Geotechnical Earthquake Engineering
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Module - 8
Lecture - 33
Site Response Analysis (Contd…)

Let us start our today's lecture for NPTEL video course on geotechnical earthquake engineering. We are going though now, module eight of this course that is site response analysis.

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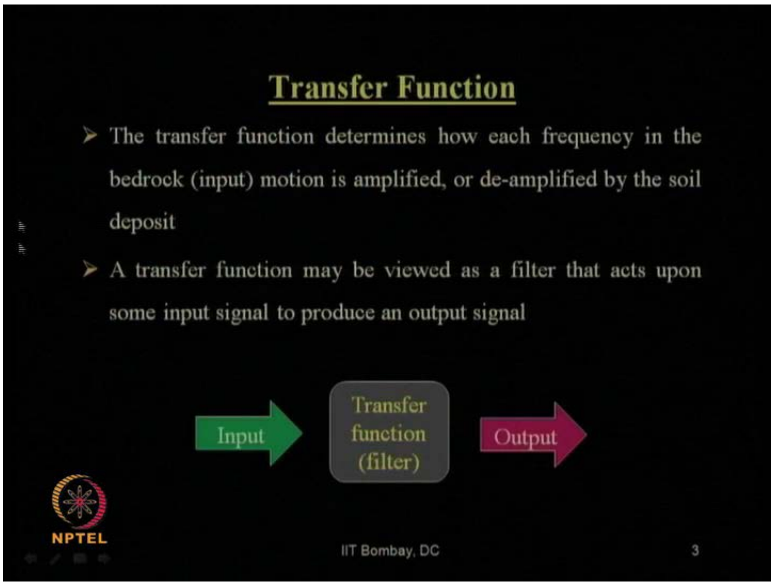

Now let us have a quick recap, what we have learnt in our previous lecture. Like we started defining, what is called transfer function? It is nothing but a filter or a multiplier which changes input ground motion to an output at another level.

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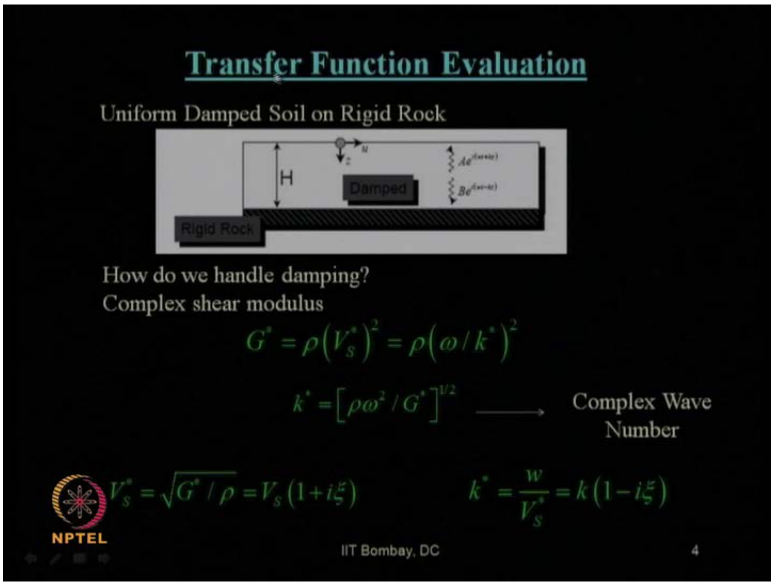

So, we have seen various cases, how to estimate the transfer function? Like for uniform damped soil, this is the way we calculate the transfer function.

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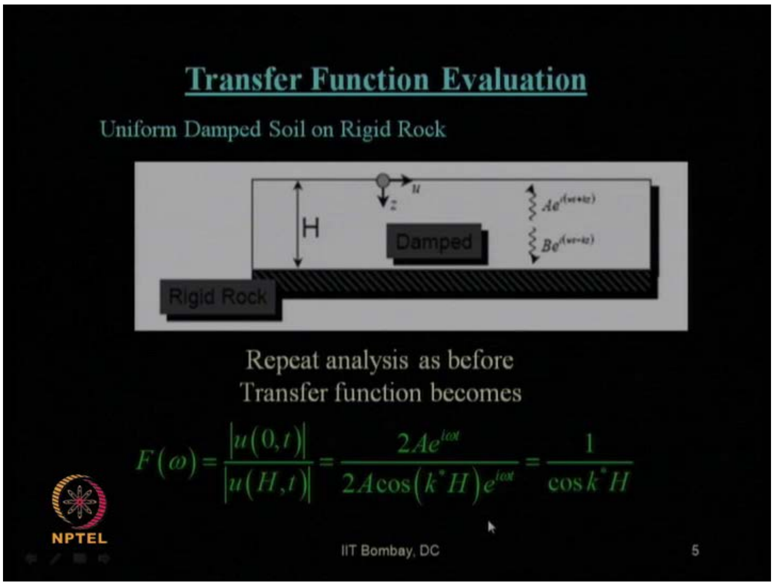

And, this is the transfer function expression which is nothing but the ratio of the displacement function at ground to that at the rock level; that is interface between the soil and rock.

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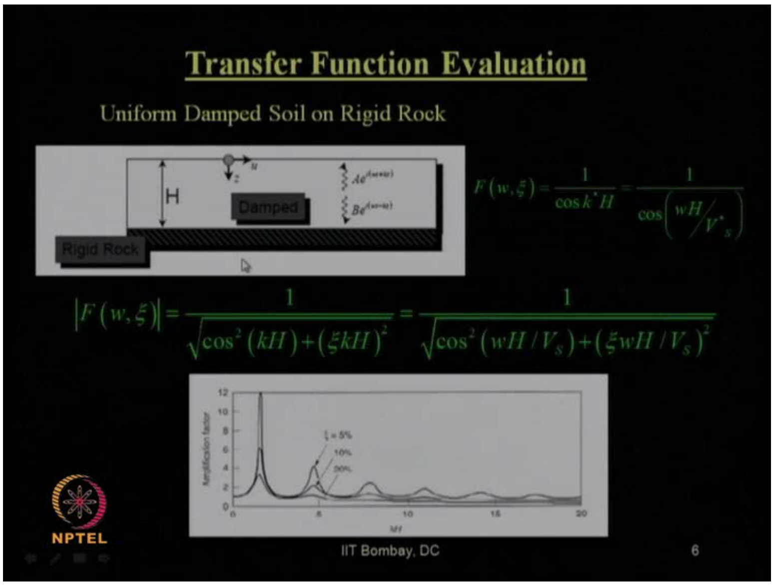

Then we have seen, what is the expression for this transfer function, which is a function of the frequency and the damping ratio expressed in this pattern. So, which is nothing but can be mentioned as amplification factor in terms of k H, so we can see, as the damping ratio increases, the amplification factor will decrease; and so on with number of cycles.

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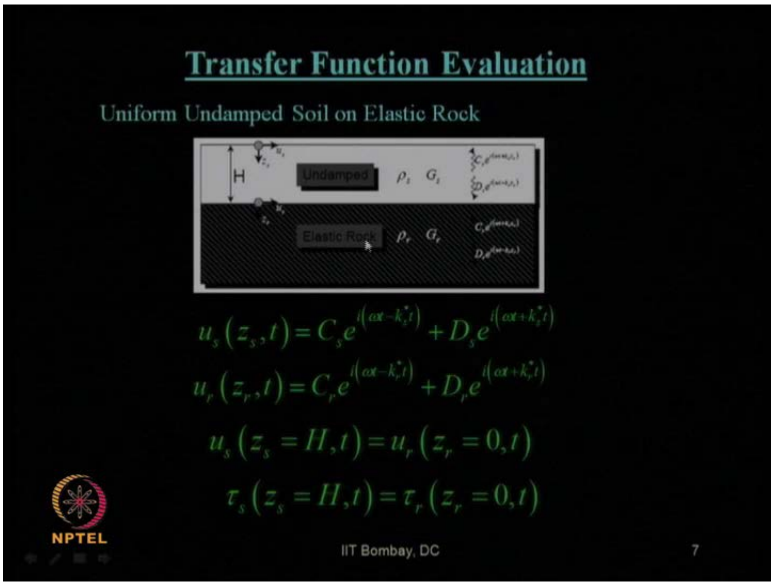

We have talked about uniform un-damped soil on an elastic rock; earlier we talked about rigid rock and then we came to consider the elastic rock which is more practical.

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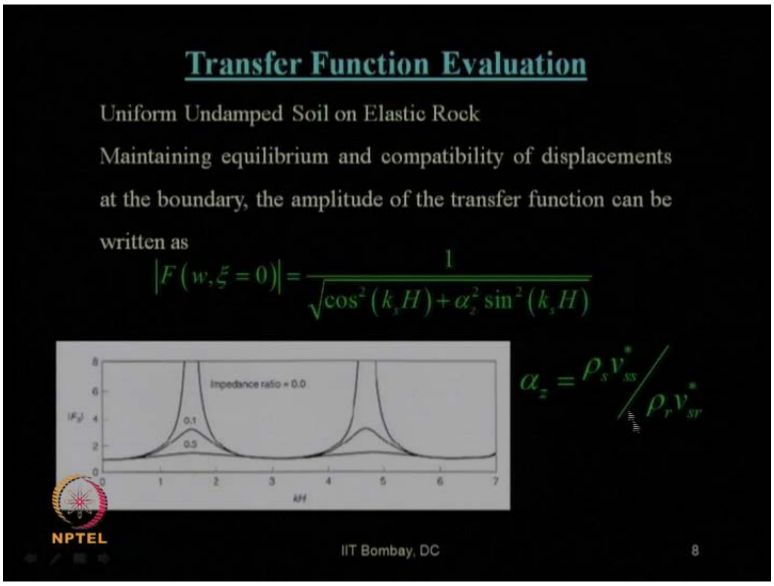

For that, we have seen, even though we do not consider the damping of the soil that is the material damping or the viscous damping of the material we have considered. But even then because of the impedance ratio of non zero value between this elastic rock and the soil, you will get the non infinite value of the transfer function, at the particular value of k H like this. So, this is the expression for transfer function when the specific impedance ratio between the soil layer and the rock layer is considered, that is considering the elastic rock.

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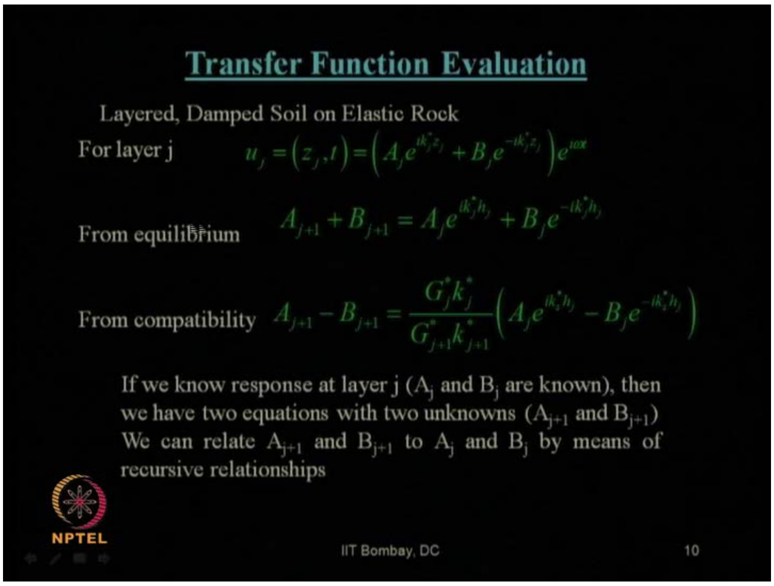

Then, we have talked about most generalized case of layer damped soil on elastic rock. So, for that we have seen that the displacement compatibility and the stress equilibrium between the layers have to be maintained which by known boundary condition will give us the values of various coefficients.

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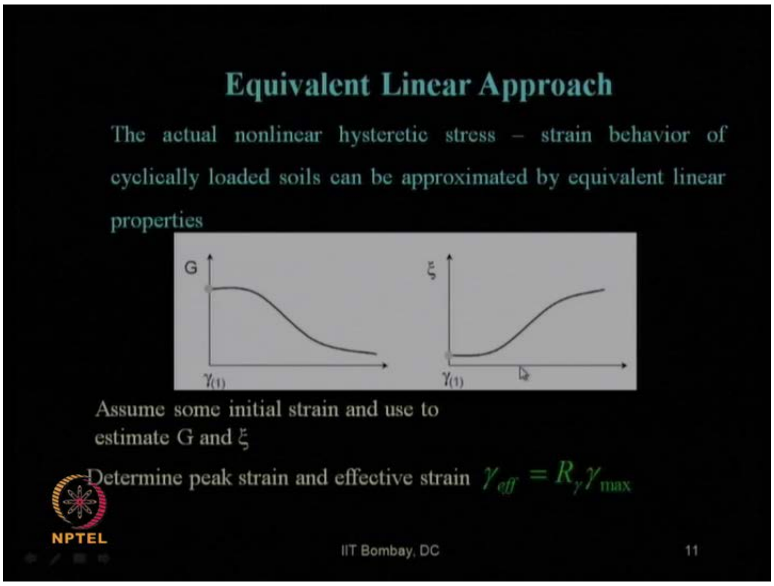

We have also discussed how to carry out the equivalent linear ground response analysis. This is how the modulus reduction curve or G by G max ratio over the cyclic shear strain varies when we are talking about g secant that is secant modulus; and the how the damping ratio varies with respect to the cyclic shear strain.

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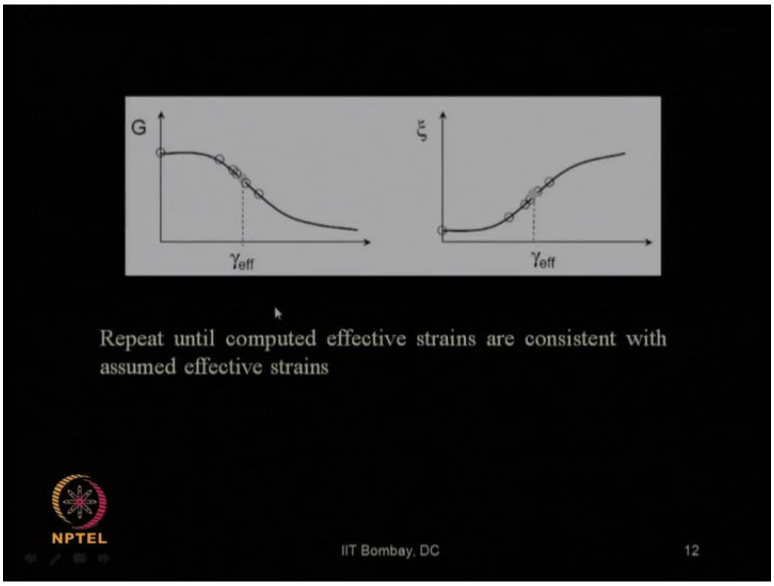

So, we need to choose initially one initial value of the cyclic shear strain to start the analysis; and until the results of assumed strain and the final output strain after using the chosen value of G and eta, we get matches, those two strains we have to repeat this. So, this is a trial and error procedure or iterative procedure by which finally we will get that gamma effective.

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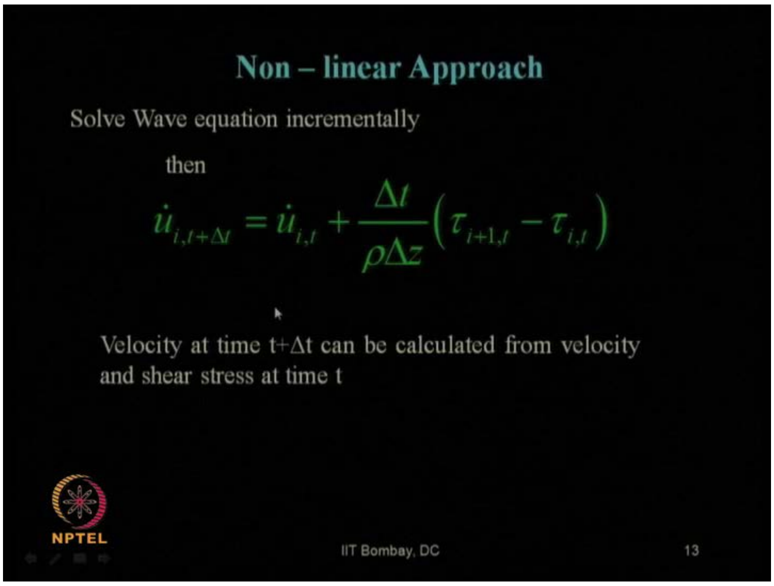

However for non-linear approach we cannot use that modulus reduction curve. We need to then use the equations, solve the wave equation incrementally that is a small steps of the back bone curve of that shear stress versus shear strain, into a small time interval steps.

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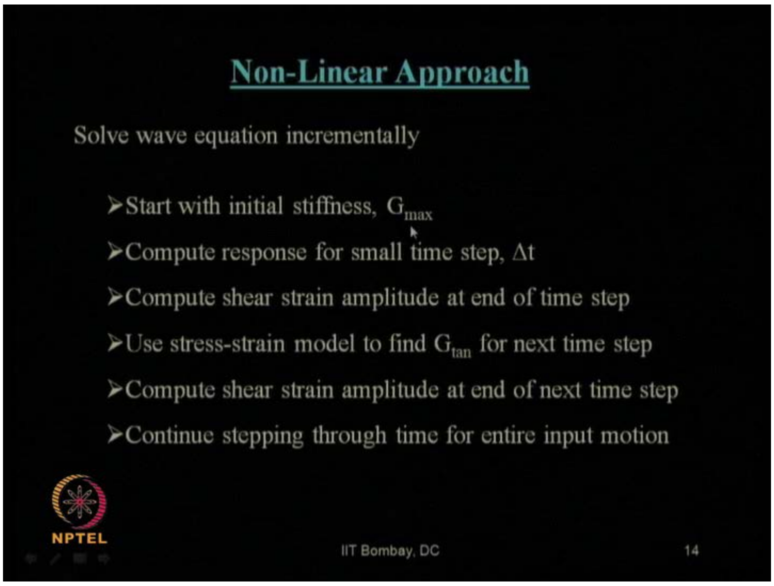

Now to do that, these are the steps we have seen. We have to start with initial tangent modulus which is nothing but G max; then use small time step of delta t that time step has to be chosen based on the entire time duration of the input ground motion, or input seismic motion whereever level we are considering, that time has to be subdivided into number of small time segments. Then you have to compute the shear strain amplitude at the each end of time step. That will give the corresponding G tangent that is individual tangent modulus for each and every time step. That finally will lead to the shear strain amplitude at end of each time step and it has to continue for the entire time duration.

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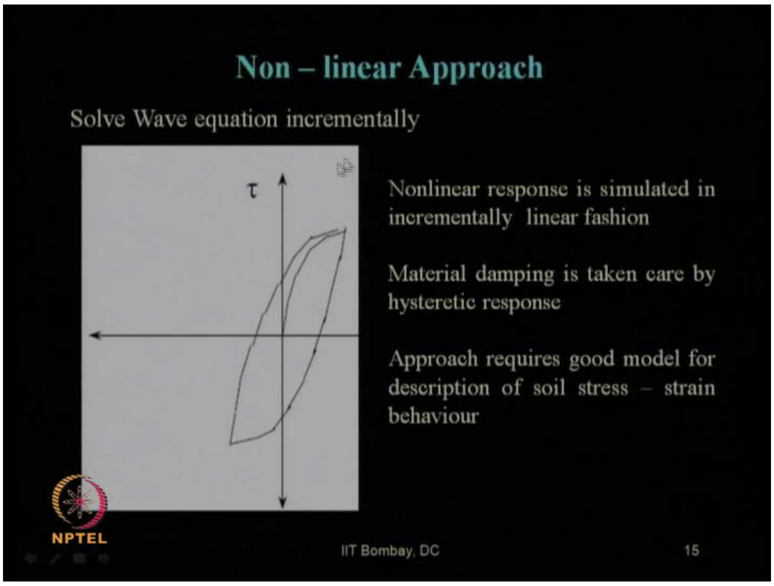

So, by using this incremental procedure of analysis, we will get this backbone curve of non-linear response which automatically considers the material damping by this hysteretic response. And this approach requires a good model to understand the behavior of this stress stain.

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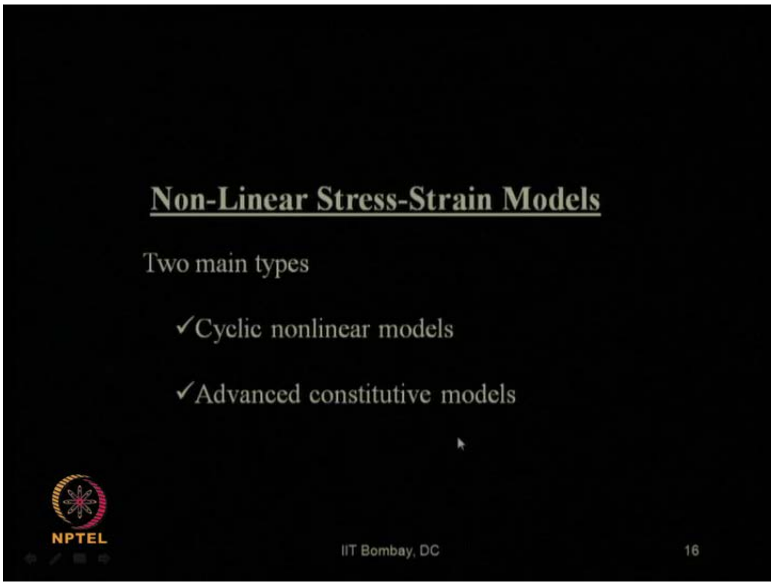

So, what are the good models? Two major types of models one is cyclic non-linear model, another is advanced constitutive model.

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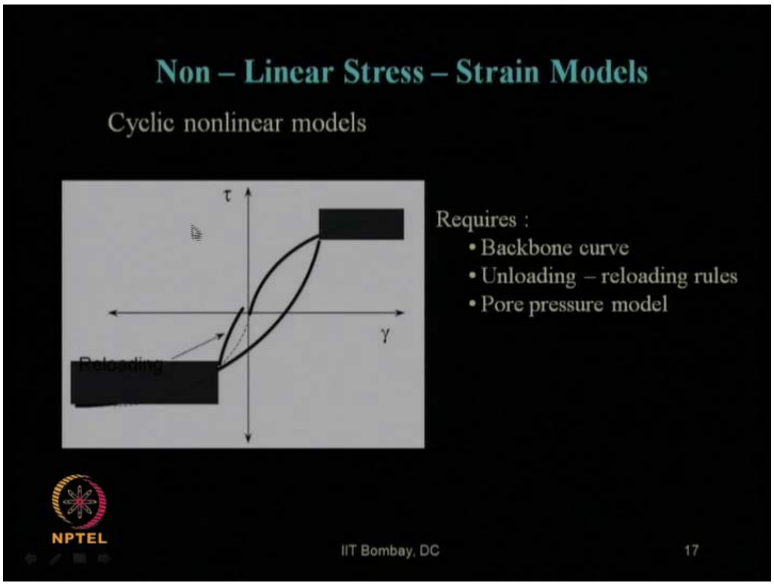

Now cyclic non-linear model one can get very easily from a lab test on a particular soil by knowing the backbone curve, unloading reloading rule, and the pore pressure model.

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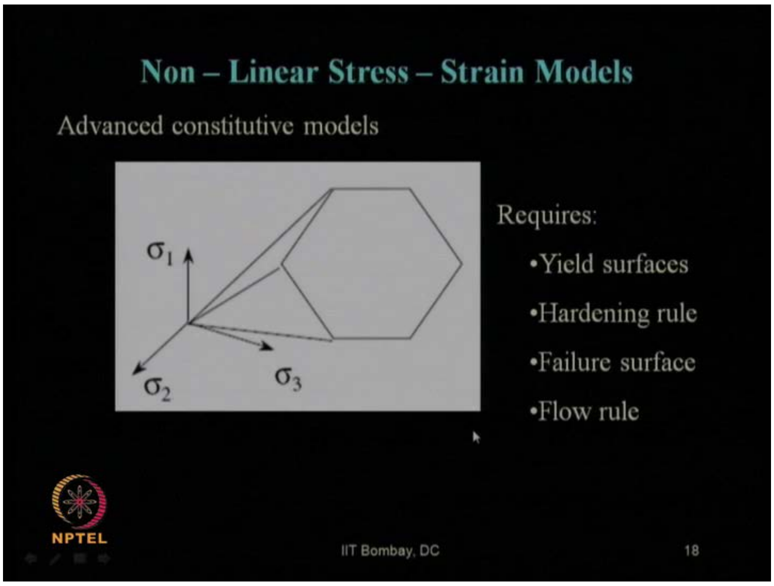

Whereas, for advanced constitutive model- yield surface, hardening rule, failure surface, and flow rule, all these information should be available.

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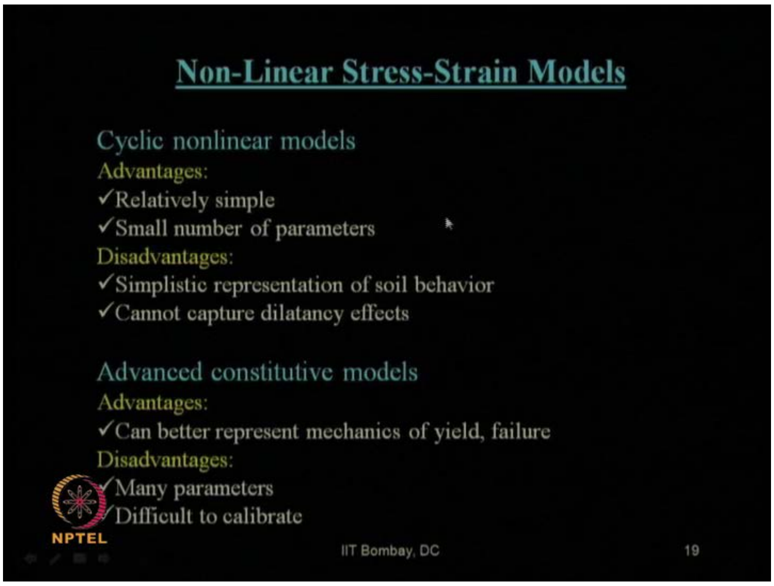

So, we have seen, also in our previous lecture, a comparative study between the use of cyclic non-linear model and the advance constitutive model. We have mentioned that obviously, the cyclic non-linear model is a simpler model to use. But there are difficulties like exact soil behavior or dilatancy effect etcetera may not be captured. Whereas advance constitutive model is the best that way, the, because it considers all the dilatancy effect, yield behavior, failure behavior, everything; but the disadvantage is it is a complex model, and it requires longer time, also it is difficult to calibrate with respect to a particular soil model. That is unless for your soil, you have your constitutive model known, then otherwise you cannot use this advance constitutive model for your analysis.

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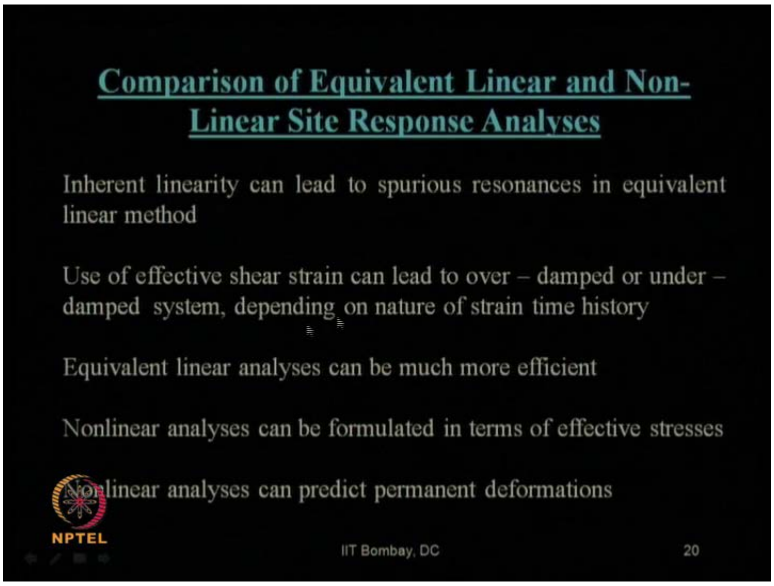

Now, in today's lecture, let us now start with the comparison of this equivalent linear and this non-linear site response analysis. So, inherent linearity can lead to spurious resonances in equivalent linear method. What inherent linearity we are considering?

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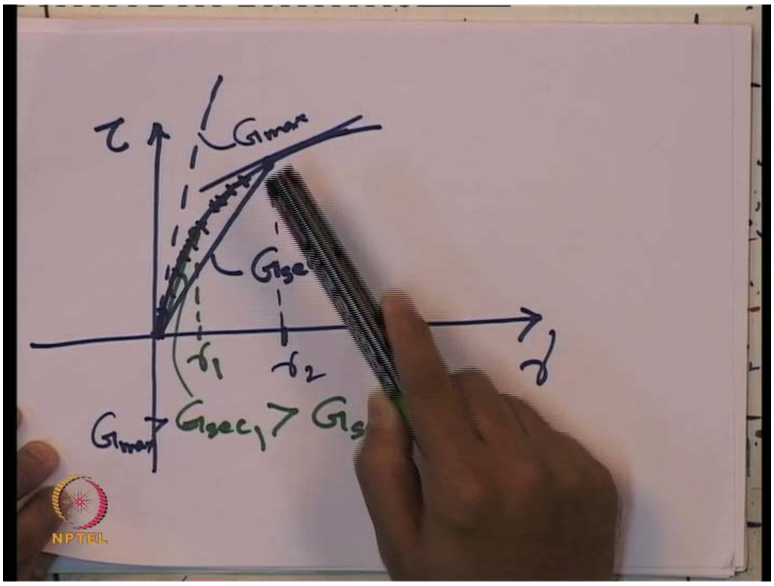

If you look at, this curve, once again which we have already discussed in previous lecture. This is the inherent non linearity which we are considering, Right. It is actually non-linear, but we are taking, assuming this linearity. So, that inherent linearity which is used in equivalent linear method may lead to some resonances, spurious resonances which is not actual. So, use of this effective shear strain can lead to over damped or under damped system depending on the nature of stain time history.

Equivalent linear analysis can be much more efficient because it is more faster, Right, than non-linear approach. Whereas, non-linear analysis can be performed in terms of effective stresses. And non-linear analysis can predict the permanent deformation because it is exactly capturing the final cyclic shear strain which the material is undergoing, through, for that time duration of input motion. So, what will be the suggestion for a practical use of this equivalent linear or non-linear approach? When we are dealing with very important problem, let us say design of any earthen dam, or design of some nuclear power plant foundation system; in those cases where the importancy of the structure is very high, in those cases non-linear ground response analysis is a must.

Whereas for medium important structures or medium important analysis like for buildings etcetera, if you want to do the ground response analysis, what you can use? Equivalent linear approach. Why? Because equivalent linear approach will save a lot of computational time which is necessary in case of non-linear approach, because you are doing incremental small time interval and repeating the method of analysis. So, that is why non-linear approach takes much longer time of computation than equivalent linear analysis; also non-linear approach is complex than equivalent linear approach. Whereas I am telling that linear approach where only G max is used should be avoided for any type of ground response analysis because we know earthquake response is a high strain phenomenon. So, in that case, if somebody is using linear ground response analysis that will be absolutely wrong. Because that does not capture the high strain phenomenon, it will be only a low strain phenomenon where G max is valid, or linearity of the model is valid.

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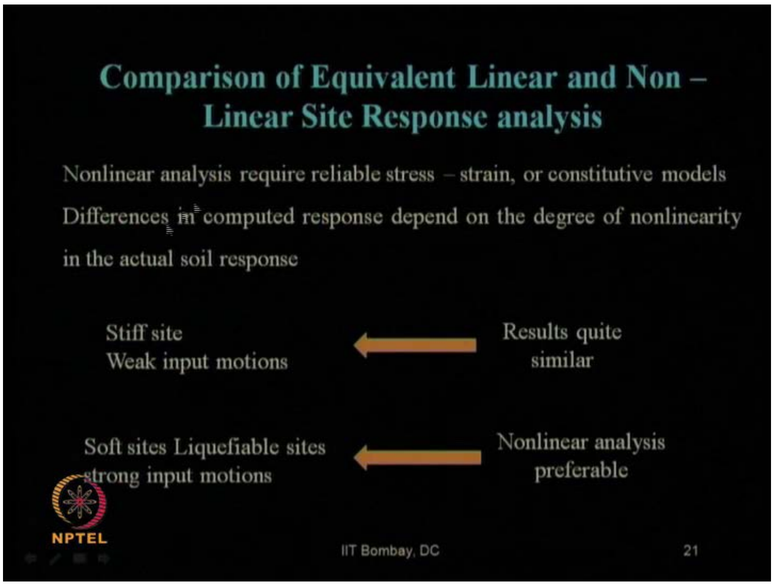

Now, let us look at this comparison further. That is non-linear analysis require reliable stress strain or constitutive model which we have already seen. So, this is one complexity in the analysis. Differences in computed response depend on the degree of non-linearity in the actual soil response what it is having.

So, suppose, if we are handling with stiff soil site, that is these are all relative terminology, of course, with weak input earthquake motion, in that case non-linear and equivalent linear will result almost the similar output. That is for a stiff soil site and subjected to weak input earthquake motion, you can either use equivalent linear or non-linear. In both the cases similar results you will get. So obviously, it is better to use equivalent linear which less time consuming, also which is simpler to analyze. Now, but for soft soil site where chances of liquefaction is there; and the soil site is subjected to strong input motion in that case non-linear analysis is always preferable. Because that will capture the actual displacement of the soil material which will help you to further, which will help you to further calculate the displacement related to this liquefaction, and which will lead to lateral spreading etcetera. So, those thing for those type of soil site and that type of input motion, strong input motion non-linear approach is preferred over equivalent linear approach.

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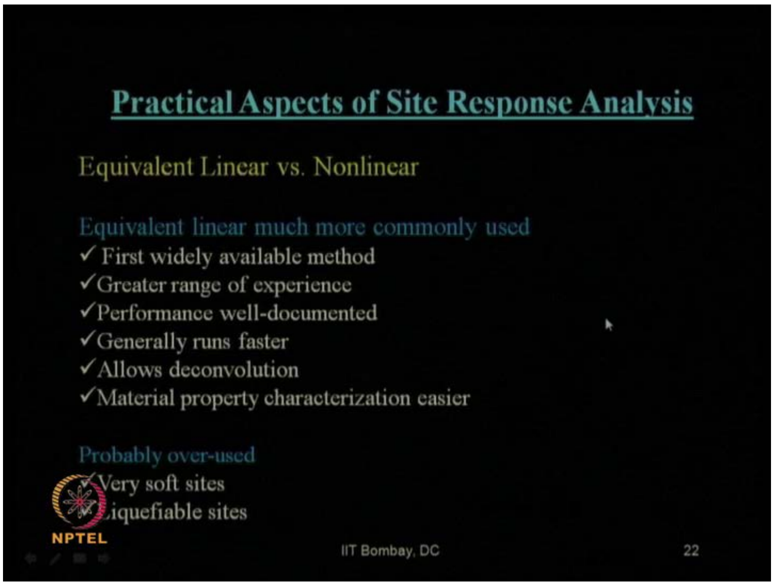

So, some practical aspects of this site response analysis what we can say, that equivalent linear much more commonly used. Why, those are commonly maximum used than non-linear? Because of its first widely available method, because it is pretty well known and widely available; greater range of experience, many people have done, so people have experience handling with that; performance is well documented, how they perform with the equivalent linear approach those are available in literature. It generally runs faster because less computational time than non-linear.

It allows de-convolution. What is de-convolution? We will see that. Convolution is nothing but when we are approaching from the ground to, from, we are approaching from the bedrock to ground surface. And deconvolution is the reverse one that is from the ground surface to the bedrock level. Material property characterization is easier in case of equivalent linear, why? Because only you are considering the G secant; at whatever strain level you are interested to that value of g sec, you can consider.

Wherever, we are mentioning this advantages of equivalent linear over the non-linear, what are the other side of it is, probably these are over used for the case of very soft site and liquefiable site, which should not be. We have already seen for this type of site non-linear approach is better; non-linear site response analysis or non-linear ground response analysis is a better option.

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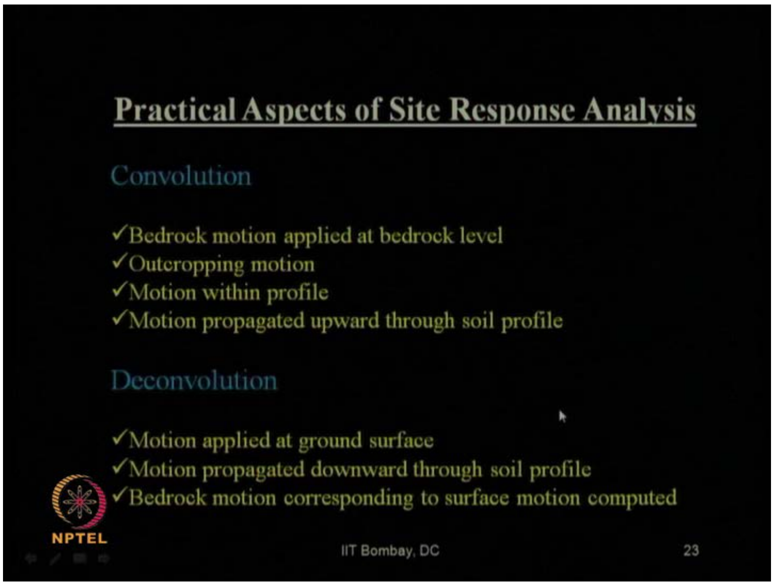

Now practical aspects of the site response analysis are- convolution, what is convolution? As I said just now, bed rock motion is applied at bed rock level; and out cropping motion is needs to be obtained; motion within the profile has to be obtained; and motion propagated upward through the soil profile. Whereas, what is called deconvolution? In this case motion is applied at the ground surface; and then that motion propagated downward though the soil profile; and bedrock motion corresponding to surface motion is computed in this case of deconvolution. So, for the case of convolution we need the transfer function to be used; whereas for the case of deconvolution we need to use, the inverse of the transfer function to be used.

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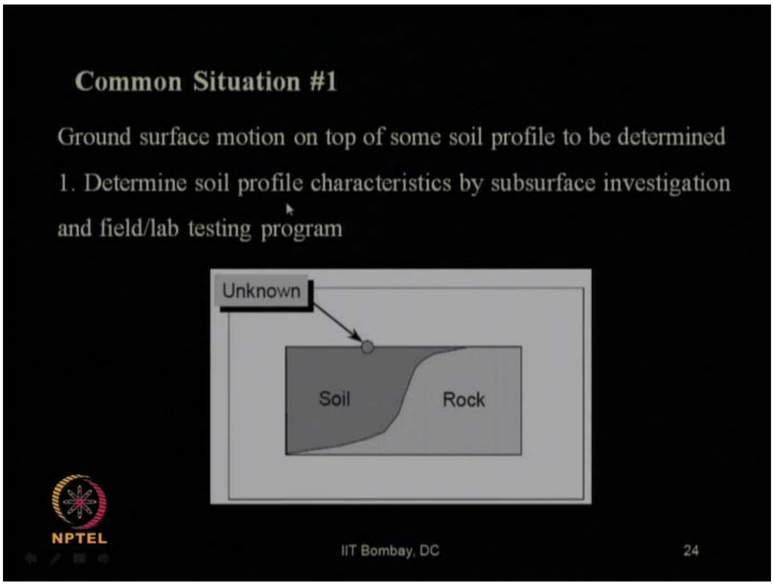

Now, as we have already discussed this common situations- two major common situation and how to find out? Now after having the complete knowledge of this site response analysis, let us look at it once again. That is common situation one is, say ground surface motion on the top of some soil layer needs to be determined. So, at this point we want to obtain. That is the problem statement, let us say. So, determined the soil profile characteristics, we need to know what is the material property of this soil layer, by sub surface investigation or the field or laboratory testing program.

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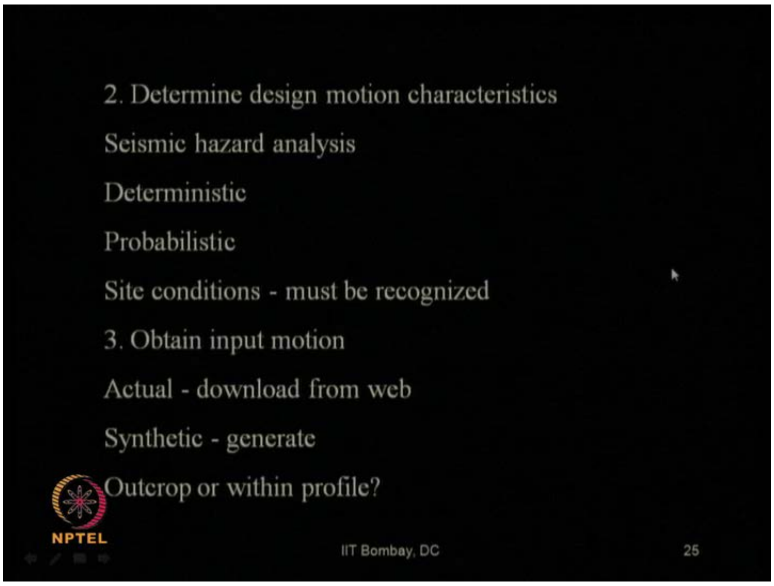

Then, determine the design motion characteristics. How we can get that design motion characteristics? Either using deterministic seismic hazard analysis, or probabilistic seismic hazard analysis. And site conditions must be recognized; what type of site. Then obtain the input motion. To obtain input motion means for which earthquake motion you want to analyze, you want to carry out this ground response analysis. Like actual input motion you can download from the website that is for the authentic data. As I have already mentioned USGS data base, or IMD data base, or PR data base, they are various other data base where actual ground response or actual earthquake responses at different depths are available. So, you can use that motion as an actual motion.

Or one can use the synthetic input motion. Synthetic ground motion is nothing but it needs to be generated. You generate the, it is synthetically designed, it is not the actually happened. And this actual is where some earthquake has really happened, in some part of world. And then using this input motion whether it is actual or synthetic, outcrop or within the profile that needs to be determined; that needs to known whether it is an outcrop motion or it is at the bed rock.

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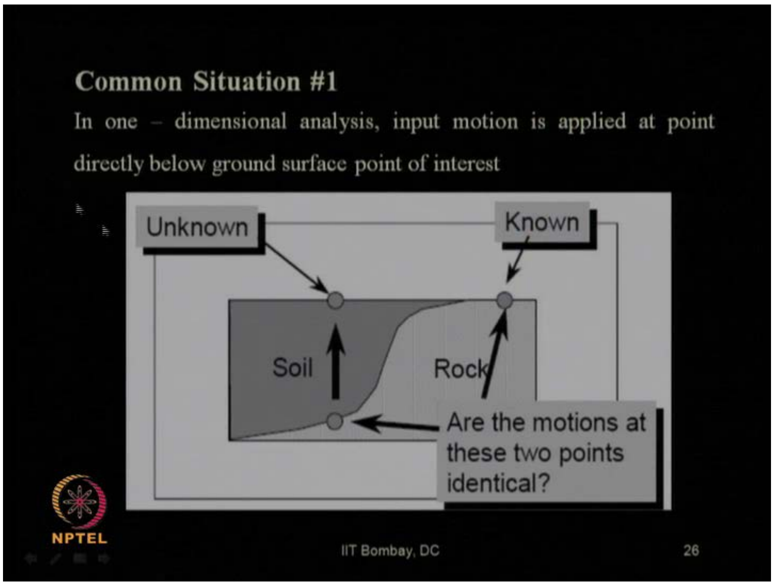

So, then once we get all these information what we do? Say this out crop motion, you are getting from some website, if it is an actual earthquake, from some reliable source. Or you are applying some synthetic motion, you can do that also. Because when you are doing site response analysis at a particular site for your design, you do not know in future what earthquake will come. So, probably either you can use the historical earthquake in the nearby region, or you can use some synthetic motion which will justify the seismic zonation of that area. So, that way the synthetic motion has to be chosen at outcrop. So, this is known.

Now, this motion and this motion should be different; or this motion at two point identical, no. They cannot be identical, because it is a free surface and this is a underlined by a soil layer. So, in one dimensional analysis input motion is applied at a point directly below the ground surface of the point of interest. So, you can apply, if the bed rock motion is known, you can apply it here also; but that should be different, you remember that. Then you apply all the soil properties etcetera, do the either equivalent linear or non-linear analysis to get the known value over here.

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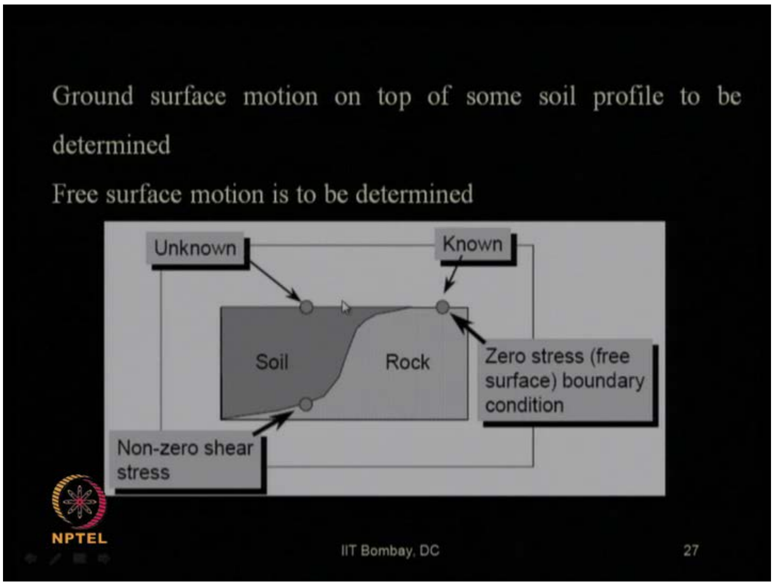

So, that is what it says. Here it is zero stress free boundary condition; whereas, it is non zero stress. So, that is why they should not be equal, as I said already.

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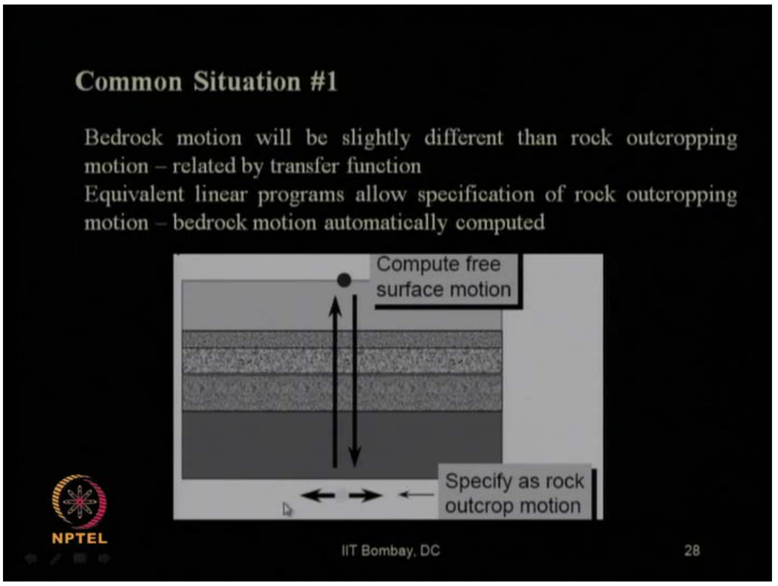

So, bed rock motion will be slightly different than the rock out cropping motion because of, it is related through that transfer function using that boundary condition. And equivalent linear programs allow specification of rock outcropping motion- bedrock motion automatically gets computed. So, this bed rock motion, once you put it, then you compute the free surface motion by using this equivalent linear or non-linear programs. So, for that as I said, either shake or deep soil can be used.

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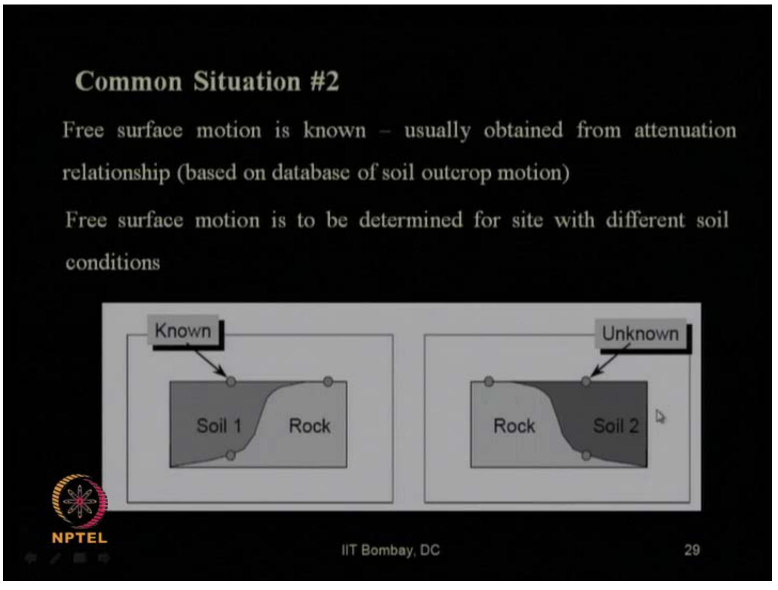

For common situation number 2, where free surface motion at one soil site is known. Suppose, just now we have computed at this soil site one, the ground response analysis. Then in another soil, site number 2, we need to find it out; so soil site number 2 we need to find it out.

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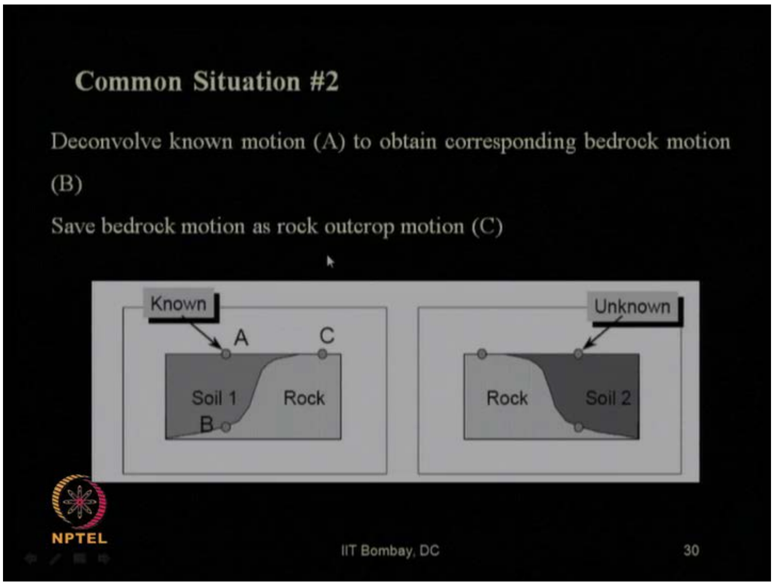

So, how we can do that? We need to transfer from point A to corresponding bed rock motion point B. And that bed rock motion you have to take at the out crop motion C; that C you have to take, if the rock profile is same at outcropping at the two different places at ground surface also. Because rock material is same, and both are out cropping, so you can used that data.

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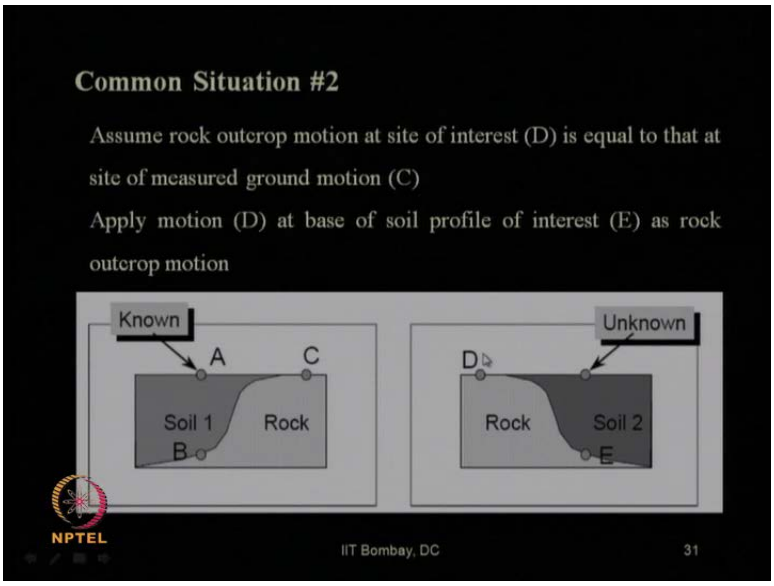

Once you use that data at C, then from C, D you equate that is assume that rock outcrop at site of interest D is equal to that of measured at C. Then once you get D, you can get it at E, again using transfer function. Once you know at E then use the ground response analysis and get this unknown at soil site to F.

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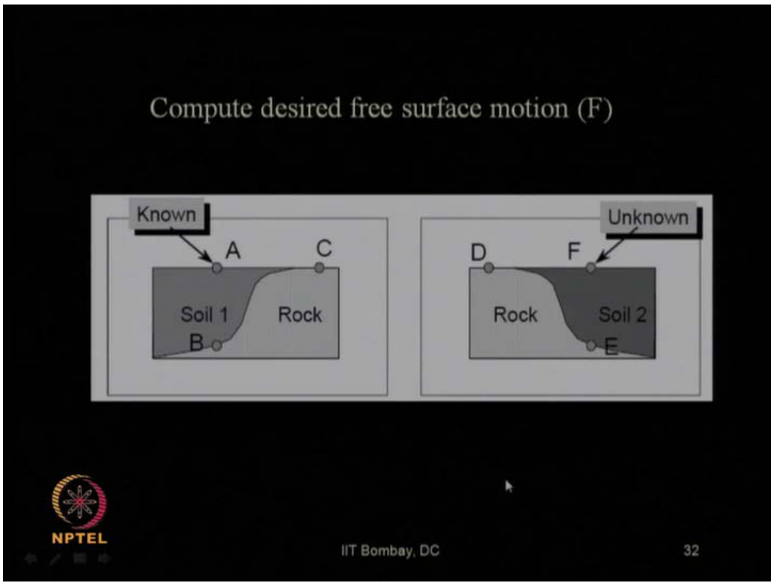

So, compute the desired free surface motion at F. So, these are the steps, how we carry out the ground response analysis or site response analysis from one site to another site. Clear.

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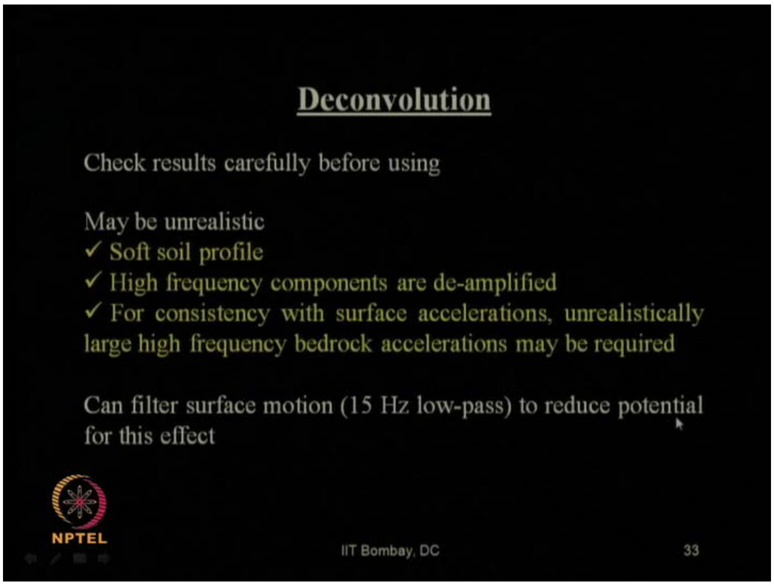

So, the deconvolution, how we can cross check? It is helpful to check the results carefully before using. Because it may be unrealistic sometimes like soft soil profile; high frequency components are getting de-amplified; whereas for consistency with surface accelerations, unrealistically large high frequency bedrock accelerations may be required. So, in that case to check the deconvolution, you can use the synthetic time history also. That is what it means. Instead if you do not have a real earthquake motion to check the deconvolution for your soil layer, you can use a synthetic time motion. So, it can filter the surface motion below 15 hertz frequency to reduce the potential for this effect.

Now, let us start going though the example problems on this ground response analysis from two case studies. These two case studies- the first example is the case study on the seismic ground response analysis for Mumbai city in India. This is a part of the PhD thesis work by Dr V S Phanikanth who completed PhD in the year 2011 at IIT Bombay, under my supervision. He is the scientist at BRC and doctor G R Reddy was external supervisor from BRC. So, from his PhD thesis work, on this ground response analysis, this journal paper has been published. So, one can go through this paper for the detailed discussion about, how this ground response analyzed using the equivalent linear approach has been carried out for some typical sites in the Mumbai city of India. This is in journal geotechnical and geological engineering of Springer publication, volume 29, issue 6, page number is given over here, year 2011.

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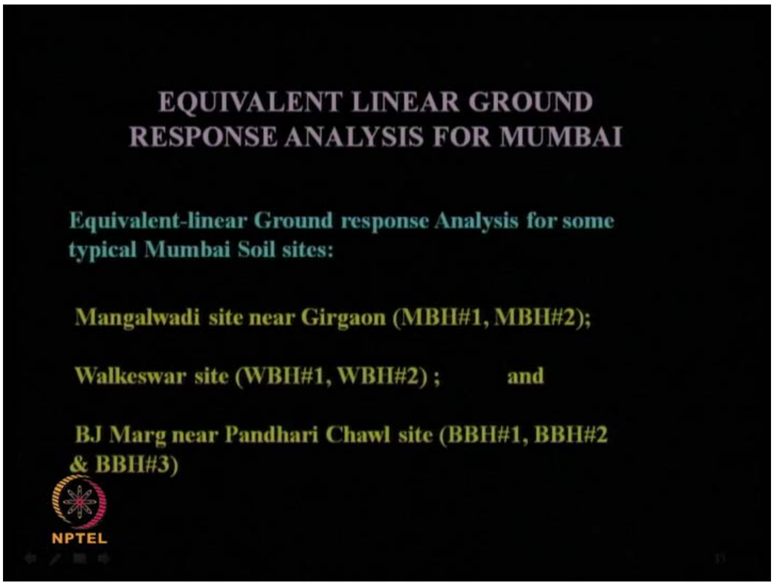

Now, let us go through, step by step. For this equivalent linear ground response analysis for some typical sites, three typical sites doctor Phanikanth had chosen. One is Mangalwadi site near Girgaon. There are two borehole data, typical borehole data. Mangalwadi site borehole number 1, Mangalwadi site bore hole number 2 were used. Walkeswar site- Walkeshwar borehole number 1 Walkeshwar bore hole number 2. And B J Marg near Pandhari Chawal site with B J Marg bore hole number 1, borehole number 2 and borehole number 3. Typically these soil data, from collected source of reliable sources of soil data, this information has been first taken care of.

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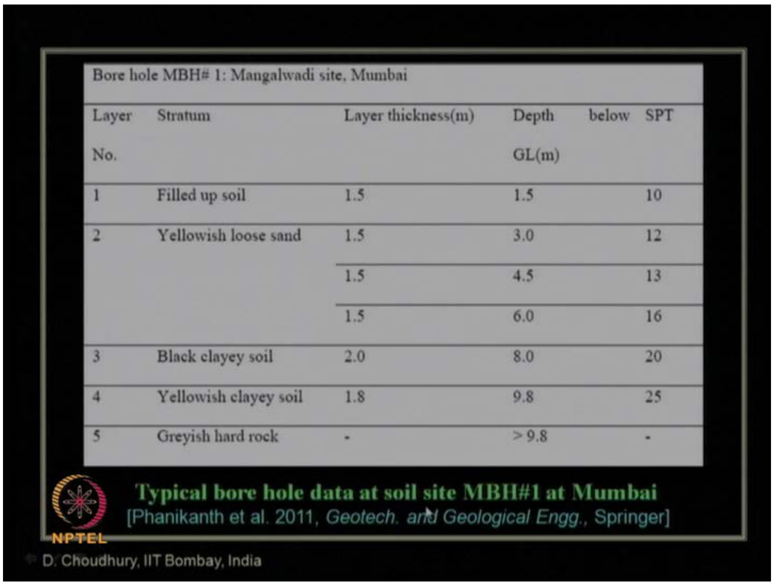

Bore hole MBH# 1: Mangalwadi site, Mumbai

Layer No.	Stratum	Layer thickness(m)	Depth below GL(m)	SPT
1	Filled up soil	1.5	1.5	10
2	Yellowish loose sand	1.5	3.0	12
		1.5	4.5	13
		1.5	6.0	16
3	Black clayey soil	2.0	8.0	20
4	Yellowish clayey soil	1.8	9.8	25
5	Greyish hard rock	-	> 9.8	-

So, this is the typical bore hole data for the Mangalwadi site number, bore hole number 1. You can see the layer thickness over here given; depth below the ground level that is beyond 9.8 meter below the ground level, the hard rock is available at that site, you can see over here.

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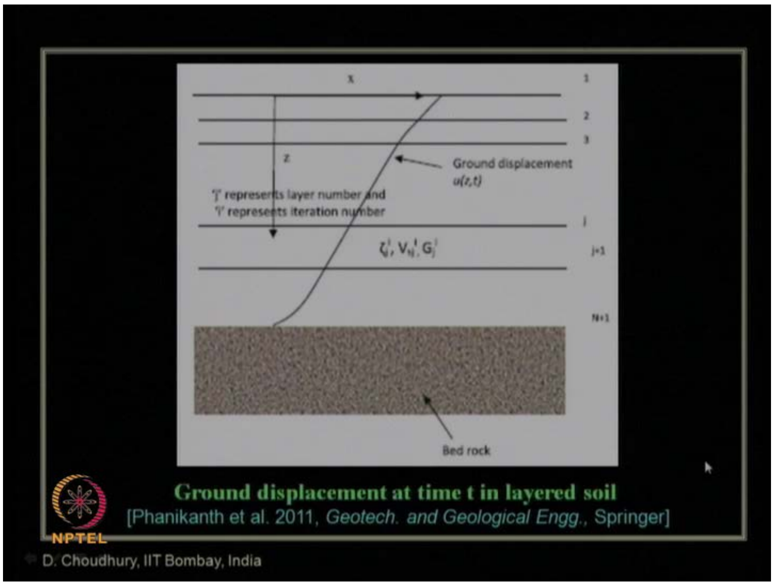

So, corresponding SPT values are given over here, as recorded at the site. Based on this input motion, what next step we need to do? This is the bedrock. We have several layers of the soil as we have seen. So, that multilayered site response analysis we need to carry out. What we have just, now we have seen? We have seen how to carry out for individual layer.

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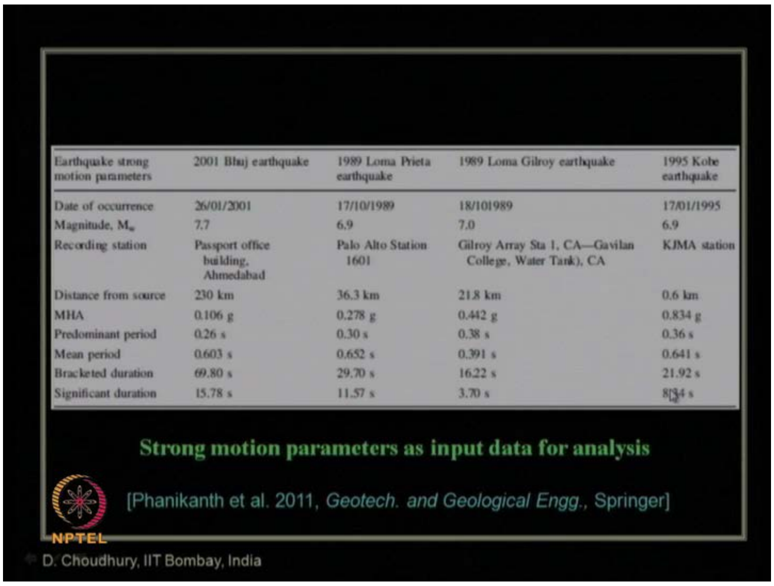

Earthquake strong motion parameters	2001 Bhuj earthquake	1989 Loma Prieta earthquake	1989 Loma Gilroy earthquake	1995 Kobe earthquake
Date of occurrence	26/01/2001	17/10/1989	18/101989	17/01/1995
Magnitude, M_w	7.7	6.9	7.0	6.9
Recording station	Passport office building, Ahmedabad	Palo Alto Station 1601	Gilroy Array Sta 1, CA—Gavilan College, Water Tank), CA	KJMA station
Distance from source	230 km	36.3 km	21.8 km	0.6 km
MHA	0.106 g	0.278 g	0.442 g	0.834 g
Predominant period	0.26 s	0.30 s	0.38 s	0.36 s
Mean period	0.603 s	0.652 s	0.391 s	0.641 s
Bracketed duration	69.80 s	29.70 s	16.22 s	21.92 s
Significant duration	15.78 s	11.57 s	3.70 s	8.34 s

So, to do that, next is we should know, what is our input motion we are using? Now for the input earthquake motion doctor Phanikanth had chosen this 4 input motion: 2001 Bhuj earthquake motion, 1989 Loma Prieta earthquake motion, 1989 Loma Gillory earthquake motion, and 1995 Kobe earthquake motion. These are the various parameters which are obtained like date of occurrence of this earthquake, magnitude M w, recording station, distance from the source that is MHA, predominant period, mean period, bracketed duration, and significant duration. So, all these data have been taken as input value of this earthquake motion, for further ground response analysis at those borehole locations or soil site.

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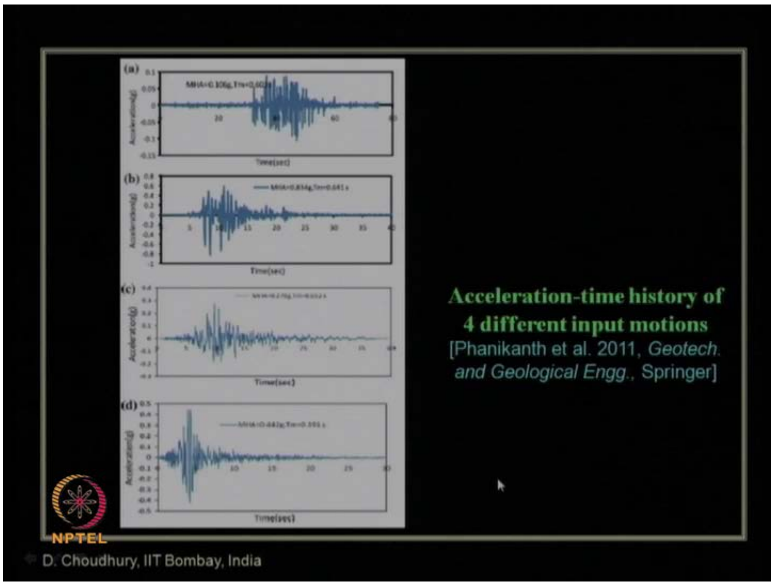

So, once again the pictorial representation of this acceleration versus time history of the input motion. So, this first one is for the Bhuj motion of 2001 earthquake; then Loma Gillory; this is Loma Prieta; this is Loma Gillory; this is for Kobe.

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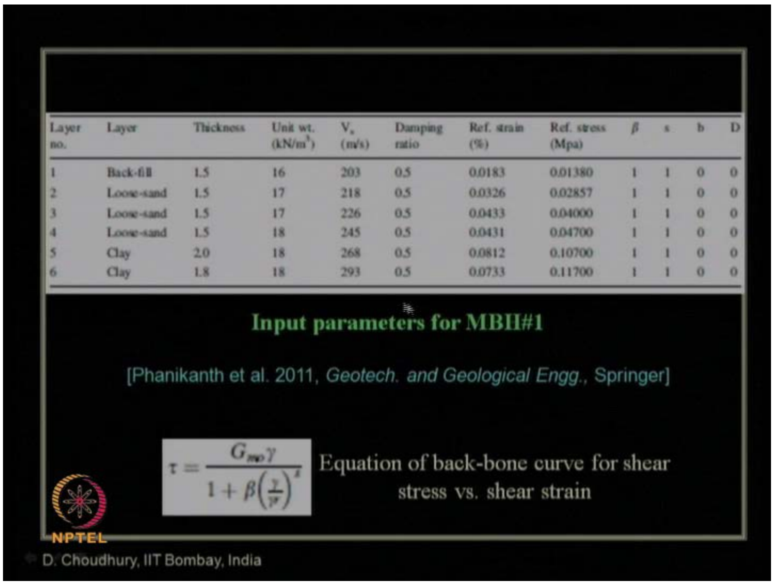

Layer no.	Layer	Thickness	Unit wt. (kN/m^3)	V_s (m/s)	Damping ratio	Ref. strain (%)	Ref. stress (Mpa)	β	s	b	D
1	Back-fill	1.5	16	203	0.5	0.0183	0.01380	1	1	0	0
2	Loose-sand	1.5	17	218	0.5	0.0326	0.02857	1	1	0	0
3	Loose-sand	1.5	17	226	0.5	0.0433	0.04000	1	1	0	0
4	Loose-sand	1.5	18	245	0.5	0.0431	0.04700	1	1	0	0
5	Clay	2.0	18	268	0.5	0.0812	0.10700	1	1	0	0
6	Clay	1.8	18	293	0.5	0.0733	0.11700	1	1	0	0

Now, input parameters for this borehole number one have been further extended to the shear wave velocity. Why we need shear wave velocity? We need shear wave velocity because unless we know the individual layers of soil shear wave velocity, we do not know the initial tangent modulus or the maximum shear modulus G max, Right. Because

G max we can calculate from rho V s square. Now this V s, either it can be estimated from the site using MASW or SASW or other field test; or in absence of that one can use the available empirical relationship between SPT N value and V s value, and obtained this values. Then starting point of damping ratio is considered as 0.5 percent. All these are 0.5 percent for all the layers. You can see 6 numbers of layers for this MBH 1 as we have already seen, has been considered.

So, this is the individual initial values of damping ratio, for an initial cyclic shear strain. So, this is reference strain in percent, initial values, reference stresses, and other coefficients beta S and so on. So, you can get the details in this paper, as I have mentioned where you should know the equation for the back bone curve that is tau versus gamma which is expressed in this from. So, GMO is nothing but initial tangent modulus that is maximum shear modulus. Beta is the coefficient; s is another coefficient; gamma is cyclic shear strain; and gamma r is reference shear stain that is you select a reference strain at beginning and then do this iteration, until this selected reference strain and the converged finally obtain reference strain matches.

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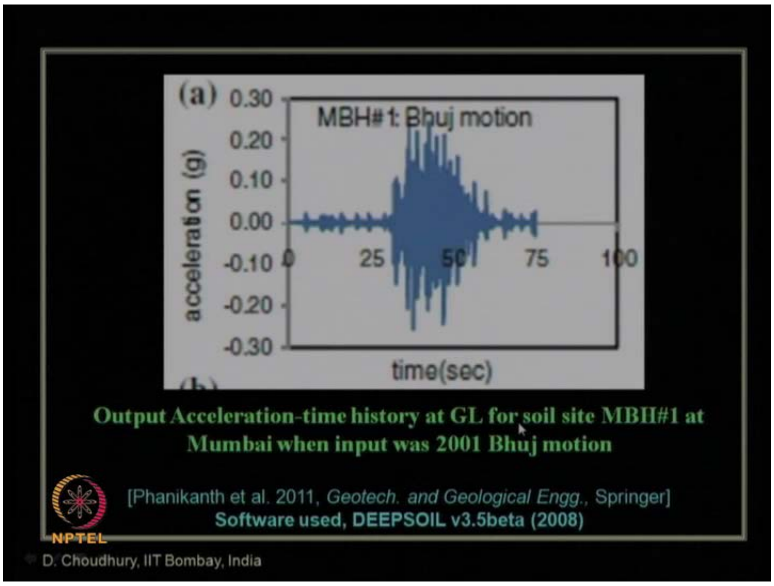

After doing that, what output you will get? So, this analysis was done using the software deep soil. Deep soil version 3.5 beta of 2008 version was used. So, one can always use the latest version to do the analysis, at that time 2008 was the latest version. Final output one can get this acceleration versus time history at any layer. So, finally, for any structural design we will be interested to know the ground, at ground level what is the output acceleration time history. So, this result shows the, at ground level this is the acceleration time history of that 2001 Bhuj input motion which was given input as the bedrock level, at that borehole number 1 at Mangalwadi site. So, using that soil property, using this we got this one. So, this was your input motion, you can see the maximum value was 0.106 g; whereas the output came more than 0.2 g, can you see that. So, there is an increase in the maximum value of acceleration from the bedrock level to ground level in that borehole number one, considering the local soil parameters or properties.

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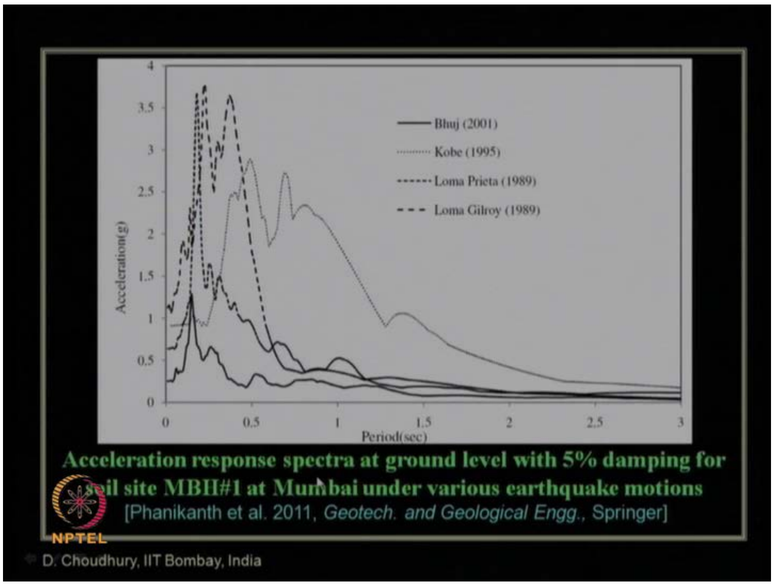

So, the response spectra, acceleration response spectra, if in that fashion, we want to express because why we are interested to know about response spectra? All the design codes they talk about the response spectra versus period. That we have already seen. Even in our IS code also. So, this is acceleration response spectra with respect to this period, for different earthquake input motion- for Bhuj motion this dark color solid line, for Kobe this dotted one, for Loma Prieta this one, for Loma Gillory this one.

So, depending on different input motion your output is also different, as you can see, for the same borehole location MBH 1 at ground level. So, this response spectra is at ground level, considering 5 percent damping at the soil site. Because, you remember, whereas, you are getting this acceleration time history result at ground level, here you are getting the exact damping ratio. But for spectral response spectra, you have to assume some damping because it is for the equivalent single degree of freedom, mass spring dashpot model. You remember, for response spectra how we determined. So, that is what it is mentioned 5 percent damping, it is nothing related to actual damping of the soil.

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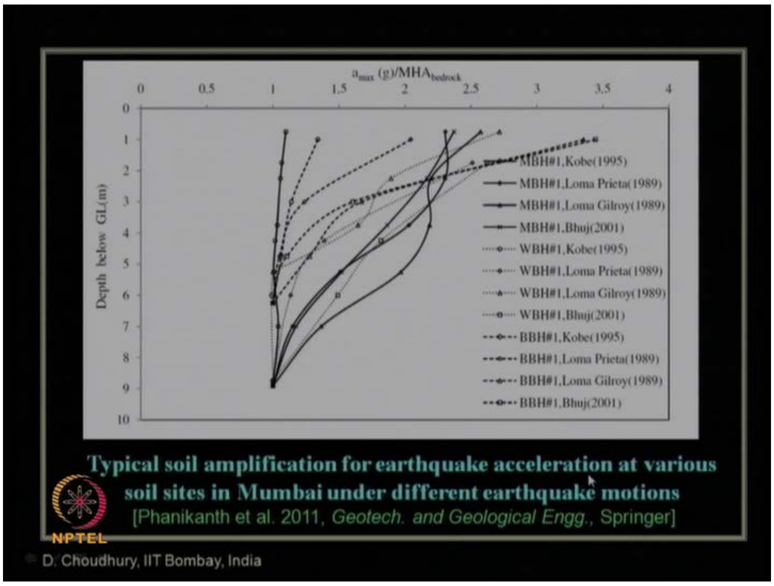

You can see the result that a max value, the maximum value at ground level for MBH 1, what we got? It was about more than 0.2 g, higher than 0.2 g. So, that if you compare with respect to your input value of 0.106 g, what is the increase or amplification of peak value? Amplification is nothing but we can see, a max in terms of g at ground level divided by maximum horizontal acceleration at bedrock level. That will give you nothing but the values of amplification with respect to depth.

So for different earthquake input motion and for all different boreholes, all the borehole number one of all these three sites are reported over here. You can see in Mumbai city, typically considering all these soil site we got a range of soil amplifications ranging between 1.2 to about 3.5. That is the range of soil amplification, when we are considering a bedrock to the ground surface level. Clear. So, that shows a typical soil amplification of earthquake acceleration at various soil sites in Mumbai.

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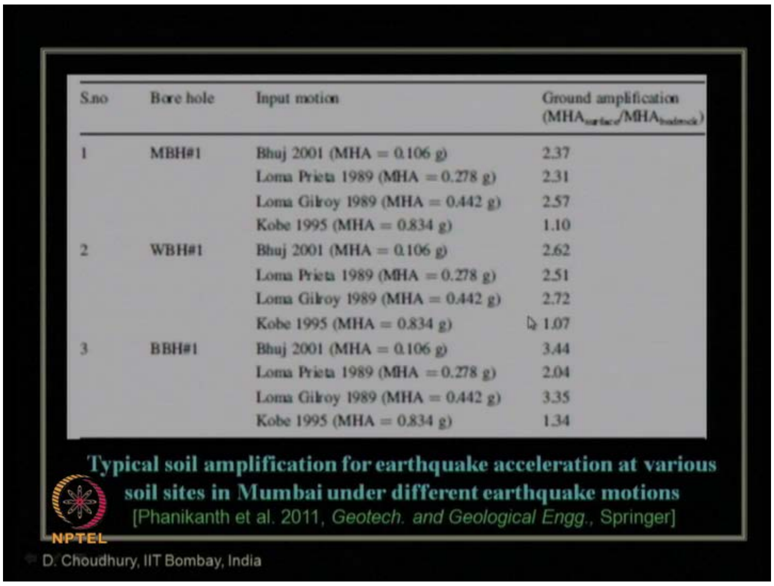

S.no	Bore hole	Input motion	Ground amplification ($MHA_{surface}/MHA_{bedrock}$)
1	MBH#1	Bhuj 2001 (MHA = 0.106 g)	2.37
		Loma Prieta 1989 (MHA = 0.278 g)	2.31
		Loma Gilroy 1989 (MHA = 0.442 g)	2.57
		Kobe 1995 (MHA = 0.834 g)	1.10
2	WBH#1	Bhuj 2001 (MHA = 0.106 g)	2.62
		Loma Prieta 1989 (MHA = 0.278 g)	2.51
		Loma Gilroy 1989 (MHA = 0.442 g)	2.72
		Kobe 1995 (MHA = 0.834 g)	1.07
3	BBH#1	Bhuj 2001 (MHA = 0.106 g)	3.44
		Loma Prieta 1989 (MHA = 0.278 g)	2.04
		Loma Gilroy 1989 (MHA = 0.442 g)	3.35
		Kobe 1995 (MHA = 0.834 g)	1.34

This is in the tabular form. The exact values are computed, the same thing MHA at ground surface by MHA by bedrock at different input motion, and at different borehole site.

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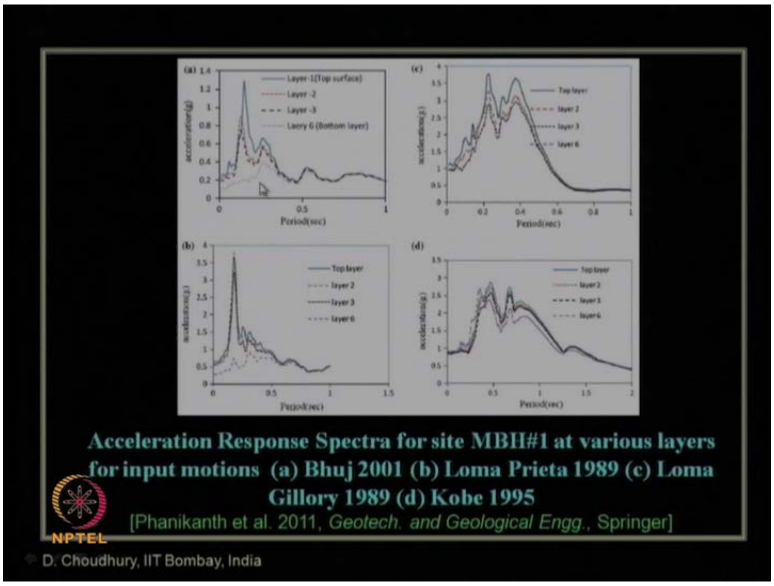

Now, we can see acceleration response spectra at different layers. Can you see layer 1 is nothing but your ground level which we have already discussed. But if somebody is interested to know the other layer, say layer 2, layer 3, or layer 6, why it is important? It is again for MBH 1, but for different input motion. This figure a is for Bhuj motion, figure b is for Loma Prieta motion, figure c is for Loma Gillory, and figure d is for Kobe earthquake motion. Why this response spectra is important? Let us look at very carefully. Suppose if we take this Bhuj motion for MBH 1 at the ground surface, see where the peak is occurring at this time period.

So, corresponding to this time period, whatever the buildings or structure will be there, that will be more vulnerable or will be subjected to maximum damage. Am I clear. Now if you are considering your structure say, you are constructing a foundation or underground structure, let us say close to your layer number 6. For layer number 6, you see here, where the peak came? Peak came here. Can you see over here, this is the peak for layer 6. So, when you are designing corresponding value of time period that will be more critical. That is a structure having time period of this value will experience maximum damage when we are constructing any structure at layer 6. So, that way individual layer wise data also very much helpful to us. So, this is about layer variation.

Now, let us compare Bhuj motion with respect to Kobe motion. What we can see? For Bhuj motion the major damage will occur, or the peak value of spectral acceleration we got at the period of say about 0.2 seconds. Whereas, for Kobe motion the peak came same at top layer ground surface is about 0.5 second. So that means, it is not that all the, means the same structure will be subjected to same amount of damage irrespective of ground motion.

But it will be subjected to damage depending on what is your input motion. That is Bhuj motion will make maximum damage to a structure which is having a period of 0.2 second; whereas Kobe motion if it comes to Mumbai, at that site, that will damage a structure which is having a period of 0.5 seconds. Is it clear. How we are using these things for further design? Hence it is very important, instead of generalizing one should do a site specific ground response analysis like this, depending on various input motion at that particular site, or using a logical synthetic motion at that site, to get the data for further design

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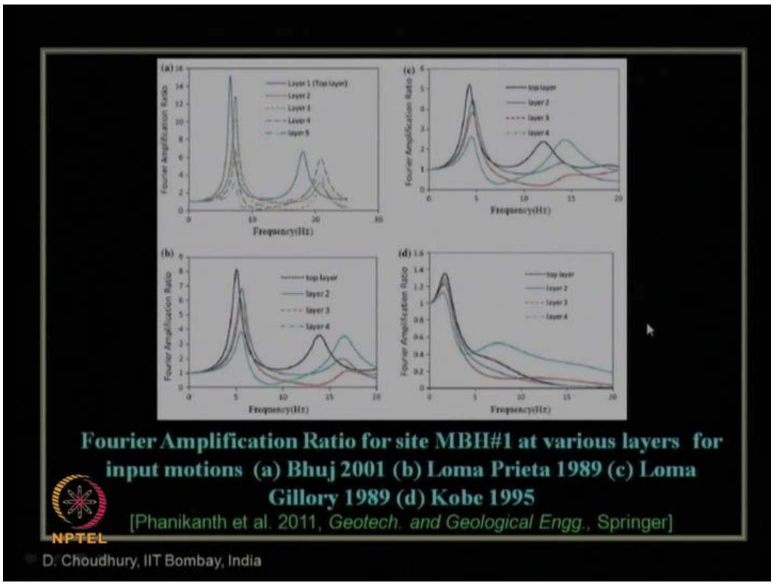

This is the Fourier amplification ratio. That is, we know the response of amplification or acceleration we can express in terms of spectral response or Fourier response. So, this is in terms of Fourier response.

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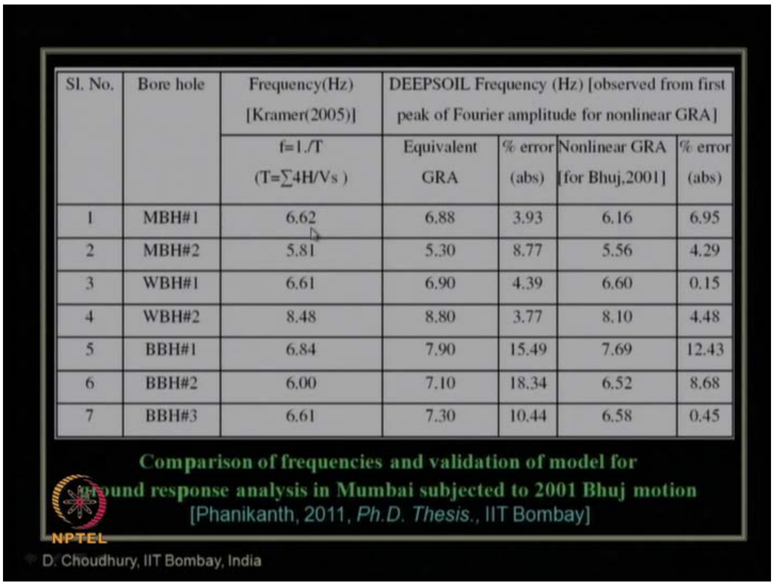

Sl. No.	Bore hole	Frequency(Hz) [Kramer(2005)]	DEEPSOIL Frequency (Hz) [observed from first peak of Fourier amplitude for nonlinear GRA]			
		f=1./T (T=∑4H/Vs)	Equivalent GRA	% error (abs)	Nonlinear GRA [for Bhuj,2001]	% error (abs)
1	MBH#1	6.62	6.88	3.93	6.16	6.95
2	MBH#2	5.81	5.30	8.77	5.56	4.29
3	WBH#1	6.61	6.90	4.39	6.60	0.15
4	WBH#2	8.48	8.80	3.77	8.10	4.48
5	BBH#1	6.84	7.90	15.49	7.69	12.43
6	BBH#2	6.00	7.10	18.34	6.52	8.68
7	BBH#3	6.61	7.30	10.44	6.58	0.45

Now, let us come to the values of this frequency, what we can compute, using the simple formula of this sum of 4 H by V s that we have already seen. So, using that simple formula, that fundamental frequency what we can get? That comes out to be this, knowing the values of V s and individual layer of value of H for all your site. Now deep soil software, also will give you the corresponding values of frequency. Now equivalent ground response analysis, values of frequency output we got like this; and non-linear ground response analysis we got like this; in all the cases Bhuj motion is used. So, if you want to see what is the percent error, percent difference, I should not mention it as error. So, it is within, the, about 20 percent in case of equivalent ground response analysis. Whereas if you use the non-linear ground response analysis, it is still lower; it is within 13 percent. Right. So, non-linear result will be, obviously closer to the actual values as it is expected.

Now, let us come to the next example of ground response analysis. That is example number 2 which is the case study on the seismic ground response analysis for 4 port sites, ports in Gujarat state of India. This is the part of the PhD thesis work of Doctor Jay Kumarsukla who completed the PhD in 2012 at IIT Bombay under my supervision. And this is the journal paper which describes about the details of this seismic ground response analysis of 4 port sites in Gujarat. That is seismic hazard and site specific ground motion for typical ports of Gujarat. It is published in the journal natural hazard's Springer publication, volume 60, issue 2, page number is given over here, in 2012.

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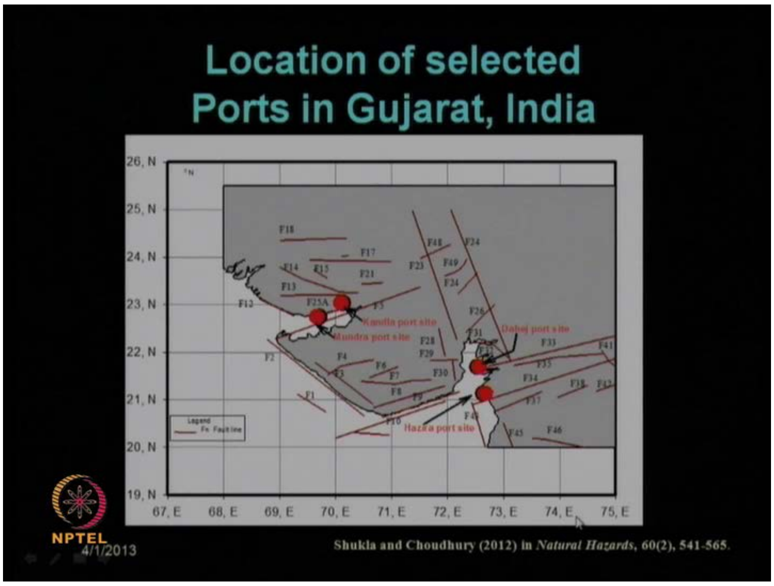

Let us look at the major 4 ports which, for which the study was carried out. These are the four ports Kandla port, Mundra port, Dahej port, and Hazira port. These are the 4 port sites; and these are the fault map, already we have discussed in the seismic hazard analysis problem.

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What are the steps followed? Like, fault map and seismicity parameters are known; logic tree frame work is already used; then we have developed UHS at the bed rock level for the synthetic ground motion; then geotechnical characterization is now necessary for each of the port site to carry out the site response analysis which will finally give us the result for this site response or ground response analysis.

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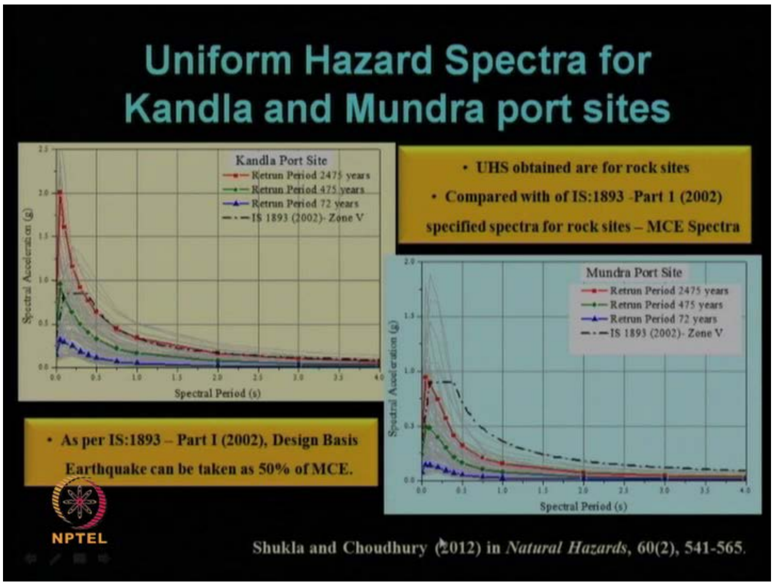

So, to do that, this is the UHS or Uniform Hazard Spectra which we have already seen for different return period of 2475, 72, and 475. And we have compared that with respect to IS code recommendation for zone 5, when we are talking about Kandla port which in zone 5 of seismic zone as per IS code. So, as we have mentioned IS code value is matching close to our return period of 475 years. Similarly, you can use that for Mundra port site; this is the spectral acceleration versus spectral period. And as we have already mentioned MCE has been used for rock site, and DBE design basis earthquake is 50 percent of that MCE.

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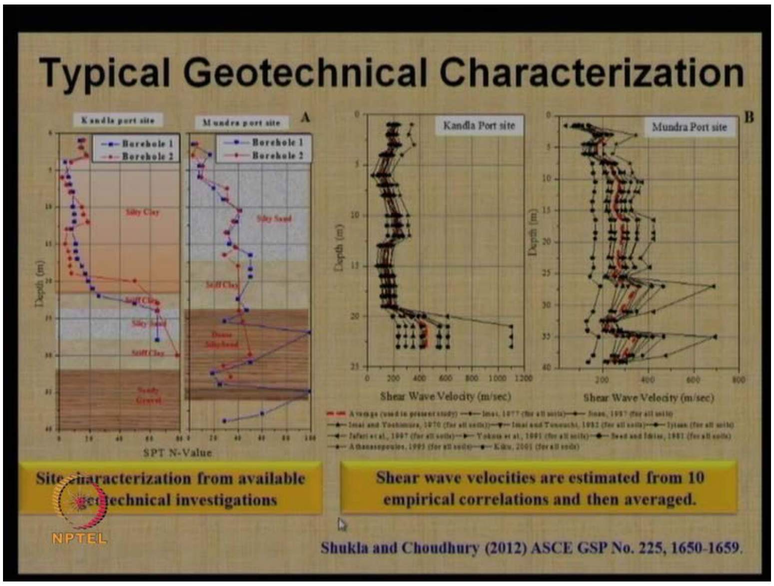

Now, we should know that geotechnical properties or geotechnical characterization at that site. So, various bore hole data were collected. Only the typical borehole 1 and 2 are shown here for Kandla port; similarly for Mundra port also bore hole one and two have been shown over here. You can see, majorly it is a silty clay followed by stiff clay then silty sand then stiff clay then sandy gravel at Kandla port site. Whereas, so up to 30 meter depth it is mostly a soft to stiff soil, and mostly clay in between a very thin sand layer, whereas for Mundra port site, it is mostly silty sand followed by stiff clay; and then dense silty sand. All the SPTN value recorded at the site also are available. Using these values, one can get the shear wave velocity with respect to depth at this Kandla port site or Mundra port site. As I have mention one can measure it using MASW or SASW or other field test; or in absence of that equipment one can use the available correlationship between SPTN values and shear wave velocity, to get this shear wave velocity values at those site. Fine.

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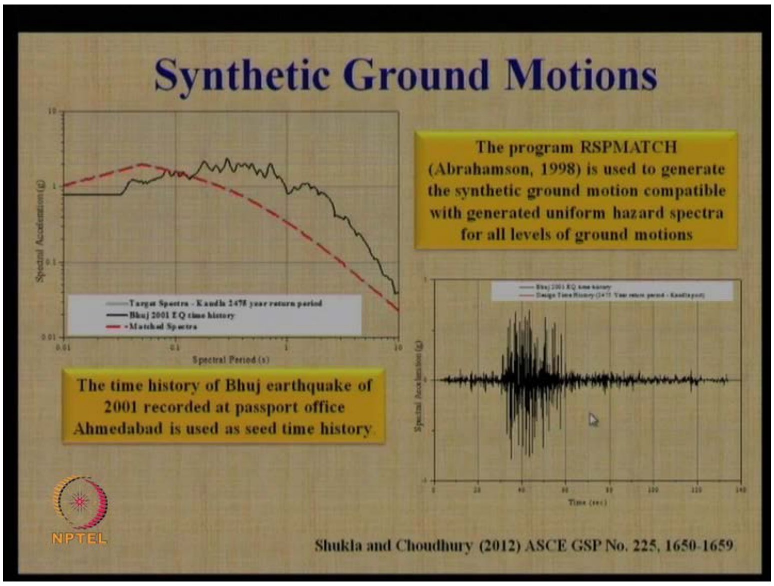

Now, once it is known, next step is the synthetic ground motion, which ground motion we are going to use for our ground response analysis. So, this is the synthetic ground motion; one can use the Bhuj earthquake motion; and the Mazz spectra is shown over here which is recorded at the passport office of Ahmadabad 2001 Bhuj earthquake data. And that program, this is the software program RSP match developed by Abrahamson in 1998 which is available, is used to generate that synthetic ground motion which will be compatible with that generated UHS at all levels of ground motion. That is you have to select such a input motion for your analysis which is compatible to that site. That is either you use form the earlier historical earthquake data at that site, or use that synthetic ground motion or that synthetic motion which will be compatible with respect to that site based on this UHS. Clear. It is not that you can intake any arbitrary value of this input ground motion. Clear.

So, input properties are very important, first of all geotechnical property, as well as which ground motion you are going to use further. So, this is the mapped one; the previous one we have seen over here, the Bhuj time history; and next one shows us, the design time history based on 2475 year of return period Tatkandla port, can you see, a similar match.

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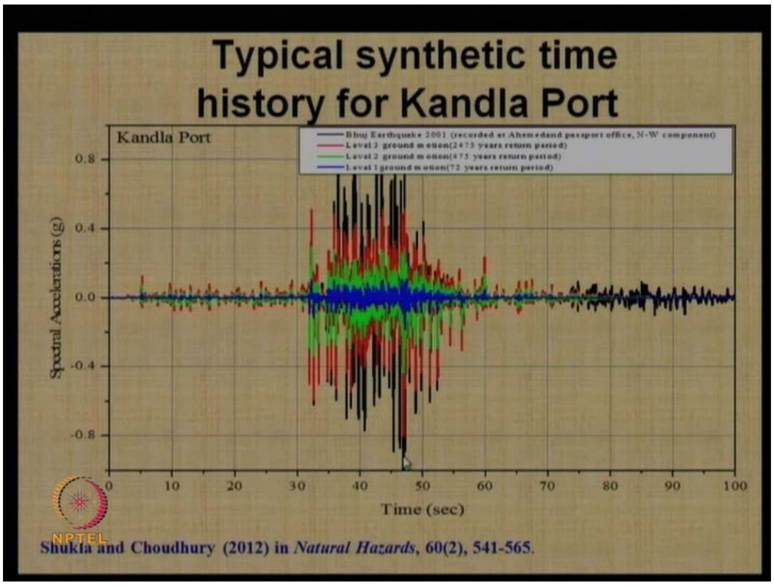

So, this is exaggerated form, you can see the Bhuj earthquake which is the black line. Then level 3 ground means 2475 which is the red one; green one is level 2 which is 475 return period; and the blue one is level 1 that is 72 years return period. So, that is the spectral acceleration versus time response.

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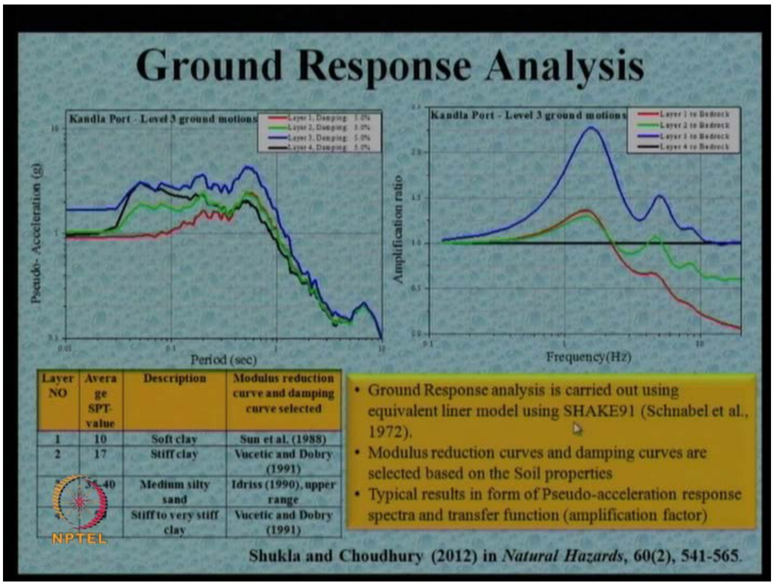

Now, we carried out this ground response analysis, at this site Kandla port with the level three ground motion. I have already mentioned what is level 3. So, for different layers layer 1, 2, 3, 4, these are the layer. You can see how the layers have been divided; and what is the average value of the SPT; and what are their descriptions. Now in this process, equivalent linear analysis was carried out. And for doing that the computer program shake has been used; shake 91 version which is developed by Schnabel et al in 1972 at U C Berkly. It has been further modified. Now latest version of shake 2000 is also available. And you can see for doing the equivalent linear model, you should have known what is the modulus reduction curve, or the damping ratio behavior for individual soil layer which you are analyzing.

So, either you can use the proposed or available design curves which are available with this shake, like we have used for this shake analysis of equivalent linear model, the model of son et al 1988 which is valid for soft clay. Son et al has given for soft clay, what is the G by G max versus gamma behavior? And, what is the damping ratio versus gamma behavior? That model we have used. For stiff clay we have used Vucetic and Dobry's model of 1991 which is already embedded in that shake software. Also in deep soil software you will find these standard or these well known available literature, modulus reduction curve, and damping curve are available. But if you want, for your own site, it is better to develop it through the actual soil property. And use that modulus reduction curve and damping curve in your input model. Then Idriss curve for medium silty, and Vuceticdobry's curve again for stiff clay were used. And for four different layers by considering 5 percent damping for the case of pseudo acceleration or spectral acceleration versus period plot, we got the result like this. Clear.

So, how the amplification ratio can be obtained? Amplification ratio is nothing but at ground level divided by the bed rock. So, that is what for different four layer, you can see this different amplification ratio versus frequency. You can see the black one is just line, horizontal line with value 1 because layer 4 is nothing but at bed rock level. So, layer 4 is at bed rock level. That is why there is no amplification, but other layers shows significant amplification. Whereas layer one is a ground surface. You can see red color for ground surface amplification is this much; but in between the amplification ratio raises upto about 2.25, can you see that; whereas, at ground surface it is only about 1.3. So, it depends on what type of material you have in between. Depending on that site response you will get different amplification ratio at different level. So, somebody is founding or planning or proposing a foundation, to, in this layer number 3, then they have to be very careful about this amplification. Clear.

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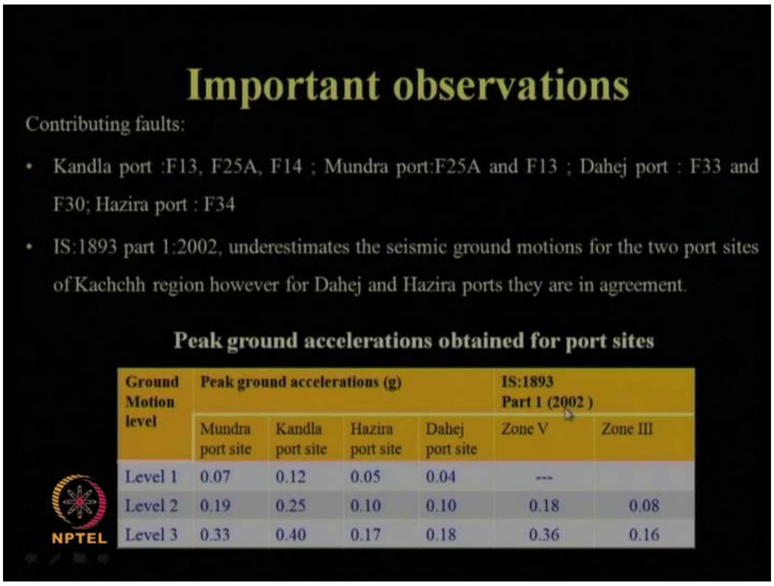

So, what are the important observations we got from this analysis. Important observations like for Kandla port, the contributing faults we got like F 13, F 25A, and F 14; whereas for Mundra port it is F 25A and F 13; for Dahej port the responsible faults or contributing faults are F 33 and F 30; whereas for Hazira port it is F 34. How you will get those things? Because at the beginning we have mentioned this fault map are available, you are using seismic hazard analysis and corresponding UHS is coming from that. So, that will give you which fault is responsible as I said, this F 25 and F 13 are responsible for Kandla port and Mundra port.

So, as per IS code of 1893 part 1 of the year 2002 version, it under estimates the seismic ground motion for the two port sites. Why it under estimates? Let us look at this comparison table, ground motion level 1, 2, 3. Level 1 means return period of 72 years; level 2 means return period of 475 years; and level 3 means return period of 2475 years. We know at which level needs to be used for which type of structure that is known to us. Now for Mundra port site the peak ground acceleration we got these values. Whereas, IS code recommends for zone 5, which ports are in zone 5? This Mundra and Kandla, this two are in zone 5; whereas, this Hazira and Dahej these are in zone 3.

So, when we are comparing the level 2 PGA, as per the IS code the recommended value is 0.18 g. And what is the value we got from the analysis for Mundra port site?

Considering the soil properties at Mundra port 0.19 which is very close to IS code recommendation when we are designing it for 475 year return period, remember that.

So, IS code we can safely use for Mundra port site when we are designing for 475 year of return period. But the same IS code is under estimate this value when we are considering the design for Kandla port. For Kandla port, the site response analysis gives us the value of 0.25 g which is much higher than the IS recommended value, even for the 475 year return period. Whereas for IS code recommended value for 2475 years of return period is, for zone 5 it is 0.36 g; whereas for Mundra port site we got 0.33 g which is very much comparable with respect to IS code.

So, if we use basically IS code recommended values for Mundra port site, design is perfectly fine. But if we use the IS code recommended values for the Kandla port site, as shown over here, using the local soil properties and the input motion, it seriously, the IS code seriously under estimates. So, we should design for these higher values. So, using this comparative study or table, one can easily find out what is the need to carry out the site response analysis before going for a particular design or important design at a particular site. Because port sites are very important; port structures are very important that has to built for several years; that should not get damaged by any earthquake. So, that is why it is mentioned, underestimate the seismic ground motions for two port sites of Kachchh region; however, for Dahej and Hazira port they are very much in good agreement.

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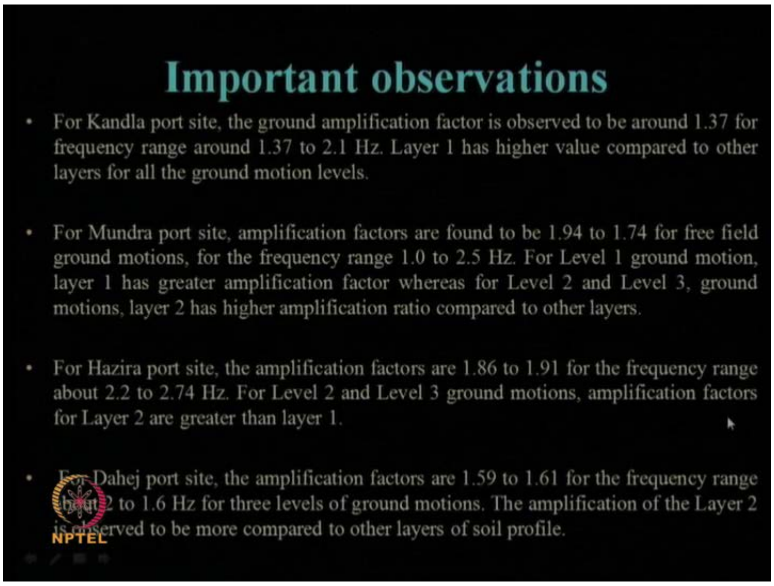

What are the other important observations we got, from this case study? That is for Kandla port site, the ground amplification factor is observed to be around 1.37 for frequency range between 1.37 to 2.1 hertz. Remember this amplification also depends on your input frequency that is what earthquake input motion you have used what frequency it was having. If you use different earthquake motion; obviously, this amplification factor also will change. And layer one has the higher value compared to other layers at all ground motion levels; whereas, for Mundra port site the value of amplification factor is ranging between 1.94 to 1.74 when the earthquake input motion is in the frequency range of 1 to 2.5 hertz. For Hazira port, the amplification factor is 1.86 to 1.91; and the input frequency ranges between 2.2 to 2.74 hertz. Whereas, for this Dahej port site, the amplification ratio is 1.59 to 1.61 with an input frequency range between 2 to 1.6 hertz. So, with this we have come to the end of our module number eight that is site response analysis or ground response analysis. We will continue further in our next lecture.