Port and Harbour Structures
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Module 4, Lecture 20
Design of Berthing, Structures-1

In the last class I showed a slide where a ship pile wall anchored to the dead man diaphragm wall as fail nearby that location we have proposed the structure,

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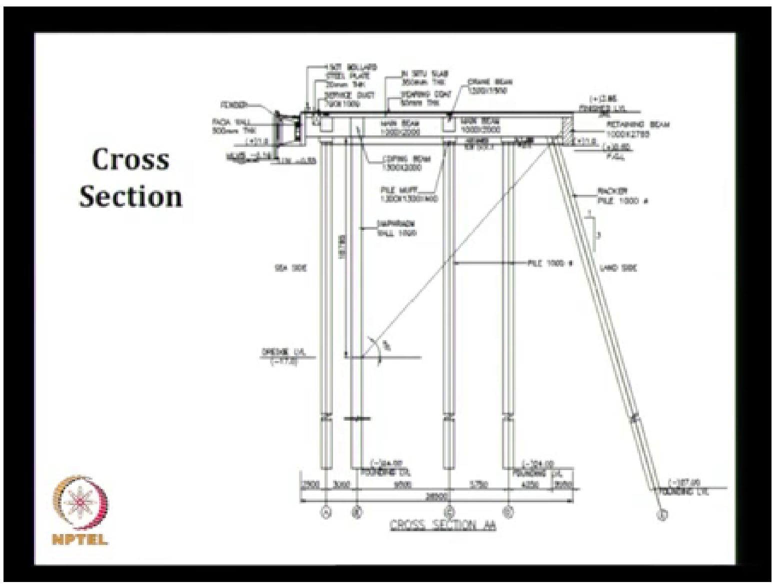

Any structure which you are planning you should have some redundancy; redundancy means you should have some additional members so that in case of failure of one of the members the other members will take load especially civil engineers will know that interest system we have some redundant members.

So there we had only one main sheet pile wall and the anchored wall with a tie bag system whereas here we have a pile, a diaphragm wall made of concrete then we have two vertical piles and one racker pile, racker pile normally is designated by these two numbers one horizontal to three vertical, we can make it stipper than this but flatter than this construction will be difficult, so we can have (one verti) one horizontal to three vertical to one horizontal to six vertical.

The construction company is prefer a stipper thing about one in four or one in five but it is preferable to have one in three because if we have a axial force in the member this axial force will have two components, one component is vertical component, another component is horizontal component, this horizontal component will take care of the horizontal forces, so we have three equations of equilibrium, sigma of all the vertical forces to be zero.

Sigma of all the horizontal forces to be zero, sigma of the moment to be zero, when you have large lateral force to be registered either a berthing force or a mooring force if we have a racker pile these racker piles will have an axial force, this axial force will have a horizontal component which will resist these forces, otherwise if we have vertical member all the horizontal forces will be taken care only by shear in the member, shear and bending moment not by axial force.

But this racker piles will reduce the deflection and doesn't have a this during seismic condition energy dissipation so in seismic zone 4 and 5 you do not recommend racker piles, what is useful in one particular condition may not be useful in other condition, the Japanese code recommend that we don't provide racker piles in seismic zone 4 and 5, another important factor why we are going in for the front diaphragm wall is they have specified surcharge load.

Of five zero 50 kilo Newton per meter square, the surcharge load if you provide a rear diaphragm wall will also exert lateral air pressure, in order to avoid the situation we are providing sufficient width of the deck so that a 45 degree line this is for clay soil where 5 is equal to zero, this 45 degree line is cutting here so any surcharge which is placed at this location will not create any air pressure, since we are constructing a deck of 2 meter.

We want to remove the soil up to plus 1 meter so that the over burden weight of the soil is not there on the active air pressure so we have to reduce the active air pressure by providing a deck and placing a surcharge beyond the 45 degree line so that the surcharge effect will not be felt up to the dredge level that is why we are providing diaphragm wall here and we have a crane beam here, another crane beam here and when you have this 45 degree line is called as active edge.

And if your pile is in the active edge this will not resist any lateral force if you are modeling springs you have to model the spring below this level similarly for this level and we provide a retaining beam at the end this will be design for the surcharge pressure and so that we transfer the load lateral load here, there is a difference between transferring the load here and transferring the load here, if you transfer the air pressure.

Here the diaphragm wall will be subjected to lot of bending moment, some portion of the pressure will go to the diaphragm wall, the passive zone some portion will be transferred as shear at the diaphragm wall beams and junction, the shear will be transferred to the vertical piles and racker piles, but if we have a retaining beam in which you have the lateral force this will be distributed uniformly, not uniformly this will be distributed to all these piles and diaphragm wall.

The diaphragm wall per say will not be subjected to additional bending moment, only whatever shear comes it will take, in this system which pile will take more load? Forget about this diaphragm wall and racker pile we have 1000 mm pile here, another 1000 mm pile here, another 1000 mm pile here as same that this racker pile and diaphragm wall doesn't exist and you are applying a air pressure here or a berthing force or a mooring pull.

In the load be distributed to these three piles equally, yes or no? Which will take more load we have the rose mark here a, c, and d which pile will take more load? It is a correct wrong answer, whenever any force is applied here since the beam is very rigid the deformation at top of all these piles will be the same this is called as consist and deformation, most of you might have studied in strength of materials, consist and deformation method.

So if the deformation at the top of the pile is same to cause let us say the deformation is unity to cause unit deformation here, and unit deformation here, and unit deformation here what is the force required, which needs more force pile a b pile a pile c or pile d, this pile is having a dredge level here so the length is more here, this pile this much portion is in the active zone there is no support, this pile this much portion as in the active zone there is no support.

This is the shorter pile, this is a longer pile suppose you take a beam cantilever beam fix to somewhere here apply a load here, what will be the delta, what is the equation plq by 3ea here the length is more delta a is constant so pa and pc and pd will be different because L is different L is from the fixity point to the top, fixity point for this is here, fixity point for this is here, we will take la is more than lc more than ld, delta is equal to plq by 3ea.

That means p is equal to 3ea delta by lq if L is more p is less, the p will be less here and this will be slightly more than that and this will be slightly more than that, that is why the force here will be more compare to this point as well as this point, but if we introduce a racker pile there will be some difference we cannot simply explain because there will be an axial force on this member and this will have a horizontal component.

In case one of the member’s file let us say this member files when we have these 3 vertical piles which will take care of this diaphragm wall, suppose the diaphragm wall is filling then there can be a slope which can be formed there may not be a permanent damage, but there will be some damage which you have to rectify at a later time, so normally this type of berthing structures we have one important requirement that is called as a utilities.

Through this utilities we have these cables which will be use for power connection will have the oil line, water line, both for bunkering as well as for fire, all these are required to be placed here and we have a bollard here which is to be fixed, so typically we need about 2.5 meter from the central line or clear 2 meter to make this utilities as well as the fixing a bollard so this is requirement and this is a fender which is called as a conical fender.

Which frontal boards so that it varies from mean no water spring as likely above the highest pile water level, the top level of the deck is kept as 3.85 and we provide weep holes in the diaphragm wall so that the water drains through this,

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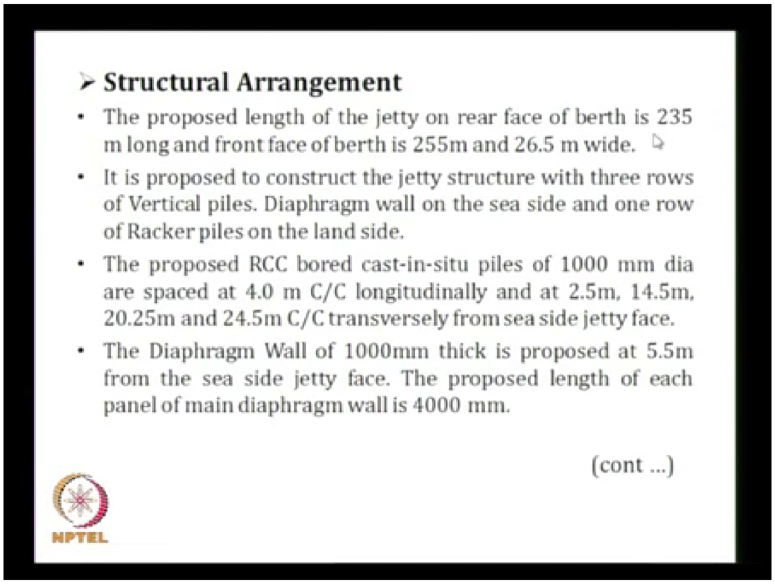

Jetty length is 255 meter on the front side, the rear side because of (some req) some additional structures on either side, rear side the length is 235 the width is 26.5 meters, we are providing 3 rows of vertical piles.

And diaphragm wall on the sea side and (row) one row on the racker piles, a purposely avoiding this diaphragm wall in the front row because during construction this is the general ground level then the water is here so when the water is here very difficult to build the diaphragm wall so we are shifting it to the second row, the piles have diameter 1 meter, this decision of the diameter of the pile is basically from the dredge level to the top level say about 20 meters.

If this is 20 meters unsupported length divided by 20 that normally gives what should be the diameter of the pile, want to decide the diameter of the pile, diameter of the pile is unsupported length divided by 20 this is for a thumb rule you start with some values, the distances are given the lance through spacing is at every 4 meters center to center, the piles are at 2.5 meter, 14.5 meter, 20.25 and 24.5 this distance between this pile a and c is governed by the crane rail spacing

You have a crane rail with track width of 12 meters, so we want one pile below the sea side crane beam, another pile below the land side crane beam, since the track width is 12 meter we want to provide 12 meters is it clear? 2.5 and 14.5 the minimum distance that can be there between one pile and another pile is 3 times the diameter of the pile, suppose diameter is (3) 1 meter the center to center distance between this point and this point.

Shall be not less than 3 into 1 meter that 3 meters, the diaphragm wall thickness also 1000 mm same as unsupported length divided by 20 that we are providing at 5.5 meter from the sea side of the jetty face already water is there near the sea side so we want to build it, fill up that and do it at 5.5 meter and each panel of the diaphragm wall is 4 meter, this will I explain later what each panel means,

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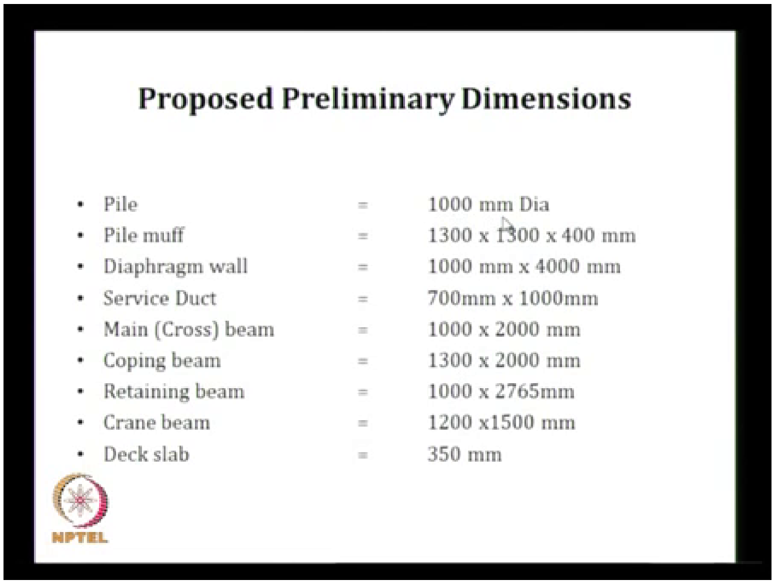

So these are the various dimension we will have a pile. And top of it we will have a pile muff and we have a diaphragm wall, 1000 mm thickness and 4 meter length each panel the service duct dimensions 700mm is the width and 1 meter is the depth, we have a main cross beam that is 1 meter by 2 meter, we have a coping beam on top of the diaphragm wall which is 1.3 by 2 meters, we have retaining beam at the end that is 1 meter by 2.765, we have two crane beams.

The width of the crane beam should be sufficient to fix the rail as well as some cable ducts, the deck slab of the class is 350mm, this thickness is governed by the span of the main cross beam that is the spacing of the main cross beam, these cross beams are at every 4 meters center to center and this design is governed by mobile Harbour cranes, this mobile Harbour cranes the load will be about 200 kilo Newton per meter square, pad loads this also we will discuss later.

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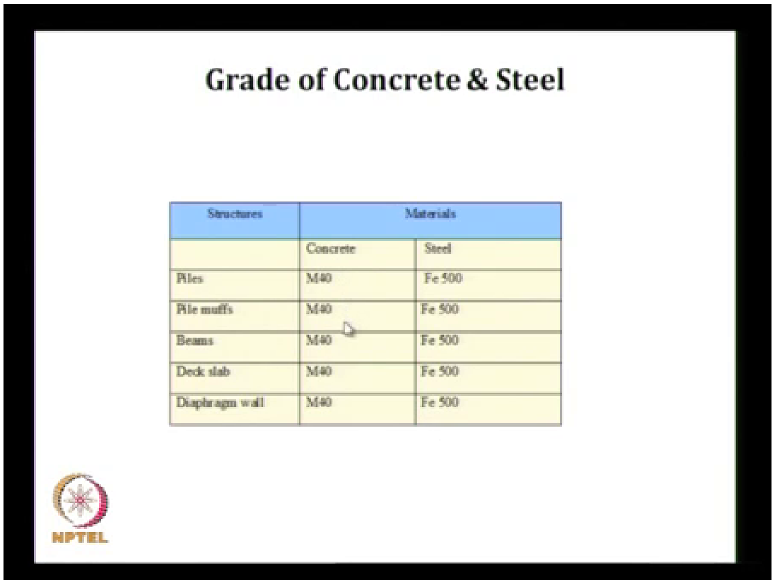

Structures	Materials	
	Concrete	Steel
Piles	M40	Fe 500
Pile muffs	M40	Fe 500
Beams	M40	Fe 500
Deck slab	M40	Fe 500
Diaphragm wall	M40	Fe 500

These are the specification for concrete and steel, we are using m40 grade for all pile, diaphragm wall fe500, there is a code call is2062 which gives these details, fe500d it is called d means it is ductile so we have 3 grades of steel which are used one is the main steel fe250 another is fe451 and another is fe500, 500 means 500 mega Pascal that is a yield strength of the material where d means elongation that is it is a ductile material elongation is about 14 percent.

So in earthquake zone we need elongation of about 14 percent, fe415 only was having a 14 percent elongation earlier now they are making fe500 also with 14 percent, now we have fe550 also, it is also being produced,

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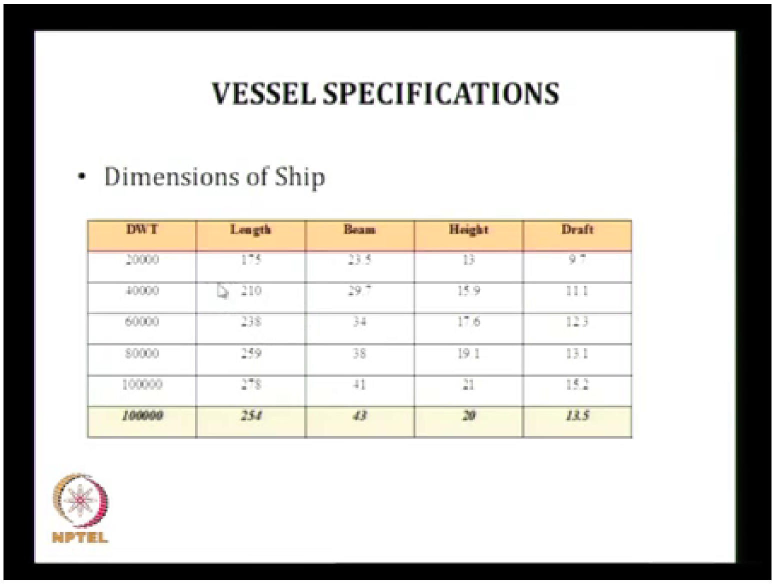

These are the various sizes of the ship which is likely to come to the berth, out of which you are choosing this ship for the design purpose,

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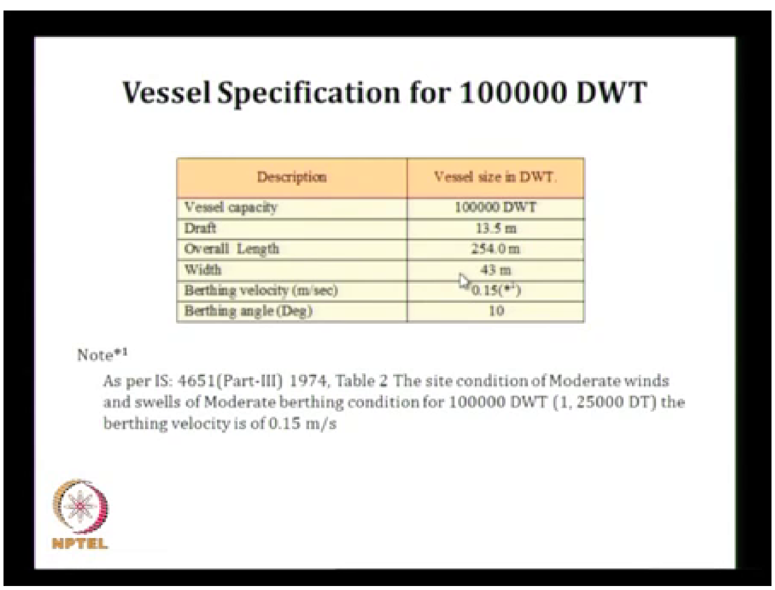

So we are getting this berthing velocity based on the dwt of the vessel, as 0.15 this is given in this code is4651 part 3 1974 table 2 in that we are choosing the site condition is moderate wind and swell and moderate berthing condition, if it is 1,00,000 dwt the displacement ten age is given as 1,25,000 the berthing velocity is taken as 0.15 meter per second, to estimate the berthing force we need this data berthing angle is taken as 10 degrees,

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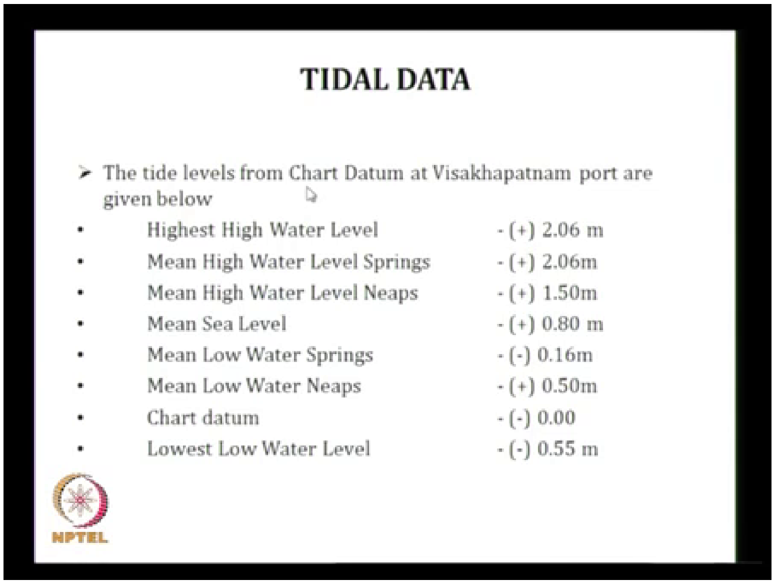

These are the various levels we have the highest high water level. Mean high water spring, mean high water neap, mean sea level, mean low water spring, mean low water neap, this is a chart datum, there is a lowest low water level, these are the various levels which have to be considered,

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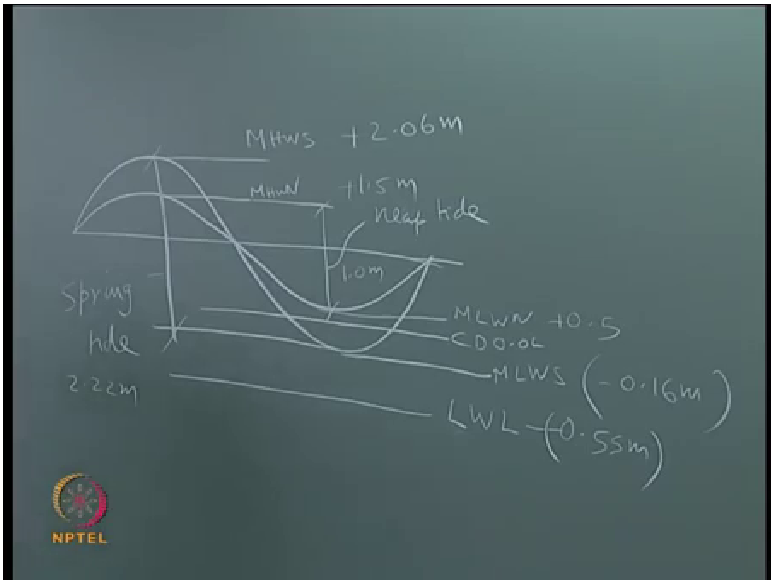

We have two tides one is called as a spring tide another is called as a neap tide, spring tides are in full moon days where the water level variation will be very high, most of the earthquake takes place during this spring tide.

The spring tide the high water level is also is high or low water level also is low, neap tide, your tidal range is less, and mean high water neap is below mean high water spring whereas mean low water neap is above mean low water spring, so in this case (mean high wa) mean high water spring is given as plus 2.06 meters and the mean low water spring is given as 0.16, so that will give you a spring tide of 2.22 meters your mean high water neap is plus 1.5.

Mean low water neap is plus 0.5, the difference is about 1 meter, we have a chart datum the sum are here 0.0 and we have a lowest water level as well as minus 0.55,

This do nor cycle they call it as a 19 years every month you will have two low water springs, so over a nineteen years you find out every month it is not the same, some months it will be very low so out of 19 years in one year it will be very low that is called as lowest low water level.

Now understood mean means average of low water spring, so every month you will have two level water spring over a period of 19 years you compute all the low water springs and find out the average that is called as mean, out of the 19 years and some months you find the lowest water level that is called as low water level, low water level is below chart datum because this is in Vizag inner Harbour in a outer Harbour in a sea side.

Generally the chart datum is a lowest water level where you have large the title variation or where the structure is Harbour is in the inner lagoon your low water level can be below the chart datum, this water level is very important for design purpose to calculate the differential water pressure as well as to calculate the dredge level.

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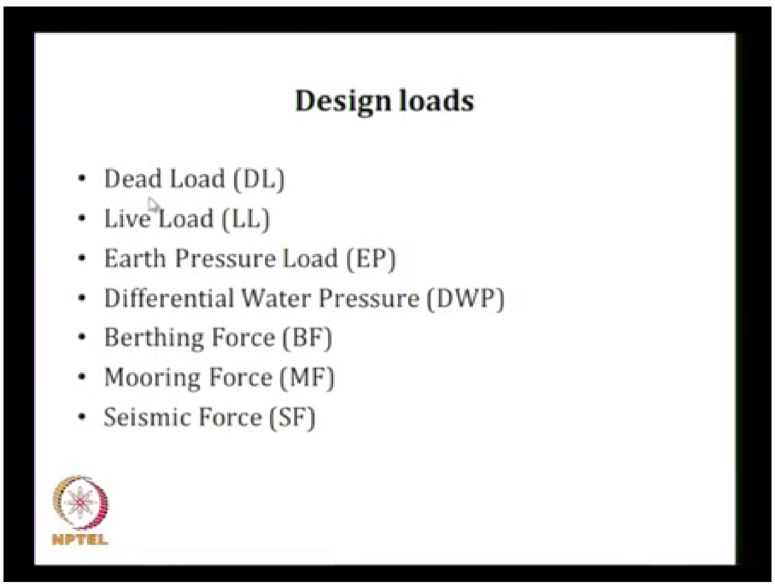

We have to design it for this various forces, dead load, live load, air pressure load, differential water pressure, berthing force.

Mooring force and seismic force, so we will be computing all these forces in the live load you will have uniformly distributed live load, you have mobile Harbour crane load, you have the key crane loads,

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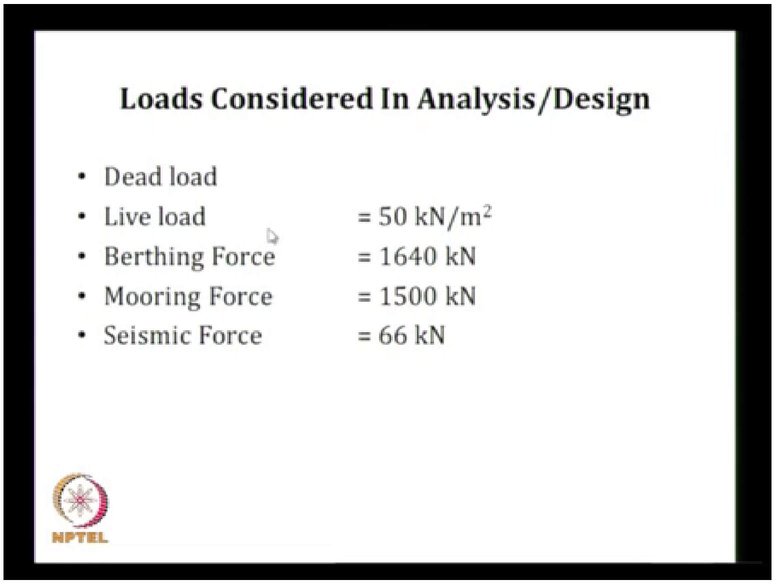

So live load is considered as 50 kilo Newton per meter square, the berthing force is 1640 kilo Newton Mooring force is 1500 kilo Newton, seismic force is 66 kilo Newton, I will be discussing the calculation of all these things in one of the classes.

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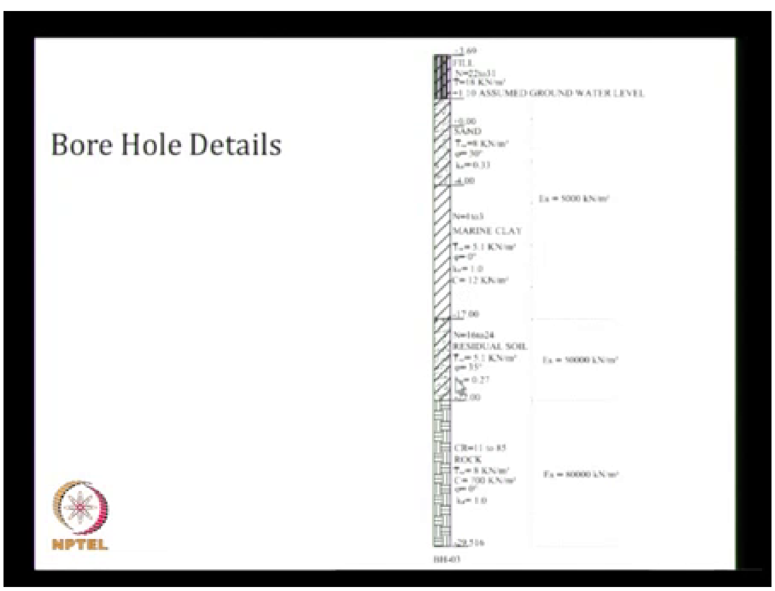

This is the bore well detail we assume the groundwater level at plus 1.1 I will discuss this later how to calculate form the 3.69 level to 1.1 we have a fill material (23:19 sptn value is) 22 to 31, Gama is dry which is about 18 kilo Newton per meter cube because it is above the groundwater level, then we have the sand layer where Gama submerged this 8 kilo Newton per meter square, 5 is equal to 30 degrees, k can be calculated from this and up to minus 4 we have the sand layer.

From minus 4 to minus 17 we have the marine clay; this marine clay is very dangerous the sptn value is between 1 and 3, the Gama is also very less 5.1 the phi value is zero active air pressure co efficient is 1.0, collusion is 12 kilo Newton per meter cube, the young's modulus of the soil from the ground level to this is taken as 5000 kilo Newton per meter square, what is the young's modulus of concrete, 25 giga Pascal, what is the value, what is the equation.

5000 into root of ck what unit, you cannot do like that, as I was telling in many classes, you cannot substitute any unit for fck say that is the result, fck should be substituted in mega Pascal right, then you get the young's modulus mega Pascal, you are a civil engineer, you know that it is a function of whatever you have told is for which grade of concrete, m25 that is correct, m25 5000 into root of 25 that is 25000 mega Pascal.

So young's modulus of concrete is how much this multiplied by root of fck, now you are using m40 grade so it will be more than 6 times that is young's modulus of concrete then we have a is called as a residual soil which is formed as a sandy soil with a mix of clay, n value varies from 16 to 24, submerged is 5.1, phi value is 35 degrees the active air pressure coefficient is 0.27, most of the people they will give only the n value.

From the n value you have to interpolate this ok, then we have a from minus 22 to minus 29 we have the rock and the rock is having a core recovery of 11 to 85, the submerged density is 8 kilo Newton per cube, the coefficient is 700 kilo Newton per meter square, you see the coefficient here it's 12 kilo Newton per meter square, there is a coefficient here is 700 kilo Newton per meter square, phi is assumed as zero degree, active air pressure coefficient is taken as 1.0.

But there is lot of difference in specifying the properties, we specify only c some people for rock they specify c and phi together, but in general from IIT Madras, professor Gandhi and others have done lot of studies on piles and weak rock, we want to follow this as only a cohesive material (if you) it's not a 5c material, some people will give 5 and c even land professors from other institutes and all, but that gives very low strength values.

Irc 78 which is bridge code that also specify something like this, I have taken the sea value from irc code only, the young's modulus also increases it is 5000, has gone to 50,000, it has gone to 80000, rock can be considered very close to m10, m15 or m20 grade or sometimes even m60 grade of concrete depending on the quality of the rock, so this bore well details were very important, if you are designing a structure, if you want to economize the structure.

In foundation if you do properly we can economize 10 to 15 percent, structures only 2 to 3 percent, we are structural engineer you cannot give much optimization but if you do thorough analysis and design, foundation design you can reduce the cost.

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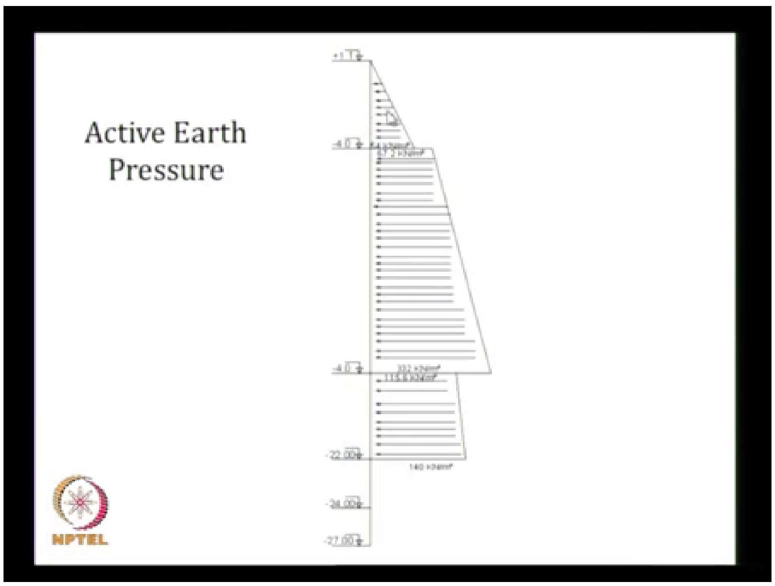

This is the active air pressure diagram at minus 4 we have this pressure and sowing the soil above and sowing the soil below we have this air pressure

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So if you see here above minus 4 we have a sandy soil. We have air pressure point of 0.33, this (guy) ka Gama h minus 2cc zero here that is air pressure at this point considering the soil above, considering the soil below ka is 1.0 ka Gama h, the over burden pressure at this point is the same Gama h is over burden pressure minus 2c is the c is here 12 kilo Newton per meter square, so at the same point you will have two values, is this point clear, (Professor scolded a student skipped that) so at the same point though the over burden pressure is same.

If we have a soil property at this point slightly above sandy soil slightly below clay soil, at the same point you will have two different pressures this cannot be explained mathematically but this is the way we have to do, is this point clear to you, at every intersection you will have two different pressures the over burden pressure is same ka Gama h minus 2c into root of ka, even if it is a sandy soil with a different phi the values will be different.

I have taken up to minus 22, below minus 22 I am not taking because ka Gama h minus 2c will become negative, so if it becomes negative don't put negative pressure, negative pressure doesn't (exs) exist k Gama h if you calculate ka is 1 Gama, h is the over burden pressure above this, over burden pressure you have to multiply Gama into this height, Gama into this height, Gama into this height, Gama varies Gama is not constant minus 2c.

2cc 700 kilo Newton per meter square, so minus 2c will be very high compare to summation of all the over burden pressures, so in that case it will become negative, don't put negative pressure it means if we cut vertically it will stand whatever be the over burden pressure,

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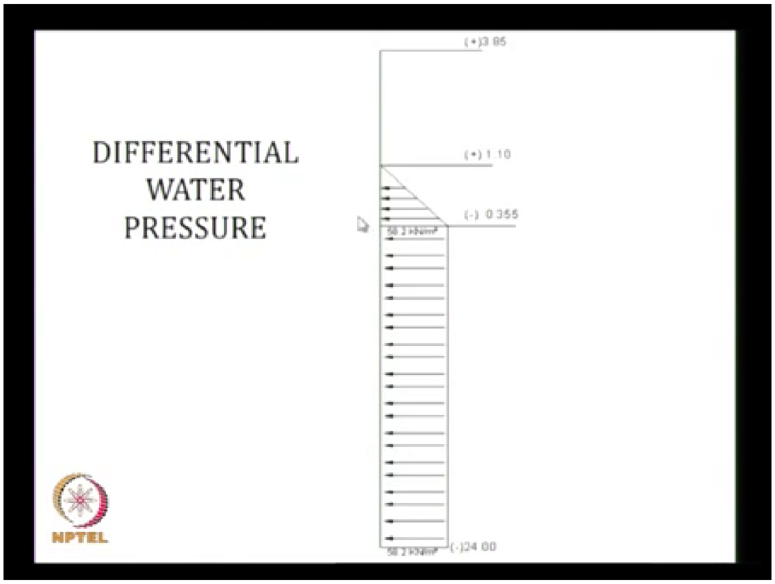

This is a differential water pressure. We are assuming the water level on the land side of the diaphragm wall is 1.1 on the sea side is minus 0.355.

This minus 0.355 depends on the tidal ranges what I have shown, so I will have a separate lecture where I will tell you how to calculate the differential water pressure, but that differential water pressure exist from plus 1.11 up to the founding level of the diaphragm wall, don't stop it here up to this it has to go right down up to the bottom of the structure.

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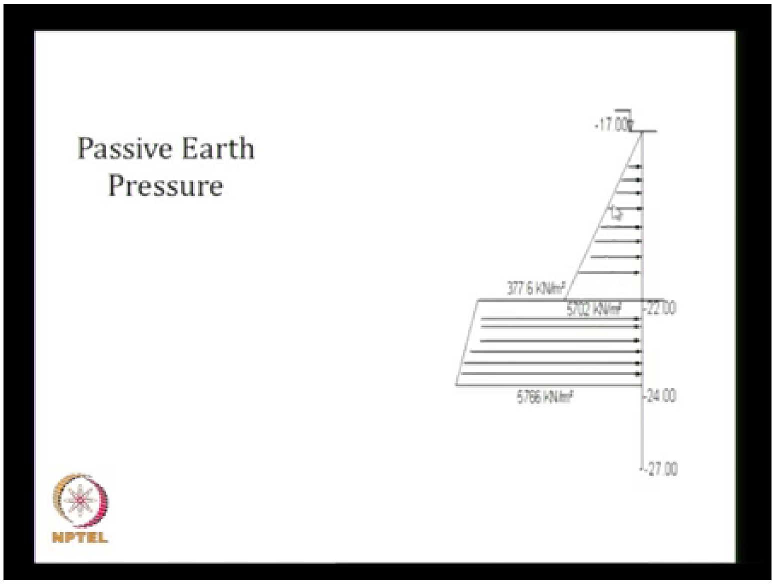

This is the passive air pressure on the sea side, for the diaphragm wall from minus 17 up to minus 24. Though the soil profile is there from minus 17 to some 29 or so we will be stopping the diaphragm wall somewhere here so we have drawn the passive pressure diagram only up to this, it starts from zero here because the over burden pressure is zero, ka Gama x minus 2c, ka Gama will be zero there is no over burden pressure on the sea side that is why it is starting from zero and we have two values here, here this is a sandy soil.

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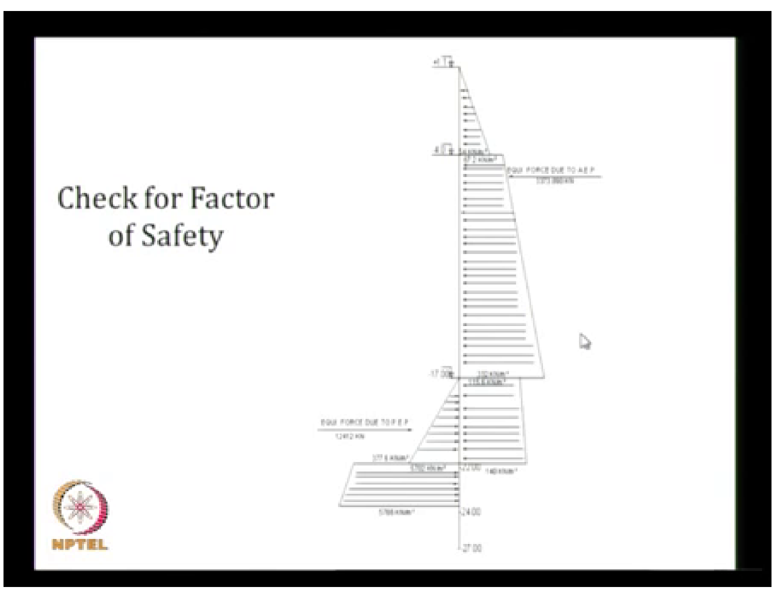

This is a rock so you have to check for factor of safety. Based on your passive pressure, active pressure and differential water pressure diagrams then that will be, that should be greater than two that is a net equivalent force due to passive air pressure, and equivalent force due to active air pressure and differential water pressure this should be two times that of the active air pressure up to that you should create the founding level, this point is clear.

Active air pressure the force that will be exerted on the structure, passive air pressure this is the maximum it can be mobilized, pressure should not be equated I have told earlier only force should be equated you calculate the force due to active air pressure as well as the differential water pressure which is exerted from right to left this is from left to right it should be twice that of this up to that you take this founding level you have to do trial and error you calculate this.

And when it is mobilized it is it will mobilized something like this it will not mobilized fully it will mobilized something like this, to create equivalent to this active air pressure we will have some riser capacity, the structure which has failed do not have this riser capacity because they have not taken the diaphragm wall into the rock I have stopped here we have taken it into the rock, what is the factor of safety here two times.

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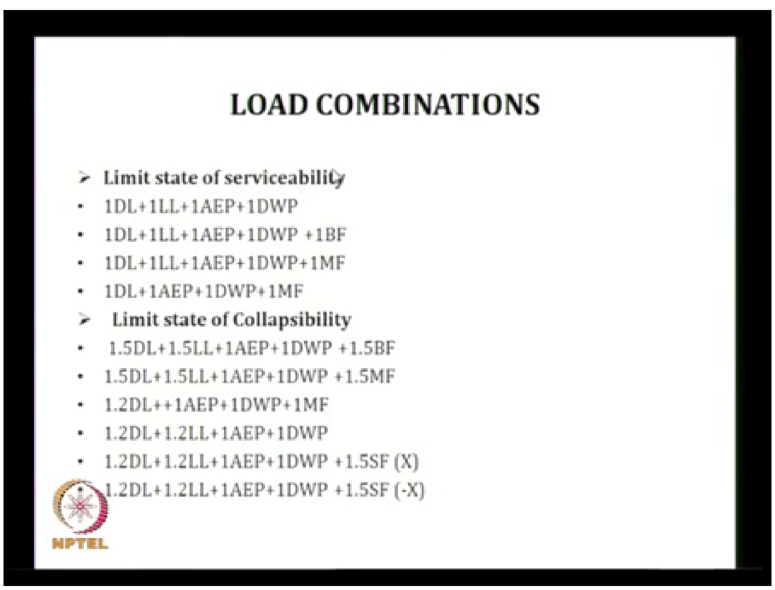

These are the various load combinations one is for limit state of serviceability another is for limit state of collapse, what is the difference between them, when you go for a job I will ask this question, civil engineers, how many are civil engineers, you are simply blinking, you answer something, (all of) all of you are studied this concrete design, what is limit state of serviceability, serviceability is, what is it, deflection is partly correct there is one more aspect.

Shrinkage and cracks is nothing to do with serviceability, don't try to put adjectives, never write adjectives either in your thesis or in your answering, if you are told crack width you would have got a job, you are saying shrinkage you are not getting the job, shrinkage crack is a material behavior the other crack width is based on the loads, what is limit state of color, see when somebody is asking a question you are suppose to answer.

When you are reading limit state of serviceability and limit state of collapse you should try to understand what they are that is not sufficient you should be able to express, limit state of serviceability we are checking the deflection and crack width, limit state of collapse what we are doing, see what you are doing you please it limit state of collapse what you are doing, I am putting a question which you can answer what exactly you are doing in limit state of collapse.

And that is right but actually what you are doing, we are not doing in limit state of collapse when you do the design what are you doing, you are doing the analysis assuming load factor for the loads what are you doing to the strength, how much it is at that, how much is the strength is assumed to be reduced in limit state of collapse, how much it is for concrete, how much it is for steel, we have rain force concrete structure both concrete and steel.

So 1.15 is for what steel, how much is for concrete, to answer this that's all, so what you are finding it difficult is you are not able to explain exactly what is limit state of collapse ok, it's not expected, so what you are doing in limit state of collapse you should know we are assuming two factors one is load factor for the load and safety factor for the material, load factor is varying 1.2, 1.5 etcetera, safety factor is 1.5, 15 for steel and 1.5 for concrete.

Then you told about deflection crack width, so if somebody comes to interview of IIT Madras they expect you to tell what is the permissible deflection and what is the permissible crack width, this Trimurthy has told the right answer, what is there you tell, what is the value, what is the permissible deflection, relates to what, span how much it is span by how much, span by 300 is a wrong answer but it is very close to the right answer.

I think it is span by 350, otherwise I will get confused between all these values, you please check is456, no no is456 is only for concrete no, please check, span by 350, what is the permissible crack width 0.3mm but it is revised now it varies from 0.1 to 0.3mm.