Geotechnical Earthquake Engineering
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Module - 9
Lecture - 34
Seismic Analysis and Design of
Various Geotechnical Structures

Welcome to today's lecture of NPTEL video course on this Geotechnical Earthquake Engineering. In our previous lecture, we had completed our module number eight, which is site response analysis.

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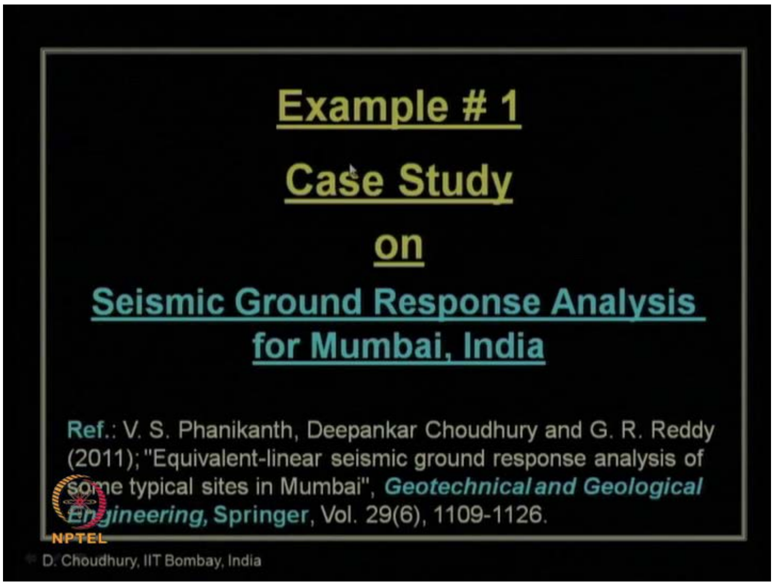

So, a quick recap, what we have learnt in our previous lecture. We had discussed about one example, which is a case study on the seismic ground response analysis for Mumbai city of India. And for that, I have referred to this journal paper by V. S. Phanikanth, Deepankar Choudhury and G. R. Reddy published in 2011, in this journal "Geotechnical and Geological Engineering", "Springer publication". This is the part of Ph.D. thesis work by Dr V. S. Phanikanth under my supervision at IIT Bombay.

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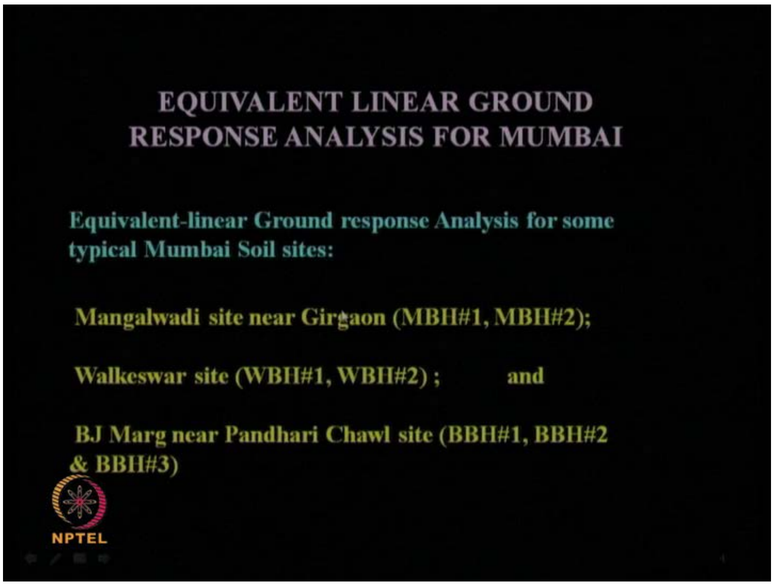

We had seen that equivalent-linear ground response analysis. And later on, non-linear ground response analysis for typical soil sites of Mumbai was carried out. So for that, the first step is to collect the geotechnical properties from the borehole data.

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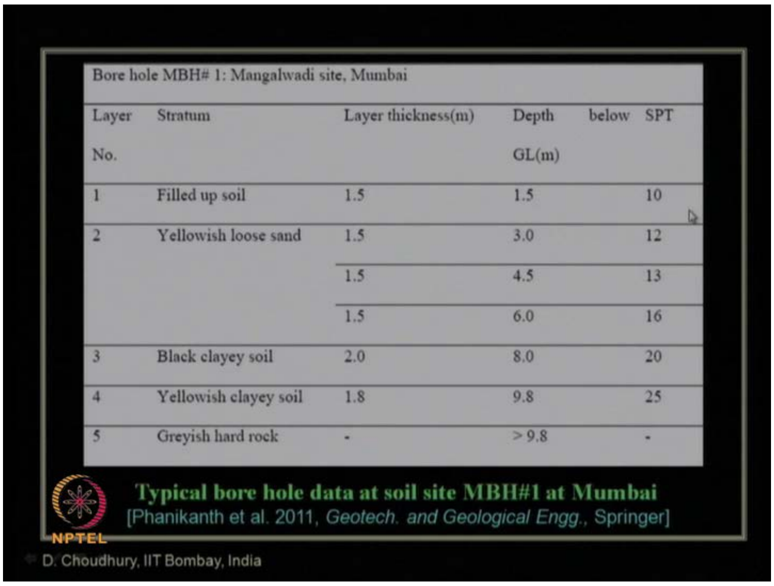
Bore hole MBH# 1: Mangalwadi site, Mumbai

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, various site borehole locations are mentioned over here, which are collected. This is one typical borehole data with the layer thickness, type of soil, the depth and recorded value of SPT; what usually at site will be available from the field data.

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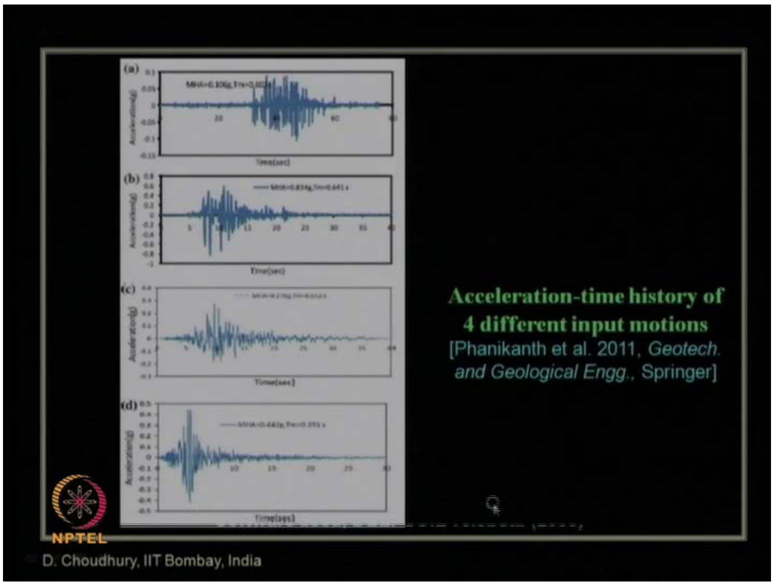

Next step was to identify for which acceleration-time history we want to carry out the ground response analysis. So, we had carried out for these four selected acceleration-time history, the ground response analysis; one is for Bhuj motion of 2001; another is, this is Kobe motion of 1995; this is Loma Prieta, 1989 and Loma Gilroy, 1989.

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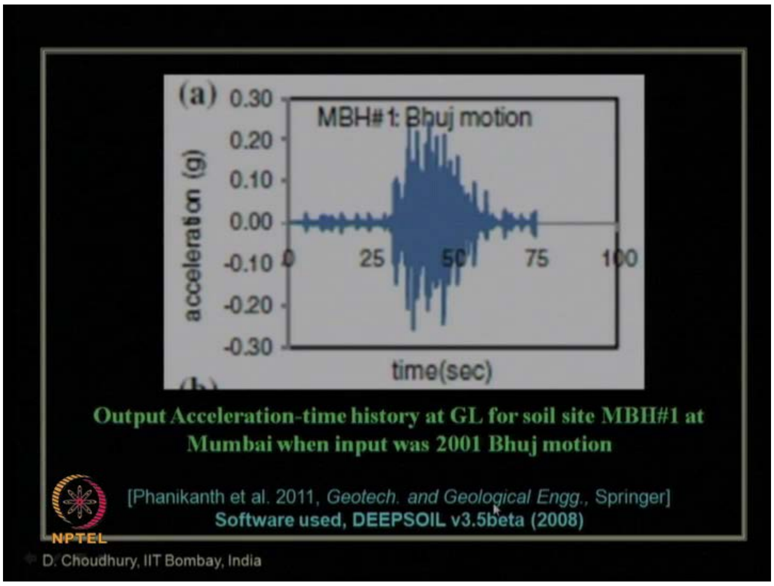

With those input motion after carrying out the equivalent-linear ground response analysis, the output what was obtained is for each borehole location. For a particular input earthquake motion, we can get the output at different levels of soil layer. So, this is at ground level or ground surface. This is the output that is, how the acceleration time response will be when a Bhuj motion is given as input bed rock motion.

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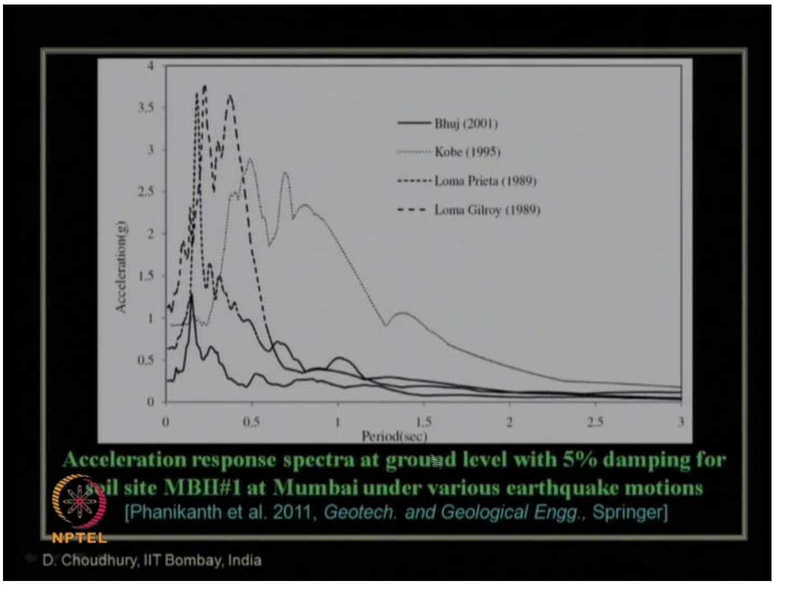

And to do that, deep soil software was used. This is another output. In this form, we obtain the spectral acceleration versus period variation at a particular borehole at Mangalbari site in Mumbai with consideration of five percent damping. As we considered the single degree of freedom system, mass spring dashpot system, to obtain the spectral acceleration for four different input motions; Bhuj motion, Kobe motion, Loma Prieta and Loma Gilroy. So, you can see that the peak value of this spectral acceleration where it is occurring, at which period that determines that, which type of structure will be vulnerable when that type of, a particular type of earthquake motion is occurring at that site of MBH at Mumbai.

So it automatically shows, for Kobe motion the tall buildings will be more vulnerable because here the time period is relatively higher compared to what you can see for Bhuj motion, where time period is relatively lower than that what we obtained for Kobe motion for this particular site. So, this will vary with respect to site to site, as well as we can see it varies with respect to input earthquake motion. That is, for which earthquake motion you are analyzing; that is also very important.

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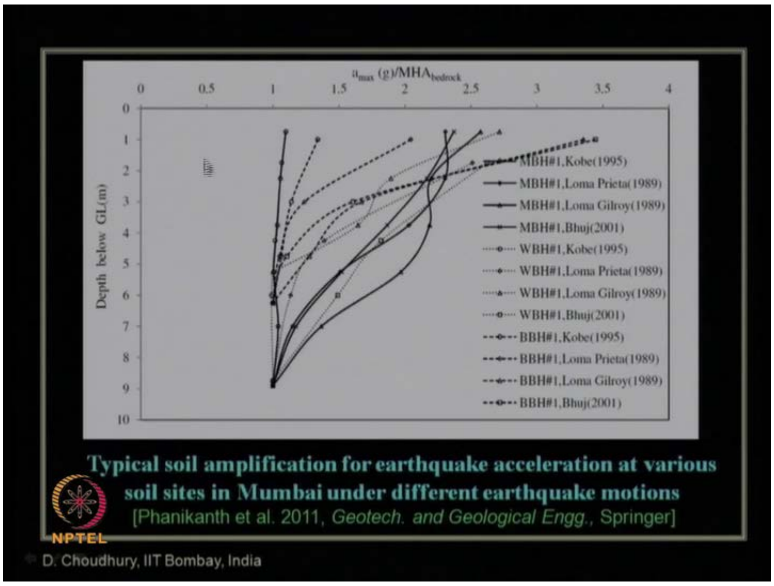

Then we have also seen how much amplification of the bed rock motion will occur, when it passes through various layers of the soil. So, at different soil layers you can compute obviously what is the value of a max by MHA at bed rock level. And these, end values shows the, at the ground level what is the soil amplification. So, you can see it ranges between about 1.2 to 3.5, depending on what is your input motion and also for which borehole this data or this analysis were carried out.

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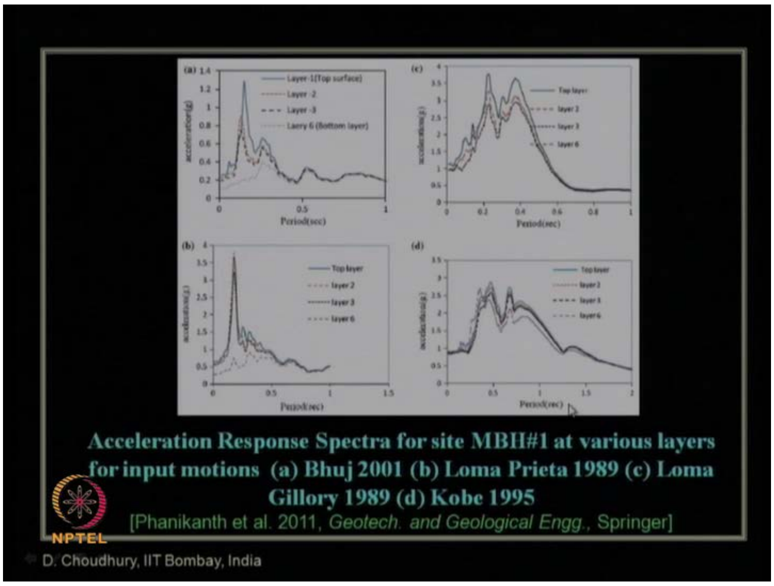

Also we had seen that for different earthquake motions, this different response acceleration spectra, which we can obtain; not only that, layer wise variation of this spectral acceleration also we can get. So, what way it will help us for the design further? Suppose, if we want to construct a foundation at a particular layer or a particular soil level below the ground surface, looking at these spectral acceleration peak values we need to decide at which depth we need to found the foundation based on the soil property; not only that, if we are find… if we are putting the foundation at a particular layer, we have to design for that particular period and spectral acceleration corresponding to different layer with respect to different input earthquake motion. So, which is very important for a detailed study at a particular site, which will not be available in any code because this is a site specific ground response analysis.

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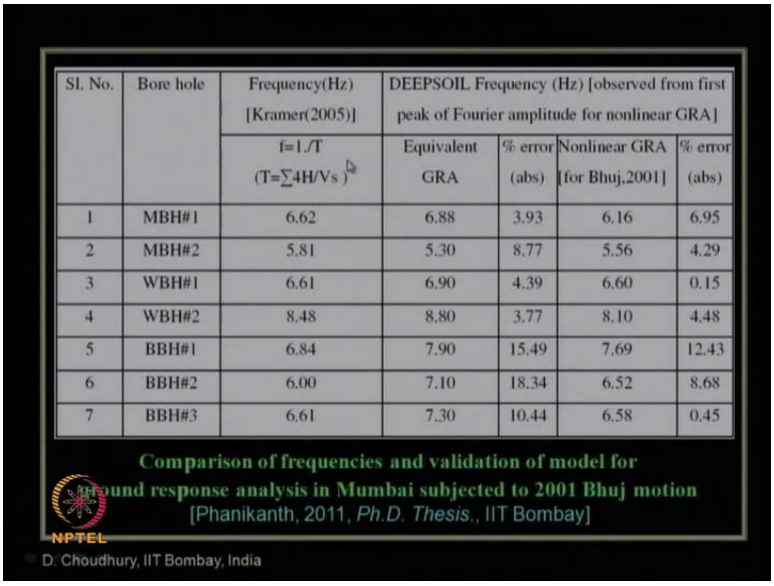

Sl. No.	Bore hole	Frequency(Hz) [Kramer(2005)]	DEEPSOIL Frequency (Hz) [observed from first peak of Fourier amplitude for nonlinear GRA]			
		f=1./T (T=∑4H/Vs)	Equivalent GRA	% error (abs)	Nonlinear GRA [for Bhuj,2001]	% error (abs)
1	MBH#1	6.62	6.88	3.93	6.16	6.95
2	MBH#2	5.81	5.30	8.77	5.56	4.29
3	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

We had also seen in our previous lecture that the frequency computed using this formula, which is a thumb rule. As I have already mentioned for a typical homogeneous soil layer, that is, the fundamental natural period, how to estimate from the V s value compared to the thickness of a soil layer. And, what is the frequency obtained using this deep soil software using either equivalent ground response analysis and non-equivalent or non-linear ground response analysis for Bhuj 2000 motion. You can see the percent difference between the theoretical value and the obtained value through this deep soil software is within the range of about 18 percent for the case of equivalent linear; for non-linear it is within about 12 percent or 12.5 percent.

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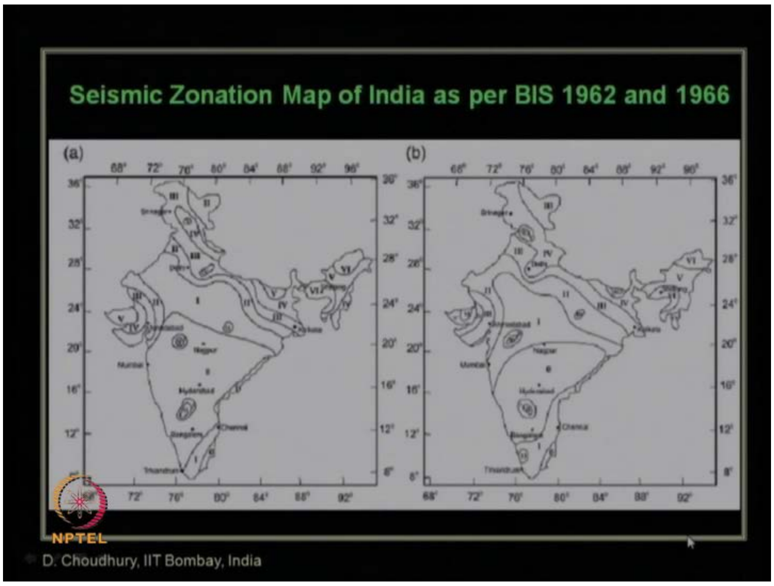

Now, coming to the development of a seismic code of India, let me emphasize on this. That is the Indian seismic code, which is BIS or IS: 1893; that code has obviously revised through several years, whenever there is some major earthquake. And, the researchers and the research community and practicing community on Earthquake Engineering in India felt that there is a need to change the seismic zonation of entire India. It has been changed. So, the latest change was done in the year 2002. As we already have discussed that is the latest version of IS: 1893 part one only; where this seismic zonation map is given for entire India. Why it has occurred or it has been given in 2002 because after the 2001 Bhuj earthquake, the researchers and practitioners in Earthquake Engineering felt that it needs to be updated.

So you can see from this slide, the first figure, figure a. This shows the seismic zonation map of entire India which was in the year 1962. I am talking about same IS code 1893 part one. So, that is the code number. And, these are the years of their changes. So 1962, if you look at this zonation map very carefully, you can see earlier entire India was sub divided into six seismic zone, starting from zone one to zone six; so, one, two, three, four, five and six. So, zone six was the most vulnerable one and zone one was the least vulnerable one.

Then, that code was revised in the year 1966. So, figure b shows the IS code: 1893 part one, where the seismic zonation map has been changed in the year 1966. Again the same zone one to six were considered for entire India, but you can see some of the zone in this western part like in Rajasthan, etcetera; which were earlier in zone one, now they have changed to zone two. Can you see?

So, there are several other changes as well. Like here, the zone five and zone six 6, their zonation demarcation has been changed. Also in this central area of India, they have been introduced this zone two region at this portion. Can you see over here? Again in the southern part also here, zone two has been introduced; which were not there. Initially, here it was zone one. And, extent of zone one also has been changed. So, like that there are several changes between the code of 1962 to 1966. And, this has happened because of the experienced people felt between this 1962 to 66, whatever earthquake occurred in India, taking care of all these effects they have changed this seismic zonation map.

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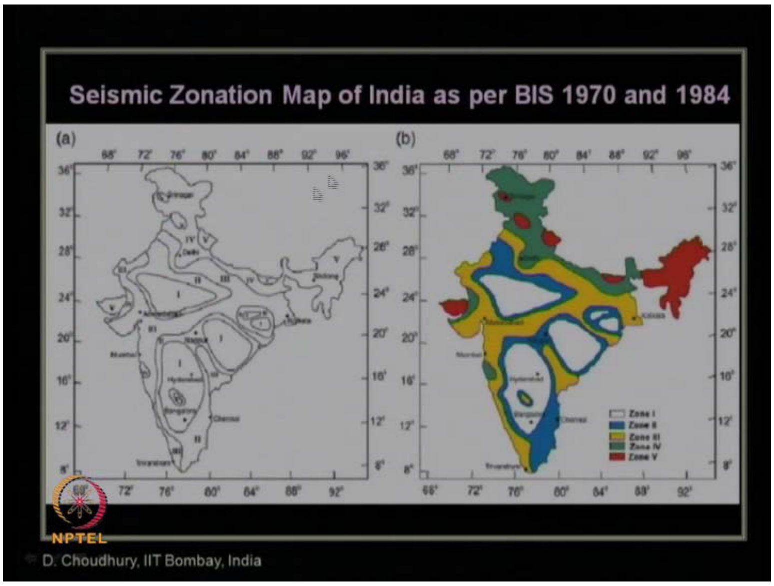

Now the 1970 version of the IS code 1893 part one, where the seismic zonation map; you can look here what are the changes from the previous version of 1966. From 66 to 1970, the number of zone has been reduced by one. Earlier it was zone number one to six. Now, it has been given zone number one to five. Can you see over here? So, five is the most vulnerable zone and zone one is the least hazardous zone. So like that, the entire India and the regions also have been changed at different locations as you can see. Even zone three has been introduced over here at this southern part; zone three has been introduced over here in the western part, etcetera. And, entire this north east has been kept under zone five, which is most hazardous one or most vulnerable one. Even this Himalayan belt or Nepal-India border; close to that, there are some regions which has been changed to zone four and zone five; the combinations as you can see.

Next, the IS code: 1893 part one was revised in the year 1984. So, this colored picture shows the version, which is the fourth revision of 1984 version of Indian code IS: 1893 part one. You can see here also it has been subdivided into five zone; zone one to five. One is least hazardous zone and zone five is maximum or most hazardous zone.

So, these red color zones are most hazardous one; whereas this white color zones are least hazardous one. But if you can look at this picture, there were zone two over here and zone three over here. Ahmedabad was coming earlier in zone three like this and their values were given. After Bhuj earthquake, people felt that this region mapping is not proper. And, also between 1984 to 2002, the latest version where it is available, there were several other earthquakes in India like Chamoli earthquake was there, Jabalpur earthquake was there. There were many other earthquakes. So, central part of India also required after the Jabalpur earthquake, etcetera; change of this zonation map.

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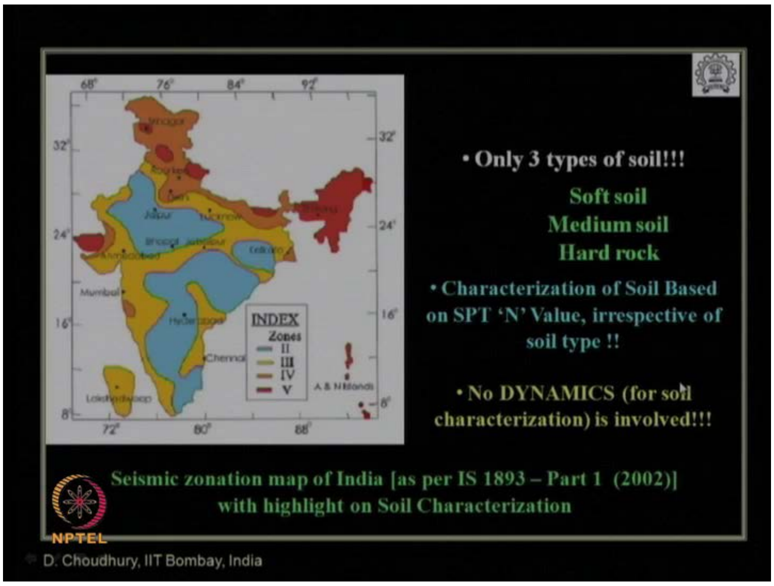

That is why in 2002, this seismic zonation map has been proposed in Indian seismic code IS: 1893 part one. This is the latest version as on today, 2002. So obviously, you can expect this version also probably will get updated whenever there are more number of earthquakes and more number of experiences, which we incur at India. And then, accordingly the changes of this seismic zone may occur in future also. And for that, to come up with this seismic zonation map, major criteria are to identify the effect of earthquake. Also, how to determine this values, peak spectral acceleration, etcetera, depending on the soil type; those what I have discussed in the site specific ground response analysis. So obviously, it, the code cannot give individual site specific values, but it will have to give a broad value which will overall match a kind of design suggestion for regular buildings. And considering the important factor, etcetera, for important structure it can be considered.

So, now in this latest version of IS code: 1893 part one, the seismic zonations are divided in four zones only. starting with zone two, which is least hazardous; with maximum one is zone 5, which is maximum hazardous. So, you can see here this zone four has been introduced, extent of zone five has been increased, this central region of Jabalpur, etcetera, the seismic zonation has been changed. So like that, there were several changes from 1970 version to 1984 version.

Now, coming to our geotechnical aspects of our IS code what it suggests, let us look at it. Unfortunately, the geotechnical aspects of Earthquake Engineering is not yet well addressed in this IS code: 1893 part one of 2002 latest version; because it specifies that, consider only three types of soil than those are mentioned as soft soil, medium soil, hard rock. But as a geotechnical engineer, we already know that hardly these nomenclature of soil signifies anything, unless their typical engineering value or the dynamic soil properties are specifically mentioned within a given range. So soft soil, whether it is sandy or clayey there will be huge difference in that; also for the medium soil as well. Even there should be the variation of these soil strata for the classification based on the dynamic soil properties, when we are talking about the Earthquake Engineering.

However our code specifies only the characterization of soil based on the… as we can see in this slide based on the SPT N value. Let us look at the slide. SPT N value irrespective of the soil type, but we know that SPT N value is not the only solution or only field test; because for pure clay you can hardly do the SPT test. There you have to probably perform the CPT; cone penetration test. Standard penetration test may not be useful there. So, all these aspects need to be yet to be addressed in our IS code part one form the geotechnical point of view. And as I said, no dynamics for the soil characterization is yet involved in this codal provision.

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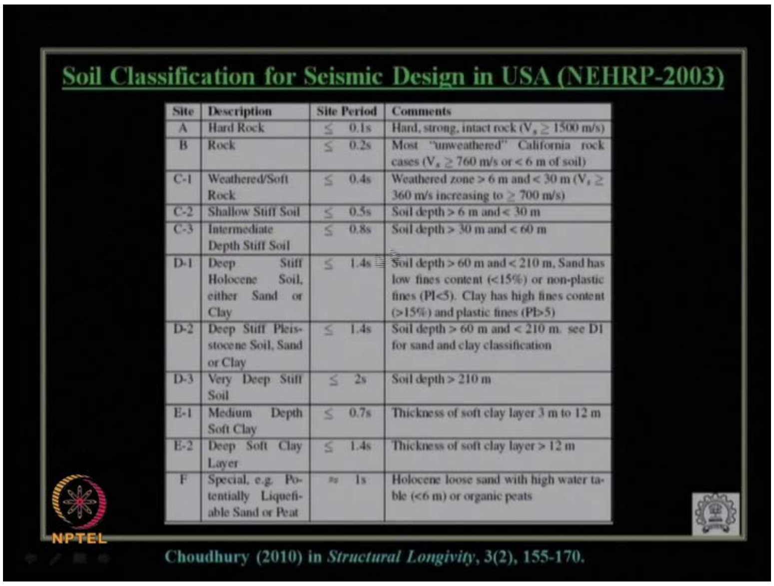

Soil Classification for Seismic Design in USA (NEHRP-2003)

Site	Description	Site Period	Comments
A	Hard Rock	$\leq$ 0.1s	Hard, strong, intact rock ($V_s \geq 1500$ m/s)
B	Rock	$\leq$ 0.2s	Most "unweathered" California rock cases ($V_s \geq 760$ m/s or < 6 m of soil)
C-1	Weathered/Soft Rock	$\leq$ 0.4s	Weathered zone > 6 m and < 30 m ($V_s \geq$ 360 m/s increasing to $\geq$ 700 m/s)
C-2	Shallow Stiff Soil	$\leq$ 0.5s	Soil depth > 6 m and < 30 m
C-3	Intermediate Depth Stiff Soil	$\leq$ 0.8s	Soil depth > 30 m and < 60 m
D-1	Deep Stiff Holocene Soil, either Sand or Clay	$\leq$ 1.4s	Soil depth > 60 m and < 210 m. Sand has low fines content (<15%) or non-plastic fines (PI<5). Clay has high fines content (>15%) and plastic fines (PI>5)
D-2	Deep Stiff Pleis-stocene Soil, Sand or Clay	$\leq$ 1.4s	Soil depth > 60 m and < 210 m. see D1 for sand and clay classification
D-3	Very Deep Stiff Soil	$\leq$ 2s	Soil depth > 210 m
E-1	Medium Depth Soft Clay	$\leq$ 0.7s	Thickness of soft clay layer 3 m to 12 m
E-2	Deep Soft Clay Layer	$\leq$ 1.4s	Thickness of soft clay layer > 12 m
F	Special, e.g. Potentially Liquefiable Sand or Peat	$\approx$ 1s	Holocene loose sand with high water table (<6 m) or organic peats

Choudhury (2010) in *Structural Longivity*, 3(2), 155-170.

Now, let us see what are the other practices worldwide. If we see the seismic design criteria of US, as per the NEHRP code of 2003; NEHRP is the guide line for the earthquake resistant design in US. They have classified their site, soil site or ground classification. They have subdivided into in six divisions starting from A to F. And within each division, also in some of the site classification they have sub classification. As you can see over here, how these have been classified with respect to the site period. What we have discussed in our ground response analysis, we get a site period which is more important for our design, rather than any other static values. Also, we should know what is the shear wave velocity because shear wave velocity is one of the major important criteria, which shows about the dynamic nature or characterization of the code.

So, in the NEHRP code they classified soil site A as the stiffest site with hard rock with site period within 0.1 second. And, what is the range of V s value? It should be more than equals to 1500 meter per second. Second site classification is site B which is rocky type site, where site period will be less than equals to 0.2 second. And, mostly it is unweathered rock and the V s value will be greater than 760 meter per second or less than 6 meter of thickness of the soil will exist.

Then within site classification C, there are sub classification; C 1, C 2, C 3. C 1 classifies weathered or soft rock with this site period value and V s value with the range between 360 to 700 meter per second. For C 2, it is soil depth should be between 6 meter to 30 meter and this is the site period where shallow stiff soil is available. And, C 3 is intermediate depth stiff soil where site period is this much and soil depth can be between 30 meter to 60 meter.

Then site classification D, again sub classified into D 1, D 2, D 3. D 1 is deep stiff Holocene soil. It can be either sand or clay, but their site period should be within 1.4 second and their depth can be between 60 meter to 210 meter with low fine content within 15 percent and non-plastic in nature. And, clay has high fines content greater than 15 percent and plastic fine greater than 5 or P i value greater than 5. D 2 is the deep stiff soil, sand or clay with this range. And, D 3 is very deep stiff soil with this value of site period with soil depth greater than 210 meters.

Whereas, site classification E again sub classified in E 1 and E 2. E 1 is medium depth soft clay, where site period is less than equals to 0.7 second and thickness of soft clay layer will be between 3 meter to 12 meter. Whereas E 2 is deep soft clay layer, where site period is this much and thickness of the soft layer can be greater than 12 meter.

Whereas, site classification F refers to the special type of soil which is potentially liquefiable soil, that is, potentially liquefiable sand or peat. So, these are most vulnerable with respect to earthquake is concerned. Their site period is about one second and Holocene loose sand with high water table, then only the chances of liquefaction will be more as we know and the organic peat contents. So, the details you can obtain in this journal paper as you can see over here, ‘Structural Longevity”; this paper.

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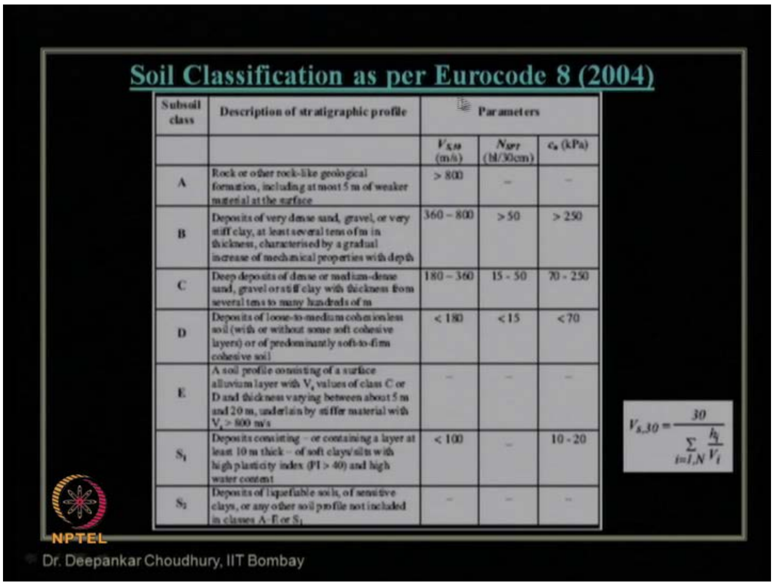

Soil Classification as per Eurocode 8 (2004)

Subsoil class	Description of stratigraphic profile	Parameters		
		$V_{s,30}$ (m/s)	N_{SPT} (bl/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	> 800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of m in thickness, characterised by a gradual increase of mechanical properties with depth	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers) or of predominantly soft-to-firm cohesive soil	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with V_s values of class C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s > 800 m/s	–	–	–
S_1	Deposits consisting – or containing a layer at least 10 m thick – of soft clays/silts with high plasticity index (PI > 40) and high water content	< 100	–	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in classes A-E or S_1	–	–	–

$$V_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{V_i}}$$

Now, soil classification as per Eurocode which is followed in entire Europe, of 2004 version Eurocode 8; you can see there also it has been sub classified into major five divisions; A, B, C, D and E. And then, another two sub classification S 1 and S 2. And, what are the parameters to identify this description of the different types of soil for this site classification? Of course the V s value.

They have mentioned another parameter which is known as V s 30. What is V s 30? Let me explain it to you. V s 30 is nothing but, it is the average value of the shear wave velocity V s within top 30 meter from the ground surface. So, how it is estimated? This is the depth 30 meter in the numerator, and denominator is the sum of all the layers. There can be several numbers of layers within that 30 meter thickness of each layer in meter unit divided by individual layers, V s value. Clear? So, that V i; when i changes from 1 to N. N numbers of layers, if it is there.

So, V i will be in the unit meter per second. So, you will get V s 30 also in meter per second. So, these are the ranges of V s value given, and corresponding what can be the typical SPT values? That also have been mentioned with respect to what can be typical values of the cohesion. You can see over here, these are mostly soft clay S 1, S 2, where V s value is even less than 100 meter per second and C value between 10 to 20 kpa; whereas, these are very stiff value. And, the first site classification a is nothing but, it is a rock; where V s value is greater than 800 meter per second.

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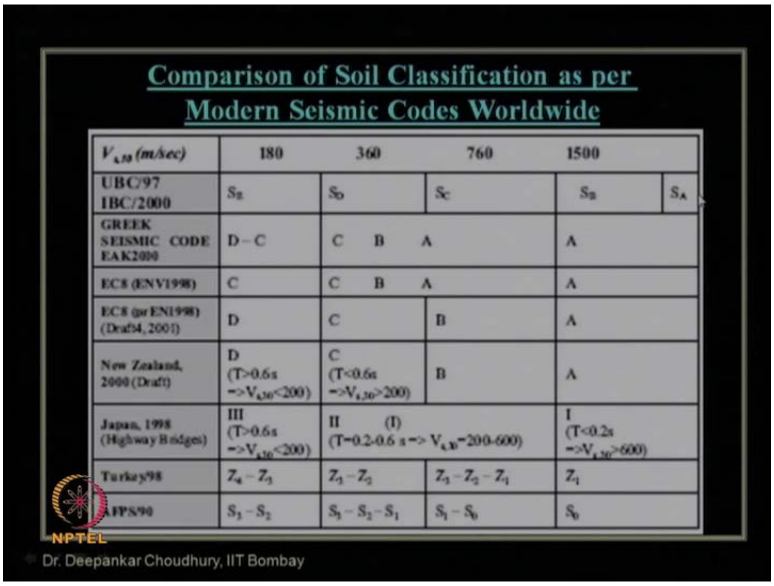

The other soil classification in modern seismic code worldwide, you can see IBC. IBC is International Building Code of 2000 or UBC of 97. They also classified the soil site with respect to the V s value. So, V s is most important parameter. Even Greek seismic code, they also classify it with respect to V s value. E C 8 already I have discussed.

Then New Zealand code; they also classify the soil with respect to V s value as well as the site period T value as you can see over here. Then Japanese code; they also classified with respect to V s value including the site period. Then Turkish code; they also classified with respect to V s value. So, V s is very important parameter for the site classification.

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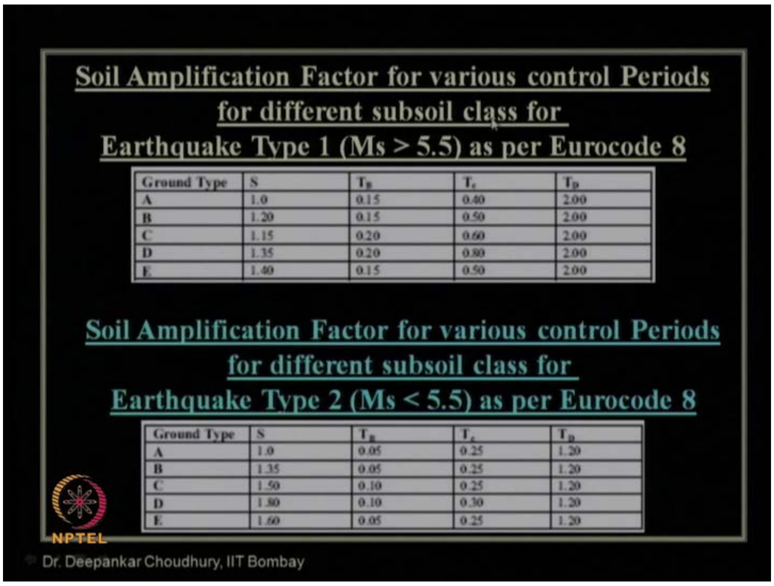

Also, if we look at what are the recommendations for the soil amplification factor for various control periods for different sub soil class; what are the different sub soil class? Already we have mentioned like A, B, C, D, and E. These five are major sub class and then S 1 and S 2 are other two different sub class.

So in Eurocode 8, they have mentioned that different types of earthquake they have first classified. One; type one earthquake means where the earthquake magnitude with respect to surface wave magnitude is greater than 5.5. For that, they mentioned what are the values for different site condition A, B, C, D, E. These are the S factors and these are various time period; mean time period, critical time period, predominant time period, all these values. And, these are the amplification factors which you can see for E type of soil.

It is mentioned that amplification factor of 1.4 should be used for design, when earthquake magnitude is greater than 5.5. And, for a low magnitude earthquake, that is, earthquake type two; when M s value is less than 5.5. These are the recommendations, where you can see the higher values of amplification factors are proposed. Why? Because we have already mentioned earlier; generally, the low value of earthquake tends to magnify more than the high value of earthquake. So, this is the reason. And that too it depends on the soil condition, whether it is a soft soil if you go from A to D E or D; that means you are going towards the soft soil condition. So, that is why these are the recommended values as per Eurocode; which are yet to be incorporated or yet to be considered in our Indian seismic design code considering the Indian sub soil condition and all the Indian site response analysis, which we have discussed in our previous lecture thoroughly.

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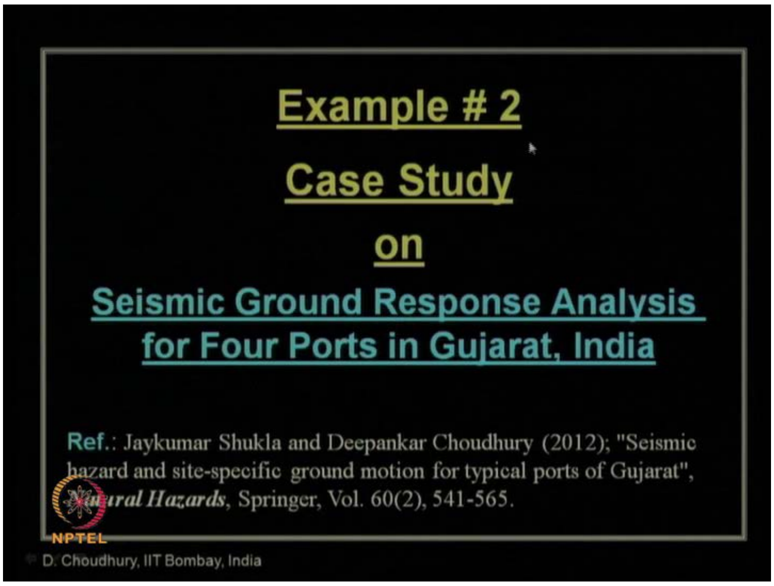

Then in the previous lecture we also discussed about the example problem two, which is the seismic ground response analysis for selected four ports in the Gujarat state of India, and the publication in the “Natural Hazards Springer” journal is available. I have mentioned, this is the detail of the journal paper.

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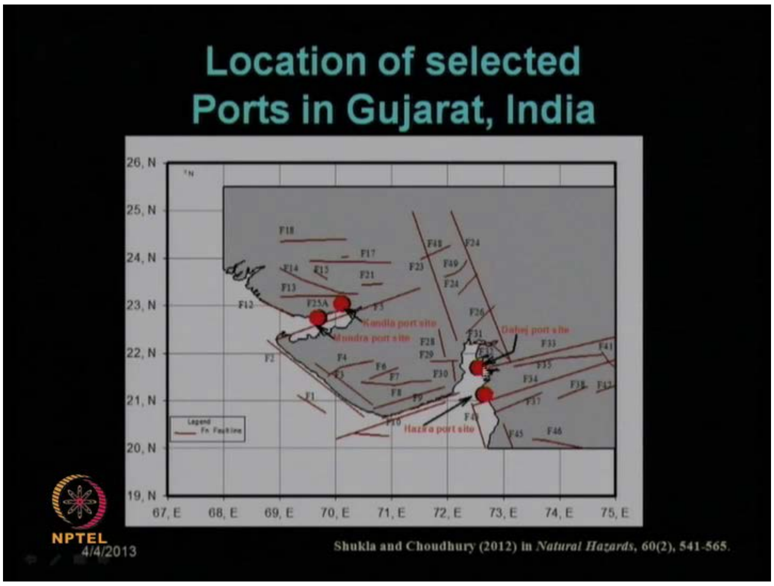

And for that, we first identified the four locations of these ports; Kandla port, Mundra port, Dahej port and Hazira port in Gujarat with their fault mapping.

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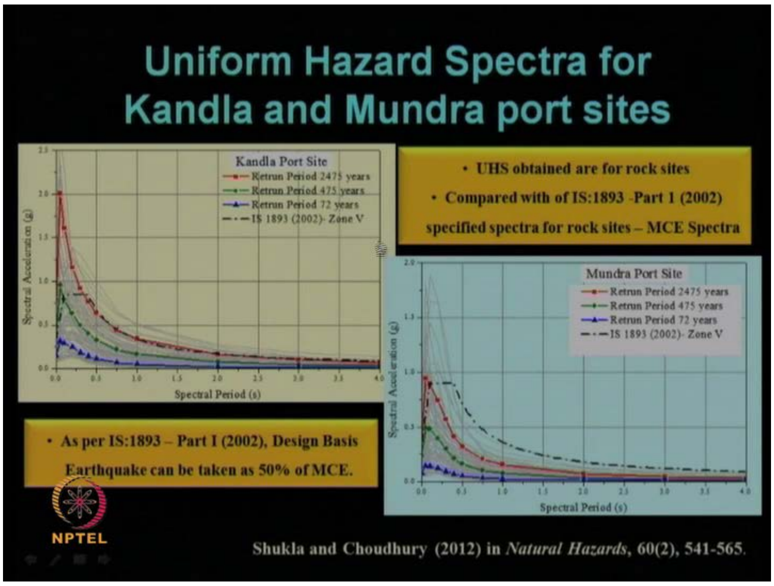

Then, we selected the uniform hazard spectra for individual port site like kandra Kandla port and Mundra port, it is mentioned over here. Compared to IS code also it has been mentioned, corresponding to their seismic zone factor as per IS code 2002 version part 1.

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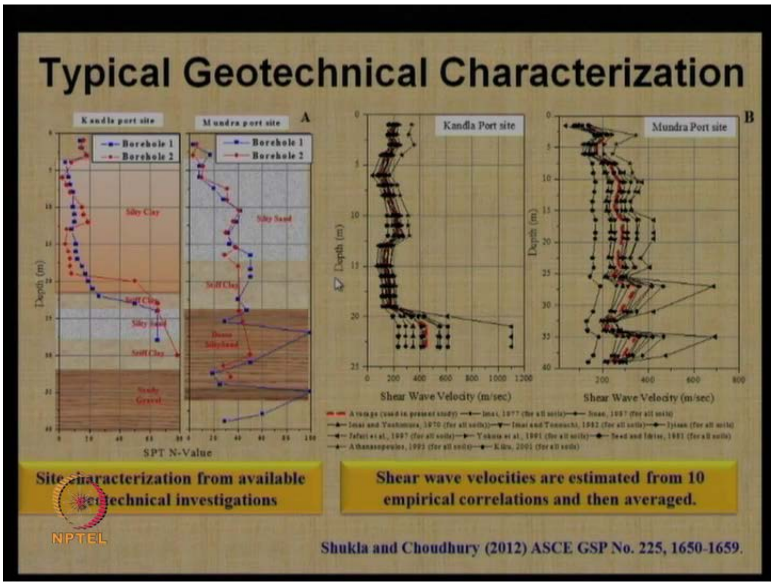

Then, we had collected the geotechnical borehole data for each of these port site at different borehole locations. And, typical data is given from which we estimated the shear wave velocity profile in each of this borehole.

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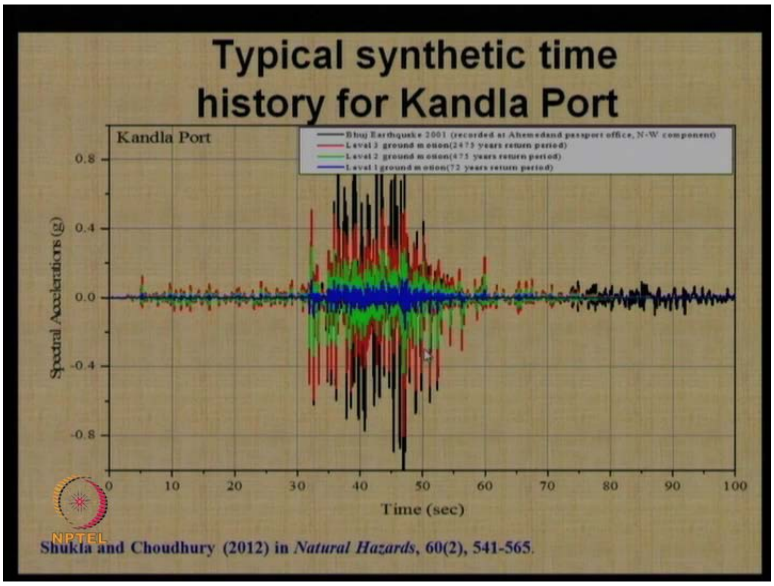

Then, for typical synthetic time history for Kandla port at different seismic level of ground motions with different return period, we got the spectral acceleration versus time value.

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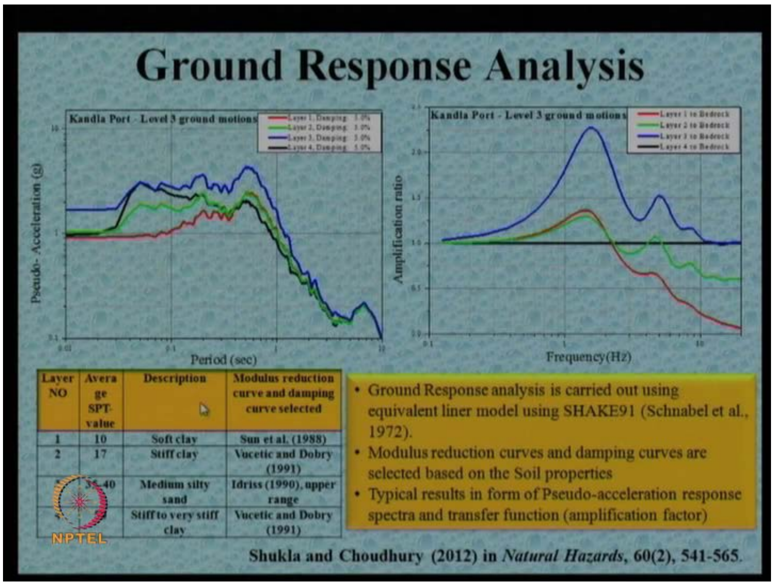

And, the ground response analysis was carried out to obtain the pseudo acceleration versus period, considering different modulus reduction curve and the damping curve as proposed by various researchers for different types of soil using shake software.

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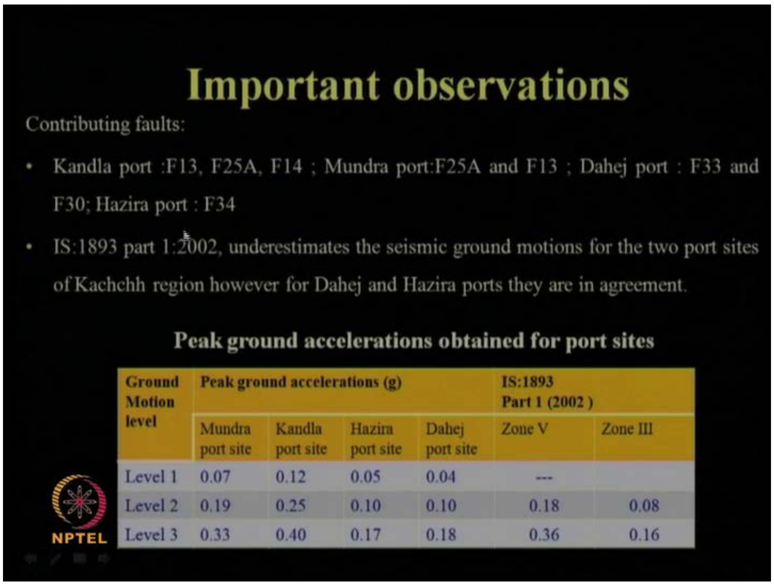

And the important observation what we found that, mostly for zone five region where the Kandla port and Mundra port are coming; actually for level three earthquake, the Kandla port site, whatever the value of peak ground acceleration, we estimate from the ground response analysis is much higher than the IS code recommended value. So, this automatically shows the need for doing the ground response analysis at an important site before going for a design; because IS code give a generalized value which may not be correct or may not match at a particular location, if there is a soft soil site or some not so good soil with respect to foundation is concerned in terms of Earthquake Engineering. So with that, in the previous lecture itself we completed our module eight.

So, in today's lecture we will start with our next module, which is module nine. So module nine, we will discuss about seismic analysis and design of various types of geotechnical structures. Various types of geotechnical structures; within that, we will try to include retaining wall foundations, water front retaining wall or see wall, MSW land fill, pile, tailing dam, slope, etcetera. So, now let us start with this sub topic within this module; seismic design of retaining wall.

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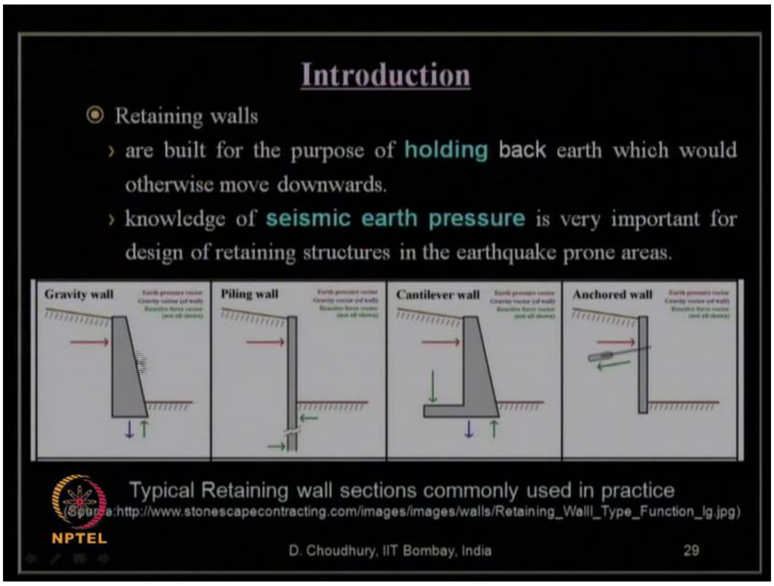

When we talk about seismic design of retaining wall, first let me introduce. As we all know, there are different type of retaining wall like gravity type, piling wall, cantilever type of wall, anchor sheet pile wall. These are different types of retaining wall.

This is gravity type retaining wall, this is the rigid wall; whereas anchored sheet pile wall, these are called flexible wall. So, those we have learnt in our conventional Geotechnical Engineering course. I will not go in to detail of this different types of retaining wall. We will consider now, how this knowledge of seismic earth pressure. That is, what are the earth pressure for which we need to design this retaining wall need to be calculated. So, what are the changes in that value of earth pressure when we are considering the effect of seismicity or earthquake.

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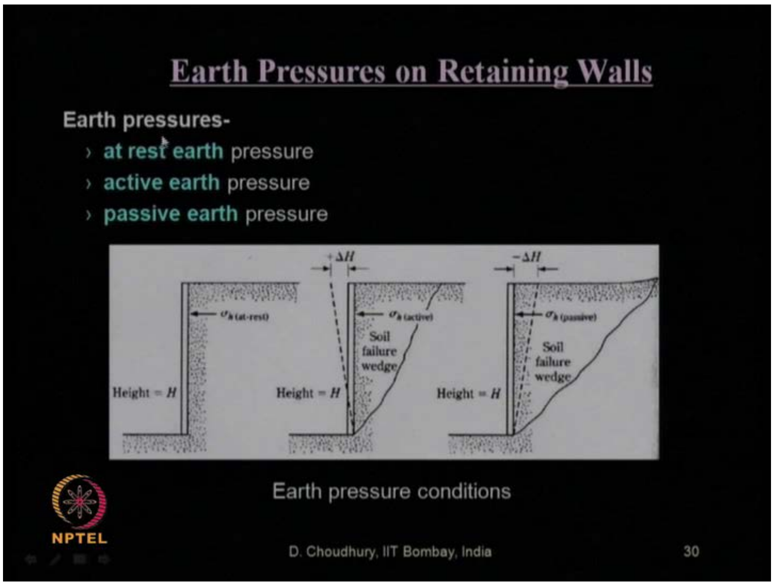

So, what are the different types of earthquake? As we know majorly, these are three types. At rest, earth pressure; when there is no movement of the wall, then the pressure exerted on the wall is nothing but at rest condition. Active state of earth pressure; when wall moves away from the backfill, then the pressure exerted by the soil on the wall is called active state of earth pressure. And, when wall moves towards the soil, then soil provides the resistance which is called passive earth resistance. That is the correct terminology. Though, we also use the terminology ‘passive earth pressure’. So, these are the three different types of earth pressure as we all know from our basic knowledge of Geotechnical Engineering.

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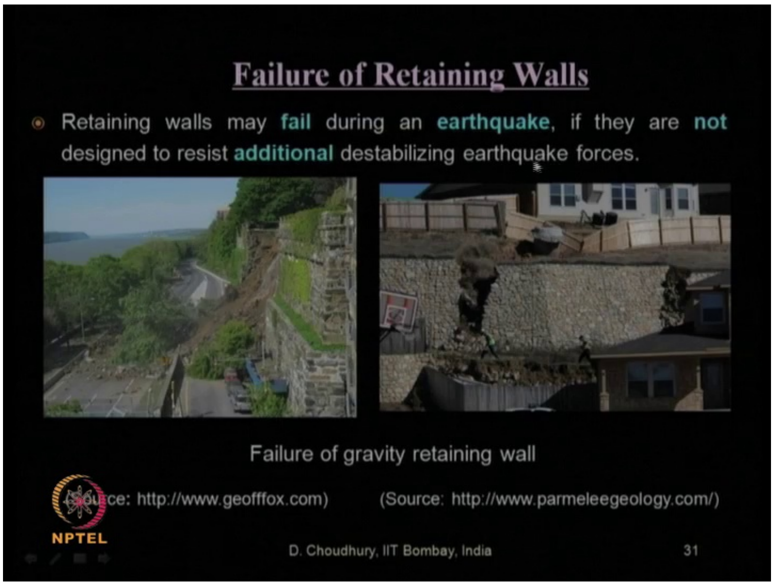

Now, failure of retaining wall; there are several cases worldwide that during the earthquake, retaining walls fail due to the additional destabilizing earthquake forces. So in what way, this earthquake forces induce the additional forces on this retaining wall, for which probably it was not designed or analyzed; that is why there were several damages as you can see from some of these pictures.

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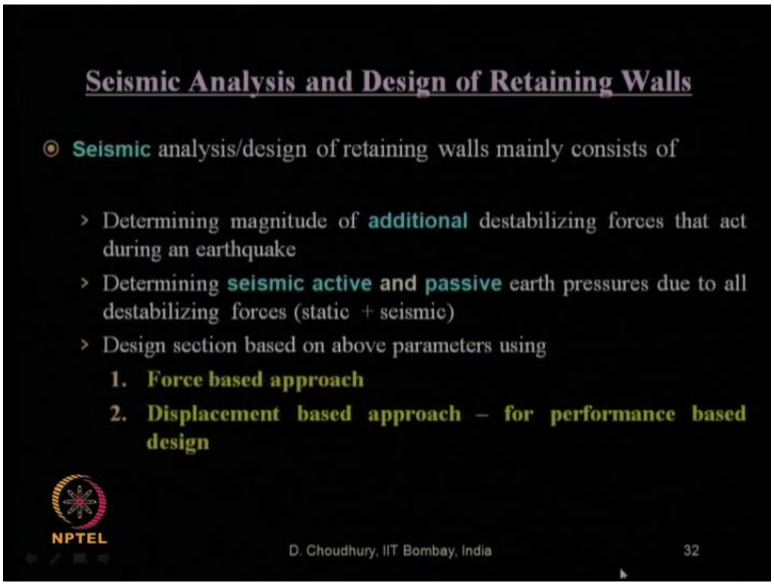

So, when we are talking about seismic analysis and design of this retaining walls, it mainly consist of determining the magnitude of that additional destabilizing force; that act during an earthquake. So, there will be static state of earth pressure on the wall depending on the movement of the wall. In addition to that, there will be some extra earth pressure which is because of the earthquake, which we need to estimate properly.

Now, determining this seismic active and seismic passive earth pressure due to destabilizing forces means, we are finally going to obtain combined earth pressure which is under static condition plus under the seismic condition. And the design section which needs to be selected after computing this seismic active or passive earth pressure, depending on which case of movement of wall is occurring, based on the above parameters by using two basic approach. There are two basic approach. What are those approach? One is called force based approach; another is called displacement based approach. In force based approach, generally we consider all the forces involved in the estimation of earth pressure, that is, weight of the failure zone, weight of the wall, reaction from the soil, etcetera, to obtain what is the pressure exerted on the wall, that is, force based approach. We can either use limit equilibrium method or limit analysis or method of characteristics, etcetera.

Another approach is displacement based approach, where we take care of how much displacement of the wall we can permit. There will be a permissible amount of displacement. And based on that, earth pressure, etcetera needs to be calculated. So, if we look at this slide for performance based design; if we talk about that any wall how it performs, we need to design for that. Then, we should go for this displacement based approach where we can monitor or we can find out or we can estimate how much displacement of the wall. It can be translational; it can be rotational displacement, how much it is occurring.

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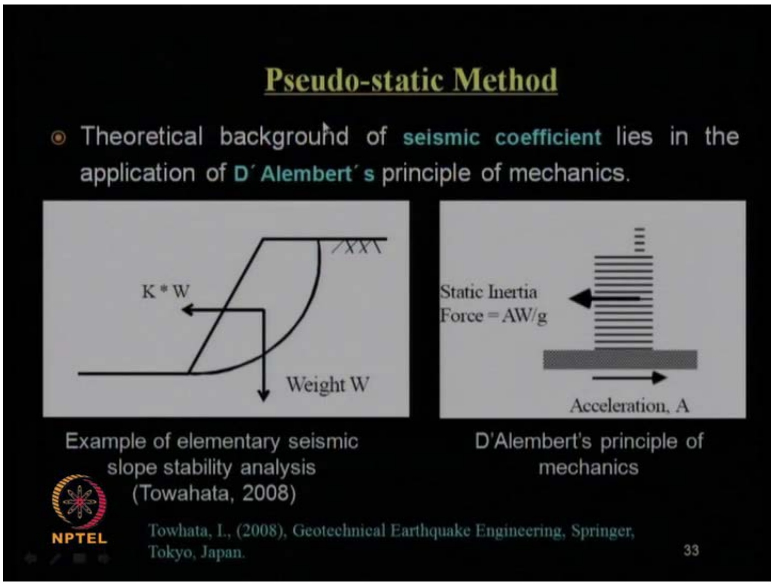

Now before we start estimating the seismic earth pressure, let me tell you the basic methods which are available. The very basic fundamental method is known as pseudo static method. What is this pseudo static method? As the name suggest pseudo static or in other words it is called quasive static also. In this case, theoretical background of the seismic coefficient lies in the application of D’ Alembert’s principle, which we have already learnt in our soil dynamics course as well as in this course also in the beginning, Principle of Mechanics.

So, suppose this is the failure zone of the soil which is having a weight of W, now there will be horizontal component of seismic acceleration. If we take the coefficient of that seismic acceleration, multiplied with that failure mass or failure weight, then what we get? That force will give us nothing but the inertia force. This is called seismic inertia force due to the horizontal acceleration of earthquake. So, this coefficient is known as pseudo static seismic acceleration coefficient. So, what will be the seismic acceleration? K times g; where small g is the acceleration due to gravity; that will be the acceleration value. And how to select that value, there are various methods like Terzhagi has proposed depending on severity of the earthquake; whether, one can consider this K value as half of the peak ground acceleration or two third of the peak ground acceleration or one third of the peak ground acceleration. That depends on what type of earthquake severity you are considering for your design.

So, this pseudo static method basically was initially proposed by Terzhagi in 1950. So after that, several researches have worked on this very basic fundamental or simplified method of pseudo static method; where this, just a coefficient is multiplied with respect to the failure mass to get the seismic inertia force. This portion has been taken from the book by Professor Ikuo Towhata. See, "Towhata 2008"; this is the book reference "Geotechnical Earthquake Engineering" published by 'Springer'.

You can see, when static inertia force acts when there is an acceleration a, there will be the inertia force that A times W divided by g. This small g is nothing but acceleration due to gravity; that is 9.81 meter per second square. So this similar way, A by g gives you this coefficient K as I have already mentioned. So, this is nothing but pseudo static coefficient, which you can multiply with respect to any failure zone, and get your seismic analysis; with respect to this, pseudo static approach can be done.

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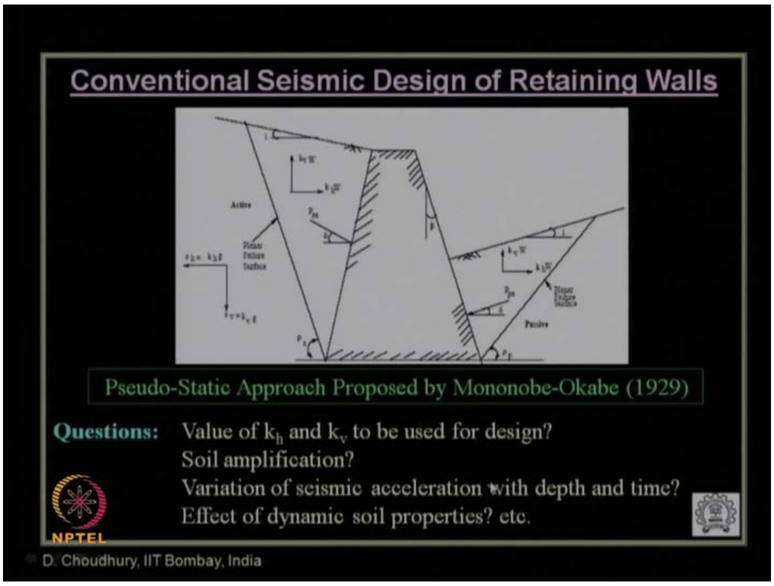

So, the basic concept of this pseudo static approach for the seismic design or seismic analysis of retaining wall was proposed by Mononobe and Matsu in 1929 and Okabe in 1926, which combinedly known as "Mononobe-Okabe method of 1929". So, this Mononobe-Okabe method is the pioneering work in this seismic design of retaining wall, using this concept of pseudo static approach. So, what they had proposed? Suppose this is the gravity type rigid retaining wall section, and if the wall tends to move in this side where you have a backfill level at this height, on the left side of the wall and on the right side you have this much height of the backfill. So, if the wall tends to move in this direction, obviously active state of earth pressure will be formed at this side and passive state of earth pressure will get generated on this side. Right.

So, Mononobe-Okabe, they considered actually they extended the conventional earth pressure theory of Coulomb's earth pressure theory. What was the Coulomb's earth pressure theory? As we know, planer failure surface was considered. And, various planer failure surface, that is, this angle of failure plane with respect to horizontal; that has been varied in such a way that this active earth pressure which is acting on the wall is maximized. Because we know what should be the design value of active earth pressure. That should be the maximum value of this; what we should get through trial and error procedure.

So, to get that trial and error procedure, either mathematically in the closed formed solution. It can be obtained in terms of by differentiating with respect to this failure angle, the total earth pressure component. What method was used by coulomb earth pressure theory? Just simple equilibrium method of all the forces. What are the forces? This is the earth pressure force, this is the weight and there will be a soil reaction.

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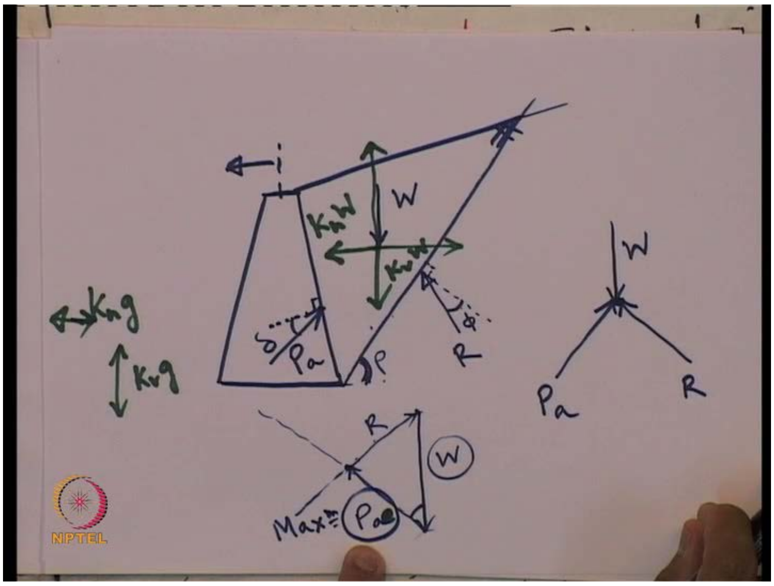

So if I show here, how the couloumbic earth pressure; let us say for active state of earth pressure, this is the rigid retaining wall. And, we are mentioning that this wall is moving towards this side. So, this is an active state of earth pressure. As coulomb's theory says, we take this failure plane. This failure plane can be considered at different failure angle as we all know. There will be forces like weight of this failure plane and this will be active earth pressure acting at an angle delta with respect to normal to this wall; delta is wall friction angle, P a is active state of earth pressure. And, on this side of the wedge there will be soil reaction R, which will act at an angle phi; phi is the soil friction angle, this is the normal to this failure plane.

So, that is simplified coulomb's method of earth pressure for active state of earth pressure as we all know. Where, these three forces must maintain the equilibrium. And, for a two dimensional problem because retaining wall is nothing but a plane strain problem. As we know it runs for several meters or even kilometers also. So, for this plane strain problem, for this two dimensional problem what we consider that, these three forces must be concurrent forces. That is W, P a and R should pass through a single point. That is what we call concurrent forces.

So, by using the force polygon like this; W, P a and R. This W is known to us for a chosen failure surface because geometry is known. By knowing the unit weight of this soil, we can compute W because area times unit weight multiplied with unit length on that side will give us the weight of this failure zone. So W is known, also its direction of acting is known; because it acts always vertically downwards. So, this vector is completely known, this force vector in terms of its direction and magnitude; whereas, for this R we only know the direction, we do not know the value. We need to obtain that. For P a also we only know the direction, we do not know the value which we need to obtain.

So, what we can do? If we know the direction with respect to W, we know the line of action of this P s. So, we first draw this line, and with respect to W we know the line of action this R. So, we draw that line. So, wherever they intersect; that is, nothing but giving us from this force polygon, the value of P a, this will be the value of P a, value of R and their direction was already known. So, in that way we get the value of design value of active earth pressure.

Now by changing this failure angle rho, we will get different weight W and different values of P a and R combination. So through that, whichever gives us the maximum value of P a; so, we have to maximize this P a. So, maximum value of P a will give us the design value that we know. So now, what are the changes Mononobe-Okabe did with respect to this coulombs earth pressure approach? They mentioned, now they added two more forces. One is this one; another is this one. What are these extra forces? This is K h times W and this is K v times W. What are K h and K v? K h times g is nothing but horizontal seismic acceleration, and this K v times g is vertical seismic acceleration. As we know, this seismic acceleration acts in both the cycles. That is, it will act both vertically down as well as up. Also, this seismic acceleration will act towards left as well as towards right. So, for the analysis we need to take all the combinations. K h W acting in this direction, also in this direction; K v W also acting in this direction, also in this direction. And then considering these forces, equilibrium of all these forces; one need to obtain this active earth pressure again under earthquake condition. So, in that case it will be P a e. right.

So, there will be a critical value or critical combinations of this K h and K v, which will give us the value of this seismic active earth pressure. So with this, if we look at this slide over here, it says K h and K v; remember these are not the fixed direction. It can change. It is in one calculation you should take K h in this direction, another calculation you should take in this direction; K v in one combination in this direction, another combination in this direction. So, there will be four iterative methods; four combinations; K h K v in this form, K h K v in this form, K h K v in this form and K h K v in this form. With that, whatever optimized value or maximized value of P a e you will you get, that will give you the seismic active earth pressure using this Mononobe-Okabe pseudo static approach. Similarly, for passive earth pressure also you need to consider. It is also an extension of coulomb’s earth pressure theory for passive case.

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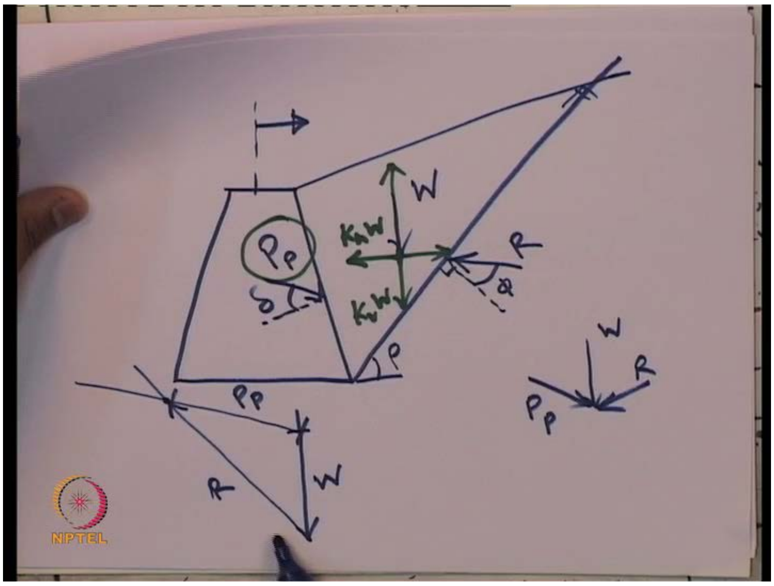

For passive case again, we know the changes of the coulombs theory like, if we consider the passive state of earth pressure; that means, the wall tends to move towards the back fill side. In that case, if we select a failure zone like this at an angle say rho, weight of this failure zone is W and this is normal to the wall. This will be P p passive earth pressure at an angle delta, friction angle, wall friction angle and soil reaction will be R at an angle phi with phi is the soil friction angle.

So, again these three forces should be concurrent, then it will be in equilibrium. So, equilibrium has to be maintained. That is, Coulomb's earth pressure theory. Here also additional parameters which were introduced by Mononobe-Okabe like this K h times W in the horizontal seismic inertia force and the vertical K v times W in vertical seismic inertia force.

Now considering these forces and maintaining equilibrium, in this case we have to… what we need to do? We need to minimize this P p value; because we know for passive earth pressure case, the design value is nothing but the minimum. Why? Because it is the resistance of the soil provided. So, as it is the resistance, it is… so, what we know in this case? W, we know the complete vector; that is, their magnitude and direction. Now, for line of action of R we know only the direction. For line of action of P p, we only know the direction. So where they intersect, that gives us the value of this R and P p. So, this value of P p what we get from the design by this equilibrium or closed force polygon that we have to minimize as I said with respect to different selection of this rho value, which is known. Now additional forces, these things come here and accordingly the analysis needs to be done or equilibrium has to be maintained.

But now, after talking about how to estimate this pseudo static active and passive earth pressure as proposed by Mononobe and Okabe, the major questions arises here like, what value of K h and K v should be used for the design. Because in the pseudo static approach we are just considering the coefficient, but after learning the Geotechnical Earthquake Engineering we all know this K h and K v is not at all constant over the depth of the soil layer. It varies with respect to depth; it varies with respect to time. It is a dynamic parameter. It is not a static or quasi static or pseudo static as it is assumed in this analysis. So, that is a big question always lies in the pseudo static approach, which can be considered as one of the major limitation of this method itself; inherent limitation.

Then, soil amplification; we already learnt that soil gets amplified when earth quake motion travels through soft soil. So, that soil amplification we cannot consider in this pseudo static approach. There is no scope to consider soil amplification in this design of pseudo static approach. Also, what is the variation of this seismic acceleration with respect to depth and with respect to time is, nowhere where we can consider these things. Another thing, what are the effects of these dynamic soil properties? whether irrespective of soil type, if we select just a single value of K h say 0.2 and K v say 0.1, will the design of the wall in clay soil, in sandy soil, in loose sand, in dense sand, everything remain same? As per pseudo static approach, it says yes. There is no other chance to take a… into all these considerations of soil parameters and their effect, dynamic nature, etcetera, in the pseudo static design.

So, these are the major limitations of this conventional pseudo static approach. Now, how to overcome this pseudo static approach? We will see soon. So, we will end our today’s lecture here. We will continue further in the next lecture.