FEM and Constitutive Modeling in Geomechanics
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Lecture – 22
In Situ Earth Pressures, Construction and Excavation Sequences

Let us look at more geotechnical aspects from this lecture onwards we in this lecture we are going to look at how to generate density two pressures and then simulate the construction and then excavation. This is very typical of geotechnical engineering we are not going to construct the entire structure in one go we are going to do stage by stage and then let us see how we can do them.

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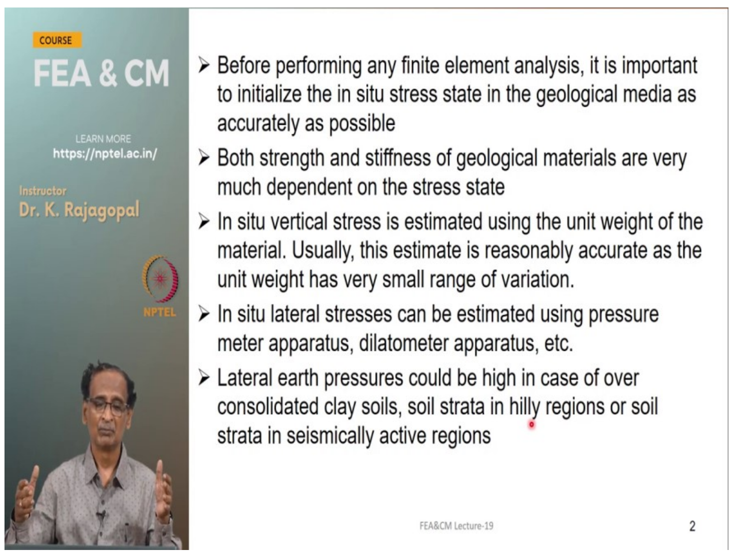

And see in geotechnical engineering before we perform any analysis is very important that we first initiate the initial stresses in the soil that is because our strength and stiffness is very much a function of the of the stress state that the soil is subjected to. And most of the geological materials like soil and rock they are dependent on the and the confining pressure for them to develop strength and also the stiffness.

And that you would have seen from our triaxial compression test. So, if we do at a low confining pressure you will get some strength and then some modulus and if you do it at a higher pressure you get a much higher strength and also much stiffer response. And so, that we are going to simulate by initial initializing the in situ stresses and how do we do that? Say the vertical stress is not a problem because we have a good idea of the unit weight of the soil.

And then gamma times z will be your vertical stress and that multiplied by some lateral pressure constant k will be your lateral pressure and that pressure constant k we can get from our either the pressure meter test or the flat head dilator meter test and so on. And then once we have this we can initiate initialize the in situ pressures. And these lateral pressures they could be very high in the case of over consolidated clays.

Like for example London Clays are known to have a k naught of about three and within Chennai itself there are some areas with a very high highly over consolidated clays they could have very high k or if you have a soil in the hilly stratum or in a deep valley they will be subjected to larger lateral squeezing in that case also we can have very high lateral pressures.

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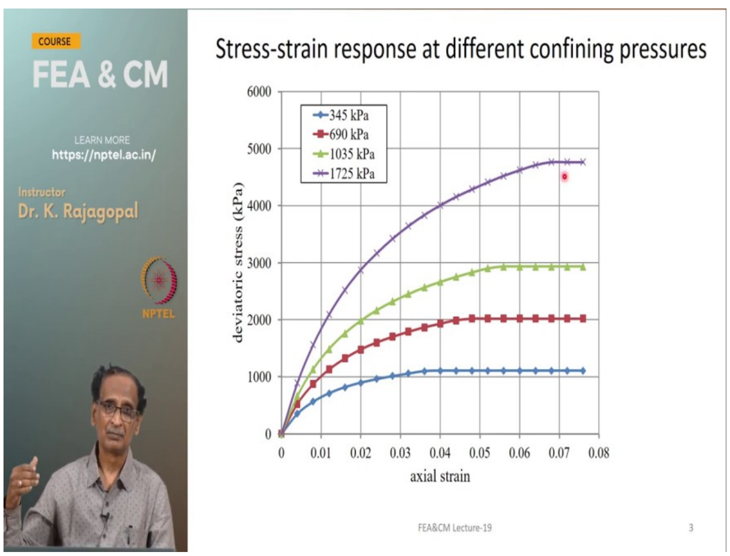
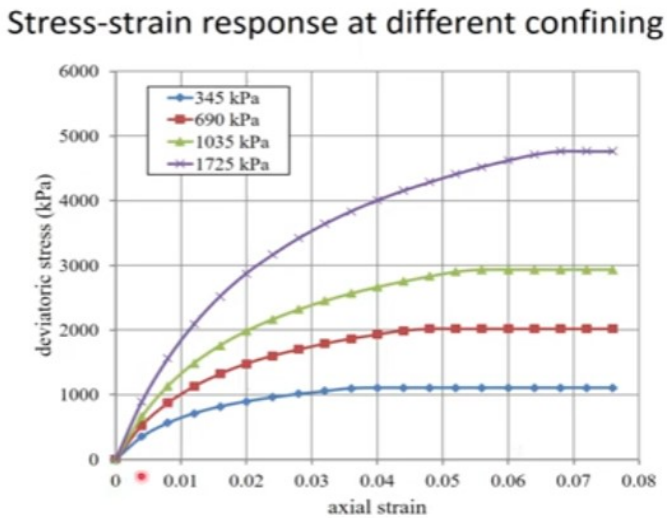


And whatever construction that we are doing say whether you are especially if you are doing the excavation if you are excavating the soil in a highly over consolidated place the locked in pressures are released and the more pressure that the soil was subjected in the geological past as your evacuating the soil is going to release that much pressure. So, that will result in larger deformations and the larger forces on our support structures.



So, let us see in this figure I have the response of soil at different confining pressures on the x axis we have the axial strain y axis and the deviated stress at 345, 690s, 1035 under 1725 kPa confining pressures and you see as the confining pressure is increasing your strength of the soil is increasing and then more importantly the slope is also increasing. Say the slope increasing means the soil is becoming stiffer.

And so, the stiffness is going to control our deformations whereas the strength is going to control our bearing capacity or the failure and so on.

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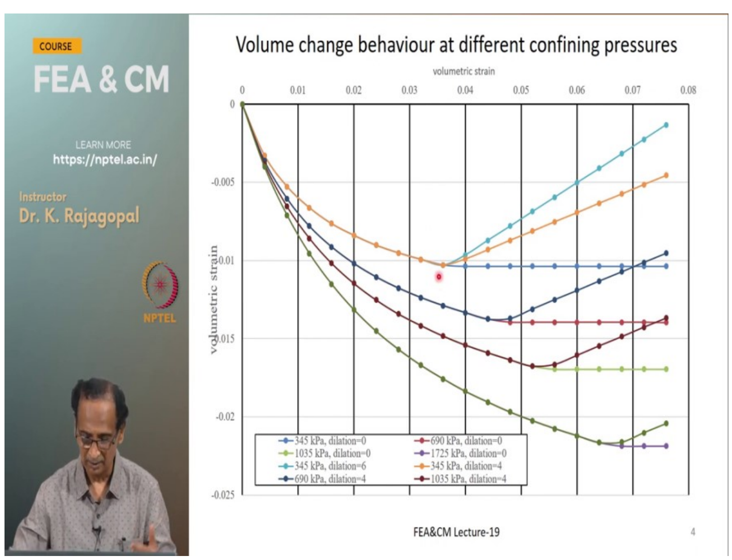
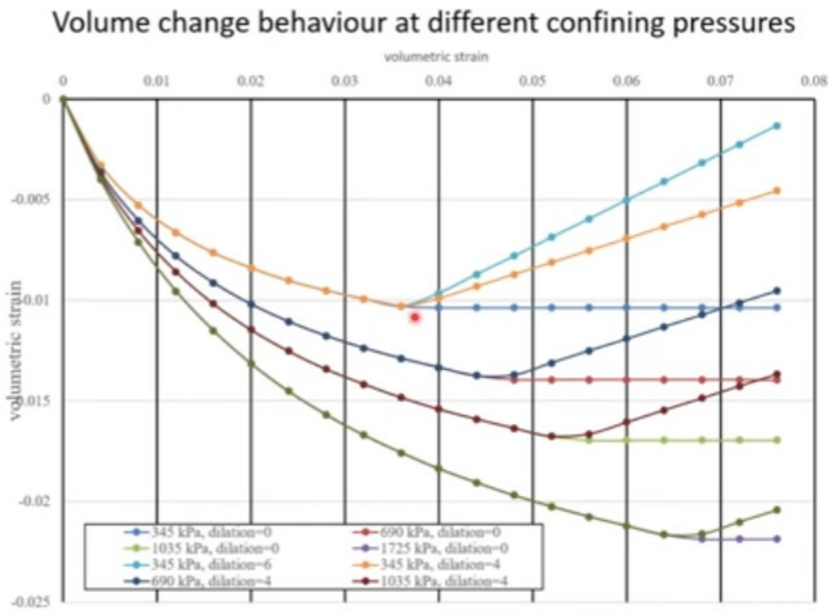


And even the volume change response is going to depend on the confining pressure. So, at low confining pressure we could have a large volume expansion that we call as dilation under very high confining pressure the soil might be subjected to more volumetric compressions these are some illustrative results.



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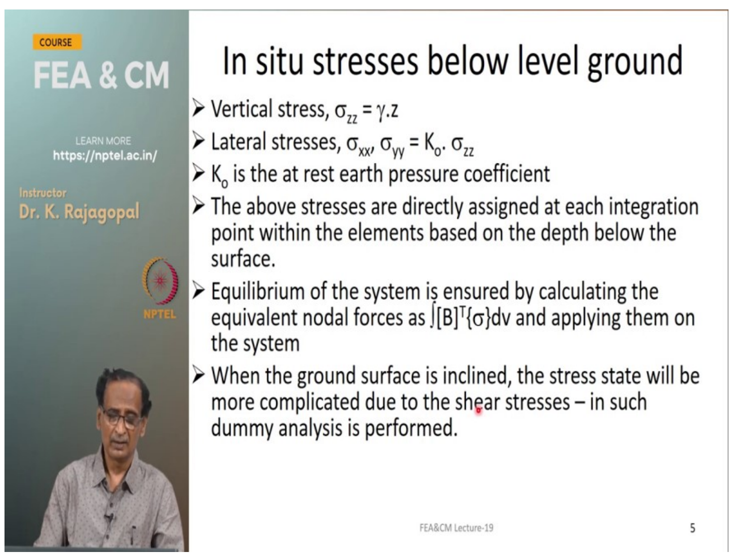

So, it is very much necessary that we initialize the in situ pressures correctly and the vertical pressure is just simply gamma z and then the lateral pressure is k naught times Sigma z. So, let us say z is the vertical axis and sigma z is gamma z and let x and y be the be the axis in the horizontal plane and we have the sigma xx and sigma yy and they are equal to k naught times Sigma z and k naught is the at the pressure coefficient at rest.

Vertical stress, $\sigma_{zz} = \gamma.z$

Lateral stresses, $\sigma_{xx}, \sigma_{yy} = K_o. \sigma_{zz}$

And we can assign all these above stress stresses at each and each integration point. So, within a mesh we know exactly where each integration point is located we can calculate x, y and z positions and then we can calculate the depth of soil above that particular integration point and then estimate Sigma x Sigma y and sigma z and then assign these normal pressures to the in situ stress states and we assume that the starting shear stresses are 0 in the soil.

And but then if you have stress without applied the force then you cannot establish the equilibrium and what we do is we need to calculate what should be the external forces that will cause this much stress. So, that we do by doing the integral B transfer Sigma dv calculation. So, whatever stress that we have applied on the system we calculate the equivalent force and apply these much force on the system so, that there is a good equilibrium between the applied force and then the reaction force.

And this method of directly specifying the the in situ pressures works only when you have a horizontal level ground because your gamma is 0 and the gamma xy is 0 or sorry the Tau xy is 0 but if you have an inclined ground there will be some shear deformations. So, in addition to your Sigma x Sigma by Sigma z there could be some Shear stresses and because of the shear stresses your normal stress magnitude also may change.

And in that case what we do is we perform an initial finite element analysis with some Poisson's ratio corresponding to our k naught.

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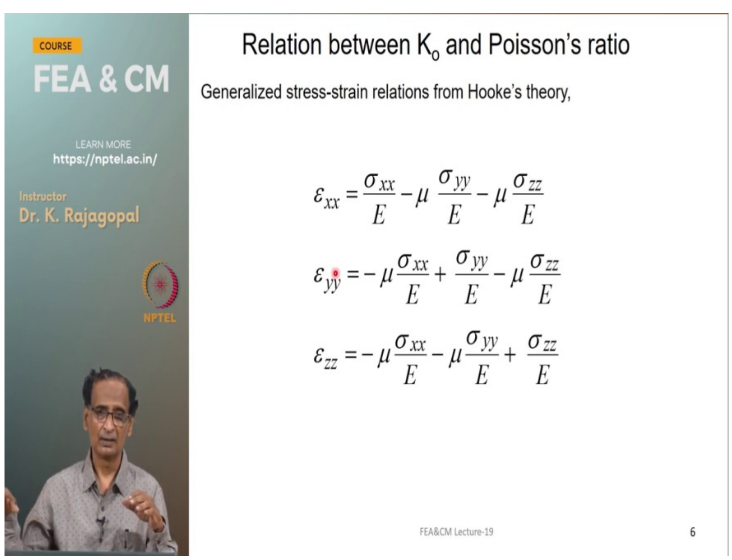

And for that we require a relation between the k naught and then the Poisson's ratio and we can go back to our original hooks relations to establish the relation between the k naught and the Poisson's ratio mu. So, if you look at the three normal strains Epsilon xx Epsilon yy Epsilon zz in terms of the normal stresses Sigma xx Sigma yy Sigma zz and then the Poisson's ratio and here by definition the rest at the pressure condition is when your lateral strains are 0 Epsilon xx and Epsilon yy are 0.

$$\varepsilon_{xx} = \frac{\sigma_{xx}}{E} - \mu\frac{\sigma_{yy}}{E} - \mu\frac{\sigma_{zz}}{E}$$

$$\varepsilon_{yy} = -\mu\frac{\sigma_{xx}}{E} + \frac{\sigma_{yy}}{E} - \mu\frac{\sigma_{zz}}{E}$$

$$\varepsilon_{zz} = -\mu\frac{\sigma_{xx}}{E} - \mu\frac{\sigma_{yy}}{E} + \frac{\sigma_{zz}}{E}$$

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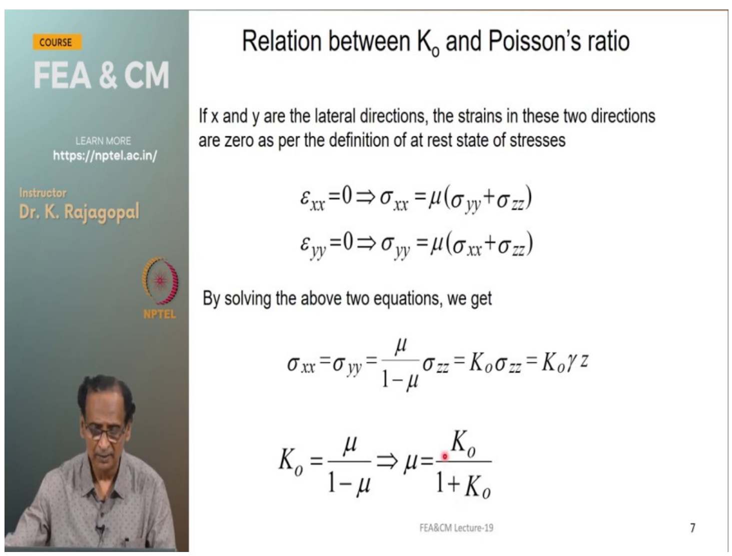

And we can set Epsilon xx to 0 that will give you Sigma xx as Mu times Sigma yy plus Sigma zz and Epsilon yy is 0 that will give you Sigma yy is a mu times Sigma xx Plus Sigma zz. And we can find a relation by for the lateral pressures in terms of the vertical stresses because between these three stress components the vertical stress is easy to compute Sigma z is just simply gamma z and by solving these two equations we can get an equation for Sigma xx and sigma yy as Mu by 1 - mu times Sigma z and it is in the form of this k naught times gamma z.

$$\varepsilon_{xx} = 0 \Rightarrow \sigma_{xx} = \mu(\sigma_{yy} + \sigma_{zz})$$

$$\varepsilon_{yy} = 0 \Rightarrow \sigma_{yy} = \mu(\sigma_{xx} + \sigma_{zz})$$

By solving the above two equations, we get

$$\sigma_{xx} = \sigma_{yy} = \frac{\mu}{1-\mu}\sigma_{zz} = K_o \sigma_{zz} = K_o \gamma z$$

$$K_o = \frac{\mu}{1-\mu} \Rightarrow \mu = \frac{K_o}{1+K_o}$$

So, this mu by 1 - mu is taken as k naught and the k naught is the lateral at the pressure coefficient at rest when your lateral strains are 0 and the sigma z is gamma z and the k naught is a mu by 1 - mu r in the reverse way we can write a mu as k naught by 1 + k naught let us say k naught is the desired value that you want to impose on the soil in turn we can calculate the Poisson's ratio for doing our dummy analysis.

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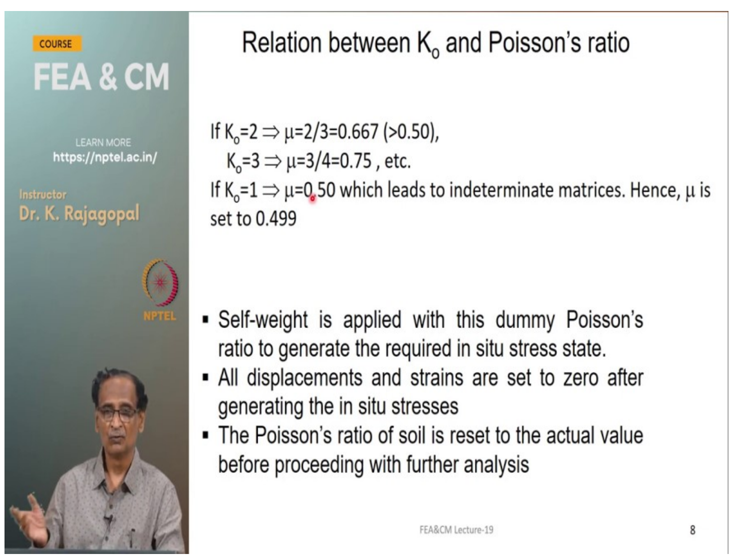

So, if k naught is 2 our Poisson's ratio is two thirds that is 0.667 and we know that the upper bound and the Poisson's ratio is 0.5 so it is greater than 0.5 which is not allowed. And the k naught of three says that your Poisson's ratio is 0.75 and once again this is more than 0.5 and our k naught of one will give us a problem because k naught of one means the Poisson's ratio is k naught by 1 + k naught that is 0.5.

If $K_o=2 \Rightarrow \mu=2/3=0.667$ (>0.50),
$K_o=3 \Rightarrow \mu=3/4=0.75$, etc.
If $K_o=1 \Rightarrow \mu=0.50$ which leads to indeterminate matrices. Hence, μ is set to 0.499

But then if you use 0.5 your constitutive Matrix will blow up because you have 1 - 2 mu the equations for plane strain axis symmetric and three dimensional. So, 1 - 2 mu becomes 0 in the denominator. So, it will just simply blow up and to prevent any numerical problems what we do is instead of using 0.5 we will round it off to some point four nine or 0.499. And so, what we do is we perform a dummy analysis with this Poisson's ratio of 0.67 or 0.75 and then initiate our in situ pressures.

And then later we set all these displacements to 0 because we are not really interested in these displacements or the soil strains this is a state where you have it before you actually construct before you construct your foundation or your embankment and we are not really interested. And then after you perform the initial in situ pressure analysis we reset the Poisson's ratio back to the original value.

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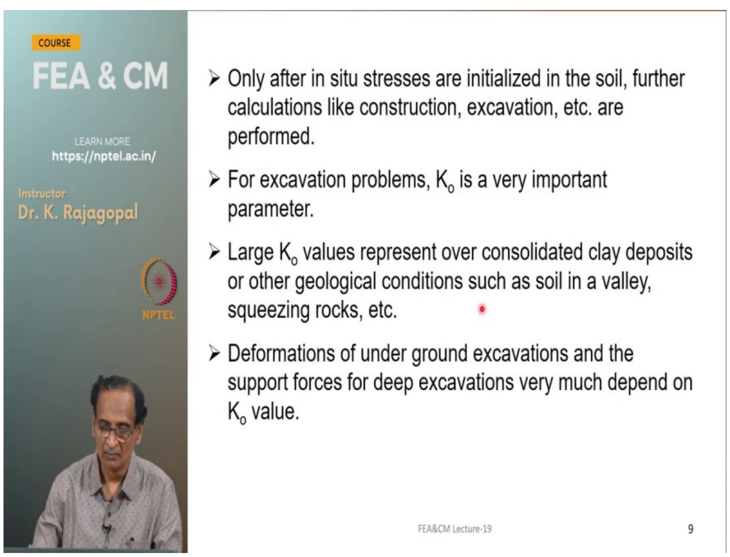

And only after the in situ pressures are initiated we can do the rest of the construction like either constructing a foundation constructing a retain wall or embankment or anything. And even for the excavation problems k naught is very very important. Because if you have very high k naught as you are releasing the soil this on the soil by excavation that much loads are being released. And this k naught of very high value happens in the case of over consolidated clays or because of some geological conditions like a deep valley or seismic conditions and so on. And the deformations when we are making deep excavations are very much dependent on the k naught.

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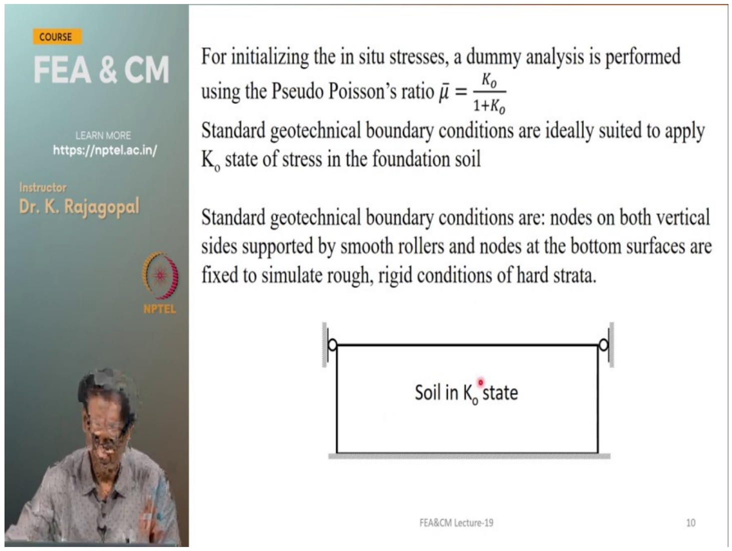
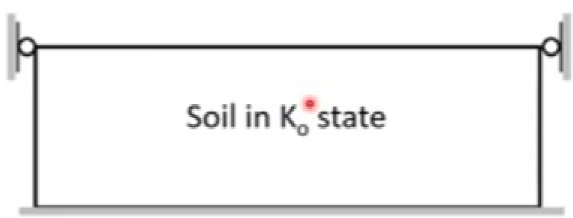


So, for doing this initial stress analysis we perform the analysis with some pseudo Poisson's ratio mu bar as k naught by 1 + k naught and then we apply standard geotechnical boundary condition that is the two vertical sides are support done and rollers smooth rollers and then the horizontal boundary at the bottom it is constrained from moving in both x and y directions to represent the rough rigid the boundary at the bottom. These boundary conditions are called as a standard geotechnical boundary conditions.



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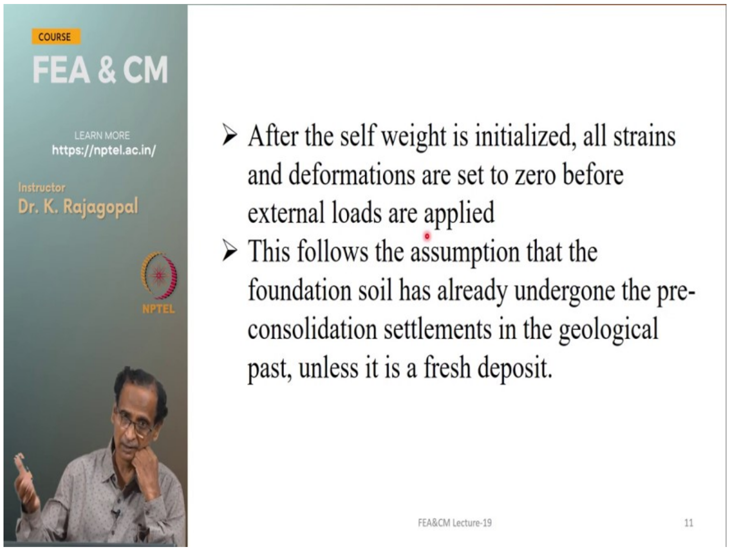

And after we initiate the self weight with the desired k naught we set all the strains and deformations to 0 and because we are not really interested in knowing what happened before we start our construction. We are more interested in knowing what happens because of our construction. Like let us say you have construct a foundation whether it will settle by 10 millimeters or 50 millimeters is our concern whether the soil is undergone some one meter settlement in the geological past that is not our concern.

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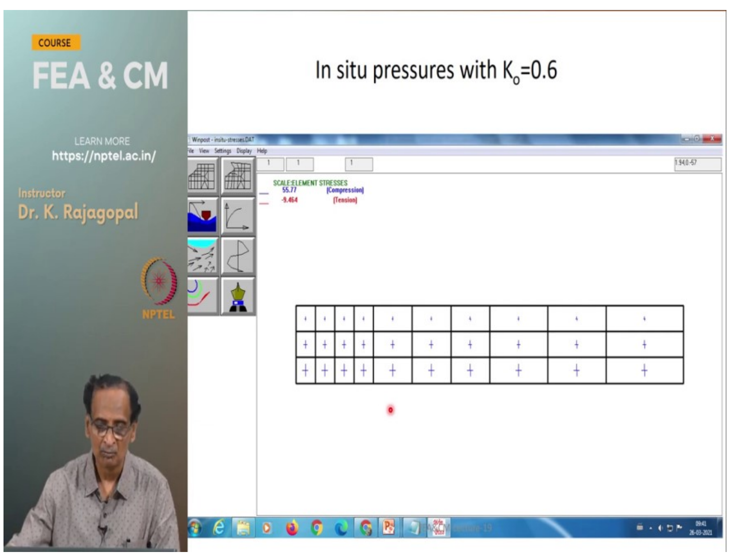

So, here you have a typical result with the k naught of 0.6 and the yellow is the original mesh and then the black line is the deformed mesh and the soil is deforming down with the k naught of 0.6 and then these are the stress vectors you see k naught is 0.6 means the sigma z should be more than the sigma x and sigma z is you see a longer Vector because these are all scaled vectors compared to the horizontal Vector with which is shorter. Because it is only Sigma x is only 0.6 times Sigma z.

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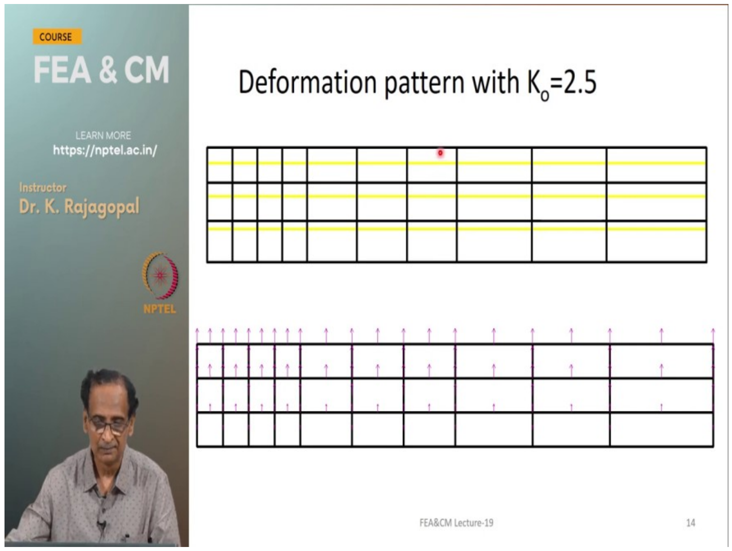

And let us; if you do with the k naught of 2.5 very very high initial pressure state the entire soil is heated up the yellow line is the original mesh and the black line is the deformed mesh and you see here the vectors they are all showing the soil is heaving up.

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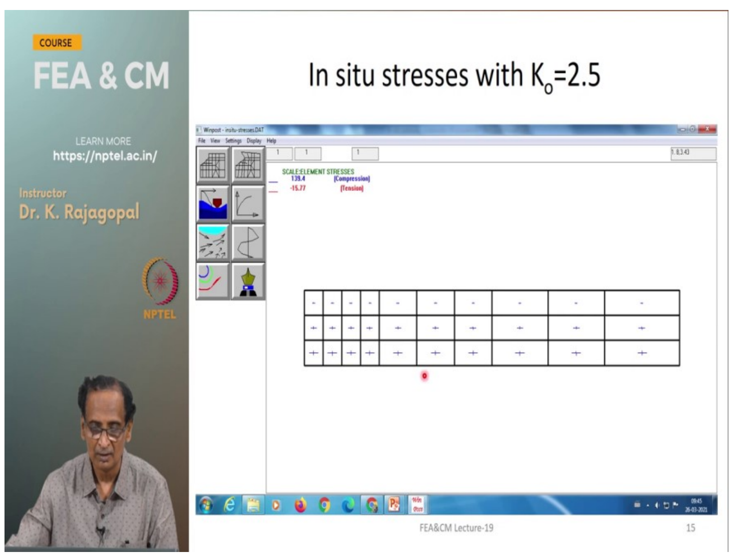

And you see the stress vectors the horizontal line is very long compared to the vertical line. So, that means that the lateral pressure is high.

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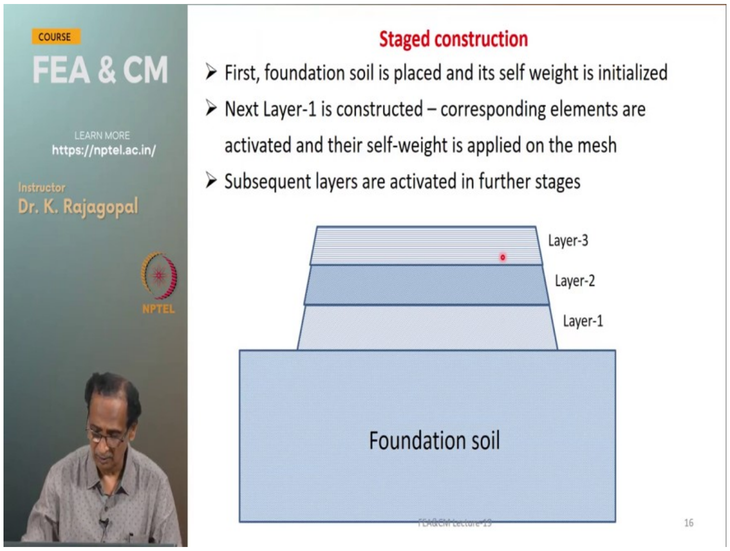
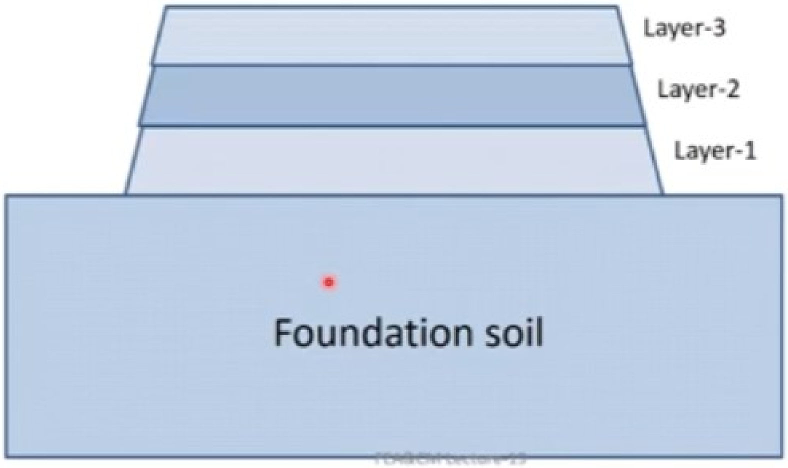


And so, the stage at construction is very important in our geotechnical engineering. Let us say you are dealing with a soft clay and we may not be able to construct our entire height of the embankment in one go because we have to go layer by layer and in a small and small heights and with some Gap. So, that the foundation soil can gain some strength now because as the soil is consolidating the soil will dissipate some pore pressure and it will undergo some compression.



So, its strength will increase and so, that is unique to a geotechnical engineering. And here just shown an illustration see here we have a foundation soil and we are going to construct the embankment in stages say let us say layer one layer two layer 3 and so on. Like we just go on placing the soil until our full height is achieved. And now we can actually use our finite element programs to not only see what happens when you have an incremental construction like this.

But we can also examine how much time we should wait for the soil to consolidate under layer one so, that before you place the soil in the layer two because if the soil is consolidated then that means that the soil has gained some strength. And so, when you place the next layer of soil the it will give a slightly better response a stiffer response and so on.

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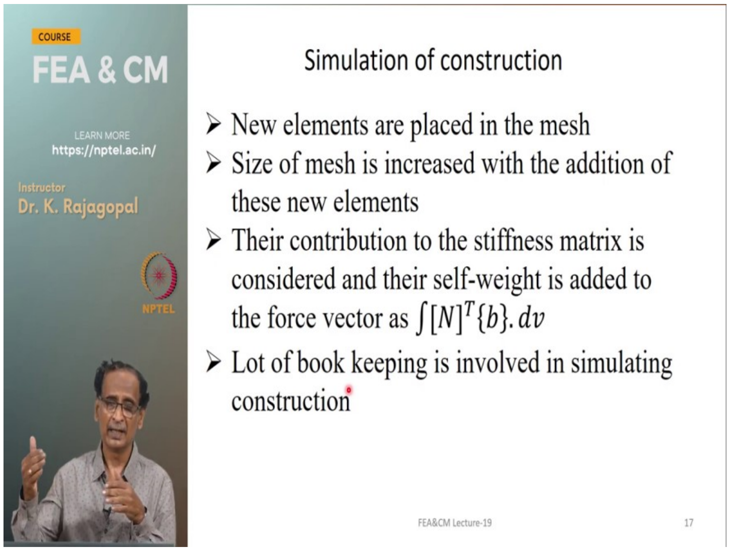

So, what exactly we mean by construction is we place new elements in the mesh and because of that the size of the mesh is going to increase because of the addition of new elements and the self weight of these elements is added to the load vector as n transpose B integrated or the volume of the elements. So, for all the newly placed elements we calculate this additional load that is coming from them and then add them to the right hand side the force vector.

$$\int [N]^T \{b\}.dv$$

And we also calculate its stiffness and assemble it for the into the global stiffness matrix. And so, actually it is actually a lot of bookkeeping see the program should keep track of what are all the new elements that are placed in the mesh and it should calculate its stiffness Matrix and then the load vector and then add them to the to the already existing stiffness Matrix and then the load vector to simulate the construction.

And so, the simulation of construction is basically N transpose b this is what we have seen in the one of the classes earlier.

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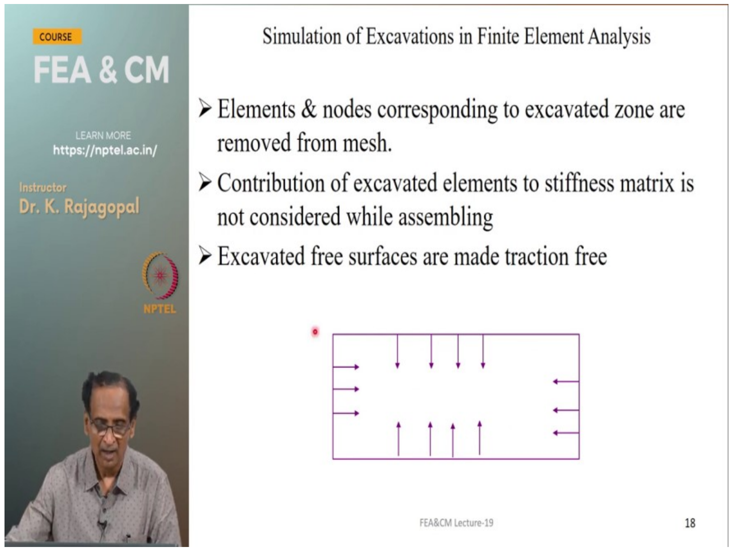
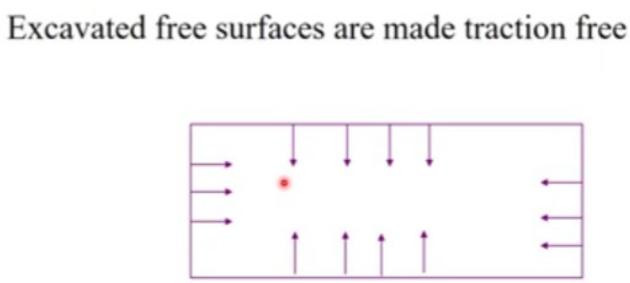


And the simulation of excavation is the opposite. See let us say we are evacuating the soil within this within this region we are going to remove these elements and then these nodes corresponding to these elements that are removed and how we simulate excavation is very simple. So, after you remove these the soil elements when the next time when you form the stiffness Matrix you are not going to consider these elements that are removed.



And then these surfaces they have become stress free surface or traction free surfaces because they are they are free surfaces.

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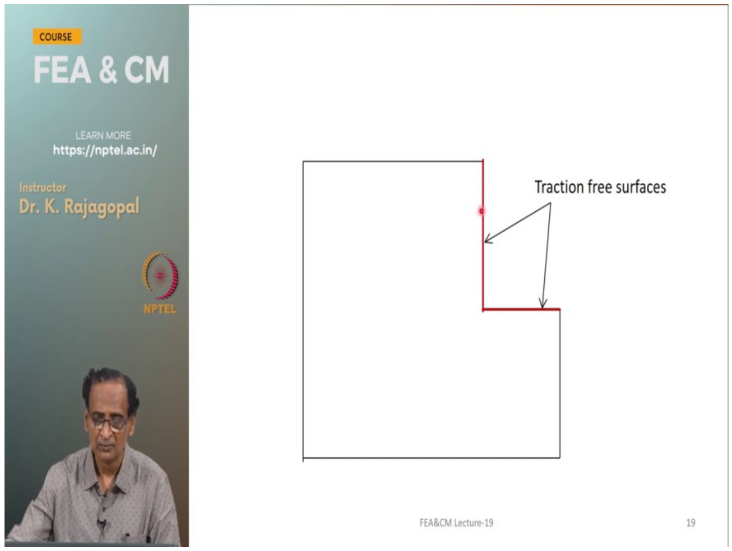

So, how do we make them traction free and for that we need a procedure and the same thing with the deep excavation. Let us say we are excavating some soil and this vertical and laterals are the horizontal surface they become stress-free attraction free after the excavation.

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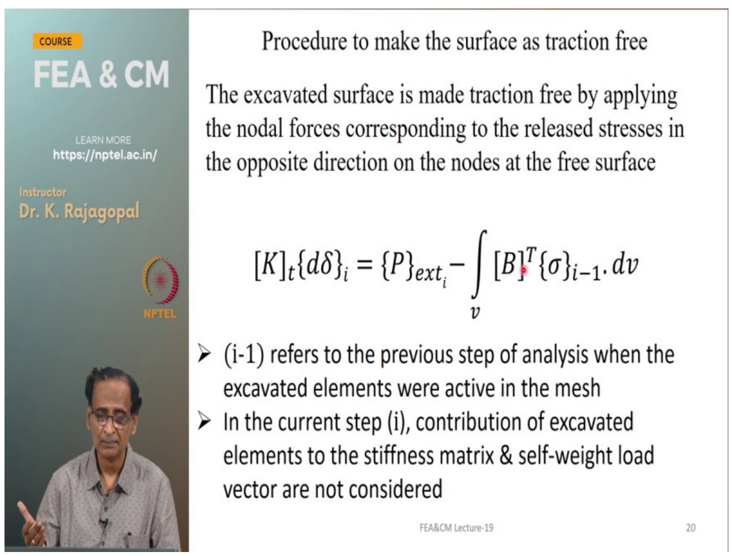

And what we do is we reformulate our equilibrium equations like this k times incremental displacement is equal to external load P - integral B transpose Sigma dv it is actually our reaction force is calculated as integral B transpose Sigma and by putting that into the right hand side load vector we can automatically take care of the load that needs to be removed because of excavation.

$$[K]_t\{d\delta\}_i = \{P\}_{ext_i} - \int_v [B]^T\{\sigma\}_{i-1}.dv$$

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And if you see this subscript I - 1 this refers to the to the to the previous step when these elements were active. And so, this the P external is the applied load that will include even the contribution from the elements that were not excavated in the previous step and now we are removing them because we are putting minus of B transfer Sigma i – 1, i - 1 refers to the previous step.

And so, when we go to the formulation of the stiffness matrix and the ith step we are not going to consider the elements that are removed but then we consider their contribution the previous step on the right hand side. So, in this way we can we can simulate the excavation and also the release of stresses.

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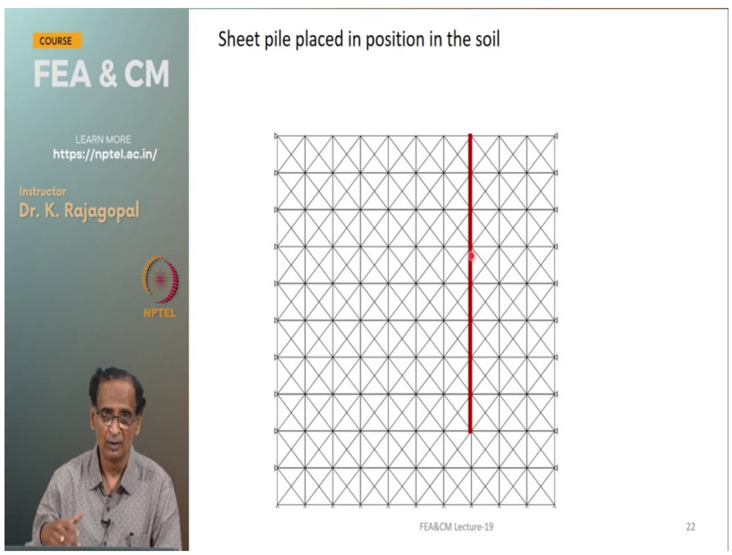

So, here let us say this is your foundation and we are going to place a sheet pile wall before we construct. And actually currently it is what we do is we call it as a vision place we just simply place the sheet pile wall to the desired depth we are not going to drive it into the ground into the soil because we are not really interested in knowing what happens during the driving.

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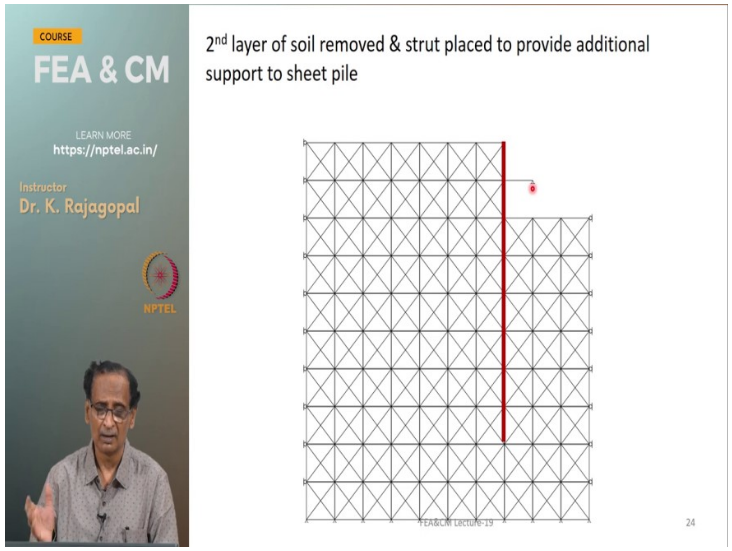

And then after you place the sheet pile wall to the desired depth we start removing the soil. And these soil these soil elements are removed. So, these surface this horizontal surface and this vertical surface they become traction free. And if you want you can provide some struts. So, that the lateral deformations are within some limits.

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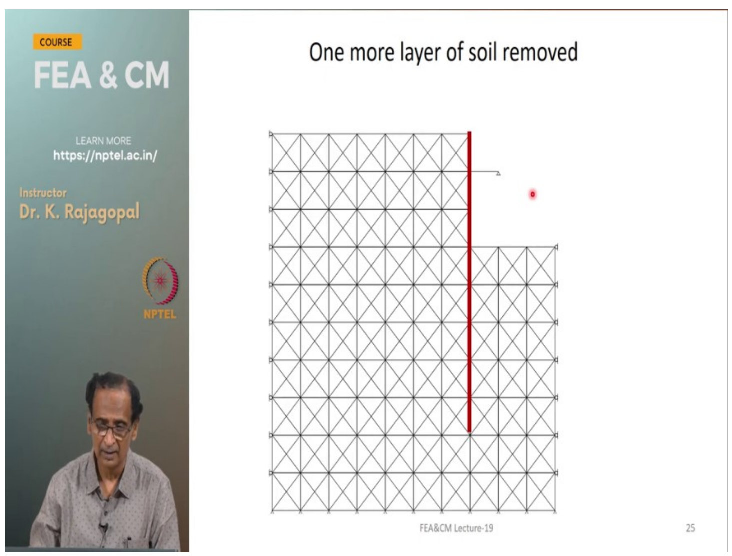

And then we can go on excavating up to the desired depth.

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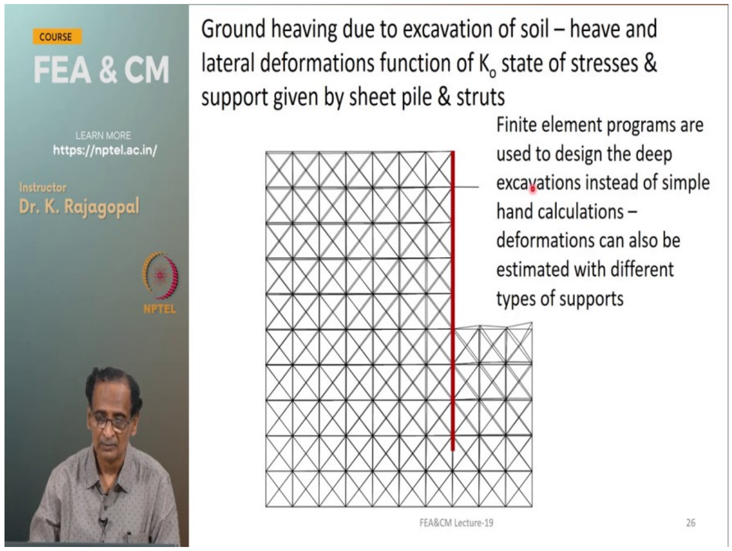

And you see here as you are excavating the soil is going to heave up.

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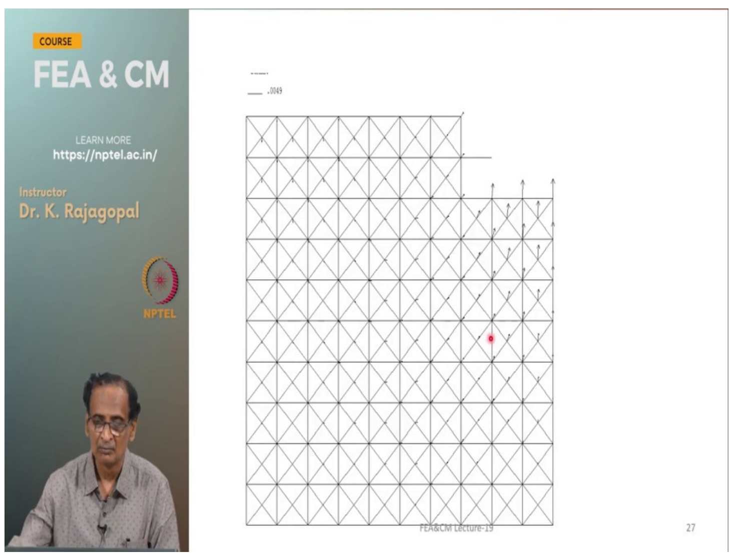

So, here these upward arrows they are showing that the soil is heaving up. And so, this excavation is a very very common thing with all the Metro constructions they are having either deep excavation or tunnels especially if you are inside a city these excavations are very very dangerous because most of the areas are built up with pre-existing buildings and your construction activities should not impair their stability.

So, it is very important that we perform detailed finite element analysis and see what is the effect of our excavation on the foundation of a neighbouring building or neighbouring structures? And then take some precautions.

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So, here I will show you one more example of excavation simulation and this particular one refers to subsidence at Singareni coal mines because of mining activities. And this mining activity is called as a like it is a pillar type sorry it is a it is a long wall mining there is a machine that cuts the cold to a very long length and this the cold seam is of approximately three meters height and the once the coal is removed from one panel we give some gap and typically this Gap is about 30 meters.

And then we do one more round of mining at the other end and this area of the soil or coal that is left behind it is called as a pillar because it is like a pillar its supporting the the superstructure or the soil above. And so, this typical overburdened depth is 100 meters thickness of the coal seam is 3 meters and the spacing between these long wall coal mines is 30 meters.

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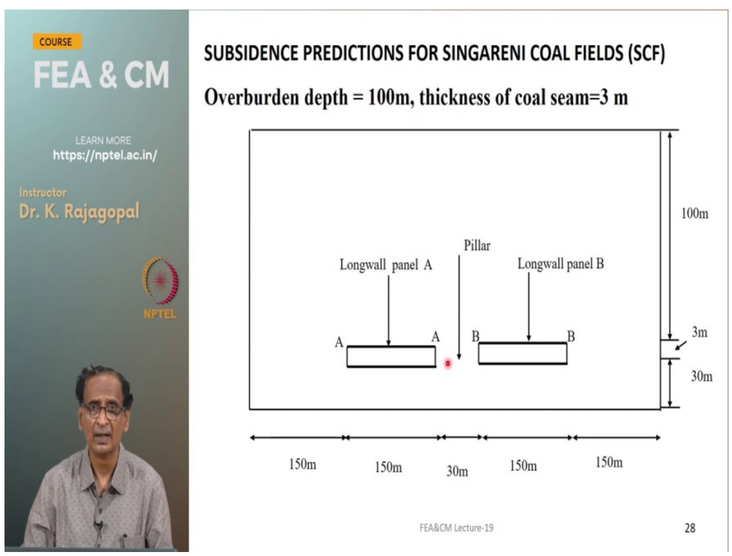
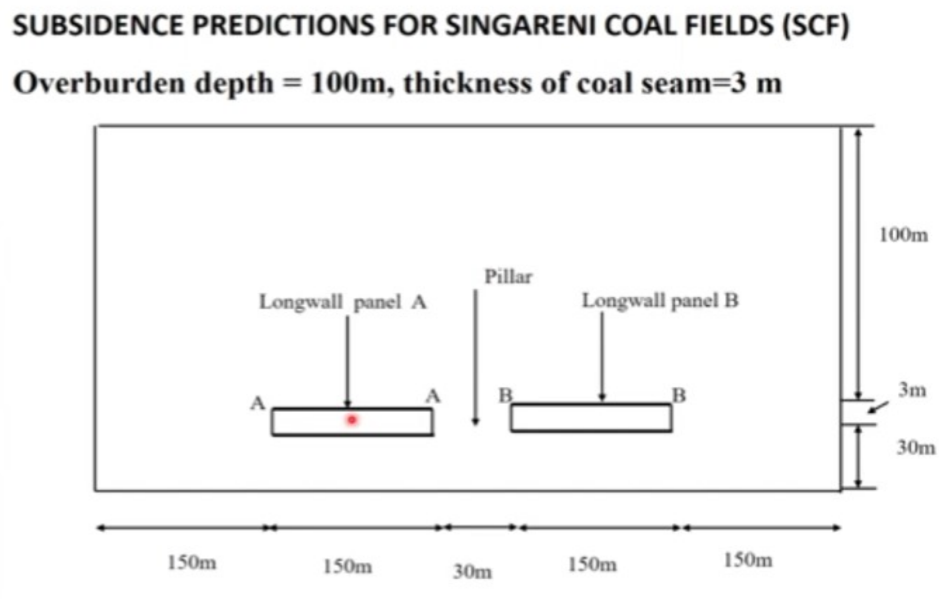


And because of this coal mining at 100 meters depth the ground at the top is undergoing some subsidence that is mainly because this entire ground is consisting of number of joint planes both horizontal and vertical joint planes. So, whatever you are doing at a great depth of 100 meters it is reflected back onto the surface.



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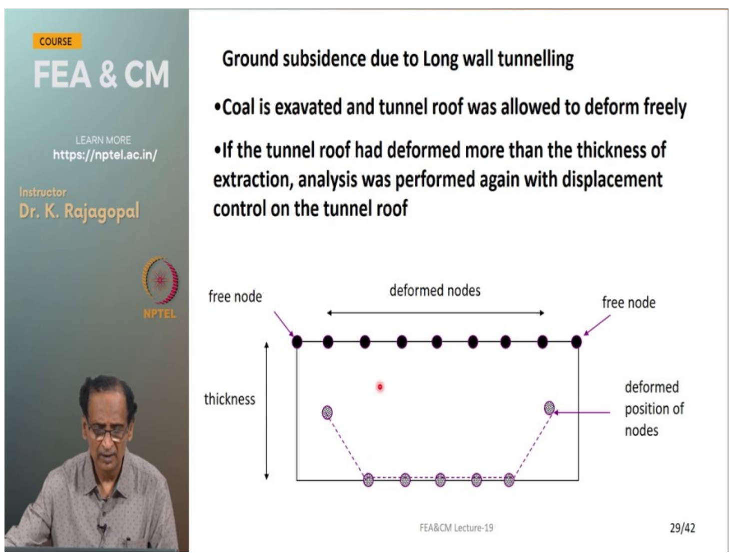
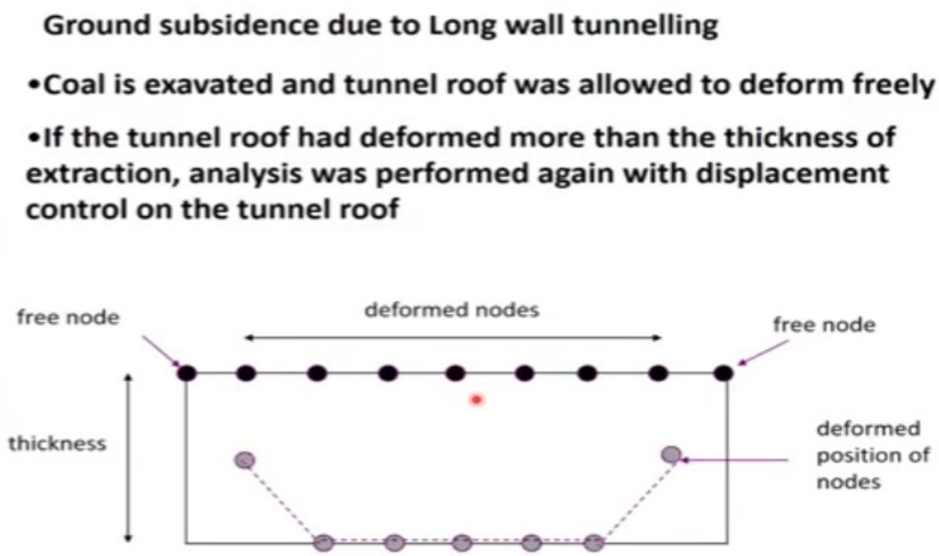


And say when you remove the soil what happens to the tunnel roof it will collapse and sometimes its deformation may be more than the thickness of the cold seam. And to prevent these nodes from deforming more than the thickness of the coal seam what is done is deformation equal to the thickness of the coal seam was given to all these nodes like this. This is the node at the other end connected to the rest of the soil mass and we give a deformation equal to the thickness of the coal seam and then see what happens in the rest of the soil.



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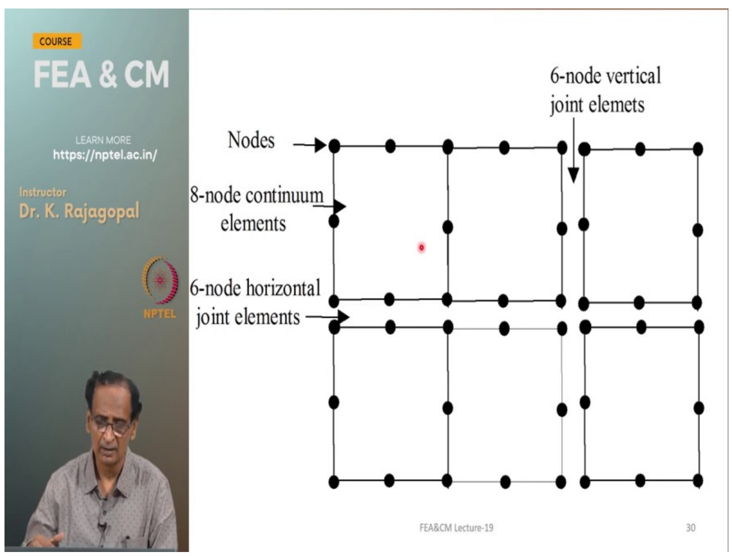
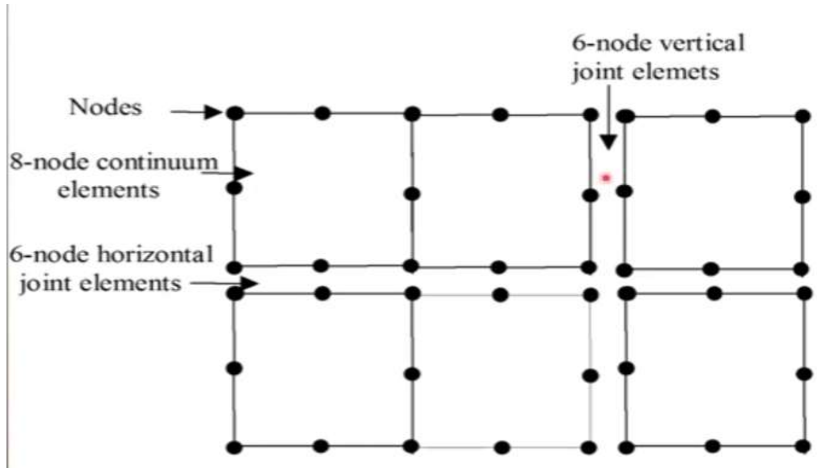


And the mesh itself is a very complicated mesh because there are number of horizontal joints and then the vertical joints. All these joints are simulated by using a six node joint elements and then the continuum was simulated by using eight node quadrilaterals square, rectangles and so on. Like here is only a schematic sketch.



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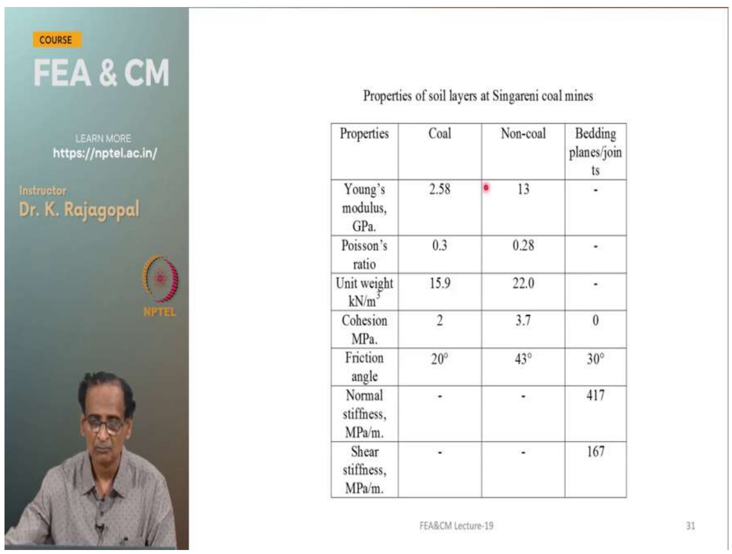
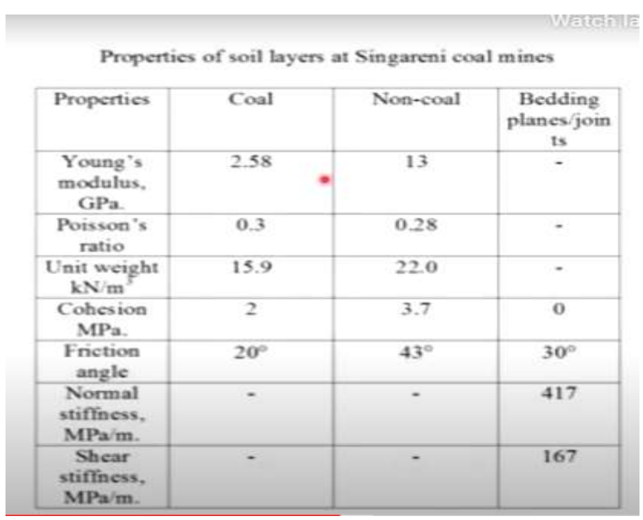


And these are all the properties of the soil and then the coal mine and the in the seam, the Young's modulus, Poisson's ratio, unit weight cohesion friction angle and so on.



Properties of soil layers at Singareni coal mines

Properties	Coal	Non-coal	Bedding planes/joints
Young's modulus, GPa.	2.58	13	-
Poisson's ratio	0.3	0.28	-
Unit weight kN/m^3	15.9	22.0	-
Cohesion MPa.	2	3.7	0
Friction angle	20°	43°	30°
Normal stiffness, MPa/m.	-	-	417
Shear stiffness, MPa/m.	-	-	167

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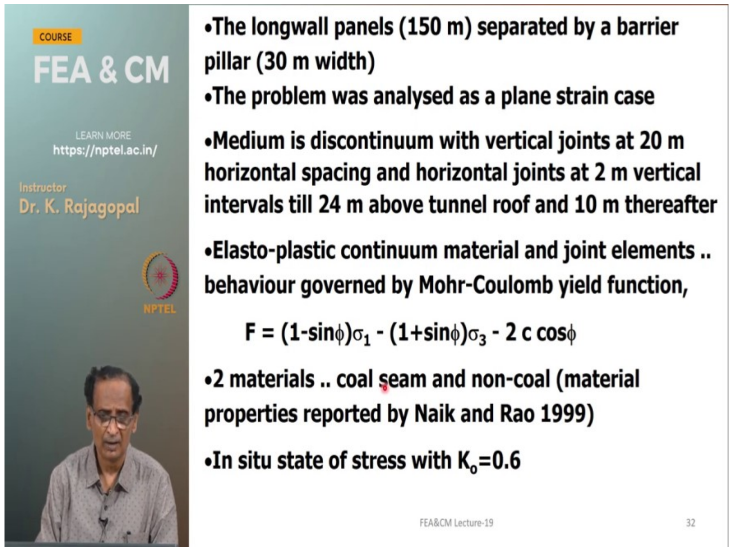

And see these long wall panels of one meter 150 meters wide they are separated by a pillar of 30 meters width and the entire problem was analyzed by using the plane strain analysis. And this medium is discontinuous with vertical joints at every 20 meter horizontal spacing and then this vertical spacing along the vertical direction also there are number of joints the entire analysis was done by using the plane strain model.

And Mohr-Coulomb relation was used for simulating the strength of this soil and the in situ a stress state was 0.6 as reported by Naik and Rao corresponding to the site.

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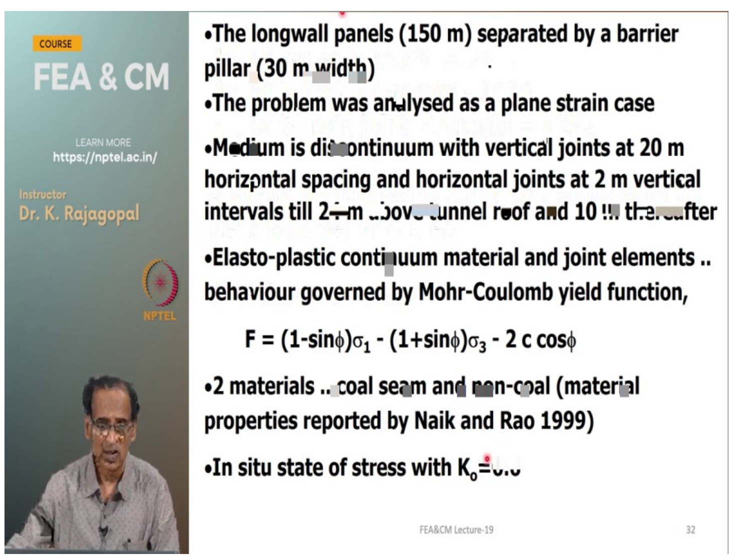

And the number of node points in the mesh is 7400 and the number of eight node elements is 1484 and the number of joint elements is 1252 and the analysis was performed in 2000 load steps with 75 iterations per load step. Actually it is action it is not an easy thing to perform this analysis because we like to have a good balance between the applied load and then the reaction load but in this type of non-linear problems it is difficult to enforce that equilibrium and this happens with very very large number of iterations.

So, each step each load step corresponds to some displacement about 75 times this solution was repeated until the equilibrium is maintained and the approximate CPU time is about 24 hours it took a long time. Some analysis they took more than one week because of the complicated geometry.

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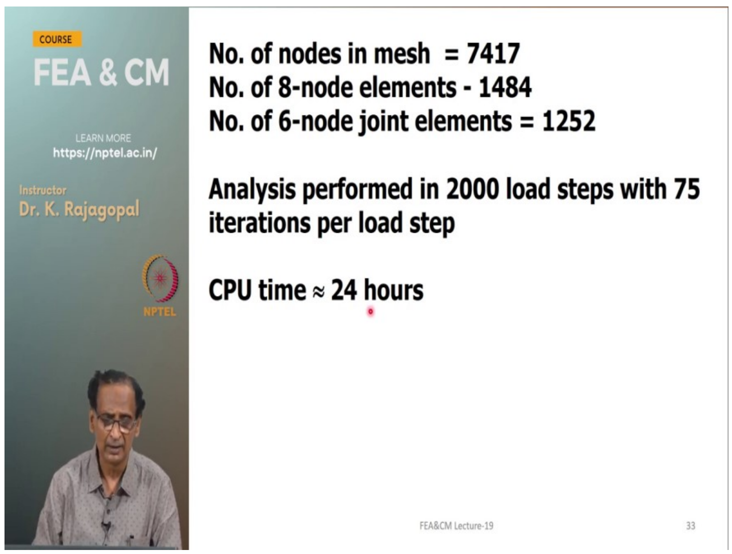

And this is a typical mesh and this plus sign is the observed ground subsidence or this site in Siliguri coal mines and then this the line with their with the round is the predicted one is actually this is the second excavation this is the first excavation because it is actually done in series with some time gap. And only the peak settlement is not predicted properly but otherwise the prediction between the finite element analysis and then the measured displacements is not bad.

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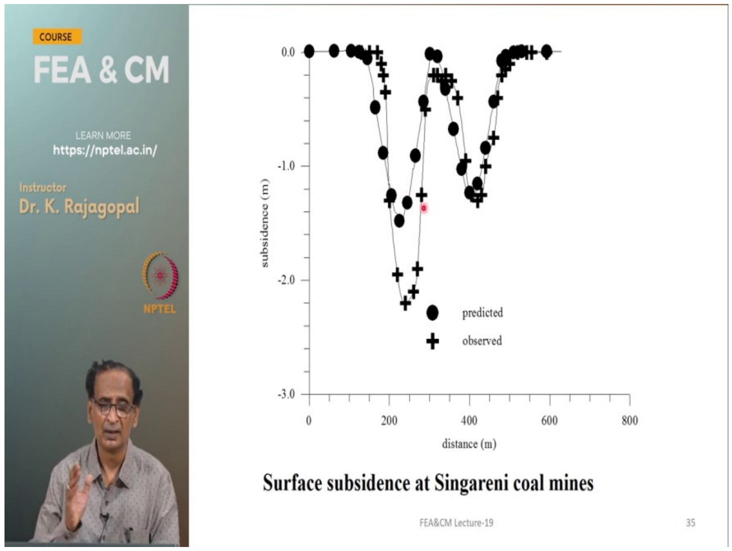

And one more prediction is done for coal mine in the USA this is in North Applachian coal mines in Virginia. And once again it is a plane strain problem long wall mining and constitute to model was Druker-Prager and this analysis was done by somebody else by Su in 1991.

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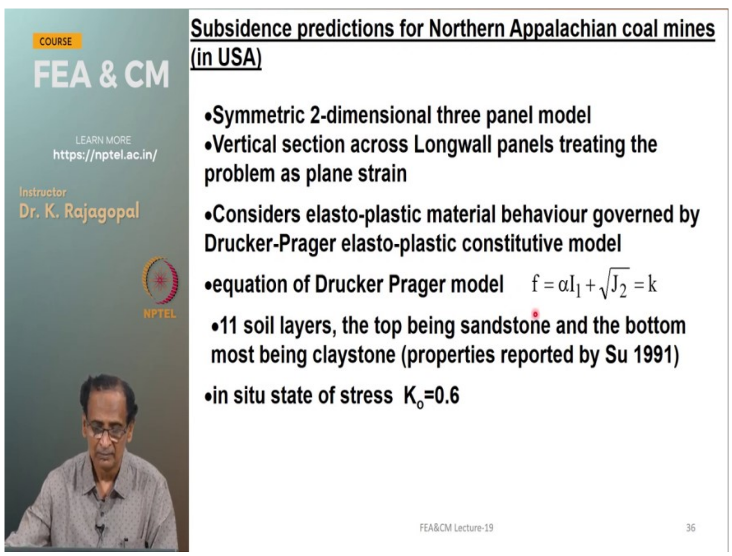

And this is the mesh and this mesh was even bigger because the area of the soil is much larger and the CPE time is about 26 hours.

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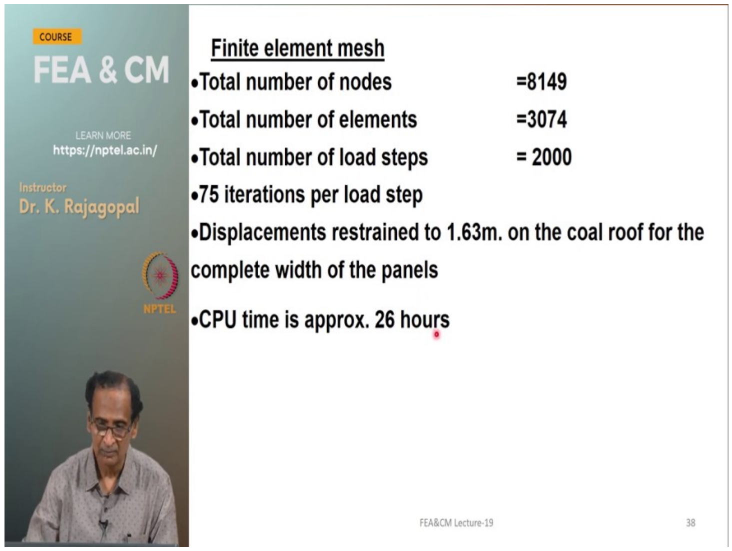

And these are the properties.

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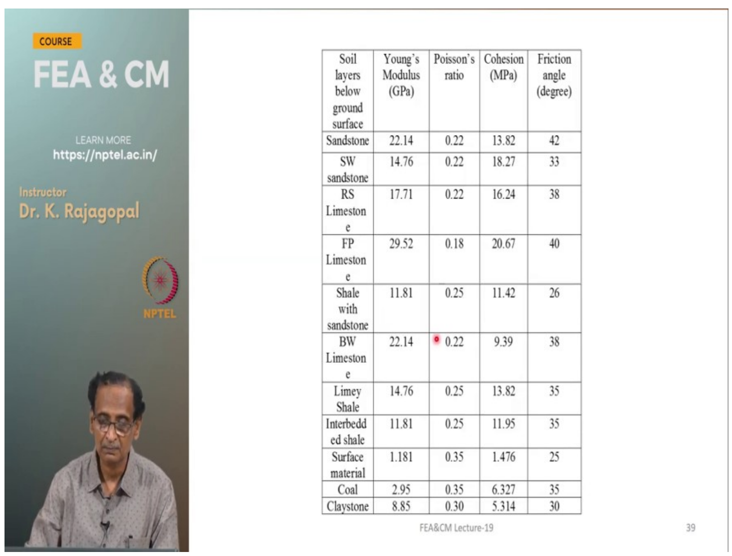

Soil layers below ground surface	Young's Modulus (GPa)	Poisson's ratio	Cohesion (MPa)	Friction angle (degree)
Sandstone	22.14	0.22	13.82	42
SW sandstone	14.76	0.22	18.27	33
RS Limestone	17.71	0.22	16.24	38
FP Limestone	29.52	0.18	20.67	40
Shale with sandstone	11.81	0.25	11.42	26
BW Limestone	22.14	0.22	9.39	38
Limey Shale	14.76	0.25	13.82	35
Interbedded shale	11.81	0.25	11.95	35
Surface material	1.181	0.35	1.476	25
Coal	2.95	0.35	6.327	35
Claystone	8.85	0.30	5.314	30

And there is a good match between the predicted displacements and then the observed displacements.

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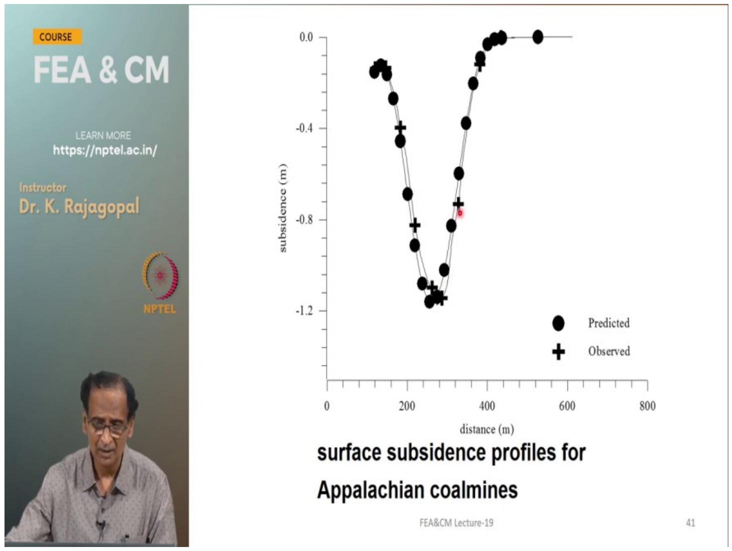

Because these soil properties are directly uptrend from their Laboratory and the site here it is a bit more complicated because there are number of soil layers as given here at different depths.

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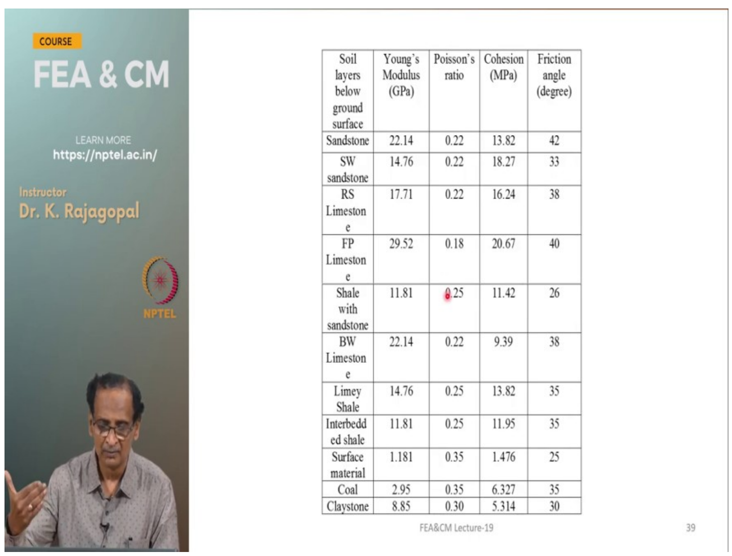

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And for the Singhuneri Coal Mine some parametric analysis were done for studying the effect of k naught on the subsidence see when the k naught was 0.43 the substance was very high and when the k naught is one the subsidence is lower. And then when the k naught is very high and the subsidence is even lower. Because actually here in this particular case because the cold seam is removed.

And then the rest of the soil Mass is at very high pressure. So, if it is at a very high pressure it may not move because it is already under a very tight squeezing position.

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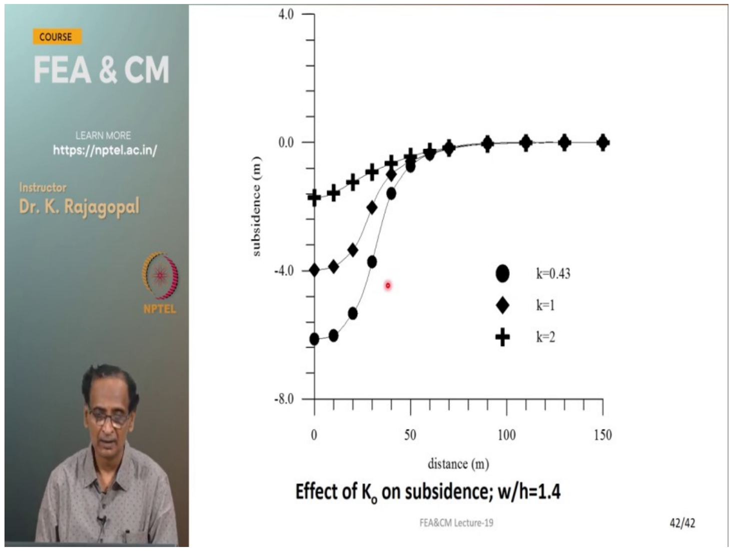

And this particular one is done with k naught of 0.6 and with the different the panel widths. And the panel width as it is increasing it becomes more and more critical with the more settlements that is what we see here with the panel width of 0.8 the settlement is less than half a meter and with a panel width of 1.2 it is more. But then when the panel width is about 1.4 the settlement is very high.

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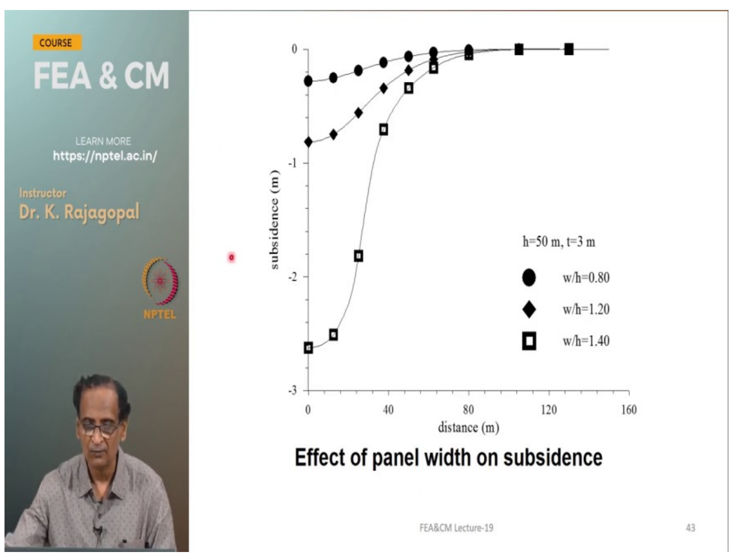

And we can actually process distress contours inside the soil and then see what happens. See these stress contours with a negative sign -100 means it is a compressive stress and if it is positive that means that the soil is failing because it is a tensile stress and when the w by h is 0.8 that is the non-critical case or a sub critical case of excavation width. The most of the tensile stresses happened only around the around the tunnel.

And away from the tunnel there are more mostly negative stresses compressive stresses and because of that your settlement is smaller.

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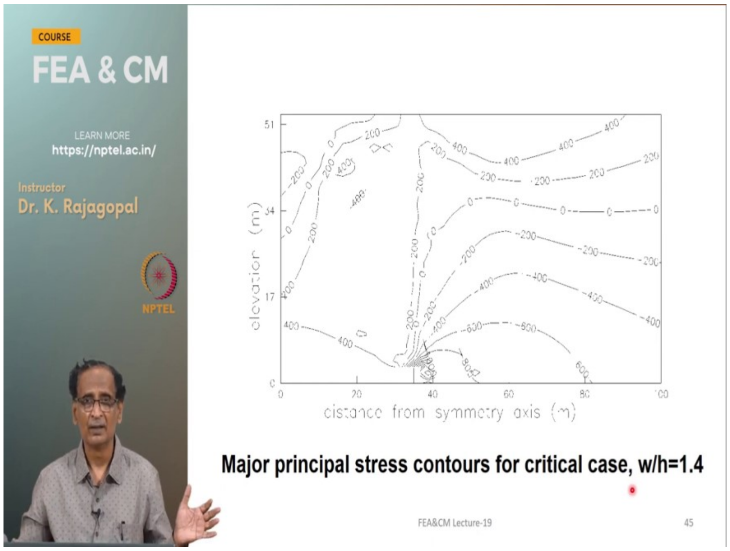

But then when you have a w by h of 1.4 that is a critical width when your failure reaches all the way to the top and that is what we see here. The tensile stresses are propagated almost up to the ground surface this is 200 tensile stress. And so, here to closer to the tunnel it is about 400 and then at some other place there is a compression. And these contours were plotted the separately what we do is after we run the Finite Element Program program there is an option to save the shear strains or shear stresses in the xyz format.

XY are the coordinates and the z is the Contour value and this is given as an input to another program and that can draw the contours and then that program has produced these contours. But fuse the commercial finite element programs like Abacus or Plaxis they have good options for drawing the contours you do not need to do this type of importing to other programs.

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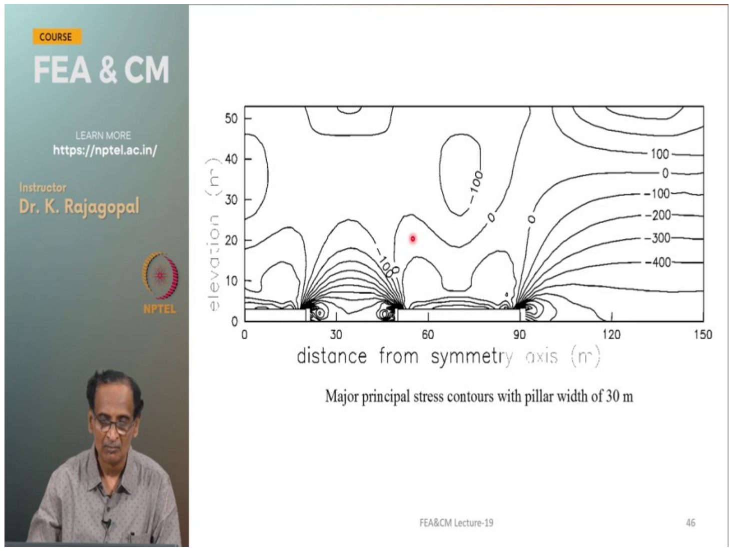

And this is the one with the two parallel mining and what happens to the stress contours.

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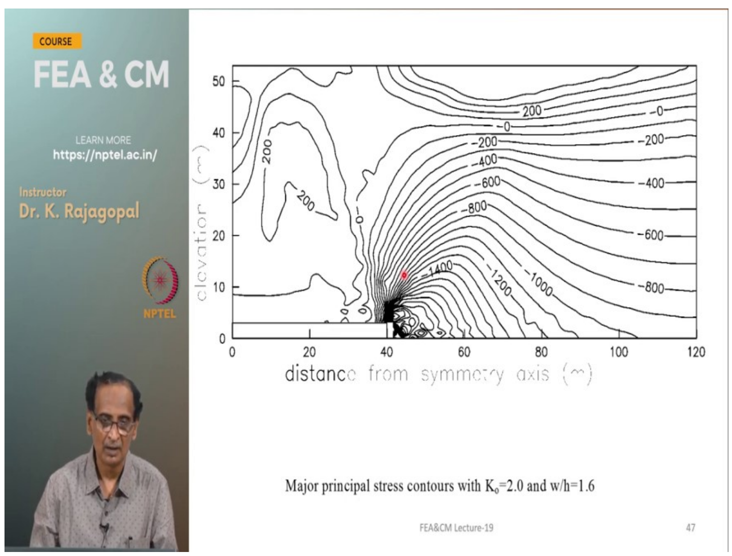

And once again with a very large width for the cold seam that they affect propagates up to the top w by h at 1.6.

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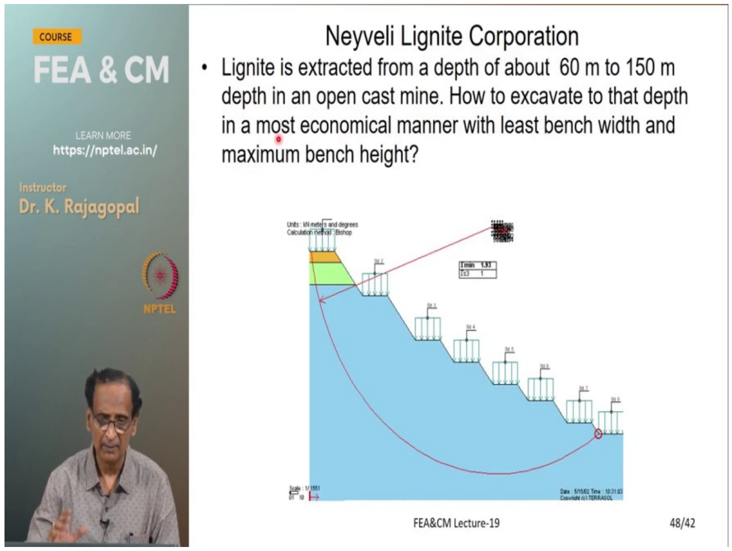

And this same thing was applied even for deciding on the on the nine slopes for another deep excavation this is for a Neyveli lignite Corporation they have open cast mines where they dig the topsoil to get access to the to the lignite. Lignite is another form of the coal and here the lignite is happening at about 60 to 150 meters depth and the entire area is an open cast mine and the mine authorities they wanted how best to excavate.

So, that they can have a very steep slope and as small a bench width as possible because if they have a larger bench width that much area they are losing for the for the lignite mining. And they asked us to recommend some slope angles and then the bench width and other things and the entire analysis was done by finite element programs and before that the analysis was done through this the slip Circle analysis also for just to get some idea.

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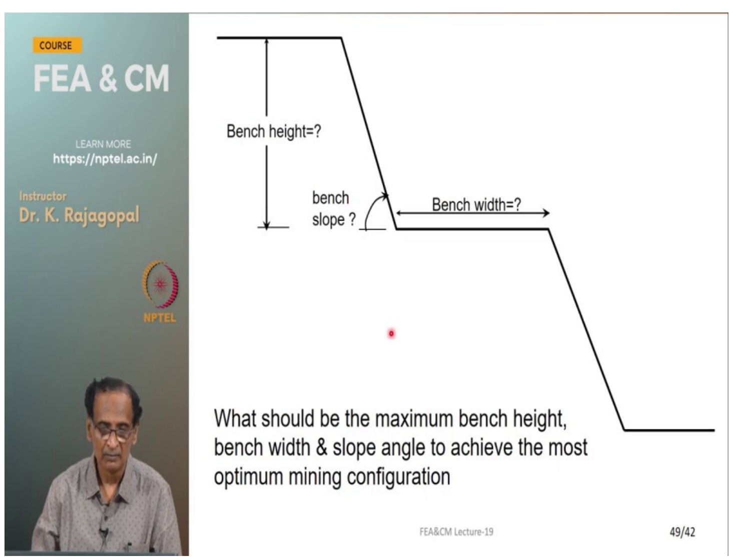

And the question is what should be the bench height and what should be the bench slope and then the bends width. So, actually they want maximum height and a very steep angle and as low bench width as possible within some constraints that it cannot be too small because their construction vehicles have to move. So, for that they need at least 10 meters and then how many benches can we have the lesser number is better because that much area they can excavate for extracting the lignite.

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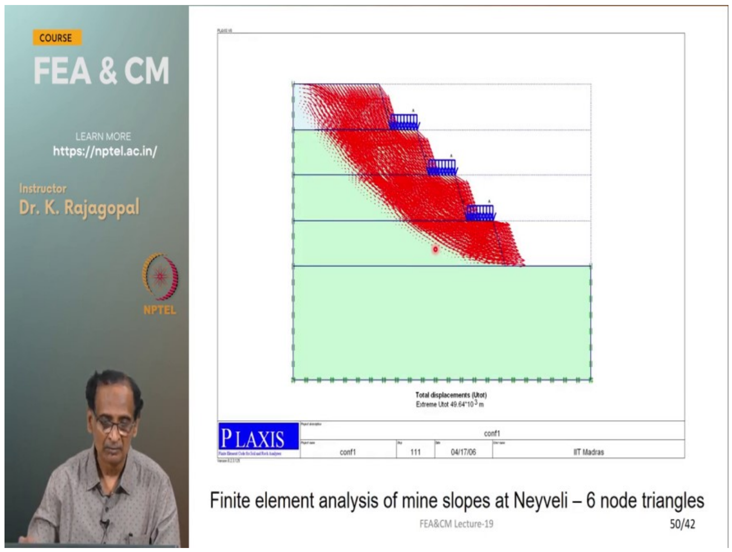

And this particular result is from plaxis program.

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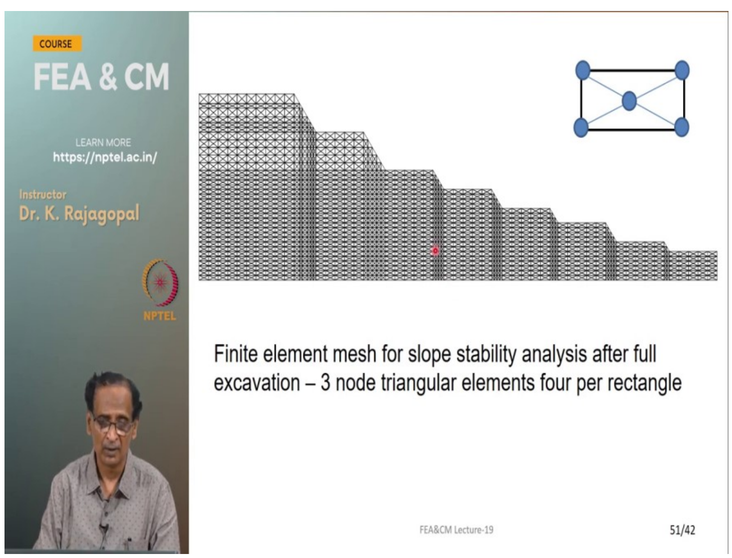

And the same analysis were also done by using the GEOFEM program by using three node triangles arranged with four per rectangle like this and very similar results were were obtained between the GEOFEM and the plaxis program.

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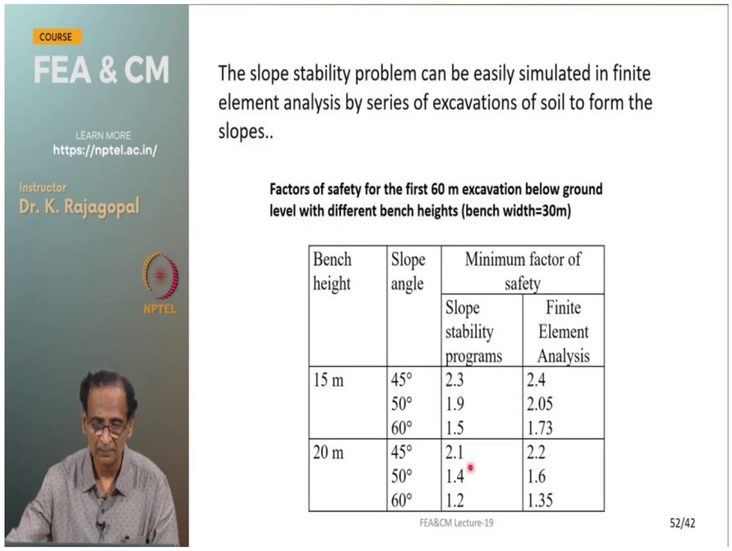

And these results are from the GEOFEM program for different bench heights and the slope angle and what are the factors of safety obtained and they wanted a minimum factor of safety of 1.5 and so, we can have 1.5 we can get with a bench height of 15 meters and the bench slope of 60 degrees now it was able to give the slope stability program was giving a slightly smaller value compared to finite element analysis.

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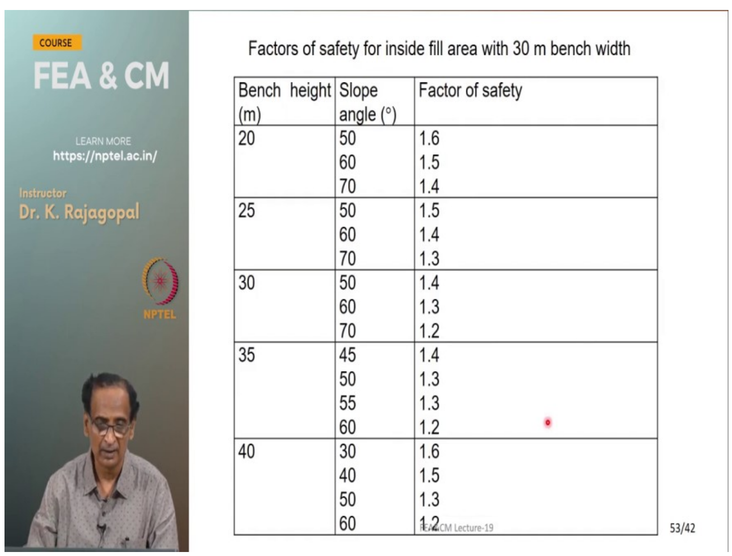

And for different bench heights and bench widths and so on

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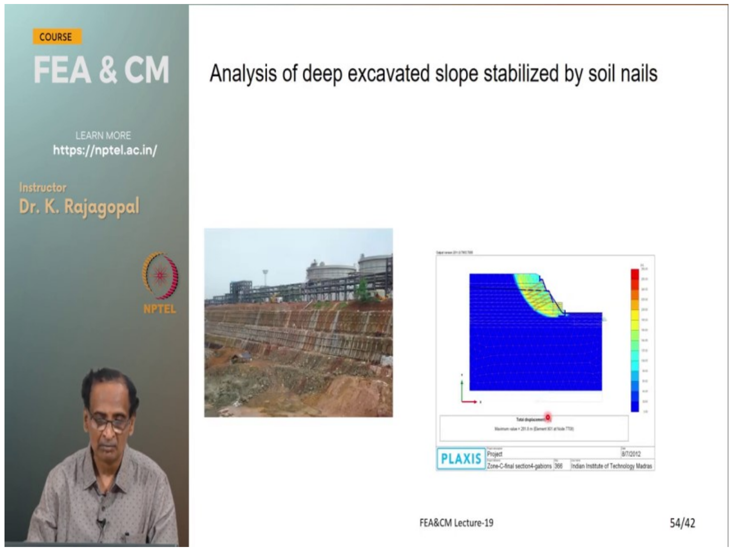

And say whenever you have a deep excavation this particular one is at some other site Manglore refineries MRPL where the deep excavation was supported by soil nailing this also we can simulate through finite element program and we can we can design the slopes. So, I think that is that is the end of my presentation. So, here in this lecture we have seen how to initiate the in situ pressures and then simulate construction gradual construction and then excavation.

And these are very typical of finite element analysis and most of the geotechnical related programs they allow you to do this. And also this B transpose Sigma is very important quantity that we can use for checking our equilibrium and also for doing the excavation problems. And the excavation problem is easy to do in our geotechnical programs but if you use a programs like Ansys or Nisa they do not give you the option for removing the elements or even if you remove they cannot release the stresses.

So, we cannot use them but Abacus has the ability to release the stresses but it is a bit complicated process whereas in Plaxis and the GEOFEM is very simple because it is they are already programmed for that. So, thank you very much we will meet next time. And if you have any questions please send an email to this address profkrg@gmail.com. So, thank you very much, bye.