A MODEL STUDY ON COHESIONLESS SLOPE DURING VIBRATION

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CERTIFICATE

CERTIFIED that the dissertation entitled 'A Model Study on Cohesionless Slope During Vibration', which is being submitted by Sri P.K. Mitra in partial fulfilment for the award of the Degree of Master of Engineering in Earthquake Engineering, University of Roorkee, Roorkee is a record of student's own work carried out by him under our supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for about 11 months from March 1969 to February 1970 in preparing this thesis for Master of Engineering at this University.

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SYNOPSIS

This investigation covers a detailed experimental study of the behaviour of sand embankments having various slopes subjected to dynamic loadings.

The deformation in cohesionless embankments starts at a particular value of acceleration. This value is called the yield acceleration. Cohesionless embankment having various slopes were constructed inside a steel tank with glass fitted on one side. This box was mounted on the horizontal shaking table. The table was vibrated at different accelerations by varying the amplitude of motion, and the deformations were studied at different values of accelerations at different number of cycles. The values of yield acceleration were calculated and the experimental values were compared with them.

It was found that the computed values of yield acceleration lie in closer vicinity with the experimental values in the case of sinusoidal excitation as compared to shock loading. The method for calculating the values of displacements has also been suggested.

CONTENTS

NOTATIONS

$K_y g$	- yield acceleration,
ϕ	- angle of internal friction of soil,
α	- inclination of the slope to horizontal
θ	- inclination of ground acceleration.
ϕ_{st}	- correction factor used in angle of internal friction at zero normal pressure
S_i	- shear strength intercept at zero normal pressure,
ϕ_{eq}	- Equivalent angle of internal friction,
g	- acceleration due to gravity,
D_{10}	- Effective grain size,
D_{60}	- Uniformity coefficient,
F_i	- initial factor of safety,
F_s	- Static factor of safety,
δ	- Angle of failure,
M, m	- Mass
F_A	- Applied force,
e	- Eccentricity
a	- Amplitude,
w	- Frequency,
T	- time period
μ	- friction factor,
N	- number of cycles,
R	- radius of the circular sliding wedge.

I N T R O D U C T I O N

The present problem under investigation deals with the behaviour of cohesionless embankments subjected to earthquake motions. With the construction of increasingly large rock-fill embankments in active seismic zones the topic of investigation has become a major source of concern. A reduction in the slope of the embankment may cause a huge saving.

In loose saturated soils the failure usually occurs due to liquefaction when subjected to dynamic loadings. But when the sand is dry or partially saturated, slope failure is due to sliding of mass of soil. In the well compacted slopes having uniform density the failure occurs in the form of mass sliding of thin surface zones. The overall failure form is in the form of flattening of the slope at the toe and the settlement at the crest.

It has been suggested by Newmark(1965) that the magnitude of slope-displacements that develop during an earthquake should be criterion for assessing the degree of stability or instability of a slope rather than the computation of the possibility of the factor of safety dropping to unity. Evaluation of this displacement due to gravitational acceleration on the stability of slope involves the concept of yield acceleration, which is defined as the acceleration at which the first movement starts. Seed and Goodman(1964) worked on this and considering a sliding wedge of soil and equating the resultant forces developed an expression for the yield acceleration in the case of cohesionless soils with strength characteristics $S = p \tan \phi$, where S = shear strength , p = normal pressure on the failure plane as follows:-

$$K_y g = \frac{\sin(\phi - \alpha)}{\cos(\phi - \alpha - \theta)} g$$

where,

$K_y g$ = yield acceleration,

ϕ = angle of internal friction,

α = inclination of slope, and

θ = inclination of ground acceleration.

It was also shown that the dense cohesionless sands develop small shear strength even under no confining pressure and the value of ϕ used was increased as shown below. Thus, the strength of the soil was given as $S = S_i + p \tan \phi$, where S_i = shear strength intercept at zero normal pressure.

Thus, a correction factor ϕ_{s1} is added with the value of ϕ and the expression is given as,

$$K_y\ g = \frac{\sin(\phi_{eq} - \alpha)}{\cos(\phi_{eq} - \alpha - \theta)} g \quad ; \phi_{eq} = \phi + \phi_{s1}$$

when the ground acceleration is horizontal the expression for yield acceleration becomes

$$K_y\ g = \tan(\phi_{eq} - \alpha) g$$

for slopes subjected to reasonably uniform acceleration the yielding starts when the acceleration exceeds the yield acceleration. But for acceleration of short duration there is a marked difference in the acceleration which induces displacement. The determination of yield acceleration requires a knowledge of strength characteristics under plane strain condition.

SCOPE OF THE INVESTIGATION

So far the experiments, for finding out the yield acceleration, were done with shock loading only. (Seed and Goodman). In the present investigation the section was subjected to sinusoidal motion generated by the movement of a cam. By changing the eccentricity of the cam the amplitude of theshaking table was varied. There was arrangement for changing the frequency of the motion also. Thus, there were three variables, frequency, amplitude and number of cycles. Out of which only the effect of two i.e. amplitude, and Number of cycles has been studied.

The effect of varying the frequency has not been studied in the present investigation. Any definite conclusion in this direction is not available because of the difficulties in conducting large number of model tests on a large scale. Though the sources of error are more in small scale tests, qualitative and semi-quantitative results will be forthcoming from them.

REVIEW OF LITERATURE

The works reviewed are of both theoretical and experimental types, although in most of the cases the theoretical studies are verified by test results.

A. Casagrande (1936) performed a number of tests on dense and loose sand embankment models with storage water. He introduced the concept of critical void ratio according to which the volume change due to shearing deformation is zero at a particular density. The corresponding void ratio was called the critical void ratio. According to him the dense sand having density higher than the density at the critical point, after undergoing shearing strains will tend to acquire the critical void ratio. He concluded that the densification of the rock-fill structures due to earthquake was not likely in view of the short duration of the earthquake motion. He conducted experiments on both loose and dense models. Loose models flowed out. The dense models did not suffer any appreciable change. He also concluded that the models are more stable than the prototypes since a reduction in shearing strength depends mainly on the extent of deformation, the amount of volume decreases with deformation, the permeability and the dimensions of model.

Hatanaka(1955), studied the free vibration of dams having a symmetrical triangular section under the hypothesis of constant elastic properties and Barnoulli's assumption about plane deformations and uniform distribution of shearing stress in a horizontal plane. These considerations led him to conclude that the shearing deformation is the most important contribution to the displacement in earth-dams of medium sizes. He studied a two-dimensional model having a triangular cross-section and lying in a canyon of rectangular shape with the dam fixed at abutments and foundations. This led him to conclude that in order to keep error below 10 percent in calculation of the first two natural period the length of the rectangular gorge should be 3 to 6 times the height of the dam. To verify this he tested agar-agar models having height of 8 cm. and various lengths. He measured transient displacement normal to dam axis under forced vibrations. He found agreement between fundamental period computed and observed; for the second errors in the period were about 20 percent. In concluding he recommends the use of seismic coefficients based on the first mode of vibrations as a design procedure.

Clough and Pirtz(1958) conducted a number of dynamic tests on 61-cm. tall model having a central and sloping core near the upstream face. The central core model had symmetrical cross-section with slopes of 29.7° in the upper third and 24° in the lower two third. The sloping core model had 24° u/s and 35.6°, d/s slopes. The latter models were tested with water at three different elevations i.e. reservoir empty, 4/10-full and full. The central core models were tested with full reservoir. The dynamic load was produced by striking the shaking table with a 68 kq. pendulum; two rebound blows were permitted. The peak acceleration ranged from .1 g to 1.25 g. The author has also discussed the dimensional similitude requirements for length, force and time. Care was taken in selecting the material to simulate the requirements obtained from dimensional analysis. Appreciable settlements of the crown and heaving near toe were measured only after maximum acceleration reached .4g. The authors have concluded that even when shearing resistance is exceeded only small displacements will result because before a great displacement takes place there will be reversal of ground acceleration. They also concluded that the sloping-core dams have superior earthquake resistance because the central core will break the continuity between the u/s and d/s portion of the dam.

Minami (1960), suggested the consideration of an extra pore pressure due to elastic vibration for calculating the effective stresses and subsequent application of the Couloumb's equation of strength. The significant contribution in his study was, however, the measurement of the distribution of internal stresses during vibration in a model of 5 cm. thick and 45 cm high made of sand. He used small stress-pickups and took measurements of normal stresses in three directions. He concluded that the internal stresses remain elastic , except close to the exposed slope surface where stresses enter plastic range. He concluded that the use of visco-elastic model will therefore correctly predict the distribution of the seismic coefficient.

Seed and Clough (1963) has utilised the test datas given by David and found the seismic coefficient according to Ambrasey's criterion, using a constant shear modulus and a spectra of table motion. The values of accelerations were varied from 0 at the base to 1.35 g at the crest for 20 percent of critical damping. Under these loads they found that the model was unstable. They have concluded that the elastic response approach for design of earth dams is a very conservative method.

Bustamante (1964) conducted number of experiments on earth dam embankments subjected to dynamic loadings. He used a shaking table of 4.5 m by 4.5 m. in plan and tested thirteen models of height varying in the range of 1.004 m. to .553 m both statically and dynamically. The slope angle of both u/s and d/s sides were also varied from model to model. He proposed a simplified analog simulation, assuming three rigid wedges with apexes at the toe of the embankment. The quantitative results given by this analog simulation was in close agreement with the physical tests conducted.

Seed and Goodman(1964) conducted number of experiments on cohesionless slopes. The experiments were conducted on slope lengths of 4.5 ft. confined laterally at 2 ft. width by glass-sides. The dynamic excitation was provided by the impact of 150 lbs pendulum. They compared different values of theoretical and experimental values of yield acceleration and plotted them for different slope angles, and displacements. They have concluded that when the surface conditions involves variations in density along the slopes, the yield acceleration is somewhat less than the value given by the expression $\tan(\phi-\alpha)$ where ϕ is the angle of internal friction corresponding to the average density of the slope, while for uniform density condition the yield acceleration will be somewhat greater than the value given by the above formula. It is necessary therefore to give careful consideration to the surface condition of the slope.

Newmark(1965) suggested that the magnitude of slope displacement that developes during an earthquake should be a criterion for assessing the degree of stability or instability rather than computing a possibility of factor of safety dropping to unity during shaking and he has analysed the embankment by block sliding procedure. He has considered three types of sliding surface such as cylindrical sliding surface, plane sliding surface and block sliding surface, and he has derived equations correlating the factor of safety and yield acceleration of the slope.

J. Krishna and S.Prakash(1965) , investigated the stability problem of Ramganga Dam. The model study was conducted on a large vibration table of 5.2 m by 2.8 m size. The model tests were done by a pendulum loading device. The acceleration pattern found with the height of the dam was very similar to the theoretically predicted one. Field study was done by conducting blasting tests with 80 percent gelatin and the acceleration, particle velocity and displacements caused by waves were recorded along and perpendicular to the axis of the dam by suitable pick-ups. The amount of the charge and the distance of the recording site were varied.

Seed and Goodman (1966) further conducted a number of dynamic and static experiments on cohesionless slope made in the length of approx. 4.5 ft., confined by glass sides of 2 ft. width. The loading was done by impact width. The loading was done by impact of a 150 lbs. pendulum. Acceleration of table and bank was recorded by Statham accelerometers. Displacements of the sliding blocks of the sand mass was recorded by the differential transformers attached parallel to the slope with a rigid block. Movable transformers core was attached to a drive rod which could slide with sand. Thus, the displacements were given by the plot of a oscillograph attached to the transformers. They have compared the experimental and computed value of the displacements. They have also found out the value of acceleration coefficients corresponding to different values of equivalent angle of internal friction. They have concluded that the displacement found experimentally and values computed lie in close agreement with each other. They also lie in close proximity with the actual measurements taken of the ground displacements with the ground accelerations. Thus, with geometrically similar conditions their method of analysis will serve as a useful tool.

TEST EQUIPMENTS

SHAKING TABLE

A steady state horizontal shaking table which could provide an horizontal acceleration upto 2 g. and vibrated with a sinusoidal motion was used. The energy for vibrating the table was supplied by a 1 H.P. A.C. motor which was connected to a speed transmission system. Uniform speed is maintained with the help of a heavy flywheel attached to the rotary parts and the rotary motion was converted to translatory motion with the help of proper device. The amplitude of vibration has a very wide range and it can be attained by changing the eccentricity of the driving cam. The number of revolutions can be counted by means of revolution counters fixed suitably.

This energy supply unit is connected to a steel platform of size 105 x 60 cm. This platform is stiffened by the channels welded in all the four sides. This platform is mounted on four wheels which can move on the rails.

The important characteristics of the table were as follows:-

Range of amplitude variation - 0 -10 mm

Range of frequency variation- 0 - 20 cps.

Size of platform - 105 cm x 60 cm

Maximum acceleration - 2 g.

The embankment model was constructed and tested inside a mild steel tank which was mounted on the steel platform. The steel tank was having a plan area of 1.1 m by .615 meters. The size of the tank was decided by the size of the model. The length 1.1 meter fitted conveniently with the size of the existing platform and could also accommodate the largest model comfortably. The width of the table and the width of the tank were kept same. Height of the tank was kept 80 cm. Though the height of model was only 50 cm. The extra 30 cms. were kept as a provision for testing the model with water on one side, so that when the tank is inclined the water is not thrown out of the tank. Though in the present investigation only the embankment models with reservoir empty conditions are tested but sufficient provisions are left in tank for testing the models with reservoir full condition, for future. One of the four sides of the box is fitted with a 1/4" thick glass plate, for observing the nature of deformation of the model. This glass plate is made water tight with the help of rubber washers. For conducting the static tests it was required to make the table inclined. Hence, a hinge device at one end and a lifting device at the other was provided as shown in **Photograph.** The hinge device consists of two mild steel rods of 1"Ø and 6" length welded on both the sides of the box. The diameters and size of the rod along with the length of welding were designed for the reservoir full-condition and sufficient factor of safety, and the rod was suitably tested against D'-shear and punching shear. The lifting device consisted of a steel rope fitted on the body of the tank and hooked by a chain-pulley block of sufficient capacity. The tank can be fixed with the platform at this end also for dynamic conditions.

PROFILOMETER

To record the deformation on the embankment surface and plot the distorted surface accurately a frame referance system was used. It consisted of a travelling frame fitted with a ruler which could move in other two directions also.

The frame work was made of 1"x 1" M.S. angles having the same length as the width of the tank. This frame work could move in the longitudinal direction along the top of the tank. Inside this a smaller frame work made of M.S. 1"x1"angle was fitted. This could move inside the bigger framework and at right angle to the direction of movement of the bigger framework, thus enabling a transverse movement. A ruler in length made of bakelite and having a pointed wooden tip at the lower end was fixed in position between the smaller frame with the help of four wooden rollers, fitted with rubber washers. The friction between bakelite and rubber enabled the vertical motion of the ruler. One of the rollers could be rotated with the **help** of a knob and handel fitted in it thus giving vertical motion to the ruler. This arrangement ensured a movement of the ruler in all the three directions. Thus the ruler could be placed in any position inside the tank and the deformation of any point on the surface could be determined. A vertical scale was fixed on the body of the ruler and a vernier was fixed on the body of the frame work. This gave us a sensitivity of 1/10th of a milimeter. Twenty five holes were made on the top of the tank, and on the framework. The frame could be fixed on any of these points and these points served as the reference points.

The readings on the vertical scale and the vernier scales were recorded accurately with the help of a magnifying glass. When the pointed end of the ruler was just in contact with the sand surface the reading was noted. To ensure the exact point of contact every time flood lights were used for throwing a sharp shadow of the ruler on the model surface.

This was the simplest possible device that could be designed for measuring the deformation. Instead of rubber friction rollers rack and penion device could be used. But the rack of 80 cm length was not available, and rubber rollers saved the cost of rack and penion.

PROPERTIES OF SAND USED

The sand used for experimentation was Solani sand. The following tests were performed on the sand:-

1. Physical Properties:

The specific gravity of the sand grains were found from pycnometer tests. The grain size distribution was obtained by using a standard machined shaker. The values obtained are presented below:

Specific gravity of sand grains	= 2.59
Effective grain size(D_{10})	= .075 mm
Uniformity coefficient (D_{60})	= .155 m

2. Angle of Repose

For determining the angle of repose sand was dropped from a funnel from different heights and the angle of the sand cone with the horizontal was determined. Different observations were taken by varying the height of fall of the sand. By varying the height of fall the density of sand in sand cone was varied and reading at nearest to the desired density was obtained. The value of angle of repose obtained from the tests was 30.9^{0}.

TEST PROCEDURE

1. Static Test:

The tests were performed to obtain informations regarding the strength of the model under similar conditions to the one that were to be found in dynamic models. These informations were useful for dynamic problem also. The static factor of safety was found to have the data to correlate the dynamic deformations to the initial conditions. Experiments conducted by Seed and Goodman in 1964, showed that tilting angle to produce failure could be predicted from the shaking tests assuming that there is no change in sand strength parameters. Another object of the shaking test was to have a preliminary idea of the progressive failure that was to be observed in the dynamic test. The tests also served as a check against wall friction between sand grains and the glass by the study of rupture surface.

The model was constructed by placing sand in layers of 5 cm thickness. Each layer was given 50-blows from the standard proctor hammer and brought to the required shape. After the fixing of the profilometer at the proper position the tilting of the box was started. The box was tilted at a very slow rate and after every movement the profile was determined. The rate of tilting was very very small and it was kept as uniform as possible i.e. between 1 to $1\frac{1}{2}$ hours were required to tilt the table through 10^{o}. Once the model was showing deterioration at some particular section reading at that sections were taken. After every noticable movement the central and end sections were studied. The movement of the coloured sand layer were also observed and their displacement to horizontal were recorded in every experiment.

2. Dynamic Tests:

The main objective of the dynamic tests were to determine the mechanism of the geometric changes and to find out the effect of different vibration on yield acceleration. After completing the construction the model was excited with the help of motor device described before and the deformation of the central and other deformed sections were recorded with the help of profilometer. Different accelerations were obtained by changing the amplitude of vibration, while the frequency of vibration was kept constant. The dynamic history was also varied from model to model. A dynamic test was stopped as soon as the displacements became large that is of the order of few centimeters. At this stage the table was tilted and a static test of the dynamically tested model was performed. Thus actually two static tests were performed of two identical geometrically similar model. The factor of safety of the dynamically failed model was calculated and compared with the static factor of safety.

Displacements	Angle of internal friction	Yield Acceleration	Slope
Peak strength(.6 mm)	32°	.201 g	
.7 mm	31°	.190 g	1:2.5
.8 mm	27.5°	.101 g	
.9 mm	22°	.06 g	
1.0 mm	21.5°	.02 g	
Peak strength(.6 mm)	32°	.1414 g	1:2.25
Peak strength (.6 mm)	32°	.095	1:2

Table -2

Tests Performed

Model No.		Initial Slope	Yield Acceleration	Amplitude of Motion	Freq-uency (cycles)	Exciting Acceleration
1.	Static Tests	1:2.5				
2.		1:2.5				
3.		1:2.5	.202g	3 mm, 2 mm	5	.3g, .2g
4.		1:2.5		5 mm	5	.5g
5.		1:2.5		5,6,7 m	5	.5g, .6g, .7g
6.		1:2.25	.1445g	2.5 mm		.25g
7.		1:2.25		3 mm	5	.3g
8.		1:2	.09 g	2 mm	5	.2g
9.		1:2		1.5 mm	5	.15g
10						

TEST RESULTS AND INTERPRETATION

STRENGTH OF SAND

Strength of sand was determined from the direct shear tests. The actual slipping of a surface layer of soil over the other is analogous to the deformation conditions in direct shear tests under low confining pressure. Henceforth it was necessary to investigate the strength characteristics at a low confining pressure. The lowest value that could be achieved from the available Direct Shear Machine was 0.1445 kg/sq.cm. Seed and Goodman (1966) achieved a confining pressure of 0.001685 kg./sq.cm. by using plastic blocks coated with glue instead of Direct Shear boxes. The results of Direct Shear Tests are plotted in Fig.1, which shows a plot of shear stress vs. shear displacement at the normal load of .1445 kg/sq.cm., .283 kg/sq.cm. , .42 kg/sq.cm., .562 kg/sq.cm. It shall be noticed from this figure that the sand exhibits a peak value of strength at about .6 mm shear displacement. The ultimate value of shear stress is attained at a displacement of about 2.0 mm.

Figure 2 is a plot of shear stress versus Normal stress at different value of shear deformation. The peak strength occurred at 0.6 mm shear displacement. The plot shows a linear relationship and the slope of the lines give the value of ϕ at .6, .7, .8, .9, 1.0, 1.1 mm displacements, i.e. at peak deformation and .1, .2, .3, .4, .5 mm after the deformation at which the peak value was mobilized.

The values of ϕ- obtained from Fig.2 for different deformations are shown in the table 1. The value of ϕ varies from 32^o to 21.5^o between the shear displacement of .6 mm to 1.0 mm. This loss in the value of ϕ is shown in the Fig. 3, which is a plot between friction angle and shearing deformation after peak strength.

YIELD ACCELERATION

The variation of ϕ values as shown in Fig.3, at different shear displacement was used for calculating the yield acceleration. Yield acceleration can be defined as the lowest value of acceleration, which is just sufficient to overcome the strength of the sand and at which the first movement starts. A simplified relation for finding yield acceleration has been developed by Seed and Goodman (1964) for cohesionless soils, by considering a wedge of soil sliding on the surface of the slope and equating the forces.

For determination of yield acceleration for any arbitrary slope α , let us consider an element of soil on the surface of slope having weight W. The effect of acceleration a, acting at an angle θ to the horizontal can be represented by an inertia force a/g. W acting at an angle θ, opposite to the direction of acceleration. Therefore , the force triangle will be of the shape shown. When $Ky = a/g$ = yield acceleration, hence from the triangle of forces,

$$\frac{Ky.W}{\sin(\theta-\alpha)} = \frac{W}{\sin[90-(\phi-\alpha-\theta)]}$$

Therefore,

$$Ky = \frac{\sin(\phi-\alpha)}{\cos(\phi-\alpha-\theta)}$$

For horizontal acceleration $\theta = 0$

$$\therefore \quad |Ky|_{Horizontal} = \tan(\phi-\alpha)$$

Furthermore the minimum value of yield acceleration is given by maximum value of denominators i.e. 1. Hence,

$$|Ky|_{min} = \sin(\phi-\alpha)$$

Seed and Goodman(1964) , **have** plotted the value of yield acceleration with the angle θ for different values of $(\phi-\alpha)$. They have found that the minimum values of yield acceleration is not constant but varies with the value of $(\phi-\alpha)$; that is the value of static factor of safety. Furthermore, if the initial factor of safety F_i is defined as

$$F_i = \frac{\tan\phi}{\tan\alpha}$$

Then the minimum yield acceleration coefficient

$$(Ky)_{min} = \sin\phi.\cos\alpha - \cos\phi - \sin\alpha$$
$$= \sin\alpha.\cos\phi(F_i - 1)$$

The values of yield acceleration has been calculated by the expression $Ky = \tan(\phi - \alpha)$ and are shown in Table 1. For the slope of 1:2.5, the yield accelerations are calculated at five displacements. They are .6 mm, .7 mm., .8, .9, 1.0 mm. and the values of yield acceleration are .201 g, .190 g, .101 g, .06 g, .02 g respectively. The value of yield acceleration decreased to a very low value after this and henceforth they have not been shown in the tabular form. For the slopes 1:2.25 and 1:2 the yield accelerations are shown at a shear displacement of .6 mm only. The yield acceleration for these two slopes at all other displacement are very small and hence they are not represented in tabular form. The values of yield acceleration for 1:2.25 and 1:2 slopes are .1414 g and .095 g respectively.

2. STATIC TESTS:

Two static tests were conducted on 1:2.5 , slope, by tilting the table. At lower tilts no appreciable motion or change in profile was observed. With the increasing angle of the box suddenly a failure would begin at some point on the slope most of the time about 1/3rd up the slope surface or near the middle. It never originated at the top. Then the sand would start rolling downwards and the motion would propagate upwards and sideways until there is a change in the whole slope. The new configuration was a curved surface with a depression in the middle and bulging at the toe. Every deformation in the range of the profilometers was recorded and plotted as shown in the Figs. 5 and 6. Figures 5 and 6 are the plot of displacements at the reference points. The reference points on which the profilometer was fixed for readings were 4 cm apart as indicated in the Fig. 5, 6. As shown in Fig. 5, the first movement was noticed at a section 20.2 cm apart. from the glass end of the box and occurred at a tilt angle of 10°-30', while in model number 2 the first movement was observed at the middle section at the tilt of 13° as shown in Fig.6. The displacement observed in the middle section of the model 2 was also considerable less than the first movement of model 1, maximum displacements are .13 cm and .07 cm respectively as shown in Fig. 5 and Fig.6. But at a tilt more than 16° the displacements are more identical.

• The experimental pattern of failure shows that the failure stated near the bottom and propagated upwards along the slope. The type of failure observed in all cases shows a case of soil structure instability triggered by interlocking of some surface grains that break the grannular structure. After obtaining the experimental data it was found that there is a variation in preliminary estimates of angle of failures. The factor of safety calculated from the value of angle of failure with the help of expression

$$F_s = \frac{\tan \delta}{\tan \alpha}$$

where,

δ = angle of failure,

α = initial angle of the slope.

It was found for the slope 1:2.5 that the first movement starts at a tilt of $10^{\circ}30'$, which gives the value of $\delta = \alpha + 10^{\circ}\text{-}30' = 32.3^{\circ}$ and the value of factor of safety = 1.585. The value of factor of safety calculated from angle of repose in the place of δ was found to be 1.49. The type of failure as observed in the model was explained by Bustamante (1964) with the help of a cylindrical mechanism.

The mechanism can be explained if the movement starts at the bottom. But when it first starts at the top one has to imagine that the upper grains force the lower grains in a sliding motion by their weight as suggested by Bustamante. The curve shows an appreciable volume of grains, which has moved in stable equilibrium having an angle to the horizontal nearly equal to the angle of repose. Under this hypothesis the instantaneous failure of the whole wedge of material covering the whole of the slope is improbable.

Bustamante (1964) offered a mechanism for this type of failure by using the analogy of cylinders and spheres. Accordingly whole of the slope is made of cylinders of unequal sizes and one of the surface cylinders are removed when the angle of slope is greater than the angle of repose then the motion of the other cylinders imply raising their centre of mass; therefore he implies the work done will be positive and shows that the system is stable. Or in other words a hyperstable pile of cylinder is not affected by the removal of a surface cylinder. However, as the angle increases the whole wedge will fail and hence for higher slope he has offered a mechanism with sphere at the surface.

DYNAMIC TEST:

Three models of 1:2.5, 1:2.25 and 1:2 slope were tested dynamically. The frequency and an amplitude was varied in each case as shown in Table 2, which lists the types of tests performed on each model. Every movement in the range of the profilometer was recorded and plotted. It was observed from the models that abrupt change in geometry similar to the one observed in static tests were observed. That is a sudden deformations were observed at some particular value of acceleration commonly originating at the lower half of the slope. It was also observed in all the models that the profile tended to become rounded, all the edges were somewhat destroyed. Due to the glass sides it was possible to observe the internal distortion of the model. The deformation of the end sections were recorded by measuring the deformation of the coloured sand lines. Though due to constructional misalignments and nonavailability of wooden former boxes the coloured sand lines were slightly below horizontal near the slope, in the initial condition, yet the relative difference between the readings gave sufficiently accurate results. Hence, it can be claimed with reasonable accuracy that the deformations recorded were unaffected by the methods of preparing the model. The importance of dynamic model testing can be realised by considering the vertical displacements at different sections. This is in a way a measure of change in P.E. of the system. The type of failure observed i.e. sudden displacement at a zone in lower half of the slope was observed by Seed and Goodman (1966) and Bustamante (1964) , in their experiments, in which the models were excited by the energy of a falling pendulum. It has been previously suggested that this type of failure occurs when the slope angle is less than the angle of repose. Bustamante (1964) has found that in models constructed with wooden former boxes having slope angle greater than the angle of repose the motion observed is the motion of a wedge fragmented into particles moving individually. Fragmentation into particles with individual motion can be ascribed to a change in interlocking condition.

The Figures 7,8, and 9 show the plot of displacements recorded at different reference points for the slope of 1:2.5. Model No.3, results of which are shown in Fig. 7, was first tested at an acceleration .2g. But no displacement was observed even at 650 cycle and similarly no displacement was observed any where between .2g and .3g. The first appreciable displacements were observed at the acceleration .3g. The displacements at 100 cycles and at 200 cycles were observed and are plotted in Fig.7. Figure 8 shows the displacements observed at the reference points at the acceleration of .5g after 20 and 100 cycles.

Figure 9 , a plot of displacements at reference points shows displacements observed at 20 cycles of .5g and another 20 cycles of .6g and another 100 cycles of .7g. Figure 10 and Figure 11 show a plot of displacements at different reference points for the slope of 1:2.25.

Figure 10 shows displacements at 100, 200, 300 and 500 cycles at acceleration. .25 g and Figure 11 shows displacement at 100, 200 , 300, 500 cycles at .3g.

Figure 12 and 13 show a plot of displacement at reference points for 1:2 slope. Figure 12 shows displacement at 100, 200, 400 , 500 cycles at 0.2g and Figure 13 shows displacement at 100,200,400, 500 cycles at the acceleration of .3g.

It may be observed from all these figures that the acceleration at which the displacements start is higher than the calculated values of yield acceleration. From Fig. 7,10,13 we get that the yield acceleration for 1:2.5,1:2.25 and 1:2 slopes are .3g, .2g and .15g as compared to .2g, .1445g and .09g calculated from the formula. $K_y = \tan(\phi - \alpha)$. Figure 14 shows the comparison of computed value of yield acceleration and the experimentally found value. It shows a plot of acceleration causing sliding Vs slope angle. Curves 1, 4, 3,2 show the experimental and computed values of acceleration for Montery sand and Watsonville Granite given by Seed and Goodman. Curves 5 and 6 show the computed and experimentally found values for Solani Sand. It shows that for experimental values of the Seed and Goodman are vastly different from the computed values and the difference goes on increasing with lower slope angle. The nature of the curve found from our experiments on Solani Sand is also the same, and shows a closer agreement between the computed and experimental value of acceleration. The difference probably can be explained by the difference of loading procedure.

Computations of Displacements:

(1) En rgy Method

The whole of the vibratory system can be represented by the line sketch shown in Fig. 15A in which energy of the moving flywheel is supplied by a 1 H.P. A.C. motor connected with it with the help of a conveyor belt.

The rotary motion imparted to the wheel is converted into translatory motion as shown. In the end of the rod a mass M vibrated on four wheels as shown in the figure. This mass consists of mass of the table, tank and the sand model collectively. Since the l/e ratio is very large in our case we can say that the motion given to the mass is sinusoidal. Let us assume that the pulse generated is of sinosoidal nature. If we consider the free body of the mass acted upon by this pulse we get the following equation of motion.

$$mx'' + (\text{Frictional force}) = \text{Applied force},$$

$$mx'' + F_f = F_A$$

How work done on the system = F_A x Distance moved,

= mx'' x Dist. moved $+F_f$xDist.

= work done in moving the mass + W.dissipated in friction.

If we can measure the current supplied to the motor we can find out the work done on the system or the energy supplied. Let us say the work done on the system is W. and work done in friction is W_f. Then ,

$$W = W_1 + W_f$$

If the mass was only made of indeformable body then the work dissipated in friction would have been the friction between the rail and the wheel but in this case some of this work is utilised in deforming the sand mass also. Hence, we can write

$$W_f = W_2 + W_3$$

where,

W_2 = work dissipated in friction between wheels and rails,

W_3 = work dissipated in moving the sand mass.

There,

$$W = W_1 + W_2 + W_3$$

The work done in moving the mass can be calculated in the following manner by considering the movement dx in time dt at an instant t after the movement starts. Hence, work done in time dt = mx". dx

$$\therefore \quad dW_1 = mx''.dx. = m\frac{d}{dt}(\ a\ \sin wt).dx$$
$$= -maw^2.\sin wt.dx.$$

$$\frac{dx}{dt} = x' = a\ w.\cos wt$$

$$\therefore \quad dx = aw.\cos wt.dt.$$

$$\therefore \quad dW_1 = -ma^2w^3 \sin wt.\cos wt.dt$$

$$\therefore \quad dW_1 = -\frac{ma^2w^3}{2}.\sin 2wt.dt.$$

$$\therefore \quad \frac{W_1}{2} = -\frac{ma^2w^3}{2}\int_0^{T/2} \sin 2\ wt.dt. \text{ in half cycle.}$$

$$\therefore \quad W_1 = +\ ma^2w^3 \times \left[-\frac{\cos\ 2wt}{2w}\right]_0^{T/2} \text{ in one cycle.}$$

$$\therefore \quad W_1 = \frac{ma^2w^3}{2w}\left[\frac{\cos 2wxT}{2} - \cos 0\right]$$

$$T = \frac{2\pi}{w} \quad \therefore \frac{T}{2} = \frac{\pi}{w} \quad \therefore \cos\frac{2wT}{2} = \cos\pi$$
$$= -1$$

$$\therefore \quad = \frac{ma^2w^3}{2w}(\ +1\ -1\ ) = 0$$

It is a closed system and work done in moving the mass is zero. Hence, whatever work is done it is done to overcome friction and distorting the slope.

W_2 = work dissipated in overcoming the friction between the wheel and rails,

Vertical weight = mg

If the friction factor = μ

$\therefore$ Frictional force $= \mu$ mg

Work done in one cycle $= 2\, a \mu$ mg

$$\therefore \quad W_2 = 2aN \mu \, mg$$

where,

N = number of cycle

a = amplitude,

Hence, $W = W_2 + W_3$

$$\therefore \quad W_3 = W - 2a \mu \, mg.N.$$

From this we can calculate the value of W_3 i.e. work done in distorting the slope. The value of μ can be taken as .2.

This value of W_3 can be equated to the actual energy utilized in distorting the slope. As we know the recorded displacement we can consider the actual wedge which has taken part in sliding. Let us consider Fig.15D. which considers the wedge that has moved by a distance ds. Its C.G. shifts from C to C' which subtends an angle $d\theta$ at the center of rotation. Hence,

$$CC' = ds' = \bar{R} \, . \, d\theta$$

$\therefore$ Vertical movement of C.G. $= \bar{R} . d\theta . \sin\theta$

$\therefore$ Force along the circumference = q x L where, L = length of the arc.

Hence, the work done = q.L.ds - $W\bar{R}$ sinθ.dθ. From the equilibrium condition the value of q can be calculated and hence, the work done can be calculated. By comparing the value of work done with W_3 we can calculate the displacements ds or if we already know the soil wedge which has participated in sliding we can compare the two values of energy supplied.

2. Yield Acceleration:

The slope of uniformly dense grannular sand is subjected throughout its height to a constant horizontal sinusoidal acceleration having magnitude greater than its yield acceleration as shown in Fig.16A. With the higher acceleration a sliding mass will generate at the surface and start rolling downwards. The velocity imparted to the sand mass can be calculated from the area of acceleration diagram. For first half of the cycle the velocity will increase in moving the sand mass down the slope. With the reversal of the acceleration, theoretically it seems that the mass of the sand will move uphill. But if we neglect the negative acceleration diagram, the velocity imparted will remain constant for the negative half-cycle equal to the velocity attained at the end of the first half of the cycle as shown in Fig.16A.

Hence the velocity attained between time 0 to T/2 is given by

$$V = \int_{0}^{T/2} x''.dt = - \int_{0}^{T/2} A.w^2.\sin wt.dt.$$

$$= 2Aw$$

This velocity is plotted and shown in Fig. 16.B., taking particular values. If Ag is yield acceleration of the slope, then the resistance mobilised before distortion can be represented by Ag.t-curve. If Ag.t - curve is superimposed on the actual velocity plot, the useful energy will be given by the area enclosed between these curves, and the displacement will be given by the area enclosed between these curves.

Considering for a particular case, say, for amplitude 3 mm and w = 5 cycles/secs, we get the velocity imparted after the end of first half of the 1st cycle will be 2Aw

Therefore, $2A.w = 2 \times .3 \times 5 \times 2\pi = 19.26$ cm/4
Velocity at the end of first half of second cycle can be given by

$$4A.w = 38.52 \text{ cm/sec.}$$

For 1:2.5 slope the yield acceleration with no disp is .2g. Superimposing the two curves we get Fig. 16.B.

The time of intersections is shown in Fig. 16.B. They are $t_1 = .0516$ to $t_2 = .0934$. Displacement is given by the shaded area.

Therefore,

$$\text{Displacement} = \int_{.0516-.05}^{.0934-.05} .3w.\cos wt.dt - \tfrac{1}{2}x(.0934-.0516)x\,(18.214-10.362)$$

$$= \int_{.0016}^{.0434} 9.42 \cos wt.dt - \tfrac{1}{2}x.0418x7.852$$

$$= \frac{9.42}{w}(\sin 39.06^\circ - \sin 1.44^\circ) - .1641$$

$$= .18 - .1641 = .0159 \text{ cm/cycle}$$

The value of yield acceleration will reduce after this displacement and the slope of the Ag.t-diagram will reduce accordingly. This new Ag.t-diagram will cut the velocity diagram after a few cycles and the displacement can be found by the area. With this displacement a new value of yield acceleration can be derived from Fig.4, by repeating this process. Thus we can find the displacement at higher number of cycles.

SUMMARY AND CONCLUSION

Static and dynamic tests of the embankment models of non-cohesive materials have been carried out. Ten models of 30 cm height and with slopes of 1:2.5 , 1:2.25 and 1:2 have been tested statically and dynamically. Direct shear tests were conducted as small pressure to find out the properties of the soil.

From the static tests we can conclude that for the slopes having the initial angle lower than the angle of repose failure occurs as a sudden change starting at lower half of the slope which generally propagates upwards until the whole slope is distorted. No sign of approach of such a failure can be detected before it. The progressive nature of this type of failure contributes tc a case of unstable equilibrium which has been explained by the help of models of cylinders and spheres.

The most important conclusionsthat can be drawn from the dynamic tests are that the values of yield accelerations found experimentally for sinusoidal excitation lies is closer vicinity with the computed values in comparison to the values obtained by the experiments conducted with the shock loading previously by Seed and Goodman (1964). However, varied accelerations were obtained with the help varied amplitude but the effect of varying the frequency was not studied. It was also observed that the slope behaved in a similar manner with sinusoidal excitation as in the case of static test and in this case also the failure originated somewhere in the lower half and the motion propagated upwards with the higher value of acceleration and with higher number of cycles. The two methods suggested for computing the displacements of any number of cycles can be utilised for finding displacement at a higher number of cycles, fairly accurately.

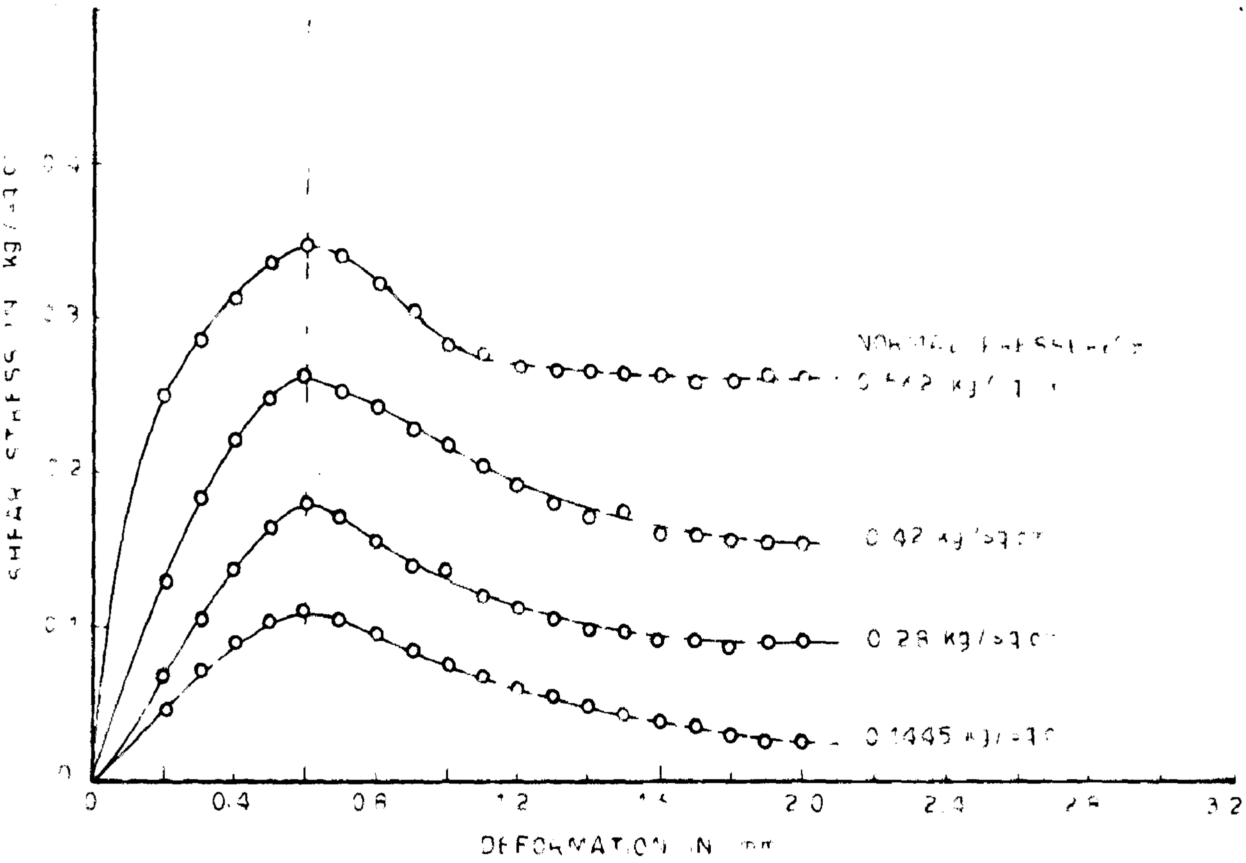


FIG. 1 STRESS-DEFORMATION CURVE FOR DRY SOLAN SAND AT DIFFERENT NORMAL PRESSURE

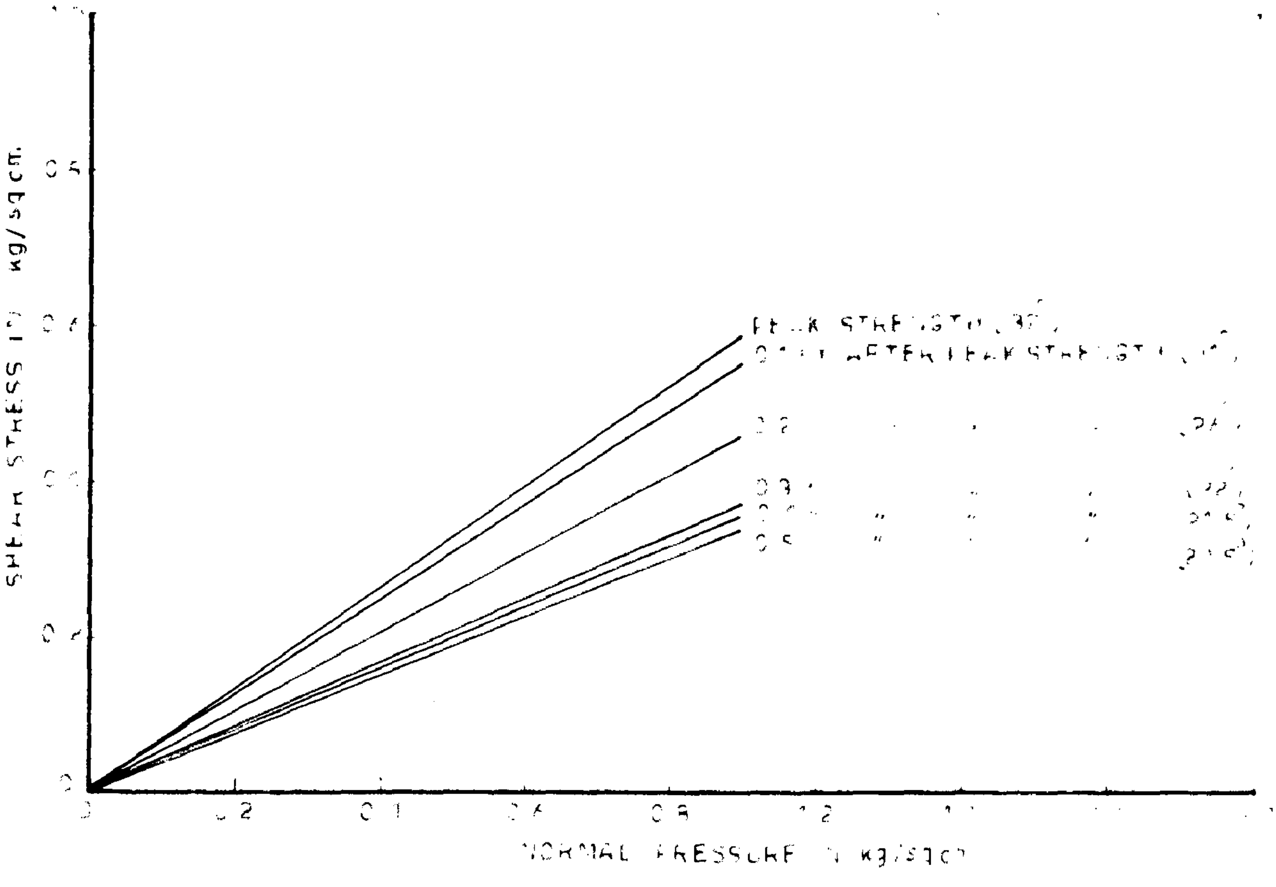


FIG. 2 CHANGE IN THE VALUE OF ANGLE OF INTERNAL FRICTION AFTER PEAK STRENGTH

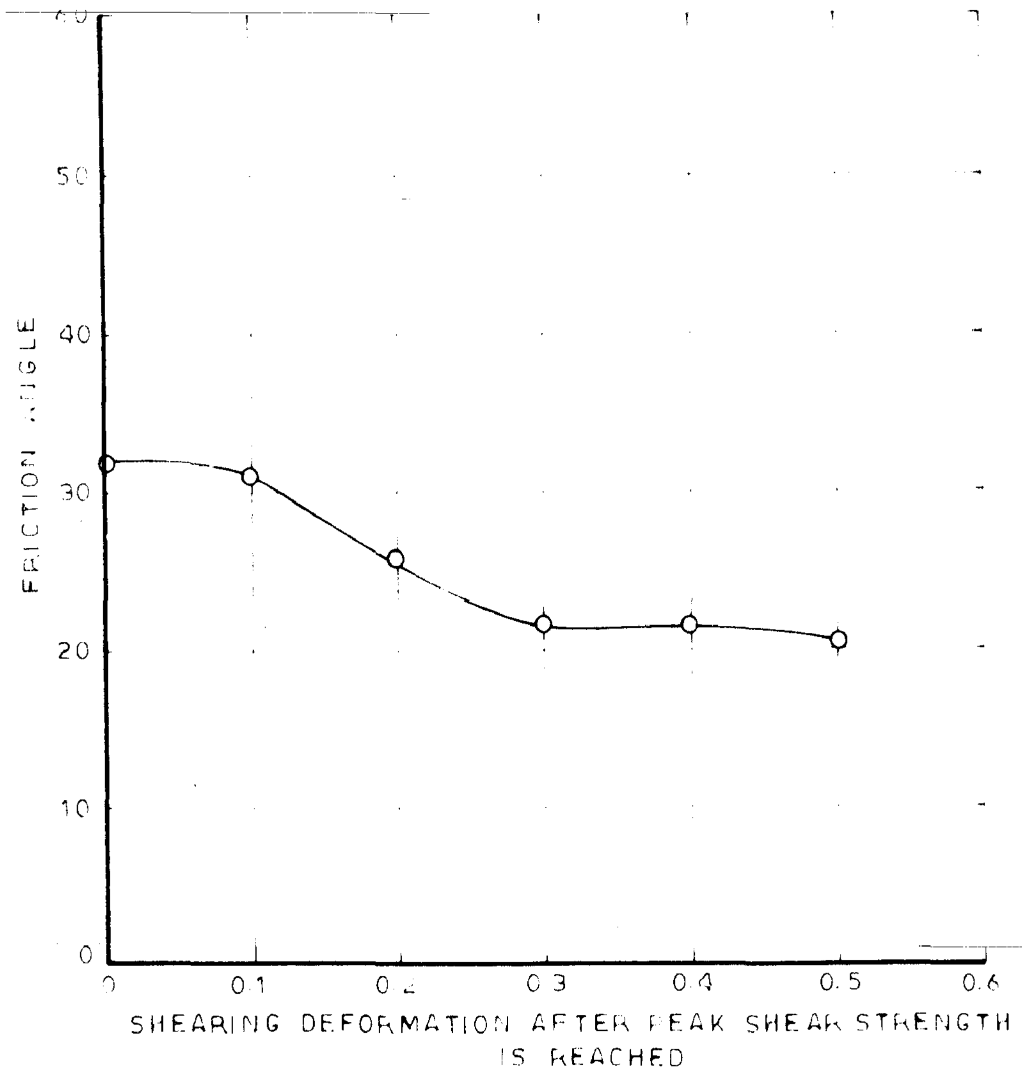


FIG. 3 _ LOSS OF FRICTION ANGLE WITH INCREASING DEFORMATION

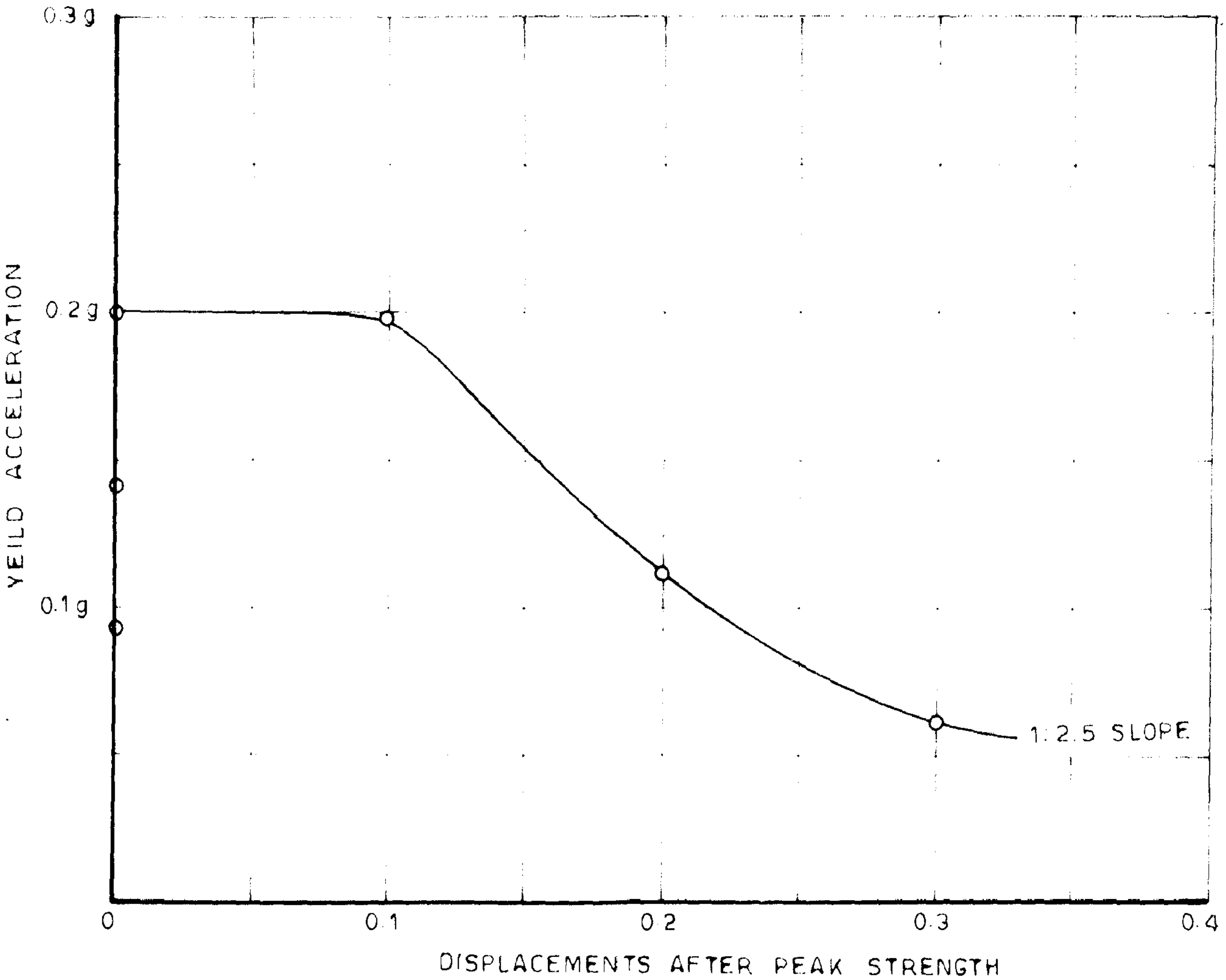
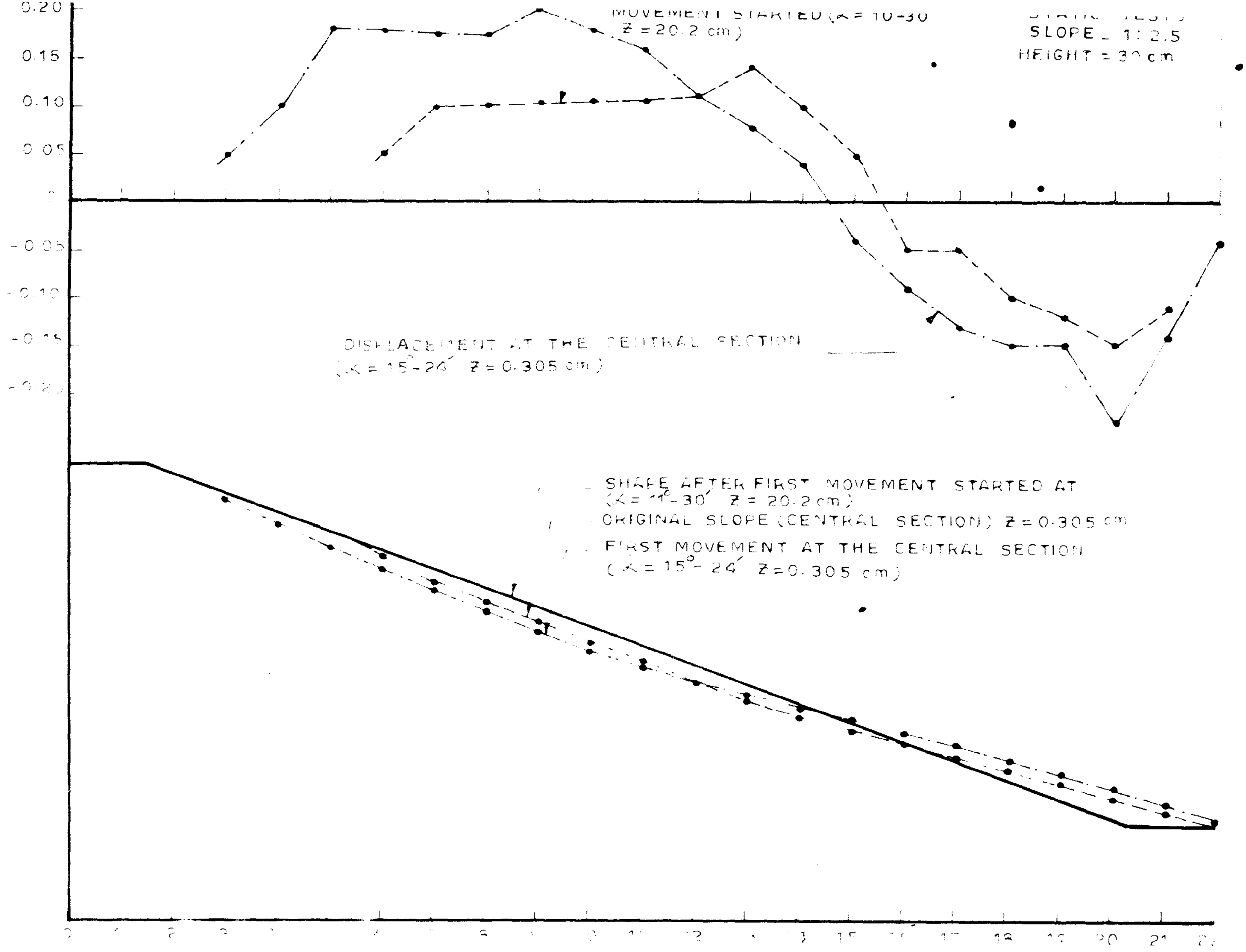


FIG. 4 _ YEILD ACCELERATION VS DISPLACEMENT CURVE



... IN ... OBSERVED AT DISTANCE z FROM THE END DURING ... REFERENCE POINTS 4 cm APART

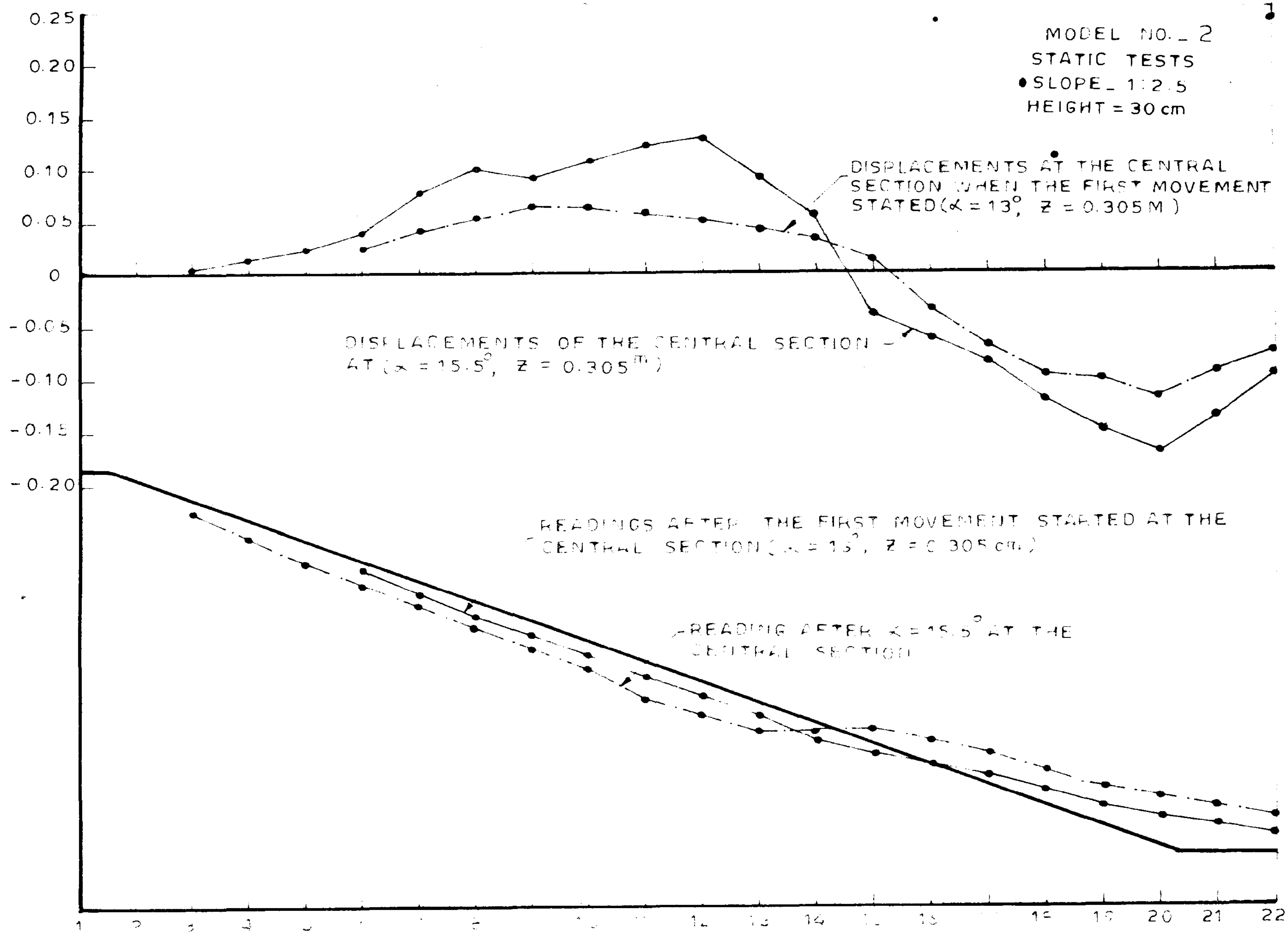


FIG 4 CHANGES IN PROFILE OBSERVED AT DISTANCE Z FROM THE END DURING THE STATIC TESTS REFRENCE POINTS 4 cm APART

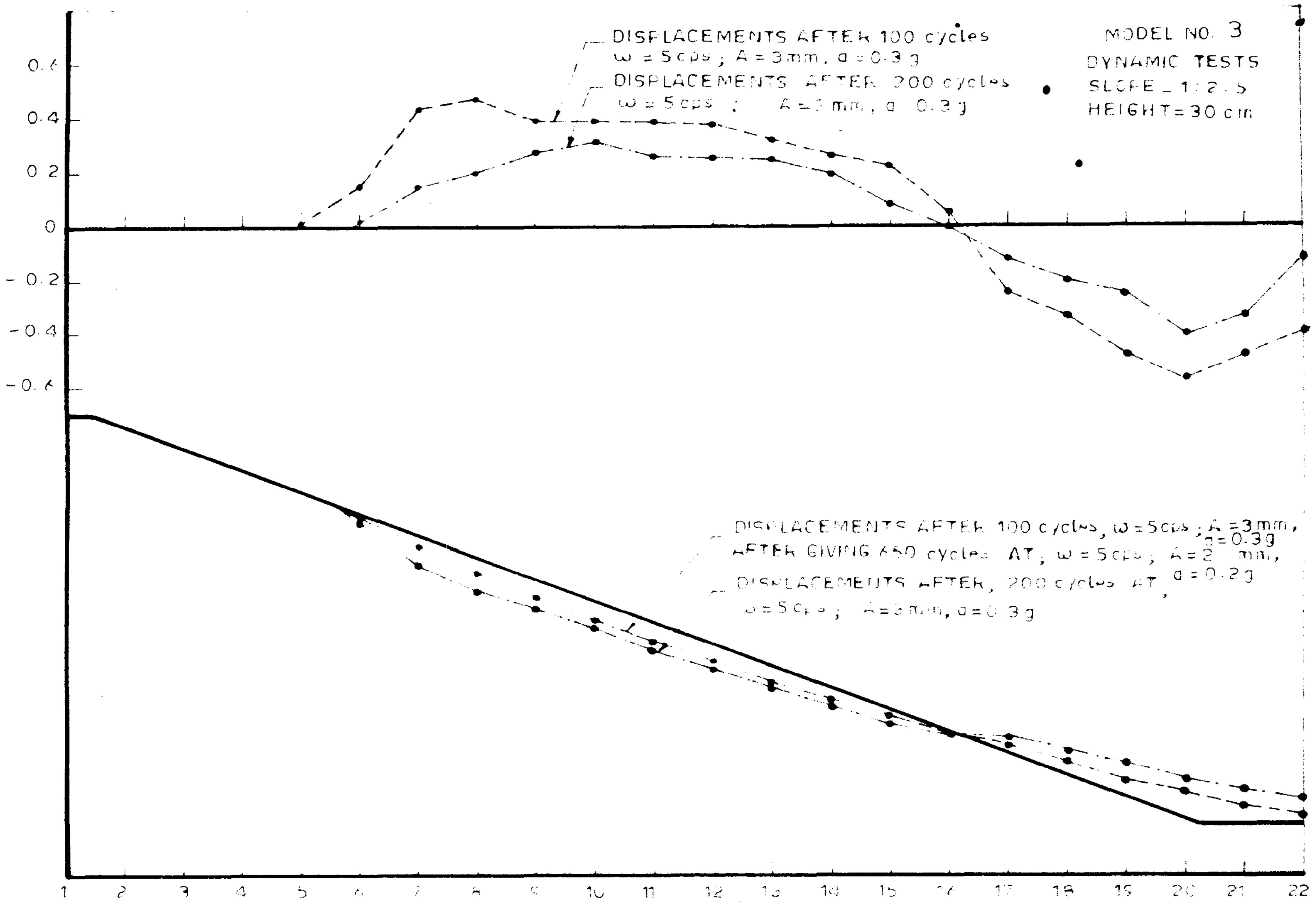


FIG. 7 _ CENTRAL SECTION DURING DYNAMIC TESTS REFERENCE POINTS 4 cm APART

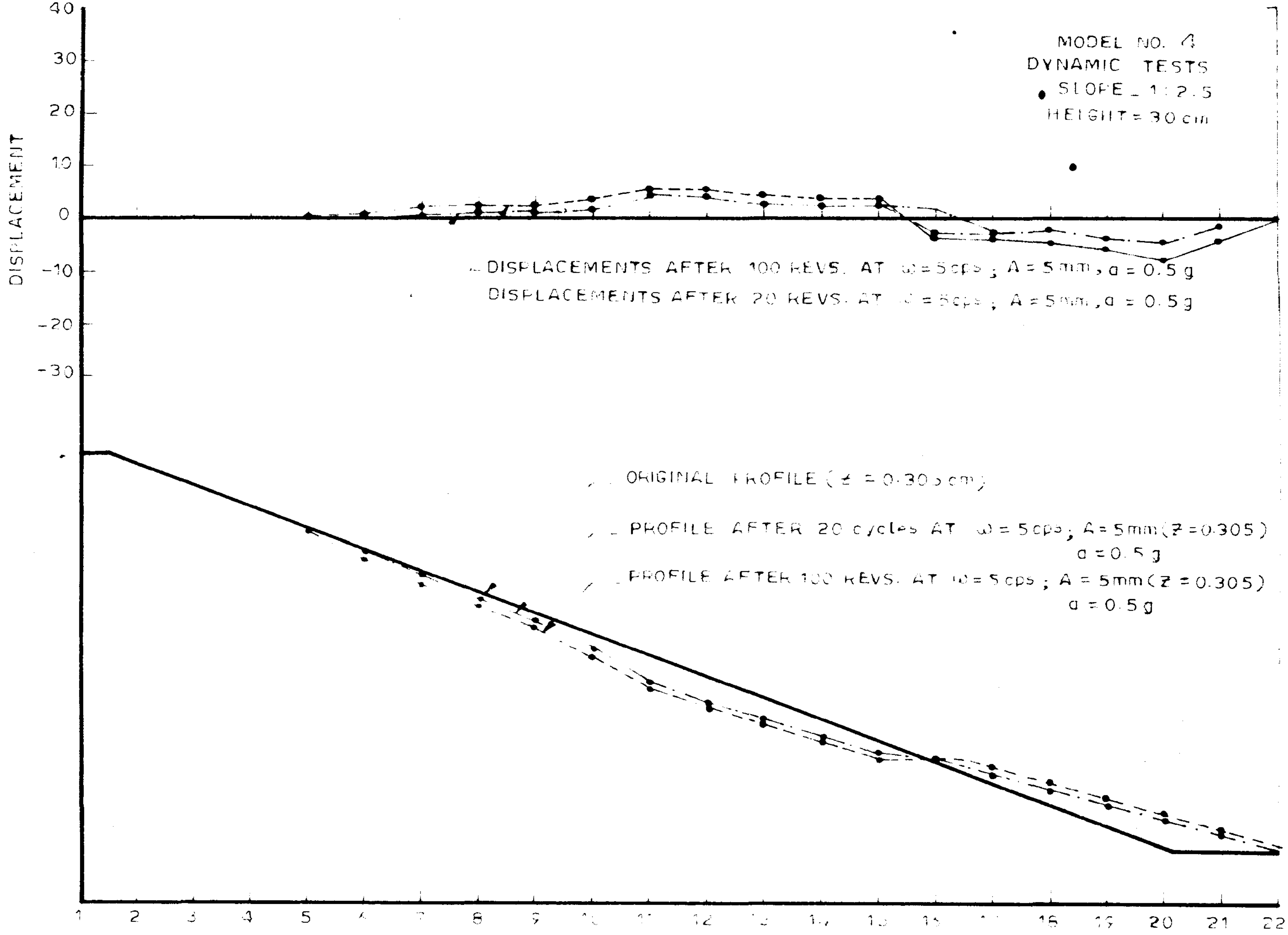


FIG. 8 _CHANGES IN PROFILE OBSERVED AT A DISTANCE Z FROM THE END REFERENCE POINTS 4 cm APART

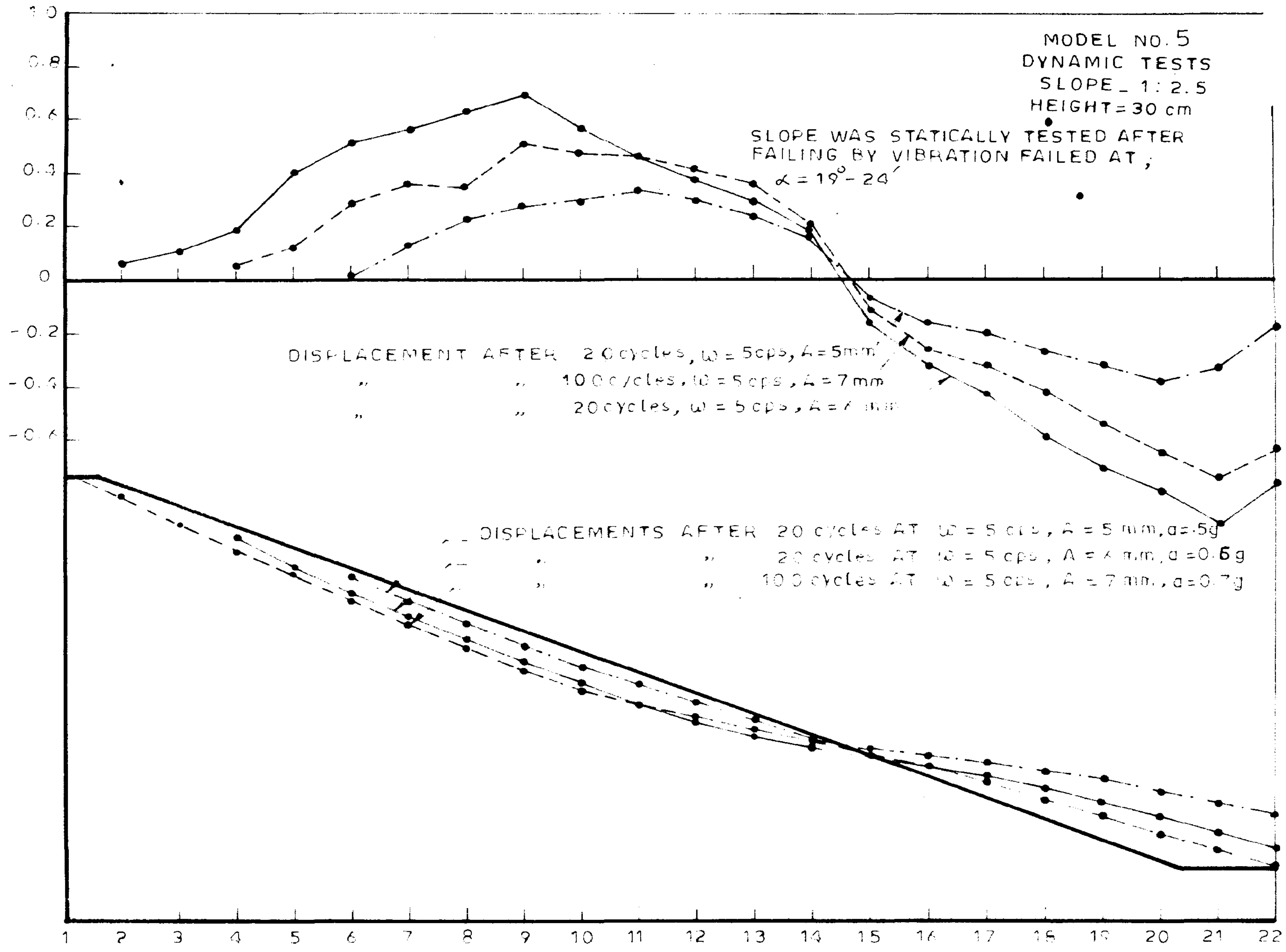


FIG. 9 _CHANGES IN PROFILE OBSERVED AT THE CENTRAL SECTION REFERENCE POINTS 4 cm APART

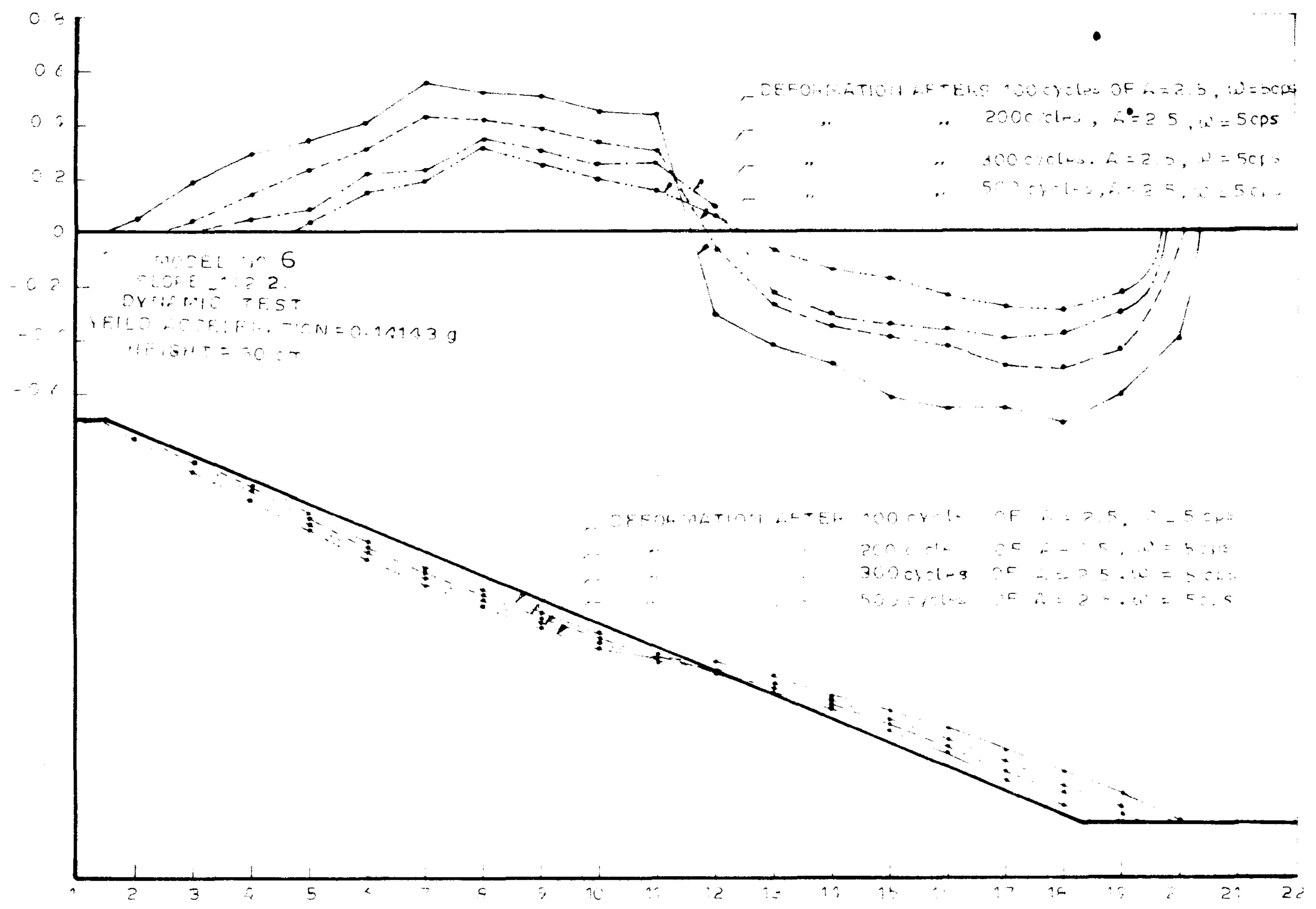


FIG.10 – CHANGES IN PROFILE OBSERVED AT CENTRAL SECTION (REFERENCE POINTS 4 cm APART, EXCITING ACCELERATION 2.5 g

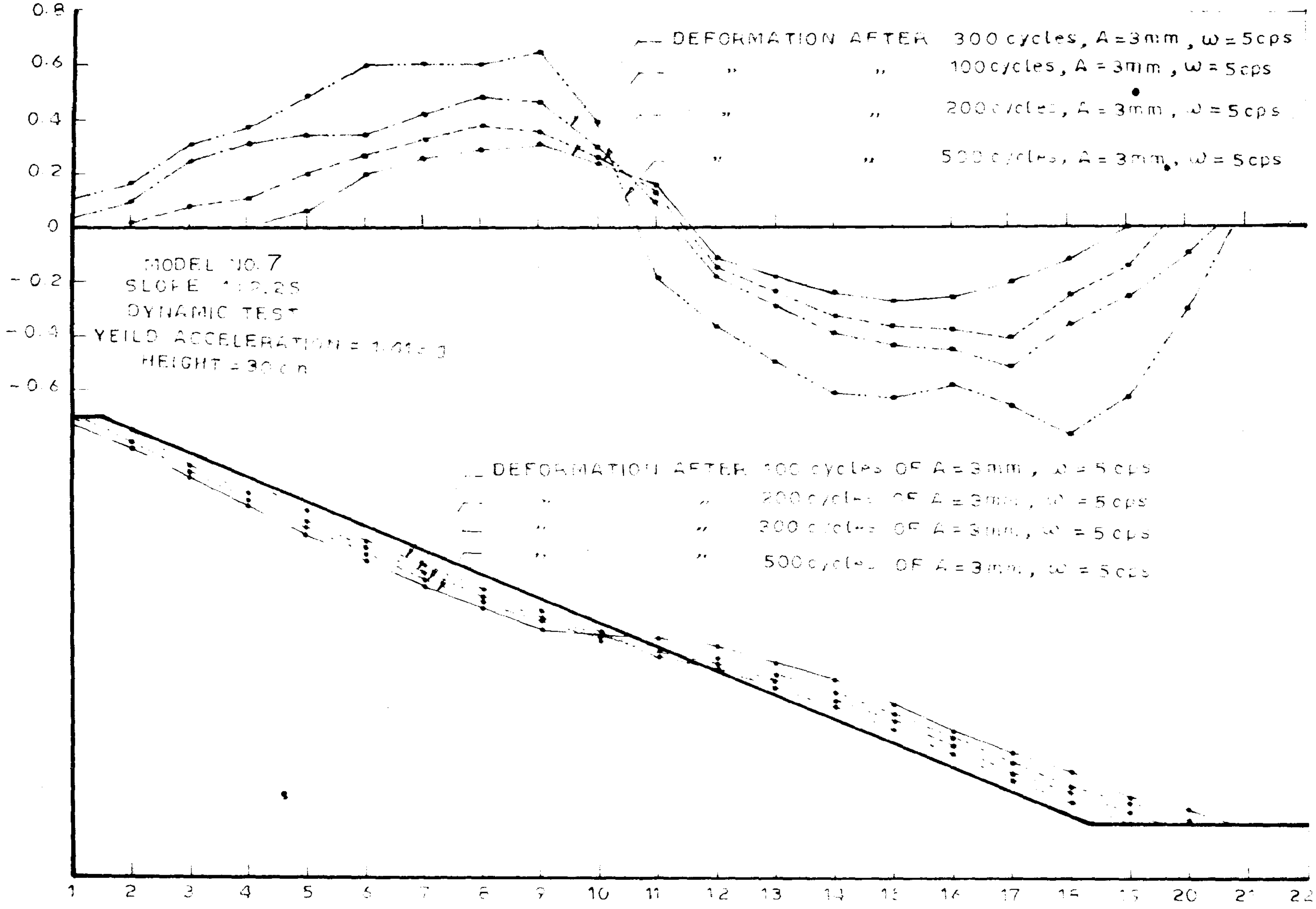


FIG. 11 – CHANGES IN PROFILE OBSERVED AT CENTRAL SECTION REFERENCE POINTS 4 cm APART SLOPE EXCITET AT 0.3 g

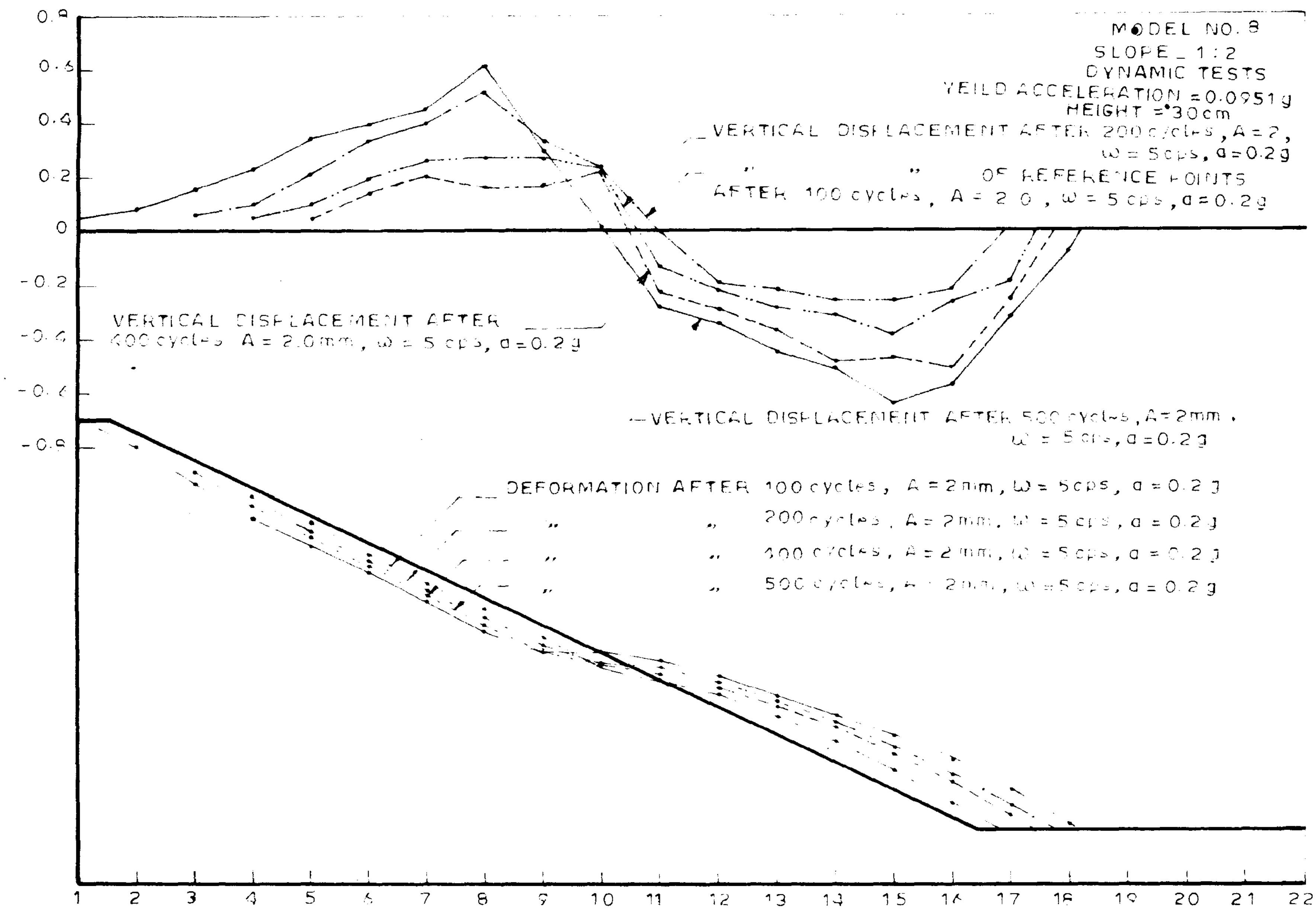


FIG. 12 _ CHANGES IN PROFILE OBSERVED AT CENTRAL SECTION DURING DYNAMIC TESTS REFERENCE POINTS _ 4 cm APART

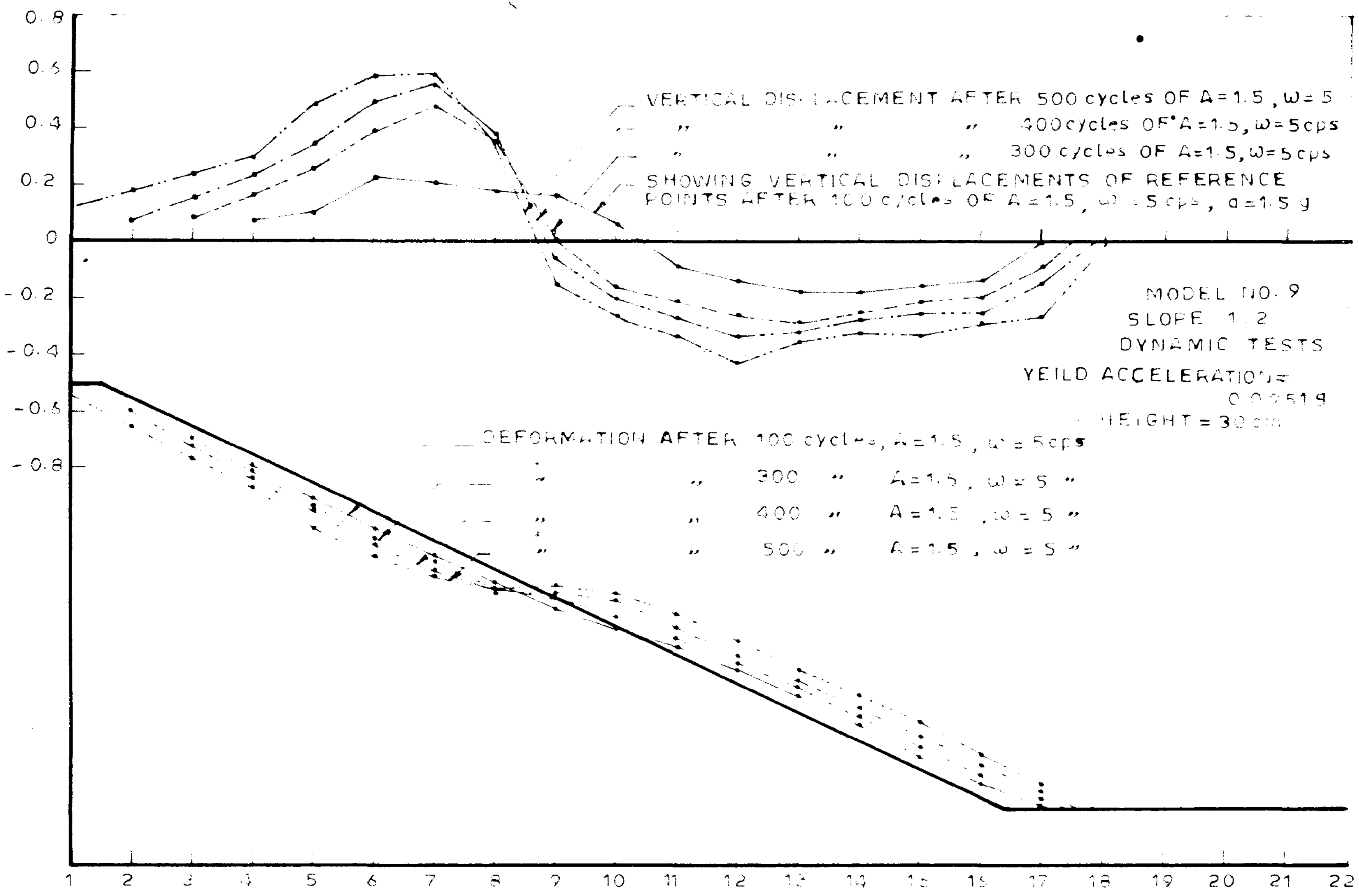


FIG.13 _CHANGES IN PROFILE AT THE CENTRAL SECTION DURING DYNAMIC TESTING REFERENCE POINTS 4cm APART SLOPE EXCITED AT 0.3 g

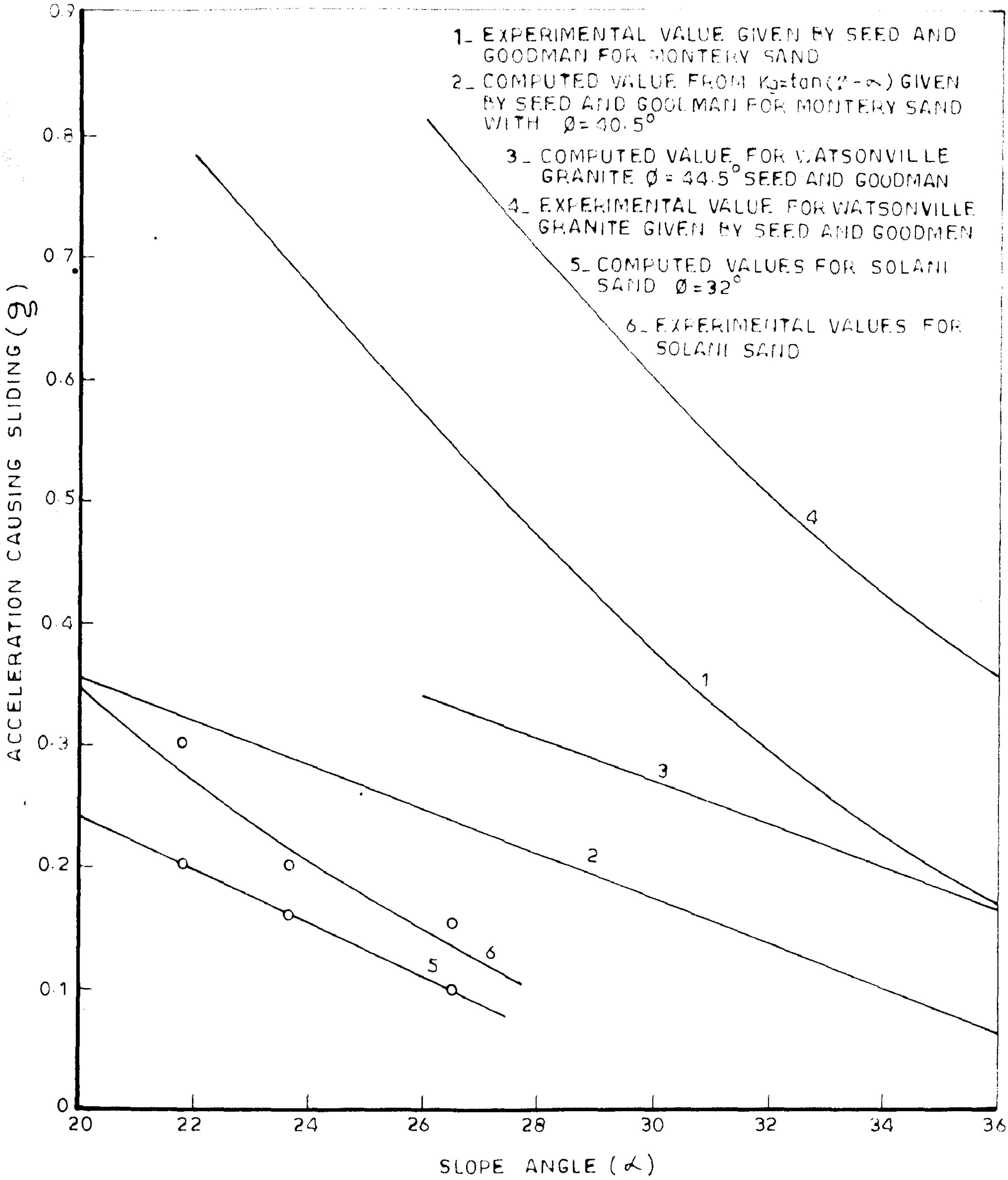


FIG. 14 _COMPARISION OF EXPERIMENTAL AND COMPUTED VALUE OF YEILD ACCELERATION FOR DIFFERENT SANDS

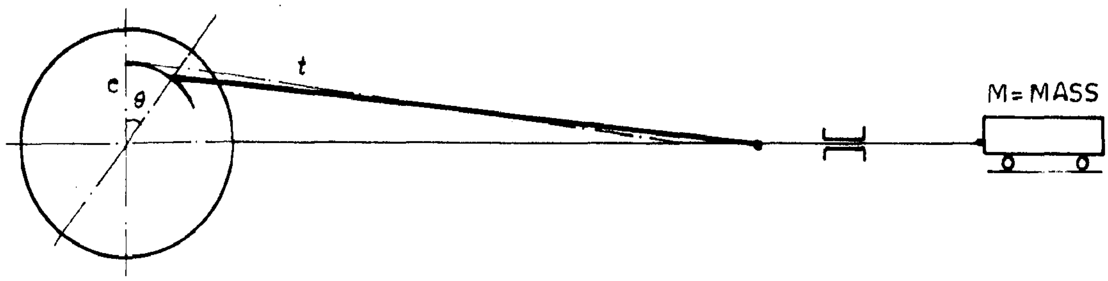


FIG. 15 A

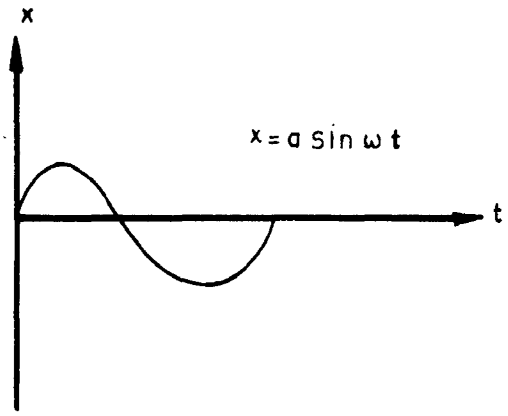


FIG. 15 B

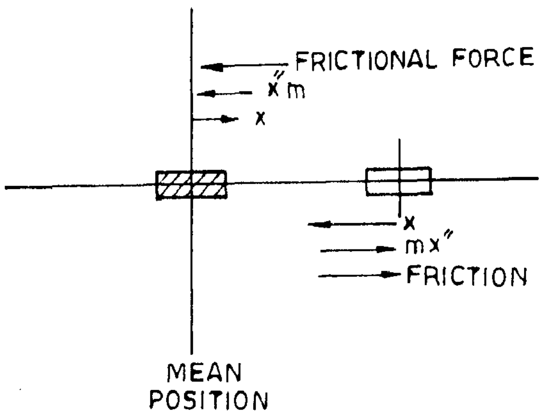


FIG. 15 C

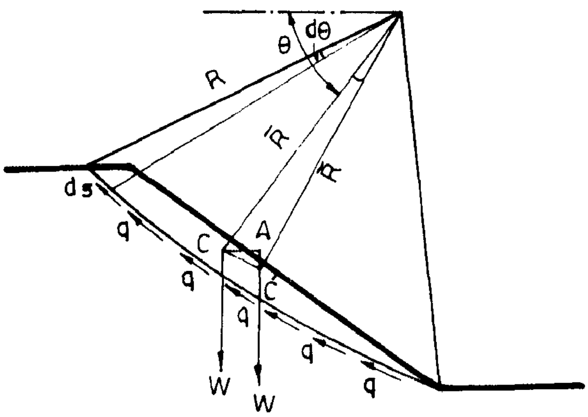


FIG. 15 D

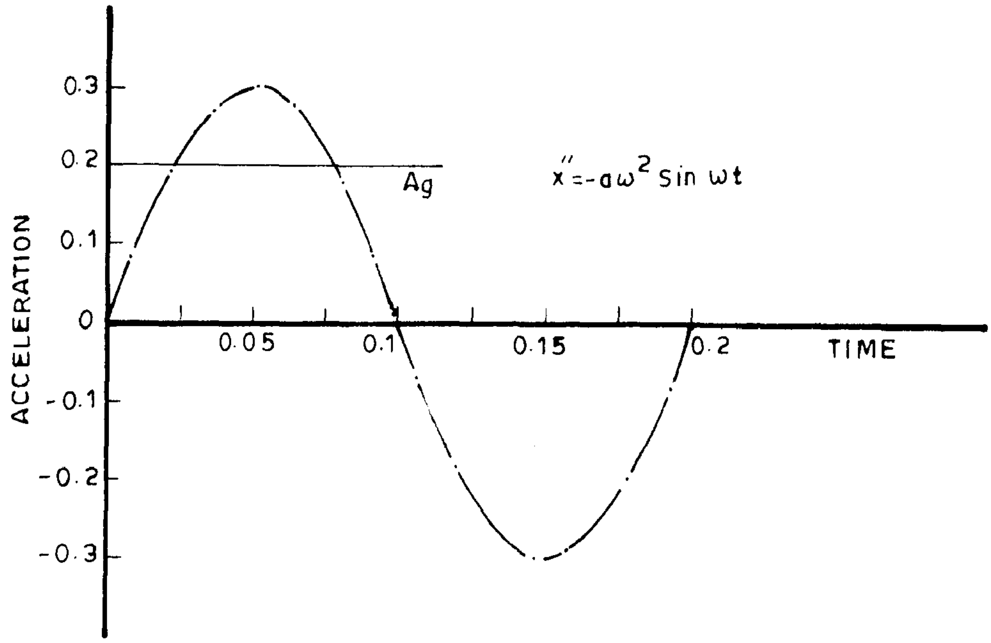


FIG. 16A

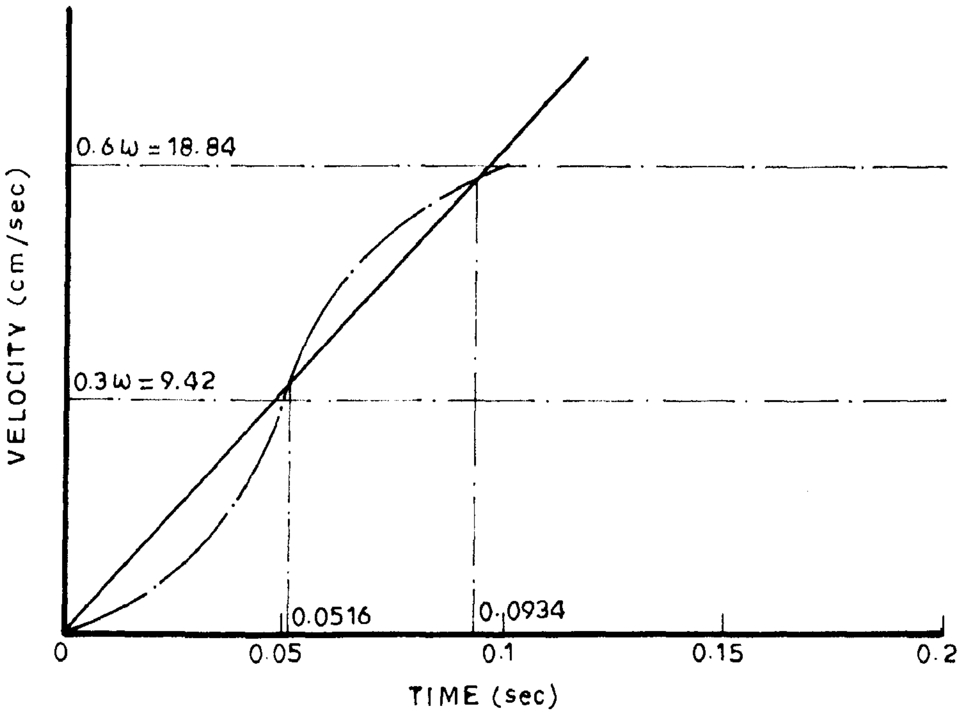


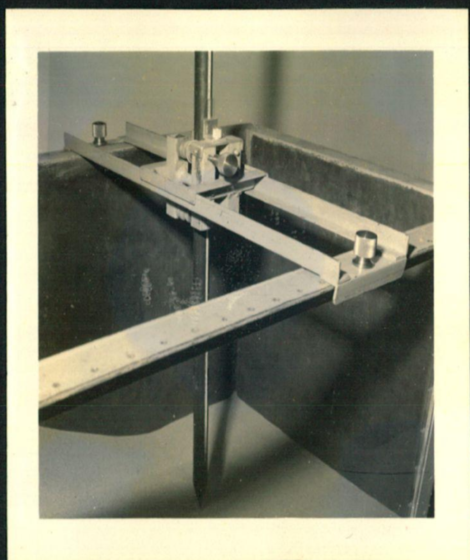
FIG. 16B_VELOCITY VS TIME PLOT FOR PARTICTLES

1. General Test Set-up

2. Tank shown in inclined position for the static test.

3. A view of the tank and the model before starting the static test.

4. Shadow of the tip of the profilometer on the slope.



5. Details of the profilometer

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