# THE EFFECT OF GROUND CHARACTERISTICS ON THE ASEISMIC DESIGN OF STRUCTURES.

by

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ABSTRACT. The first part of this paper refers to evidence from damage surveys of earthquakes that structural damage is greater with increasing softness and depth of alluvium, and varies with soil characteristics. An additional method of soil testing is proposed which would assist in the estimation of seismic settlement of structural footings. The second part discusses lateral earth pressure on model quay walls, and the determination of the mode of failure under vibration conditions. In the final part of the paper, an example of a model to represent a structure and clay soil characteristics is given, together with comments on raft type versus pile foundations.

INTRODUCTION. In the design of structures to resist forces from an earthquake, the first consideration should always be to obtain full information on the dynamic characteristics of the ground. Considering the subject broadly, the earth's crust is divided into soil and rock, and the soil is divided into sand, silt and clay. There are considerable differences in the vibrational responses of these fundamental materials depending on their mechanical properties, their mixture with one another, their moisture content and other conditions. It is well established from damage surveys of earthquakes, and reports drawn from all parts of the world, that the effect of ground characteristics on the behaviour of structures under seismic conditions is a matter of primary importance. Let us take two examples: first, compare the action of sedimentary rock with that of granular alluvial soil, and then compare saturated clay with consolidated materials, as foundations, together with the response of structures based upon them.

An informative example of the first case was provided in a striking manner in the New Zealand town* of Napier during the great Hawke 's Bay earthquake of 3rd. February, 1931, (I). Napier is located on the coastal boundary of a practically level alluvial plain with a hill of considerable area and some 100 feet high overlooking the town. The business area is on the alluvial flat and is founded on mainly fine and coarse gravel to a depth of approximately 85 feet. The elevated feature called Bluff Hill consists of limestone of Tertiary geological age. During the major earthquake the difference in damage to structures was very marked. On Bluff Hill, except for one notable case of poor construction, there were no serious foundation troubles, and the damage to structures was comparatively light. On the alluvial flat there was considerable earth movement and serious settlement of ground. This caused heavy damage in foundations and, in turn, heavy damage to superstructures. There was considerable loss of life in this area due to the earthquake and to the fire that followed. Structural damage was very closely related to ground conditions.

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An impressive example of this second comparison of soils mentioned above is provided by the report on soil conditions and damage in the Mexican earthquake of 28th. July, 1957 (2). In this report it is stated that the Modified Mecalli intensity was a maximum of VII, in the vicinity of the epicentre near Acapulco Bay on the Pacific Coast. At Mazatlan, a town 43 miles away, which was founded on consolidated materials, the MM intensity was V, while at Mexico City , at 163 miles distant, or four times further away, the MM intensity was VII in the lake area of saturated clay where tall buildings stand. Again at San Remon, a village 46 miles distant from the epicentre, and founded on sand, the MM intensity was VII. This should be compared with damage to towns at similar distances from the epicentre and founded on consolidated materials, where the MM intensity was only V.

In other examples of earthquake damage, the cause has not always been recognised at once, and the damage has been regarded as due to exceptional intensity of ground motion, instead of being attributed to differential settlement. One of the most obvious cases of differential settlement, however, was demonstrated in the modern reinforced concrete Daiwa building in Fukui, Japan, on 28th. June, 1948, where beams of the first floor showed a difference in level of 25 inches after the quake. This building was founded on 300 feet of sand, alternating with clay strata, and is referred to by Housner (3).

In commenting generally on the above, two main types of structural damage are, firstly, that due to lateral forces and horizontal movement of the superstructure, and, secondly, that due to induced vertical forces, and general differential settlement of the substructure. It is practicable to accomodate stresses in the superstructure resulting from horizontal movement, as was shown by the almost entire absence of damage in the 43-storied Latino-Americano building at Mexico City, but it may not be practicable to accomodate stresses arising from slight differential settlement in foundations. Differential settlement is common in sandy soils which are lacking in density, as in the case of Napier and San Remon. This represents a difficult situation because the position is not always apparent from the usual shear strength and penetrometer tests. It therefore becomes important to check the density of sandy soils in a site investigation.

Summarising the reports then, we have reliable information that

(a) buildings founded on certain granular or sandy types of soil can suffer approximately twice as much damage as those founded on harder compacted soils at the same distance from the epicentre.

(b) buildings founded on soft deep clay soils suffer damage which is very much greater than those founded on consolidated ground. Even at distances greater than 160 miles from the epicentre, major damage is possible to certain types of structures founded on very soft clays.

It is with regard to the first type, that is, sandy soil, that the writer believes that there is scope and a definite need for improvement in site investigation, and in the preparation of a soil foundation to withstand earthquakes. Suggestions for improvements are therefore made, firstly with reference to site investigations and associated laboratory tests by a proposal for the addition of the "Isolated Vibrating Load" test (referred to here as the I.V.L. test.).

SOIL TESTING FOR DYNAMIC SOILS. Relative Density Test.

In granular soils, the degree of compaction, and not the shear strength, may govern the design of foundations and site preparation to resist earthquake forces. The relative density test is the one most favoured in determining the degree of compaction before and after the use of compaction methods. This test is a most valuable one for that particular type of soil and, as its use is becoming common, the expression for relative density is given below. Three dry unit weights are required: the minimum, the maximum, and the natural (in situ) dry unit weight. The minimum weight is that determined in its loosest possible state, and is found by pouring the soil through a large funnel while it is held at the apex of the conical sand surface. The maximum is that in its densest state produced by the Modified Proctor Compaction method. The natural dry unit weight is found from undisturbed samples. If W is the dry unit weight, and the subscripts L, D, and N refer to the soil in loose, dense and natural states, the relative density expressed in terms of dry unit weight is:

$$D = \frac{\frac{I}{W_L} - \frac{I}{W_N}}{\frac{I}{W_L} - \frac{I}{W_D}}$$

In seismic areas, the degree of compaction (relative density) necessary at a particular site is generally taken as 85 to 90 per cent. Site preparation and field compaction methods are now available for economically producing this specified degree of compaction, provided the soil is clean, free-draining and granular. (3a). It is understood, however, that there is a definite limitation in the use of methods such as "Vibroflotation" to soils having the above qualifications. Moreover, it is quite evident that relative density tests are unworkable in soils containing 5 to 10 per cent of silt or clay, and this may be seen to some extent in Fig. I (c), where the Napier sample has a low relative density when compacted by vibration and yet has a high resistance in the I.V.L. test.

ISOLATED VIBRATING LOAD TEST.

The writer has extended the use of an experimental arrangement similar to that used by Professor Okamoto (4), to become a standardised test on granular soils which is here called "The Isolated Vibrating Load" test. This test is designed to assist the designing engineer in identifying granular soils which are liable to become troublesome in regions subject to heavy vibrations, and its value may be best judged by considering an example as described in the following study.

For the purpose of illustrating the essential characteristics of various types of granular soils, three typical granular soils have been selected, two being from the Wellington City area, and one from the City of Napier.

A STUDY OF PARTICLE DISTRIBUTION AND RELATIONSHIP TO SETTLEMENT.

We begin with the study of the distribution of grain size and sieve analysis in Fig. I (a).. We have plotted the two Wellington sands, one a crushed sand, and the other a beach sand from Lyall Bay, having a grain size of limited variation. These sands generally produce a typical curve which is only slightly inclined from the vertical. These should be compared with the curve for the Napier sample, see Fig. I (a). The main feature of this sample is its well balanced grading, shown by its general uniform inclination. Reference now to the graph in Fig. I (b) for each sample in turn will reveal the fact that the samples having limited variation in size of grain are not resistant to subsidence of model footings when vibrated, and in fact, the amount of subsidence or settlement is 3 or 4 times as much as that for the sample having a well balanced curve in the sieve analysis. There are two important conditions. This statement holds true

(a) when the material is composed of strong metallic particles of similar unit weights.

(b) when the amount of compaction is comparable in each sample, that is, when their relative densities are similar, if dealing with granular soils.

The presence of more than 8% to 10% of silt or clay in the sample renders the rule inoperative.

It was found that vibration alone would not produce maximum densities comparable with the Modified Proctor Compaction test. These values are plotted on Fig. I (c) for the purpose of obtaining the relative densities of all samples after vibration preparatory to the I.V.L. test. This preparation consisted of vibrating the sample at 25% G for three minutes, and then at 36% G accelleration for another three minutes, when it was considered that all settlement from vibration had ceased. At this point, the sample is ready to undergo the I.V.L. test. If it is of interest to study how the vibrating load acts upon the sand particles, the I.V.L. test can be made in a glass box, and the grey sand inlaid with vertical columns of white sand. In this way, one secures a picture of the movement of the sand particles in the area under the load, as is described in some detail in the following experiment.

<u>A MODEL EXPERIMENT ON A RIGID FOOTING IN COHESIONLESS SOIL.</u>

A special box was made with plate glass sides and this was fastened to a table capable of being vibrated horizontally. Dark grey sand was placed in the glass box, in doing so, six vertical columns of white sand were inlaid in the dark sand so as to present a regular pattern which would show through the glass. At this stage the sand was considerably compacted by means of 6 minute vibration. A large wooden frame was screwed to the box so that a steel shaft could be guided through the frame in a vertical direction during the course of the experiment. At the base of the shaft was welded a flat steel plate $2\frac{3}{8}$ inches x $3\frac{3}{4}$ inches and $\frac{1}{2}$ inch thick. The latter represented a rectangular rigid footing, and this was placed on the surface of the sand as was shown in Fig. I.(d) Owing to the previous compaction by vibration, the sand showed no visible settlement when the full load of approximately 30-lbs was taken. The outside surface of the plate glass was marked in the manner shown in Fig. Id by marking dots of white paint to correspond with the centres of the white columns of sand, and to act as reference marks for the subsequent lateral movement of the sand particles in the box. The original position of the column footing was marked in white paint, together with white dots,at $\frac{1}{2}$ inch intervals along the centre line of footing. An Ames dial was clamped in position on the shaft in order to obtain readings of settlements.

The vibration apparatus was arranged in the same manner as that shown in Fig. 8. The eccentricity adjustment was set and fixed so that the amplitude of movement was 0.215 centimeters. The intensity of loading in contact with the sand was 0.20 tons per square foot.

The shaking table was set in motion and immediately brought to an acceleration of 250 gals with a period of vibration of 0.185 seconds. The progress of the footing as it settled in the sand was marked on the glass sides of the box in white paint and photographed as shown in Figs. 2 and 3. After 100 seconds of vibration, the footing had settled approximately $1\frac{1}{2}$ inches as can be seen in Fig. 3. The sand was dry up to this stage and it was found that no further settlement could be obtained by vibration. The sand was then submerged in water and vibration was continued. This produced a further settlement of $\frac{1}{2}$ inch before all settlement finally ceased.

From this experiment, it is clear that something more than unloaded vibration tests are necessary before making a practical decision on design loads for footings. The main feature of the last experiment is the demonstration of the fact that the supporting sand particles travel sideways away from the footing and therefore confinement conditions surrounding a footing in practice would apparently exert an influence on the rate of settlement. Also it is felt that variation in the width of footing would have an effect on the shape of the "bulb of disturbance". This is apparent from the photographs, as the sideway travel of particles is larger than, and only slightly influenced by the vertical displacement. The above experiment gives an insight into the mechanical action of particles of cohesionless soil in sustaining footing loads and in compacting the area around the footing. Small scale standard tests, such as the I.V.L. test may be used in practical design in an impirical manner, for comparative purposes only at first to test the comparative loading capabilities of areas where some definite change in strata and conditions are noted in any site investigation.

As previously mentioned, Professor Okamoto carried out some experiments of the same type in 1954, and his results are partly summarised in Fig.4. The lefthand graph of Fig. 4 is plotted for settlement versus acceleration for a footing load of 20.8 kgs., and the righthand graph records the same information for a load of 29.1 kgs. A most significant change in the rate of settlement always appears at about an acceleration of 300 gals.

## SHEET PILE WALLS.

The use of sheet pile walls is fairly common in the design of earthwork bounded on one side by estuaries or harbours. The basic requirements for stability of such a wall are that the resisting forces should exceed the actual forces by a suitable factor of safety. The forces resisting the direct overturning of the wall are provided by an anchor and tie, and the forces resisting sliding are provided by the depth of penetration of the sheet piles and the shearing capacity of the soil at the toe. Before dealing with the dynamic experiments on models of walls, it would be well to examine briefly the static condition with regard to failure planes which occur when rotation begins to take place. The photographs in Figs.5, 6, and 7, are more or less self explanatory. In Fig. 6, a simple rotation of the wall about its base is made to represent the yield of an actual wall under static conditions of lateral earth pressure. The wedge action instantly reveals itself in the pattern of the alternate layers of white and grey sand. Failure is occurring on all planes that have a slope of 45 plus $\phi/2$ degrees to the horizontal. If the same wall were moved laterally without rotation, approximately the same angle of failure would be produced, and would have a value of 62°, if $\phi$ equals 34°. The great disparity between angles of failure for the above case, as compared with the dynamic case, is shown in the following experiment.

## A MODEL OF A SHEET PILE WALL SUBJECTED TO VIBRATION IN COHESIONLESS SOIL.

Fig. 8 shows the general arrangement of the experiment, and Figs. 9, 10, and 11, show the results of model experiments representing a sheet pile wall subject to lateral vibrations in cohesionless soil. The model was constructed as follows: the tie, which consisted of a strand of round-sectioned rubber, was fixed to the model wall and passed through the wall to the box where it was firmly anchored. The model wall was made of solid rubber ¾ inch thick, and covered with white paper to make its movements clearer through the glass. The depth of toe penetration into the sand was kept at a minimum to ensure some movement taking place and to enable observations to be made on the mechanics of failure. For the latter purpose also, the face of the grey sand against the glass was inlaid with vertical columns of white sand both in front of and behind the wall. Then the glass was marked with white paint to fix the original position of the wall, the columns of sand, and the levels in front of and behind the wall. It will be observed from Fig. 9, that the wall was slightly curved in taking the lateral pressure in the static position before starting the dynamic test. After 20 seconds of vibration, at an acceleration of 250 gals, the pattern of the active and passive forces in the sand are beginning to take shape. It will be observed that the resisting forces of the tie and the toe have yielded an almost equal amount, so that the wall has moved laterally while still keeping vertical. After a certain build-up has taken place at the toe, the anchor tie was manually released to simulate failure of an actual anchor tie. The planes of shear failure, before and after release of the tie, are seen to be parallel and at an angle of about 35° to the horizontal. The second plane was brought about by bending the rubber wall about its built-up support at the toe, The angle of the plane of passive failure in front of the wall was found to be about 18° which is also less than that for the static case of the passive Rankine state, and shown in Fig. 12.

The two main conclusions of the experiment are that the active pressures under dynamic conditions exceed those of the static case, and that the active wedge forms behind the wall has a line of failure which is much flatter than that of the static case. Any anchorage, therefore, must be placed at a safe distance beyond the effect of the actual wedge. In addition, some extra length should be given to the anchor tie to allow for the reduction of shear resistance resulting from dynamic conditions. The increase in cost to provide for these practical changes would be brought about mainly by the lengthening of the tie and would not be excessive. It should be noted, however, that the above conclusions are based on an idealised soil, and are qualitative only.

## GRAVITY QUAY WALLS.

The behaviour of gravity walls under earthquake conditions can best be studied from full scale examples of partial failures, but models are informative and useful in relation to certain aspects of general behaviour. Earthquake damage reports show that there is a tendency for quay walls to tilt and slide rather than to overturn, but obviously a great deal depends upon the toe conditions as regards both depth and shear strength of the soil. Gravity walls which chiefly depend on weight for stability, frequently have anchor ties as well to assist in the resistance to lateral earth pressures. The facts recorded by Amano, Azuma and Ishii (5) show that as in the case of the Shimizu Harbour quay wall, the anchor tie may be one of the most important items. The usefulness of such well illustrated details of partial failures, as are contained in the above report, cannot be over emphasized. It is noted that some of the conditions and dimensions are similar to those of New Zealand harbours, but there has not been an earthquake of sufficient severity to test them thoroughly. It was therefore considered useful to make a model test of a typical wall in New Zealand on a vibrating table.

The object was to study at least some of the points of general behaviour of a wall in a simulated earthquake of major intensity.

## MODEL OF A GRAVITY WALL SUBJECTED TO VIBRATION IN A COHESIONLESS SOIL.

The model was built of solid rubber to a scale of 3/16 inch to a foot or 1:64. As far as the "mass" ratio was concerned, the forces were not proportional at all points with the prototype, but the total weights were sufficiently proportional to give a qualitative result. Froude's law of similitude states that the "time" scale must be equal to the square root of the "length" scale so that the period of vibration of the model at 0.185 secs. corresponded to 1.48 secs. in the case of the prototype or a ratio of one in eight (1:8).

The period of vibration of 1.48 secs. is reasonably near the dominant period to be expected in an actual earthquake in the harbour locality. The acceleration imposed on the model will be 250 gals. which is just within the limit of proportional behaviour of the soils.

Fig. 13 shows the model before starting the test. The figures indicate inches, and reference marks are painted on the glass to record the starting position of the model and that of the white inlaid sand. Fig. 14 shows the position after 1 minute of vibration at the period and acceleration given above. Rotation of the wall is shown by movement at top of the model. No actual plane of failure has yet developed in the sand, although the surface of the sand behind the wall has subsided slightly and there has been a general lateral movement to the right. Fig. 15 shows a rather advanced stage of failure when the vertical columns of white sand had moved one complete space distance down the plane of failure. During the rotation of the wall, the toe has subsided and moved forward. Fig. 16 is a diagram which illustrates the various stages of the wall overturning and the angle of inclination of the line of failure as being 35° to the horizontal for both active and passive pressures. It is interesting to note that the same inclination occurs for the plane of failure in Figs. 11 and 15. As a matter of interest also, Fig. 17 has been drawn from information given in the paper by Amano, Azuma and Ishii, on aseismic design of sea walls (5), in order to show that his same angle of inclination of plane of failure probably occurred in a quay wall failure at Shimizu Harbour in 1930.

The main conclusion of this experiment is similar to that of the sheet pile wall experiment in that anchor ties should be of sufficient length to pass beyond the region of the active wedge. The experiment also draws attention to the great importance of the toe conditions in maintaining stability. There is sometimes a demand for greater depth of berthing facilities in the vicinity of the toe of quay walls. The obvious danger of dredging near the toe is well illustrated in the model.

MODEL INVESTIGATION TO DECIDE ON EFFECTIVENESS OF RAFT FOUNDATION IN SEISMIC CONDITIONS.

In foundation design for stiff clay soils of moderate thickness, there is usually no fear of sudden settlements under dynamic loading. The main problem is frequently when the stiff clay strata is underlaid by softer clay and continuous loading can cause a long term settlement. If, on investigation, it is found that such conditions can be satisfactorily dealt with, the question of seismic loading should then be considered. Such was the case in a recent industrial building design where the loading was relatively high and fairly heavy construction was required to accomodate heavy locomotive and gantry loading. A thorough site investigation was made and the shear strength of the soil was determined at all levels over the whole area. The usual spread footings were first considered as a foundation but, on analysis of soil tests, it was found that the size of the footings would have to be unduly large unless assisted by a heavy system of foundation tie beams. This requirement virtually changed the foundation into a raft in which deep beams formed an essential part.

The cost of the raft foundation was found to be much less than that of a piled foundation which would have had to be driven to a lower consolidated strata independent of soft clay. Then the question of seismic loading was considered, and it was felt that a model might prove useful in reviewing all factors.

Two gelatine models were made of the soil and the shear strengths of the soft stiff clay stratas were reproduced in correct proportion at the appropriate scale depths. Two alternate models of the superstructure were made in perspex, one being the piled model and the other being the raft model. In these models the elastic stiffnesses of each part of the frame, foundation, or pile construction was correctly proportioned to reproduce the stiffnesses of corresponding parts of the prototype. Photographs of the models are shown in Figs. 19 and 20.

When the models were subjected to lateral vibration, it was found that (I) the piled foundation model did not contribute appreciably to the lateral stability, and (2) the surface interruptions due to the continuous pits in the floor created a distinct tendency to split in the case of the pile foundation model, whereas the raft foundation model proved more affective against this action. Fig. 18 shows the piled model after a test and the fractured corners of the pits. Fig. 19 shows the special features of the raft foundation model which were effective against such action.

It was decided to adopt the raft foundation proposal. It was found to be more important to strengthen the building floor and foundation as a horizontal ribbed unit capable of taking vertical and horizontal forces rather than to depend upon piles to take such forces.

It was found that, in tying the foundation together, any notch effects or offsets in alignment need smoothing out as much as possible.

## CONCLUSION.

It has been established from reports of earthquake damage that ground characteristics greatly influence the type and amount of damage which occurs in an earthquake. The experiments described, show that a close examination of granular soils in particular during the site investigation stage of building construction would assist the design engineer in avoiding differential settlement which is obviously one of the most destructive types of seismic damage.

The conclusions with regard to quay walls are given under their respective headings, but it is emphasized that any deficiency in the toe conditions are a potent source of trouble, as well as deficiencies in length of anchor ties.

In designing important buildings of moderate height on soft clays, the established soil mechanics procedures for vertical loads are the first consideration and model studies can form a good source of information for special cases or special conditions, with regard to aseismic design.

In general it has been found that models, when carefully reproducing the prototype, and when used in the light of earthquake damage reports, form a good basis for a realistic approach to problems of structural design.

## ACKNOWLEDGEMENTS.

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## BIBLIOGRAPHY.

(1) Report on the Hawke's Bay Earthquake, 3rd. Feb.,1931. N.Z. Dept. of Scientific and Industrial Research, Bulletin No. 43, 1933.

(2). Soil Conditions and Damage in Mexico Earthquake, July 28th., 1957. Bull. of Seism. Soc. of America, April 1959.(by C.M. Duke and D.J. Leeds.)

(3). Geotechnical Problems of Destructive Earthquakes, by G.W. Housner. Geotechnique Vol. IV, Dec. 1954, Inst. of Civil Engineers, London.

(3a). Loose Sands - Their Compaction by Vibroflotation, by E. D'Appolonia. Special Technical Publication 156, published by A.S.T.M, 1953.

(4). An Experimental Study on Settlement of Foundation Due to Vibration, by S. Okamoto, and K. Seimiya. Proc. 4th.Japan, National Congress for App. Mech., 1954.

(5). Aseismic Design of Quay Walls in Japan, by Amano, Azuma & Ishii. Proc. of the First World Conference on Earthquake Engineering, 1956.

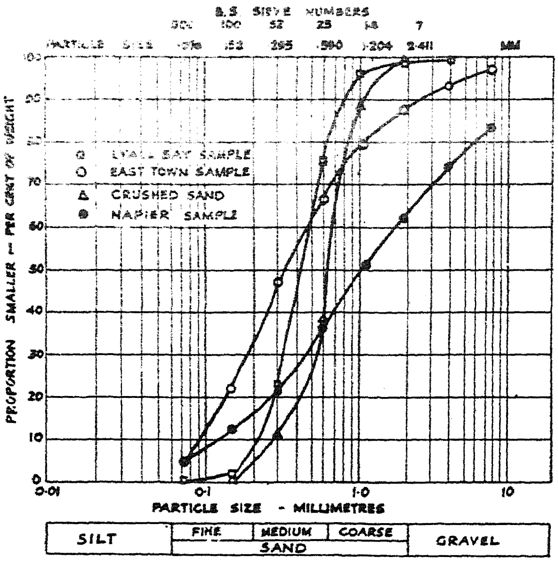


Fig. 1 (a) Soil Tests for Particle Size Distribution.

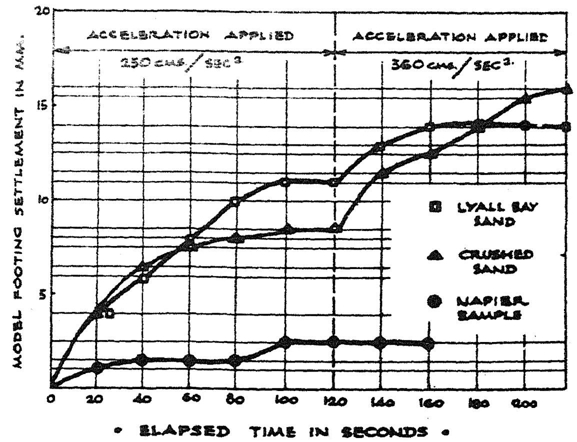


Fig. 1 (b) Results of I.V.L. Tests.

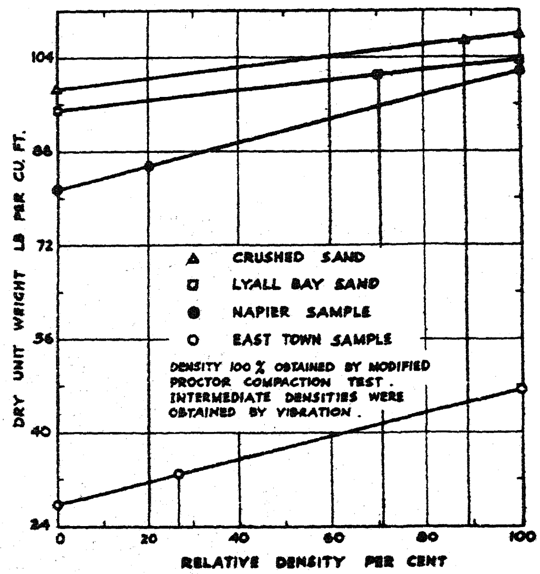


Fig. 1 (c) Results of Relative Density Tests.

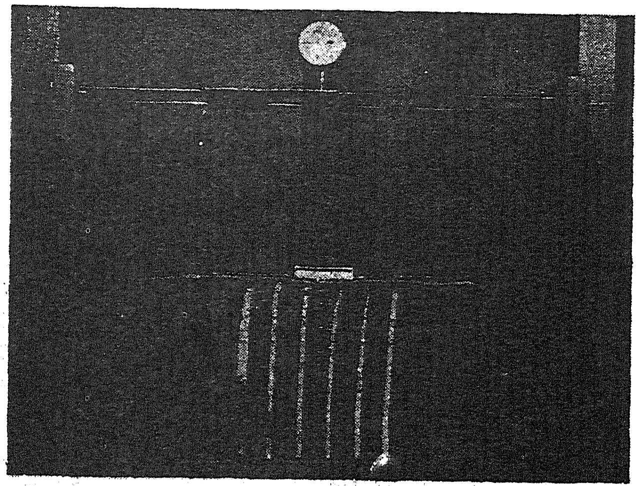

Fig. 1 (d)
Model Footing before starting, I.V.L. Test.

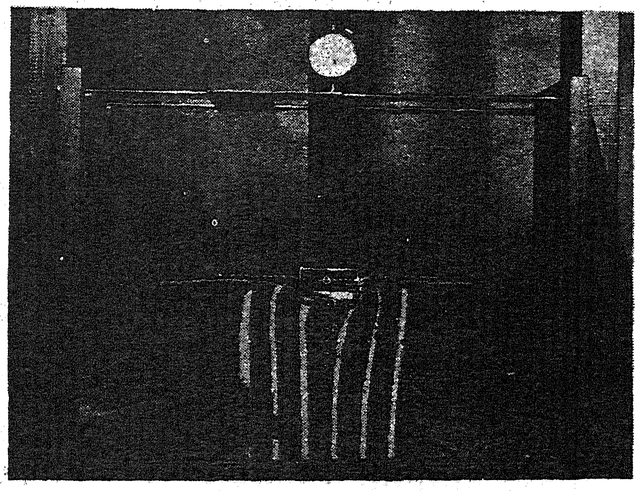

Fig. 2
Condition after 40 seconds of Vibration in I.V.L. Test.

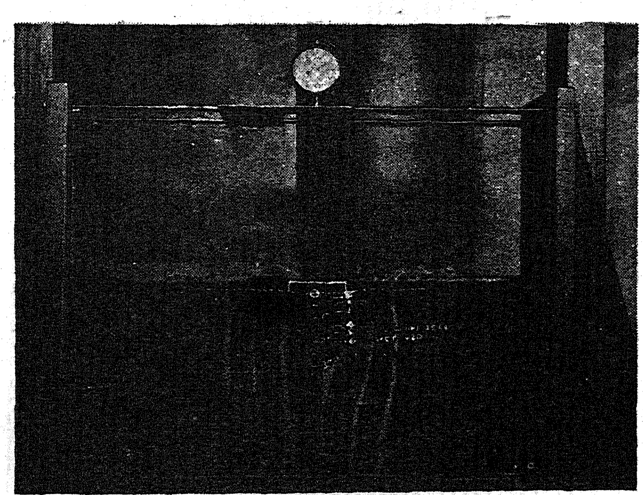

Fig. 3
Condition at conclusion of I.V.L. Test.

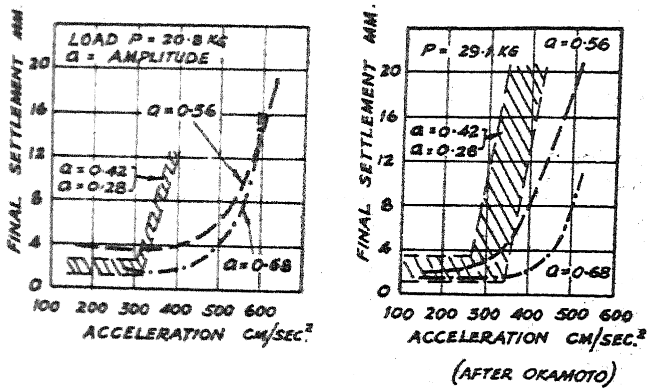


Fig. 4

Graph of Results of Okamoto's experiments.

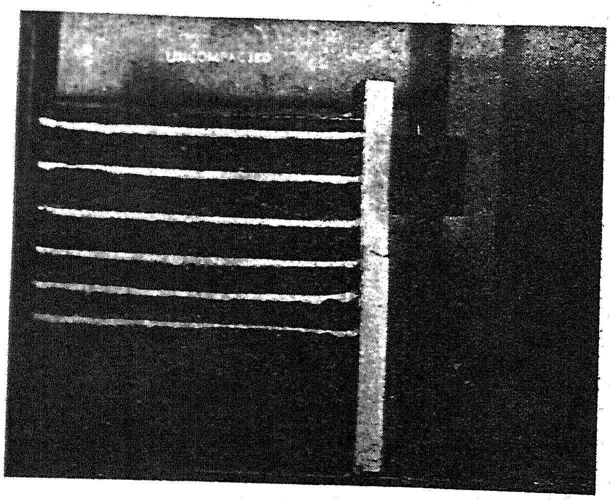

Fig. 5

Before Starting Static Test of sand.

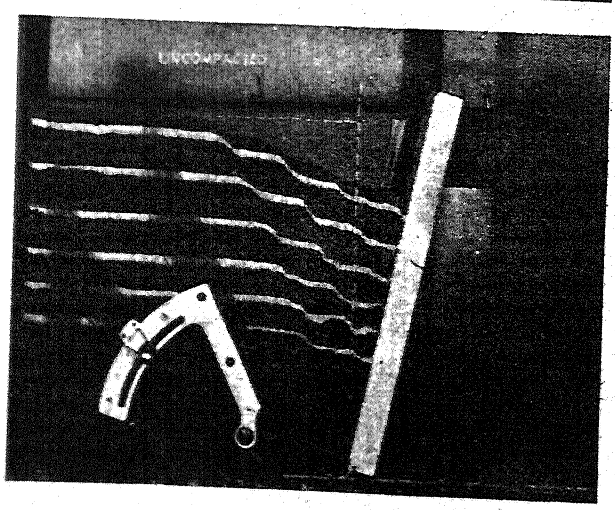

Fig. 6

Angle of Failure for Static Case.

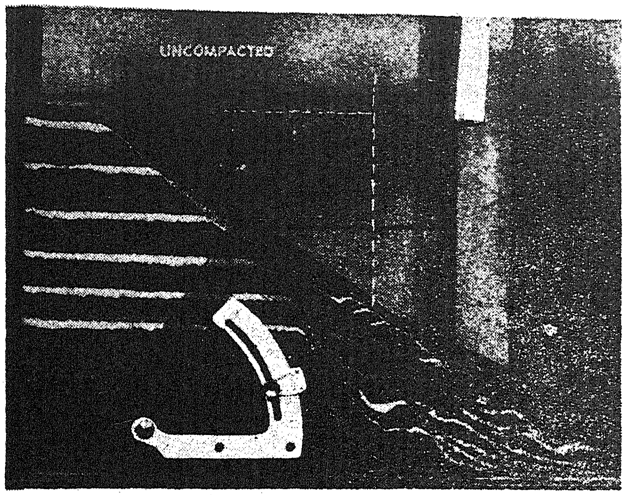


Fig. 7

Angle of Repose for Static Case.

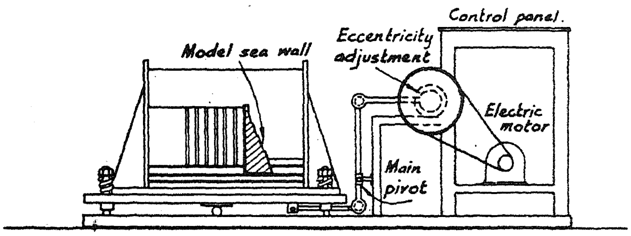


Fig. 8

General arrangement of Gravity Wall Model experiment.

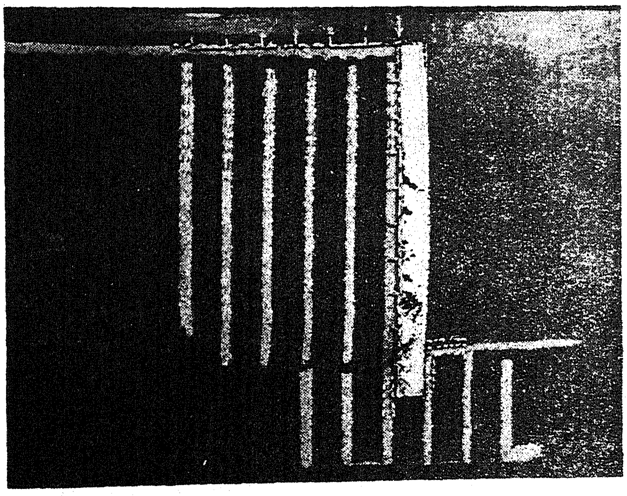

Fig. 9

Before Starting Test of Sheet Pile Wall

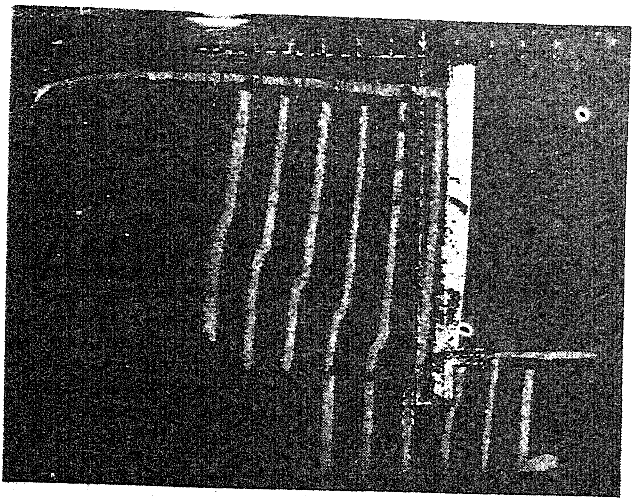

Fig. 10

Condition of Sheet Pile Wall after 20 seconds of Vibration.

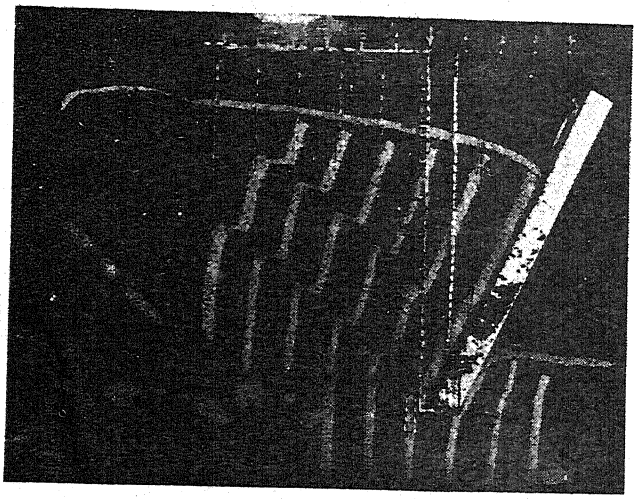

Fig. 11

Condition after release of anchor tie.

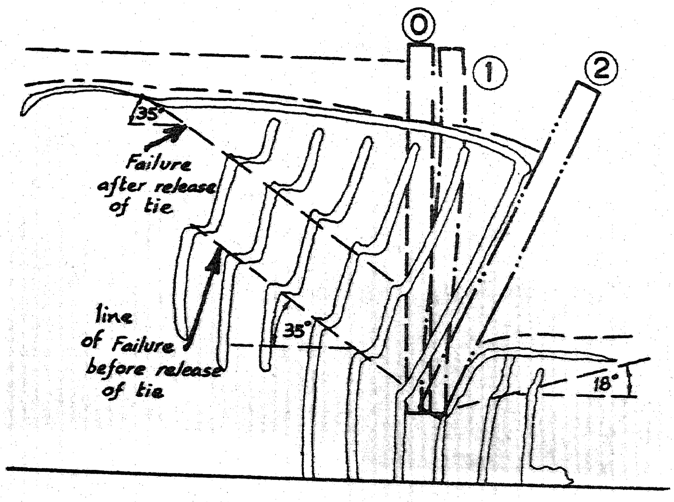


Fig. 12

Diagram showing development of failure of Sheet Pile Wall.

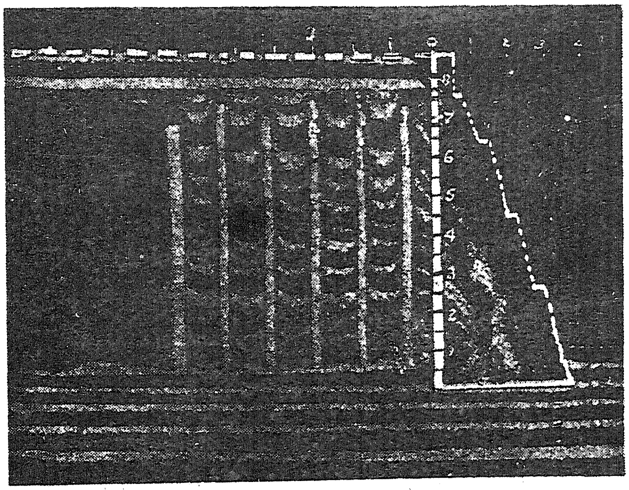

Fig. 13

Before Starting Test of Gravity Wall.

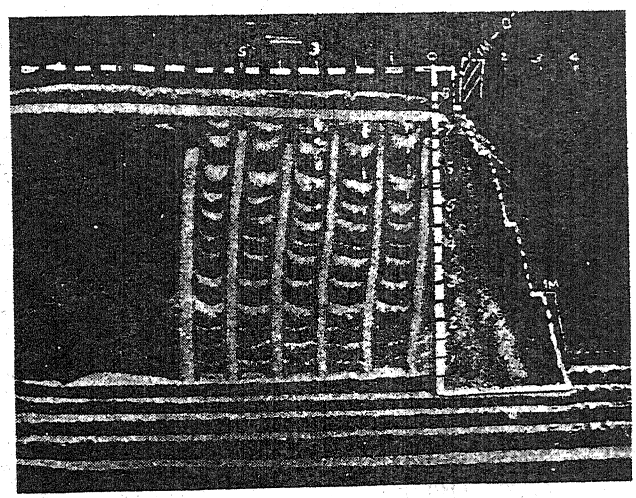

Fig. 14

Condition of Gravity Wall after 1 minute of vibration.

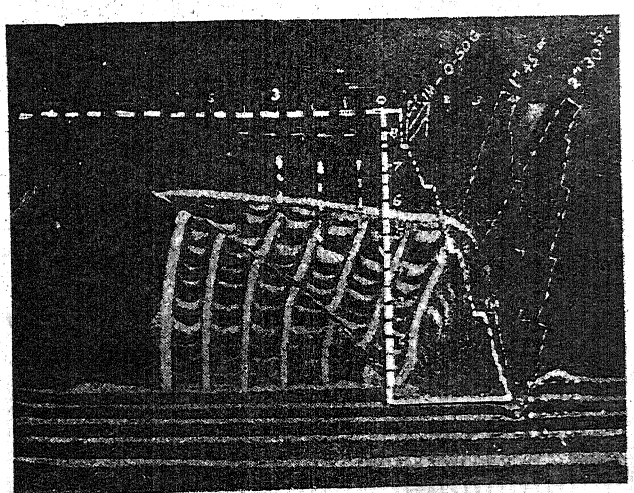

Fig. 15

Condition of Gravity Wall after 2½ minutes of vibration.

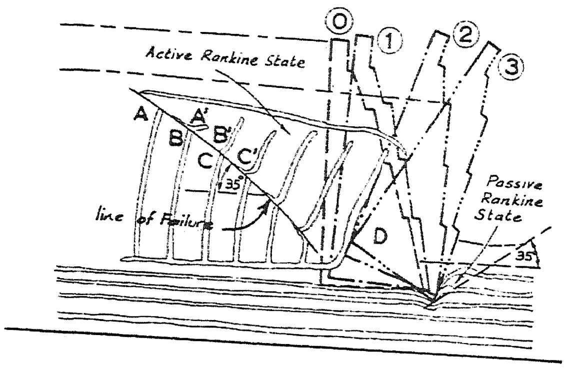


Fig. 16 Diagram showing development of failure of Gravity Wall.

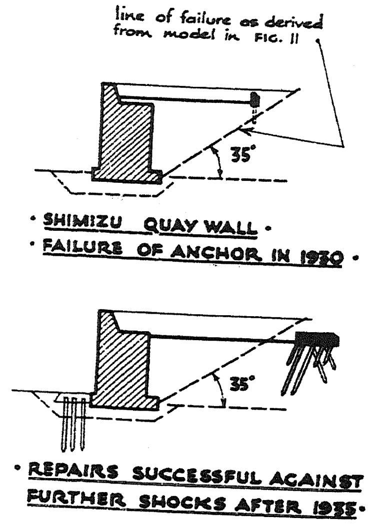


Fig. 17
Applying Model Results to Damage Surveys.

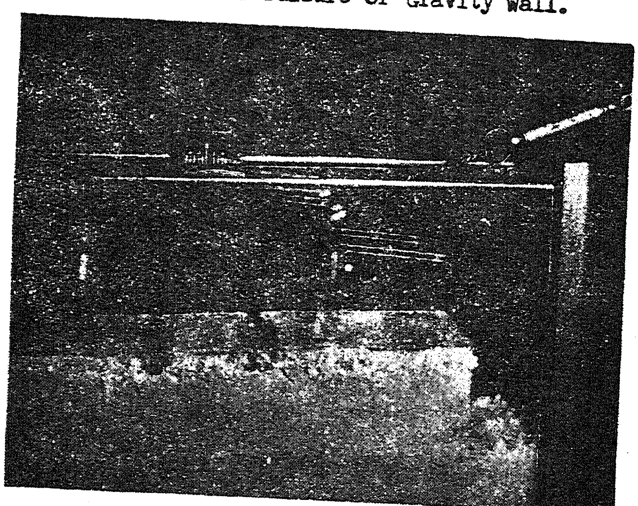

Fig. 18
Gelatine Model fractured at corners of pit.

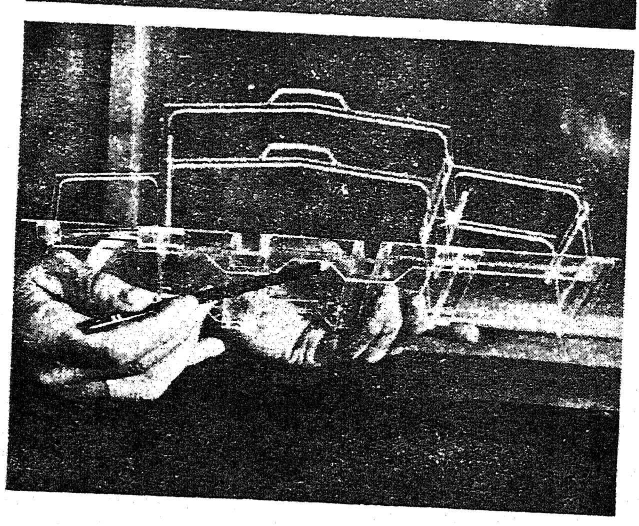

Fig. 19
Special Features of Raft Foundation Model.

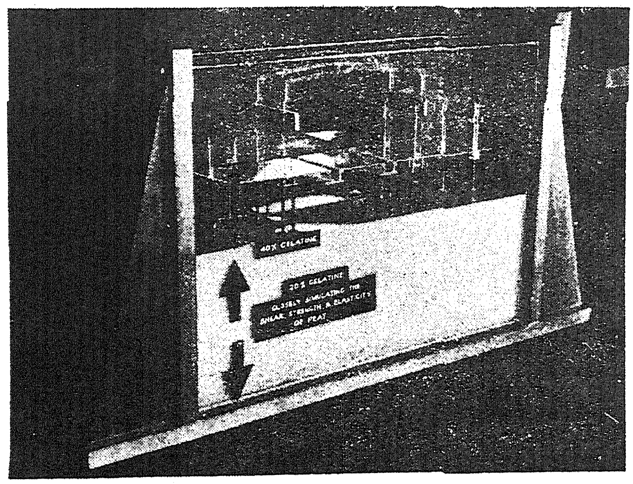


**Fig. 20** **Pile Foundation Model being constructed.**