Earthquake Geotechnical Engineering

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Lecture 58

Ground Improvement Techniques: Vertical Drains

I welcome you again for this NPTEL online lecture on Earthquake Geotechnical Engineering. We are at lecture number 58, and we are discussing ground improvement techniques that is the last module of this course. And in the last lecture we started a topic ground improvement using vertical drains, we will continue with that, that means chapter number third of this module. Coming to vertical drains, we are going to discuss two things here, one case study from vertical drains and then we are going to talk one of the different issue which is called repeated liquefactions. That means a site which is already liquefied, it may be again liquefied, so that we are going to discuss in the second case. So, we are going to talk about a case study on the vertical drains and this case study data has been taken, let me acknowledge from my paper which is published in Indian Geotechnical Journal and the authors names are here and this has been like the data, what I am going to show is from this paper.

So, for the case study preloading coupled with vertical drains is a successful ground improvement technique for soft clay deposits. We already discussed in the last lecture that these vertical drains are mostly used for the clay deposit. And if we coupled preloading with vertical drains, preloading means basically as we discussed that it is basically your densification. You can treat it like preloading means you are densifying the soil using some load densification.

You couple this densification with vertical drains, then this combination is very successful. Though surcharge in the form of sand pits is conventionally used, vacuum consolidation is also an option. This lecture presents a case study on ground improvement using vacuum consolidation and surcharge preloading techniques. So, the two techniques will be used, one is called vacuum consolidation, and another is surcharge preloading. And this technique has been carried out near the shoreline in the city of Kakinada, Andhra Pradesh and this has been published by Ganesh Kumar et al which we have just discussed, so in the last.

Here in India large parts of the coastal area along the, it is more than 7500 kilometer stretch of the coastline of Indian mainland are covered with very soft clay deposits. So, near the coastlines, you have the soft clay which is marine clay. The shear strength of these deposits is very low resulting in very low bearing capacity. Apart from this, these soils exist with high water contents and possess poor compressibility characteristics. So, so many things are there with this.

First of all, bearing capacity is low and why bearing capacity is low? Because shear strength is low. Then another issue is they exist with high water contents because they are near the water bodies near the seashore, and they also possess poor compressibility characteristics. That means, like the water is when like when you talk about primary consolidation, it takes longer time to in primary consolidation because due to the dissipation of water. Surchar preloading after installing sand drains or prefabricated vertical drains is a common type of ground improvement technique adopted to this type of deposits. So, we are going to use vertical drains which is basically sand drains or PVDs.

So, the city of Kakinada which is situated about 170 kilometers south of Visakhapatnam and about 60-50 kilometers north of Chennai with a geographical position of this is with the latitude 17 degree north and 82 degree east. The foundation soil in this region is characterized by the presence of thick layers of soft marine clay deposits which we already discussed in this area which is typical for the coastal regions in India. In view of increasing demand for development of fertilizer plants and pores, there is a need for ground improvement. Two boreholes were drilled in the treatment area to characterize the subsoil conditions. So, these are the location of boreholes shown.

So, this is kind of a plan. In the plan, this whole area has been shown and then within this area, the two areas have been selected. Area number 1 which is here, the size of area number 1 is 10 meter to 10 meter. 10 meter is the length and as well as 10 meter width. And another area is also similar size.

Then you have two boreholes. Near area 1, the location is given. It is 1 meter apart and 1.5 meter away from the top part. Similarly borehole, another borehole is there in area number 2. So, these are the characteristics. What are the borehole details? The borehole details are here. The type of soils in both the boreholes are given here where water table is very, because as I said it is in the coastal region. So, water table is very shallow. It is about 0.5 meter from the ground surface. Then you have in the top part medium dense sand, below then fine loose sand, but fine loose sand and this get over at 3 meter. But below 3 meter, you have very soft black clay below 3 meter and this is the problematic soil. Then you have stiff clay. So, here in this borehole data, 3 to 13.5 meter, this is the problematic soil from the drainage point of view from here to here. And like you know that this try to protect the dissipation of water, because when it is inside the clay. So, this in clay and as a result if you have only clay, then the primary consolidation will take quite longer time. So, what we do? We provide some drain. So, the water inside the soil come out of this drain and the prefabricated vertical drains of 100 mm in width, that is 10 centimeter in width and thickness is 4 mm.

So, basically you have the kind of like 100 mm in width and then thickness could be 4 mm. And 14 meter length was installed. So, 14 meter is basically you say that it will cover quite much because here to here distance is 3 into 13, this is only 10.5 meter. So, 14 meter drain will be enough in the length.

In a triangular pattern at a center to center spacing 1 meter, the center to center spacing between two drains is 1 meter. There were two test areas of 10 to 10 meter which we already discussed. One test area was used for vacuum induced pre-consolidation treatment and the another is conventional surcharge induced pre-consolidation. So, pre-consolidation in this can be done by two way, one is using surcharge which is conventional or traditional method. Another method is vacuum induced pre-consolidation.

So, a total of 126 numbers of prefabricated vertical drains were installed in each of the test sites. So, you have two test sites. So, all together if I combine both the test sites, then the drains which is installed is 126. The typical example of prefabricated vertical drains that is PVDs are given here. So, this is your PVD which is in the form of like looks like a belt and here you have an anchor plate.

A plate is used like at the last end of the PVD and another end you have the rubber connector. This is at the anchor plate on one end of the PVD, another hand you have rubber connector and through this rubber connector you are connecting at 8 mm diameter polythene tube. This is tube which is 8 mm in diameter and this is made of polythene tube, so which will be used. So, here further details are given for particular with vacuum pump PVDs with as we said it can be used by two way one is using vacuum pump, another is like you have the surcharge loading. So, this is with the vacuum pump.

So, a vacuum pump is here and then you have water collection chamber. So, this is plan basically of PVDs. This is plan and this is nothing but your PVD. So, this is basically what you have. In the plan you have a row of PVDs.

So, for example, 10 PVDs are in this side and the 6 are this side. So, all these 60 PVDs in one case and the 60 PVDs are for the vacuum pump. Similarly, it is done for the surcharge loading also. So, this and the arrangement of PVDs for vacuum and surcharge treated area. So, A is vacuum preloading where you use vacuum pump and all the things here, another is surcharge loading where you do not require any vacuum pump.

What is done in the vacuum preloading? You have these are the PVDs, vertical drains like in numbers about 10 and then like water will come out of here, this is 1 meter thickness below fine sands and then ultimately the water will go on the top and then pass through the collection chamber at B. While in case of surcharge preloading, like you are directly putting this surcharge which is like sand fill. So, this surcharge is heavy compared to what you use as the sand fill here. So, all these consolidations are accelerated like due to this surcharge. So, the surcharge is, naturally the surcharge in this case B will be higher than the case A because here you are not providing any vacuum rather directly using the surcharge.

So, and this is black is very soft clay. So, this is basically if you compare this figure 3, 10, 11 and 14 meter. So, 14 meters can be compared here. So, the 14 meters are the 1 meter is in the top and that 10 meters in the down and then you have so, another like 3 meters on the top fine sand. So, fine sands can be compared like this is fine loose sand.

So, this is up to 3 meter and then you have below you have another 10 meter. Coming to this continue with this case study, the instrumentation in the vacuum treatment area which is considered of 3 numbers of settlement gauges and 3 numbers of lateral movement markers. So, in the short you will see that settlement gauges are referred with using SG while the lateral movement markers are basically LMM. The instrumentation in the surcharge loaded area consists of one settlement gauge, two lateral movement markers and water resistant pipe and two kesagrand type piezometers at approximately 7 and 12 meter depth below the ground level. The vacuum and the temperature gauges were separately fitted on the vacuum pump to monitor the vacuum pressure level and the temperature developed during the operation.

So, it is here in this figure for this is for vacuum loading, this figure is for the vacuum loading. Three types of the transducers, one is called settlement gauge SG, which is SG1, SG2, SG3 are given. Then you have the second lateral displacement markers which is LM, so you have LM1 and then you have LM2 also like LM2 and LM3. So, there are two SGs, three SGs, SG1, SG2, SG3, similarly LM1, LM2, so you have in this case three settlement gauges and three this lateral displacement markers. Then beside that you have LCPT light cone penetration test, which is second is here, first is here, so two LCPT two.

These are the instrumentation done for the vacuum loading case. Similarly the instrumentation has also been done for the surcharge loading case. In that case you have two piezometers, one is P1, P2 to measure the pore water pressure. Then you have SG pore settlement gauges 4 and then you have another water strength 5 WSP is also used. So, this was the instrumentation done at the Kakinada site for this measurement.

So, continue with this case study, the vacuum pressure was applied during the period of 8th October 2012 to 14th November, 4th November 2012. That means it is about approximately about a month for approximately 28 days. The surcharge pressure was applied on 20th November, so in between like after that surcharge pressure was applied. So, the first treatment was done using vacuum pressure. Then on 20th November the surcharge pressure was applied and monitored for next 24 days.

However, the test had to be abundant due to cyclonic rains and flooding due to the cyclone Nilam during North East monsoon period. So, there was North East monsoon period and cyclone name Nilam came in between and then test had to be abundant. During this period several parameters such as vacuum pressure, settlements, water pumping rate, diesel consumptions were monitored. So, all these things was monitored during this on the case study. The time which was taken by settlement under surcharge load is available for a period of 24 days and which is plotted as shown in the next slide.

So, in the next slide what is given here, you have one axis is time which is in days, and this is on the log scale which is varying from 1 to 10 days 100 or 1000 days. So, most of the recharge is up to 100 days. Then on y axis you have settlement which is in millimeter which is varying from 0 to 175 mm. So, what is this curve says predicted and measured time settlement curve under surcharge loading. So, what you have, the measured one are shown with the arrow here circles here.

So, this is measured one and these the curve shows the predicted one. One is based on back analysis, and another is based on the laboratories. Then C h and C v are coefficient of consolidation for the horizontal direction, and one is for vertical direction. And you could see that this is based on back analysis we have the higher coefficient of consolidation and as a result they will provide you the more settlement for the same time period. While on the based on laboratory test you have the lesser values compared to this one.

So, the settlement is relatively less compared to the back analysis. Assuming that the time settlement data follows a rectangular hyperbola, which is kind of situation here. The ultimate settlement is predicted as 149 mm. So, that is the ultimate settlement. So, you could see here this is the last value of the settlement here which is about 150 mm or 149 mm which is very close to the estimated settlements of 143.5 mm. So, this is for the one is estimated settlement which was there calculating using the consolidated test data. The prediction is better because the soil deposit uniform and the estimated parameters from the consolidated tests are reasonable. So, what has been predicted was the same as monitored or measured in the field. So, this was for the surcharge loading. Similarly, the data has been collected for vacuum consolidation also and the time settlement data under the vacuum loading is also shown in this figure.

The settlement has not reached steady value like this was the case earlier that it was like there it becomes flat. So, it will just started reaching here and even the settlement was large about 200 mm more than 200 mm compared to the last case here and this is based on the vacuum consolidation. This was done for the surcharge loading the time settlement response was assumed as a rectangular hyperbola and the ultimate settlement was predicted. The predicted ultimate settlement is 213 mm which is more than 149 mm which we have said. This value is much smaller than the calculated settlement of 734 mm which is expected under a vacuum of 86.5 kPa. So, here the like the value which is coming out this is 213 mm but the calculated settlement was going up to 734 mm and the predicted was very small.

So, this in this case the results was not so good compared to what was the surcharge settlement consolidation. So, this was all about the case history using what we say the PVDs and this case history as we discussed was from the Kakinara site near Visakhapatnam. So now we will discuss like one of the like different topics which is quite relevant to the ground improvement techniques only. What happens like you have repeated shaking and as a result you have repeated liquefaction.

So, one of the like what is called repeated liquefaction or in one word it is also called reliquefication. So, you can understand this like this. Suppose some site is undergone liquefaction already. Now, if another shaking comes what will happen whether it will liquefy or it will not liquefy.

So, naturally it will depends on two factors. One was what is was the state once some site get liquefied after some time there will be dissipation of water whether what is enough time is there to water is dissipated or not number one. The second the reliquefication will depends on your further loading whenever you are applying the load again whether this load is enough to cause liquefaction or not. So, that will also there. So, one of my research scholars like Gautam Padmanabhan he is working on this topic and already one publication has been done which is in ISET Indian Society of Earthquake Technology journal and which is also like in the form of a book from the Springer and the title of the manuscript is Assessment of Reliquefaction behavior of Solanisand especially using 1G shaking table experiments. So, some of the data which I am going to show is from this published paper.

What has been observed that since the 1964 Niigata earthquake and 1964 Alaska earthquake the liquefaction was focused primarily on the independent shaking events which we have discussed earlier also that liquefaction studies are lot of liquefaction studies have been done but most of them are on the independent shaking events. That means the shaking have come and you assume it is kind of fresh shaking. That means it is your soil condition soil is subjected to some shaking but we do not account for the prior history. Historic evidence witness that successive earthquakes are more catastrophic in nature. Successive means if after one earthquake or for example before an earthquake you get foreshock and then you get men shock and then again you get the aftershock that is basically series of like you know kind of motion shaking and they if you have more number of shakings compared to only one shaking which is independent shaking then chances are there that like the damage will be more catastrophic damage will be more.

For instance the extensive damage which was observed during the earthquake such as 2010, 2011 Canterbury in New Zealand. Then you have another example 2011 Tohoku earthquake from Japan, 2016 Kumamoto earthquake from Japan, 2018 earthquake Lumbok from Indonesia and 2021 Assam earthquake in India. The bears the testimony of two successive shaking events. There are evidence clear evidence that the behavior of like the soil will not be the same as the fresh shaking or like this. If you have the successive shaking if some soil is already gone liquefaction, then what happens in the next shaking we are going to see that.

The phenomena of repeated liquefaction can examined by has been also examined by other researchers like Padmanabhan and Maheshwari using shaking table experiment at earthquake engineering department of IIT Roorkee. So the therefore the experiments the table which has been used which we call the liquefaction table or simply people called as vibration table also which is used. This is one dimensional table where it is kind of a tank inside the tank you can put your sample with this let us say solani sand in this case and then this sand sample has been like soaked for saturation for quite some time until all the water goes inside the voids and then that extra water from the top is removed before any experiment is conducted. So it is basically the saturated sample of the soil is taken into what you have in this mechanism in this table you can move using motor in one direction horizontally. So it is called one direction 1D motion horizontal direction you can control the amplitude of the motion that is in terms of acceleration as well as you can control the frequency of excitation using this table.

And then you have three tubes in this table which is for basically for measuring of pore water pressure at different heights you put this at different heights, and this is tested. To monitor the pore water pressure at different depths three piezometers were embedded at 40 millimeter 200 mm and 360 mm from the bottom of the tank and name is the bottom middle and top piezometers. So, you have 40 mm, 200 mm, 360 mm. So, the 40 mm will be the distance from the base.

So, this distance will be 40. Then you have the distance from the base itself 200 mm of the middle this will be 200 mm and the top one from the base will be 360. So, at the top three piezometers are used. So, this table is used to and in this figure what is shown experimental setup with instrumentation has been shown in the part A and in the figure part B of this figure sample provision at halfway stays and location of piezometers has been shown in this case. For this study a total number of 24 shaking events and out of 24, 12 shakings was given with incremental and 12 with with decremental patterns were performed to study the reliquefaction behavior of the sand specimen. As for reliquefaction is concerned I think we already discussed that a site which is already liquefied in the past some earthquake or some event if it is again liquefied in the next shaking then it will be treated as a reliquefication.

Acceleration amplitude, shaking duration and pattern of the shaking events were varied and experimented using shaking table which we already discussed. Excitation frequency and relative density of the specimen were kept constant as 3.5 hertz and 25 percent respectively for all the events. So, you have excitation frequency as well as relative density. Coming to the test matrix of the study, how many numbers of tests has been conducted.

So, the two shaking patterns has been used to conduct the tests, one is called incremental shaking pattern another is decremental. In incremental shaking pattern three shaking durations which is 20 seconds, 40 seconds and 60 seconds has been used and this is called S1, S2, S3. And the acceleration amplitude is used is varying from 0.1 to 0.4 and this is incremental. That means, for each shaking duration you select four acceleration amplitude in incremental one is starting from 0.1 g then 0.2 g, 0.3 g, 0.4 g. And here you need to understand the same sample which is subjected to a shaking of 0.1 g will now go to subjected to 0.2 g and again therefore, 0.3 g or like this. It is not that it is a independent shaking. Then in case of decremental shaking you start from 0.4 g then you do the next shaking 0.3 g, 2 g and 0.1 g. So, I am going to discuss some of the results which are obtained for this test matrix from the like the test which has been conducted in our laboratory.

Here in this case maximum pore pressure ratio that is Ru and Ru you know that has been basically we discussed that earlier when we talk about liquefaction. Excess pore pressure divided by the confining pressure overburden pressures effective overburden pressure. So, this will be defined and sometime it is written as simply u divided by sigma v effective. And this is dimensionless quantity. If your excess pore pressure becomes as a equal to effective overburden pressure then you will say the Ru is 1 otherwise Ru will be less than 1.

So, what you see for example, for each pattern for incremental shaking pattern when you increase the acceleration amplitude whether s1, s2, s3, s1 is the largest like shaking duration is the largest for s3. So, it is on the top when s1 is on the bottom. So, like s1 is for 20 seconds, this 40 seconds and 60 seconds. Now for a given shaking duration if I increase the acceleration amplitude the pore pressure is increasing which is expected with acceleration amplitude. Similarly when we talk about decremental shaking pattern when I decrease the acceleration amplitude the pore pressure will decrease and again it will be higher for the higher shaking duration.

So, if I compare for the same acceleration value like in one like incremental and decremental shaking then you can see that for the same shaking duration the pore pressure ratio is higher for the same acceleration compared to incremental in the decremental. So, if we compare these points so that means, decremental shaking in this case like will give the higher pore pressure ratio which is more dangerous. These has been plotted also here particularly this has been plotted for shaking duration like 60 second shaking duration this plot has been done for the 60 second duration that means the top portion. So, what you could see that in case of incremental shaking the pore pressure was maximum peak value if we see the pore pressure peak value of pore pressure Ru.

So, naturally it will be maximum for 0.4g and so it is for 0.1g, 0.2g, 0.3g, 0.4g. The 0.4g values are here, but if this was if we directly shake independent if we apply independent shaking instead of the incremental shaking then for 0.4g you get this peak. So, this peak is certainly quite high compared to which can be observed compared to there.

So, that means, independent shaking which is given as a 0.4g is more dangerous than this incremental shaking and the reason being here in the incremental shaking what you get the advantage of that because when you shake from 0.1 to 0.2 the sample get densified. As a result in the when you go for the higher shaking period it does not pore pressure develop is not so much as was in the fresh shaking.

So, these some of the conclusions has been listed out later. Similarly when we talk about settlement we have discussed about pore pressure in these two figures two results, but when we talk about for the sheeted settlement then naturally the settlement will be also higher when the shaking duration is higher and the settlement will keep increasing when acceleration amplitude increases from 0.1 to 0.4g. So, in this case this was for incremental shaking, but in case of decremental shaking when you start your shaking from 0.4g first of all the value of settlement at 0.4g is quite high compared to the end value as this one. So, even we have compared this 0.4g and 0.4g there is a loss. So, that means decremental shaking will give the last settlement for the same value of amplitude.

Then further after 0.4g shaking if even I decrease the acceleration amplitude then also the shear this settlement is going to increase it is not going to decrease because settlement will be add up this is basically cumulative settlement it is not the independent settlement rather than after 0.4g already some settled.

So, the now when you applies whether 0.3g or 0.1g it will add to the already settlements. So, this was all. So, what the summary of the results which we have discussed acceleration amplitude and shaking durations are critical in influencing the relay extraction phenomena under both the incremental and decremental shaking pattern. So, it will depends on your amplitude number 1 and shaking duration. The larger shaking duration will provide more settlement as well as more pore pressure ratio.

Similarly higher the amplitude of acceleration it will provide higher which is expected. When the sand specimen is subjected to repeated shaking events the first initial event contribute to the majority of the soil displacement and sand density which you have seen particularly in the decremental shaking pattern 0.4g is contributing loss. And you get the beneficial effect of pre-shaking in case of incremental shaking and the associated incremental shaking also depends on the initial shaking event. The beneficial effect of pre-shaking in the terms of increase in liquefaction resistance was much pronounced in case of incremental shaking compared to decremental shaking pattern which we have observed. So, this was the kind of conclusion obtained from the liquefaction studies or you can say studies done for on the reliquefaction.

So, with this thank you very much for your kind attention and we have left with two more lectures other lecture number 59 and 60. So, we will continue later. Thank you very much for your kind attention. Thank you.