Earthquake Geotechnical Engineering

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Lecture 42

Slope Stability Analysis (Conti.)

I welcome you again for this NPTEL online course on Earthquake Geotechnical Engineering. So, we are under module 5 of this course and we already covered introduction to earthquake induced landslides. First chapter is over, the second chapter static slope stability analysis is partly covered. So, we will discuss in this lecture. The second chapter that will be over and then we are going to talk the third chapter also seismic slope stability analysis.

Coming to the static slope stability analysis, there are two methods which we have discussed one is limit equilibrium analysis and another is stress deformation analysis. So, we already started in the last lecture discussion on limit equilibrium analysis, we will continue from there and this I have told earlier that most of the material is from Kramer's book. The static slope stability analysis, in case of this limit equilibrium analysis concept any slope theoretically if the factor of safety is above 1 in that case the slope should be stable or in practice the level of stability is seldom considered acceptable unless the factor of safety is significantly greater than 1 and criteria for acceptable factor of safety recognize that uncertainty in the accuracy which the slope stability analysis represent the actual mechanism of failure. So, those factors of safety may be more than 1, but we need to understand this factor of safety has been determined with some uncertainties.

What are those uncertainties? The first of the uncertainty itself the methodology that how accurate the slope stability analysis is the actually representing the actual failure mechanism which is occurring in the field. Normally these methods may be conservative side, but still it is not guaranteed that they represent the actual failure mechanism which is occurring. The second uncertainty could be in accuracy with the input parameters for example, shear strength, groundwater conditions, slope geometry it has been determined because when we carried out the slope stability analysis then we assume some geometrical property or the material properties that should represent the actual condition, but there are they are uncertain. The third could be the likelihood and duration of exposure to various types of external loading and the fourth the potential consequence of slope failure because if slope get failed then there may be a lot of catastrophic damage. So, as a result normally the factor of safety is kept quite high compared to only one in case of slope stability. Typically, it is suggested that minimum factor of safety used for slope design for a static case for the long term is at least about 1.5 for normal long term loading condition and about 1.3 for temporary slopes or end of construction conditions in permanent slopes. So, in any case the factor of safety should not be suggested, the factor of safety is suggested is at least 1.3 in any case.

Even if you keep factor of safety equal to 2 no problem if it is quite high compared to 1.5 it is alright. Coming to this continue with the limit equilibrium analysis when the minimum factor of safety of slope reach to a value 1 then the available shear strength of the soil is fully mobilized on some potential failure surface and the slope is at the point of incipient failure. So, what happens like you determine the factor of safety and you said that factor of safety is greater than 1, but then what happens some loadings have increased and the factor of safety start decreasing then factor of safety reaches to the value 1 then the failure will likely to take place along some the potential failure surface. Any additional loading will cause the slope to fail that is to deform. Now, when your slope start getting fail then again it will come in the equilibrium and until it reaches a configuration in which the shear stress is required for equilibrium are less than the equal to the available shear strength of the soil. So, this is the kind of phenomenon the landslide also when the landslide started occurring then again that will stop when again there is equilibrium between the shear stresses and strength. So, similar is the case for the slope stability also that once again there is your shear stress become less than the shear strength then it will be okay. Continue with this the limit equilibrium assumptions of reach perfectly plastic behavior suggests that the required deformation will occur in a ductile manner. Many soils however exhibit brittle strain softening stress strain behavior.

So, the behavior of the many soils is brittle, it could be straining softening and, in such cases, the peak shear strength may not be mobilized simultaneously at all points on the failure surface. So, you have a failure surface that does not mean that peaks strength will be mobilized at each and every point on the failure surface simultaneously. For example, this in this slide it is explained like in a good way how it has been explained here. You have let us say this what is in this figure a tau versus gamma that means shear stress versus shear strength and the peak value peak strength is mobilized at point A and if we see in the slope this is the potential failure surface, this which is maybe circular. Point A peak strength have reached.

Now other points B and C they will have the less strength compared to the peak value. So, what will happen? When the failure will start from point A. Now when the point A has get failed then what will happen? It continues in then after some time then this failure will start reaching to point B and C. So, what will happen? There will be redistribution of stresses. Now the stress at point A becomes a residual strength. So, it was the peak value in the first part A but now at point A you left with a shearing resistance drops from peak to residual strength. But now the point B got your peak strength and so that means earlier failure point was A now it moved to B and then C as a result slowly slowly your entire this surfaces this potential failure surface will get failed. So, this is the progressive failure in slope comprised of strength softening material. So, whatever is written is what is already explained here. So, the and the peak strength in n case may not be mobilized simultaneously which is the case first it is mobilized at point A then move to point B and then later on point C. In case of continue with the limit equilibrium analysis this should be first of all must be formulated with great care. Since the available shearing resistance of the soil depends on pore pressure drainage conditions those conditions must be considered carefully in the selection of the shear strength and pore pressure conditions for the analysis. In fact, you may be aware that there are two types of parameters one is called total stress another is effective stress or we say that you have the shear strength parameter total or the effective shear strength parameters. So, when there is increase in pore water pressure then your let us say if you have the C and phi first the without like in case of when without pore water pressure then the pore water pressure is generated then it will become C dash and phi dash. And normally the effective parameters are the values are less than the total.

So, that means effective parameters so this will reduce the strength basically. So, when the pore water pressure is present due to the excess pore water pressure present then there will be decrease in the shear strength parameters and those conditions must to be considered when we deal with the slope stability. Now, guidelines for the selection of the input parameter for limiting curve are available in literature and what those guidelines some of the guidelines says like we will discuss like particularly when we talk about seismic slope stability. Now, this was about the limit equilibrium analysis for static slope stability analysis. In another case static slope stability analysis also can be carried out stress deformation and method.

Why this is required? Because stress deformation analysis allows the consideration of the stress system behavior of soil and rock and most commonly performed using a finite element method. When applied to slopes can predict the magnitude and pattern of stresses. So, first of all this will in case of a limit equilibrium analysis you get only the factor of safety. But here using stress deformation analysis you can get the magnitude and as well as the pattern of the movement of the stresses at different locations and it can also help you to find out excess pore pressures in slopes during and after construction deposition. So, the good point that like if you move from limit equilibrium analysis then you can get the more information related to stress, excess pore pressure and most importantly deformations or which is completely missing in the limit equilibrium analysis.

Further nonlinear stress system behavior can also be considered. Complex boundary condition, irregular geometries and a variety of construction operations can all be considered in modern finite element analysis. So, the first of all the stress deformation analysis is done using FEM finite element method. Naturally it is not going to be as easy like as you have done for the limit equilibrium analysis can be carried out using the hand calculators but or maybe at the most spreadsheets but for the stress deformation analysis you require finite element method. So, as a result some software to be used.

The accuracy of stress deformation can be strongly influenced by the accuracy with the stress system model parameters. Many different stress system models have been used for stress deformation analysis of slope and each one has different constitutive models have their advantage as well as limitations. The accuracy of the simple models is usually limited to certain range of strength and strength paths. So, this was about stress deformation analysis which is related to static slope stability analysis. So, with this we covered both the topics of static slope stability analysis.

We finished limit equilibrium analysis as well as stress deformation analysis. Now, we are going to move the third chapter of this module which is on seismic slope stability analysis. We are going to introduce first after introduction we are going to talk about analysis of inertial instability and then we are also going to talk about pseudo-static analysis. In fact, pseudo-static analysis is a counterpart you can say in the kind of an extension of limit equilibrium analysis. In case of pseudo-static analysis is used for the dynamic loading or for or earthquake loading while limit equilibrium analysis is used for the static case. So, let us discuss first with the introduction seismic slope stability analysis. In this case, when we talk analysis of the seismic slope stress further complicated by the need to consider the effects of dynamic stresses which is induced by earthquake shaking. The effects of those stresses on the strength and stress system of slope deform materials. So, the effects of dynamic stresses which are induced by the earthquake shaking.

So, that will be one part. The second is what is the effect of these stresses on what is the constitutive relationship or we say stress system of slope materials. So, coming to this part based on these effects seismic slope stability may be grouped into two categories. One is called the inertial instabilities another is called weakening instabilities. The shear strength of the soil remains relatively constant, but slope deformations are produced by temporary exceedance of the strength by dynamic earthquake stresses. So, in this case there are two categories. One category is in case of inertial instability, the shear stresses which is generated due to the external loading may pass the shear strength. So, it may be more than the shear strength and but slope deformations which are produced by temporary exceedance and this is as a kind of a temporary exceedance. But in case of weakening instability, it could be earthquakes may serve like to weaken the soil sufficiently that it cannot remain stable under earthquake induced stresses. So, when earthquake induced stresses are there for example, liquefaction. So, flow liquefaction and cyclic mobility are common causes of weakening instability. In that case, the equilibrium will not be maintained and it will be failure. So, we will discuss the first part inertial instability in very much detail and we will introduce the weakening instabilities. Coming to the analysis of inertial instabilities, earthquake motions can induce significant horizontal and vertical dynamic stresses in slopes. So, when due to the earthquake there will be like stresses generated which could be horizontal and vertical in the slope. These stresses which are generated due to earthquake produce dynamic normal and shear stresses along potential failure surface within a slope.

When you have slope, then there will be a potential failure surface which is known many times called slip circle. So, and along this potential failure surface, dynamic normal and shear stresses will be produced. When superimposed upon the previously existing static shear stresses, the dynamic shear stresses may exceed the available shear strength of the soil and produce what is called the inertial instability of the slope. So, like you have the additional stresses which is generated due to earthquake or due to this loading and when these additional stresses are generated, these additional stresses are superimposed on already existing stresses which was due to the static loading. So, already because due to the static loading there will be stresses and when this slope is subjected to earthquake motion or earthquake shaking, then additional stresses will be superimposed upon that and this total stress may produce what is called inertial instability of the slope.

Now, when we talk about a number of techniques are there for the analysis of inertial instability, one of the methods is called pseudo-static analysis and the pseudo-static analysis produces a factor of safety against seismic slope failure and which is in the similar way as we discussed limit equilibrium analysis for the static case and the similar way as the limit equilibrium analysis produce a factor of safety. So, in the pseudo-static analysis you get a factor of safety even when you consider the loading due to earthquake and then there are other approaches which you will need permanent slope displacement produced by earthquake shaking and which is other approaches one of the stress deformation analysis similar to what we discussed for the static case. Now, we will discuss the first pseudo-static analysis in very much detail and other approaches will be discussed later. Coming to the pseudo-static analysis, the effects of an earthquake is represented by constant horizontal or vertical acceleration. So, what is considered like in the pseudo-static analysis the effect of earthquake is like two additional forces when the earthquake shaking is considered horizontal force as well as vertical force is considered.

Usually, the effects of earthquake shaking are represented by pseudo-static accelerations that produce what we call the inertial forces FH and FV which act through the centroid of the failure mass. So, first of all we check through the centroid, let us see here. So, in this case you have the slope A and B, in this slope W, W is the basically this W what is this W, this W is nothing but the mass or weight of this load of this slice triangle. So, if you go AB and let me put this point C, so if you have ABC and in this case the total weight is represented by W and mind it that irrespective of the slope angle beta, whatever the value of beta, this W will always vertically downward because it is a force, it is in the direction of gravity and in fact W is nothing but if you know the mass then you know W is mass of this multiplied by g acceleration due to gravity and this load will act always in the downward direction. Then you have two forces FH and FV and these two forces are due to seismic loading or earthquake loading.

FH is a force which is a horizontal seismic force. This horizontal seismic force though in actual earthquake condition there will be to and fro. So, let us see if I consider horizontal, then one side this will act this side, in another reversal this will act like this side. However, because when this FH is acting towards the slope then it will provide more stability to the slope rather than decreasing the factor of safety. So, that condition the loading of the earthquake loading lateral loading by which the slope is getting more stable is not considered rather a condition where slope is getting weakened is considered when FH is acting outward that condition is considered.

So, always FH will consider outward direction like going away from the slope. While the FV which is the force vertical force due to the earthquake loading in this case is shown upward that means opposite to the W but it could be upward or it could be downward it can be considered. So, what is in this case the magnitude of the forces is given by this relation FH equal to AHW by GE and simply KH and W and FV as also are finally KV and W. Where AH and AV are nothing but the horizontal and vertical pseudo-static accelerations while KH and KV are dimensionless horizontal and vertical pseudo-static coefficients respectively and W is the weight of the failure mass. So, W is the basically weight seismic weight. Now, the force horizontal force due to like due to this earthquake or shaking is represented by KH and into W where KH is dimensionless and KV is also dimensionless or in another words KH and KV can also be linked KH is nothing but AH horizontal acceleration divided by G while KV is horizontal vertical acceleration that is AV divided by G. So, the KH and KV are linked with the if you know the KH and KV then simply AH and AV can be find out by multiplying these factors with G. Now, if we know the value of KH and KV which are dimensionless horizontal and vertical pseudo-static coefficients then you can find the seismic weights. So, this is how it is done the value of KH and KV is assumed or it is like based on some PGA value this is decided once KH and KV is decided then rest of FH and FE is known to you and then we can proceed to find the factor of safety. The magnitude of the KH and KV should be naturally related to the severity of the anticipated ground motion.

So, naturally if suppose nothing is given then the KH and KV should be higher in higher seismic zone compared to the lower seismic zone. So, this was about this value of how to calculate FH and FE and this FH and FE is used here. So, forces acting on the triangular wedge of soil above a planar failure surface in pseudo-static slope stability analysis. Now, in this case when this force FH and FE is also acting FH is acting outward to the slope and FE is acting in the negative towards opposite to the W. In that case the factor of safety of this potential failure surface is given by this relation where C and phi are the parameters shear strength parameters because these are the effective shear strength parameters while

L is the length of the like length of the code length of the basically in this case L will be simply length from here to here.

Here this for the simplicity we are assuming that this is a triangular rectangular or triangular, but this could be possible that you have a slope and in let us say for example this is slope, but you consider a circular failure surface. So, in that case circle there could be the circular failure surface also. So, that also be considered and that is not an issue. Coming to this like L, L is this distance from A to B along the failure surface and C and phi we already defined W is the weight of the slice and FH and FE are already defined where FH is nothing but KH into W and FE is nothing but KV into W. C and phi are the more column strength parameters thus describe the shear strength on the failure plane and L is the length of the failure plane.

Continuing the pseudo-static analysis, it can be observed that the horizontal pseudo-static force decreases the factor of safety which is very clear from this equation. You see the FH is decreasing the numerator because negative sign is coming while it is increasing numerator or denominator. So, as a result the factor of safety will decrease when you consider some value of FH and while FV as far as FV is concerned FV is decreasing numerator as well as denominator. So, as a result the effect of FV is not so much. So, the vertical pseudo-static force typically has less influence on the factor of safety since it reduces or increases depending on its direction.

If direction is upward then it reduces. If direction is downward then it increases both the driving force and the resisting force. So, it will increase the driving force as well as resistance either increase or decrease. As a result the effect of vertical accelerations are frequently neglected in pseudo-static analysis. So, when you use this equation many times it is done that FV is assumed 0 there is no this one.

So, this is cancelled out and you left with FH only. This equation can be further simplified when c equal to 0. If you have the c equal to 0 then FH and FV can be represented in terms of w and then w will be cancelled out from the numerator as well as the denominator and the equation becomes more simplified. So, the result of pseudo-static analysis is critically dependent on the value of the seismic coefficient k H because k v can be neglected k v can be assumed as 0, but in any analysis, you should not do other way that you are assuming k v and k H equal to 0. In any case k H should be considered k v may be considered or may not be considered that is optional. So, when the k H is considered now the issue comes what value of k H should be taken for the analysis.

The selection of an appropriate pseudo-static coefficient is the most important and most difficult aspect of a pseudo-static stability analysis. The seismic coefficient controls the pseudo-static force on the failure mass. So, its value should be related to some measure of the amplitude of the inertial force induced in the potential unstable material. So, when we want to link with that in that case naturally the value of this coefficient k H will depend on your zone in which zone, seismic zone whether seismic activity is high or low depending on that it can be decided. If the slope material is the inertial force induced on a potential failure surface slide would be equal to the product of horizontal acceleration and mass of the unstable material. Whatever the view of the horizontal acceleration and the mass of the material which is unstable if you multiply by that then you will get the force. However, actual slopes are not reached which is the considered in limit equilibrium analysis and which is extended in the pseudo-static analysis. So, the actual slopes are not reached and that the peak acceleration exists for only a very short time the peak value comes only momentarily it is not last for a long. The pseudo-static coefficients used in precise generally in practice generally correspond to acceleration values which is linked with the A max or you can say that the value just for pseudo-static coefficient you should give you the acceleration corresponding acceleration which is quite below A max. So, basically what is said here if you have the value of A H which is nothing but k H multiplied by g.

So, this way you can find the value of A H and g acceleration due to gravity and once this is known A H is known then the value of A H should be less than the value of A max it cannot exceed the value of A max which is a p g a. So, in any case it can be more than p g a value that this acceleration due to this horizontal acceleration or vertical acceleration. Now, coming to this one like the how the values of this k H can be estimated. Terzaghi in 1950 originally suggest that the k H can be assumed 0.1 for severe earthquake while k H can be assumed 0.2 for violent or destructive earthquake. So, this was by Terzaghi long back very conservative approach. For pseudo-static coefficients should be based on the actual anticipated level of acceleration the failure mass then it should be a correspond to some fraction of the anticipated peak acceleration. So, it is said that it should be linked with the peak acceleration the value of it should be some fraction of the peak acceleration.

So, for example, if we are living in seismic zone 5. So, in the seismic zone 5 you have the value of k H will be one third of 0.36 one third of 0.36. So, will be 0.12 to half will be 0.18. So, the value of k H which should be taken for the analysis for zone 5 like from as a thumb rule should be lying between 0.1 to 0.18 and an average value k H can be taken as 0.15 and this 0.15 is the value for the sesame zone 5 and this is for the highest zone which is zone 5. So, for the zone 5 it could be taken as 15 percent 0.15. Representation of the complex transient dynamic effects of earthquake shaking by a single constant and unidirectional process and naturally this is quite crude that we are representing by earthquake force by a single parameter k H, k H multiplied by W will give the horizontal force due to earthquake. Terzaghi stated that this slope could be unstable even if the computed pseudo-static factor of safety is greater than 1. This we already discussed. Experience has already shown that pseudo-aesthetic analysis can be unreliable for soils that should be produce large excess pore pressure or show more than about 15 percent degradation of strength due to earthquake loading. So, there may be two reason one is the soils which produce large earthquake pore pressure.

The second reason could be that there could be degradation of strength which could be as high as about 15 percent degradation of strength is taking place then we need to call off this. Now, it has been observed in the past then some dams have been designed using pseudo-static approach and those dams design having the factor of safety quite high than 1, more than 1 factor of safety and they have been designed using pseudo-static approach. The limitation is saying that even those dams have designed using pseudo-aesthetic approach but they get failed during some of the earthquakes. Like for example, Sheffield dam in US have complete failure even the factor of safety was more than 1.2 and the k H value was considered 0.1 which is quite reasonable. For lower San Fernando dam or upper San Fernando dam, the value of k H is taken 0.15 that means higher than what has been considered for Sheffield dam and still the factor of safety for the lower San Fernando dam was 1.3 but for upper San Fernando dam it is 2 to 2.5. So, that means this is like quite high compared to what you have for other two cases.

Then tailings dam in Japan has a factor of 1.3. So, downstream shell including crust slip 6 feet downstream, failure of dam with release of tailings. So, this data you have the factor of safety more than 2 that is between 2 to 2.5 that is for this case is quite heavy value for this factor of safety. But still even you have this much factor of safety but still your dam gets failed and the reason being is here because you have downstream shell including crust slip.

So, this was about. Now, coming to the advantage of the pseudo-static approach, the analysis is relatively simple and straightforward. Indeed, it is significantly to the static limit equilibrium analysis routinely conducted by geotechnical engineers make computation easy to understand and perform. So, here it is easy, you know the pseudo-static analysis easy to grasp, easy to implement and easy to understand. It produces a scalar index of stability that is a factor of safety. How the accuracy of the pseudo-static approach is governed by the accuracy with which the simple pseudo-static inertial forces represent the complete dynamic inertial forces that actually exist during an earthquake.

So, with this I conclude this session that is the lecture number 42 and we already covered in the seismic slope stability analysis one part which is pseudo-aesthetic analysis. In the next lecture we will discuss other approaches including Newmark's method. Thank you very much for your kind attention. Thank you.