Foundation for Offshore Structure
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Module 1
Lecture No 29
Special Foundations 1

So we will start with a new topic on special foundations you will be looking at some special cases where pile foundations maybe not suitable or alternate foundation types is required because of the ground condition, type of structure and also to some extent applicability for deep water areas where pile foundation may be difficult to install.

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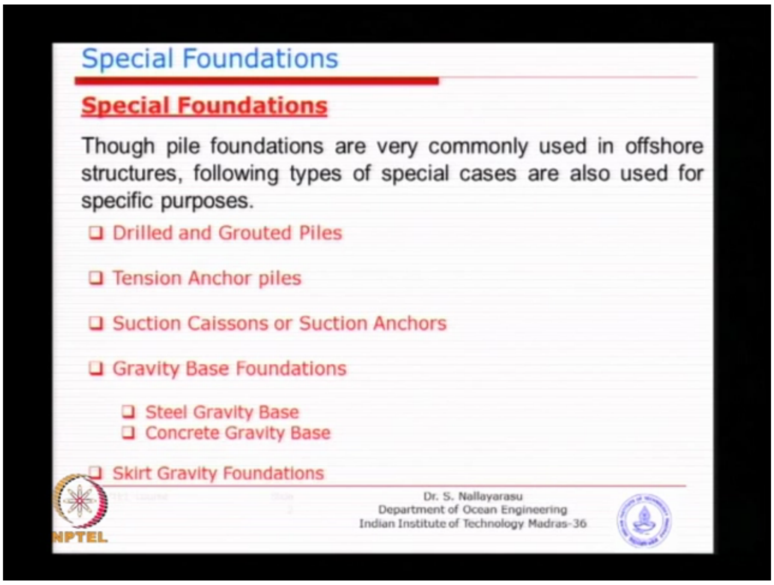

So will just quickly look at the areas in which we need to focus, so one of them is the drilled and grouted piles which I think we have discussed in brief earlier on you know wherever ground conditions like shallow, depth, rock (())(0:51) or maybe hard layers expected earlier on wherein driving piles becomes too difficult. In such situations we can still use even if it is in offshore we can use the drilled and grouted piles means instead of driving through the annulus, through the whole inside the leg you will actually send the drilling rig and then completely drill out to the diameter which is equivalent to the diameter of the inside of the leg and then you insert the pile and basically grout from the bottom of the pile toe up to the top of the jacket together, so instead of only earlier on we will be grouting the annulus space between the leg and the pile.

Now you are going to grout from bottom but only thing is the pile inside space also needs to be filled with grout, so that you know you get the end bearing which is something that we need to achieve otherwise only depending on the skin friction you may not be able to transfer the load, so that is the grout the grout properties is something that you need to work out, the amount of grout required is enormous because you talking about largest depth, so one of the idea is the drilled and grouted piles we got 2 interfaces one is the grout to rock interface the other one is the steel pile to grout interface, we just need to look at both of them in design. The second one is the tension piles, even in offshore platforms we are doing analysis of jackets we could see that if there is a substantial horizontal load, the pile opposite will get the compression loading, the pile in the vicinity of the loading direction will actually get tension because of the overturning.

Now if the tension is so large that your skin friction capacity like I think most of you now you all are able to do the calculation for pile capacity both basically the plugged case unplugged case, tension and compression I think 4 cases you are able to visualise and do a configuration for a multi-layered soil. Now for tension you will not be able to take any end bearing and if you add the internal and external skin friction you will get maybe a smaller capacity depending on type of piles design. If you have design the pile as the predominantly end bearing pile means you are terminating the pile at a very good soil hard layer then the skin friction contribution is very small.

If the skin fiction contribution is smaller then that means then tension capacity is going to be very nominal, you will not have too much of attention capacity, so in a such cases when you encounter a large tension load on a jacket or any type of pile foundation it may not be sufficient to only drive but also you may have to anchor the pile I somehow means either you drilled and grout the inside to some depth where you can get sufficient pull out capacity or you drill a smaller hole like the one that we discussed like a micro pile, you drill a smaller hole inside another pile which is also difficult or you drill even a small hole inside steel wires, so that is what we are going to look at how we obtain our required tension capacity.

See to get a required compression capacity what we were doing? We were searching for a good layer, very hard soil layer can you drive and then go to the depth and ultimately the end bearing gives you the required compression capacity. Now we need exactly opposite when you look for that then you have to just think about what can we do? So that is where the tension anchor piles are very useful especially foundation for mooring systems you know if you if you look at some of the moorings for deep water structures like spar, TLPs and other forms of FPSOs you will see that the mooring load will be very large depending on what design condition you are doing there, so that type of tensile load you may have do this anchoring purely depending on only the skin friction capacity may not be sufficient, so if that is the case where we try to do this kind of tension anchor pile.

Modification to the same tension anchor piles is basically system by which we try to achieve (())(5:22) capacity by means of suction means the available water pressure which we will discuss in detail later on and then we have gravity-based foundations which is slight modification to the mud mat what we have learned, mud mat is just temporary foundation when you spread the area of loading under the jacket wearing gravity-based foundations we try to make it similar fashion because their mud mat is a foundation but the gravity load is a jacket weight we try to make the weight of the jacket as much as larger so that you get a stability there.

In here we purposely designed a foundation mostly using concrete for some cases we have still concrete composite structure that means bottom will be concrete and you have a steel frame you know such type of structures very few and then completely gravity means completely concrete with the base concrete with the Tower also concrete many of these platforms have been in existence and then the last one modification to the concrete gravity with a skirt very similar to a combination of this and this, we are trying to form a combination of gravity base with a skirt all around the foundation to obtain additional of instruction and the weight of the soil plus you get some kind of passive resistance against horizontal loading you know like very similar to retaining well I think we discussed about active and passive pressure last time in previous class.

So you can see here several cases of ideas these are all just ideas developed due to situation changing according to the necessity, so you can see here each one is applicable to individual case or you can come up with something new nobody is going to prevent it prevent you from thinking and developing a new concept depending on what the situation is, so we will go through some of the design aspect one by one and installation aspects in fact quite useful.

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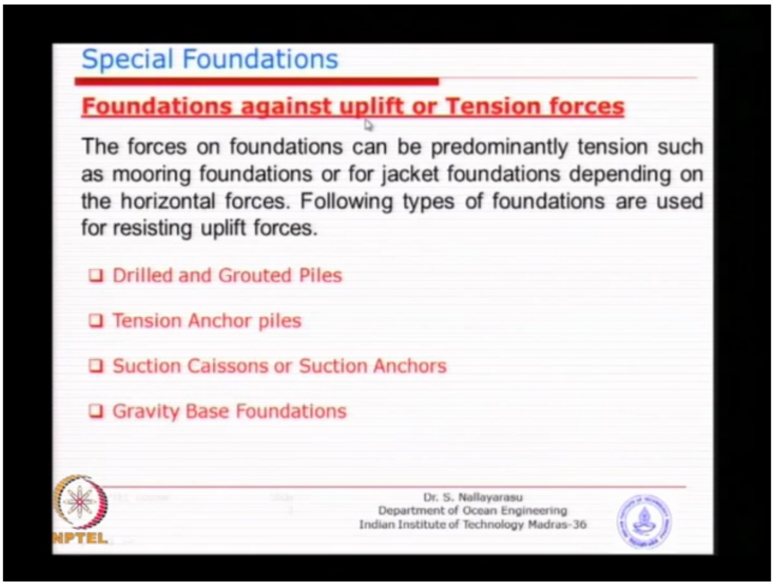

So primarily these foundation we are looking at you know the foundation design always we were talking about foundation design means against huge vertical load compression and also horizontal load that is what we have been talking about lateral capacity and vertical capacity mostly compression but some cases wherein you know the deadweight or the structure weight and the facility weight is smaller but the horizontal various too much then you will encounter such kind of situation where the foundation will try to get a uplift and that is basically the area where we are going to focus the design against uplift forces.

Onshore situations many of the cases will be there similar you know one of the pile will be getting compression the other one gets injured and depending on the loading direction. In offshore anyway the ratio of the horizontal to vertical load is larger than any structure on land, is not it? You know if you look at horizontal base here for an offshore platform could be as much as 30 percent of the vertical load, so in such case you can see the ratio in on land could be less than 1 percent you know because most of the structures are heavy and predominantly the vertical loads are going to the ones (())(8:52) whereas this is exactly opposite.

So these are the 4 cases we are trying to do design against uplift direct uplift or tension forces, in some cases some of the you know foundations may actually get compression at some time and tension at some other time, so you cannot purely design a foundation just only for tension whereas when you apply a compression load it will just fail, you understand the difference? You cannot design exactly only for one function because the same pile can actually get at a different type of loading conditions may get a compression loading, so in such a scenario we need to see whether the pile is able to take compression loading as well when the loading is reversal you are able to transfer the tension loading, so that is something very useful when we see this idea of tension anchor piles.

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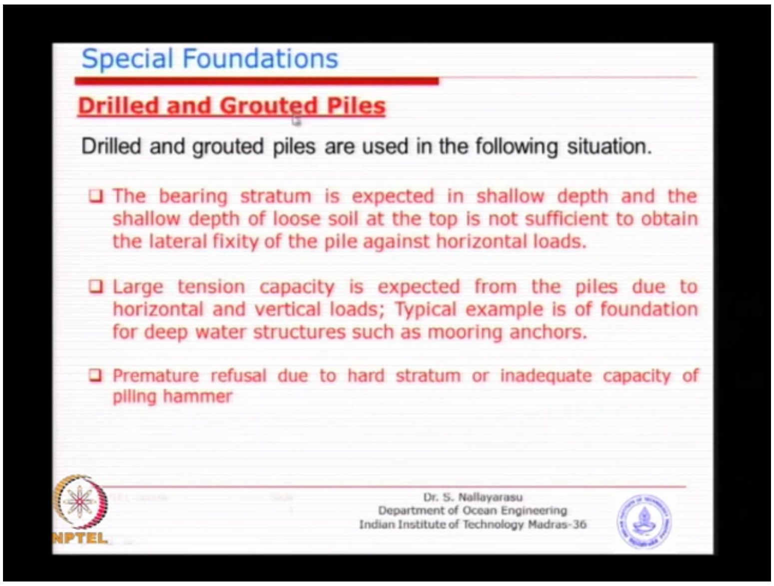

So drilled and grouted piles is very simple idea the bearing stratum is expected in shallow depth this this type of scenario do exist in several places where the top 5 - 6 metres or 10 metres is clay and then you have a very good rock you should not be very happy that I am getting a hard rock in 10 metres because I can just drive the pile to refusal to the depth and then get enough capacity even if you do not have tension load the problem is sufficient fixity against horizontal load may not be there.

For example you have a jacket of 100 metres water depth you just drive the pile to 10 metres when you apply a horizontal load pile may not get enough fixity against a bending and rotation, so the pile will actually try to rotate and topple the whole structure, so that means even if there is no tension load you may actually have to anchor this foundation into a bedrock in which case you will have to not possible to drive then you have to drill and then grout the situation there, so basically the bearing stratum decides what type of foundation that you are going to adapt within the pile foundation itself whether you are going to drive it, you are going to drill it or you are going to have anchoring you can decide. Of course the reason why it is going to topple is basically an idea that horizontal load is going to induce tension and that is where if you do not have an anchorage definitely the jacket will try to rotate.

Large tension capacity is expected from piles due to horizontal and vertical loads, typical example is deep water structures you know almost all structures you see there, you take a TLP the (())(11:37) are anchored to the seabed and it is under a heavy upward lift forces. You take spark platform or you take other types of mooring systems, most of them will be sustaining sufficient amount of tension loads to resist, so that is the idea where we need to find easy way of installation.

Number 1 if you go to deep water structure in 1000 meter water depth, driving piles becomes potentially difficult, imagine you have to take a hammer 1000 meter water depth and the pile is not going to come up above water the hammer has to go down, so in a situation like this not all hammers can actually go and drive the piles, usually to design a special hammer which can actually take 1000 metres of water hydrostatic pressure and take it to the near the seabed drive the pile and then make a connection to the structure somehow you have to connect the (())(12:35), so imagine this activity become, so you need to work out as sophisticated idea by which you can avoid a hammer, if possible, you can have a complete fabricated system you can avoid underwater work, so we need to develop ideas that can actually work for us and reduce the amount of offshore works, so that is where alternate schemes have been worked out.

The third one the premature refusal due to hard stratum which is what we saw the earlier class wherein prepare of or on-site decision to get sufficient capacity in case if the pile refuse earlier due to various reasons, it could be due to inadequate information on soil or pile drivability was not carried out for the hammer selected was adequate, soil condition different from what you have understood, so many things can happen, so premature refusal is something even after doing all this you may have variation of the soil within the whole group, if you have 5 4 corners one of the pile may get refusal because of the localised soil change or the soil parameters are different from or expected Boulder located in that location, so the inadequate capacity means your pile has not achieved sufficient penetration, so in such cases you can actually go for further drilling and then grout it.

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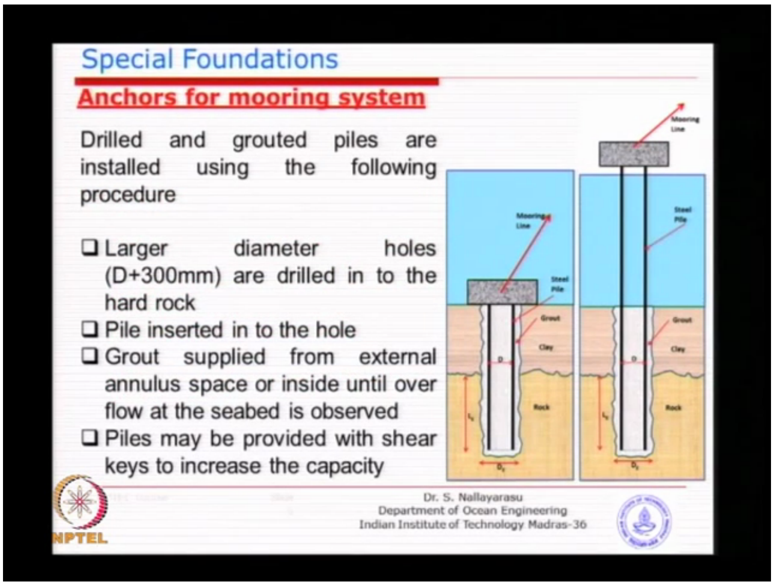

A typical anchoring system, for example if you look at drilled and grouted piles or a mooring system, you may have a mooring system anchor, wires connected in this way or you can actually have slack mooring system in which most of the loads are going horizontal you know depending on what is a mooring configuration, so if you actually drill out a hole larger than the pile diameter typically we give a gap of 75 to 150 mm between the pile surface to the rock surface not too much because then the transfer of loads from the steel interface to grout, if you have a larger grout you may not be able to transfer as a shear, it may become different load behaviour.

So normally we do not also want to make it too small, if it is too small like what we do it in pile to steel pile to leg, we talk about 30 mm, 40 mm, 50 mm may be difficult because you are unable to drill perfectly nice surface, so you may have undulation in the surface, so you want to give a little bit bigger that too we want to make larger diameter because your interface diameter become larger, the smaller the diameter because what you have is 2 interfaces, you have a steel to grout interface which is very good controllable because you can control the grout, you can also control the steel surface by means of shear keys.

Remember we were designing the shear key interface between the skirt leg and skirt pile, similar you can actually weld shear keys before election of piles into the de annulus or the hole, so you have a control over the interface strength between the steel pile and the grout whereas the strength between the rock and then grout control is not hundred percent, you have a control over grout then the surface and the nature of surface and the strength may have to be not possible to hundred percent control it, so the diameter can be increased so that pi D times L becomes slightly larger because pi D the diameter plays a major role in the capacity itself but if you make it too large also not good, the recommended is about 150 either side.

So you will get about 300 mm pile inserted into the hole, grout supplied from external annulus normally we supply the grout from the external annulus through here and normally you will have a grout pipe going and starting to fill from the bottom, now you cannot just start the feeling of the grout from the top allowing the grout to segregate go down the hole, so you have to insert the grout pipe to the bottom. Very similar to our bored concrete pile exactly same manner we start the grout filling from the bottom and start pumping the grout, if you are doing a deep water structure you have to pump with a high-pressure grout system, so that the grout will flow and start coming up and flow outwards exactly same as what we do for jacket skirt piles also. So once you see the grout flowing outside on the seabed means it is filled the hole between the rock to pile and pile inside the complete area is filled that gives you a conformation that there is no void space anywhere.

Imagine if you start filling from the top you do not know whether everything is filled 100 percent whereas if you start filling from the bottom using a grout port or grout hole with a pipe the grout is rising from the bottom for sure due to gravity the hole grout has to be settled. So normally we adapt this stage either through the inside or through the outside does not matter, one of the area you can use it normally we do it from the inside, so grout because our interest is to get a proper interface between the rock and the grout, so you make one grout pipe put it at the bottom and start allowing the grout to flow from the bottom on either side and just start growing up.

At the same time you can lift the grout pipe depending on the timing and you will conclude the volume of grout required to fill and you will also have the similar amount of grout prepared you know you normally calculate the volume just see that whether you are pumping in rightly or is there any wide space which makes the grout to flow laterally. You know sometimes what happen you calculate the volume of this grout requirement say 20 meter cube or 50 metre cube and you prepare the grout, you keep pumping the grout is not coming about the seabed means there is something not right because you may actually have a grout leakage through a wide and poor space keep going around unable to get the grout up so that is a situation where you know several cases you may have a loss of 20 percent 50 percent, so you will have to bring that grout extra.

If the rock is porous rock, if the intact rock is available then it is not a big problem, so grouted piles very few jackets, for jacket structure very few this this idea has been adapted because mostly before designing a structure example I want to go for concrete based gravity structure or jacket structure, the rock out craft will decide whether such type of foundation is feasible, which one is better but even after finding the rock out craft is there, if you decide to go for pile foundation only in such cases you will go into this kind of drilled and grouted piles.

Mostly if you have a very shallow hard rock stratum you will go into gravity foundation, but one of the typical situation is I think I have shown you some video the other day one of the platforms where in the top 30 to 40 metres is soft clay or medium clay and then rock, now you see the problem if the clay is only 5 to 10 meters then you can go for a gravity foundation and will not be a major problem but if you have 40 metres of clay and then you bring in concrete gravity platform, that 40 meters actually cause a major threat against bearing capacity as well as stability and that is the time you have a problem to go for concrete foundation even if you have a very good rock at 40 meters.

So you will go for steel foundation but then you are not able to achieve design penetration according to what you require, so you may have to embed the steel pile, so wherein you do this kind of design. You see this the side picture also very often we use this type of structure or coastal areas when you design a (())(21:15) or harbour wherein the horizontal loads due to either mooring systems because of (())(21:21) or due to berthing structures where horizontal loads are applied from the moving ship. So basically this type of foundations have been used many times in fact I have myself because as many of the locations in the coastal areas if you find (())(21:41) very close to the seabed just on the top you do not even have a clay layer so you can use this kind of foundation in case you cannot use the steel then you can use the concrete bore pipe and exactly the same thing we do bore it and use the reinforcement cage said of steel pile.

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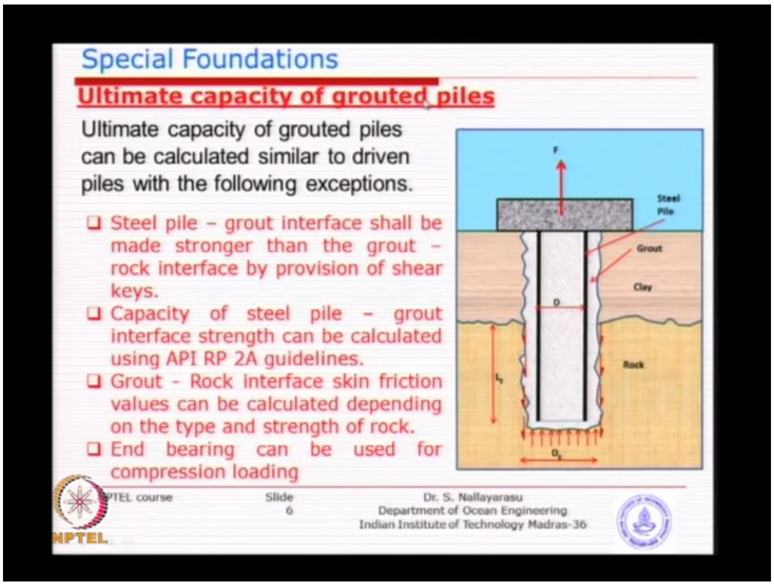

Ultimate capacity which we have just discussed about the interface between steel to grout you can leave it as plain pipe which is 0.138 mega Pascal I think we were discussing about the strength between the steel pile to grout 0.138 and 0.168 something like this for normal and extreme conditions but when you introduce shear keys in terms of weld bits or plain pipes or rectangular strips welded at a particular spacing then you have tremendous increase in the strength between the pile and the grout, so you could actually design them to make sure that in the whole system designed the grout steel is not weaker that means you do not want the design to be governed by your pile steel pile and grout interface but the other way round the grout to rock interface should be governing the design.

Now normally when you have a situation like this you do not want to take or it may not actually contribute the the soft clay or soft layer grout interface may be very small in terms of frictional resistance, so you can ignore them, so we call the reminder length which is called socket length, socket length is nothing but the amount of or the length of pile embedded into the hard rock which is called socket length, so you may get some amount of end bearing to some extent depending on what is the type of loading, if it is a compression loading sometimes tension loading sometimes, so you may have to take the compression end bearing into account the wise if it is a pure tension is then you actually do not have to worry.

The rock and the grout interface is something that we need to evaluate, what could be the type of capacity? I think when we were looking at skin friction between clay, you have alpha methods which I think most of you all are familiar. It depending upon the over burden pressure and the untrained shear strength, you could calculate alpha, alpha times CU you could find out what is the adhesion value or the skin friction value. Similarly for sand you have or beta method which is 30 percent, 50 percent, 60 percent of the overburden pressure was and also limiting values were given.

Now when it comes to rock is almost similar to concrete you take a case of a very hard granite rock, I think most of you are familiar with concrete strength M 20 concrete, M 30 concrete means is the number denotes the characteristic compressive strength of concrete, is not it? 30 mega Pascal is a is the compressive strength, so imagine you have 40 mega Pascal concrete and granite you will be able to see almost similar characteristics 30, 40 depending on the fissures and the whites in granite or any other rock.

If you have intact rock hundred percent no pores no whites then probably it is very similar to your high-strength concrete of 30 mega Pascal or 20 mega Pascal whichever, so you can see when you cut a concrete and try to do a grout interface very similar you can find out whether the rock will behave but unfortunately getting 40 mega Pascal type of granite for drilled and grouted piles if you encounter drilling itself will become problem, so you expect less than 30, 40 type of rock during your encounter the foundation system. So you make the grout stronger by making grout minimum strength of say 40 mega Pascal, so you are going to expect the rock definitely less than that, so in such cases what could be the expected bond value because you are going to make the surface of the drilled surface not very smooth, you are going to have rough surface.

So this we need to evaluate but we cannot possibly use the API equation what we have been using for steel pile interface to grout, we were using API equations or we were also using equation from Department of energy but we cannot use it because it is a different type of surface. So what we need to do is? Look at the literature and try to use…see one of such code which we have been using as the British code which is 8081, code of practice for ground anchorages especially for tension system which we have been using for some time. It recommends a skin friction value of 10 percent of unconfined compressive strength. When you actually take a rock core and take it to the laboratory and do a unconfined compressive strength very similar to your compressive strength test.

I think some of you might be familiar with concrete cube testing in strength of materials lab you know you do a compressive strength. Very similar only thing yes it will be a cylindrical specimen, so you do a unconfined compressive strength of rock and find out the FCU value and take 10 percent of that as the bond that is what is recommended by this code but seems to be very small you take a 30 mega Pascal concrete. Remember when we were studying about concrete design, bond strength is 30 mega Pascal concrete you will only get one mega Pascal for bond between the reinforcement and the concrete, so which is 1 in 30, so here you take about 10 percent means if it is 30 mega Pascal rock you only have 3 mega pascal is the…not too bad but then it also suggest depending on type of rock the porousness you could take values anything between 10 to 35 and leave it to the designer but as you can take as a minimum or conservative design 10 percent of the FCU value.

For a typical example 30 mega Pascal rock you can almost if it is a 5 mega Pascal in only you will get a very low value what will be there is 0.5 mega Pascal pitches 50, 0.5 mega Pascal basically 50 TPA, remember we were talking about bond strength between pile to clay, pile to sand varies from as low as very small value over 300, 400. In fact if you look at sand if you remember the limiting skin friction value is around 156, if you look at API table and for clay the maximum, the hard clay the very strong clay goes as much as 200 - 250 maximum, so if you take even hundred percent as the adhesion value, so you will not get anything more than 300 as skin friction between any type of soil to steel pile interface. So in here if you get a very good rock straight away you get 50 and if you have 10 percent of the granite rock you are going to get will very large value, so that is the advantages of grouted but then you have to pay for it.

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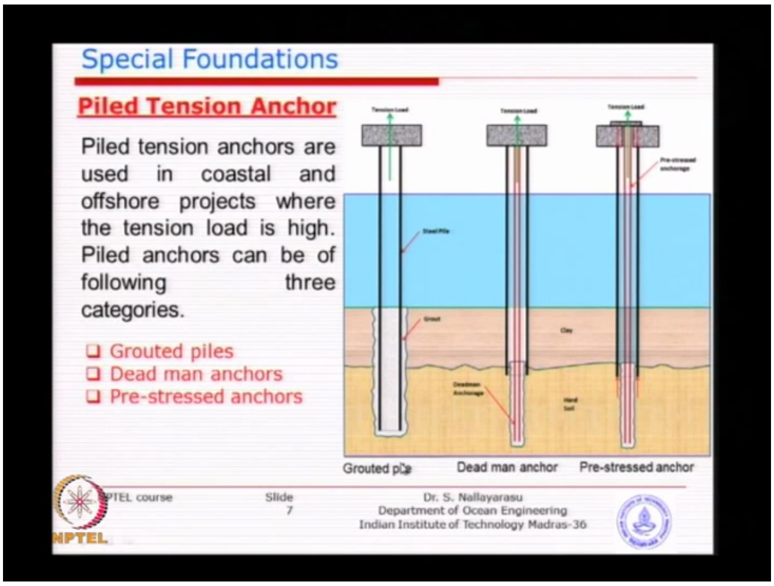

Coming back to this previous picture also you see here piled tension anchor is used in different scenarios, for example we have been talking about this particular is of just only insert the pile and grout it that means when the tension load is applied, it passes through the steel pile to the grout and from the grout to the rock by means of pure shear transfer. Imagine if this is not feasible you are unable to do this drilling because this large diameter, I do not have a drilling rig to do that exercise.

So I try to drive the pile instead of drilling, I drive the pile to refusal probably one or 2 meter if possible to drive it into the hard striatum stopped and then remove the soil inside something like this and then bring in a smaller drilling rig and do an opening inside which I can use it for either a micro pile which is very similar to the one that we discussed earlier on or I insert small diameter rods like reinforcement rods 20 mm, 30 mm rods bundle of them calculated in occurrence with the required pull capacity and you put it inside and bring them all the way up to the superstructure and then grout the pile and anchorage interface an anchorage rock interface.

So you see here the loads are going to be transmitted from the superstructure to the rods because it is already embedded and cast together so when pull is applied the rods will take care of the of course some amount of load will be trying to share with the pile itself but as it progresses downwards you can see here in this location pile and the anchorages are cased together with a the grout inside.

Now what happens is the load redistribution will happen, ultimately the tension will be taken by these wires because they are very strong and compared to simple embedded pile into a one diameter or 1 metre below it the hard rock, so what happens is the anchorages or the wires or the steel rods takes the tension load, transmit into the ground through the grout interface, so this we call it dead man anchor basically we are having a wire placed or a steel reinforcement place inside and trying to get the load transfer.

When you go to the right side one the difference is only just the wires or the steel reinforcements are stressed very similar to the pre-stressed concrete but your if you have studied, so instead of simply casting after pulling steel rods were required tension capacity to be estimated as the difference between the compression loading and tension loading because what we are worried is when this pile is getting compression loading in some stage sometimes, some relaxation will happen on the wires which is not very good because then it cannot take tension, it will take too much tension because it already has to go through the slack slackening procedure because whatever the compression has done damage it has to get extended.

So in order to prevent that the pile is to be designed for compression loading at some stage tension loading at different load scenarios, you pre-stress this anchor that means you pull these wires after the grout is fully strengthen or cured then you pulled this wires and then just anchor against the structure itself, the reaction is basically the pile transferring the reactions onto the hard ground you know so you are actually pulling the wire against the pile itself and then once you pull it to required capacity and put the wedges on top and then you have the top edge connection and grout and then concrete the whole structure. So this we call it pre-stressed anchor just to avoid relaxation of wires and you can do this when the pile is having both compression and tension capacity.

Typical example will be a berthing structure, I do not know whether you are familiar with…if you look at berthing structure in in a port or jetty, the same structure will be used for berthing and later you will connect the morning wire, so berthing means the front pile will get actually tension whereas actually mooring means the front pile will get compression exactly opposite alternating forces will occur due to change in direction of wind or due to change in direction of loading, so that is the type of situation wherein when you encounter such ideas you will have to design pre-stressed anchor, of course if you have a large diameter rods you are not really worried about this is slackening and relaxation but many a times we use small diameter rods bundle of them, so this pile the tension anchor any of these things you can design, depending on situation you can come up with ideas and implement them.

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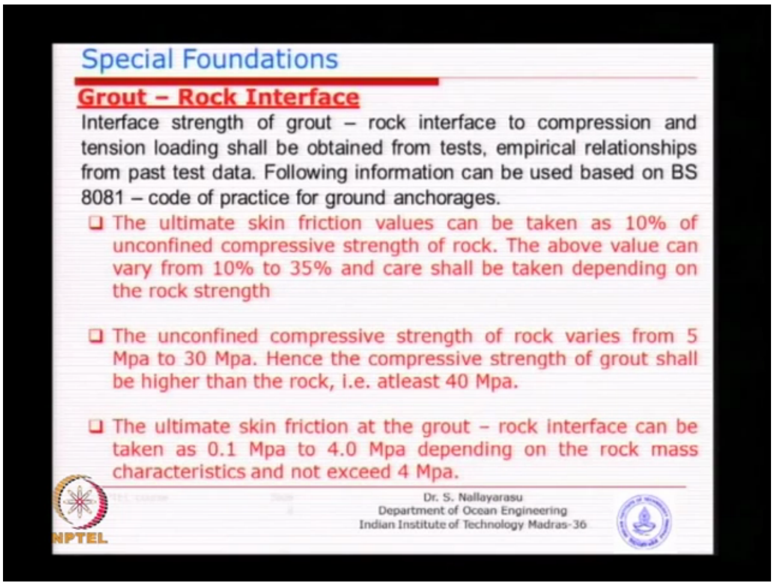

So we have seen this rock grout interface, the only information that is available for us is BS 8081 API does not give any guidance on the grouted steel pile or grouted rock interface, no guidance, so some of the literature if you review through you may get some information but most of them are scattered because not many locations you encounter such situations.

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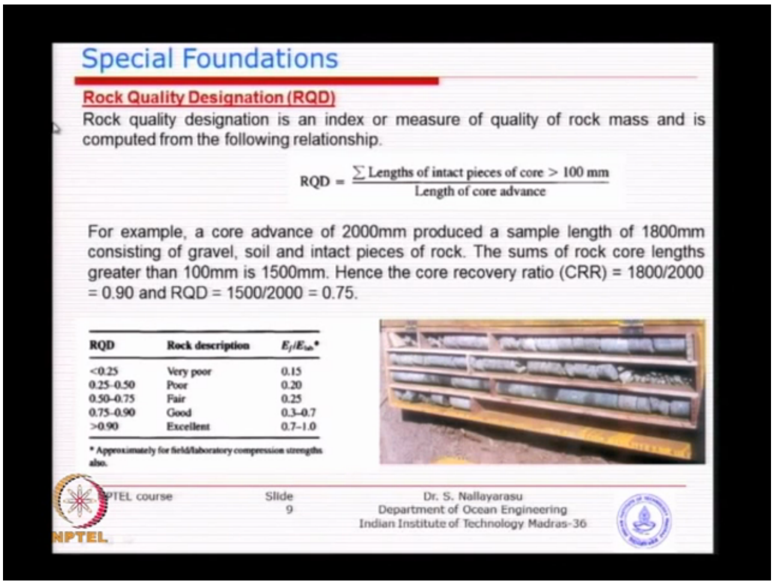

One of the important aspect which we need to understand when you are designing piles bearing in rock or embedded in rock or drilled and grouted you need to find out the quality of rock whether it is intact rock or fissured rock or you know loose boulders because they are going to change the way they are going to carry the load, so one of the important characteristics which we did not discuss during our early stage of basic soil mechanics of soil testing and borehole which I think I thought it will be useful, so as you drill through for soil sampling if you encounter rock then you have to use a different drilling tool, normally you cannot use split barrel or other methods you may have a different drilling core making device, so when you drill through and just take the core and then look at the whole thing put it on a table.

If you receive for example you try to drill at a stretch 2 metres, 3 metres normally about less than 2 metres typical about 1 and half to 2 metres 1 stretch you drill and take the core and you look at the core completely one piece of rock has come. Imagine if you get just you drill through 2 metres and you get to metres as an intact rock means it is hundred percent solid rock and if you get less than that indicates that there are fissures in over the rock is not intact is you an idea how we can treat the rock in terms of strength, so that is the idea behind is called rock quality designation we call it RQD and there are several literature available to relate this RQD with compressive strength and other properties, modulus of elasticity.

So that is why we have to define this. Historically how we define the RQD is the length of intact pieces of core greater than 100 mm, so if you have 2 metres several places it is broken but we just only count which one of them has length more than 100 mm, so if you have all or broken but everyone only broken pieces is greater than hundred then what you need to do is this add them all of them, you will still get example you have 200 mm, 300 mm, 500 mm but none of them less than 100 mm then you will get almost same as the intact rock but if you have lot of broken pieces less than 100 they will not be considered into the length of the rock for which will be used for rock quality designation.

A typical example is given here example if you have 2000 and only you are able to get the recovery is 1800 that means there are some portions wear loose materials, whitespace, so that means you can now understand this chance of 20 percent the rock is not very good the remaining is something better, so the rock core recovery ratio is 1800 by 2000, is not it? So some amount of…so that means 90 percent you are able to recover it means there is material remaining is not very good. Similarly if you look at additive of all the pieces together you get 1500 as your measure of rock quality divided by original of 2000, so the ratio is 75, 75 percent or 0.75 which gives you an idea that it is not 100 percent intact rock, some amount of broken pieces which gives an understanding to the geotechnical engineer that what type of rock we are encountering.

If you get one single piece of 2000 it is going to be a very tough further drilling and from a literature you can see here RQD versus type of rock description which is what we are looking for based on that is you can just do an interpretation trying to understand what type of rock than 25 percent very poor and greater than 90 percent is going to be a solid rock is similar to a mass concrete and also the ratio of the E value between the field and the lab you could find the ratios of this kind which will be useful hours later on when you do laboratory testing using unconfined compression test then you can extrapolate the values like this. The reason why we need to do this when you look at the literature many a times they refer RQD versus something, so you can use them for your design purposes.

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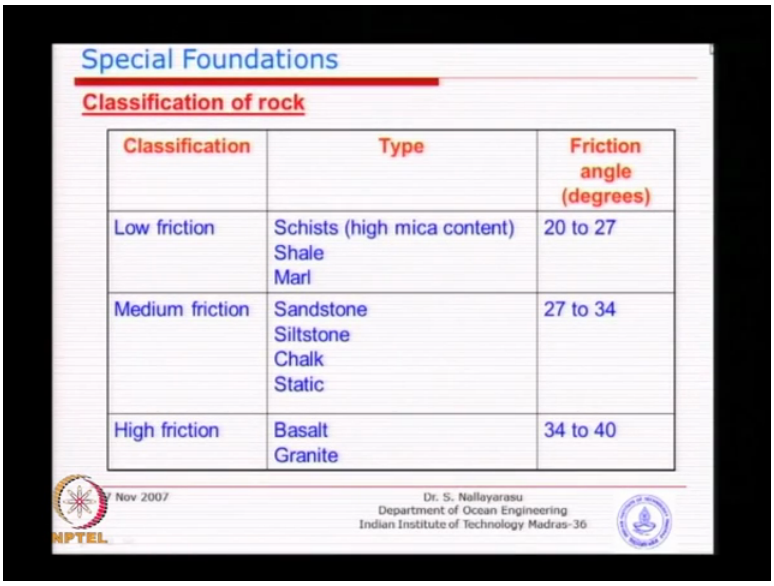

Typically you can see here the rocks can be classified into low friction, medium friction, high friction and you can see here granite goes to the extreme end with typical values of friction angle between steel pile to grout or grout to rock is numbers like this you know basically very similar tower angle of internal friction for sand say typical angle because later you will also use these numbers, some of the literature is if you look at it they will relate this friction angle with bond strength friction angle with end bearing values.