Geotechnical Earthquake Engineering
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Module - 6
Lecture - 22
Dynamic Soil Properties (Contd…)

Let us start our today's lecture for NPTEL video course on geotechnical earthquake engineering. So, for this video course, currently we are discussing with module number 6, which is dynamic soil properties.

So, a quick recap of what we have learnt in our previous lecture. As I have already mentioned the basic reference for this course, I will refer to my another video course of NPTEL, that is on soil dynamics; module 4 of that course, where I have discussed in detail about this dynamic soil properties.

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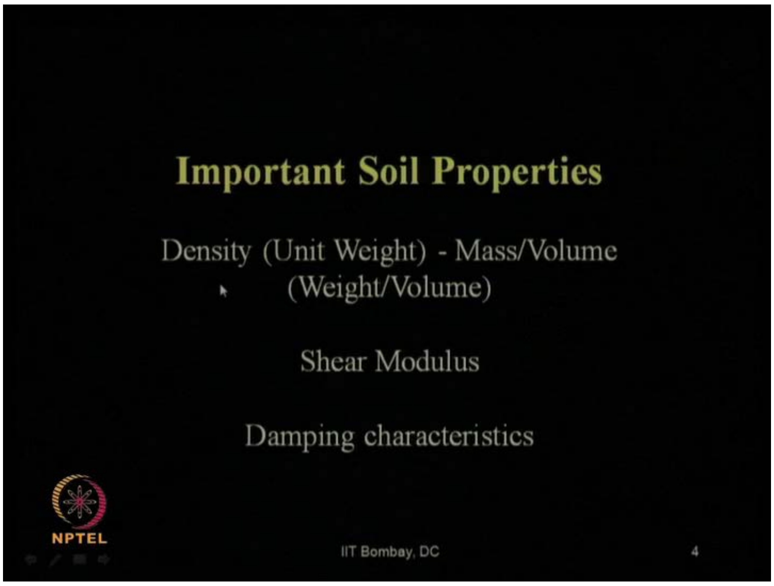

Now, in the previous lecture, we have discussed about what are the most important dynamic soil properties. In that, we need to know first about the density or unit weight of the soil then shear modulus, which is an important dynamic soil property and damping characteristics, another important dynamic soil property.

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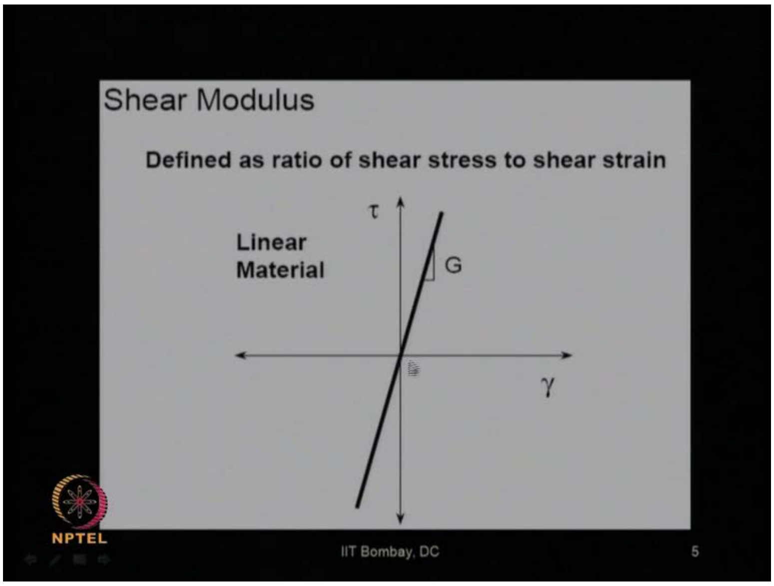

We have seen how shear modulus is defined. It is nothing but the behavior of any material, for our case it is the soil, when it is subjected to cyclic shear stress. So, this is the shear stress versus shear strain plot. The slope of that curve will give us the shear modulus. So, it is defined as the ratio of shear stress to shear strain as we all know.

For a linear material, that is a material which is behaving linearly, we will find out that this tau versus gamma or shear stress versus shear strain relationship is something linear like this. Hence, we will get a single value of this shear modulus from this curve.

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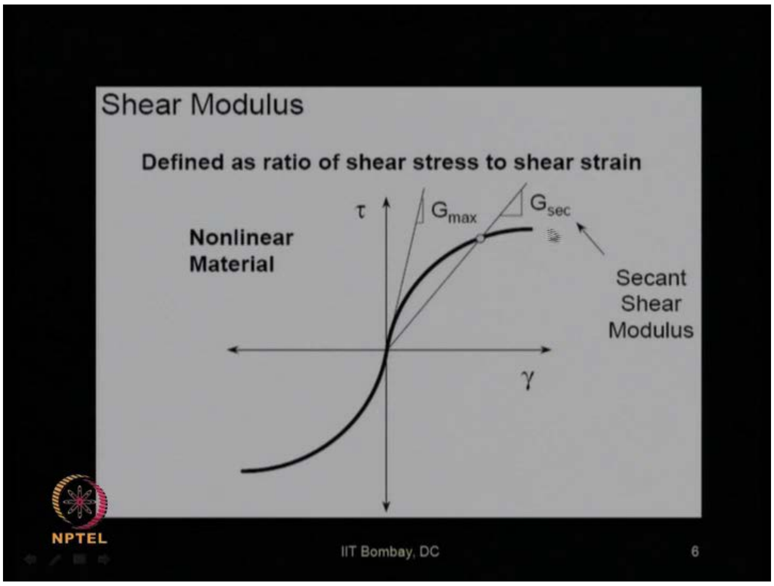

Whereas for actual material or most of the material like for soil, we will get it will behave like a non-linear material, where the shear stress versus shear strain of the material will behave nonlinearly like this. If we want to find out the slope of that curve, it will keep on changing at different point. The initial tangent of that will be considered as maximum shear modulus or G max. If we want to find out shear modulus at any higher shear strain value, then from that point, if we join to the initial point, then the slope of that line will give us the secant shear modulus.

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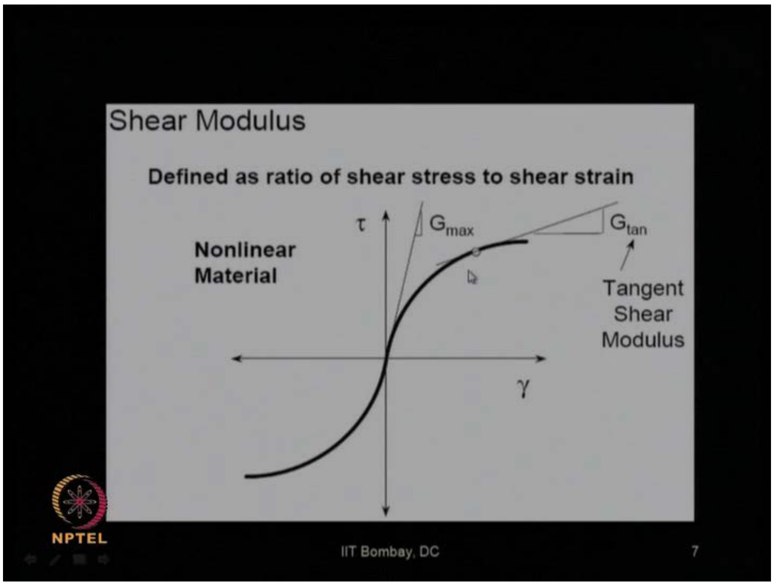

If we draw a tangent at that curvilinear portion of that material behavior, the slope of that line will give us the tangent shear modulus.

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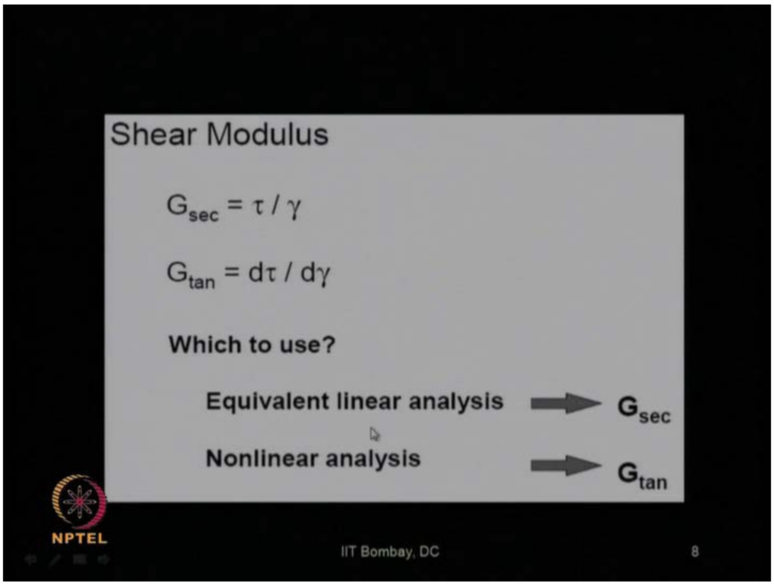

Now, where these three shear modulus are used. For equivalent linear analysis, we use G secant modulus. For linear analysis, of course we use G max and for non-linear analysis, we use the G tan.

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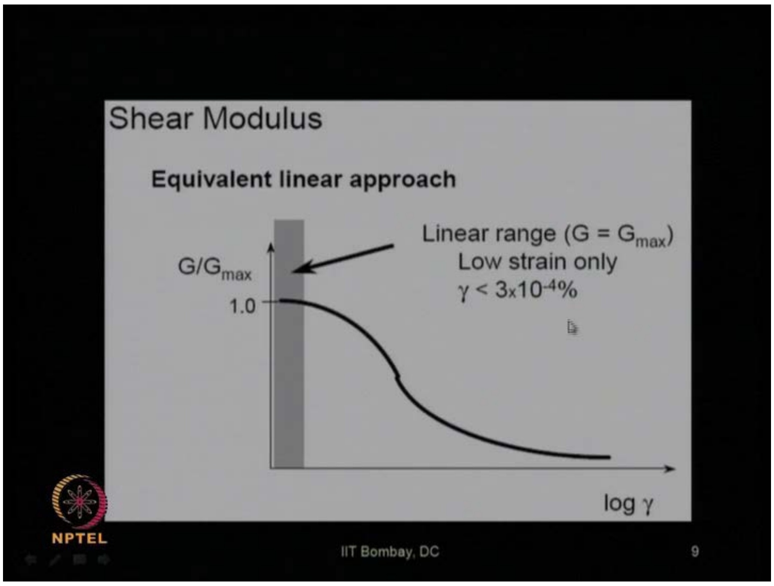

Then, we have seen how we carry out the equivalent linear approach. We generally define the material property in terms of G by G max and non-dimensional parameter in the y-axis. So, maximum value of this G by G max ratio will obviously be 1. As the shear strain increases, which is plotted in the x-axis in the log scale, the material degradation will show like this for soil. We will say that it is almost close to 1, G by G max value for very low shear strain. That is called the linear range, where G equals to G max for low strain. For that, the low strain range should be within 3 into 10 to the power minus 4 percent.

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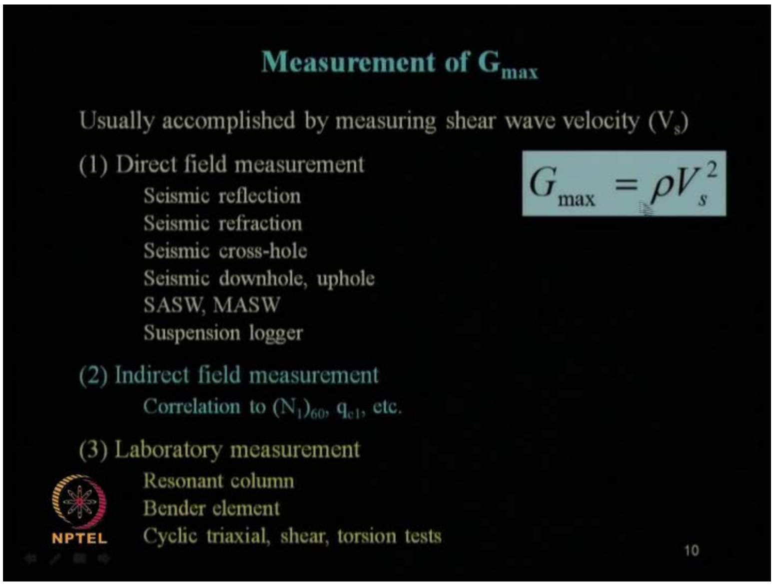

Now, how to find out this G max value or maximum value of shear modulus? The measurement involves basically three major methods. One is direct field measurement by using these techniques or indirect field measurement by using this technique or laboratory measurement using these techniques. Also from the shear wave velocity value, that is V s value, we can calculate the value of G max using this relationship, using this density of the material or density of the soil as rho.

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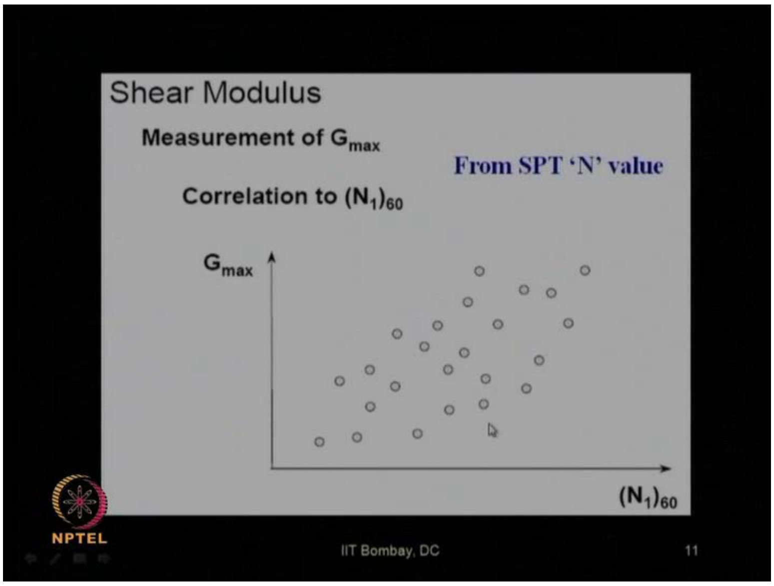

Now, coming to the various relationships of this G value with respect to SPT N value, that is standard penetration test N value. Several researchers has obtained correlation between the corrected SPT N value, which is N 1 60 with respect to G max.

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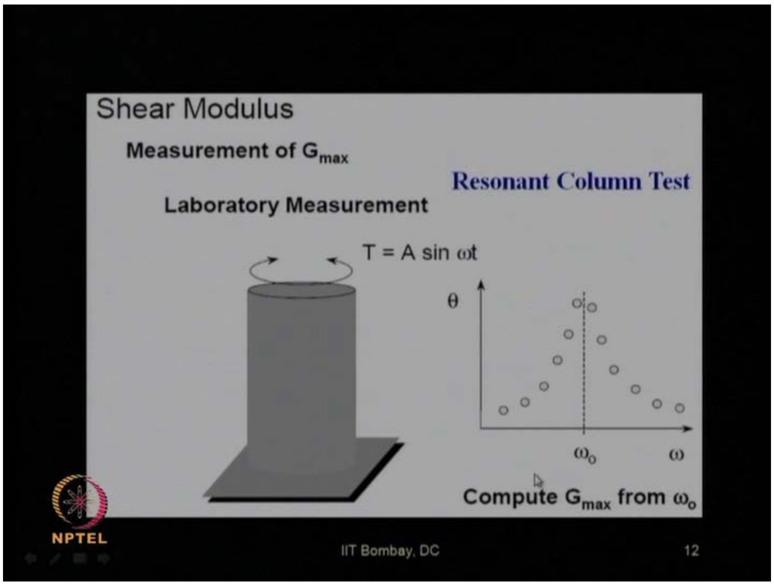

Also in the laboratory, one can find out using resonant column test like this. That is, what is the value of G max at the resonant frequency from this plot of the distortion at different frequency on the resonant column with the torque applied on it. Also from cyclic triaxial test, one can find out the G max value.

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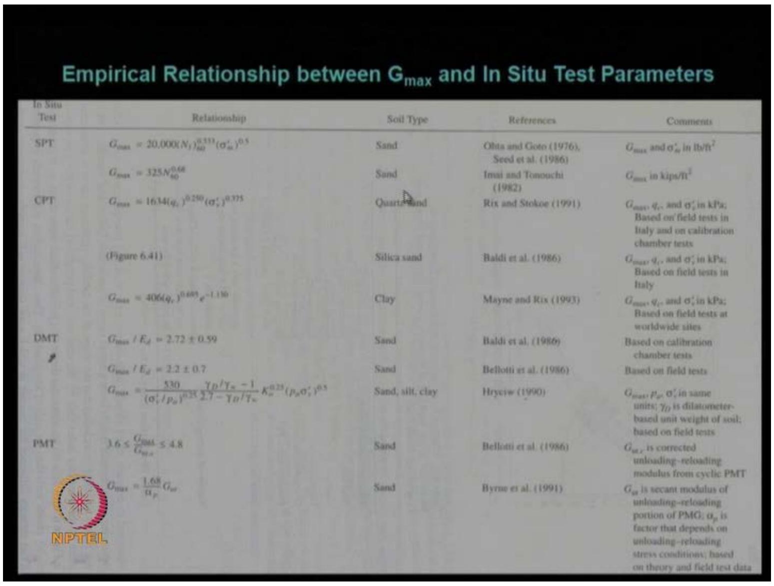

In Situ Test	Relationship	Soil Type	References	Comments
SPT	$G_{max} = 20{,}000(N_1)_{60}^{0.333}(\sigma'_m)^{0.5}$	Sand	Ohta and Goto (1976), Seed et al. (1986)	G_{max} and σ'_m in lb/ft^2
	$G_{max} = 325N_{60}^{0.68}$	Sand	Imai and Tonouchi (1982)	G_{max} in kips/ft^2
CPT	$G_{max} = 1634(q_c)^{0.250}(\sigma'_v)^{0.375}$	Quartz sand	Rix and Stokoe (1991)	G_{max}, q_c, and σ'_v in kPa; Based on field tests in Italy and on calibration chamber tests
	(Figure 6.41)	Silica sand	Baldi et al. (1986)	G_{max}, q_c, and σ'_v in kPa; Based on field tests in Italy
	$G_{max} = 406(q_c)^{0.695}e^{-1.130}$	Clay	Mayne and Rix (1993)	G_{max}, q_c, and σ'_v in kPa; Based on field tests at worldwide sites
DMT	$G_{max} / E_d = 2.72 \pm 0.59$	Sand	Baldi et al. (1986)	Based on calibration chamber tests
	$G_{max} / E_d = 2.2 \pm 0.7$	Sand	Bellotti et al. (1986)	Based on field tests
	$G_{max} = \frac{530}{(\sigma'_v / p_a)^{0.25}} \frac{\gamma_D/\gamma_w - 1}{2.7 - \gamma_D/\gamma_w} K_o^{0.25}(p_a\sigma'_v)^{0.5}$	Sand, silt, clay	Hryciw (1990)	G_{max}, p_a, σ'_v in same units; γ_D is dilatometer-based unit weight of soil; based on field tests
PMT	$3.6 \le \frac{G_{max}}{G_{ur,c}} \le 4.8$	Sand	Bellotti et al. (1986)	$G_{ur,c}$ is corrected unloading-reloading modulus from cyclic PMT
	$G_{max} = \frac{1.68}{\alpha_p} G_{ur}$	Sand	Byrne et al. (1991)	G_{ur} is secant modulus of unloading-reloading portion of PMG; α_p is factor that depends on unloading-reloading stress conditions; based on theory and field test data

Another way to obtain the G max value is from in situ test parameters using various empirical relationships, which are available worldwide. Like SPT N value, CPT cone penetration value, DMT dilatometer test value and PMT pressure meter test value. So, from each of these tests, we can get these empirical relationships for different types of soil as proposed by various researchers worldwide. But we need to remember that these relationships are developed for limited number of dataset point and also for a typical soil of that region.

So, acceptability of these empirical relationships always needs to be checked, when somebody is trying to do a rigorous analysis for obtaining the G max value and further use of that G max for dynamic analysis.

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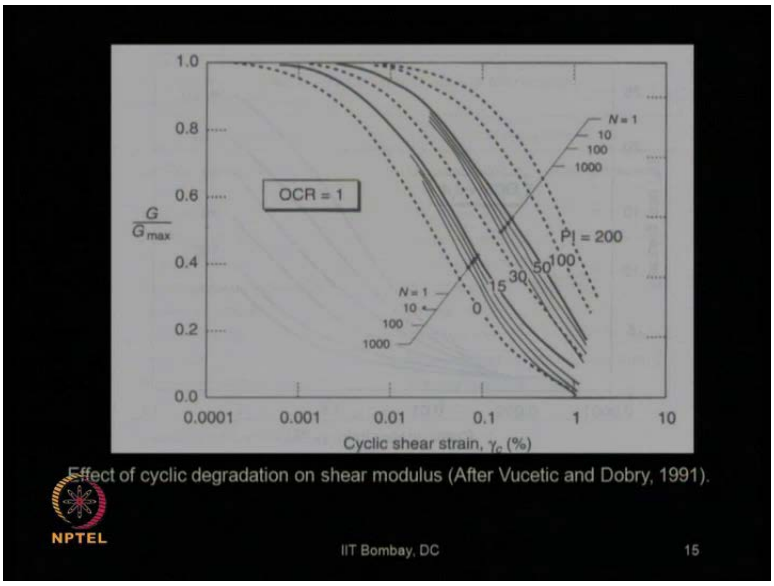

The variation of this G by G max with respect to cyclic shear strain for different values of plasticity index was reported by Vucetic and Dobryin in 1991, which shows like this type of variation and also with respect to the number of cycles. That is, if number of cycle increases, then there will be a change in the G by G max value. Obviously, it is going to decrease as it can be seen from here.

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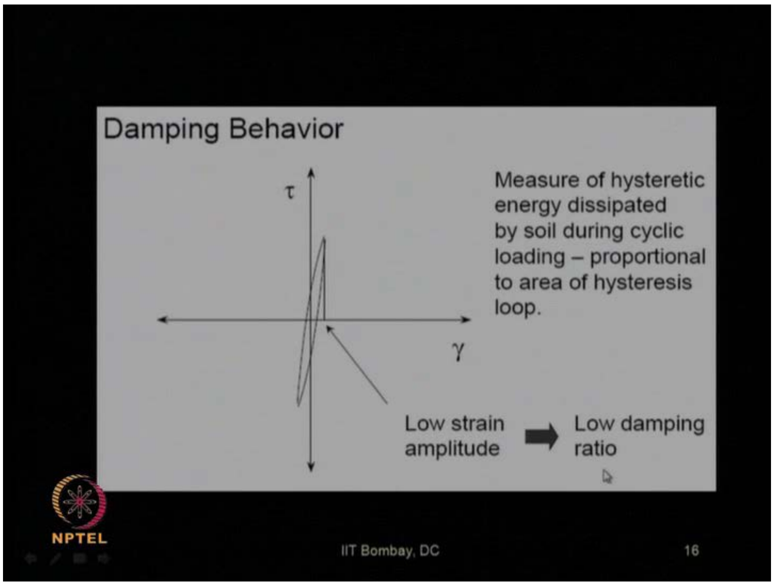

Another important parameter damping behavior, which we have discussed in our previous lecture like, at low strain, we will expect low damping ratio, whereas at high strain, we will get high damping ratio value.

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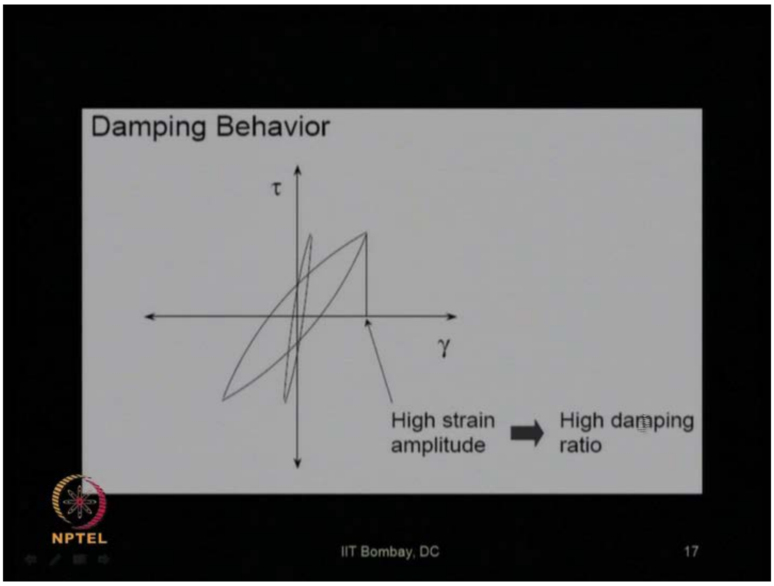

Variation of damping ratio with respect to cyclic shear strain for a typical soil material will be something like this. This result is also given by Vucetic and Dobry in 1991 for different values of plasticity index for fine grained soils.

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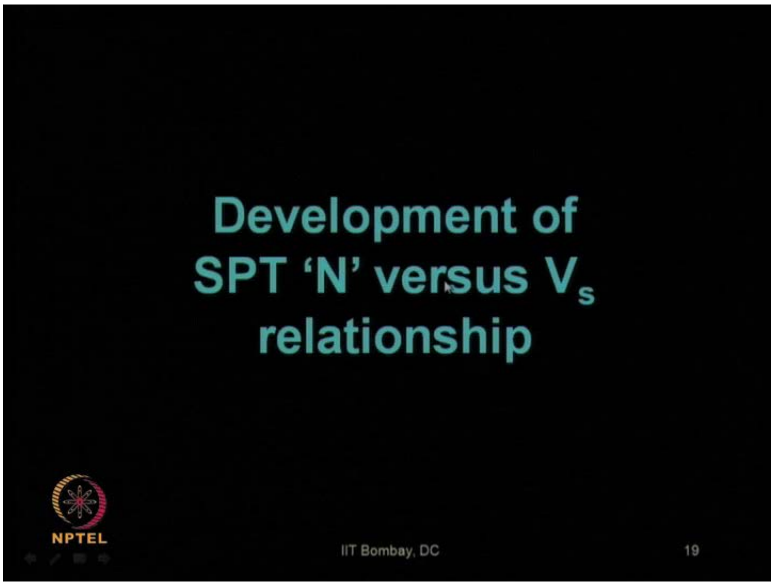

Now worldwide, there are developments on the SPT N value versus the shear wave velocity relationship because many times people cannot obtain the dynamic shear wave velocity at the soil site or field. So, instead of that, people can reasonably use from the static test which is nothing but static penetration test SPT N value. The value of V s can further be used to compute the G max value and for further dynamic design.

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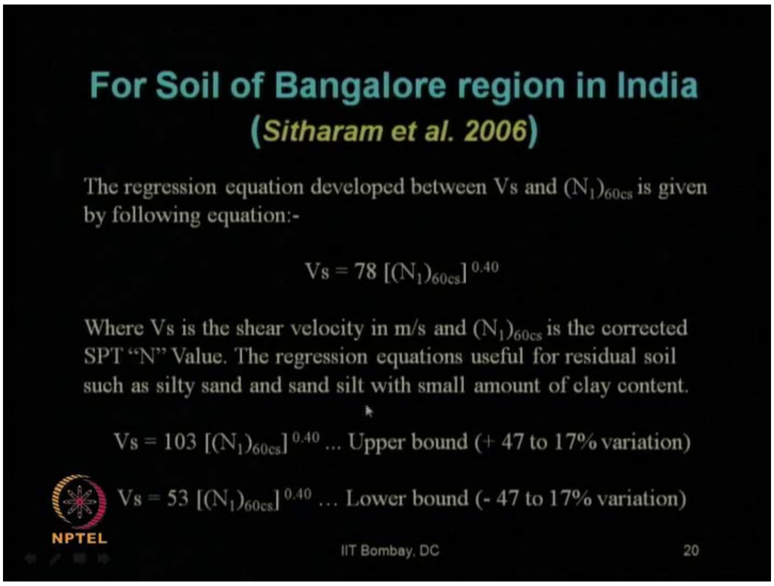

So, how it is done? The example we had discussed in our previous lecture for soil of Bangalore region in India, Sitharam et al in 2006, proposed this empirical relationship V s in terms of the corrected SPT N 1 60 value and also corrected with respect to clean sand. So N 1 60 c s, in terms of that, how one can obtain the V s value in the absence of the actual data obtained through field test for the V s value. Also for other types of residual soil and silty sand and sand silt, the upper bound and lower bound equations were proposed by these researchers.

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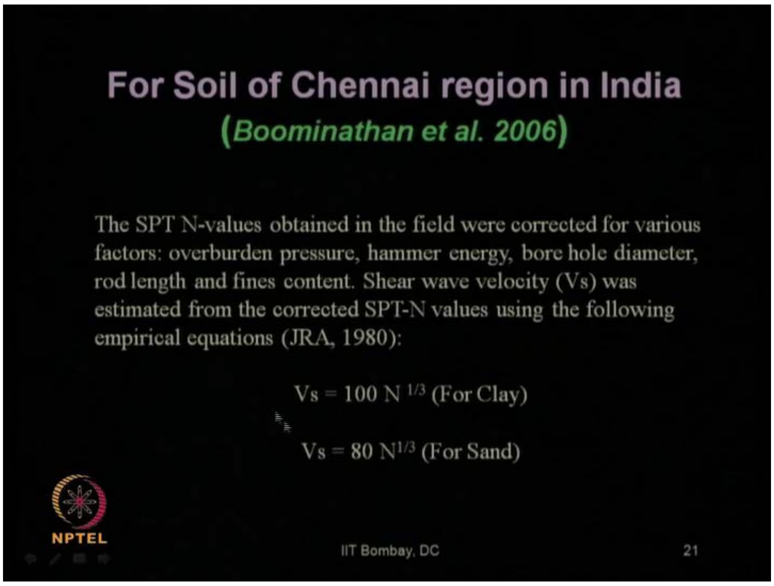

Another group of researchers had proposed for soil of Chennai region in India, Boominathan et al in 2006, for clay soil and for sandy soil, the relationship between V s and corrected SPT N value.

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Then, the application of research for dynamic soil characterization of Mumbai city, we had started in our previous lecture. Reference for that, I have mentioned Sumedh Y.Mhaske's PhD thesis. Dr.Sumedh Mhaske has completed his PhD in 2011 at IIT Bombay under my supervision.

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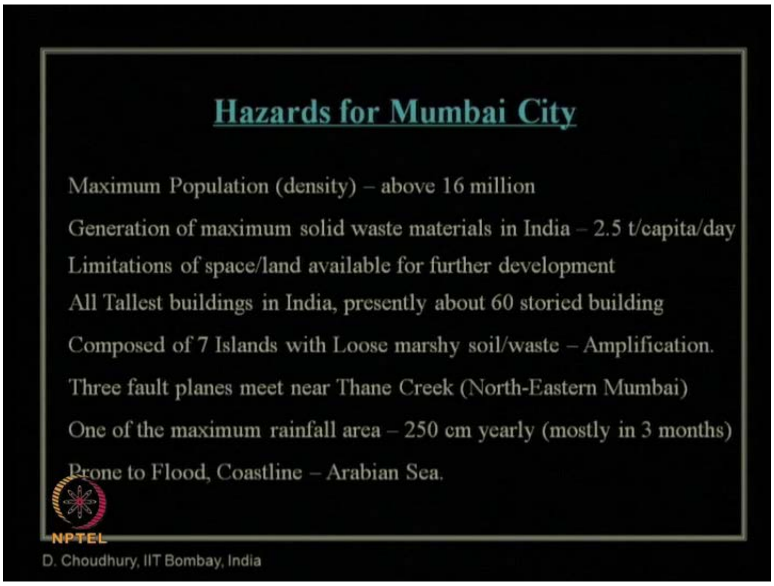

So, first we had discussed what are the various hazards for Mumbai city, that shows the need of doing the study of geotechnical earthquake engineering for Mumbai city.

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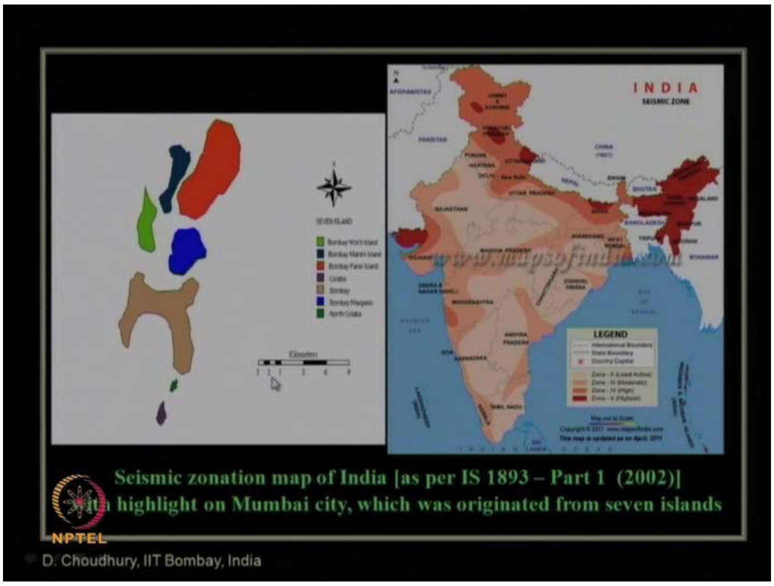

This is the seismic zonation map of India as per IS 1893 part 1 of 2002 version, which places Mumbai city in zone 3. These are the seven original islands of Mumbai city, which has been combined together by land filling etcetera over a period of time.

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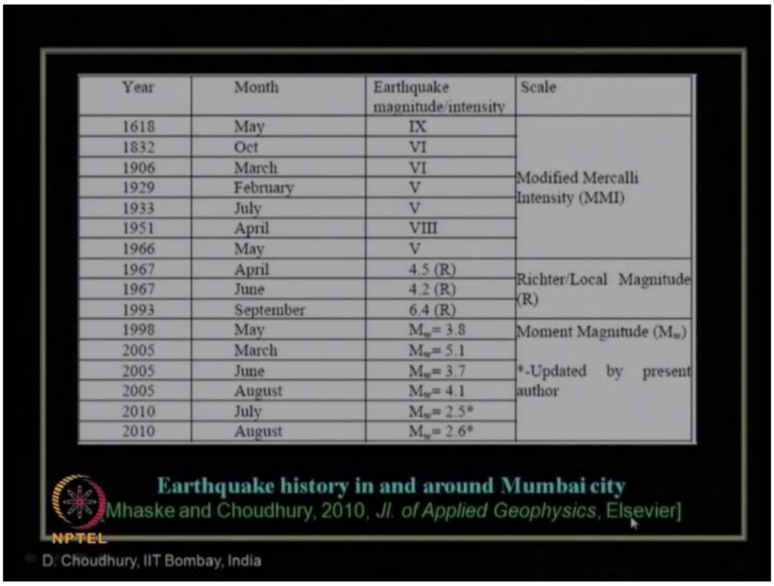

Year	Month	Earthquake magnitude/intensity	Scale
1618	May	IX	Modified Mercalli Intensity (MMI)
1832	Oct	VI	
1906	March	VI	
1929	February	V	
1933	July	V	
1951	April	VIII	
1966	May	V	
1967	April	4.5 (R)	Richter/Local Magnitude (R)
1967	June	4.2 (R)	
1993	September	6.4 (R)	
1998	May	M_w= 3.8	Moment Magnitude (M_w)
2005	March	M_w= 5.1	
2005	June	M_w= 3.7	*-Updated by present author
2005	August	M_w= 4.1	
2010	July	M_w= 2.5*	
2010	August	M_w= 2.6*	

This lists the earthquake history in and around Mumbai city, which has been reported in this journal paper by Mhaske and Choudhury, 2010, journal of applied geophysics, Elsevier.

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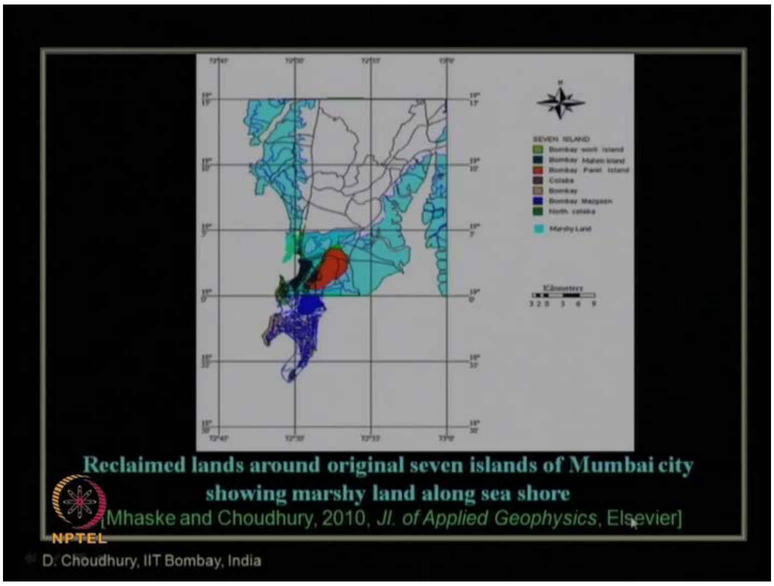

This GIS based map shows the original seven islands of Mumbai and the surrounding marshy land that is reclaimed land around Mumbai city, which actually forms today's Mumbai city.

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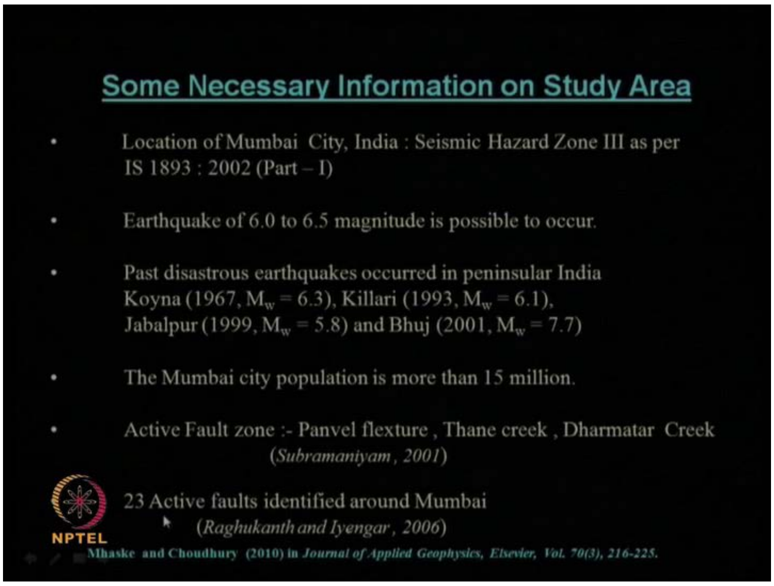

We will see some other necessary information to study the Mumbai city area for this seismic hazard analysis. First of all, location of the Mumbai city is in seismic zone 3 as per IS 1893, 2002 version part 1. So, earthquake of 6 to 6.5 intensity or magnitude is possible to occur. Past disastrous earthquake occurred in the peninsular India is not exactly in Mumbai, but in peninsular India and Mumbai is also a part of that. The population in Mumbai city is more than 15 million as per the census of 2011.

Active fault zone are present close to Mumbai like Panvel flexture, Thane creek and Dharmatar creek as mentioned by these researchers. Also, there are various 23 small active faults in and around Mumbai city as mentioned by Raghukanth and Iyengar in 2006.

So, with these details, we had completed our previous lecture. So, let us see how to take it forward today in our present lecture. Now, when we want to do any dynamic soil properties study on that for Mumbai city, first we should know Mumbai's typical soil property.

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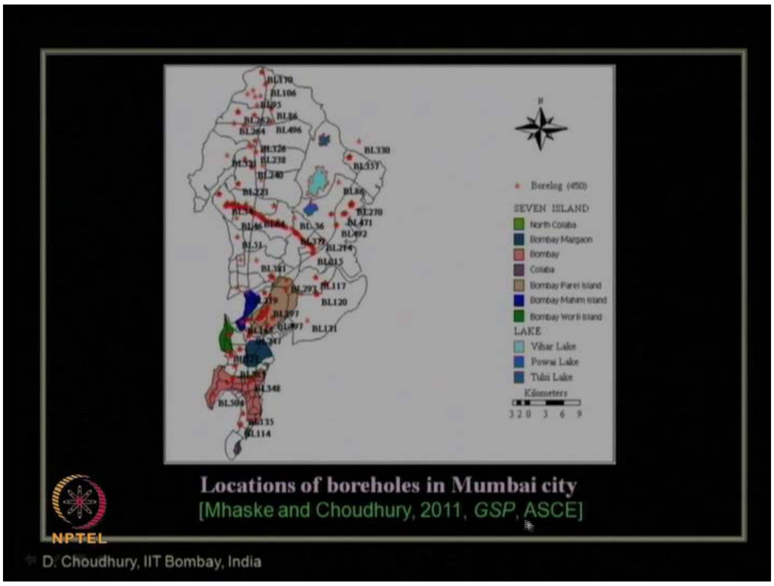

So, to find out the typical soil property, you can see in this slide. For the entire Mumbai region, various numbers of borehole data has been collected from reliable government and private agencies, so that we get the soil profile of entire Mumbai city in this fashion. This GIS based map shows the locations of boreholes, which are collected all around Mumbai city, which is available in the paper of Mhaske and Choudhury, 2011, geotechnical special publication of ASCE.

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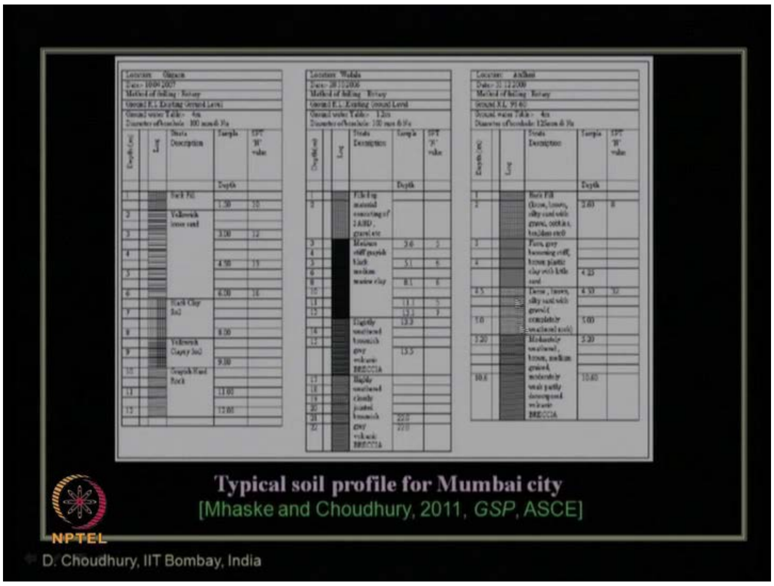

This is the typical soil profile for Mumbai city at various locations like Gurgaon, Wadala Andheri, and etcetera.

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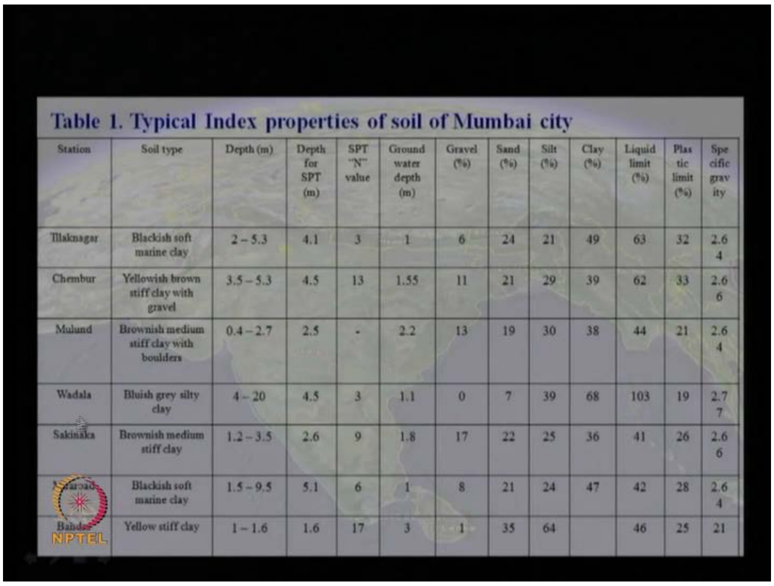

Table 1. Typical Index properties of soil of Mumbai city

Station	Soil type	Depth (m)	Depth for SPT (m)	SPT "N" value	Ground water depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Liquid limit (%)	Plastic limit (%)	Specific gravity
Tilaknagar	Blackish soft marine clay	2 – 5.3	4.1	3	1	6	24	21	49	63	32	2.64
Chembur	Yellowish brown stiff clay with gravel	3.5 – 5.3	4.5	13	1.55	11	21	29	39	62	33	2.66
Mulund	Brownish medium stiff clay with boulders	0.4 – 2.7	2.5	-	2.2	13	19	30	38	44	21	2.64
Wadala	Bluish grey silty clay	4 – 20	4.5	3	1.1	0	7	39	68	103	19	2.77
Sakinaka	Brownish medium stiff clay	1.2 – 3.5	2.6	9	1.8	17	22	25	36	41	26	2.66
[illegible]aroad	Blackish soft marine clay	1.5 – 9.5	5.1	6	1	8	21	24	47	42	28	2.64
Band[illegible]	Yellow stiff clay	1 – 1.6	1.6	17	3	1	35	64		46	25	21

This table shows the various stations, borehole stations, various soil types, their depth, SPT N value corrected, originally recorded, corrected, ground water depth and various other soil parameters like amount of gravel, sand, silt, clay, liquid limit, plastic limit, and specific gravity. So, all these information were collected from reliable and authentic borehole data from various locations like Tilaknagar, Chembur, Mulund, Walada and etcetera.

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This chart shows the worldwide used correlations available for SPT N value versus the shear wave velocity V s value. For all types of soil, Obba and Torimani, 1970, proposed this equation of V s versus N. Several other researchers also, as shown in this table, have mentioned different equations of V s versus N as reported over here, which are used worldwide extensively. The details are available in this paper by Mhaske and Choudhury, 2011, in the journal Natural Hazards published by Springer.

So in this, you should again remember that these empirical relationships were developed based on the soils collected from a particular region and also it is based on certain number of data set points. So, application of these equations for any particular region, one has to be very careful whether that type of soil exists at the same area or not or whether the soil behavior at the location is similar or not. If not, then obviously, this relationship should not be used and a new relationship is required to be developed for that particular area.

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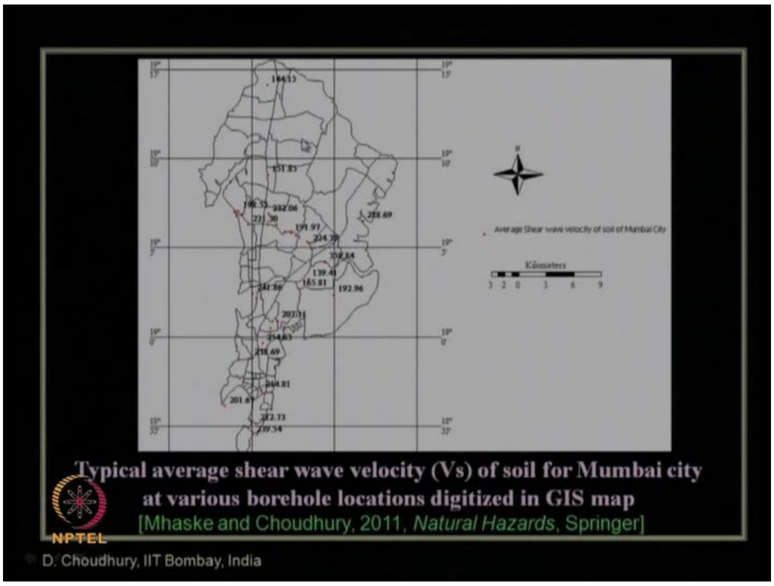

So now, let us look here that when we want to develop some particular relationship of that SPT N value versus the shear wave velocity V s for Mumbai region from the collected borehole data, you can see that these are the typical average shear wave values all around Mumbai city at various borehole locations, which are digitized in the GIS map. These details are available in this paper.

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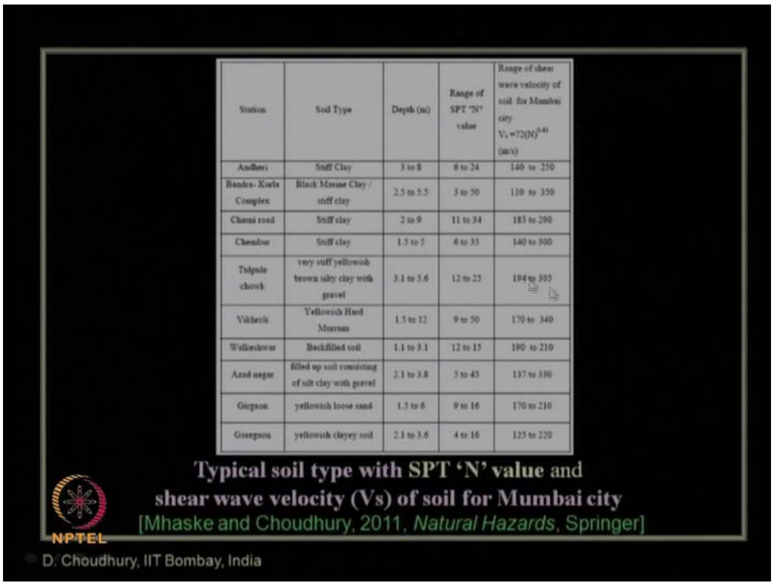

Station	Soil Type	Depth (m)	Range of SPT "N" value	Range of shear wave velocity of soil for Mumbai city $V_s = 72(N)^{0.40}$ (m/s)
Andheri	Stiff Clay	3 to 8	6 to 24	140 to 250
Bandra- Kurla Complex	Black Marine Clay / stiff clay	2.5 to 5.5	3 to 50	110 to 350
Charni road	Stiff clay	2 to 9	11 to 34	185 to 290
Chembur	Stiff clay	1.5 to 5	6 to 35	140 to 300
Tulpule chowk	very stiff yellowish brown silty clay with gravel	3.1 to 5.6	12 to 25	194 to 305
Vikhroli	Yellowish Hard Murrum	1.5 to 12	9 to 50	170 to 340
Walkeshwar	Backfilled soil	1.1 to 3.1	12 to 15	190 to 210
Azad nagar	filled up soil consisting of silt clay with gravel	2.1 to 3.8	5 to 45	137 to 330
Girgaon	yellowish loose sand	1.5 to 6	9 to 16	170 to 210
Goregaon	yellowish clayey soil	2.1 to 3.6	4 to 16	125 to 220

These stations, soil type, depth and SPT N value are also shown over here. This is the equation which has been developed for Mumbai city by Mhaske for his PhD work under my supervision. These are the ranges of values of SPT N value and corresponding shear wave velocity value in and around Mumbai at different locations.

So, one can easily see that it is not in the similar range in all the places. It depends on location to location. It varies from location to location. So, one needs to be very careful when somebody is planning to do any earthquake engineering study or analysis or design at different locations of Mumbai, incorporating this dynamic soil properties.

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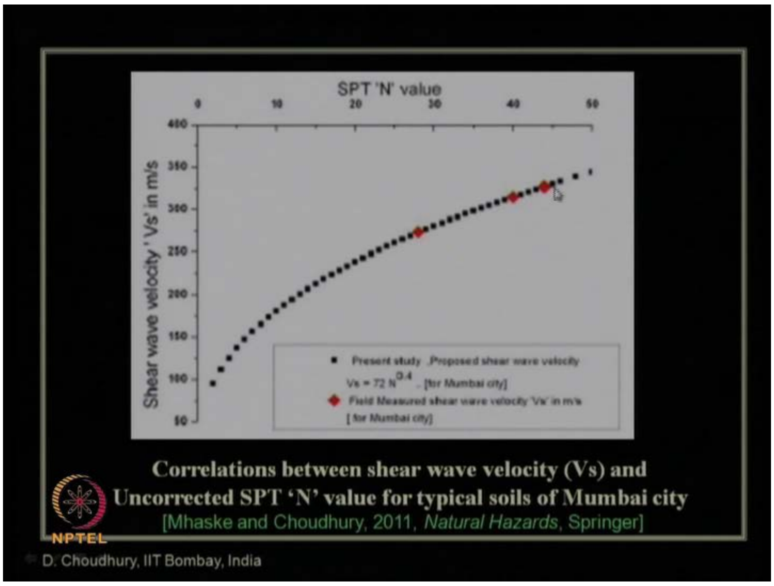

This result shows the correlation between shear wave velocity in the unit meter per second versus SPT N value. This is uncorrected SPT N value and it may be noted. So, from the present study, this is the observed and proposed equation. That is, V s equals to 72 N to the power 0.4 for entire Mumbai city soil. Field observed or field measured, actual shear wave velocity at three different locations are obtained at different values of SPT N value and corresponding V s values are plotted, which are matching very well with the proposed entire region in Mumbai soil.

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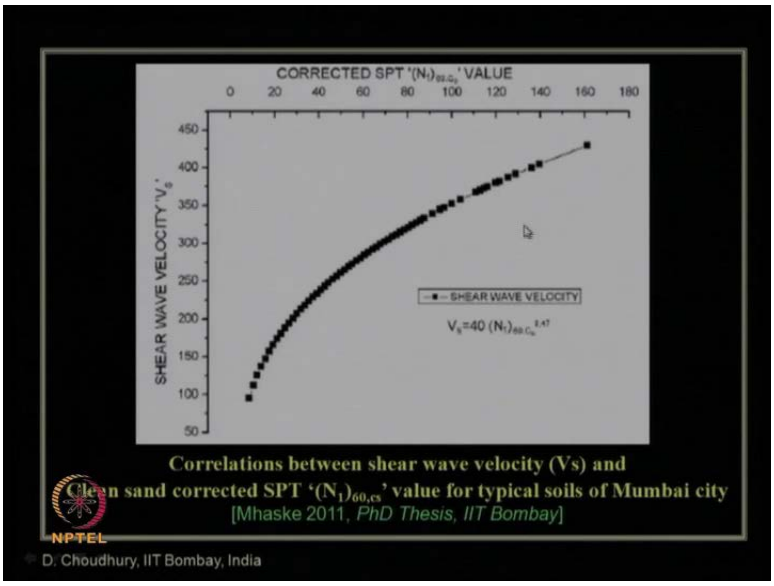

This result shows the correlations between shear wave velocity V s in meter per second unit versus the clean sand corrected SPT N 1 60 value. So, this is clean sand SPT N value. You can see, the equation proposed here is V s equals to 40 N 1 60 c s to the power 0.47. So, there is a minor change from the uncorrected SPT N value to the corrected SPT N value. This is observed from Dr.Mhaske's PhD thesis as reported over here. It was concluded from this study that there is hardly any large variation or significant variation between the use of original raw SPT N value or corrected clean sand SPT N value. The same was concluded by previous other researchers. Why? Because there are several uncertainties involved in the SPT N value corrections also.

So, instead of adding up the uncertainties, it is also suggested that, for basic study or first step of study or design, it is better to use the basic equation of V s equals to 72 N to the power 0.4, for the Mumbai city soil from the uncorrected SPT N value, where it will give reasonably correct result for the shear wave velocity. It is also verified and authenticated from the field measurements as shown in this slide.

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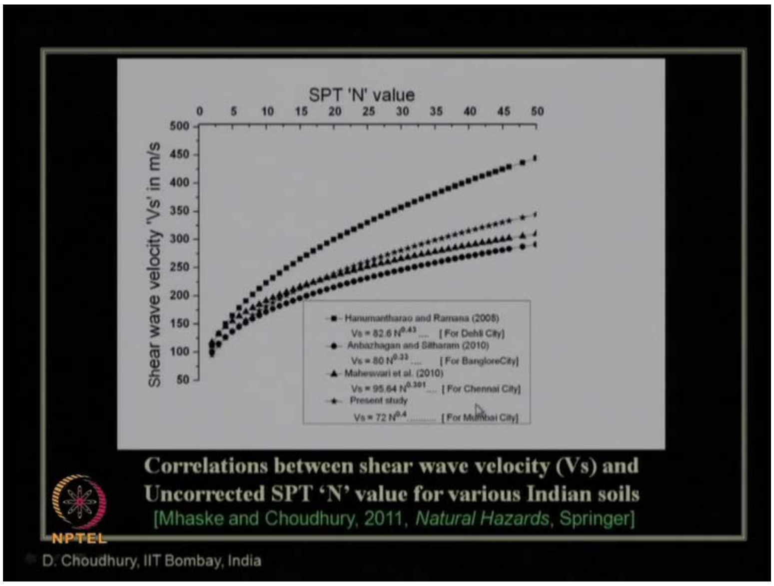

Now, let us see the comparison of various Indian soils. The different correlationships available between shear wave velocity v s and uncorrected SPT N value, as reported by four researchers group for four different cities as on today, it is available like this. That is, shear wave velocity V s in meter per second unit in the y-axis, and in x- axis, it is SPT N value uncorrected. This line, the top one, shows the results for Delhi city, which is given by Anumantrao and Ramana in 2008. The researchers from IIT Delhi, they have developed this equation V s equals to 82.6 N to the power 0.43. For Delhi city, this is the equation proposed to be used. Whereas, Anbazagan and Sitharam in 2010, this lower most line with the circle symbol, they proposed for the Bangalore city, that is V s equals 80 N to the power 0.33. This is the proposed equation for the soil of Bangalore city.

Maheshwari et al in 2010, as shown by these dark triangles, which is for Chennai city V s equals to 95.64 N to the power 0.301 is the proposed equation for the soil of Chennai city. For Mumbai city, the present study shows the results, which is star marked over here. This one, that is V s equals to 72 N to the power 0.4 is the proposed correlation for Mumbai city soil. That is, from SPT N value, how to calculate the shear wave velocity.

So, it can be seen that for these four major cities in India, like Delhi city, Bangalore city, Chennai city and Mumbai city, these are the corresponding proposed correlations between SPT N value and the shear wave velocity V s value, which will finally help to compute the maximum shear modulus G max value from this V s value for that particular region of soil. That will be finally used for the seismic design of any structures in that locality.

In similar fashion, it is today’s necessity that for most of the seismically active region or important locations, where major construction is required or proposed to be carried out, the seismic design or earthquake resistant design is necessary to find out this kind of relationship of V s versus SPT N. Since, at many places, we will not be able to carry out the shear wave velocity test at field due to several reasons.

One of that as I have mentioned in previous lecture is the presence of several obstruction. If obstruction is present, many times SASW MASW results will give us the wrong result because it is not giving the result of the that V s value of that particular soil. It is giving a result of shear wave velocity passing through composite material. That is, whatever structure or hidden objects are present in the soil, that material including the soil, does not capture the exact value of the stiffness or shear wave velocity etcetera of that particular soil. So, that is the reason why it is proposed worldwide and also in India. It is today’s need to find out this kind of V s versus N relationship for most of the important cities and locations and the seismically active regions.

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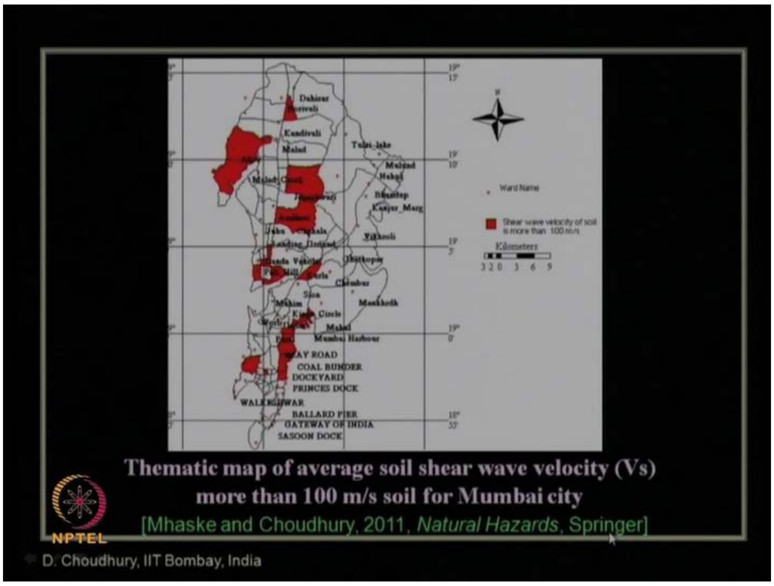

This GIS based map shows the thematic map of average soil shear wave velocity, more than 100 meter per second for the Mumbai city. You can see over here, all these red colored patches are nothing but those regions where shear wave velocity, average shear wave velocity to a particular depth is greater than 100 meter per second. So, one can say that these are relatively stiffer soil as compared to other regions. These details are available in Mhaske and Choudhury, 2011 Natural Hazards journal paper in Springer.

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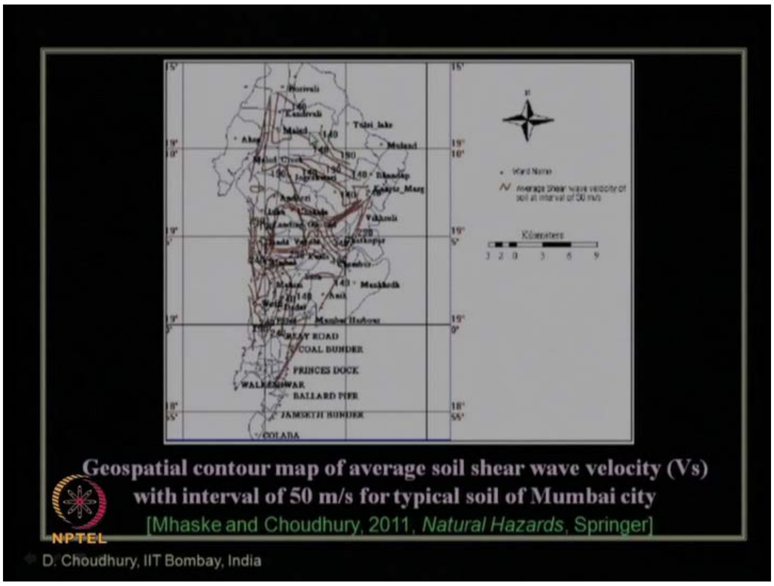

This GIS map shows the geospatial contour map of average soil shear wave velocity V s with interval of 50 meter per second for typical soil of Mumbai city. So, with 50 meter per second interval, the shear wave velocity values are plotted and shown over here like 140, 190, 240, 290 etcetera are shown over here. It is useful, why because if somebody is planning to do any construction at any region, say at Malad region, then they know at this location, typically, remember this is a typical representation; it may vary within that location also. But typically, the range of shear wave velocity will be within this value whatever is mapped over here. So, these details are very much useful for practicing engineers and design engineer to further carryout earthquake resistant design or earthquake related design at that site.

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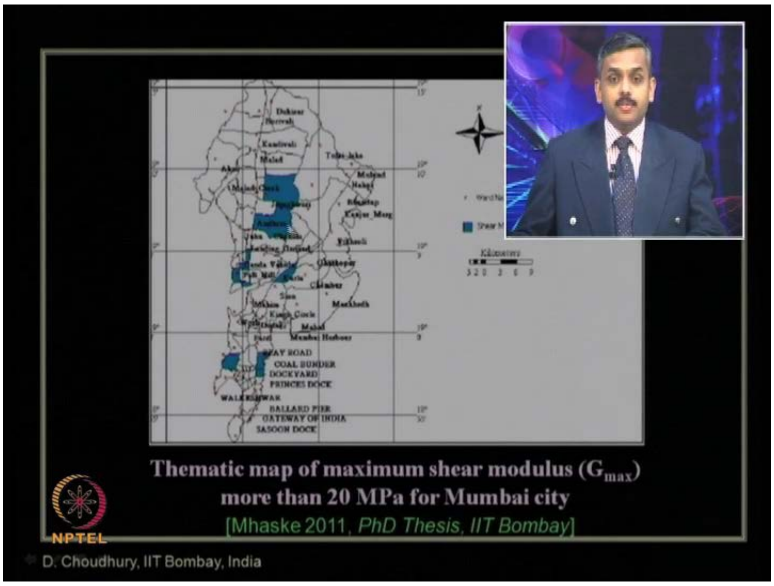

This GIS based map shows the value of maximum shear modulus, G max value, where it is more than 20 MPa for Mumbai city. How it is obtained? From the values of V s, the G max value can be easily computed by knowing the density of the soil. So through that, the G max of more than 20 MPa, those locations are marked over here. That means, these are relatively stiffer soil zone.

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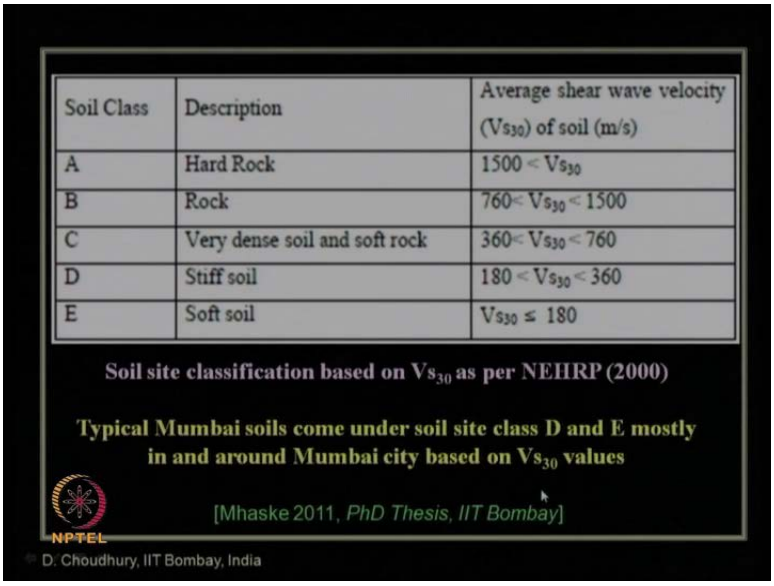

Soil Class	Description	Average shear wave velocity (Vs_{30}) of soil (m/s)
A	Hard Rock	$1500 < Vs_{30}$
B	Rock	$760 < Vs_{30} < 1500$
C	Very dense soil and soft rock	$360 < Vs_{30} < 760$
D	Stiff soil	$180 < Vs_{30} < 360$
E	Soft soil	$Vs_{30} \leq 180$

Now, this table shows the classification of the soil site into 5 different categories like soil class A, B, C, D, and E with their descriptions as hard rock, rock, very dense soil and soft rock, stiff soil and soft soil based on the dynamic property of the soil, which is expressed in terms of average shear wave velocity V s 30. What is 30? 30 indicate the average shear wave velocity up to 30 meter depth from the ground surface. So, that is why this number 30 came here. So, V s 30 of the soil in the unit meter per second, that value is used to classify the soil into different classes.

This soil site classification is based on as per NEHRP, standard of 2000 which is nothing but codal guide line or provision as mentioned in USA and practiced worldwide. So, for soft soil, V s 30 value will be less than or equal to 180 meter per second. For stiff soil, it is in between 180 to 360 meter per second. For very dense soil, it is in between 360 to 760 meter per second and so on. So, for Mumbai soil, a study has been carried out by Dr.Mhaske for his PhD thesis at IIT Bombay. It has been observed that most of the soil of Mumbai region comes under soil class D or E. That means, stiff soil and soft soil because their V s 30 value was found out to be within the range of 100 to 360 meter per second as mentioned over here.

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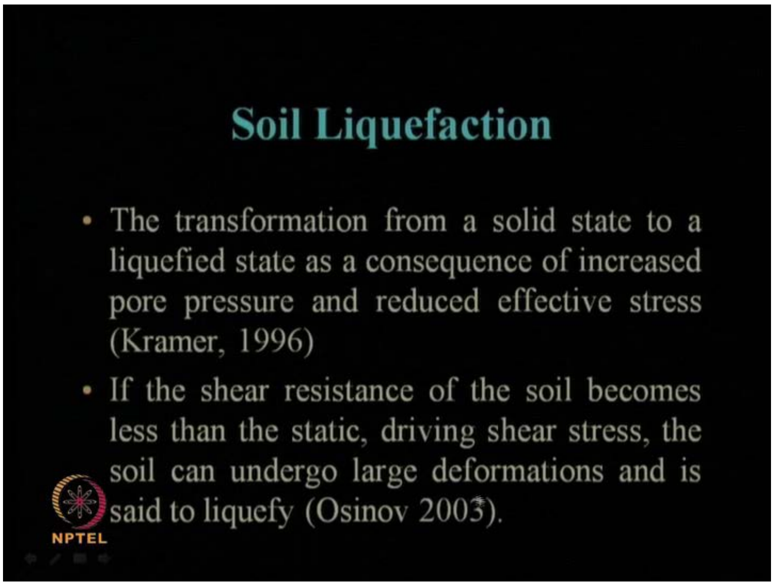

Now, coming to another important sub topic, that is soil liquefaction. First, I will mention that for a detailed basic understanding of soil liquefaction, one should refer and listen to my another video course on Soil Dynamics which is developed for this NPTEL course once again. In that Soil Dynamics video course, module number 4 discusses about this soil liquefaction.

What is soil liquefaction? As mentioned in this book by Kramer in 1996, it is nothing but the transformation from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress for soil. As mentioned by Osinov in 2003, if the shear resistance of the soil becomes less than the static, driving shear stress, the soil can undergo large deformations and is said to liquefy.

So, in the state of soil liquefaction, soil liquefaction can occur due to several dynamic loads. Earthquake, is of course one of the reason. So, even due to earthquake liquefaction can occur.

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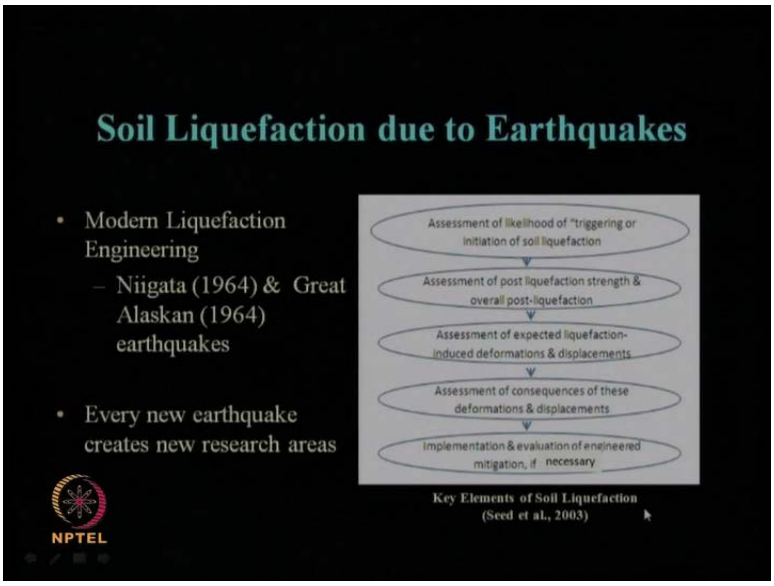

Now, let us see soil liquefaction due to earthquake. Modern liquefaction engineering is developed after Niigata 1964 earthquake and great Alaskan earthquake of 1964. After that, the research etcetera extensively started in this area of soil liquefaction. So, key elements of soil liquefaction as proposed by Seed et al in 2003 are mentioned over here. We should always remember that every new earthquake, whenever another earthquake comes, it creates a new research area because we get lots of more data and lots of understanding about how the soil behave during and after the earthquake.

So, all those information like whether it got liquefied, or if it has been liquefied, what type of soil was there, and if it has not been liquefied during an magnitude of earthquake, why it has not liquefied, what are the characteristics of the soil and then try to correlate between various physical and engineering soil parameters with respect to the liquefaction estimation and so on.

So, this assessment of likelihood of triggering or initiation of soil liquefaction needs to be carried out. Then assessment of post liquefaction strength and overall post liquefaction needs to be studied because after the liquefaction, how much strength of the soil is remaining and whether it can be further used for construction of structure or not are important to know assessment of expected liquefaction, induced deformations and displacements, assessment of consequences of these deformations and displacement. And then finally the implementation and evaluation of engineered mitigation; if necessary at that site.

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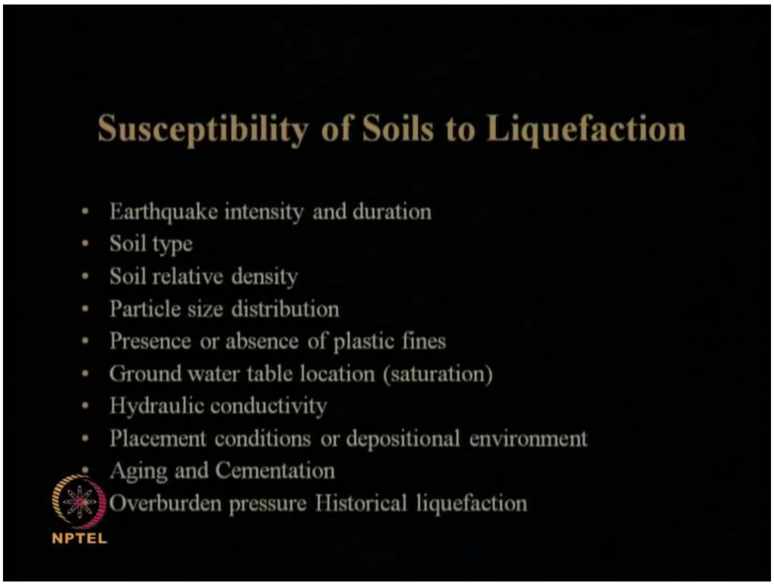

Now, susceptibility of the soil to earthquake induced liquefaction. These are the parameters which influences susceptibility of the soil to earthquake induced liquefaction. That is, earthquake intensity and duration, what type of soil is present there, soil relative density, particle size distribution of the soil, presence or absence of the plastic fines, ground water table location, that is amount of degrees of saturation, hydraulic conductivity or permeability of the soil, placement conditions or depositional environment of the soil, aging and cementation of the soil structure, overburden pressure and finally, the historical liquefaction. So, all these factors influence the earthquake induced soil liquefaction.

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For liquefaction susceptibility criteria, there are several methods that can be seen over here. For fine grained soils and as well as course grained soil, there are various methods to obtain the liquefaction susceptibility criteria. They are Chinese Criteria and Modified Chinese Criteria, Andrews and Martin Criteria of 2000, Youd et al 2001 criteria, Seed et al 2003 criteria, Bray et al 2004 criteria, Bray and Sancio 2006 criteria, Boulanger and Idriss 2006 criteria and various other studies. So among these, the most important or maximum widely used one is by Youd et al 2001. This is maximum used worldwide for the liquefaction susceptibility estimation.

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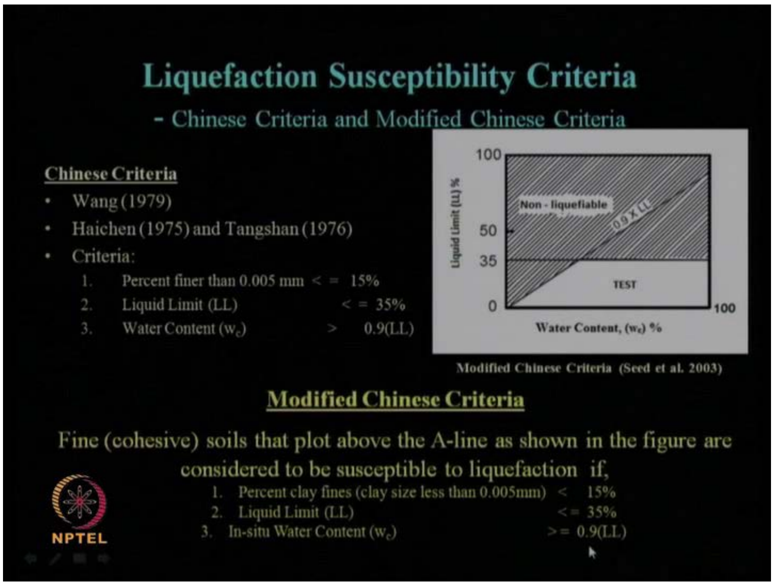

Now, let us see Chinese Criteria and Modified Chinese Criteria. Chinese Criteria was developed by Wang in 1979 based on the study by Haichen and Tangshan in 1975 and 1976. The criteria says that, the percent finer than 0.005 mm should be less than 15 percent, liquid limit should be less than equal to 35 percent and water content should be greater than 0.9 times of liquid limit for that fine grained soil. This criterion is basically for fine grained soil.

So, this is the typical plasticity chart. As we know, liquid limit versus water content, this is the A line equation. So, Modified Chinese Criteria says that, fine soils that plot above the A line as shown in the figure are considered to be susceptible to liquefaction, if these three conditions are met. That is, percent of clay present in the soil is less than 15 percent, liquid limit is less than equals to 35 percent and in-situ water content is greater than or equals to 0.9 times liquid limit.

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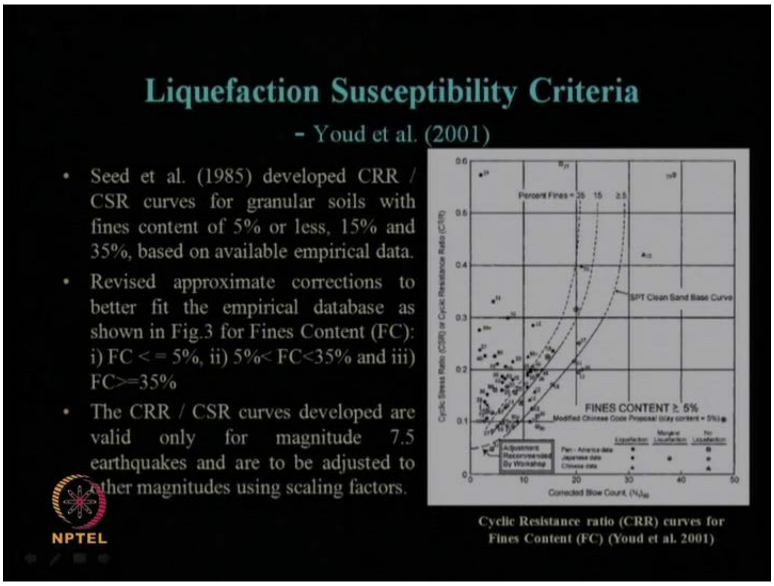

Then, the Youd et al 2001 criteria is a combination of several other researchers findings. This is a kind of report or methodology proposes which is accepted worldwide. So, Seed et al in 1985 developed the ratio of this CRR versus CSR curve for granular soil. CRR is cyclic resistance ratio and CSR is cyclic stress ratio. Regarding the details about these things, I have already discussed in my another video course for NPTEL, that is on Soil Dynamics. So, I request all the viewers of this course, to also go through my another video course on Soil Dynamics, module number 4. We can get all these basic details there and hence, I am not covering it here.

So, it has been proposed how to estimate this cyclic resistance ratio or CRR from corrected blow count SPT N 1 60. So, these are all the collected historical data points of earthquake from actual field test results, as to where the soil got liquefied and where it did not get liquefied. That shows the three curves like percent fines less than equals to 5 percent, another is between 5 to 15 percent, this range between 15 to 35 percent and more than 35 percent. So like that, this percent fine, if it is less than equals to 5 percent, then it is called SPT clean sand based curve. Clean sand correction is also required to be computed to estimate this CRR value from this curve.

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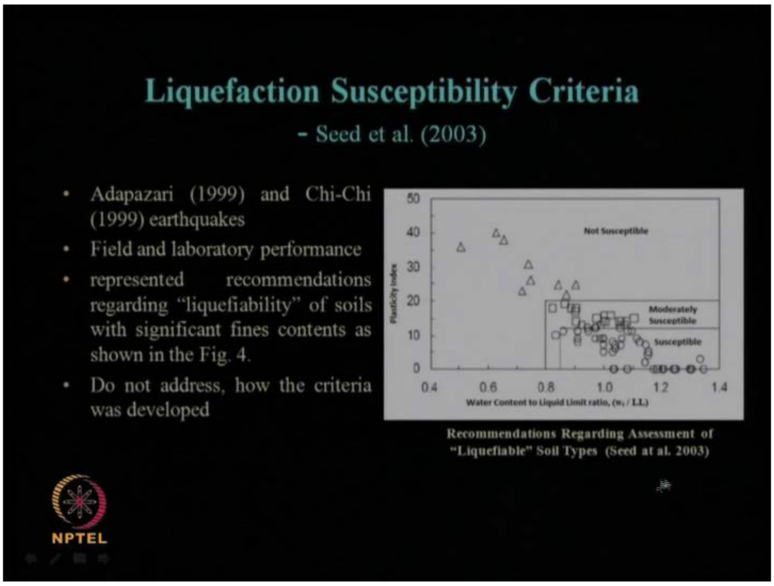

Seed et all in 2003, they proposed the recommendations for fine grained soil. That is, plasticity index versus water content to liquid limit ratio in the x-axis. They have divided into different three zones, that is susceptible to liquefaction, moderately susceptible and non susceptible. So, accordingly if somebody wants to put their soil in this region and try to find out whether it is coming in susceptible range or non-susceptible range, this recommendation may be used.

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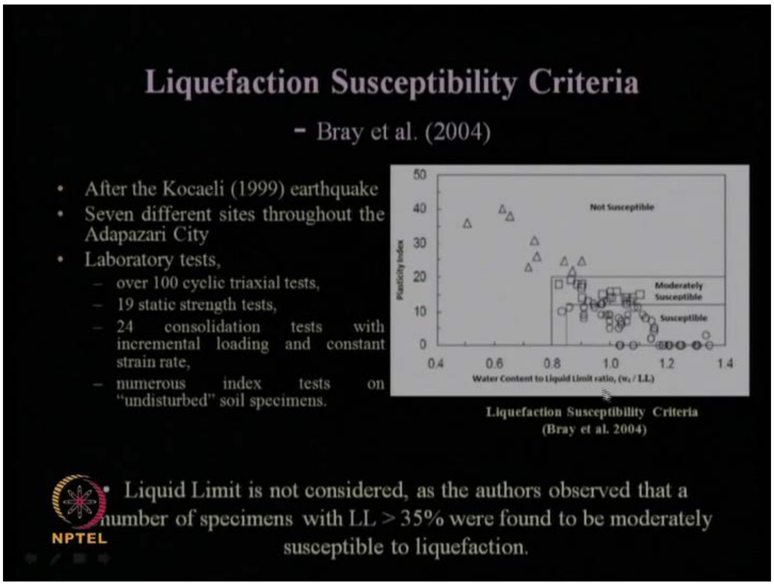

Further, Bray et al in 2004 proposed another methodology using the similar concept of plasticity index versus water content to liquid limit ratio like this. It is mentioned that liquid limit is not considered as the authors observed that a number of specimens with liquid limit greater than 35 percent were found to be moderately susceptible to liquefaction.

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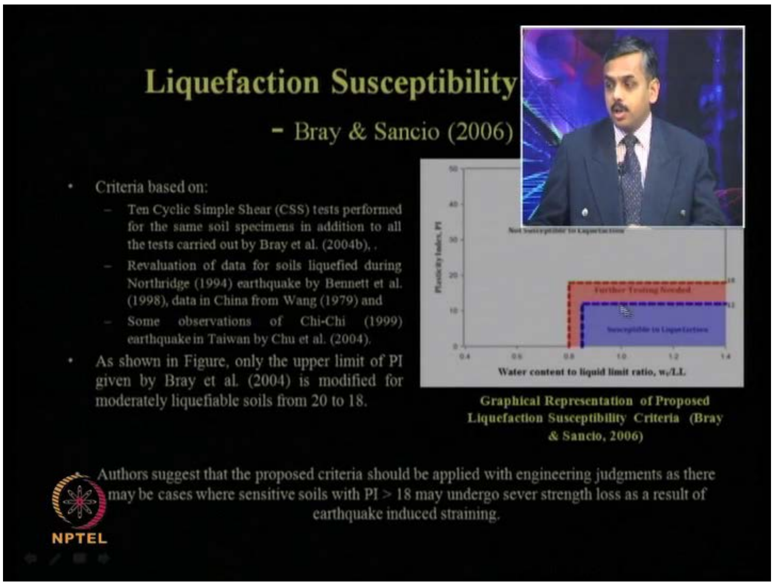

Another set of researchers like Bray and Sancio in 2006, proposed the criteria based on ten numbers of cyclic simple shear test performed for the same soil specimen, in addition to the test carried out as we have explained in the previous slide just now. Some observations of Chi-Chi earthquake of 1999 as reported by Chu et al in 2004 are also incorporated in that. In the same pattern of plasticity index verses the water content to liquid limit ratio have been zincified into three zones. One is susceptible to liquefaction, another further testing is needed and another is not susceptible to liquefaction zone.

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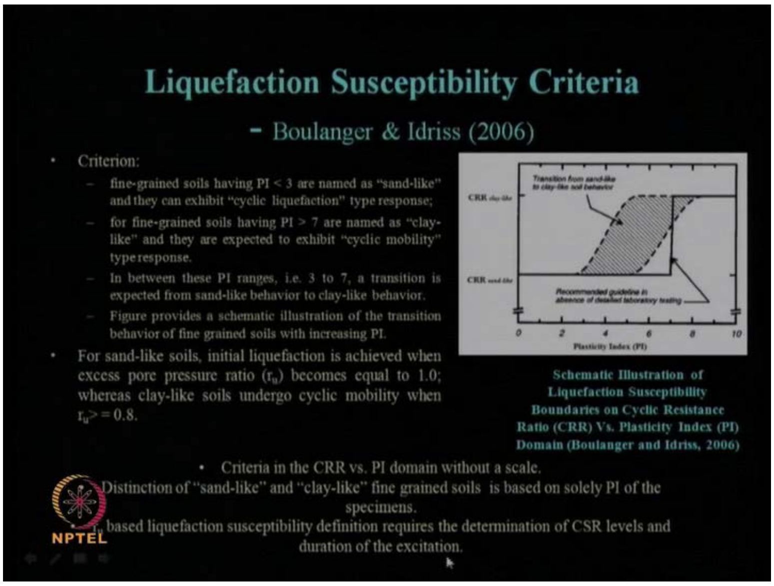

Another criteria is mentioned by Boulanger and Idriss in 2006, where CRR of clay like material and CRR of sand like material has been considered as two boundaries with respect to the x axis plot of plasticity index PI. So, different values of plasticity index, the typical range from transition of sand like to clay like soil behavior is mentioned by these researchers, which is showing the exhibit of cyclic liquefaction.

So, fine-grained soils having plasticity index less than 3 are named as sand like and they can exhibit the cyclic liquefaction type response. Whereas, for fine-grained soils with plasticity index greater than 7 are named as clay like material and they are expected to exhibit cyclic mobility type response.

So, in one case it is cyclic liquefaction and another case it is cyclic mobility, depending on whether it is sand like behavior or clay like behavior. In between range, that is when plasticity index is in between 3 to 7, a transition between this sand like to clay like behavior is proposed to occur. This figure provides the schematic illustration as to how this transition from sand like to clay like behavior is occurring. So, this criteria in the

CRR versus plasticity index domain without a scale and distinction of sand like and clay like fine grained soils, is based is solely on the plasticity index on the specimens and r u value, that is pore pressure ratio as it is mentioned over here. Excess pore pressure ratio for sand like soils, initial liquefaction is achieved when excess pore pressure ratio r u becomes equals to 1. So, if it is less than 1, then it is not liquefying.

For clay like soil, it undergoes cyclic mobility when this r u value exceeds 0.8. So, for sand like material it has to be equals to 1, so that, one can say liquefaction is going to occur. For clay like material, it should be more than equals to 0.8, and then one can say it is going through the cyclic mobility.

So, this r u based liquefaction susceptibility definition requires the determination of CSR levels and the duration of excitation. So, these things are the topic of research even today, in this year 2013. Still further researchers are carrying out various researches from the collected field test data of liquefied zone, non liquefied zone, and transition zone, etcetera and also doing laboratory tests and combining these dataset for result, as many datasets, so that, this interpretation of results and then further proposing of some new criteria will be valid and can be used by various researchers.

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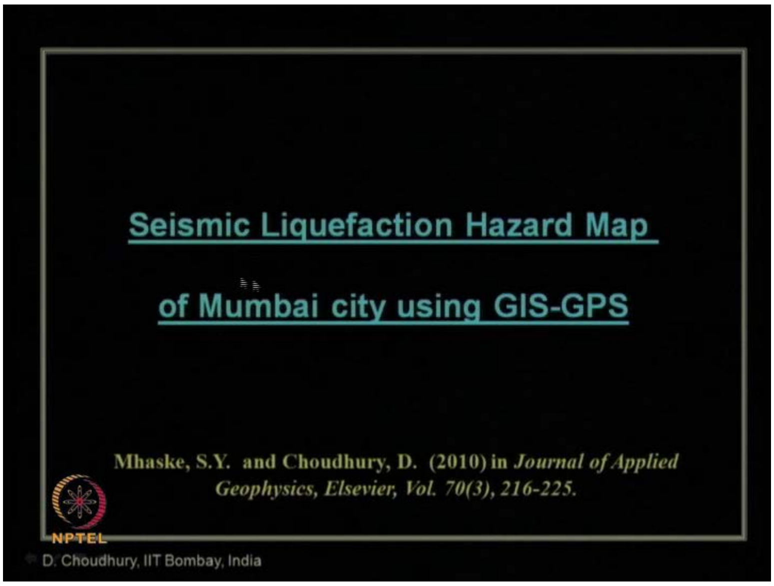

Now, let us see seismic liquefaction hazard map of Mumbai city using this GIS and GPS. The details of this work can be obtained in this journal paper of Applied Geophysics, Elsevier publication, volume 70(3), page number 216 to 225 by Mhaske and Choudhury.

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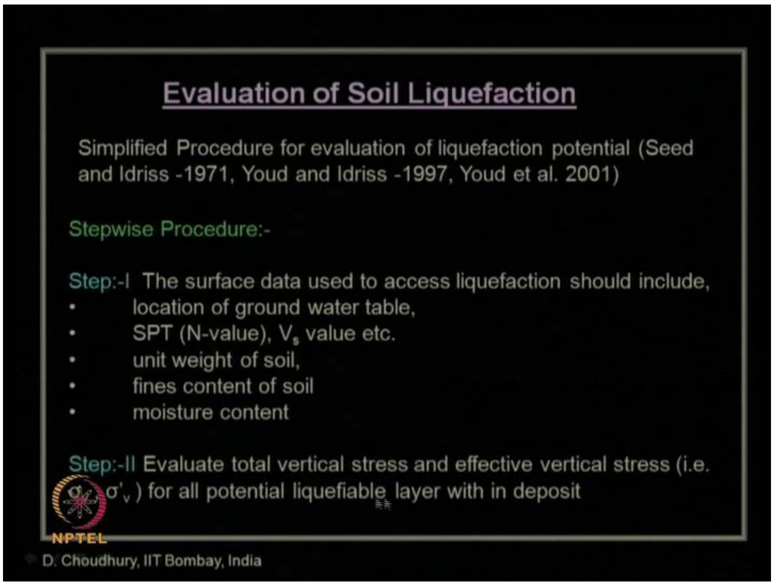

How is the evaluation of soil liquefaction carried out? From the entire set of borehole data collected for the entire Mumbai region as the dynamic soil properties, the V s value has been estimated and reported just few slides back. We have discussed here. Then for each borehole location, the soil liquefaction susceptibilities are estimated using the simplified procedure for evaluation of liquefaction potential.

This simplified procedure was basically proposed by Seed and Idriss in 1971 and further modified by Youd and Idriss in 1997. The final one is by Youd et al 2001, which is widely used worldwide as I have mentioned. So, discussion about this simplified procedure to evaluate liquefaction potential is available in my other video course of N PTEL, which is on Soil Dynamics. Module 4 of that discusses about this simplified procedure for liquefaction estimation.

So, these are the stepwise procedure. In step 1, the surface data used to access liquefaction should include location of ground water table, SPT N value, and shear wave velocity value, unit weight of soil, fines content of the soil and moisture content. In step 2, evaluate the total vertical stress and effective vertical stress, that is sigma v and sigma v dash for all potential liquefiable layer within the deposit.

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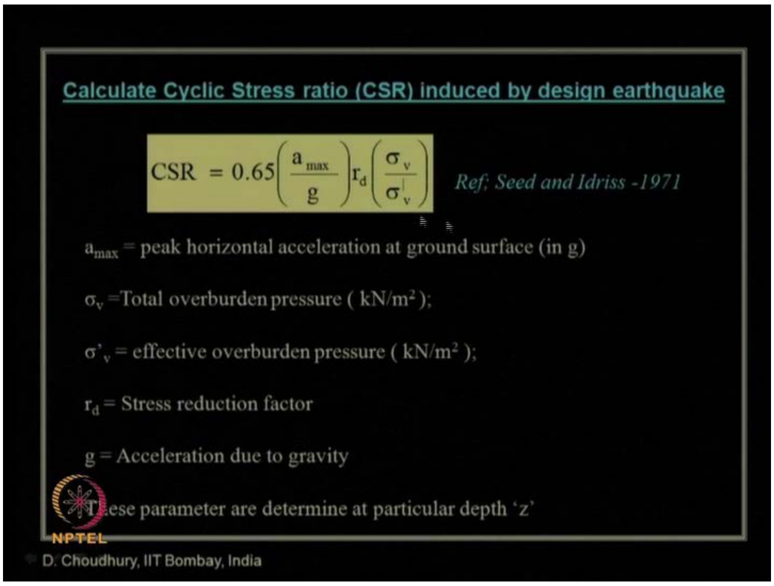

Then, one needs to calculate this cyclic stress ratio or CSR as induced by the design earthquake. So, for a particular region, we all know what is design basis earthquake. As per the codal provisions or the zonation map or the seismic micro zonation point of view, one can find what is the value of this a max for a region. That a max value can be used. This g is acceleration due to gravity. So, this is just a number and r d is nothing but stress reduction factor due to flexibility of the soil. This sigma v is total vertical stress and sigma v dash is effective vertical stress. So, with that a non-dimensional parameter CSR cyclic stress ratio can be obtained, which is proposed by Seed and Idriss in 1971.

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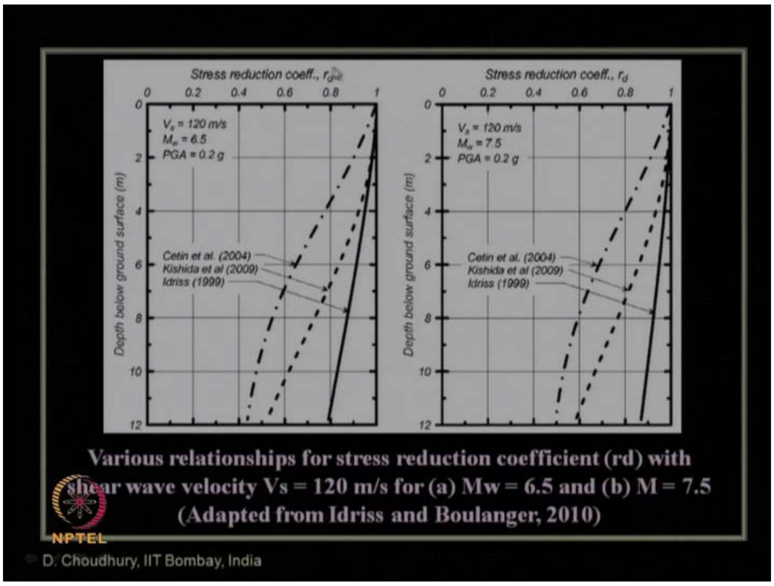

Now, how to select this stress reduction coefficient r d. There are various researchers who had proposed different ranges or values or equations for r d. We can see over here, as we go deeper and deeper inside the ground, so from the ground surface where the depth is 0, if we go deeper below the ground as the depth increases in meter unit, it is shown over here, the r d value reduces also from one to this one. So, one means there is no correction or stress reduction coefficient in this equation. It is not required. One means, that is at ground surface. But as we go deeper because of flexibility of the soil, this stress reduction coefficient needs to be incorporated.

One can see, Idriss in 1999 proposed this line to calculate the value of r d for a certain value of m s, which is certain value of V s shear wave velocity 120 meter per second. Moment magnitude of earthquake is about 6.5 p g a value of 0.2 g. For that, this is the line. Whereas, Kishida et al 2009 mentioned that this line to be used whereas, Cetin et al 2004 mentioned that this value to be used. Whereas, for another range of same soil, that is V s value 120 meter per second but under higher magnitude of earthquake. When higher magnitude of earthquake M w of 7.5 is coming at that location, these are the values of r d as proposed by different researchers.

It is adapted from the research paper of Idriss and Boulanger of 2010, one can easily see that there is a wide variation in the value of this r d, which can influence this calculated value of this CSR. So, the question is, which is the correct value to calculate this r d. So, in the absence of a correct value, one can easily use the method proposed by Youd et al 2001 to calculate the reasonable range or value of r d.

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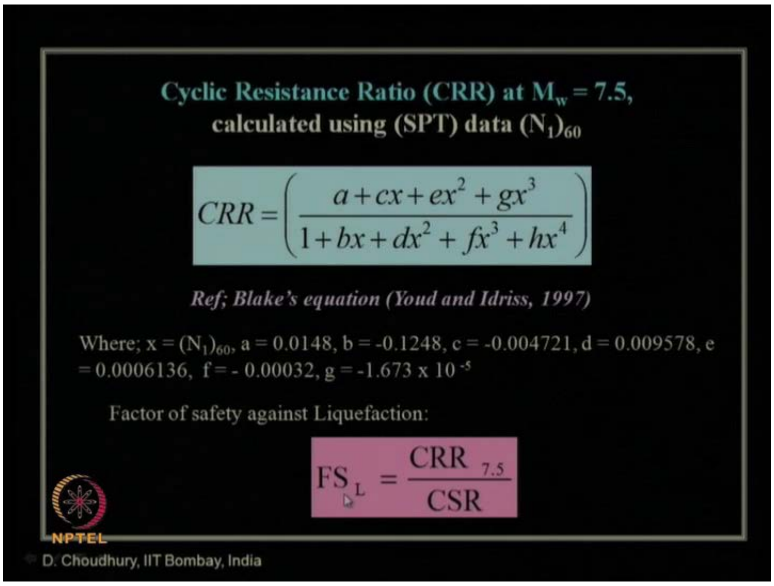

Then, let us see another parameter cyclic resistance ratio, also called as CRR. At the reference magnitude of 7.5, it can be calculated using SPT data of N 1 60, using this expression which is known as Blake's equation as proposed by Blake. It is available in the paper of Youd and Idrisss 1997 and also Youd et al 2001.

These are the x s' in this equation. x is nothing but N 1 60, corrected SPT N value and various coefficients a, b, c, d, e, f, g, and h, all are mentioned over here. Factor of safety against liquefaction is computed using this expression. Factor safety against liquefaction is nothing but CRR 7.5. 7.5 is nothing but at the moment magnitude of 7.5. If the earthquake of that zone for which the design is considered is different than 7.5, then correction due to magnitude correction needs to be carried out.

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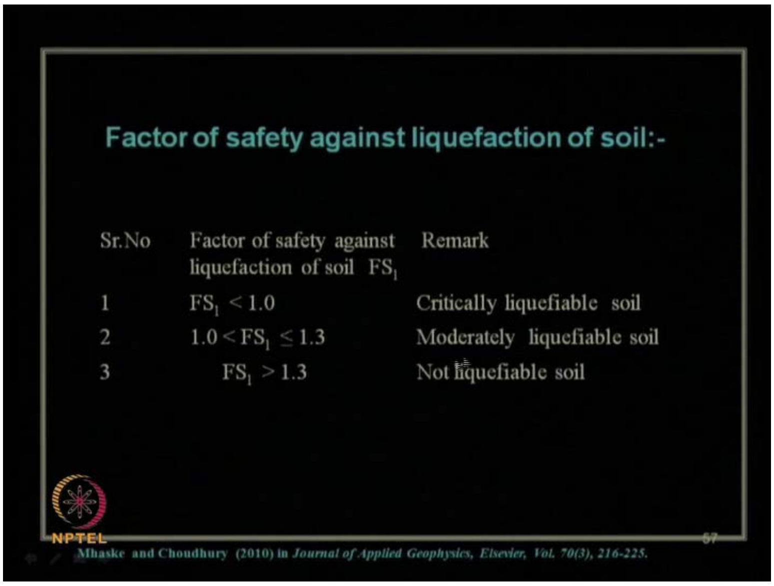

So, CRR by CSR will give the factor of safety with respect liquefaction. Using this concept, this paper of Mhaske and Choudhury in journal of Applied Geophysics, Elsevier, classified the 3 ranges of factor of safety with respect to liquefaction. That is, their value to identify or remark their soil as critically liquefiable, moderately liquefiable, and non-liquefiable soil.

What is critically liquefiable? When factor of safety is less than 1. When factor of safety is in between 1 to 1.3, it is mentioned as moderately liquefiable soil and when factor of safety is greater than 1.3, it is considered as non-liquefiable soil. So, using these ranges of factor of safety for entire Mumbai city in this paper, the calculations for factor of safety against liquefaction with respect to depth and with respect to all boreholes were carried out.

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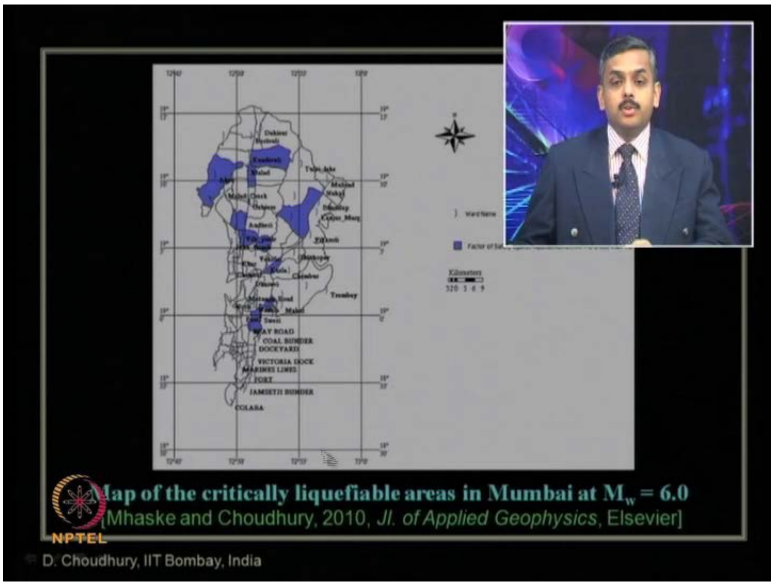

Finally, this is the GIS based map for entire Mumbai, which shows the critically liquefiable area. Critically liquefiable means that, in these locations, in these patches as shown by this color, shows critically liquefiable areas in Mumbai at a moment magnitude of 6. So that means, if a moment magnitude of 6 earthquake comes in Mumbai, these are the region where soil is going to liquefy according to the present scenario. Critically liquefiable means, factor of safety with respect to liquefaction will be less than 1.

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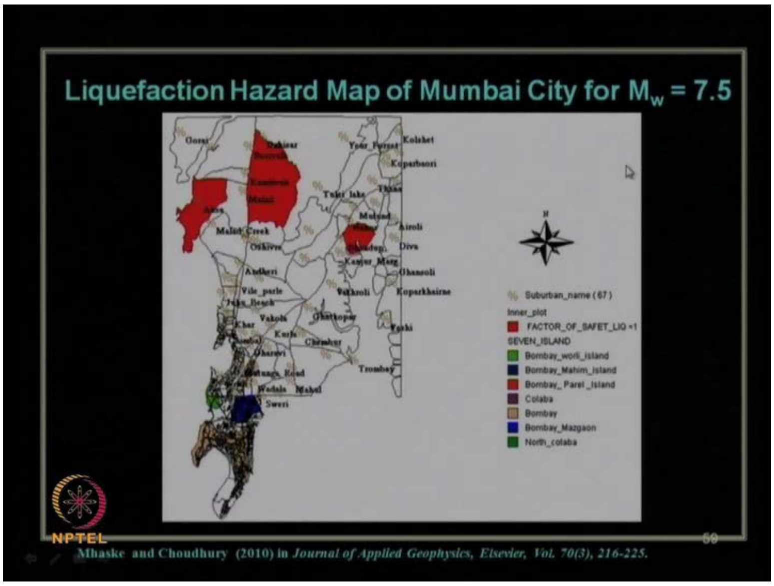

This is another liquefaction hazard map for Mumbai city. It shows for moment magnitude of 7.5, if it comes in Mumbai, these are the regions where it is going to critically liquefy. One can easily see that these are nothing but the areas where it is the reclaimed land. So, that is why it is another kind of validation. As we know, the reclaimed land or field up land are more prone or susceptible for liquefaction during an earthquake, which is also got validated from the property and from this liquefaction hazard map.

So, how people can use this hazard map for further design? So, if somebody is planning to construct any big high rise building in these locations, which is quite possible in Mumbai city, extra design care and design measure needs to be taken for the foundation design and other designs. If somebody is constructing a pile foundation in these locations, then pile foundations need to be designed with respect to the liquefiable zone, which I am going to discuss in subsequent lecture modules in this course.

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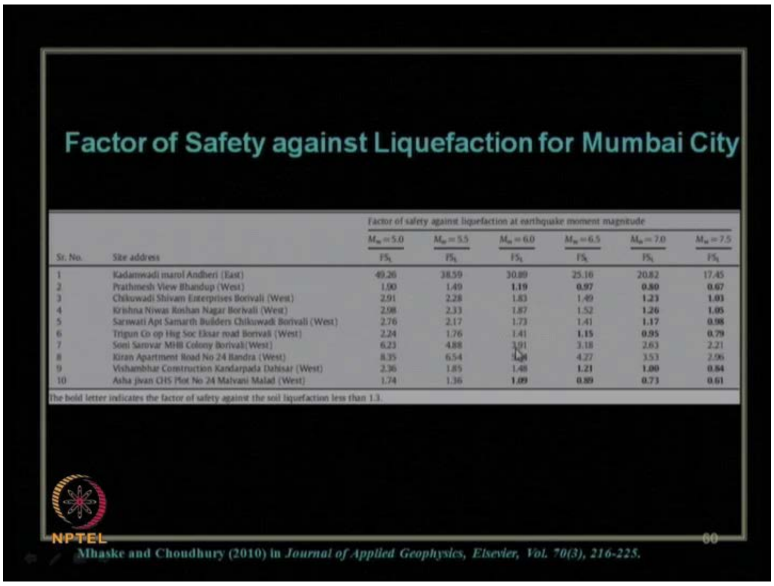
Factor of Safety against Liquefaction for Mumbai City

Sr. No.	Site address	Factor of safety against liquefaction at earthquake moment magnitude					
		$M_w = 5.0$	$M_w = 5.5$	$M_w = 6.0$	$M_w = 6.5$	$M_w = 7.0$	$M_w = 7.5$
		FS_L	FS_L	FS_L	FS_L	FS_L	FS_L
1	Kadamwadi marol Andheri (East)	49.26	38.59	30.89	25.16	20.82	17.45
2	Prathmesh View Bhandup (West)	1.90	1.49	**1.19**	**0.97**	**0.80**	**0.67**
3	Chikuwadi Shivam Enterprises Borivali (West)	2.91	2.28	1.83	1.49	**1.23**	**1.03**
4	Krishna Niwas Roshan Nagar Borivali (West)	2.98	2.33	1.87	1.52	**1.26**	**1.05**
5	Sarswati Apt Samarth Builders Chikuwadi Borivali (West)	2.76	2.17	1.73	1.41	**1.17**	**0.98**
6	Trigun Co op Hsg Soc Eksar road Borivali (West)	2.24	1.76	1.41	**1.15**	**0.95**	**0.79**
7	Soni Sarovar MHB Colony Borivali(West)	6.23	4.88	3.91	3.18	2.63	2.21
8	Kiran Apartment Road No 24 Bandra (West)	8.35	6.54	[illegible]	4.27	3.53	2.96
9	Vishambhar Construction Kandarpada Dahisar (West)	2.36	1.85	1.48	**1.21**	**1.00**	**0.84**
10	Asha jivan CHS Plot No 24 Malvani Malad (West)	1.74	1.36	**1.09**	**0.89**	**0.73**	**0.61**

The bold letter indicates the factor of safety against the soil liquefaction less than 1.3.

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Mhaske and Choudhury (2010) in *Journal of Applied Geophysics, Elsevier, Vol. 70(3), 216-225.*

This table shows the various values of factor of safety against liquefaction for entire Mumbai city as obtained in this journal paper by Mhaske and Choudhury 2010 in journal of Applied Geophysics in Elsevier publication. One can see these are the different site address like Andheri, Bhandu, Boriveli, Bandra, Malad, Dahisar etcetera. So, factor of safety against liquefaction for different moment magnitude are mentioned over here. Like moment magnitude of 5, 5.5, 6, 6.5, 7, and 7.5, all values are given over here. One can see that the non-bold values are perfectly fine. That means, if in Mumbai, magnitude of 5 to 5.5 earthquake comes, there is absolutely no problem in terms of liquefaction is concerned, in these mentioned 10 locations. That is, in these 10 sites.

But, if magnitude of 6 to 6.5 or more than that comes, the soil tends to start liquefying at certain locations. Like one can see at Bhandu west, if a magnitude of earthquake magnitude 7.5 comes, the soil is going to critically liquefy. That is, it is going to fully liquefy. That means, the factor of safety less is than 1, even at magnitude 7 also.

Similarly, for other locations also, the values that are given over here, which are very much useful for any designers to utilize this concept to take further protection and necessary design steps and methodology and construction steps for earthquake resistant design in and around Mumbai city using this data.

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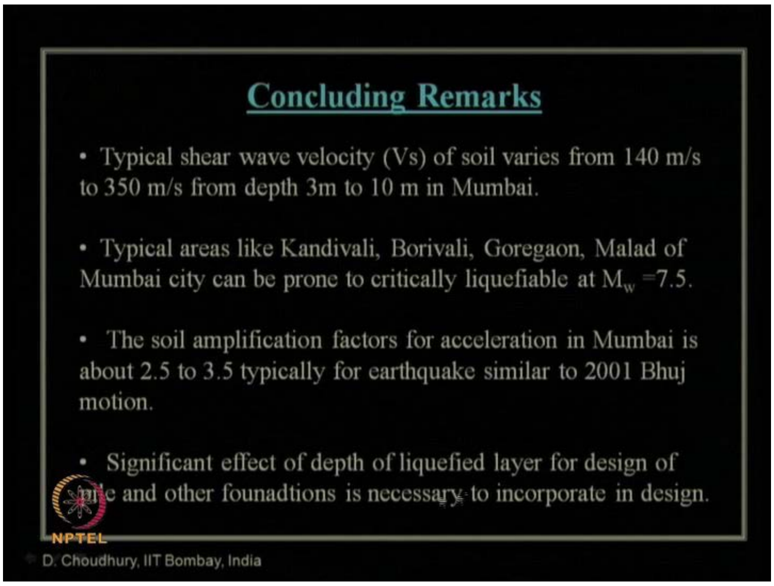

So, in concluding remarks, we can say that typical shear wave velocity what we have obtained for the soil in Mumbai region between 3 meter to 10 meter range, varies typically between 140 to 350 meter per second. Typical areas like Kandivali, Borivali, Goregaon, Malad of Mumbai city can be prone to critically liquefiable condition, when an earthquake magnitude of 7.5 hits in an around Mumbai. The soil amplification factors for Mumbai, which has been obtained earlier, it can range between 2.5 to 3.5, if a similar type of Bhuj earthquake motion of 2001 hits Mumbai city.

From this known knowledge of geotechnical earthquake engineering, one can further take precautionary measures to find out, what are the significant effect of depth of this liquefying layer, which needs to be considered for design of pile foundation and any other type of foundation. Even for shallow foundation also, which an necessary to incorporated in the design. So with this, we have come to the end of the present module, module number 6. We will continue further in our next lecture.