Earthquake Geotechnical Engineering

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Lecture 29

Local Site Effects (Continue)

I welcome you again for this NPTEL online course on earthquake geotechnical engineering. We are discussing module 3 which is on ground response analysis and local site effects including soil-structure interaction. So, we have already discussed ground response analysis which was the chapter number 1 with 4 lectures and then we talk about soil structure interaction in next 2 lectures and local site effects we have the 4 lectures. So, we have already started this local site effect 2 lectures are already over. Today we are going to have another 2 lectures on the local site effects that is lecture number 29 and lecture number 30. So, what we are going to talk today in this lecture is design parameters which are used for the constructing the local site effect, what will be the difference on the parameters.

Then one important issue is design earthquakes that is there are for the design of the structures and facilities what should be which earthquake we should use. Then design spectra, then next is development of design parameters and finally, we are going to talk about site specific development which is also part of the topic number 4 that is development of design parameters. In fact, development of design parameters can be done either using codal provisions or using site specific development. So, let us talk about design parameters.

When we talk about design parameters first before I go ahead let me acknowledge that this Kramer's book, this is a major source for this information which has been collected in this lecture. So, coming to design parameters, earthquake resistant design of structures and the evaluation of the safety of existing structures will require the analysis of their response to earthquake shaking. So, how they respond? When these structures are subjected to earthquake shaking what will be their response? Whether it is for the new structure or for existing structure it is important. And evaluation of geotechnical hazards for example, which include the liquefaction and slope failure will also require analysis with respect to some level of shaking. Now the issue is level of shaking should be used. For which satisfactory performance is expected like often referred as a design level of shaking and will be described by design ground motion.

So, design ground motion we are going to talk later. So, we need to select the level of shaking that which is used for the design of the structure's satisfactory performance during the earthquake. Coming to this point what is the design earthquakes? We are going to talk in the next few slides about design earthquakes. When we go in the history design parameters were most commonly determined from a specified design earthquake. So, for design you need a design parameters and to have the design parameters we require design earthquake.

And some regulatory agencies still require that earthquake resistant design of structures we perform with respect to the motion produced by the design earthquake. So, you need to select first design earthquake and on the basis of the design earthquake the structure of facility need to be designed for that earthquake. The specifications of a design earthquake will implies a level of determination in the seismic hazard analysis that we already discussed which is called SHA or there are two part of seismic hazard analysis DSHA and PSHA we already discussed in detail. In that case the design earthquake is characterized either could be deterministically or probabilistically we as we discussed earlier and its effect at the site of interest or that need to be computed deterministically. So, we can select the design earthquake based on the seismic hazard analysis which could be deterministic or probabilistic.

But once you select a design earthquake then rest of the things can be done as a deterministic. Continuing with that historically design earthquakes have been associate with the two level of design in which a structure of facilities required to remain operational at one level of motion. So, one level of motion will be that level where even that our level of earthquake or shaking occurs, but still your structure or the facilities operational functional no issue. And the second is a level of shaking where it may not be operational, but we need to avoid catastrophic failure. So, that earthquake comes, but there is no catastrophic failure to the structure.

Now, to define design earthquakes many different terms have been used to describe the level of severity which is associated with the design earthquake and different organizations may put different names for design earthquakes. The maximum credible earthquake which is in the short called MCE and this is normally defined as the largest earthquake that can reasonably expected from a particular source. Then you have SSE what is called the SSE safe shut down earthquake, which is mostly used in the design of nuclear power plants, and this is maximum peak horizontal acceleration where the operation of nuclear facility need to be stopped automatically. This is normally done whenever this occurs then the nuclear power plants are stopped with its operation. In many organization the MCE and SSE have similar characteristics.

So, MCE are the earthquake, which is the highest level of shaking where one can assume to avoid the catastrophic damage. The other terms to use this maximum MCE instead of

MCE which are popular maximum capable earthquake, maximum design earthquake, so many terminology are there, but mostly MCE and SSE are most popular. So, the two level design approach will require that structure of be designed to avoid catastrophe. The second level is basically to first level is either it is MCE or SSE. So, the first level is to design to avoid catastrophic failure at the levels of shaking and the second one is upper level design earthquake.

So, the upper level like you have one side MCE or SSE on another side you have OBE. So, when we design our facility and the structure they may be designed for MCE in general and OBE. MCE will be maximum credible earthquake where what we have in case of maximum credible earthquake will be based on that the earthquake where you design the structure, but there should not be catastrophic damage. At MCE level we do not expect there your facility or the structure to be operational or functional, but it should not collapse. Well at OBE which is basically operating basis earthquake, at OBE and this is an earthquake where it is expected during the life of a structure that means during when the OBE occurs still the structure or facilities functional or operational.

Normally the OBE is taken as an earthquake with half the peak acceleration of the SSE which was the different authors have given different like Christian in 1988 said that it could be half of the peak acceleration which is taken for SSE or an earthquake which produces motion with chances are that 50 percent probability of exceedance in 50 years. So, in a span of 50 years there are chances that this level of shaking may be exceeded, but chances are only 50 percent not 100 like no more than that. This is by United States called that is the committee on large dams in 1985. Then another researchers have said that with a return period of about 110 year. So, normally OBE is linked with MCE and most of the time it is taken half of the MCE approximately half of the MCE in the simple definition.

Continuing with this, other terms that are used for OBE operating level earthquake, maximum probable earthquake, design earthquake and resistance level earthquake. So, for MCE you have different terms similarly for OBE you have a number of terms, but ultimately you have two design earthquakes one is MCE, and another is OBE and these two level design requires that the structures and facilities be designed to remain operational after being subjected to the level of shaking which is associated with lower level design earthquake that is OBE. So, after OBE it should be operational, but if it is structure of facility subjected to an earthquake shaking which is equivalent to MCE at that point, we may not expect that the structure or facility is operational. However, there should not be collapse or there should not be catastrophic damage even at MCE level. So, this was about design earthquakes.

Now the second part that is design spectra based on the design. So, as for the design spectra there are two types of spectra one is called response spectra and another is design ground motion. So, the response spectra are often used to represent seismic loading for the dynamic analysis of structures. So, one side you have response spectra. Then design ground motions are aspects in terms of design spectra.

So, you have the response spectra and using the response spectra we prepare the design spectra which will be used for design. Response spectra can be said like when you carried out the seismic analysis of a structure what is the response of the structure that will give you a response spectra which is normally response versus period or response versus the natural frequency. So, these design spectra and response spectra are not the same that is based on the response spectra you prepare the design spectra, but they are not exactly the same thing. The response spectra from earthquakes are highly irregular, their shapes reflect all the details of their specific frequency content and phasing. So, when we prepare the response spectra of a particular structure which is the response against a given ground motion and that spectra is not smooth, but the design spectra is smooth.

So, on another hand the design spectra generally required quite smooth thing, they are usually determined by smoothing, averaging or enveloping the response spectra of multiple motions. So, that is the answer here that to prepare the design spectra you use multiple motions, you collect the response spectra from different many earthquakes which may be applicable and then you prepare kind of envelope, and that envelope will be yielding your design spectra which will be used for the design of the structure or facility. The use of a smooth design response spectra implicitly recognize the uncertainty with which soil and structural properties are known by avoiding sharp fluctuations in spectral acceleration with small changes in structural period. So, smoothing is done because we want to design a spectra as a smooth spectra. As a result, we avoid the sharp fluctuations which comes in a spectral acceleration and due to small change in structural period.

So, Newmark and Hall in 1973 recommended that the design response spectra be developed from a series of straight lines on a tripartite plot corresponding to this plot tripartite means 3, there will be 3 axis in fact 4 axis on this plot. So, on x axis you have period, but on y axis there will be split into 3 axis that is why it is called tripartite plot which we are going to discuss in detail which corresponded to acceleration, velocity and displacement control portions of the spectrum. And a Newmark Hall design spectrum is obtained by multiplying the Peak ground acceleration, velocity and displacement values by the factors which are shown in the next table. So, the spectrum for this amplifying factor you obtain the Peak ground acceleration and for Peak ground factors once you obtain the Peak ground you know the peak values then, amplifying factor for displacement velocity and acceleration they depends on the damping ratio. As the damping ratio increases from 0 to 20 percent these factors decreases.

While at the same damping ratio the factors value is maximum for acceleration and minimum for displacement. So, for example 2.5, 4 and but the first line because you may not have 0 damping. So, let us say I may have 5 percent damping in the system then the realistic value will be this one 20 percent will be too high. So, in that case the factor which is multiplier acceleration is 2.6 for velocity 1.9 and displacement 1.4. So, using this table one can select depending on the damping ratio the multiplier for this value. Now, as we said that tripartite plot is there. So, let me discuss that tripartite plot first.

In this case tripartite plot is shown here on x axis you have the period which is in seconds and then you have corresponding velocity. So, this line will give you the velocity. So, here we see for a particular case spectra the velocity remain constant between this period and this period 22.8 second. So, in this case one side you have the velocity for the same period if you want to calculate the acceleration. So, for the acceleration this axis which is 45 degree should be used. So, the axis for acceleration is this one. This is the axis which is for acceleration in g while another axis 45 diagonally position displacement in h. So, let us say from 1 second when I hit this here then I can read the velocity and for wherever it is hitting this will be corresponding acceleration axis and then you have corresponding this is acceleration value on this axis while displacement is given from this axis. So, to calculate the value of velocity acceleration displacement you will be using these two axis while for velocity vertical axis is used.

For acceleration displacement the axis are 45 degree. So, at periods below the continue with this Newmark Hall design spectrum what was suggested by Newmark Hall design this table is coming from the Newmark Hall design spectrum. It was also suggested if you have period of below about 0.17 second that is frequency above about 6 hertz in that case the spectral accelerations are tapered down to the peak ground acceleration, a peak ground velocity of 48 inch per second and peak ground displacement of 36 inch are assumed to be consistent with p g a of 1 g. So, here if you have this below about 0.17 second for example, the period in this case is even for this velocity is less than 1 second that is. So, here at this point at 0.17 second this is tapered down and it is tapered down to 0.25 g.

So, this tapering is done because this period is at 0.17 second. So, if this is the case in that case for 1 g acceleration you get this peak ground velocity is 48 inch per second. If so, suppose your g is increased then you need to multiply whatever the g value you have that should be multiplied by 48 inch per second to get calculate the velocity and similarly to calculate displacement it should be multiplied by 36 and naturally this table should be also used. So, here is one example develop a Newmark Hall design spectrum for 5 percent damping and peak ground acceleration 0.25 g.

So, here whatever data for 1 g that should be multiplied by 0.25 g. So, you have 48 inch per second was for 1 g. So, naturally at 0.25 g it will be one fourth. So, this factor comes out to be one fourth and then the multiplier which is from the table 1.9 for the 5 percent damping these factors are 2.6, 1.9 and 1.4. So, for the spectral velocity this should be multiplied by 48. Similarly for spectral displacement 36 inch is for 1 g and for 0.25 g it will be one fourth. So, this factor is one fourth of nothing but 1.4. So, it will be one fourth

36, 9 into 1.4. So, you get 12.6. Here 12 into 1.9. So, you get 22.8. For acceleration directly you multiply by factor because for 0.25 g you have 2.6. So, 0.25 g is there. So, multiply by 2.6 give you 0.65 g. So, these are the values of spectral acceleration, spectral velocity and spectral displacement using what you call the Newmark Hall design spectrum, and these values are at 4, 5 percent damping. Now we continue and how we go to development of design parameters.

As for the development of design parameters are concerned you have the characteristics, what are the parameters of the design of ground motion at a particular site. At a particular site these characteristics will be influenced by the location of the site, relative to potential seismic sources, the seismicity of those sources, the nature of rupture at the source, travel path effects between the source and the site, local site effects and the importance of the structure of facility for which the ground motion is to be used. So, there are so many factors which will influence the characteristics of design ground motion including the importance of the structure. The manner in which the motion is to be used should also be considered for this. As for development of design ground parameters are concerned, they are developed by two ways.

One way is what we call the site-specific analysis, and another is called by the provision of building codes and structure. The first one, site-specific analysis we will discuss in detail. As for the building codes are concerned these design parameters are already given for the different seismic zones like for example, in our country we have 4 seismic zones and highest is 4th and 5th seismic zone. So, for site-specific development, site-specific design ground motions reflect the detailed effect of the particular subsurface condition at the sites of interest. You may have some sites of the interest and those sites of the interest we want to develop we want to determine site-specific design ground motion.

The usual process for developing site-specific ground motions involves a seismic hazard analysis and a ground response analysis. So, we have one side as such, on another side we have GRA. So, GRA we have discussed in very much detail that is one dimensional ground response analysis and two dimensional ground response analysis. Developing a site-specific ground motion will involve this is seismic hazard analysis where seismic hazard analysis was also discussed as a DSHA and PSHA. The use of probabilistic seismic hazard analysis will require the design motion be associated with some level of risk or return period.

Selection of such a quantity can be quite complex, various social, economic and political considerations are also involved when we develop site-specific development. In fact, it would be rather than political, it would be more on the importance of the structure, how big is your project. For example, if you are going to have a bridge or dam or a hospital, then they are considered to be very important structures and in that case, we use what we call the stringent criteria for the design. So, continue with the site-specific development, the seismic hazard analysis that is SHA will produce a set of ground motion parameters that may or may not correspond to the subsurface condition at the site of interest. So, normally our target is that they should be corresponding to the subsurface condition at the site of interest.

Both the deterministic and probabilistic seismic hazard analysis utilize predictive relationship that usually correspond to a fairly narrow range of subsurface conditions if the site of interest is located on a similar profile. So, you have like site of interest is located, one side you have site of interest. On another side, you have a record, the record which has been recorded by the seismograph or the accelerograph may be recorded at different sites, not at the same site where your site of interest or site of construction is there. So, if both sites have the similar characteristic, in that case, the parameters may be then taken directly as the design ground motion parameter. If not, in that case, seismic hazard analysis must be modified to account for the effect of local site conditions and how we can account and how we can modify this design ground motion accounting for the effects of local site condition we are going to discuss in the next slides.

For example, here the parameters modification process may be performed empirically using prior observation or analytically. So, if you have the prior observation, then you can know that by thumb rule or that way you can modify them. But in the analytical approach, what we say both deconvolution and conventional ground response analysis may be required and as shown in the slide, and this is how it is carried out. I will explain this slide.

This slide is in fact in two parts. So, you can say that this is like here, rock outcropping motion at C is there. So, I divided like this. So, in the first part, you are starting from A, B and going to C. In the second part C is common, in both parts C is common.

From C you are going D and D to E. So, this is the way how like you are having. So, let us say the surface motion from seismic hazard analysis is available to point A and point A is not at the rock outcropping motion. Rather, the point A is located on a top of a soil column. Below point A, you have a soil column and below the soil column, you have the point B which is called bedrock motion at site corresponding to predictive relationship. Now, for this site, you would have the predictive relationship between A and B.

Between A and B, you have the soil, but for soil, predictive relationship is known, what could be the attenuation relationship or that is known to you. So, what is done because predictive relationship is known, we use this as a result like characteristics of the site and using those data, whatever the motion given at point A is deconovulated to find the motion at point B. Now, point B, you find the bedrock motion at site which is corresponding to predictive relationship. Using that B, we determine what we call the rock outcropping motion point C and then point C, we go to point D like a rock outcropping motion is known, then we find the bedrock motion at site of interest and here there is a difference between B and D. B and D are not going to be the same motion. At this point C and point C will be the same, there is no difference at all, but at point B and D because at on the top of B, you have some soil cover, its property may not be the thickness, or the properties may not be the same as you have the soil cover above the D. So, even motion at point C is same, but it is still motion at point B and point D will not be the same. So, C is common here and once C is known, then you would remain the motion at point D which will have the influence of the soil cover on the top of D. Once motion at point D is known, then we carried out conventional ground response analysis or convolution and find the surface motion of site interest. So, we started from point A, this is the beginning, this is the large, the beginning and this is at E, this is end.

So, in this case, when you do this exercise, the properties of the site between A to B is also involved and D to E is also involved soil profile. So, these steps are detailed out and like they are like listed here. The seismic hazard analysis will produce parameters that describe at ground motion at the surface point A of a site with subsurface conditions that correspond to those of the site in the database from which the predictive was developed. So, that means the motion at point A is given and the point A is a site with subsurface conditions which correspond to those of the sites in the database which is available to you where the predictive relationship was developed, predictive relationship between point A and B. To determine the corresponding parameter of the surface of the site of interest at times of ground surface motion that is consistent with the predictor is generated and how it is done? The motion first is deconvoluted.

So, this is the first step. In the first step, the motion is then deconvolated through soil profile corresponding to the predictor to determine the time history of bed rock motion at point A that would produce the time history of motion at point A. This corresponding rock at cropping motion produces the bed rock motion applied at the base that is point D. So, like you have this. So, this is the first step and in the second step, once you find at point B, then you from point B to point you go to point C and then finally you go to point, so this will be, so point A to there will be one step, go to point C and then this will be the second step and in the third step point C to point D, point C to point D. And point C to when you go point D, then your soil profile comes in picture.

Then you carried out a conventional ground response analysis. So this is the fourth step basically. So, in the fourth step, you carried out the GRA. So if I go back, so you can see the first steps, second step.

In the first step, you go A to B. In the second step, you go B to C. In the third step, C to D and in the final step, fourth step D to E. So, using these different steps, you carried out the site specific development. So finally, the motion which you find at point E will be consistent with the results of seismic hazard analysis and also with local site conditions and can be taken as a design ground motion. Because why this can be carried out design ground motion? This will be your final answer.

Because it will have effect of seismic hazard analysis as well as local soil conditions. And it can be used to compute site specific design parameters such as peak acceleration or peak velocity and response spectra, ordinates and duration. So thank you very much for your kind attention. Thank you.