"THE TRANSMISSION OF GROUND MOVEMENT TO STRUCTURES"

by

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PREFACE

The nature and extent of ground movements resulting from the extraction of coal can be predicted with a fair degree of accuracy as a result of extensive research carried out in the past. There is, however, little quantitative information about the effect of such movements on structures. This information is necessary for the protection of existing structures from damage due to mining operations and for the design of new structures for erection in mining areas.

The work described here was carried out to study the factors governing the transmission of ground movement to structures, and the damage arising from the transmitted movement.

Section 1 is concerned with theoretical aspects of the transmission of ground movement to structures. The effects of vertical and horizontal movements are considered separately. In each case an expression is derived for the critical length of structure at which damage will occur.

Tests on existing structures which were subject to mining subsidence are described in Sections 2 to 4. The development and application of suitable techniques for the measurement of horizontal and vertical movements of ground and structures, and for the measurement of wall tilt, are described. Damage which occurred to the structures is listed and is related where possible to measured ground movements.

Section 5 describes tests which were carried out with sand models to study movements in a cohesionless mass subject to horizontal extension.

A paper entitled "SUBSIDENCE - The Transmission of Ground Movement to Surface Structures" was read to the Mining Institute of Scotland by the author and his supervisor. This paper has since been accepted for publication by the Institution of Mining Engineers.

CONTENTS

Section 1 THEORETICAL ASPECTS OF THE TRANSMISSION OF GROUND MOVEMENT TO STRUCTURES

1.1. Introduction.

Ground movements which occur at the surface over a mine working are three dimensional in character though, as a result of the methods used in their measurement, it is usual to consider the vertical and horizontal components separately. The factors affecting the distribution of such vertical and horizontal movements have been sufficiently discussed and documented by other workers [1.1, 1.2, 1.3] to render description of them here unnecessary. The object of the research described herein is the study of the effect of such ground movements on structures, with reference to the type and amount of ground movement which would cause damage and to the factors which govern the transmission of ground movement to structures. Though the damage caused by ground movement is a result of the combined effect of vertical and horizontal movement, for the purpose of the dissertation which follows the effect of each type of movement will be considered separately.

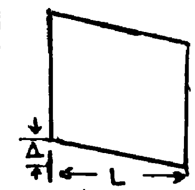
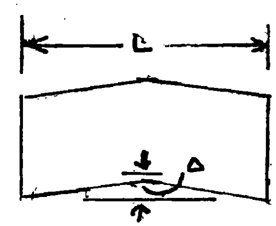
1.2. The Effect of Vertical Movement.

Differential vertical movement of the ground surface results in tilting which can be dangerous in the case of tall structures where it may lead to instability. Some structures must be kept level in order that they function properly (e.g. bridges, foundations of machinery etc.) and are, therefore, adversely affected by tilting; methods of relevelling such structures have been described [1.4]. The efficiency of sewage and land drainage layouts may also be impaired by reduction in the flow gradients necessary for the proper operation of the systems.

The strength characteristics of most conventional structures are such that a structure set on ground which is subject to vertical movement will, in general, deflect under the action of its own weight to conform to the shape of the ground surface. Thus the curvature of the ground surface, which is an essential feature of subsidence movement, will give rise to differential vertical movement over the area of a structure set thereon. Such differential movement, if large enough, will cause cracking of the structure.

A survey of published work was made to determine the magnitude of the relative deflection between adjacent parts of a structure which would result in cracking of the structure [Ref. 1.5 to 1.10]. The data obtained are shown in Table 1.1 ; it will be noted that the data fall into two groups which relate to two distinct types of deflection. Some of the authorities quoted in Table 1.1. relate the incidence of damage to the tilt of sections of a structure while the others relate it to the ratio Δ/L of the displacement 'Δ' at the centre to the length 'L' of the structure. The first concept is more useful in considering the incidence of damage to the brick infilling of framed structures when the frames are distorted out of square. In the second category the figures given relate to the displacement of load bearing brick walls set on concrete strip foundations. Consideration of the data given (Table 1.1.) suggests limits of deformation of $\frac{1}{300}$ in the first case and $\frac{1}{4000}$ in the second.

T A B L E 1.1

Authority	Type of Test	$\frac{\Delta}{L}$ for cracking	Tensile Stress	Remarks
Skempton & McDonald [1.5]	Analysis of damage to a number of buildings	$\frac{1}{300}$	-	Type of deflection 
F.G. Thomas [1.6]	Racking of brick infilled, concrete encased steel frame.	$\frac{1}{300}$ - $\frac{1}{400}$	80 - 100 lb/in^2	
Myerhoff [1.7]	Observation of brick walls of buildings	$\frac{1}{280}$	60 lb/in^2	
Terzaghi (Quoted by Myerhoff)		$\frac{1}{1000}$	-	Type of deflection 
Myerhoff	Consideration of published work	$\frac{1}{2000}$		
Rigby and Decoma [1.8]	Lightly reinforced brickwork on concrete strip footing the ends of which were jacked up.	$\frac{1}{7000}$ - $\frac{1}{45000}$	40 - 50 lb/in^2	
R.H. Wood [1.9]	Brick wall on span of 10 ft 6 in.	$\frac{1}{20,000}$	50 lb/in^2	
Polshin and Tokar [1.10]	Single storey buildings with plain brick wall length L and height H	$L/H \leq 3$ $\frac{1}{3000}$ - $\frac{1}{2500}$ $L/H \geq 5$ $\frac{1}{2000}$ - $\frac{1}{1400}$	Breaking Strain 0.5 m.m./m.	

Consider the deflection of a load bearing brick wall of length 'L' feet which is subject to curvature as a result of mining subsidence. If, when the radius of curvature of the ground and building is 'R' feet, the deflection of the centre of the wall relative to its ends is '$\triangle$' feet it can be shown that

$$R = \frac{L^2}{8\triangle} \qquad \text{------------------------------} \qquad (1.2.1)$$

Now consider the equation of the subsidence curve given by Whetton & King [1.11], namely

$$x = \frac{S}{2}\left(1 - \tan h \quad \frac{2\,y}{h \tan \alpha}\right) \qquad \text{-------------} \qquad (1.2.2)$$

where

x = subsidence at a distance 'y' from the point of inflection of the subsidence development curve.

S = amplitude of the subsidence development curve.

h = the depth to the panel being extracted.

α = the angle of draw.

The above equation was derived by Whetton and King as a result of experiments on gelatine models; it was shown to be in reasonable agreement with actual subsidence profiles measured in the field and is, therefore, suitable for use in this analysis.

Using equations (1.2.1) and (1.2.2) it can be shown that

$$\frac{1}{R} = \frac{d^2x}{dy^2} = \frac{8\Delta}{L} = \frac{4S}{h^2 \tan^2\alpha} \quad \frac{\sec h^3 2y}{h \tan\alpha} \quad \frac{\sin h\, 2y}{h \tan\alpha}$$

and that curvature is a maximum when $y = \pm 0.33\ h \tan\alpha$. As the important factor of the curvature is its magnitude the signs need not be considered.

Therefore maximum curvature $\dfrac{d^2x}{dy^2} = \dfrac{1.52\ S}{h^2 \tan^2\alpha} = \dfrac{8\Delta}{L.}$

which for average values of $\dfrac{\Delta}{L}$ and α of $\dfrac{1}{4000}$ and 35^{o} respectively gives

$$L_c = \frac{h^2}{1550\ S.} \qquad \text{------------------------} \qquad (1.2.3)$$

where L_c = critical length of structure in respect of the incidence of damage due to curvature.

Section 1.3. THE EFFECT OF HORIZONTAL MOVEMENT

1.3.1. General.

Differential horizontal ground movements at the surface are transmitted in a greater or lesser degree to structures set thereon, causing them to strain and, if the movements are large enough, to break. Research work over the years has provided a considerable amount of information about the magnitude and distribution of horizontal ground movement and has enabled methods of prediction to be evolved [1.12]; in addition a reasonable estimate can be made of the amount of movement a particular structure can stand. There is, however, a great lack of information about the factors which govern the transmission of ground movement to structures.

If structures had their foundations set into bed-rock there would be little difficulty in determining the relationship between ground and structural movement. Most structures, however, have foundations resting in unconsolidated material and it is necessary to investigate the factors affecting the transmission of movement through this unconsolidated material to relatively rigid structures.

<u>1.3.2. The transmission of horizontal ground movement by unconsolidated materials</u> is a problem which could best be investigated by field measurement, over mine workings, of movement at the rock head and at the surface of the unconsolidated material. Suitable sites where measurements of this type can be made are difficult to find, but the application of simple soil mechanics theory helps to elucidate the problem in a general way.

Consider the state of stress in a soil mass with horizontal surface, subject to horizontal movement. The shearing resistance 'τ' on a plane in a soil mass is related to the normal stress 'σ' on that plane, by the equation

$$\tau = \sigma \tan \phi + C \qquad \text{------------} \qquad (1.3.1)$$

where ϕ = angle of internal friction

$c \quad = \quad$ coefficient of cohesion of the material.

It can be shown by reference to standard soil mechanics theory [1.13] that a state of tensile stress cannot exist in a cohesionless mass (i.e. one in which $C = 0$) even under a condition of elongation. In such a case failure takes place along planes inclined at an angle of $(45 - \frac{\phi}{2})^{\circ}$ to the vertical. It can be shown that under this condition the relationship between the horizontal principal stress 'σ_H' and the vertical principal stress 'σ_V' is

$$\sigma_H = \sigma_V \tan^2\left(45 - \frac{\phi}{2}\right) \text{ ---------- } (1.3.2)$$

Similarly in a cohesionless mass, subject to compression, it can be shown that failure takes place along a plane inclined at $\left(45 + \frac{\phi}{2}\right)^{\circ}$ to the vertical when the relationship between σ_H and σ_V is

$$\sigma_H = \sigma_V \tan^2\left(45 + \frac{\phi}{2}\right) \text{ ---------- } (1.3.3)$$

Thus, considering the case of failure in a mass of dry sand, the nearest practical approach to an ideal cohesionless material, and taking a typical value of $\phi = 40^{\circ}$ it can be shown that

$$\sigma_H \simeq 0.22\ \sigma_V \text{ -------------------- } (1.3.2a)$$

for failure resulting from elongation of the mass

and $$\sigma_H = 4.56\ \sigma_V \text{ -------------------- } (1.3.3a)$$

for failure resulting from compression of the mass.

In the case of a soil mass with cohesion, subject to extension, it can be shown that a state of tensile stress can exist in a horizontal direction for certain values of σ_V , namely

$$\text{when} \quad \sigma_V \leqslant 2\,C \tan\left(45 + \frac{\phi}{2}\right) \geqslant \gamma h. \qquad \text{———————— (1.3.4)}$$

where γ = specific weight of material

h = depth to point under consideration

Taking typical values of C, ϕ and γ of 500 lb/ft^2, 15° and, 110 lb/ft^3 it can be shown that a state of tensile stress can exist to a depth of approximately 12 ft.

For a cohesive material the relationships between σ_H and σ_V when failure due to elongation or compression takes place, are as follows

$$\sigma_H = \sigma_V \tan^2\left(45 - \frac{\phi}{2}\right) - 2C \tan\left(45 - \frac{\phi}{2}\right)$$

for elongation ———————— (1.3.5)

$$\sigma_H = \sigma_V \tan^2\left(45 + \frac{\phi}{2}\right) + 2C \tan\left(45 + \frac{\phi}{2}\right)$$

for compression. ———————— (1.3.6)

The typical values of 'ϕ' and 'C' given above give

$$\sigma_H = 0.59\,\sigma_V - 770 \qquad \text{———————— (1.3.5a)}$$

for failure due to elongation

$$\sigma_H = 1.69\,\sigma_V + 1300 \qquad \text{———————— (1.3.6a)}$$

for failure due to compression.

Since in the undistributed state the horizontal principal stress

Since in the undisturbed state the horizontal principal stress 'σ_H' is approximately equal to the vertical principal stress 'σ_V' it is obvious that compressive ground movements will give rise to considerable lateral pressure on parts of structures below ground level. (1.3.3a, 1.3.6a).

In an attempt to test the applicability of this theory in the case of cohesionless material, some tests with sand models were carried out. These tests are described in Section 5.

<u>1.3.3. The transmission of horizontal movement from soil to a structure</u> can easily be visualised in the case of a cohesive material in which tensile stresses can exist at or near the surface. Bearing in mind that tensile stresses cannot exist in a purely cohesionless material, it is difficult to visualise how a material such as dry sand can transmit tensile movement to a structure. To elucidate this matter consideration will be given to some theoretical and practical aspects of the transmission of ground movement to structures.

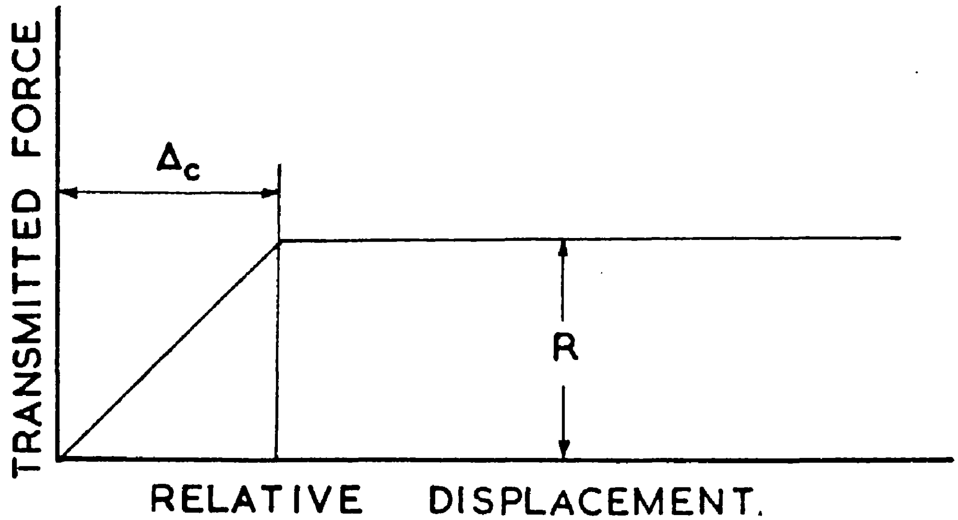
When a structure is sited on soil which is subjected to movement as a result of mining operations relative movement of the soil and the foundation of the structure may take place. Experiments at the Road Research Laboratory [1.14] where concrete slabs were pushed and pulled over different types of soil showed that, when relative movement took place between slab and soil, the frictional resistance to movement varied with the magnitude of the relative movement up to a maximum value;

subsequent relative movement took place against this limiting resistance. Values of maximum resistance between 100 and 300 lb/ft^2 at relative displacements of 0.01 to 0.05 in. were found for materials ranging from smooth sand to rough clinker. Though these results were obtained by moving slabs over soil there is justification for using the results in considering the movement of soil past slabs, or other flat bottomed structures as it is the relative movement between the two which is important in each case.

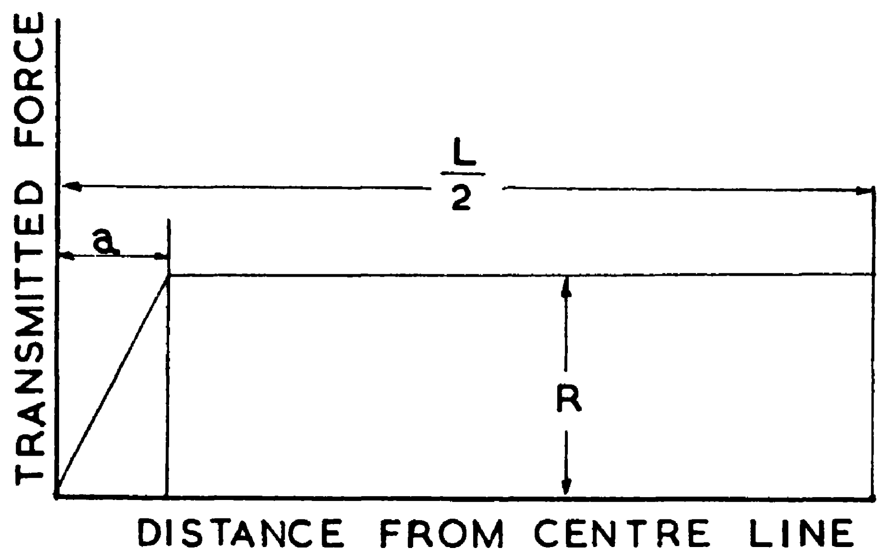
Consider the transmission of strain to a structure with a modulus of elasticity 'E' lb/ft^2, base width 'W' ft, length 'L' ft and sectional area 'A' ft^2. Suppose that the structure is set on unconsolidated material subject to a horizontal extension of '$\mathcal{E}_G$' mm/m. The following assumptions will be made

1) The force transmitted to the structure increases uniformly with increase in relative displacement between soil and slab up to a limiting value of 'R' lb/ft^2, at a relative displacement of 'Δ_c' ft. For relative displacements greater than Δ_c the force transmitted is constant at R lb/ft^2. Fig. 1.1a.
2) The centre line of the structure does not move relative to the ground.
3) The relative displacement at points away from the centre line is proportional to the distance of the points from the centre line.

The assumed relationships regarding the force transmitted between



(A)



(B)

<u>ASSUMED RELATIONSHIPS</u>

<u>FIG. 1.1.</u>

the soil and structure are as shown in Figs. 1.1. a and b.

It is obvious that the distance, 'a' ft from the centre line of the structure to the point where maximum transmitted force obtains, will depend on the magnitude of the ground strain $\mathcal{E}_G$. At small values of $\mathcal{E}_G$ the critical displacement may not be reached within the length, 'L' ft, in which case the force at the centre line of the structure will be given by

$$F_{\text{℄}} = \frac{WRL^2}{8a} \qquad \text{-----------} \qquad (1.3.7)$$

At large values of $\mathcal{E}_G$ the critical displacement Δ_c will be attained within the length of the structure and the force at the centre line will be given by

$$F_{\text{℄}} = \frac{WR}{2}(L - a) \qquad \text{-----------} \qquad (1.3.8)$$

There will be a value of $\mathcal{E}_G$ which will just cause the critical relative displacement 'Δ_c' to be attained at the end of the structure i.e. there will be a value of $\mathcal{E}_G$ where $a = \frac{L}{2}$. In this case the force at the centre line of the structure will be given by

$$F_{\text{℄}} = \frac{WRL}{4} \qquad \text{-----------} \qquad (1.3.9)$$

Now it can be shown that the strain set up in the structure is negligible compared with the ground strain $\mathcal{E}_G$. Therefore

$$\Delta_c = \frac{\mathcal{E}_G . a}{1000} \qquad \text{-----------} \qquad (1.3.10)$$

If the value of $\mathcal{E}_G$ at which $a = \frac{L}{2}$ is $\mathcal{E}_G'$ then from (1.3.10)

$$\mathcal{E}_G' = \frac{2000\,\Delta_c}{L} \qquad \text{------------------------} \quad (1.3.11)$$

The total force at the centre line of the structure will be given by eq.(1.3.7) for $\mathcal{E}_G < \mathcal{E}_G'$ and by eq.(1.3.8) for $\mathcal{E}_G > \mathcal{E}_G'$

Consider next the stresses set up at the centre line of the structure as a result of the above forces; suppose that the sectional area of the structure is 'A' ft^2 and its modulus of rigidity is 'I' ft^4. If the length of the structure is small in comparison with its height then the stress at the centre section will be eccentrically distributed over the height of the structure. If the distance from the neutral axis to the bottom edge of the slab is 'y_1' ft. then the stress at any section at a distance 'y' ft from the neutral axis is given by

$$\sigma = F_{\mathrm{CL}}\left(\frac{1}{A} + \frac{yy_1}{I}\right) \qquad \text{------------------------} \quad (1.3.12)$$

The maximum stress will occur at the bottom edge and will be given by

$$\sigma_{max.} = F_{\mathrm{CL}}\left(\frac{1}{A} + \frac{y_1^2}{I}\right) \qquad \text{------------------------} \quad (1.3.12a)$$

or $\sigma_{max.} = F_{\mathrm{CL}}\ K_1$

where $K_1 = \dfrac{1}{A} + \dfrac{y_1^2}{I}$

If, on the other hand, the length of the structure is large in comparison with its height the stress at the centre section will be more or less uniformly distributed and will be given by

$$\sigma = \frac{F_{\mathfrak{t}}}{A} \quad \text{------------------------} \quad (1.3.13)$$

In general the maximum value of stress at the centre section will have some value between that given by eq.(1.3.12a) and (1.3.13) but as the most critical condition is that expressed in eq.(1.3.12a) this equation will be used in the analysis which follows.

Failure of the structure will occur when $\sigma_{max} \geqslant \sigma_{U}$, the ultimate tensile stress of the material from which it is constructed, i.e. for cracking

$$F_{\mathfrak{t}} \; K_1 = \sigma_{U}$$

For values of $\mathcal{E}_G \geqslant \mathcal{E}_G'$ and using eqs.(1.3.8) and (1.3.10) it can be shown that

$$L_c = \frac{2\sigma_{U}}{K_1 RW} + \frac{1000\,\Delta_c}{\mathcal{E}_G} \quad \text{---------------} \quad (1.3.14)$$

where L_c = critical length of structure with respect to damage due to extension.

For values of $\mathcal{E}_G \leqslant \mathcal{E}_G'$ and using equations (1.3.7) and (1.3.10) it can be shown that

$$L_c^{\,2} = \frac{K_2}{\mathcal{E}_G} \quad \text{------------------------} \quad (1.3.15)$$

where $K_2 = \dfrac{8000\,\Delta_c \sigma_U}{R\,K_1 W.}$

The graph of L_c against $\mathcal{E}_G$ is shown in Fig. 1.2.

As $\mathcal{E}_G$ tends to ∞ L_c, eq.(1.3.14), tends to a limiting value of $2\,\sigma_U/K_1 RW$ which is a constant for a given set of circumstances. Let this limiting value of L_c be called the 'maximum safe length' of structure. It is obvious that a structure of length less than the maximum safe length will not suffer damage due to ground strain, irrespective of the magnitude of that ground strain.

Consider next the factors which affect the maximum safe length of structure, L_{max}, where

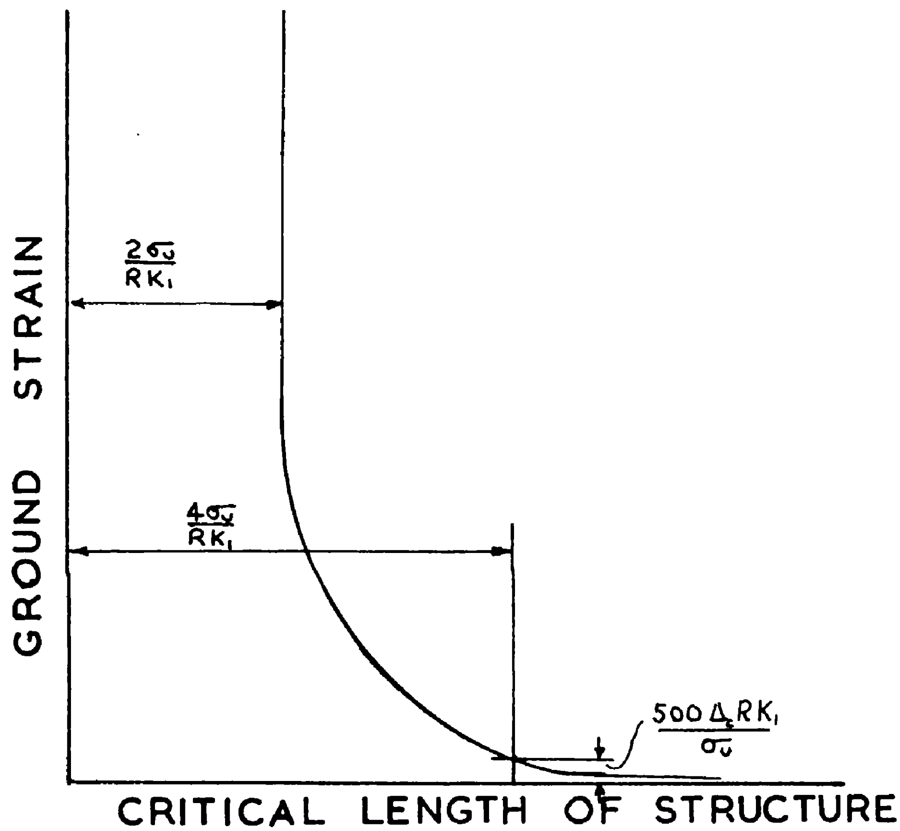
$$L_{max} = \frac{2\,\sigma_U}{K_1 R W.} \qquad \text{------------------------} \quad (1.3.16)$$

The effect of each of the quantities in the above expression will be considered in turn.

The strength of the material from which the structure is constructed, σ_U, will be fairly constant for the normal brick or concrete structure. In general the strength of the structure can be increased by the provision of tensional reinforcement. The most effective position for this tensional reinforcement can be seen from consideration of the effect of the constant K_1

where $K_1 = \dfrac{1}{A} + \dfrac{y_1^2}{I}$

For a given cross sectional area, 'A' it is obvious that L_{max} will



EFFECT OF GROUND STRAIN ON CRITICAL LENGTH OF STRUCTURE.

FIG. 1.2.

increase with increase in 'I', the modulus of rigidity of the structure, and with decrease in 'y_1' the distance from the neutral axis to the base of the structure. These desiderata can be achieved simultaneously by the provision of tensional reinforcement as near ground level as possible.

In general the width of the foundation 'W' will be a constant for a given structure.

Reduction of the value of 'R' will cause increase in 'L_{max}'. This reduction can be achieved by laying the foundation on a smooth layer of a material with low shear strength. Such a procedure has been described by Gibson [1.15].

1.3.4. CONCLUSIONS

The foregoing analysis has been made on the basis of experiments, with concrete slabs, carried out at the Road Research Laboratory [1.14]. Certain simplifying assumptions were made. In order that the validity of the analysis can be tested field trials are necessary. The object of these trials should be

a) to determine the relationship between the force R, transmitted to a structure and the relative displacement between ground and structure for different types of soil,

b) to determine the relationship between the bearing pressure and the force, R, transmitted for different types of soil.

This information could be obtained by laying a series of long, narrow slabs of concrete on ground which is to undergo movement arising from mining operations; measurements could be made of ground strain, slab strain and relative horizontal displacement between ground and slab. By siting the slabs close together and laying them on different types of underlay the effect of different soil types, for a given set of strain conditions, could be investigated. Such field tests could not be carried out as no suitable sites could be found.

Several sites were made available by the National Coal Board where existing structures were to be undermined. Though there was not the variation in soil type, foundation type and structure type that was desired it was felt that valuable information could be obtained by making measurements, at these sites, of ground and structure movements. The work carried out on these sites is the subject of Sections 2, 3 and 4.

Section 2.1 CARDOWAN HOUSE SITE

A plan of the site is shown in Fig. 2.1.

When work commenced in October 1957 a face in the Meiklehill Wee Seam was being worked in a south easterly direction under the grounds in front of Cardowan House. The following data relate to the working :-

Width of face	500 ft
Thickness of extraction	26 in. - 29 in.
Dip	1 in 15 North east
Depth of Cover	1,300 ft

The ground in front of Cardowan House dips in a southerly direction at about 1 in 12. The plot marked 'A' in Fig. 2.1 is enclosed on the south and east by a brick wall which varies in height between 5 and 10 feet, the lowest portions being on the eastern, i.e. the sloping side.

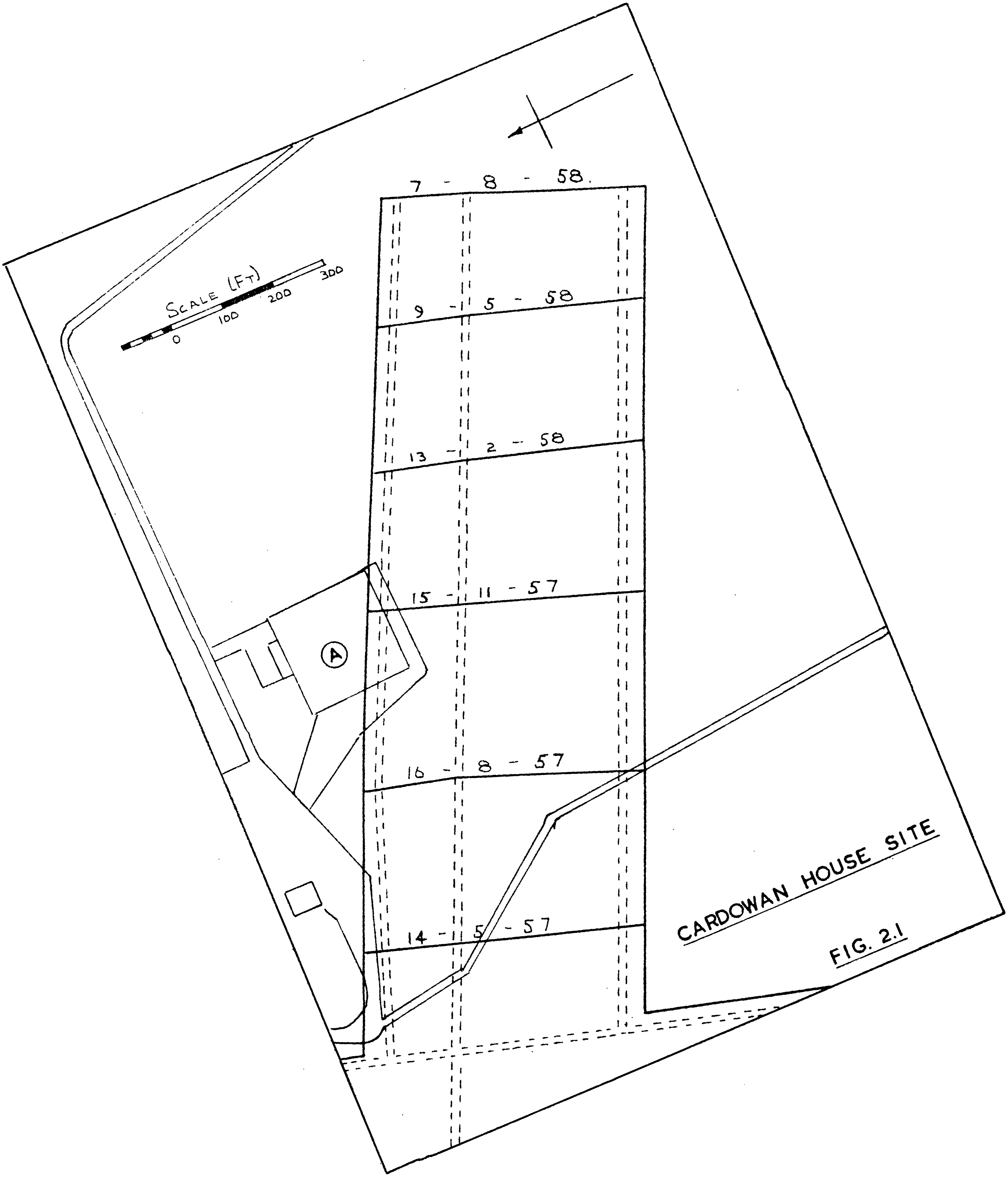
To determine if differential vertical movement was taking place between the ground and the wall it was decided to measure -

a) the change of slope of the wall and ground with time,

and b) the change of level of the wall and ground with time.

To determine if differential horizontal movements were occurring in the wall and ground it was decided to measure -

a) change in distance between stations in the wall,



CARDOWAN HOUSE SITE

FIG. 2.1

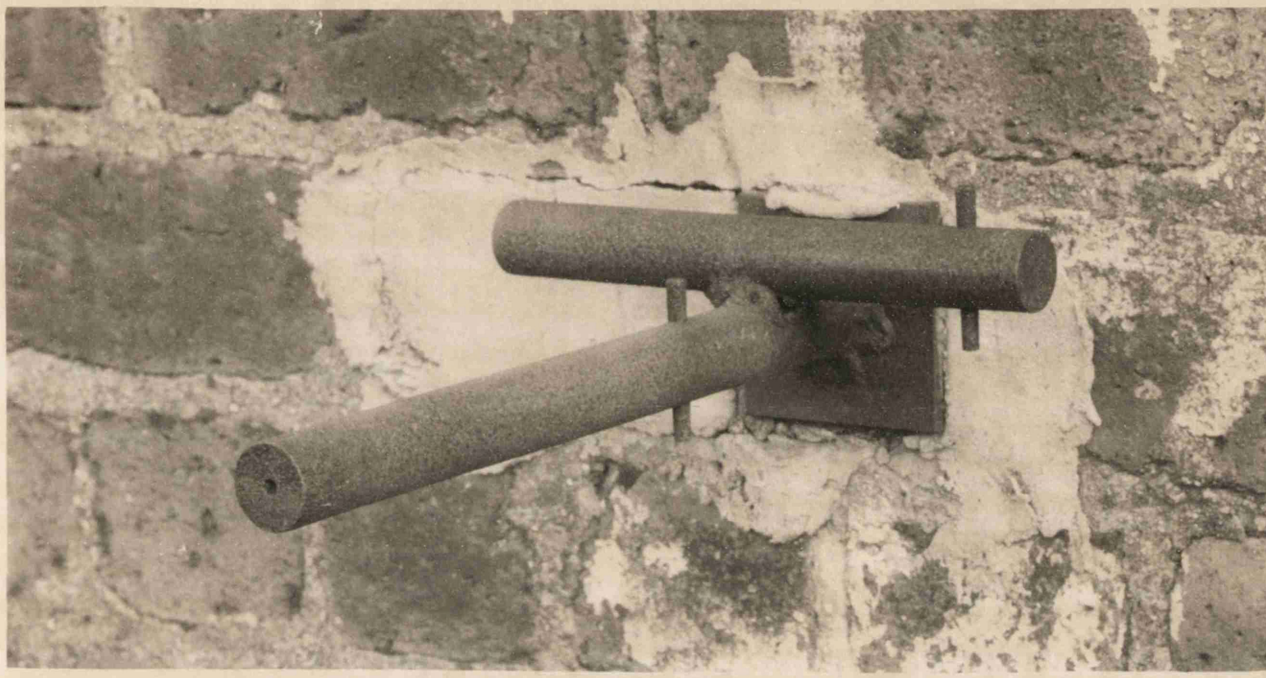


PLATE 2.1.   WALL STATION.

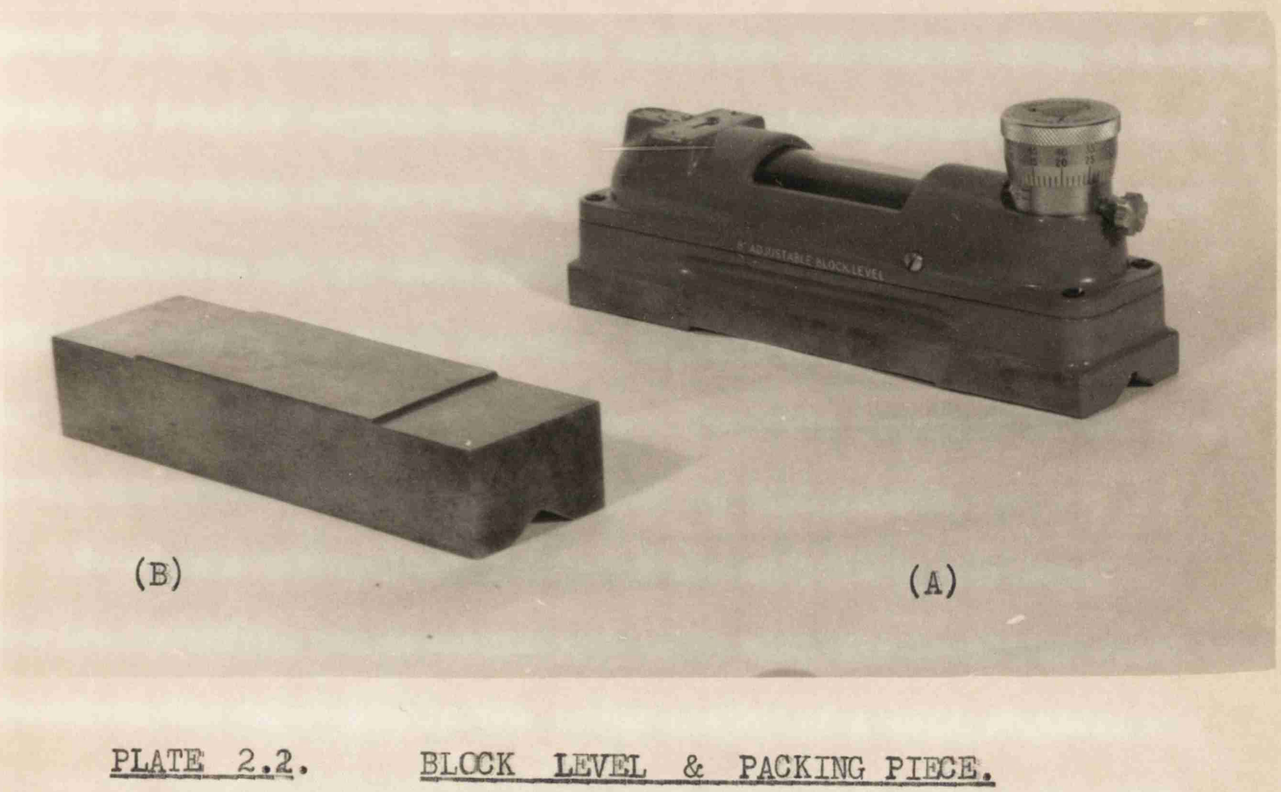


PLATE 2.2.   BLOCK LEVEL & PACKING PIECE.

and b) change in distance between stations in the ground.

Section 2.2 MEASURING STATIONS

2.2.1. Wall Stations.

Details of the wall stations are shown in Plate 2.1. The stations were fixed in prepared holes in the wall (with the main and cross members horizontal) by means of nut. Cement grout was placed in the hole to provide a firm anchorage. Thirty two stations in all were installed at intervals of approximately 12 feet. The spacing was dictated by the presence of strengthening butts built at 12 foot intervals along the wall. The 16 stations on the south wall were installed on a level course approximately $5\frac{1}{2}$ feet above ground level. Those on the east wall were installed along a uniform grade at heights varying from 4 to 6 feet above ground level.

The tilting of the wall was recorded by measuring the changes in inclination of the main and cross members of the stations using a Watts' Adjustable Block Level. By this means the tilt perpendicular and parallel to the wall was found. Positive location of the block level was provided by a taper pin set in each member.

To enable vertical movements of the wall to be detected, precise levelling measurements were made with the levelling staff placed on the taper pin in the cross member of a wall station.

2.2.2. Ground Stations.

The ground stations (see Fig. 2.2) consisted of 3 ft lengths of $\frac{3}{4}$ in. bore malleable iron pipe set in concrete. The top end of each was screwed internally and externally. A brass plug with a domed head and engraved cross was screwed into the internal thread. The outside thread was to take a detachable, screw-on extension piece. The station heads were set below ground level and protected by cast iron drain covers from accidental damage by animals.

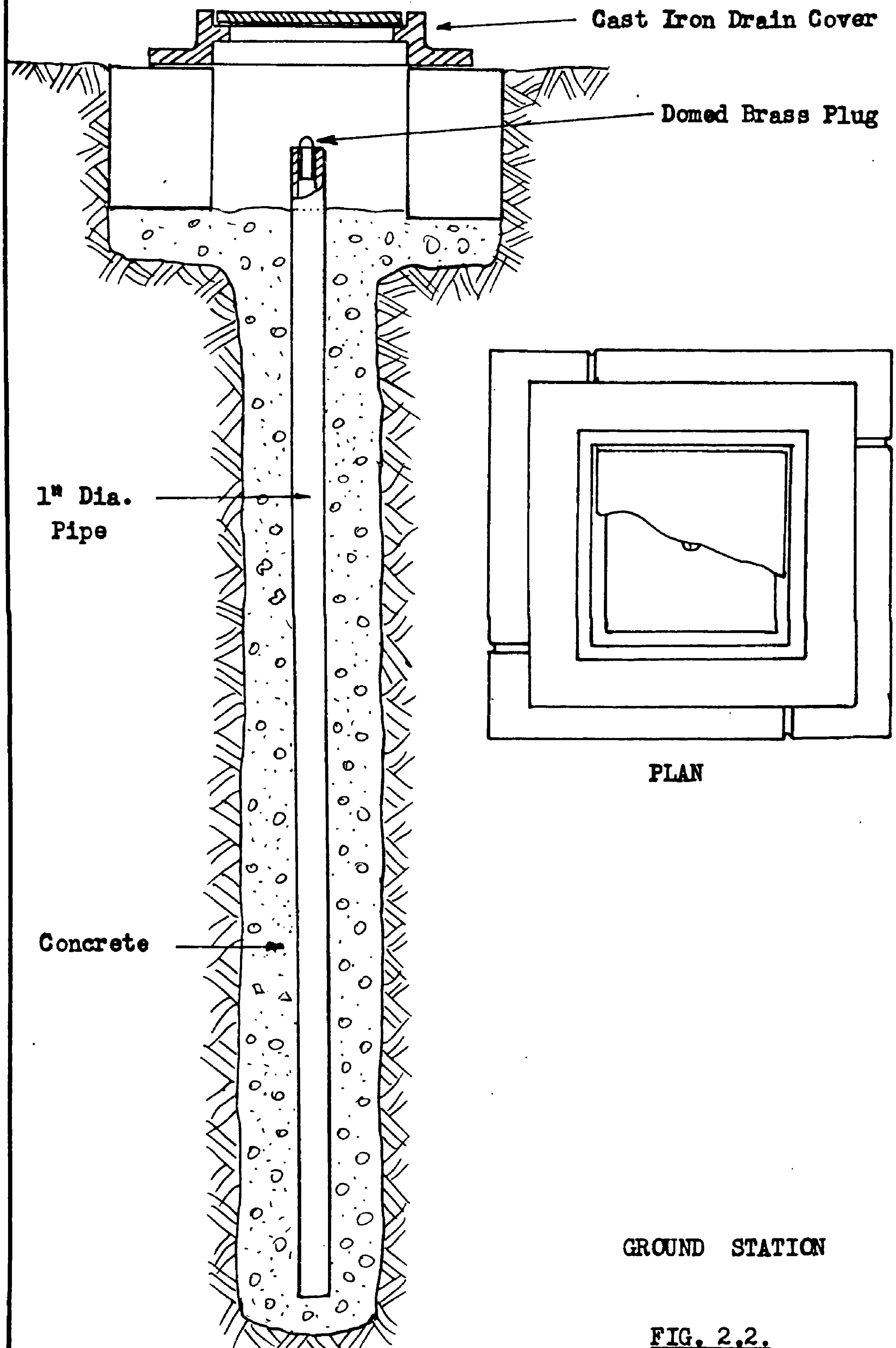
Section 2.3 MEASURING APPARATUS AND TECHNIQUES

2.3.1. Vertical Movements.

The apparatus used for the detection of vertical movements was of a type previously used and described by other workers in the field of subsidence research. [2.1] ; it comprised a precise level (Cooke, Troughton & Simms S.500 geodetic level) and invar staff.

The levelling procedure which was evolved for use on the site was as follows :-

The ground stations were levelled and all levels related to station 32 Ground. Six instrument settings were required due to the slope of the ground and the presence of trees. The staff was read with the level on both faces to eliminate collimation errors, and the mean value calculated. Fore sights were made immediately following back sights to minimise error resulting from settlement



GROUND STATION

FIG. 2.2.

of the instrument. Intermediate sights were then made. The back sight reading was checked to see if settlement had occurred. Any error noted was allocated proportionately to all intermediate sights on the assumption that settlement had taken place at a uniform rate with time.

As a check the line was relevelled using the same stations as change points. No intermediate sights were taken. The mean value of the vertical distance between adjacent change points was then calculated and used in the calculation of reduced levels, provided that the two measured values of a vertical distance did not differ by more than 2/1,000 ft. Where the discrepancy was greater than this value, the vertical distance between the two stations was remeasured till two readings were obtained which came within the prescribed limits. Checking was done during the second levelling so that discrepancies would be discovered in the field and any supplementary measurements made forthwith.

The vertical distance between ground and wall stations was then obtained by positioning the instrument so that both stations could be levelled at the one setting. When making measurements on wall stations the staff had to be held by the staff man while sitting on top of the wall. As steadying poles could not be used in this position it was difficult to hold the staff steady in a wind. In an attempt to overcome this difficulty a short staff was constructed from a draughtsman's scale and fitted with a spirit level. On test the small staff proved to be more unsuitable than the large on the following counts.

(a) The staff was difficult to read due to the faint graduations and to the lack of light in the shadow of the wall.

(b) The staff was difficult to hold steady while standing on a ladder, a procedure rendered necessary by the height of some of the stations.

(c) The rate of measuring was slowed down since the level had to be moved and re-set for every station on the sloping east wall.

The measurements were therefore continued using the large staff, measuring being limited to calm days.

The vertical distance between corresponding ground and wall stations was measured twice and the mean value accepted, unless the individual measurements differed by more than 2/1,000 ft. In such a case further measurements were made.

2.3.2. Inclination Measurements.

A Watts' Adjustable Block Level was used for measuring the inclination of the main and cross members of wall station. The instrument is shown in Plate 2.2a. The level has a range from -2° to $+3^{\circ}$. The micrometer drum is graduated in degrees and minutes, and the bubble tube in intervals of 20 sec.

As slight roughness on some of the station members was causing damage to the machined faces of the V-groove on the level, a packing piece was constructed (Plate 2.2b). The packing piece has a machined V-groove to fit over the station members and a flat machined upper surface with a locating spigot to take the block level. One end of the packing piece was marked to ensure that the packing piece was always presented to the measuring station correctly oriented. This precaution was taken to allow for any lack of parallelism between the upper and lower surfaces of the packing piece which would introduce an error if the packing piece were turned end for end between two measurements on a station. The packing piece was tested for parallelism by making several measurements on a station, turning the packing piece through 180° between consecutive measurements. It was found to be parallel within the limits of accuracy of the block level.

The measuring procedure with the block level was as follows :-

The packing piece was set on a station member with the marked end butting against the taper pin. The block level was placed squarely on top of the packing piece and the cross level bubble brought to zero by rotating the assembly about the member. The micrometer drum was rotated to bring the main bubble near the centre of its run; the micrometer was set to the nearest minute and the reading noted; both ends of the main bubble were read, estimating to 1/10th of a division. The block level was then turned through 180° and the reading repeated to allow for the axis' of the bubble being not parallel to the base of the level. The mean of the two readings of inclination was taken as the inclination of the member.

2.3.3. Measurement of Horizontal Movements.

The magnitude of the maximum horizontal ground movement likely to occur in the vicinity of the measuring stations was estimated to be 0.4 m.m./m. tension. This would cause an increase in the distance between stations of 0.06 inches. To measure the development of such movements a measuring device with a high degree of accuracy was required. The measurement was rendered difficult by the following factors.

(i) Wall stations were not uniformly spaced (distance between stations ranged from 11 feet 9 inches to 12 feet 3 inches) nor were they parallel to each other.

(ii) Ground stations were of necessity below ground level.

W.H. Ward [2.2] claims an accuracy of ± 0.001 inches on a 10 ft. gauge length when using a rod fitted with a micrometer under steady temperature conditions. Since steady temperature could not be expected at the site, any device similar to that used by Ward would require to be of invar to minimise error in the application of a correction for thermal expansion. The use of an invar strip as a gauge length, with a micrometer for measuring lengths in excess of the gauge length, was considered. The means of locating the ends of the measuring device on the stations, especially those below ground level, proved a difficulty. After unsuccessful trials with mechanical means of location it was decided to try to locate the ends by optical means - using optical plummets to set up over or under the station. Macca base line measuring equipment (Plate 2.3) was tested to find if it could be applied to the job in hand. The apparatus was set up over the measuring stations then centred over the stations using the optical plummets 'A'. The plummets were replaced by tape micrometers 'B' and the distance between the measuring marks read from the invar tape.

Measurement of the distance between the tape micrometers could be repeated to within 1/10,000 ft while, with replumbing between measurements, a range of 10/10,000 ft was obtained. It was thus obvious that the main source of error lay in the plumbing operation. Since it was impossible to modify the existing stations to provide positive location for the tape micrometers it was decided that optical plumbing be used for measurements on the site. By the time the necessary equipment was delivered by the manufacturer ground movements on the site had virtually ceased. No measurement of the horizontal movements in the ground or wall were made. With the absence of such measurement the main object of the test was lost.

It was obvious from trials with the base line measuring equipment that the technique was very time consuming and required at

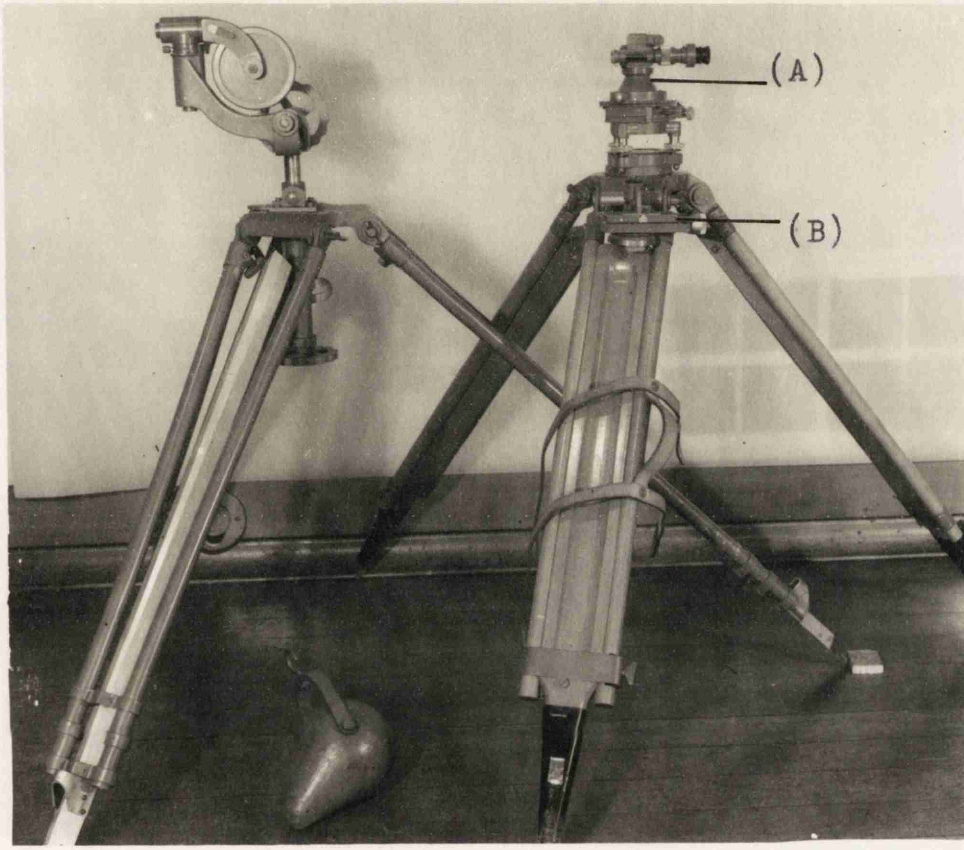


PLATE 2.3. MACCA EQUIPMENT

Parts (A) and (B) are interchangeable

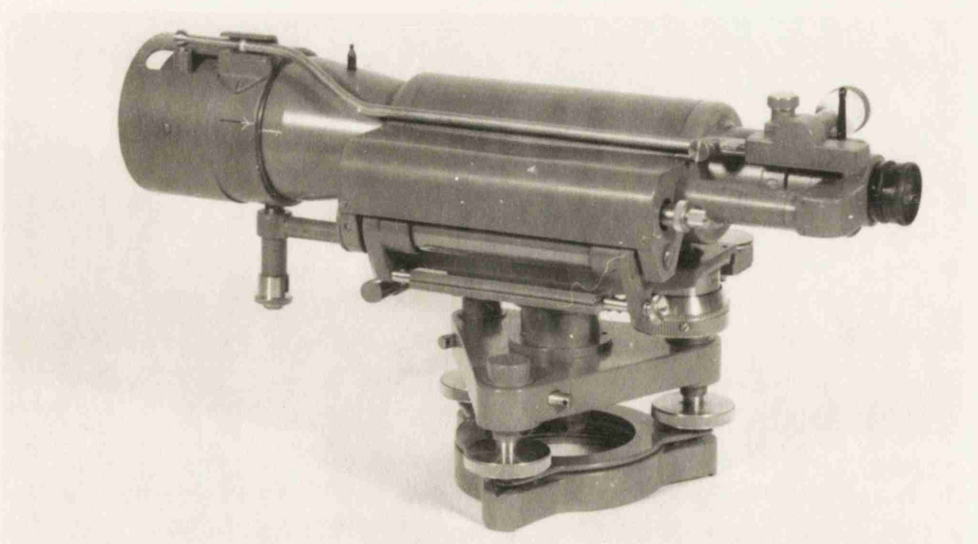


PLATE 2.4. S.500 GEODETIC LEVEL

least four men for its operation, also the accuracy obtained was not sufficient to justify the time and labour expended. The 95% confidence limits for the mean of three measurements of the distance between ground stations, (the measuring heads being replumbed between each measurement) was found to be $\pm$ 1.5/1,000 ft. It was, however, apparent that an invar tape, under standard tension and used in conjunction with tape micrometers, was capable of giving a measure of the distance between two stations to within 1/10,000 ft provided that a positive means of locating the micrometers on the station markers could be developed. This could be done in the case of a wall station by mounting a clamping device on the wall; in the case of a ground station a concrete beacon with its top some inches above ground level would be required. Three difficulties are at once apparent.

(1) Cost, (initial and replacement in event of damage).

(2) The difficulty of finding a site where such fittings would not interfere with or be damaged by normal use of the land or buildings.

(3) Measuring stations would be conspicuous and therefore liable to malicious damage.

The first difficulty can be reduced by using fewer measuring stations set at greater distances apart. This would also reduce the fractional error of the measuring technique as the error in reading the tape is independent of the length, long lengths thus giving smaller fractional errors than short ones. The use of widely spaced stations is not permissible when measuring over shallow seams since under these conditions the magnitude of ground movements may change considerably over a comparatively short distance.

The difficulties listed as 2 and 3 were in fact the governing factors in the choice of the measuring technique which was evolved for use on other sites.

Section 2.4 RESULTS

2.4.1. Vertical Movements.

Periodic measurements were made of the difference in level between

a) ground stations,

and b) adjacent ground and wall station.

The results obtained are summarised in Tables 2.1 and 2.2.

From the data in Table 2.1 the following were calculated -

Average tilt in ground adjacent to East Wall - 2.3 sec.

Average tilt in ground adjacent to South Wall - 1.5 sec.

The ground tilted uniformly along the length of each wall.

Reference to Table 2.2 shows that there was no significant differential vertical movement of the ground and wall.

TABLE 2.1.

SUBSIDENCE OF GROUND STATIONS RELATIVE TO STATION 3.2.

Stn.	13/1/58 to 20/2/58	8/4/58	30/4/58	2/6/58	3/7/58	5/8/58
	Total Sub. (Ft)	Total Sub. (Ft)	Total Sub. (Ft)	Total Sub. (Ft)	Total Sub. (Ft)	Total Sub. (Ft)
1						
2						
3		0.0629	0.0786	0.0994	0.1189	0.1307
4		0.0590	0.0767	0.0976		
5		0.0584	0.0749	0.0957	0.1140	0.1260
6		0.0577	0.0723	0.0930		
7		0.0571	0.0713,	0.0918	0.1095	0.1217
8						
9	0.0414	0.0962	0.1099	0.1295	0.1481	0.1612
10	0.0407	0.0936	0.1116	0.1312		
11	0.0383					
12	0.0367	0.0867	0.1012	0.1203		
13	0.0344	0.0830	0.0941	0.1128	0.1300	0.1430
14	0.0336	0.0800	0.0921	0.1102		
15	0.0320	0.0741				
16	0.0295	0.0734				
17	0.0294	0.0708	0.0802	0.0972	0.1136	0.1238
18						
19	0.0248	0.0577	0.0654	0.0794	0.0921	0.0101
20	0.0221	0.0520	0.0583	0.0716	0.0820	
21	0.0194	0.0459	0.0497	0.0622	0.0717	0.0779
22	0.0167	0.0405	0.0448	0.0553	0.0645	
23						
24	0.0152	0.0347				
25	0.0141	0.0293	0.0298	0.0372	0.04	0.0429
26	0.0116	0.0242	0.0249	0.0304	0.0355	
27	0.0097	0.0202	0.0229	0.0260	0.0301	0.0318
28	0.0067	0.0152	0.0155	0.0188	0.0224	
29	0.0052	0.0111	0.0107	0.0143	0.0156	0.0169
30	0.0035	0.0073	0.0072	0.0082	0.0105	0.0114
31	0.0010	0.0024	0.0048	0.0052	0.0074	0.0075
32						

TABLE 2.2.

VERTICAL DISTANCE BETWEEN GROUND AND WALL PEGS

Stn.	Vert. Dist. (Ft) 14th April		Vert. Dist. (Ft) 1st May	Period Change (Ft)	Vert. Dist. (Ft) 13th June	Period Change (Ft)	Vert. Dist. (Ft) 3rd July	Period Change (Ft)	Vert. Dist. (Ft) 5th Aug.	Period Change (Ft)
3	5.2187		5.2171	-0.0016	5.2175	+0.0004	5.2168	-0.0007		
5	5.2849		5.2845	-0.0004	5.2857	+0.0012	5.2850	-0.0007	5.2844	-0.0006
7	5.1783		5.1786	+0.0003	5.1777	-0.0009	5.1782	+0.0005		
9			5.8973		5.8968	-0.0005	5.8973	+0.0005		
11			6.0645		6.0658	+0.0013	6.0659	+0.0001		
13			5.9115		5.9117	+0.0002	5.9114	-0.0003	5.9109	-0.0005
15			6.0523		6.0507	-0.0016	6.0519	+0.0012		
17	6.1022		6.1055	+0.0033	6.1036	-0.0019	6.1048	+0.0012	6.1033	-0.0015
19	6.7308		6.7301	-0.0007	6.7299	-0.0002	6.7314	+0.0015	6.7314	-0.0000
21	6.3622		6.3631	+0.0009	6.3623	-0.0008	6.3629	+0.0006	6.3641	+0.0012
23			5.9307		5.9318	+0.0011	5.9326	+0.0008	5.9316	-0.0010
25	5.6531		5.6523	-0.0008	5.6527	+0.0004	5.6544	+0.0017	5.6531	-0.0013
27	5.1925		5.1920	-0.0005	5.1922	+0.0002	5.1917	-0.0005	5.1917	0.0000
29	4.5356		4.5359	+0.0003	4.5354	-0.0005	4.5355	+0.0001	4.5357	+0.0002
31	3.8493		3.8493	0.0000	3.8492	-0.0001	3.8499	+0.0007	3.8500	+0.0001

2.4.2. Angular Movements.

The change in inclination of the main and cross members of the wall stations was measured. The data obtained are shown in Tables 2.3 and 2.4. The sign convention adopted for the angular movements is as follows :-

Clockwise rotation in the plane of either wall, viewed from inside, is positive.
Movement outwards in the plane at right angles to the wall is positive.

Graphs were drawn to show how the movement of the wall varied with time. Fig. 2.3 shows that movements of the cross members of the stations in the south wall were similar for all stations except station 3. Movement at station 3 was markedly in excess of that at other stations. A different pattern of movement was shown by the station members at right angles to the wall (Fig. 2.4). It will be seen that in this case the curves fall into two groups corresponding to the two parts of the wall. Movement at station 3 was not markedly different from that at other stations in the same group. Fig. 2.5 also shows the difference in movement of the two sections of the south wall. The movements in sections 9 to 16 were uniform and comparable in magnitude to the ground movement; movements in the other sections were comparable with ground movement at stations 7 and 5, but were greatly in excess of ground movement at stations 3 and 2. There were no noticeable peculiarities in the ground or wall in the vicinity of

TABLE 2.3.

CHANGE OF INCLINATION OF MAIN PEGS

	CHANGE OF INCLINATION								
STATION	22/1/58 to 31/1/58	31/1/58 to 13/2/58	13/2/58 to 27/2/58	27/2/58 to 16/4/58	16/4/58 to 29/4/58	29/4/58 to 4/6/58	4/6/58 to 30/6/58	30/6/58 to 6/8/58	PERIOD
1	-0° 1' 42"	+0° 0' 5"	+0° 1' 14"	+0° 0' 38"	+0° 0' 49"	+0° 0' 38"			
2	-0° 1' 10"	-0° 0' 7"	+0° 1' 17"	+0° 0' 45"	+0° 0' 25"	+0° 0' 9"			
3	-0° 2' 26"	-0° 0' 33"	+0° 1' 45"	+0° 0' 3"	+0° 0' 37"	+0° 0' 14"			
4	-0° 2' 16"	-0° 1' 9"							
5	-0° 2' 30"	-0° 0' 48"			-0° 0' 13"	+0° 0' 51"			
6		-0° 0' 22"	+0° 1' 40"	-0° 1' 30"	+0° 0' 32"	-0° 0' 15"			
7	-0° 1' 46"	-0° 0' 47"	+0° 1' 41"	-0° 0' 38"	+0° 0' 55"	+0° 0' 22"			
8	-0° 1' 14"	-0° 1' 0"			+0° 0' 21"				
9		-0° 0' 35"	+0° 1' 46"	-0° 0' 40"	+0° 0' 39"	+0° 0' 0"	+0° 1' 28"	-0° 1' 7"	
10	-0° 4' 4"	-0° 1' 0"							
11	-0° 5' 56"	-0° 0' 18"			+0° 0' 37"	+0° 0' 1"	+0° 1' 25"	-0° 0' 5"	
12	-0° 8' 24"	-0° 0' 15"	+0° 1' 52"	-0° 1' 6"	+0° 0' 48"				
13	-0° 2' 44"	-0° 1' 21"	+0° 2' 1"	-0° 0' 47"	+0° 0' 55"	-0° 0' 24"	+0° 1' 25"	-0° 0' 5"	
14	-0° 0' 58"	-0° 1' 20"	+0° 2' 2"	-0° 0' 20"	+0° 0' 22"	+0° 0' 41"	+0° 2' 1"		
15	-0° 1' 52"	-0° 1' 12"	+0° 1' 38"	-0° 1' 27"	+0° 1' 6"	+0° 0' 1"	+0° 1' 7"	-0° 0' 16"	
16	-0° 0' 16"	-0° 0' 46"	+0° 1' 7"	+0° 1' 7"	+0° 0' 21"	+0° 0' 25"	+0° 0' 25"	+0° 0' 53"	
17	-0° 2' 26"	-0° 0' 19"			+0° 0' 2"	-0° 0' 28"	+0° 0' 43"	-0° 0' 27"	
18	-0° 2' 14"	-0° 2' 4"	+0° 0' 12"	-0° 0' 46"	+0° 0' 4"				
19	0° 0' 0"	-0° 3' 54"	+0° 0' 0"	-0° 1' 34"	+0° 0' 18"	+0° 0' 15"	+0° 0' 37"		
20		-0° 6' 14"			+0° 0' 16"	-0° 0' 58"	+0° 0' 36"	+0° 0' 14"	
21									
22	-0° 2' 28"								
23	-0° 3' 48"	-0° 1' 18"	+0° 0' 0"	-0° 4' 7"	+0° 0' 5"	+0° 0' 5"	+0° 0' 23"	-0° 0' 5"	
24									
25	-0° 7' 8"	-0° 2' 21"	-0° 1' 0"	-0° 1' 21"	+0° 0' 2"	-0° 0' 16"	-0° 0' 4"	-0° 0' 24"	
26	-0° 3' 16"	-0° 4' 13"	-0° 0' 9"	-0° 0' 54"	-0° 0' 29"	+0° 0' 2"	-0° 0' 5"	-0° 0' 1"	
27									
28	-0° 9' 10"	-0° 2' 55"			+0° 0' 8"	-0° 0' 5"	-0° 0' 2"	+0° 0' 10"	
29	-0° 9' 38"	-0° 1' 52"	-0° 0' 23"	-0° 2' 2"	+0° 0' 12"	+0° 0' 34"	-0° 0' 41"	-0° 2' 1"	
30	-0° 0' 0"	-0° 0' 40"	-0° 0' 30"	-0° 1' 23"	-0° 0' 9"	-0° 0' 3"	+0° 0' 3"	+0° 0' 7"	
31	-0° 3' 44"	-0° 0' 5"	-0° 0' 17"	-0° 0' 36"	+0° 0' 12"	-0° 0' 37"	+0° 0' 16"	+0° 0' 4"	
32	+0° 3' 10"	-0° 1' 5"	+0° 0' 5"	-0° 1' 40"	-0° 0' 2"	-0° 0' 21"	+0° 0' 6"	-0° 0' 12"	

TABLE 2.4.

CHANGE OF INCLINATION OF CROSS PEGS

STATION	CHANGE OF INCLINATION								PERIOD
	21/1/58 to 31/1/58	31/1/58 to 13/2/58	13/2/58 to 27/2/58	27/2/58 to 16/4/58	16/4/58 to 29/4/58	29/4/58 to 4/6/58	4/6/58 to 30/6/58	30/6/58 to 6/8/58	
1	+0° 0' 34"	+0° 0' 16"	-0° 0' 8"	+0° 1' 28"	+0° 0' 1"	+0° 0' 22"			
2	-0° 0' 18"	+0° 0' 40"				+0° 0' 0"		+0° 0' 5"	
3	-0° 1' 38"	+0° 1' 36"	+0° 0' 2"	+0° 8' 52"	-0° 0' 6"	+0° 0' 7"		+0° 0' 0"	
4	+0° 0' 26"	+0° 0' 5"	+0° 0' 23"						
5	+0° 0' 22"	+0° 0' 10"	+0° 0' 2"	+0° 1' 7"	+0° 0' 5"				
6	-0° 0' 2"	+0° 0' 31"	+0° 0' 9"	+0° 0' 47"	-0° 0' 20"	+0° 0' 12"			
7	+0° 0' 2"	+0° 0' 20"	+0° 0' 14"	+0° 0' 37"	-0° 0' 22"	+0° 0' 19"		-0° 0' 5"	
8	+0° 1' 2"	+0° 1' 28"	-0° 0' 2"	+0° 1' 31"	-0° 0' 21"	+0° 0' 7"			
9	+0° 0' 4"	+0° 0' 18"	+0° 0' 18"	+0° 1' 45"	+0° 0' 53"	+0° 0' 4"		-0° 0' 3"	
10	+0° 0' 6"	+0° 0' 18"							
11	+0° 0' 28"	+0° 0' 12"				+0° 0' 9"	+0° 0' 4"	-0° 0' 1"	
12	+0° 0' 26"	-0° 0' 16"	+0° 0' 33"	+0° 3' 58"	+0° 0' 6"	+0° 0' 26"			
13	0° 0' 0"	+0° 0' 12"	+0° 0' 12"	+0° 1' 7"	+0° 0' 3"	+0° 0' 16"	+0° 0' 5"	+0° 0' 6"	
14	+0° 0' 4"	+0° 0' 25"	+0° 0' 4"			+0° 0' 2"	-0° 0' 5"		
15	+0° 0' 4"	-0° 0' 10"	+0° 0' 23"	+0° 0' 55"	-0° 0' 9"	+0° 0' 7"	+0° 0' 1"	+0° 0' 12"	
16	+0° 0' 4"	+0° 0' 0"	+0° 0' 25"	+0° 1' 8"	+0° 0' 6"	-0° 0' 2"	+0° 0' 20"		
17	-0° 4' 10"	+0° 0' 31"	+0° 0' 20"	+0° 8' 19"	+0° 0' 12"	+0° 0' 28"	+0° 0' 27"	+0° 0' 5"	
18				+0° 1' 31"	+0° 0' 25"				
19	-0° 1' 10"			+0° 8' 36"	+0° 0' 17"	+0° 0' 18"	+0° 0' 10"		
20	+0° 1' 8"	+0° 0' 1"	+0° 0' 21"	+0° 2' 16"	+0° 0' 13"	+0° 0' 35"	+0° 0' 14"	+0° 0' 26"	
21	-0° 0' 10"	+0° 0' 31"	+0° 0' 3"	+0° 1' 56"	+0° 0' 16"	+0° 0' 37"	-0° 0' 3"	+0° 0' 20"	
22	-0° 1' 16"								
23	-0° 0' 16"	+0° 0' 6"	+0° 0' 14"	+0° 2' 29"	+0° 0' 15"	+0° 0' 20"	+0° 0' 18"	+0° 0' 20"	
24	-0° 3' 30"	+0° 0' 35"	-0° 0' 3"	+0° 1' 46"	+0° 0' 14"				
25	+0° 1' 16"	-0° 0' 27"	+0° 0' 33"	+0° 1' 52"	+0° 0' 13"	+0° 0' 1"	+0° 0' 31"	+0° 0' 22"	
26	-0° 2' 10"	-0° 0' 28"	+0° 0' 7"	+0° 2' 47"	+0° 0' 8"	+0° 0' 24"	+0° 0' 30"	+0° 0' 19"	
27	+0° 1' 42"	+0° 0' 16"	+0° 0' 0"	+0° 4' 29"	-0° 0' 18"	+0° 0' 15"	+0° 0' 2"	+0° 0' 14"	
28	+0° 0' 4"	-0° 0' 54"	+0° 0' 25"	+0° 5' 57"	-0° 0' 27"	+0° 1' 2"	+0° 0' 7"	+0° 0' 17"	
29	+0° 0' 50"	-0° 0' 42"	+0° 0' 14"	+0° 9' 3"	-0° 0' 5"	+0° 0' 16"			
30	+0° 0' 2"	+0° 0' 22"	+0° 0' 17"	+0° 3' 35"	+0° 0' 12"	+0° 0' 11"	+0° 0' 17"	+0° 0' 15"	
31	+0° 0' 26"	+0° 0' 6"	+0° 0' 14"	+0° 4' 14"	-0° 0' 3"	+0° 0' 45"	+0° 0' 11"	+0° 0' 19"	
32	+0° 0' 18"	+0° 0' 6"	+0° 0' 6"	-0° 0' 29"	+0° 0' 4"	+0° 0' 14"	+0° 0' 3"	+0° 0' 19"	

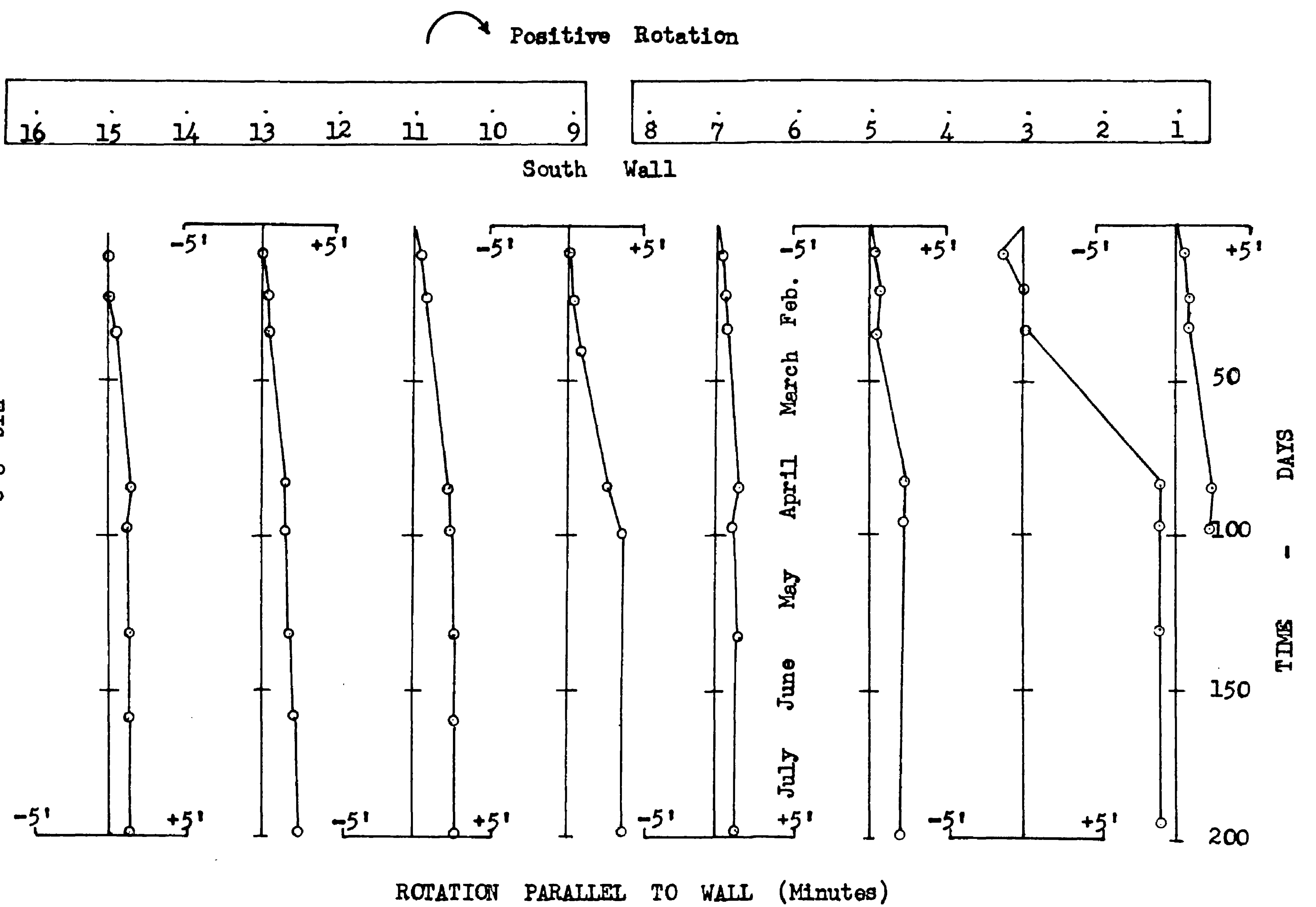

Positive Rotation
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
South Wall
-5'
+5'
Feb.
March
April
May
June
July
50
100
150
200
TIME - DAYS
ROTATION PARALLEL TO WALL (Minutes)

FIG. 2.3.

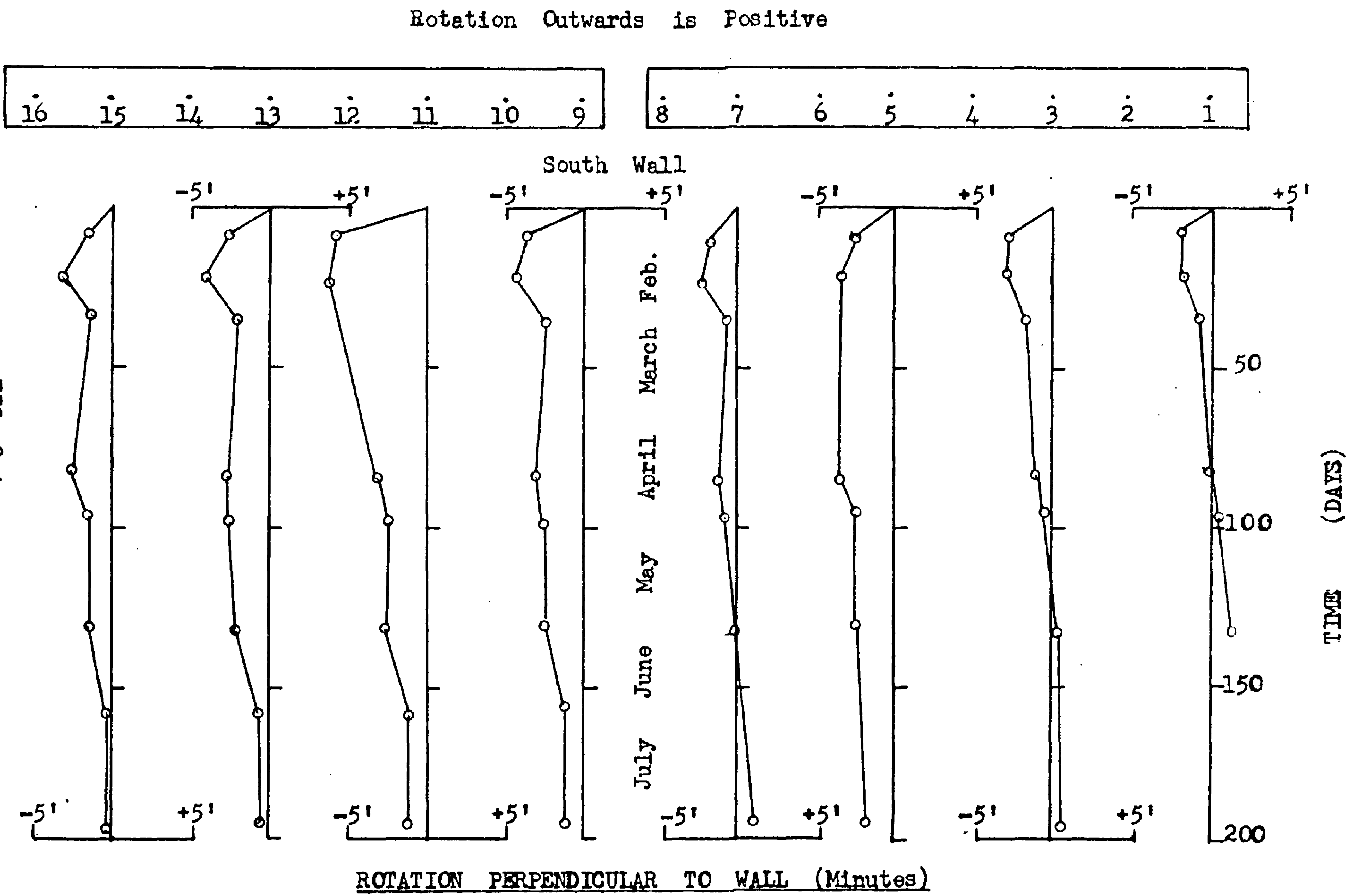
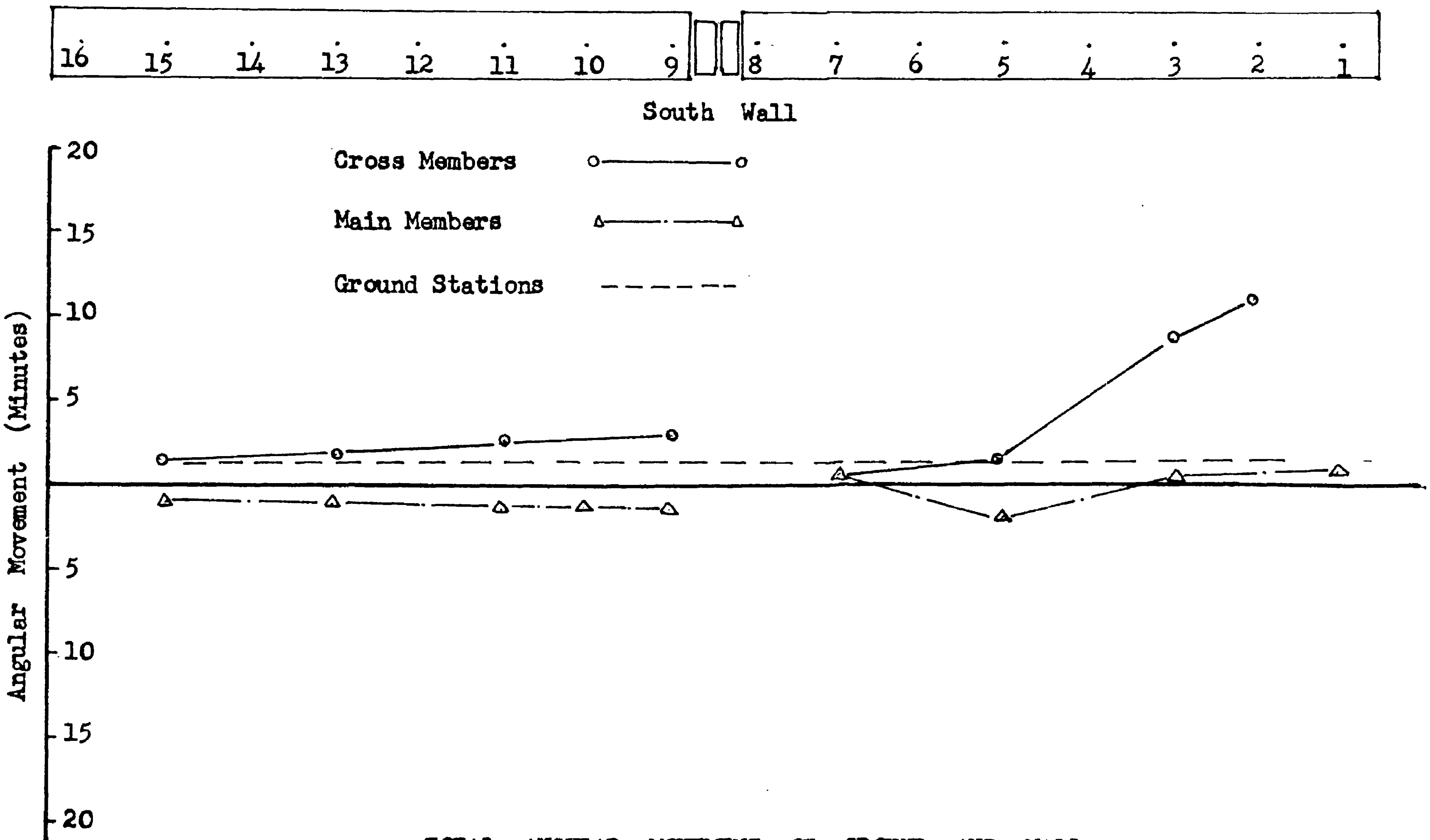

Rotation Outwards is Positive
16 15 14 13 12 11 10 9
8 7 6 5 4 3 2 1
South Wall
-5'
+5'
Feb.
March
April
May
June
July
50
100
150
200
TIME (DAYS)
ROTATION PERPENDICULAR TO WALL (Minutes)

FIG. 2.4.



TOTAL ANGULAR MOVEMENT OF GROUND AND WALL

FIG. 2.5.

the latter stations.

Figs. 2.6 and 2.7 show the movement of the cross members and main members of the stations on the sloping east wall. In both figures the curves fall into groups according to their shape. It will be noted that there is a variation in the magnitude of movement within each group. This is more clearly seen in Fig. 2.8 which shows the total angular movement which occurred at each station on the east wall. Movement over the centre section of the wall was fairly uniform and only slightly in excess of ground movement. At the ends, movement was more erratic and was much in excess of ground movement. It is suggested that this was caused by a lean-to garage backing on the wall between stations 29 and 30, and by dead trees standing against the wall between stations 17 and 19. Another factor contributing to the effect was the presence of cracks which allowed adjacent parts of the wall to move independently of each other.

Fig. 2.9 was constructed to compare the rate of development of tilting movements in the ground and the wall. Average values of tilt for the ground and wall were used. It can be seen that, except during the period from February till April tilting of the ground and wall took place at the same rate. In the period February to April a large increase in the tilt of the wall was recorded. This increase can also be seen in Figs. 2.3, 2.4, 2.6 and 2.7 where it will be noted that the magnitude of the change varies from station to station. Movements during the period in a given direction, e.g. parallel to the wall, are in the same direction for all stations. Now if weather

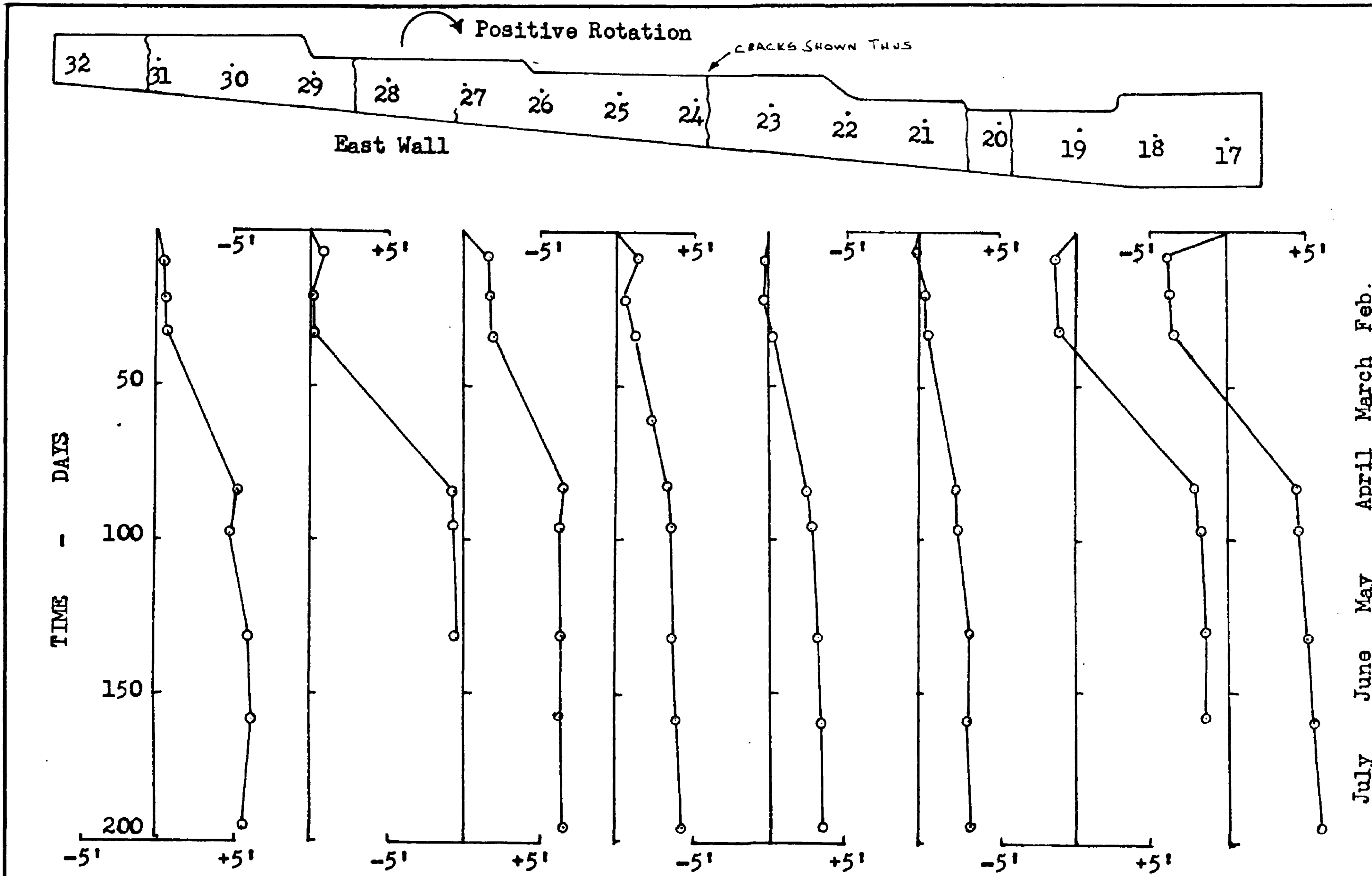


FIG. 2.6. ROTATION PARALLEL TO WALL (Minutes)

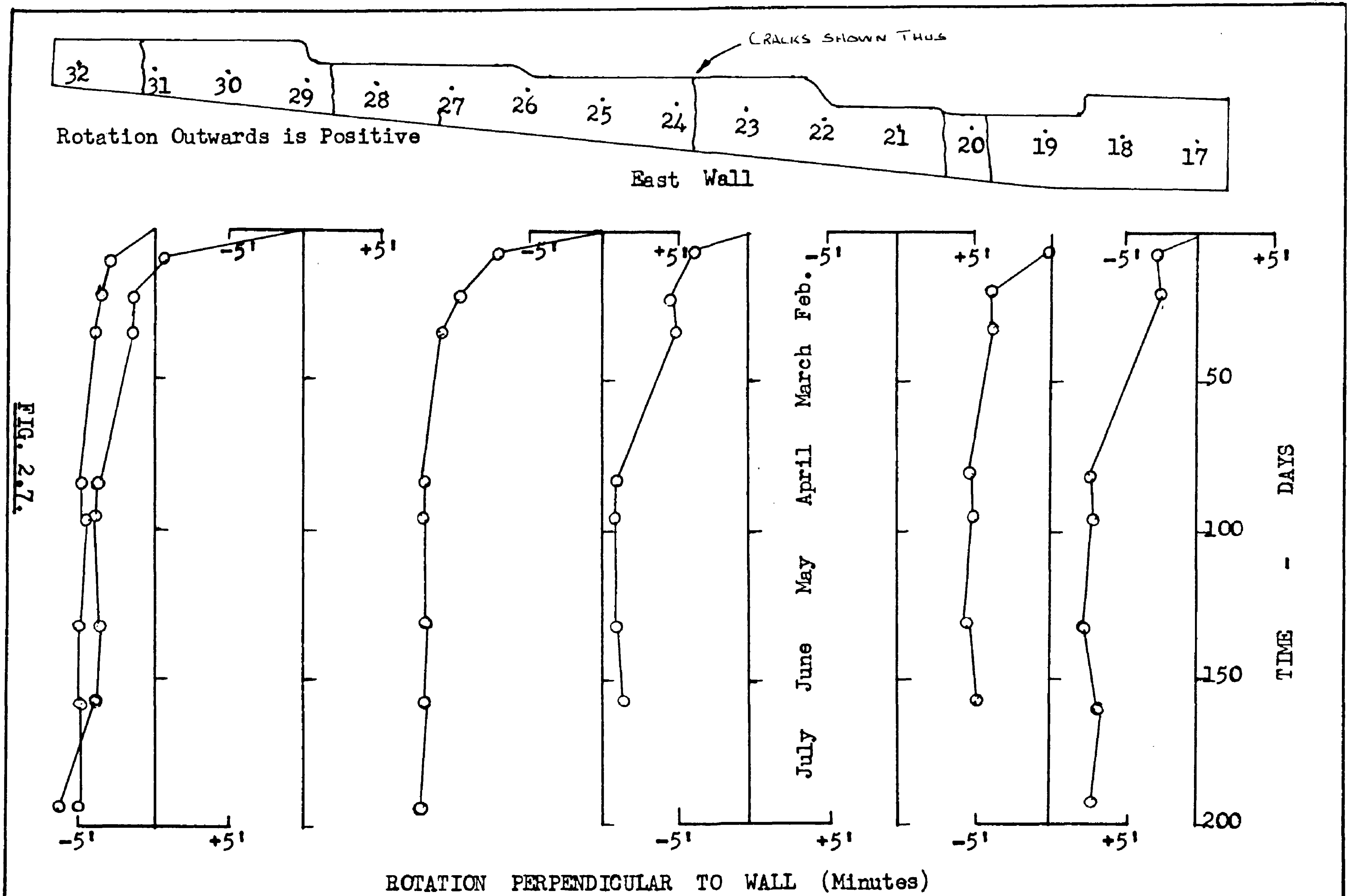
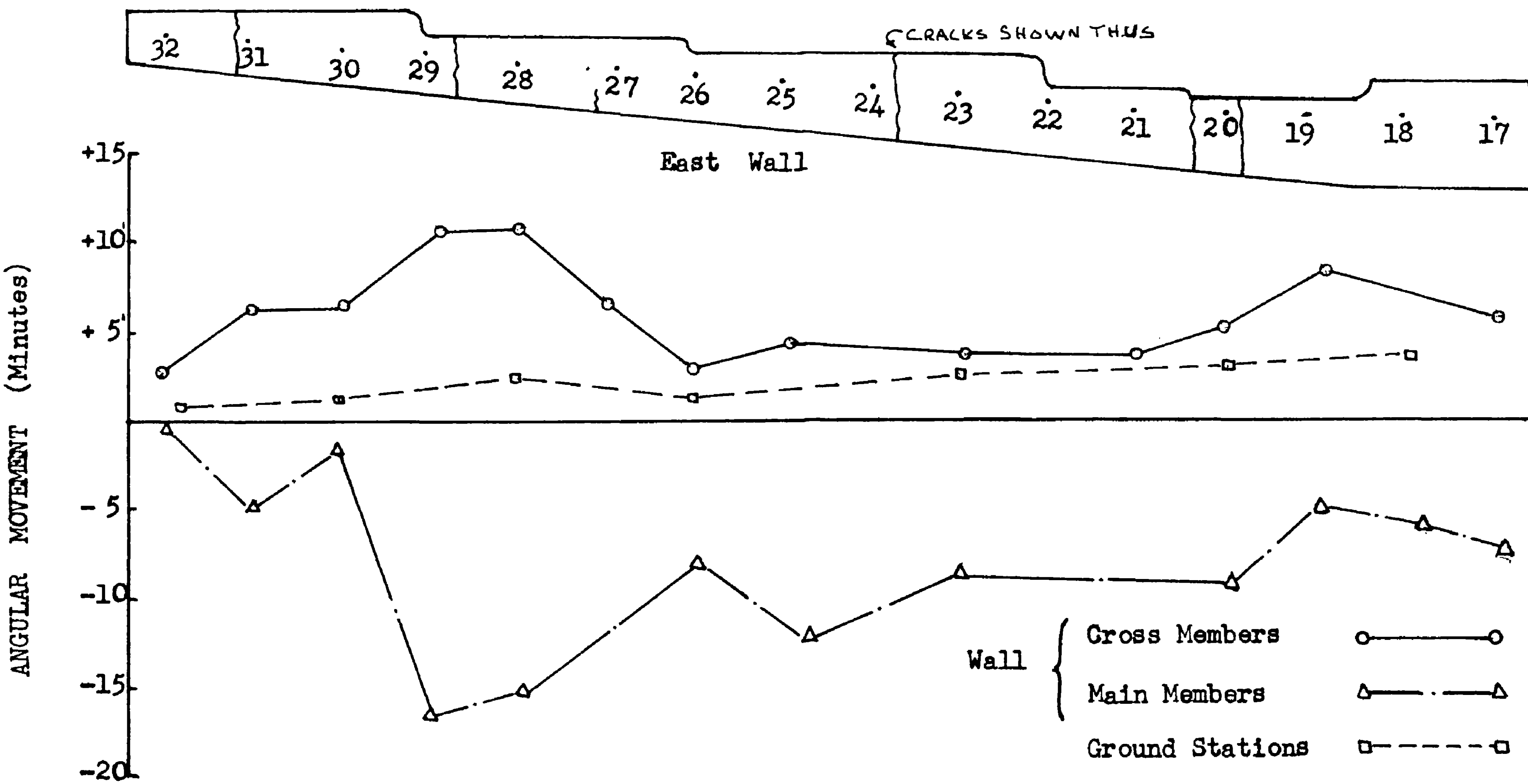
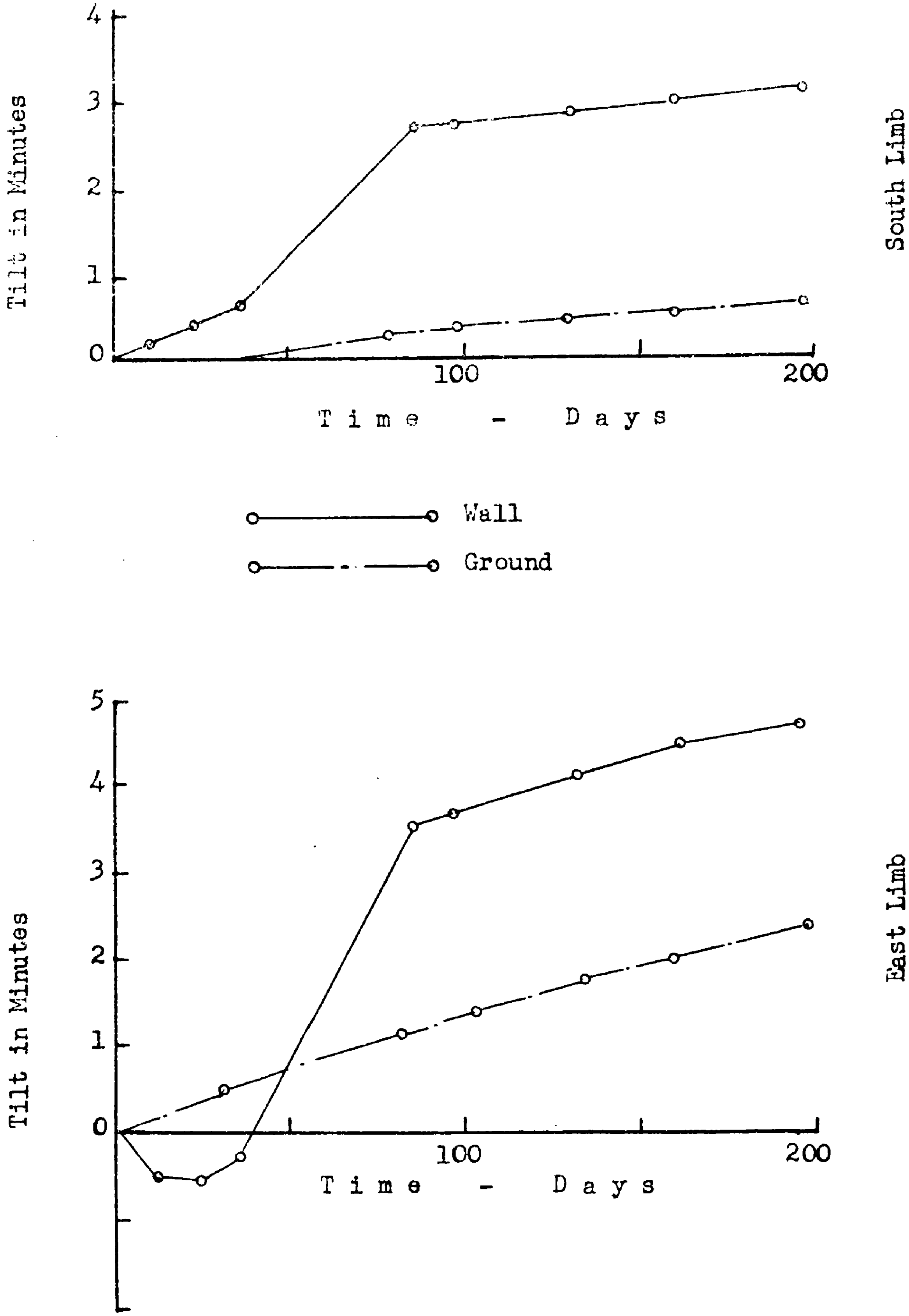

CRACKS SHOWN THUS
32
31
30
29
28
27
26
25
24
23
22
21
20
19
18
17
Rotation Outwards is Positive
East Wall
-5'
+5'
Feb.
March
April
May
June
July
50
100
150
200
TIME - DAYS
ROTATION PERPENDICULAR TO WALL (Minutes)

FIG. 2.7.



TOTAL ANGULAR MOVEMENT OF GROUND AND WALL

FIG. 2.8.



AVERAGE TILT OF WALL AND GROUND

FIG. 2.9

had been the main cause of the rapid change in tilt it is probable that changes in either direction would have occurred. The rapid change cannot, therefore, be attributed solely to the effect of weather.

CONCLUSIONS

Differential vertical displacement of the ground and wall, if such occurred, was not detected by the measuring procedure used in the test.

Tilting movements were not uniformly distributed along the length of the wall. The variations in movement were attributable in most cases to the presence of cracks or other discontinuities in the wall.

Except during the period from February to April the average tilt in the ground and wall, in a given time, were comparable in magnitude. The reasons for the large wall movements during the above period are not clear.

Section 3.1 DESCRIPTION OF SITE

3.1.1. Workings.

The seam being worked in the vicinity of the church at the time of the investigation was the Stoney Seam. Details of the seam and the workings under the church are :-

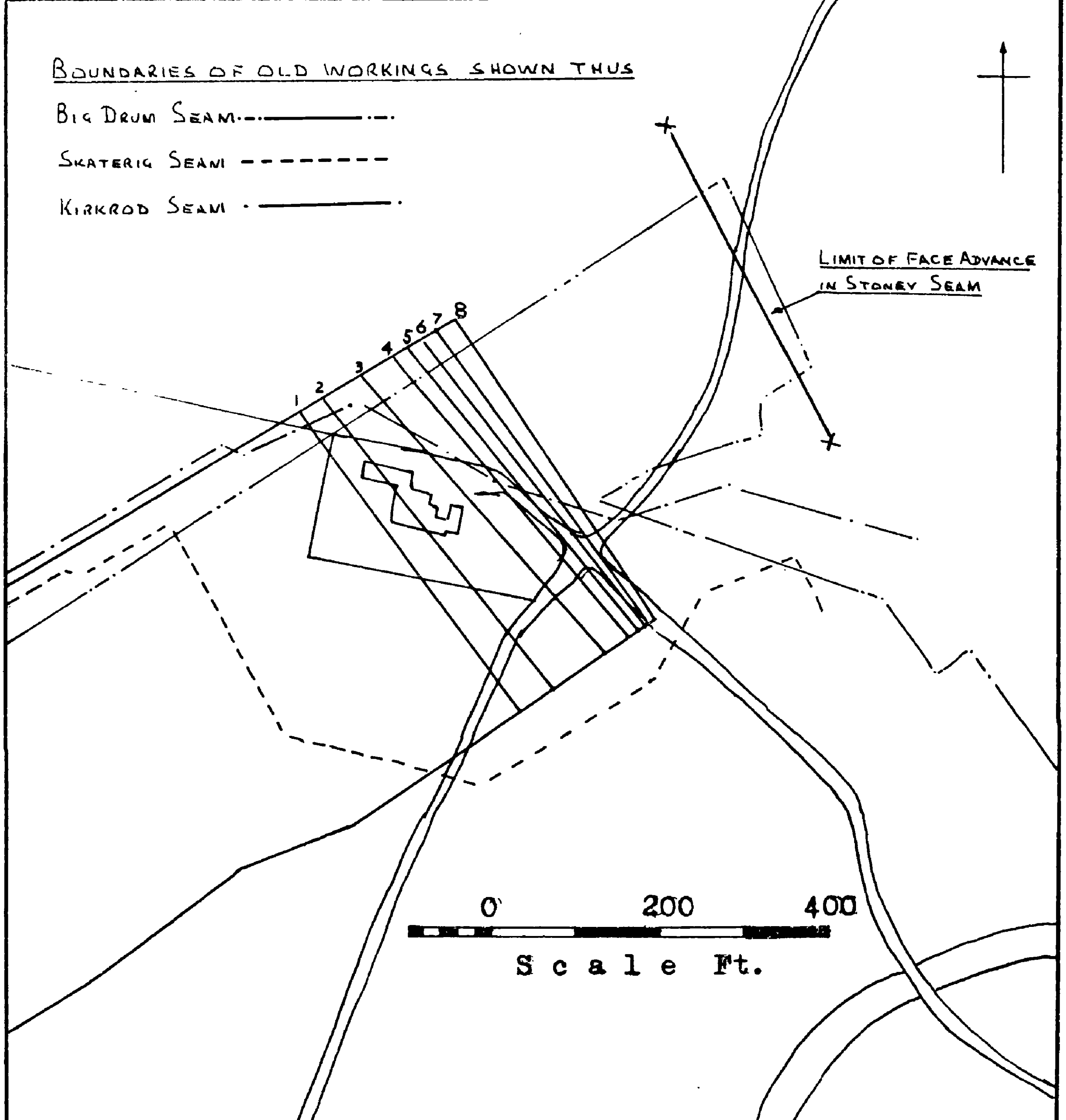
Thickness of seam	26 in.
Depth of cover at church	550 ft.
Width of face	450 ft.
Pack Details	
Main Gate rise side	20 ft.
Main Gate dip side	15 ft.
Tail Gate rise side	12 ft.
Tail Gate dip side	30 ft.
Strip packs on face	8 ft. at 22 ft. centres.

Figure 3.1 shows

(a) the working in the Stoney Seam

(b) the boundaries of old workings in the three seams lying within 100 feet above the Stoney Seam,

(c) the limit of advance in the Stoney Seam which was rising towards unconsolidated deposits.



Dates on which measurements were made.

1) 24-1-59	4) 4-4-59	7) 13-6-59
2) 7-2-59	5) 18-4-59	8) 27-6-59
3) 7-3-59	6) 16-5-59	

DOUGLAS WATER SITE

FIG 3.1

3.1.2. Buildings.

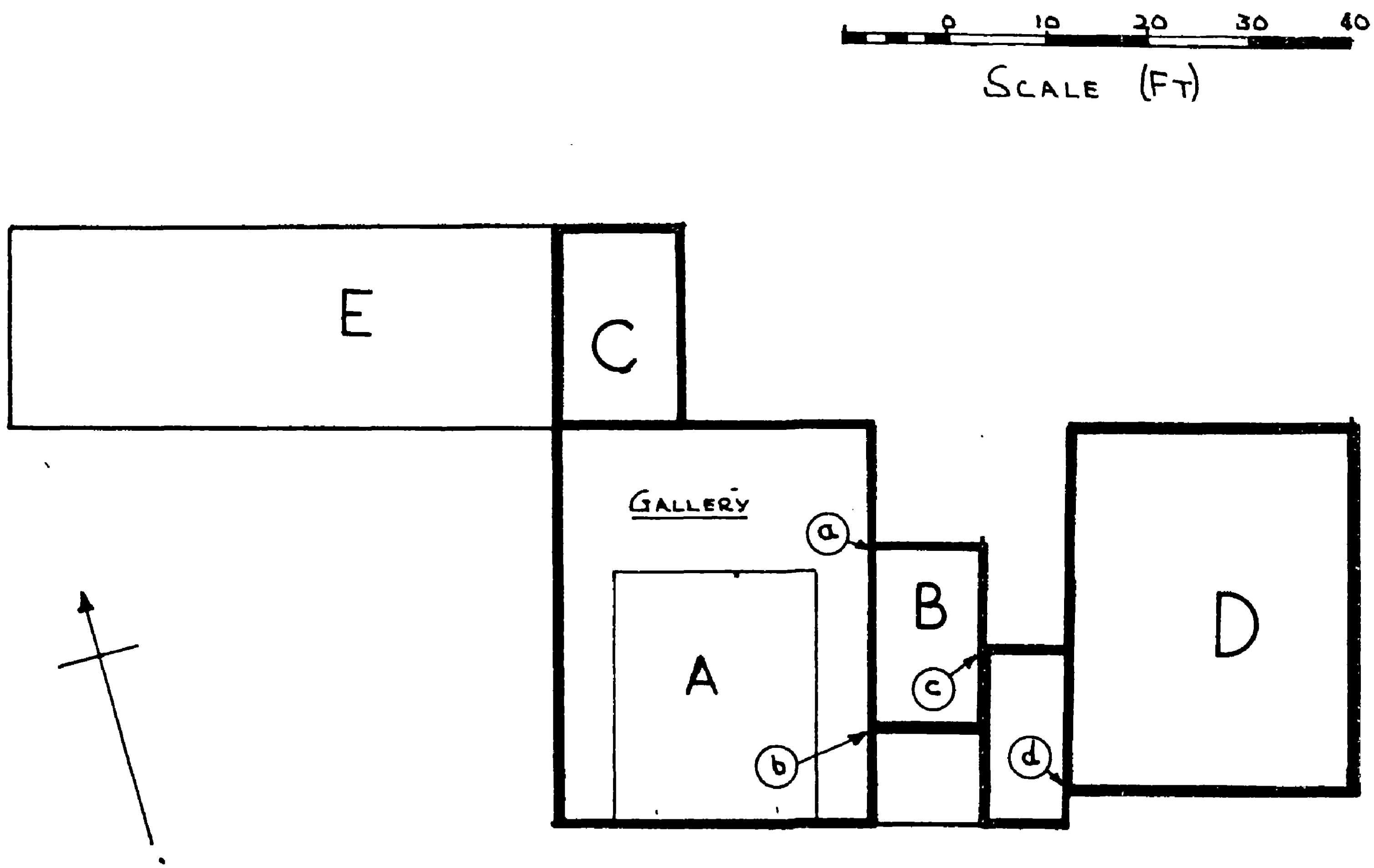
Details of the buildings are shown in Fig. 3.2. The sections A, B, C and D were built over 100 years ago and comprise the church, vestry, kitchen and manse respectively. The walls of these buildings are about 2 feet thick and consist of sandstone blocks, set in level courses, backed by random rubble. Lime mortar was used as the bonding agent. The walls are built on sandstone footings set at a depth of 1 foot on a hard packed sandy soil.

Church. The gallery shown in Fig. 3.2 is built into the wall and supported at its inside edge by wooden beams set on wooden pillars. The interior finish, on both ground floor and gallery, is wood panelling with plaster above. The ceiling is of plaster and is without ornamentation; it is flat over the centre portion and slopes downwards at the pitch of the roof to the junction with the walls.

The vestry, B, and the kitchen, C, are single storey buildings; the manse, D, is two storey.

The hall, E, is of more recent construction. It is a single storey brick structure built on concrete footings laid at a depth of 1 foot on a hard packed sandy soil.

All the buildings had been damaged by previous workings in the area. Repairs, mostly in the form of repointing and plastering of open cracks, had subsequently been carried out.



PLAN OF BUILDINGS FIG 3.2

3.1.3. General.

When work commenced at the site some movement of the buildings had taken place. Also, the distance the face had still to advance was limited to about 500 feet by the presence of unconsolidated deposits. As this distance was less than the diameter of the area of influence of a surface point (700 feet in this case with a 35° angle of draw) it was impossible to lay out a line of measuring stations which would undergo a complete cycle of subsidence movement.

Section 3.2 OBJECT OF INVESTIGATION

The objects of the investigation were

(a) To determine the magnitude and distribution of ground movements in the vicinity of the buildings.

(b) To determine the magnitude and distribution of structural movement and to record the nature and extent of damage.

(c) To correlate where possible the information obtained in (a) and (b).

Section 3.3 DESCRIPTION OF MEASURING STATIONS

3.3.1. Ground Stations.

The station markers consist of 3 ft. lengths of malleable iron pipe about 1 inch in diameter which were hammered to a point at one end and fitted with an anchor plate at the other. The pipe was screwed internally at the anchor plate end to take a brass socket. (See Plate 3.1). The main member 'A' of the station marker is driven into the ground before the socket 'B' is screwed into position. A special drive head with wood inserts was used to prevent damage to the internal thread. The station markers were inserted so that they did not protrude above ground level, to minimise the risk of damage to them by animals or farm implements. To enable linear measurements to be made, two extension pieces 'C' were provided. The extension pieces were a sliding fit in sockets 'B'. The extension pieces were numbered to aid identification. Both the extension pieces and the sockets had locating marks scribed on them.

The distance between adjacent stations was measured by means of an invar tape.

3.3.2. Wall Stations.

Tape Stations. Cylindrical brass plugs 1 inch long and $\frac{3}{4}$ inch diameter were fixed to the wall by rawlbolts to form station markers. A fine line was scribed on each plug, after fitting, to form a reference mark for linear measurements. The plugs were set in pairs 24 feet apart and the distance between them measured by means of an invar tape.

Micrometer Stations. The plugs used were similar to those used for tape stations but, in addition, were fitted with a pin to provide

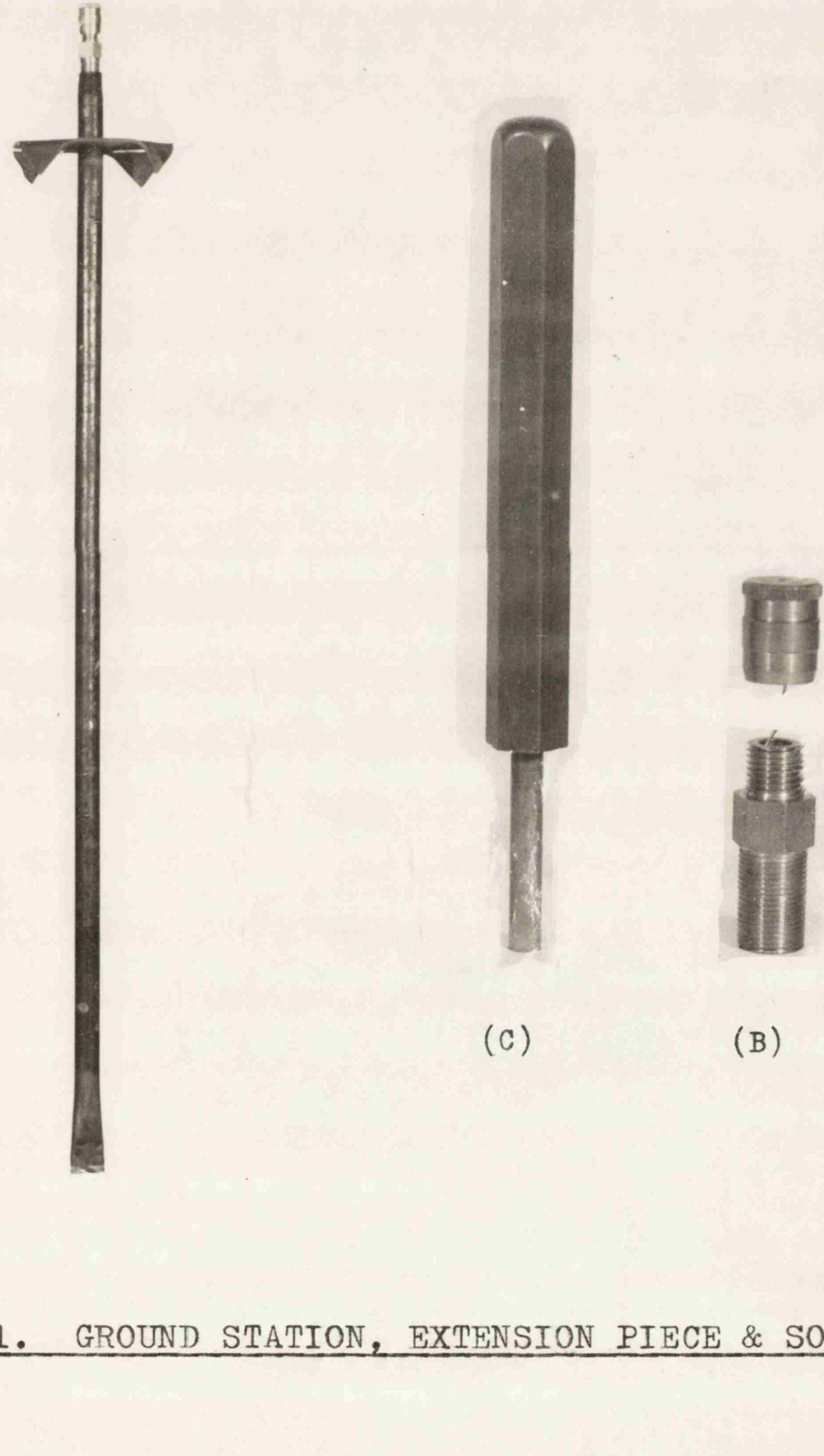


PLATE 3.1. GROUND STATION, EXTENSION PIECE & SOCKET.

positive location for the micrometer (See Plate 3.3.C). The plugs were inserted in pairs 15 inches apart. The distance between adjacent plugs was measured by means of a micrometer.

Section 3.4 LAYOUT OF MEASURING STATIONS

3.4.1. General.

The station layout was determined by the factors discussed in section 3.1.3. Stations inserted adjacent to the church would undergo only that part of the movement cycle which had not taken place before they were inserted. To enable some assessment to be made of the magnitude of the movement which had occurred near the church prior to the insertion of the stations, a second series of stations was required. This second series of stations was laid out in the same position relative to the face as the first series, but in a position in advance of the face where no movements had occurred.

In order to obtain the maximum possible information about the magnitude and direction of the horizontal ground movements the stations in the field were installed in a triangular pattern as recommended by King and Whetton [3.1.]

Measurements of horizontal movement only were made. Vertical movements were not measured because the nature of the terrain, and the lack of a suitable stable bench mark, made levelling difficult and time consuming.

3.4.2. Stations Adjacent to the Buildings.

The stations were set out to form two bays A-B , and C-D (Fig. 3.3). By measuring the change in length of these bays from their length at the start of the test, and applying a correction for the movement which had occurred prior to the start, (obtained from the results of the measurements on the field line) an estimate of the maximum movements occurring near the wall was to be made.

3.4.3. Stations on Buildings.

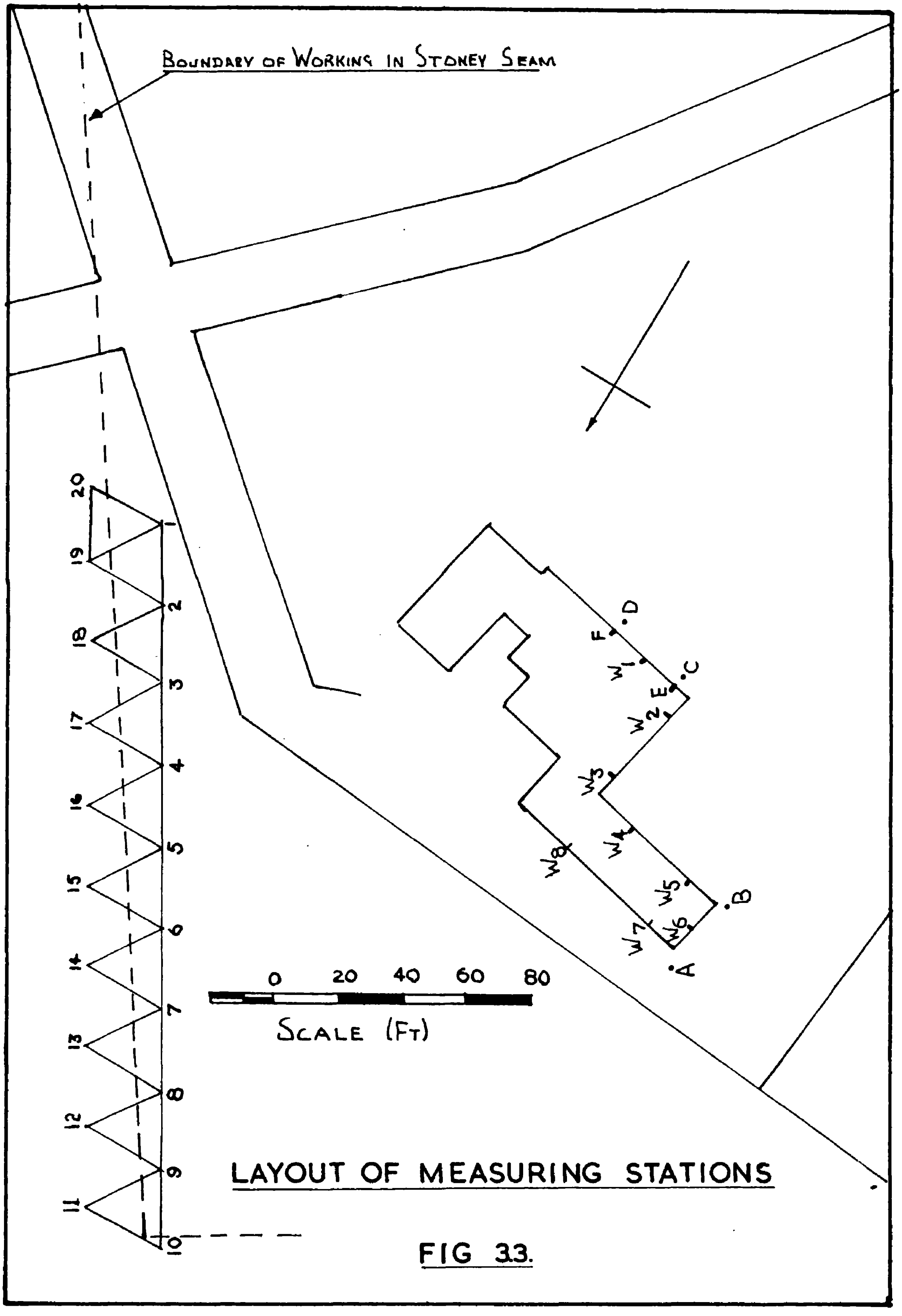
E and F were a pair of tape stations on the south wall of the church.

W_1 to W_8 were micrometer stations.

3.4.4. Stations in the Field.

The stations were set out in a staggered line to form a number of equilateral triangles with side of length 24 feet. The line was set out at right angles to the gate roads in the Stoney Seam and extended from the ribside on the rise side to the centre of the face.

The change in length of the sides of the triangles was measured and used to compute the magnitude and direction of the maximum change in length. The Mohr circle method used for this calculation is described in the Ref. [3.1].



LAYOUT OF MEASURING STATIONS

FIG 3.3.

Section 3.5 MEASURING APPARATUS AND TECHNIQUES

3.5.1. Ground Stations.

The distance between adjacent ground stations was measured by means of a 25 ft. long invar steel tape graduated in feet, 1/10th ft, 1/100th ft. The tape was suspended in catenary under a standard tension of 20 lb. applied by means of straining frames of the type shown in Plate 3.2.

The procedure for measuring the distance between two stations was as follows :-

An extension piece was fitted into the socket of each station and pushed firmly home. (A record of the number of the extension piece was made in the field book beside the station with which it was used so that in subsequent measurements the same extension piece could be used in the same station. This was done to minimise error arising from the extension pieces being of slightly different dimensions). The index mark on each extension piece was aligned with that on the socket in which it was used. (This procedure was to prevent error resulting from eccentricity of the measuring mark on the extension piece).

The straining frames were fixed in position by means of the anchor pins; the tape, swivels and spring balance were fitted as shown in Plate 3.2. The tape was aligned over the measuring mark on both extension pieces by using the vertical and lateral adjustment provided on the straining frames. The tape was then tensioned by means of the adjusting screws till the spring balance registered 20 lb. Vertical and lateral adjustment was carried out if necessary.

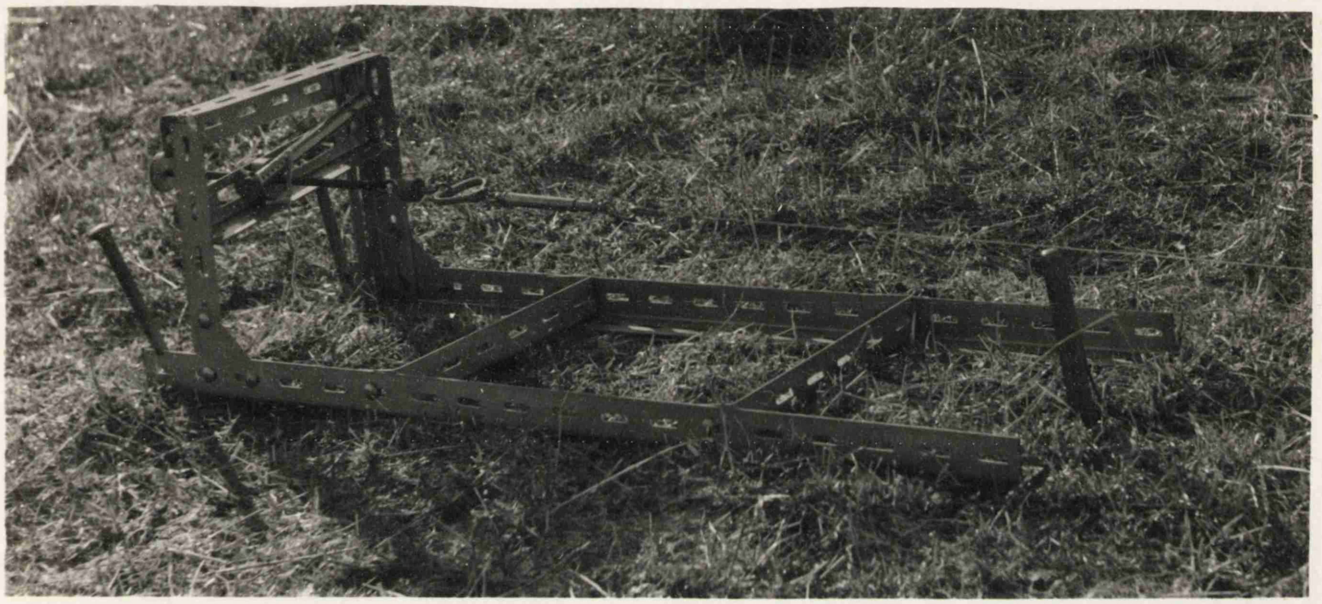


PLATE 3.2. TAPE STRAINING FRAME

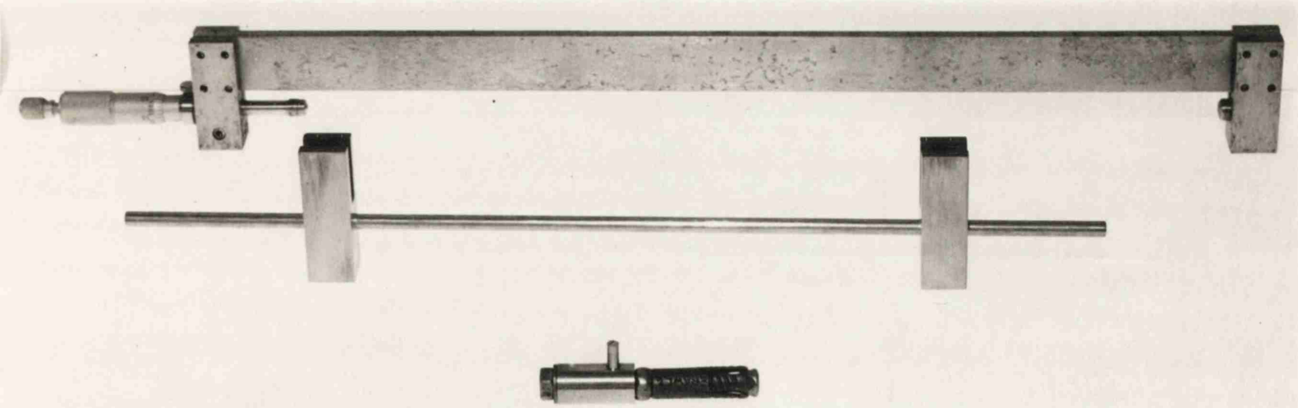


PLATE 3.3. MICROMETER, SETTING ROD & WALL PLUG

Both ends of the tape were read, estimating to one tenth of a division i.e. to 1/1000 ft.

The tape was then moved by slackening one adjusting screw and tightening the other.

Both ends of the tape were read again.

This procedure was repeated till six readings of the length had been obtained. The mean of these six readings was then accepted as the measure of the distance between the two stations. The air temperature was read on an encased thermometer suspended from the top member of one of the straining frames.

The whole procedure was then repeated for the other stations.

This measuring procedure was evolved after other methods of measuring the distance between stations had been tried and found unsatisfactory. (See Section 2.3.3). To determine the accuracy of the measuring technique the following test was carried out.

The distance between two adjacent stations in the ground was measured as described in 3.5.1. Twenty five measurements were made. After every fifth measurement the extension pieces were removed and replaced. From the results obtained the 95% confidence limits of the mean of six readings of the length were found to be $\pm$ 1/1000 ft. For the length of bay used, i.e. 24 feet, this is equivalent to limits of $\pm$ 0.04 m.m./m.

The tape used in the field was checked periodically against another invar tape which was kept as a standard. The standardisation was carried out using the Macca Base Line Equipment described in Section 2.3.3.

3.5.2. Wall Stations - Tape.

The distance between stations 'E' and 'F' on the wall of the church was measured by means of the invar steel tape suspended in catenary between two small brackets fitted to the wall outwith the gauge length defined by 'E' and 'F'. Adjusting screws, swivels and a spring balance were used as before. The tape was adjusted for height by moving the adjusting screws in vertical slots provided on the brackets. Six readings of the distance between the stations were made and the mean accepted. The temperature was read and noted as before.

3.5.3. Wall Stations - Micrometer.

At stations 'W_1' to 'W_8' the distance between the brass plugs was measured by means of a specially constructed micrometer (Plate 3.3.A). The micrometer is fitted with a vernier to read to 1/10,000 in. and has a range from $14\frac{1}{2}$ to $15\frac{1}{2}$ inches. The measuring faces of the micrometer are spherical to ensure point contact. The device was used in conjunction with an invar setting rod. The setting rod acts as a standard with which all measurements are compared. This eliminates error caused by variation in the length of the micrometer whether caused by rough handling or thermal expansion.

The measuring procedure was as follows :-

The length of the invar rod was measured. To facilitate this, brass blocks (Plate 3.3.B) were used to hold the micrometer and rod in the correct relative position. The micrometer was placed on the measuring plugs and pushed against the locating pins; the distance between the plugs was measured, reading to the nearest vernier division (1/10,000 in.). The micrometer was removed and replaced; the distance was re-measured. Three such measurements were made and the mean noted. The temperature was read from a thermometer placed in contact with the wall; this was to enable a correction for the thermal expansion of the wall to be applied.

To check the accuracy of the micrometer measuring technique a series of 50 consecutive measurements was made on a wall station. The micrometer was removed after each reading and replaced for the next. A check measurement was made on the invar setting rod after every fifth measurement on the wall station. For a significance level of 95% the confidence limits for the mean of three measurements were found to be $\pm$ 4/10,000 in. As the temperature read on the thermometer may differ from the true temperature of the wall by an unknown amount it was felt that the confidence limits used to test the significance of recorded movements should be widened. A 5 degree difference from the true temperature of the wall would cause an error of 5/10,000 in. As such a temperature error is possible the confidence limits were set at $\pm$ 10/10,000 in.

Section 3.6 RESULTS

3.6.1. General.

The positions of the face on the dates when measurements were made are shown in Fig. 3.1. It should be noted that the rate of face advance was much reduced from April onwards, the face being finally stopped on June 27th 1959, under the line of field stations. This stoppage prevented the measuring programme being carried through as planned, as the face did not advance as far as had been anticipated. Measurements, however, were continued for six months after the face had stopped. The results obtained are shown in Tables 3.1, 3.2 and 3.3.

3.6.2. Field Stations.

Using the data shown in Table 3.1 the magnitude and direction of the principal movements were calculated by the method given by King [3.1], and are shown in Fig. 3.4.

It will be seen that the maximum tensile and compressive movements occurred on bays 7-8 and 2-3 respectively. Graphs were constructed to show the development of these movements with time (Fig. 3.5). Measurable movement is seen to have ceased approximately 70 days after the face stopped under the line of measuring stations. It should, however, be noted that the rate of face advance was much reduced in the two months prior to the stoppage.

TABLE 3.1

DATE / BAY	11-3-59	16-4-59	13-5-59	12-6-59	15-7-59	3-8-59	1-9-59	5-10-59	7-1-60
1-2	-0·27	-0·52	-0·79	-0·92	-1·05	-0·93	-1·04	-0·98	-0·72
2-3	-0·43	-0·67	-0·99	-1·40	-1·83	-2·05	-2·25	-2·25	-2·28
3-4	+0·11	+0·03	-0·01	-0·04	-0·34	-0·64	-0·57	-0·49	-0·51
4-5	+0·21	0	-0·36	-0·98	-1·19	-1·23	-1·32	-1·46	-1·39
5-6	-0·20	-0·24	-0·40	-0·44	-0·49	-0·45	-0·57	-0·48	-0·45
6-7	+0·20	+0·25	+0·35	+0·41	+0·30	+0·30	+0·38	+0·39	+0·38
7-8	-0·03	+0·17	+0·60	+1·13	+1·25	+1·34	+1·38	+1·41	+1·34
8-9	-0·15	-0·11	+0·09	+0·27	+0·28	+0·28	+0·28	+0·24	+0·31
9-10	-0·04	+0·07	+0·27	+0·46	+0·50	+0·58	+0·60	+0·70	+0·71
10-11	-0·08	-0·18	-0·24	-0·26	-0·27	-0·23	-0·27	-0·25	-0·27
11-9	+0·10	+0·10	+0·01	+0·07	+0·10	+0·13	+0·15	+0·20	+0·25
9-12	0	-0·07	-0·13	-0·31	-0·45	-0·45	-0·40	-0·36	-0·44
12-8	-0·17	-0·22	-0·16	-0·03	+0·12	+0·13	+0·08	+0·06	+0·17
8-13	+0·09	-0·03	+0·03	+0·25	+0·22	+0·21	+0·25	+0·28	+0·25
13-7	-0·16	-0·12	+0·13	+0·47	+0·58	+0·60	+0·60	+0·58	+0·61
7-14	-0·12	-0·20	-0·34	-0·68	-0·78	-0·74	-0·72	-0·69	-0·70
14-6	+0·03	+0·07	+0·24	+0·57	+0·58	+0·59	+0·66	+0·65	+0·61
6-15	+0·03	-0·22	-0·43	-0·82	-0·99	-1·06	-1·17	-1·21	-1·16
15-5	-0·02	+0·05	+0·10	+0·17	+0·18	+0·06	-0·05	-0·05	0
5-16	-0·02	-0·22	-0·56	-1·01	-1·28	-1·38	-1·46	-1·56	-1·58
16-4	0	-0·09	-0·09	-0·05	-0·06	-0·02	-0·10	-0·17	-0·06
4-17	-0·04	-0·20	-0·19	-0·12	-0·06	-0·04	-0·04	-0·03	0
17-3	+0·10	+0·08	+0·15	+0·25	+0·15	0	-0·17	-0·13	-0·19
3-18	+0·05	-0·08	-0·25	-0·50	-0·73	-0·84	-1·02	-1·02	-1·11
18-2	-0·01	-0·08	-0·26	-0·64	-0·73	-0·70	-0·97	-1·00	-0·82
2-19	-0·02	-0·17	-0·40	-0·21	-0·21	-0·19	-0·25	-0·21	-0·15
19-1	+0·01	0	-0·19	-0·50	-0·61	-0·70	-0·77	-0·77	-0·75
1-20	+0·01	-0·01	-0·10	-0·13	-0·10	-0·25	-0·19	-0·34	-0·43
20-19	-0·10	-0·27	-0·46	-0·83	-0·67	-0·89	-0·97	-0·98	-1·00

STRAIN (MM/M) ON FIELD LINE

T A B L E 3.2.

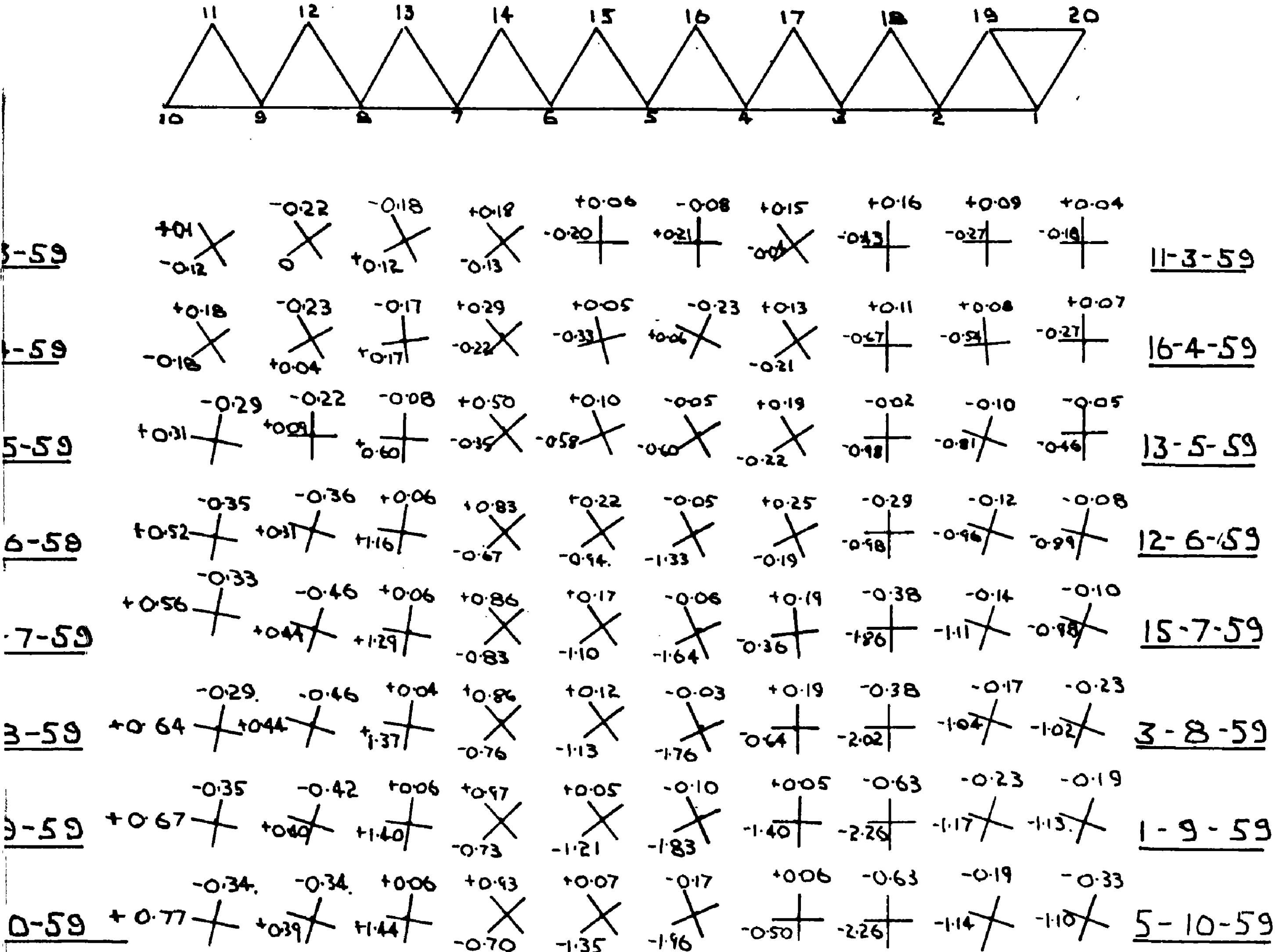
Date / Bay	11.3.59	3.4.59	13.5.59	15.6.59	30.6.59	3.8.59	1.9.59	5.10.59	7.1.60
A - B	+0.14	+0.44	-0.48	-1.29	-1.35	-1.45	-1.47	-1.53	-1.57
C - D	+0.31	-0.99	-2.38	-2.84		-3.00		-3.03	-3.15
E - F	+0.05	+0.04	+0.05	+0.12	+0.07	+0.15	+0.13	+0.06	-0.05

STRAIN (MM/M) ON AND ADJACENT TO CHURCH

T A B L E 3.3.

Date / Station	28.2.59	12.3.59	3.4.59	12.5.59	15.6.59	3.8.59	1.9.59	5.10.59	7.1.60
W_1	-1	-3	0	-8	-6	-11	-16	-6	+9
W_3	+55	+198	-20	-1240	-1560	-1610	-1610	-1607	-1590
W_4	+4	-11		-16	-9	-18		-13	-8
W_5	+7	-6	-3	-111	-247	-253		-258	-247
W_6	+2	-5	-3	-10	-18	-23	-28	-25	-15
W_7	+5	-1	-2	+4	+25	-5		+5	+20
W_8	+1	0	-7	-17	-16	-25		-20	-15

CHANGE IN LENGTH (1/10,000 FT) AT MICROMETER STATIONS



MAGNITUDE & DIRECTION OF PRINCIPAL STRAINS (MM/M.) AT FIELD STATIONS.

FIG. 3.4.

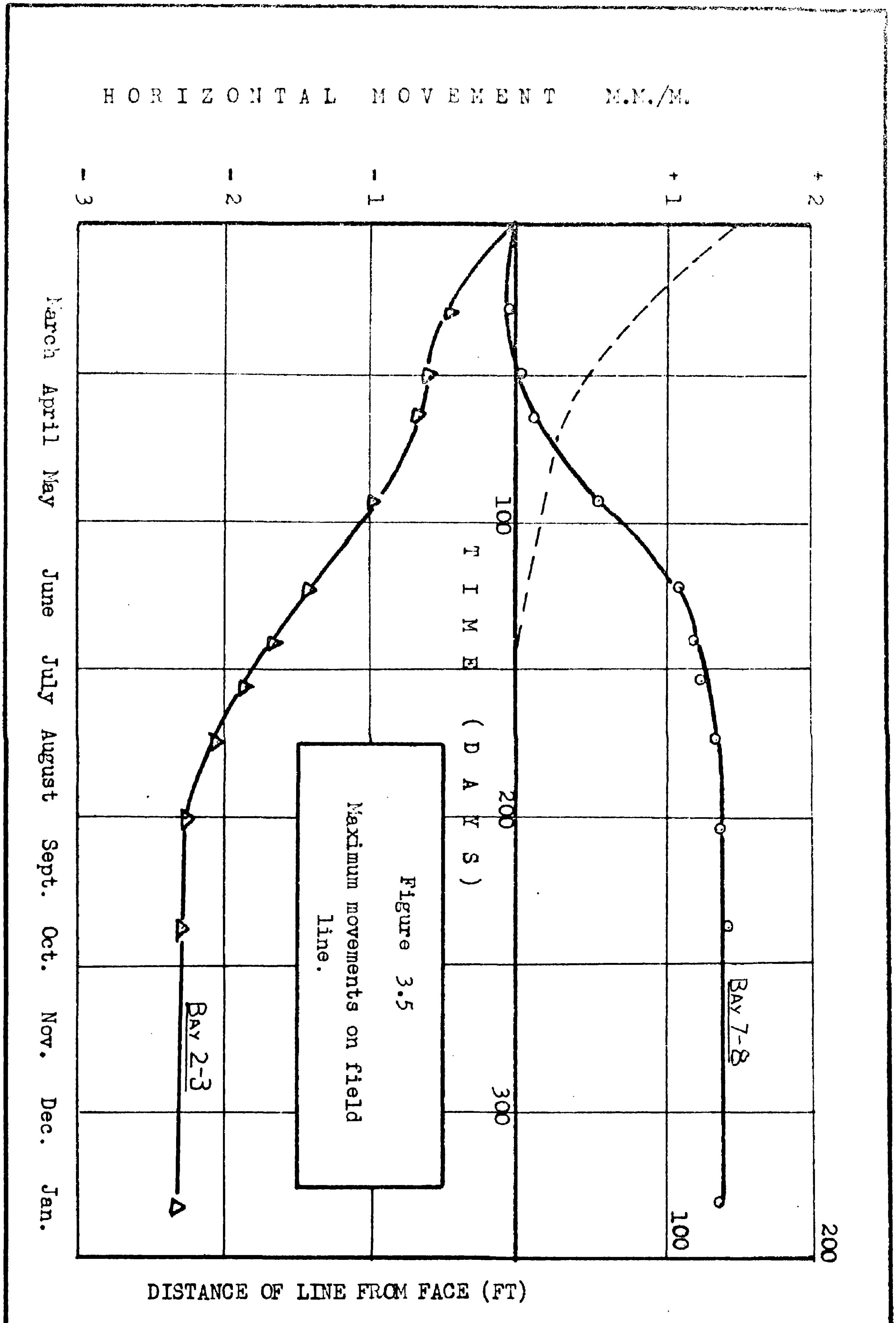


Figure 3.5
Maximum movements on field line.

3.6.3. Stations on and, adjacent to, Church.

Fig. 3.6 was plotted from the data shown in Table 3.2, to show the relative movement of the church and the ground adjacent to it.

Bay C-D is situated in the ground parallel and adjacent to the wall containing bay E-F, yet the wall showed negligible movement when compared with the 3 m.m./m. compressive movement of the ground. Slight damage was apparent on the outside of the wall in the form of dislodged pointing but the window openings showed little sign of compressive damage. A doorway at the end of the wall in question, however, showed considerable distortion especially near the bottom. (Plate 3.4).

There was little noticeable damage to the west wall of the hall though bay A-B adjacent to it registered compressive movement of 1.5 m.m./m.

Table 3.3 shows the results obtained from the micrometer measurements. Corrections to allow for the thermal expansion of the masonry have been applied. It will be seen that, with the limits of error at $\pm$ 10/10,000 in., the movements at stations W_1, W_4, W_6, W_7 and W_8 cannot be considered significant; the movements at stations W_3 and W_5 were sufficiently in excess of the limits of error to be considered significant. It should be noted that station W_3 was set up over a crack, while at station W_5 a crack appeared between the station markers during the course of the investigation.

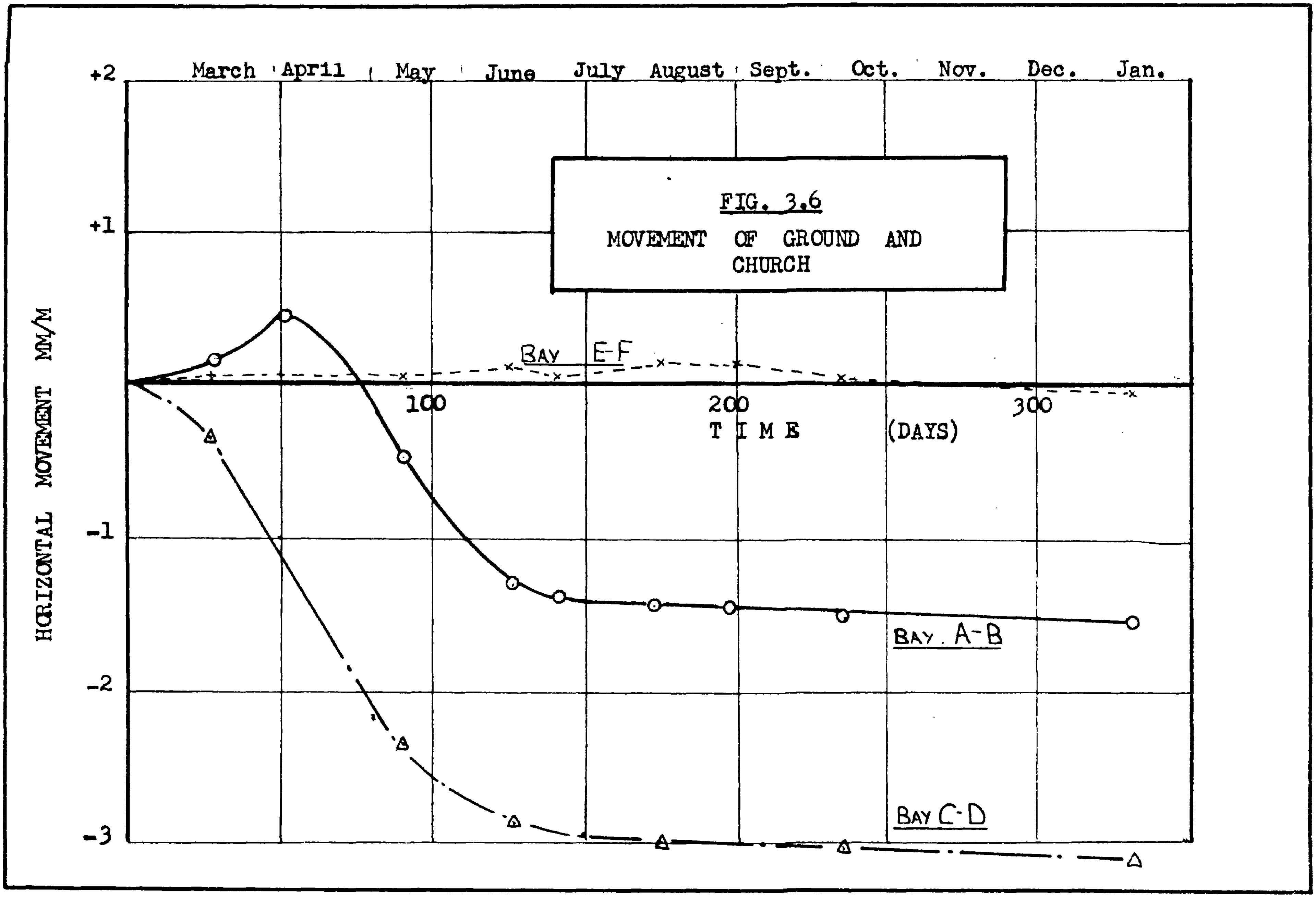


FIG. 3.6
MOVEMENT OF GROUND AND CHURCH
HORIZONTAL MOVEMENT MM/M
+2
+1
-1
-2
-3
March
April
May
June
July
August
Sept.
Oct.
Nov.
Dec.
Jan.
100
200
300
T I M E
(DAYS)
BAY E-F
BAY. A-B
BAY C-D

PLATE 3.4.

PLATE 3.5.

Section 3.7 DAMAGE TO THE BUILDINGS

3.7.1. Tensile Phase.

The buildings had been damaged by ground movements before the observations commenced. Cracks caused by previous workings had re-opened as a result of the curvature and tensile movements accompanying the initial phase of the subsidence movements. Typical cracks in existence when the tests commenced are shown in Plate 3.5. A micrometer measuring station (W_3 Fig. 3.3) was installed to measure the movement across a crack in the sill of one of the downstairs windows on the west wall of the church. The movements recorded are shown in Fig. 3.7. It will be noted that the crack reached its maximum width when the face was 60 ft (0.15 R) beyond the church. The crack subsequently closed with the advent of the compressive phase of movement.

3.7.2. Compressive Phase.

The maximum horizontal movement, to which the buildings were subjected, was compressive in nature and of magnitude 3 m.m./m. This caused the worst damage to the buildings.

The damage which occurred during the compressive phase can be classified under three headings.

(a) Damage resulting from the interaction of two walls.

(b) Damage caused by lozenging of the structure.

(c) Damage resulting from the displacement of walls relative to a roof or floor in the presence of a plane of weakness.

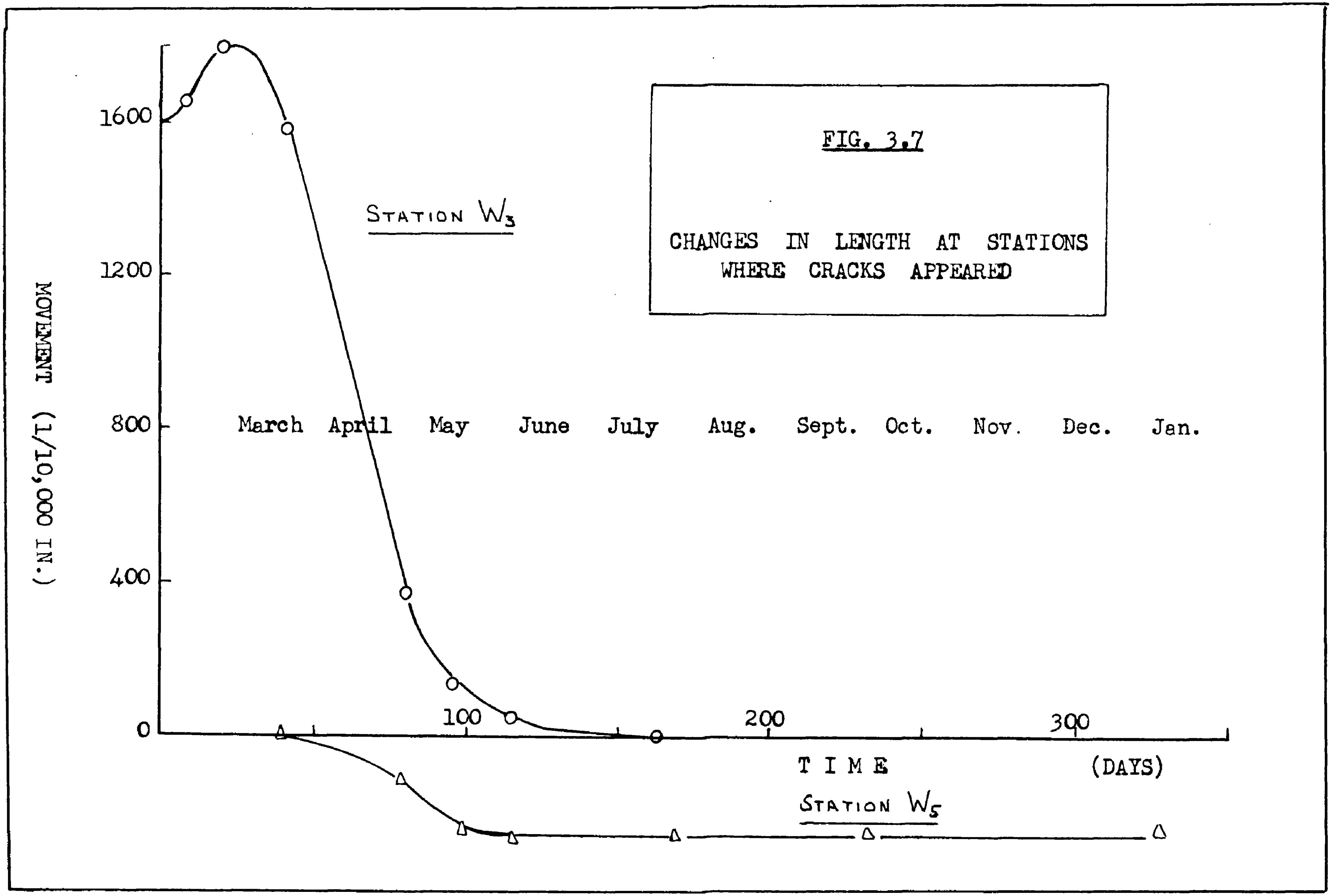


FIG. 3.7

CHANGES IN LENGTH AT STATIONS WHERE CRACKS APPEARED

The damage will therefore be discussed under these headings :-

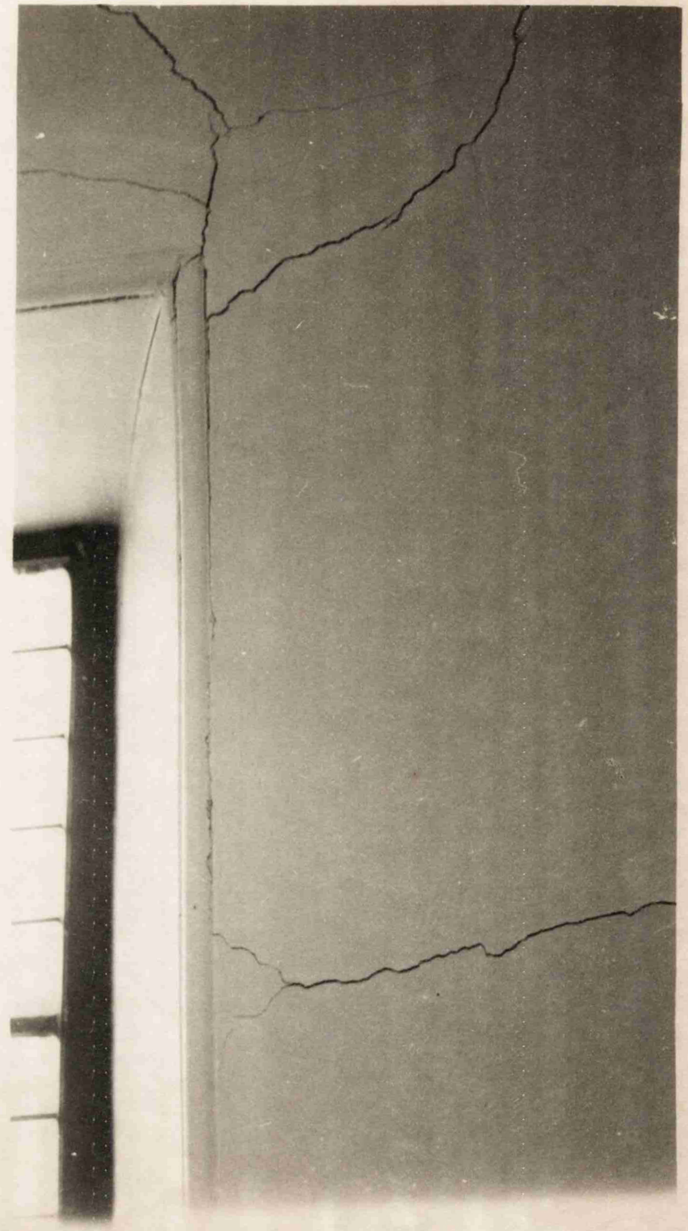
A. <u>Damage resulting from the interaction of two walls.</u> This type of damage was not noticeable in the church, vestry and manse; the places at which it occurred are shown thus (a) (b) etc. in Fig. 3.2.

The interaction of the walls of the vestry with the east wall of the church ((a) , (b)) caused this wall to be displaced inwards locally with considerable damage to the interior plaster finish. (Plate 3.6). A similar effect occurred in the vestry at (c) and in the manse at (d). Had the church and vestry been built separate from the manse it is probable that this damage would not have occurred. As it was, the differential horizontal movement in the ground, acting over the cumulative length of the church, vestry and manse, resulted in concentration of structural movement at weak points.

B. <u>Damage caused by lozenging of the structure</u> was most noticeable in the church where it was aggravated by the rigidity of the gallery.

The maximum movement to which the church was subjected was compressive in nature; it took place in a direction parallel to the face, and to the diagonal from the north west to the south east corner of the church. The maximum differential movement occurred, therefore, between the parts of the building which were furthest apart when measured in the direction of maximum

PLATE 3.6.



movement (i.e. between corners 'B' and 'D' in Fig. 3.8a.
This caused a deformation of the structure, as shown in Fig. 3.8a, which resulted in cracking of the interior plaster work in each of the four corners. The side portions of the gallery moved with the side walls of the church and pulled away from the end wall at 'A' and 'E' (Fig. 3.8). A large crack developed under the east side of the gallery at its junction with the south wall; maximum movement occurred at 'E', minimum at 'B' (Plate 3.7a). A crack in a similar position on the west side showed maximum movement in the corner 'A'.

Damage to the exterior of the church at corner 'A' was also consistent with the suggested deformation of the church (Plate 3.7b); rotation of the walls 'AB' and 'AD' relative to each other would produce the stepped effect at the corner stones.

C. <u>Damage resulting from the displacement of walls relative to a roof or floor in the presence of a plane of weakness</u> was most noticeable in the hall. It will be seen from Fig. 3.9 and Plate 3.8 that movements took place along three distinct planes,

(i) at the top of windows,
(ii) at the bottom of windows,
and (iii) below floor level at the damp course.

Movements at (i) and (ii) are attributable to the resistance to compression offered by the roof. This resulted in

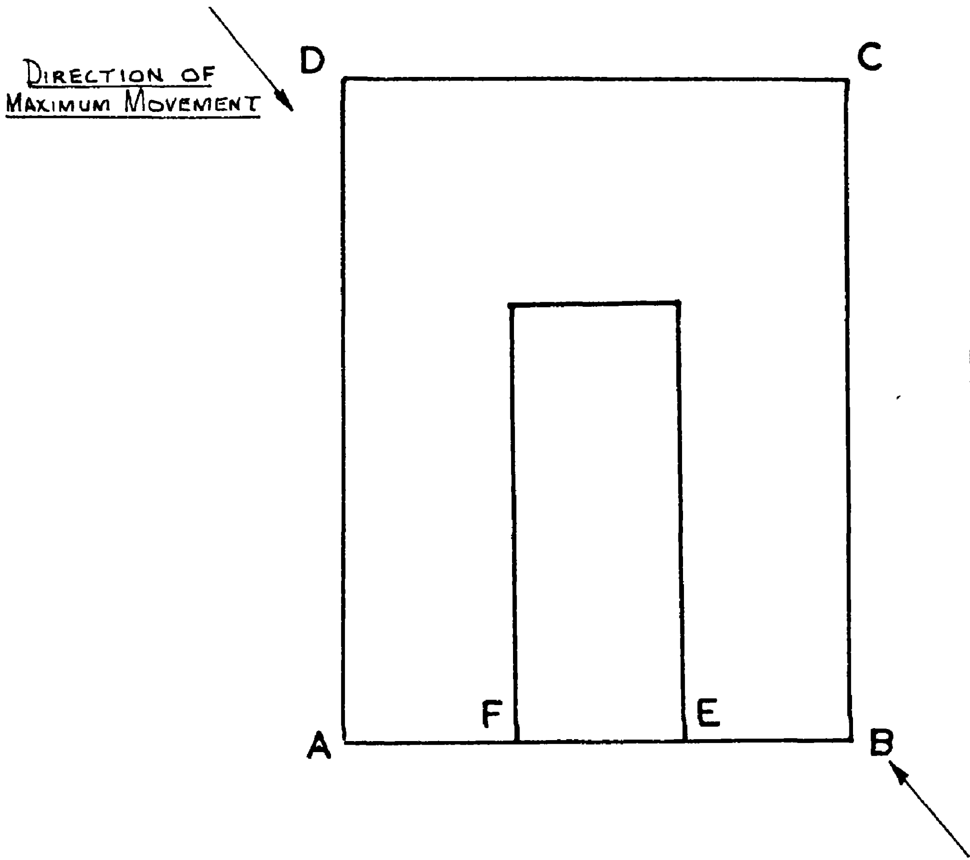


FIG. 3.8 A.

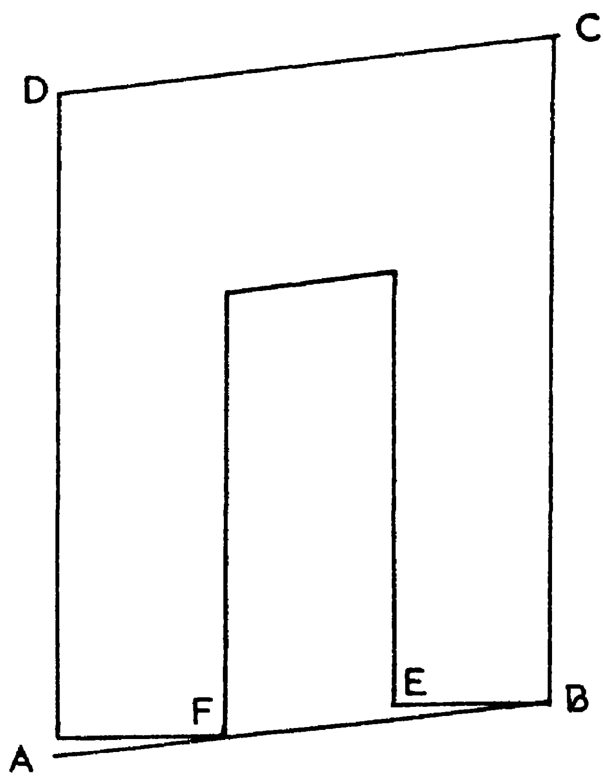


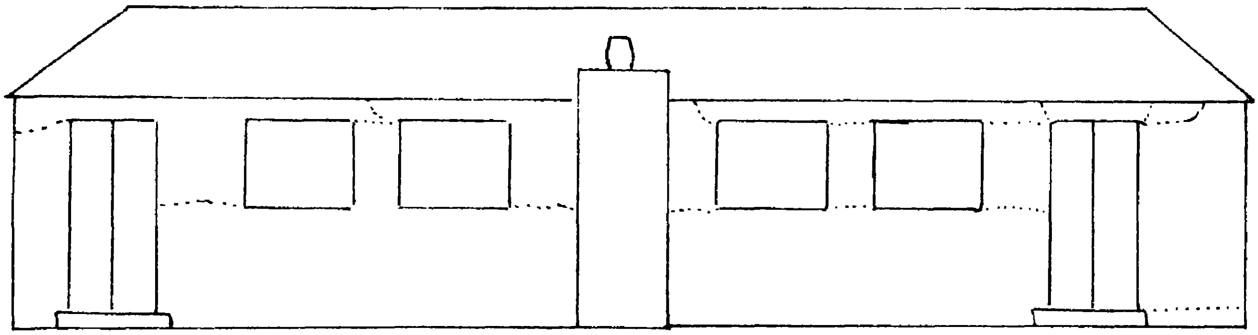
FIG. 3.8 B.

DEFORMATION OF CHURCH

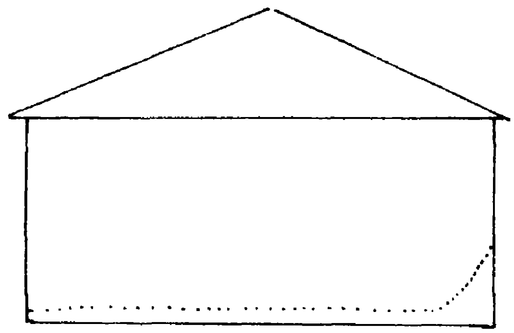
(A)

(B)

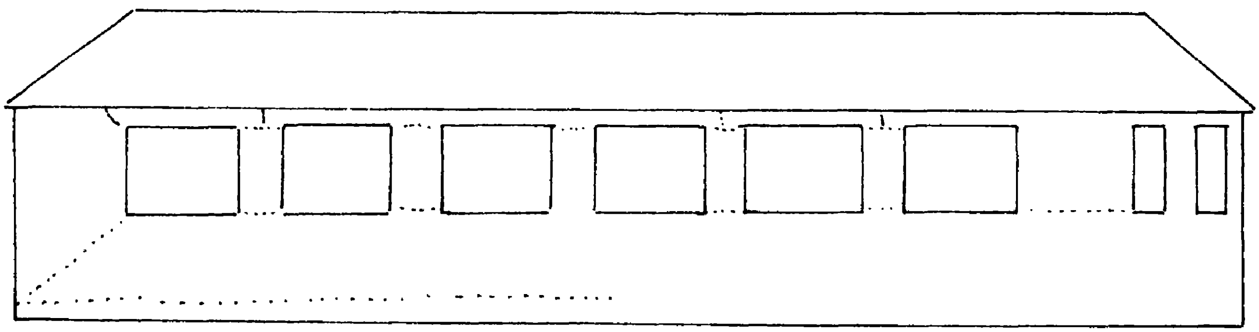
PLATE 3.7.



North Elevation



West Elevation



South Elevation

DAMAGE TO HALL

FIG. 3.9

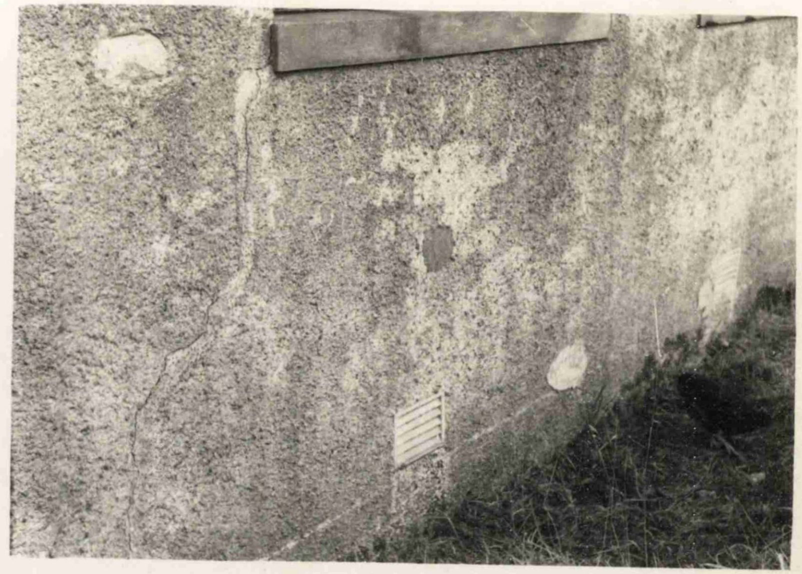
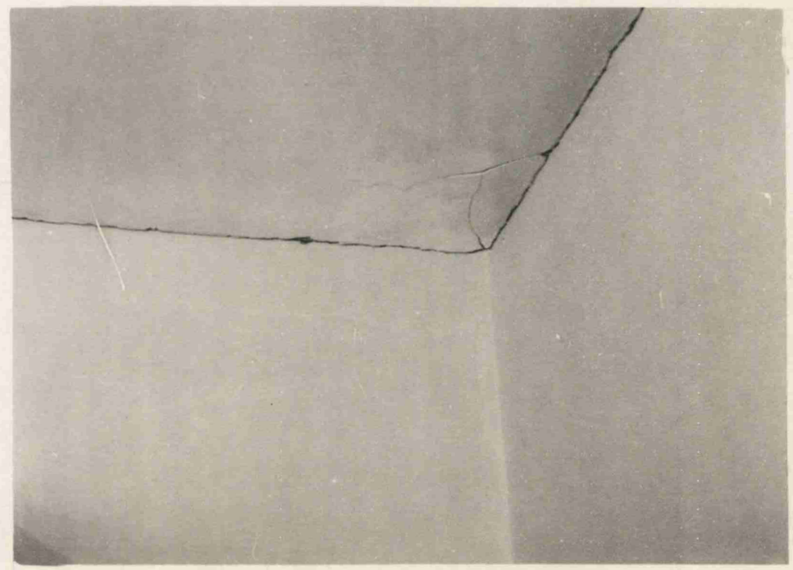


PLATE 3.8.



concentration of movements below the roof along lines of weakness.

Similarly the stiffness of the floor resulted in movement taking place along the damp course (iii). Displacement along the damp course was greatest at the west end of the hall and was not apparent halfway along the length of the hall.

Movement of the walls relative to the roof was apparent all round the inside of the church. This movement resulted in the formation of horizontal cracks at or near the ceiling-wall junction. Displacements occurred in the direction of the length of the wall; maximum displacements occurred at corners, minimum at the centre of each wall (Plate 3.3).

3.7.3. General.

Some 60 days after the start of the test the recorded ground strain adjacent to the church was of the order of 1.5 m.m./m. compressive. Fairly extensive cracking of interior plaster work had occurred; damage to the hall had occurred along the three planes of weakness already discussed; reports had been received that doors in the manse were jamming; distortion of the doorway at the end of the south wall of the church had occurred. According to the scale of damage referred to by Orchard [3.2] the damage could be classified as appreciable. The greatest continuous length of structure is about

90 ft (measured over the church, vestry and manse). Data given by Orchard show that when a building of length 90 ft is subjected to a strain of 1.5 m.m./m. the damage to be expected falls under the 'slight' category. There is thus a difference between the expected and recorded damage. The difference, however, is not significant when the fact that the buildings had been cracked by previous workings is taken into consideration.

When the ground movements had attained their maximum of 3 m.m./m., compressive, the damage to the buildings still fell under the 'appreciable' category. In this case good agreement is obtained with the prediction made from data given by Orchard [3.2].

Section 3.8 CONCLUSIONS

The stoppage of the face under the measuring line prevented an overall picture of the movements in the vicinity of the church from being obtained. However, from the data obtained the following factors emerged.

1) The tape measuring technique used on ground and wall stations was satisfactory; with the 25 ft tape used it gave 95% confidence limits of $\pm$ 0.04 parts per thousand.

2) The measuring micrometer proved suitable for recording movement across cracks. It was not sufficiently accurate

to record movements on uncracked portions of masonry.

3) Horizontal movements continued to develop at the surface over the coal face for some 70 days after the face had stopped.

4) Movements in the walls were generally not of measurable magnitude. Ground movements were accommodated in openings or were concentrated at cracks.

5) Different layouts and methods of construction of the buildings gave rise to different types of damage; the hall was the least damaged of the buildings as a result of some the ground movement being accommodated by slipping on the damp course; the worst damage was caused in the church, vestry and manse due to the concentration of the movement over their combined lengths at a few weak points.

Section 4.1 DESCRIPTION OF SITE

4.1.1. Geology.

The Little Splint Seam as worked from Woolmet Colliery is approximately 3 feet thick. In the vicinity of Newton Village it lies at a depth of 400 feet and dips in an easterly direction at 1 in 20. Bed-rock in the area is overlain by a layer of boulder clay 15 feet thick. The thickness of the unconsolidated layer was determined by a seismographic technique described in appendix 2.

4.1.2. Details of Workings.

To minimise damage to buildings the Little Splint Seam was worked under Newton Village by a series of stepped faces as shown in Fig. 4.1. The faces were approximately 350 feet wide with steps of about 200 feet between them. Details of the packing system used are :-

Road side packs	-	20 ft
Face packs	-	16 ft., with 12 ft dummy roads between.

4.1.3. Newton Village.

The buildings in the village are blocks of four flats. The blocks are of two types, both of conventional brick construction with an exterior finish of either plain brick or rough-cast.

The wall surrounding the school is of brick, 9 inches thick,

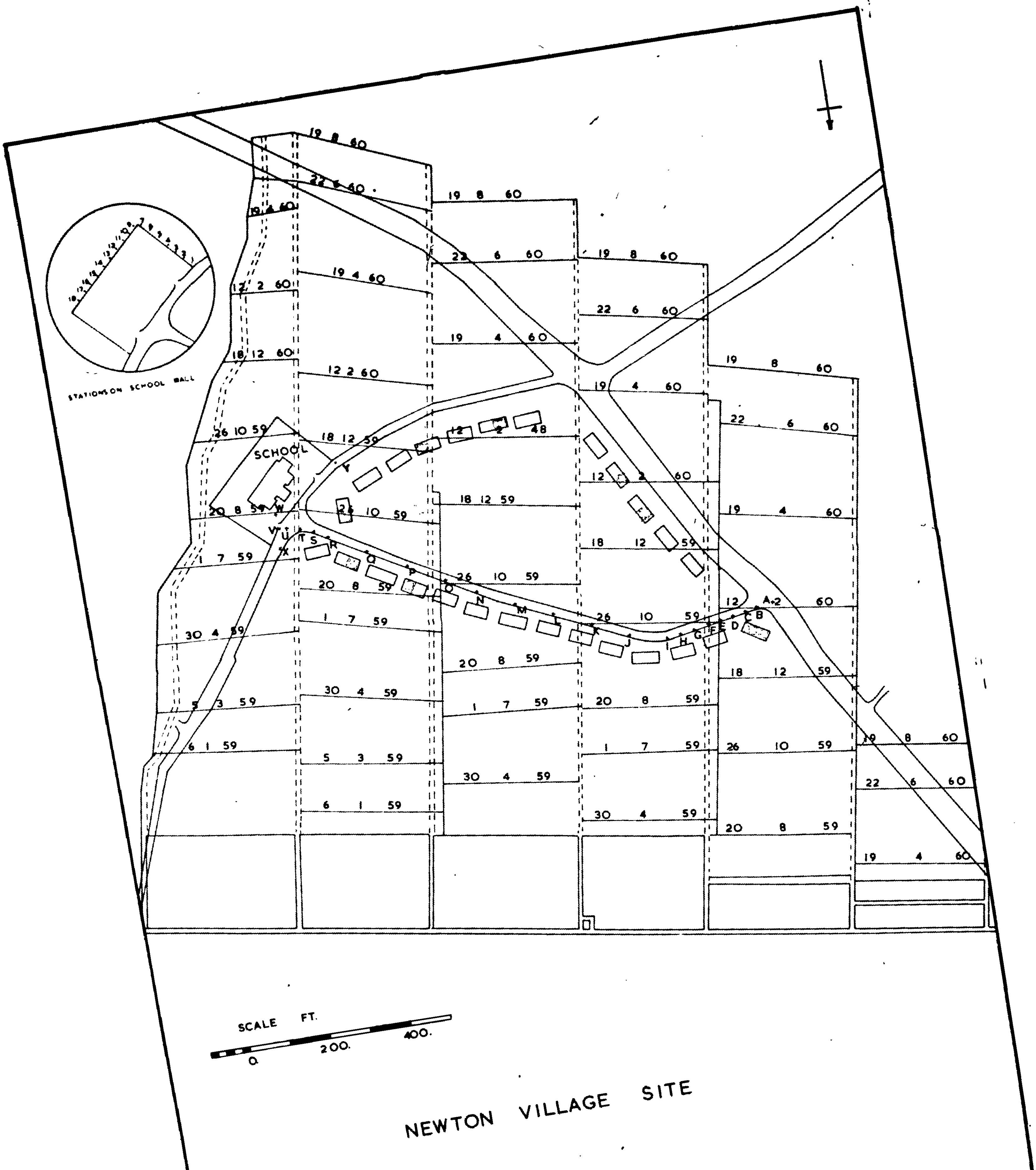

SCHOOL
STATIONS ON SCHOOL WALL
SCALE FT.
0
200.
400.
NEWTON VILLAGE SITE

FIG. 4.1

5 to 6 feet high, with concrete coping stones on top (See Plate 4.2). It rests on a strip foundation on a stiff clay soil.

Section 4.2 SCOPE OF INVESTIGATION

4.2.1. Stepped Face Layout.

The reasons for adopting a stepped layout were :-

(a) To reduce the magnitude of the ground movements and so lessen the risk of damage to the buildings in the village.

(b) To provide information about the efficacy of the stepped face method of work prior to its being used in the extraction of coal from under Newton Church.

4.2.2. The Test Programme.

To provide the information required for (b) a number of measuring stations were installed (A to X Fig. 4.1) to record the magnitude of the horizontal and vertical movements caused by the working under Newton Village.

The scope of the investigation was widened as follows :-

(a) By noting the incidence of damage to buildings in the village an attempt was made

i) to determine which of the two types of building in the village was more liable to damage,

ii) to determine the effect of the orientation of buildings on the incidence of damage, and

iii) to determine the effect of the magnitude of ground movement on the incidence of damage.

(b) By measuring the movements in the wall surrounding the school, and the movements in the ground adjacent to the wall, to investigate the factors governing the transmission of movement from ground to wall. For this purpose two series of stations were established, one series in the wall the other in the ground adjacent to it. (Stations 1 - 19, Fig. 4.1).

Section 4.3 DESCRIPTIONS OF MEASURING STATIONS

4.3.1. Road Stations.

Domed headed steel bolts about 9 inches long were used as station markers at positions 'A' to 'X' (Fig. 4.1). A measuring mark for the linear measurements was made with a centre punch after the bolts had been driven into the gutter at the side of the road.

4.3.2. Ground Stations.

The stations in the ground adjacent to the school wall, (i.e. stations 1 to 19) and the extension pieces for use with them, were of the type used at Douglas Water and described in Section 3.2.1. In addition, a short extension piece was provided for use during levelling.

4.3.3. Wall Stations.

The station markers in the wall were constructed from O. B.A. cheese headed brass bolts. The bolt head was machined flat and scribed with a cross. The markers were grouted into holes drilled in the wall.

Section 4.4 MEASURING APPARATUS AND TECHNIQUES

4.4.1. Levelling.

The apparatus used comprised a Cooke, Troughton & Simms, S 500, geodetic level and invar staff. (See 2.3.1). All stations were levelled using the standard procedure recommended by the National Coal Board [4.1] the levels being referred to a stable bench mark at Woolmet Colliery.

4.4.2. Length Measurements on Road.

The apparatus and procedure used when measuring between ground stations 'A' to 'X' was in accordance with National Coal Board recommendations [4.1]. A 100 ft steel tape graduated in feet, tenths and hundredths on one side, and in metres, centimetres and millimetres on the other, was used. The tape was stretched along the ground between stations under a tension of 20 lb. The tension was applied by means of straining poles and measured on a spring balance. Both ends of the tape were read simultaneously. Six measurements of the length were made and the mean accepted as the length of the bay. In order to apply a correction for the thermal expansion of the tape the temperature was read on a thermometer attached to the tape.

4.4.3. Length Measurements on Ground in Field.

The apparatus and technique used in measuring the distance between stations in the ground adjacent to the school wall were identical to those used at the Douglas Water site and described in Section 3.5.1.

4.4.4. Length Measurements on Wall.

In the light of experience gained at the Douglas Water Site a taping technique was adopted for measuring the change in distance between station markers on the wall. The measuring technique was identical to that used at Douglas Water. The brackets, between which the tape was suspended during measuring, were demountable. Each bracket was constructed from a piece of slotted angle to which was bolted a 2 inch pin (See Plate 4.1). During measuring the brackets were fitted to the wall by inserting the pin in a hole drilled in the wall.

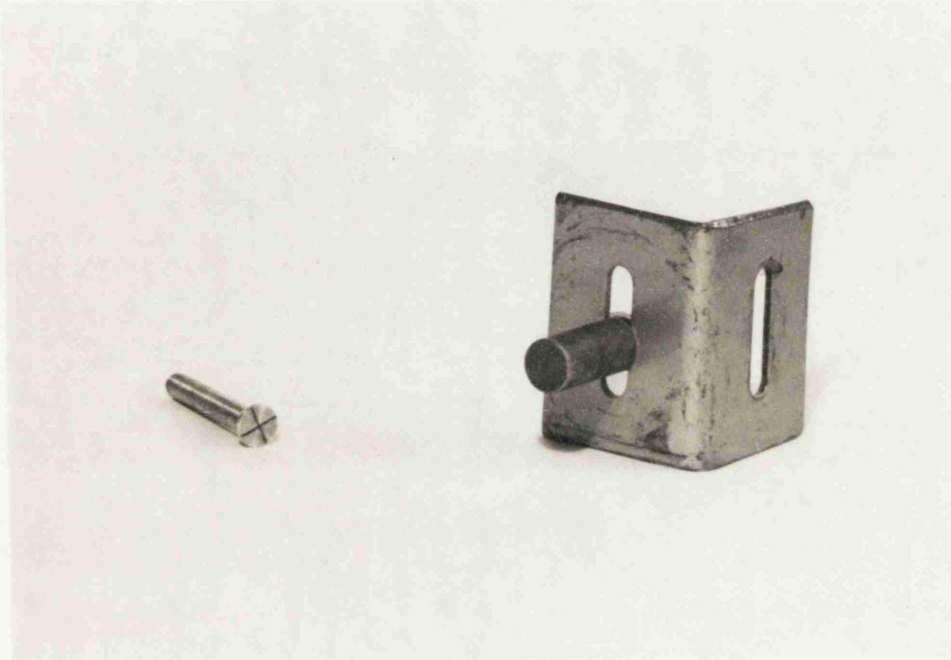


PLATE 4.1. WALL STATION & STRAINING BRACKET

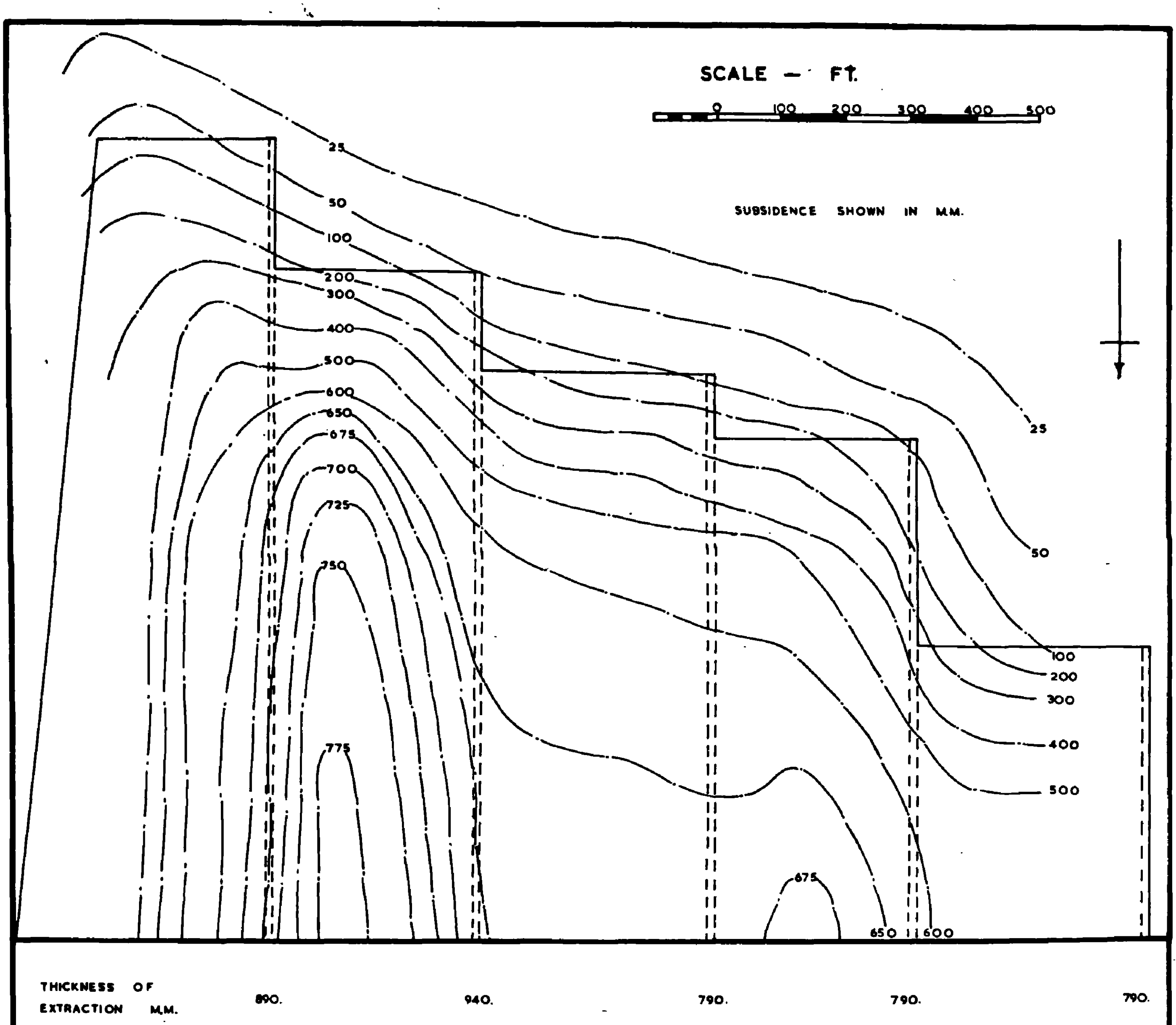
PLATE 4.2. DAMAGE DUE TO COMPRESSION

Section 4.5 RESULTS

4.5.1. Village.

Periodic measurements were made on stations 'A' to 'X' to determine the magnitude and distribution of horizontal and vertical movement. The data obtained from the levelling were used to construct the subsidence contour diagram shown in Fig. 4.2. This was done by plotting the measuring stations on a scale diagram in their correct position relative to the face on each of the dates when measurements were made; i.e. the subsidence measured at a point on a particular measuring day was plotted on the diagram at a distance from that measured on the previous measuring day, equal to the face advance in the time between the two measurements. Such a procedure assumes that the rate of development of surface movement is equal to the rate of face advance. In the light of experimental evidence obtained in Yorkshire [3.1] it was felt that the assumption was justified. The distance between each of the faces remained fairly constant throughout the experiment, except during the last three months when slight changes took place (Fig. 4.1). These changes were neglected when showing the relative position of each face in Fig. 4.2.

It will be seen from Fig. 4.2 that subsidence developed uniformly over all faces to a distance approximately 300 feet behind the faces. At distances in excess of this three distinct zones developed - two regions of high subsidence separated by a region where the final subsidence was less. Maximum subsidence occurred over the waste of



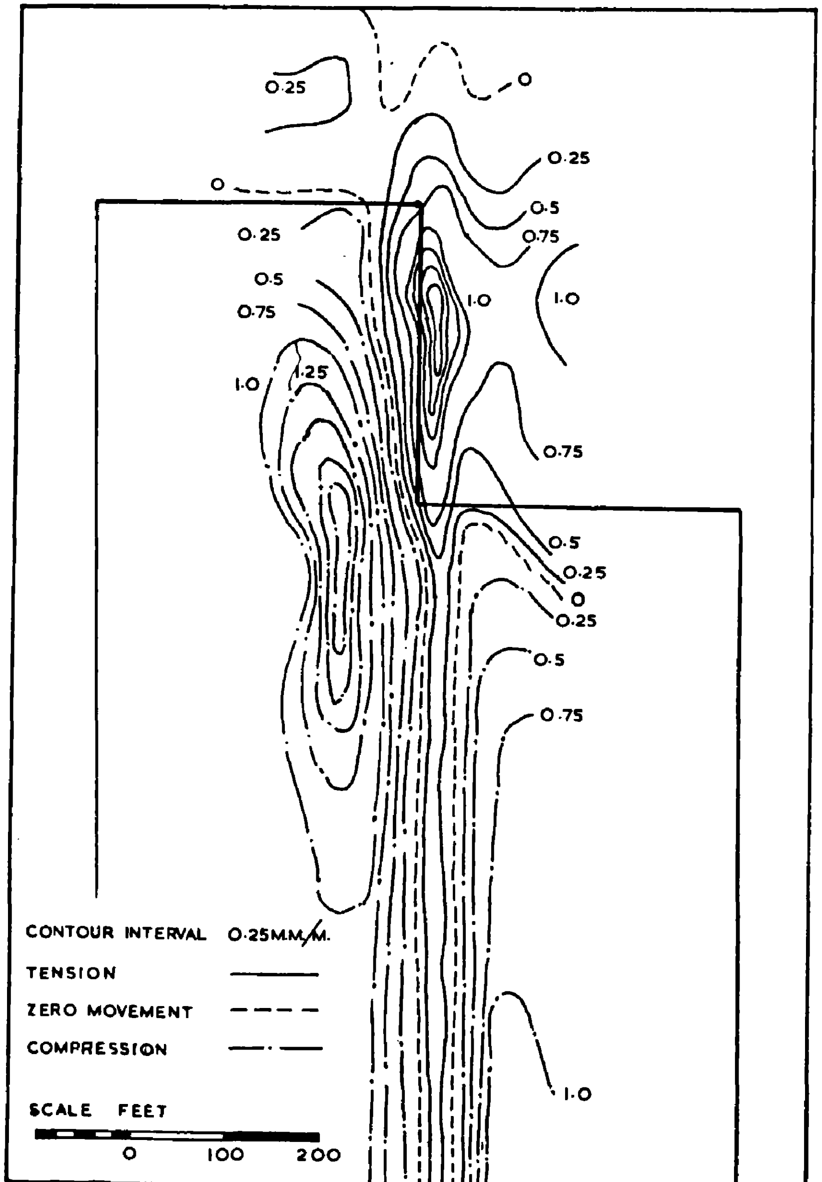
NEWTON VILLAGE SITE. SUBSIDENCE CONTOURS.

FIG. 4.2

the second face from the eastern end of the excavation where the subsidence is some 15% greater than in the corresponding zone over the fourth face. It was noted that there was a variation in the thickness of the seam along the line of measuring stations which amounted to some 13% between faces 2 and 4. In the light of this, it is suggested that the irregular pattern of subsidence at a distance behind the face is due to variations in the thickness of extraction over the area.

On consideration of the movements of station 'R', the station at which maximum vertical movement occurred, the following factors emerge. The subsidence which occurred when the face was directly underneath was 30% of the final subsidence at that point. At a distance of 300 feet behind the face 90% of the total subsidence had taken place. Maximum subsidence was not attained until the face had advanced a further 300 feet. This is in accordance with the findings of other investigators. [4.2].

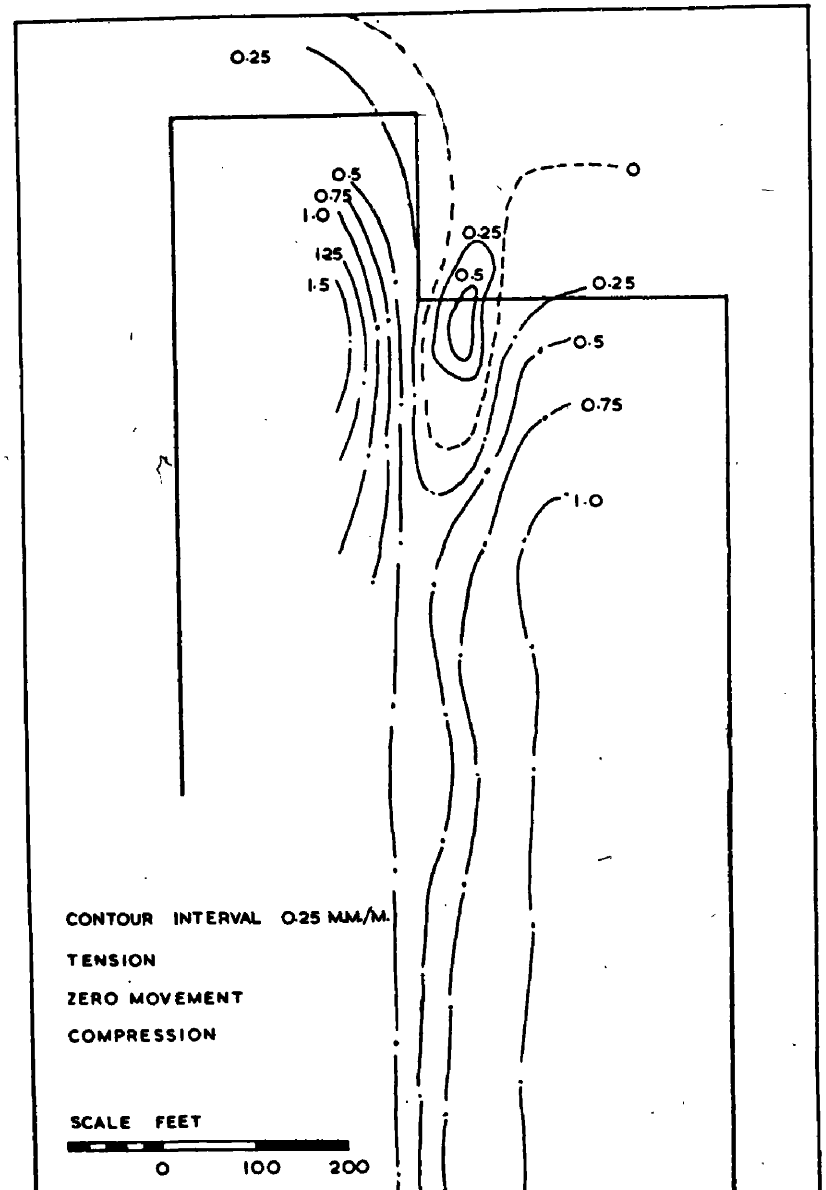
A comparison of the results of linear measurements at stations 'A' to 'I' with those at stations 'Q' to 'V' yielded information about the effect of the distance between faces on the magnitude of ground strain. Strain diagrams were constructed to show the development of movement in these two regions (Figs. 4.3 and 4.4). Fig. 4.3 shows the development of strain over the junction of two faces separated by a step of 300 feet. A maximum tension of about 2 m.m./m. was recorded over the rib-side of the first face at a distance of 150 feet behind the face; the corresponding compressive strain was 1 m.m./m. This compressive strain occurred over the waste of the first face and built



STRAIN CONTOUR DIAGRAM

STATIONS A TO I

FIG. 4.3



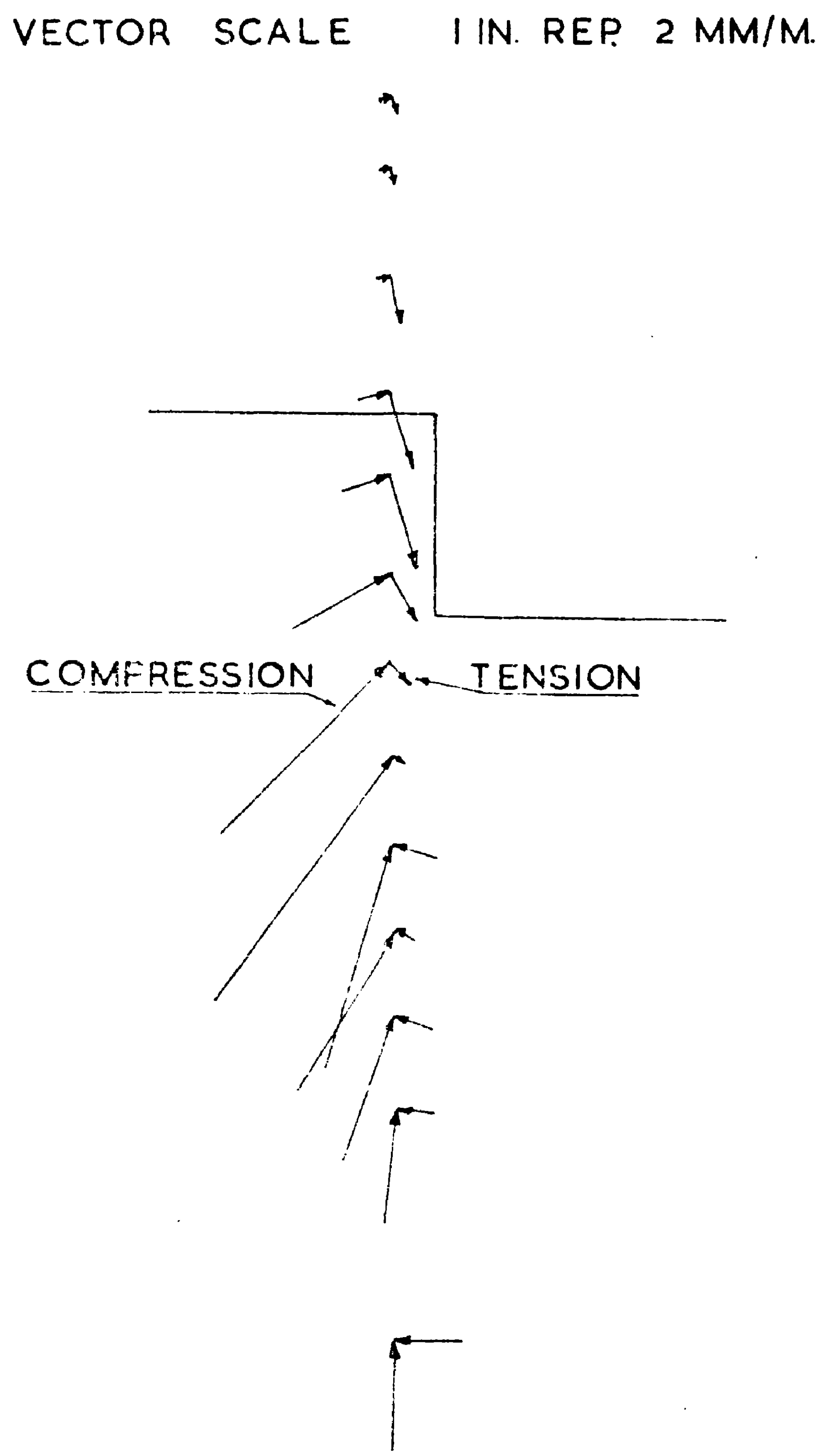
STRAIN CONTOUR DIAGRAM

STATIONS Q TO V

FIG. 4.4

up to a maximum value of 2.3 m.m./m. at a distance of 350 feet behind the face. With the passage of the second face under and beyond the measuring points, the peak values of strain gradually reduced. By comparison Fig. 4.4 shows that the magnitude of the peak strains over faces separated by 200 feet was much less. In this case the maximum tensile strain was 0.5 m.m./m. and the maximum compressive 1.5 m.m./m. The magnitude of strain at a distance behind the face is comparable with that shown in a similar position in Fig. 4.3.

Using the results of measurements between stations 'U', 'V' and 'X' the magnitude and direction of the principal horizontal movements in the vicinity of these stations was estimated. These movements are shown vectorially in Fig. 4.5 in the correct relative position to each other and to the face. The maximum movement was compressive in nature and acted in the direction of maximum slope; it occurred at a distance of 150 feet behind the face. It is of interest to note that on the day this maximum movement of -3 m.m./m. was recorded the movements parallel and perpendicular to the face were -0.80 m.m./m. and -1.60 m.m./m. respectively. This shows clearly the effect that the direction of the measuring line can have on the magnitude of movement recorded and is strong evidence in favour of the use of a staggered line of measuring stations (Section 3.4.1.). At distances greater than 150 feet behind the face the compressive movements progressively reduced in magnitude to a limiting value at a distance of 500 feet behind the face. The major movements there took place more or less in the direction of face advance and were approximately 3 times greater than the corresponding



PRINCIPAL MOVEMENTS AT STATIONS 'U' 'V' & 'X'

FIG 4.5

minor movement. This is contrary to the general assumption that at a distance behind a working face the residual movements occur in a direction parallel to the face.

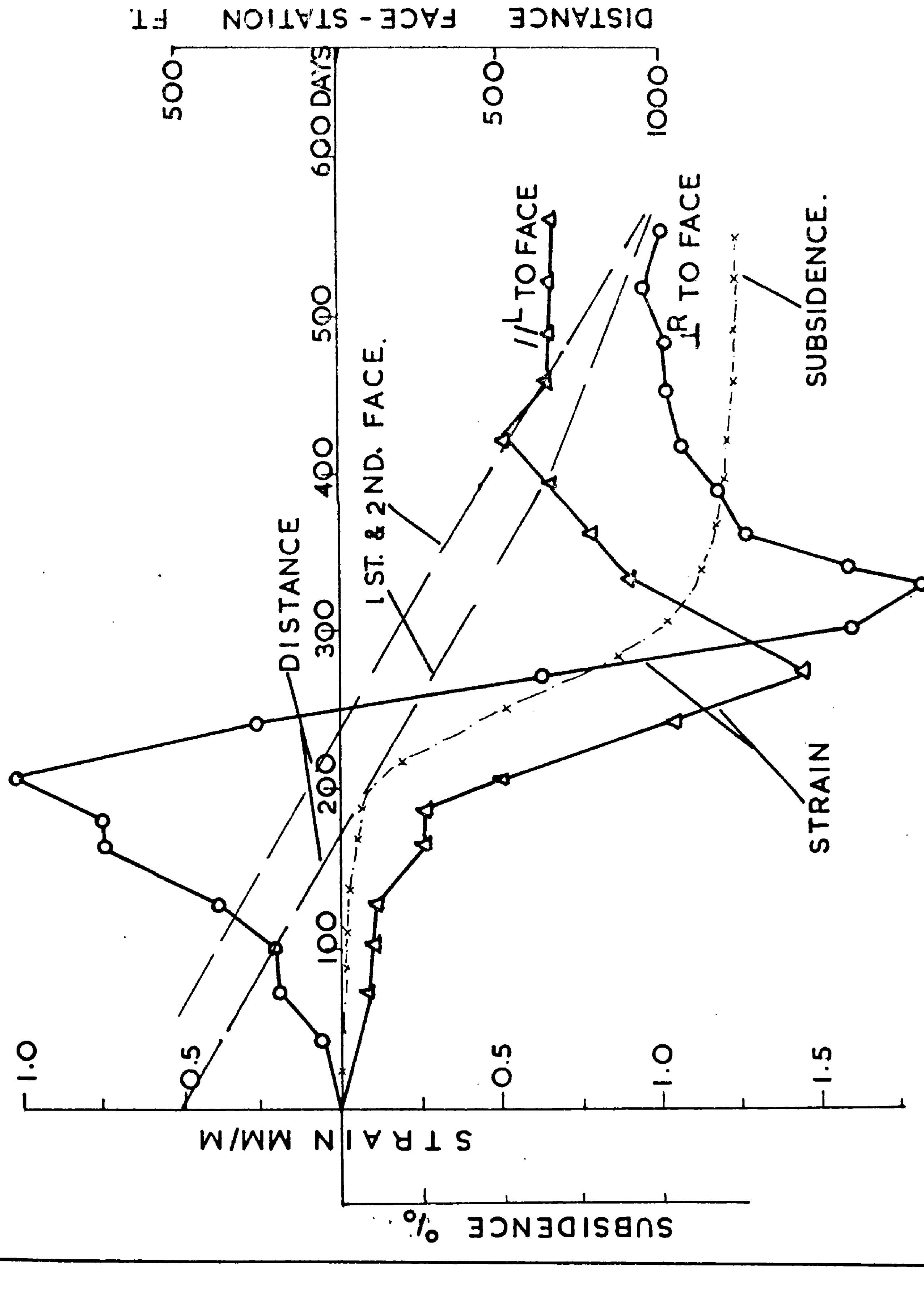
Fig. 4.6 was drawn to show the relationship between subsidence and horizontal movement in two directions at a point over the step between two faces.

Subsidence developed uniformly as the two faces worked in sequence under the point. The build-up of tensile movement, perpendicular to and in front of the face, was broken by two steps corresponding to periods when the rate of face advance was considerably below normal. Maximum tension was recorded at a distance of 100 feet behind the first face. Thereafter the extension in the ground decreased rapidly and changed to compression as the second face moved under the measuring stations. Maximum compression was recorded at a distance of 250 feet behind the second. The compression subsequently dropped off rapidly and reached a limiting value about 600 feet behind the second face. It will be noted that the peak values of tension and compression took place where the curvature of the vertical movement profile was greatest.

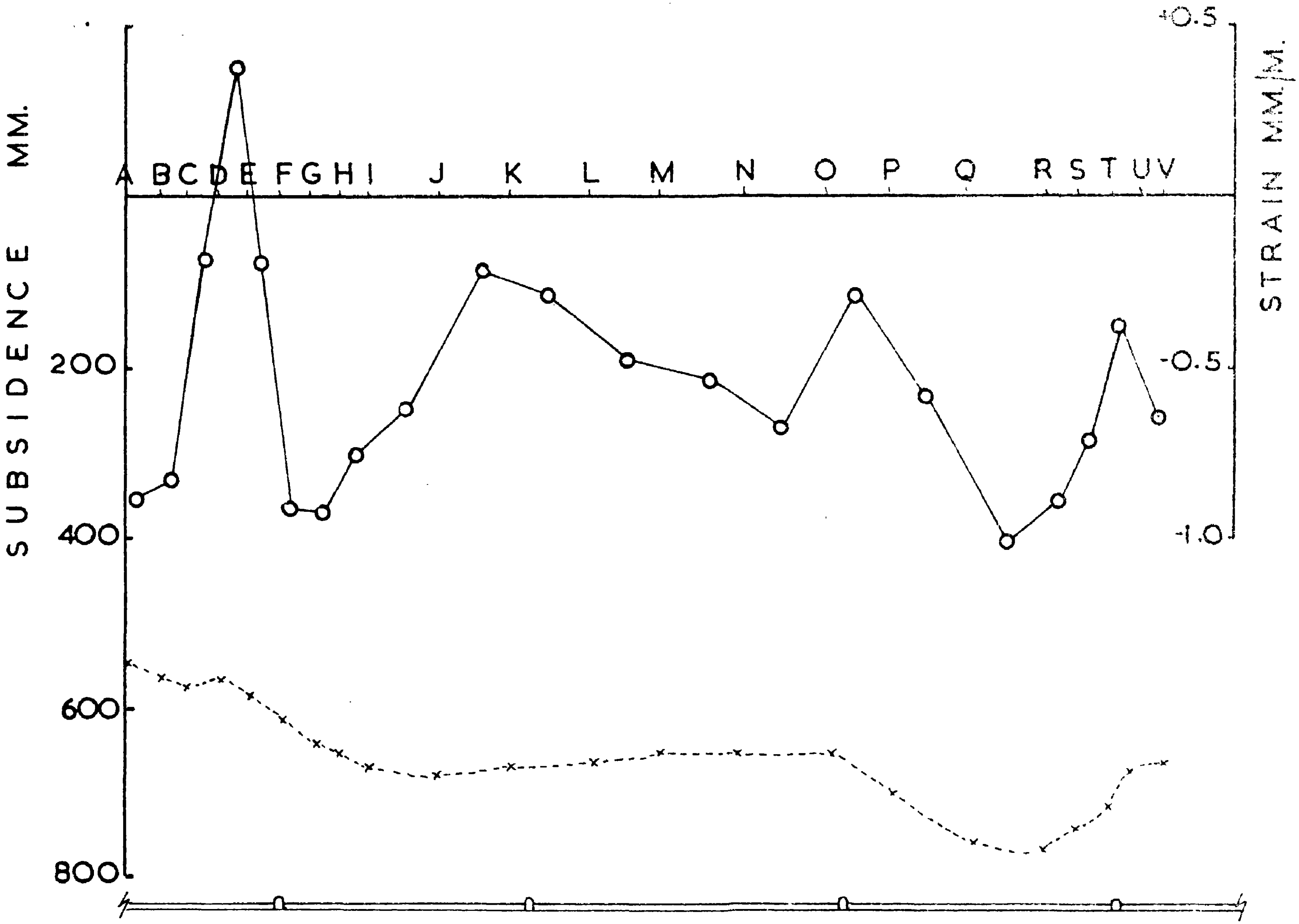
Movement parallel to the face was at all times compressive in nature. The peak value of movement occurred at a distance of 100 feet behind the second face i.e. much earlier than the corresponding peak in the movement at right angles to the face.

The final movements, horizontal and vertical, along the whole line of measuring stations are shown in Fig. 4.7. The peaks of the

FIG. 4.6



DISTANCE FACE - STATION FT.
500
600 DAYS
500
1000
100
200
300
400
500
1.0
0.5
0.5
1.0
1.5
STRAIN MM/M
SUBSIDENCE %
DISTANCE
1 ST. & 2ND. FACE.
// TO FACE
⊥ TO FACE
SUBSIDENCE.
STRAIN



SUBSIDENCE MM.
STRAIN MM/M.
A B C D E F G H I J K L M N O P Q R S T U V
200
400
600
800
+0.5
-0.5
-1.0

FIG. 4.7

horizontal movement curve are clearly related to the position of the gate roads in the seam and to the curvature of the vertical movement profile. The greatest difference between positive and negative peaks occurs between stations 'E' and 'H', i.e. where the step between the two faces was 300 feet.

The houses in the village where damage occurred are shown in red in Fig. 4.1. There is no definite pattern in the distribution of the damage and there is insufficient evidence to show whether the orientation of the houses and differences in their construction had any effect on the incidence of damage.

4.5.2. Newton School Wall.

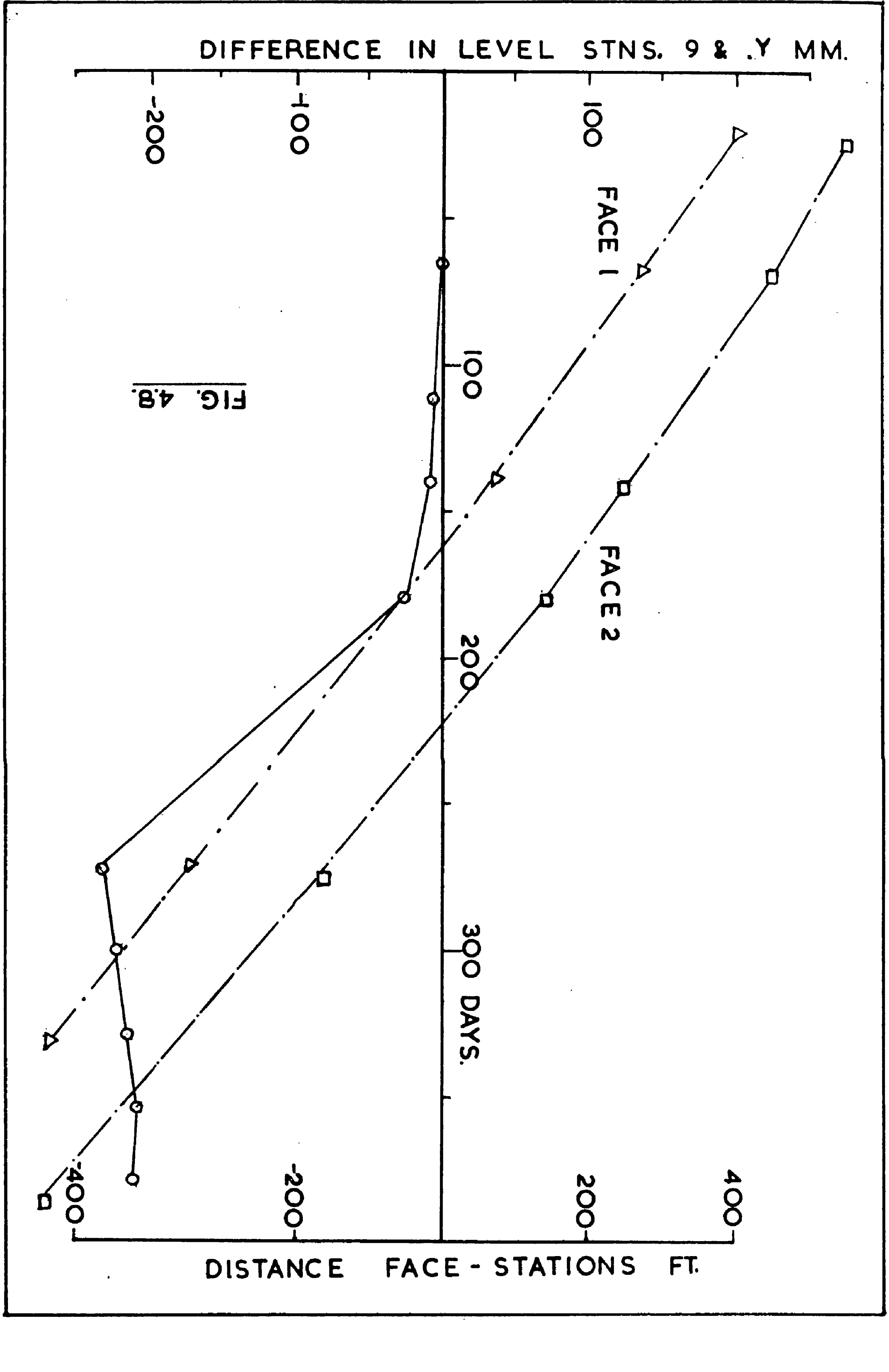
The results obtained from measurements on, and adjacent to, the wall are shown in Tables 4.1.

The vertical movement of station 'Y' relative to station 9 is shown in Fig. 4.8. The values plotted are thus measures of the slope between the two stations, while the gradient of the curve is a measure of the rate of change of slope. The gradient of the curve and the rate of change of slope in the ground, were greatest during the period from 200 to 250 days after the start of measurements. This was the period during which maximum tension was recorded at stations 2 to 7 (See Fig. 4.9). There is also a correspondence between the change in sign of the gradient of the slope curve with the change from tension to compression shown on the strain curve (Fig. 4.9). Similar relationships

TABLE 4.1.

DATE / BAY	10/5/59	30/6/59	7/8/59	14/9/59	23/9/59	8/12/59	13/1/60	10/2/60	9/3/60	7/4/60	4/5/60	1/6/60	30/6/60	27/7/60	
2-3	+0.07	+0.32	+0.46	+0.44	+0.46	+0.82	+1.185	+0.37	-0.04	-0.04	-0.09	-0.16	-0.14	-0.21	GROUND
4-5	-0.23	-0.03	-0.22	-0.21	-0.17	-0.16	0	-0.38	-1.01	-1.43	-1.48	-1.76	-2.01	-1.85	
6-7	-0.05	-0.12	-0.21	-0.21	-0.19	-0.28	+0.01	+0.27	-0.14	-0.42	-0.47	-0.73	-0.74	-0.63	
9-10	0	-0.02	+0.04	+0.19	+0.28	-0.30	-1.23	-1.84	-0.91	-0.02	+0.38	+0.35	+0.53	+0.49	
11-12	-0.10	-0.32	-0.61	-0.25	+0.04	-0.62	-1.58	-1.82	-1.13	-0.93	-0.46	-0.60	-0.32	-0.18	
13-14	-0.05	0	+0.23	+0.44	+0.36	-1.32	-2.16	-1.66	-0.88	-0.40	-0.18	-0.10	+0.25	+0.10	
15-16	+0.07	+0.18	+0.39	+0.12	-0.12	-1.75	-1.58	-0.83	-0.33	+0.05	+0.16	+0.17	+0.14	+0.26	
17-18	+0.21	+0.43	-	+0.29	-0.13	-1.06	-0.70	-0.17	+0.14	+0.28	+0.47	+0.41	+0.47	+0.52	
2-3						+0.32	+0.62	-0.17	-0.93	-1.05	-1.08	-1.09	-1.20	-1.15	WALL
4-5						-0.05	+0.04	-0.29	-0.58	-0.78	-0.77	-0.80	-0.87	-0.87	
6-7						-0.17	+0.01	+0.26	-0.40	-0.46	-0.50	-0.49	-0.46	-0.47	
9-10						-0.82	-0.95	-0.87	-0.16	+0.57	+0.92	+1.19	+1.23	+1.23	
11-12						-1.57	-1.85	-1.69	-1.03	-0.58	-0.33	-0.21	-0.11	-0.11	
13-14						-1.01	-1.15	-0.87	-0.20	+0.09	+0.24	+0.35	+0.44	+0.47	
15-16						-0.98	-0.81	-0.21	+0.18	+0.33	+0.45	+0.56	+0.59	+0.61	
17-18						-1.04	-0.82	-0.16	+0.19	+0.38	+0.45	+0.55	+0.64	+0.70	
Days	26	66	105	142	151	197	233	261	289	318	345	373	402	431	

STRAIN (MM/M) IN GROUND AND WALL



DIFFERENCE IN LEVEL STNS. 9 & Y MM.
-200
-100
100
FACE 1
FACE 2
100
200
300 DAYS.
400
200
-200
-400
DISTANCE FACE - STATIONS FT.

FIG. 4.8.

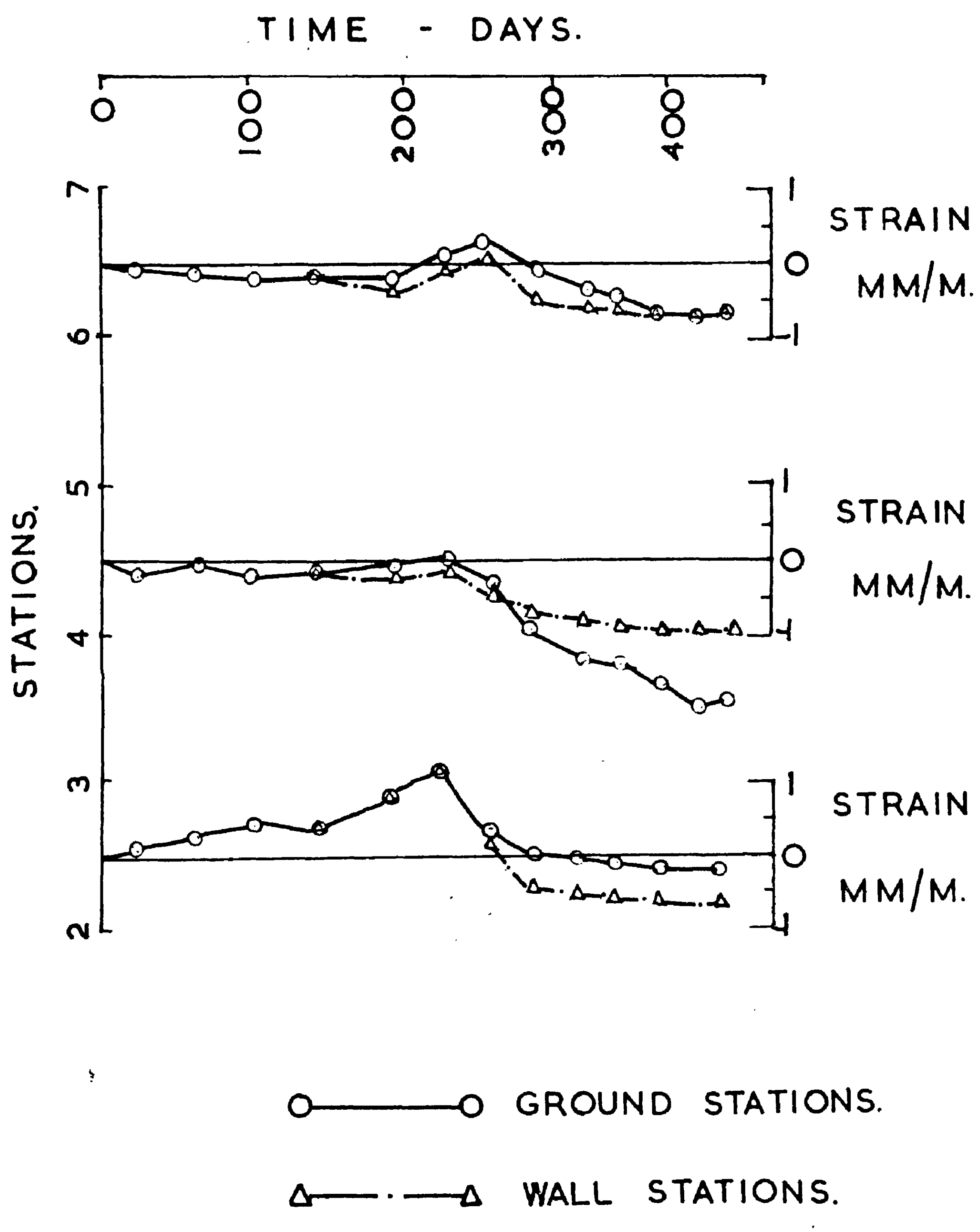
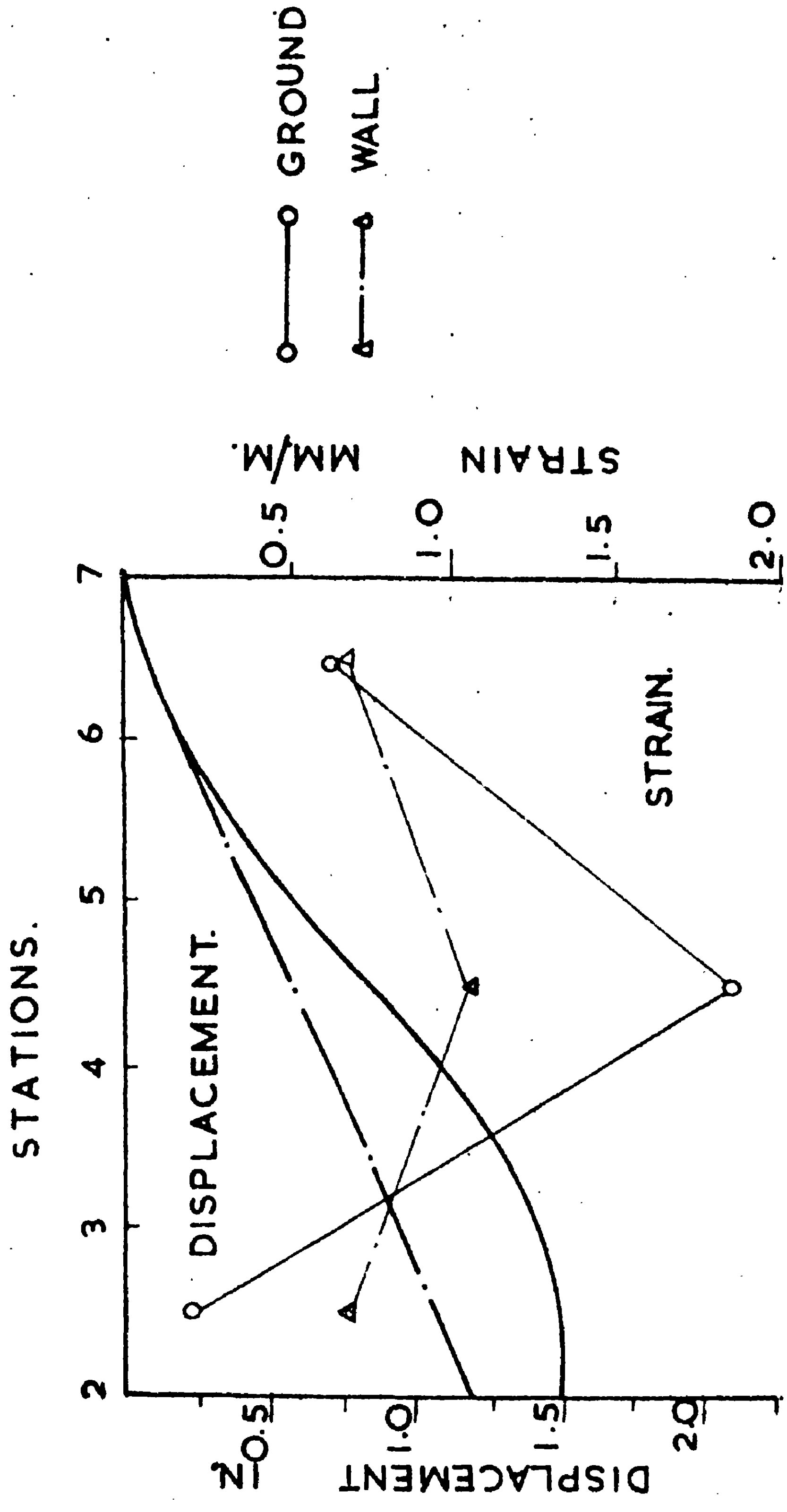

TIME - DAYS.
0
100
200
300
400
STATIONS.
7
6
5
4
3
2
STRAIN
MM/M.
1
0
-1
STRAIN
MM/M.
STRAIN
MM/M.
GROUND STATIONS.
WALL STATIONS.

FIG. 4.9.



GROUND
WALL
STATIONS.
2
3
4
5
6
7
DISPLACEMENT.
STRAIN.
STRAIN MM/M.
0.5
1.0
1.5
2.0
DISPLACEMENT IN.
0.5
1.0
1.5
2.0

FIG 4.10.

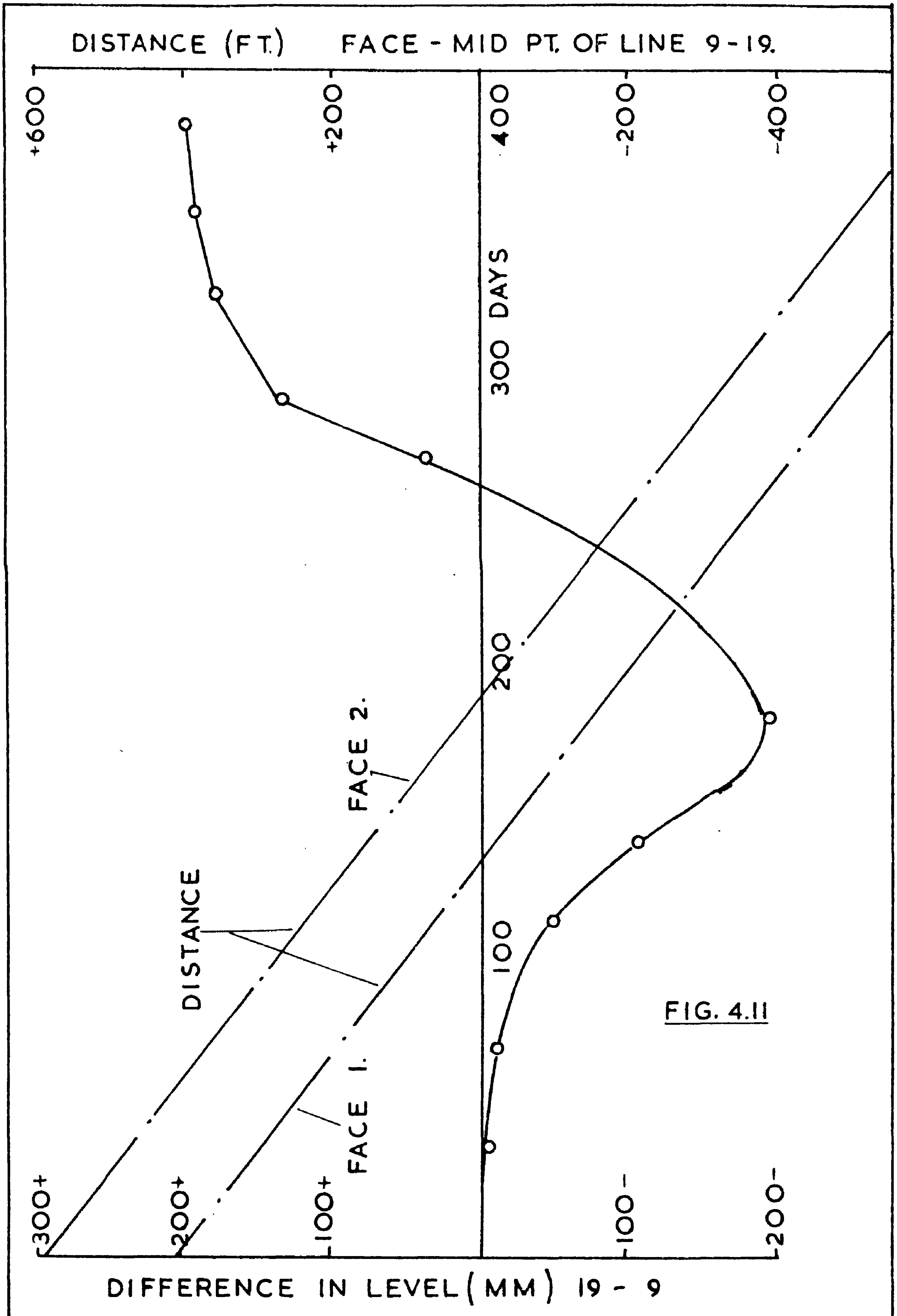

DISTANCE (FT.) FACE - MID PT. OF LINE 9-19.
+600
+200
400
-200
-400
300 DAYS
200
100
FACE 2.
FACE 1.
DISTANCE
300+
200+
100+
100-
200-
DIFFERENCE IN LEVEL (MM) 19 - 9

FIG. 4.11

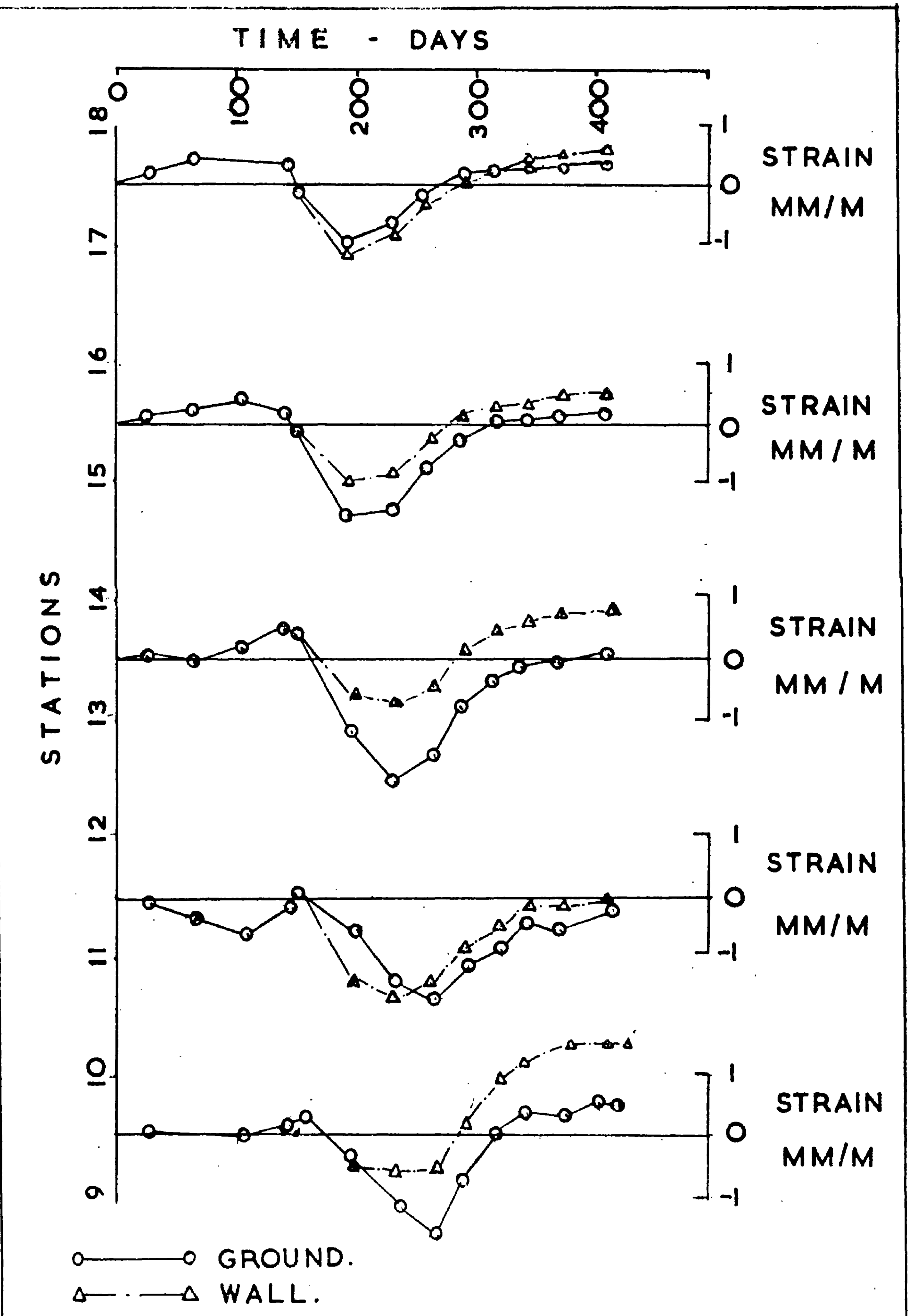

TIME - DAYS
0
100
200
300
400
STATIONS
18
17
16
15
14
13
12
11
10
9
STRAIN
MM/M
1
0
-1
GROUND.
WALL.

FIG. 4.12

between rate of change of slope and ground strain are shown by the results obtained at stations 9 to 19 (Figs. 4.11 and 4.12).

Graphs were plotted to show the development with time of horizontal movement in the ground and wall (Figs. 4.9 and 4.12). Wall measurements were commenced at a later date than ground measurements and were adjusted to allow for movement which occurred between the two starting dates.

Fig. 4.9 shows the horizontal movements at stations 2 to 7. Wall movement between stations 6 and 7 was almost equal to ground movement; between stations 4 and 5 ground movement was greater than wall movement; between stations 3 and 4 the reverse was the case. The wall movements were much nearer an average value than were the ground. On average the shortening of the wall was less than that of the ground. The displacement of the ground relative to station 7 was greater than the corresponding displacement of the wall, the difference being some 30%.

Fig. 4.12 shows the horizontal movements between stations 9 and 19. In three cases the ground movement was considerably in excess of wall movement (Bays 15-16, 13-14 and 9-10). In the two cases where ground and wall movements were nearly equal throughout the measuring period, the wall between the measuring stations was cracked before the start of the test (11-12, 17-18). In this case ground movement could be accommodated at the cracks; wall movement was therefore equal to ground movement. In the other parts of the wall, which were weathered but contained no cracks between measuring stations, compressive movement in the ground was resisted by the Wall. As a result, the recorded wall movement was considerably less than the ground movement. After maximum compression was reached further ground and wall movements were more or less equal in magnitude. This suggests that though the wall offered resistance to compressive movements it was unable to withstand extension. Such behaviour is consistent with the weathered condition of the mortar.

Fig. 4.13 shows the strain and calculated displacement of the wall containing stations 9 to 18 at the time of maximum compressive movement. It will be seen that, relative to station 18, the ground was displaced 1.5 inches further than the wall. This compares well with the observed displacement between the lower and upper part of the wall at station 9 (See Plate 4.2).

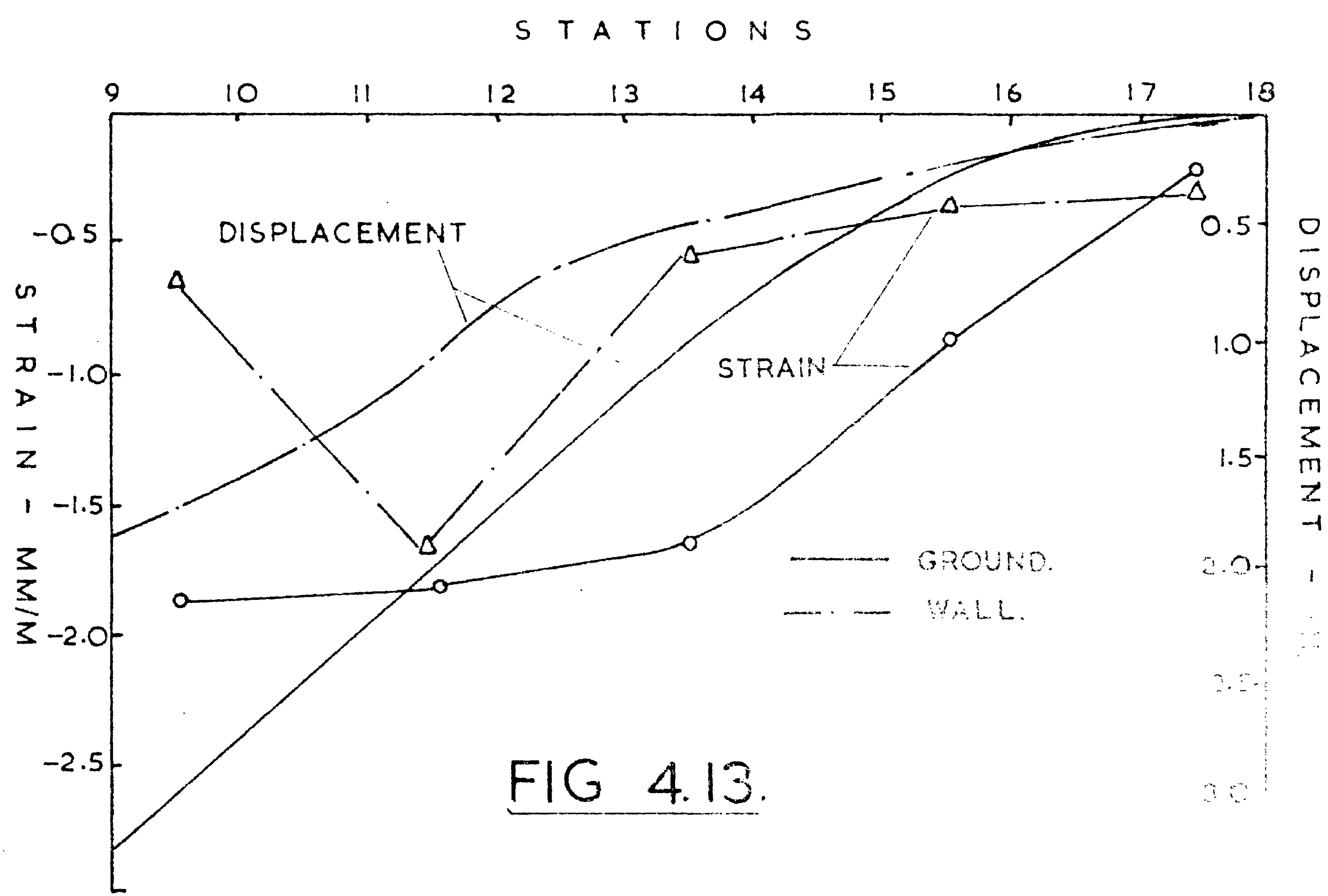
Fig. 4.14 shows the strain and calculated displacement of wall 9-18 at the end of the test. The difference in displacement in this was of the order of 1 inch.

Section 4.6 CONCLUSIONS

4.6.1. Newton Village Site.

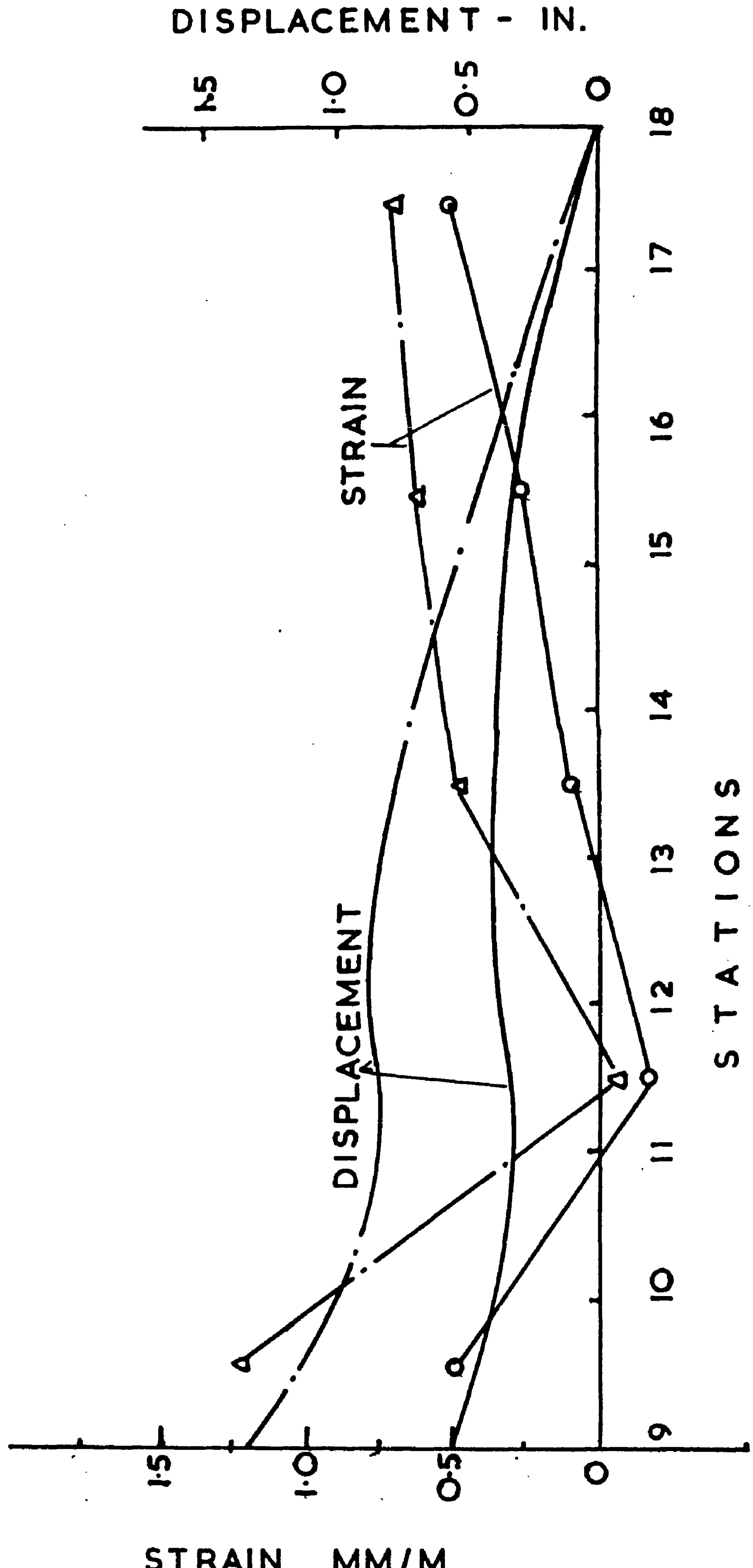
The efficiency of the stepped face layout as a means of reducing ground strain was clearly demonstrated by measurement and by the fact that relatively few houses in the village suffered damage.

The greater reduction in strain occurred where the distance between consecutive working faces was 200 ft ($\frac{1}{2}$ Depth of working).



STATIONS
9
10
11
12
13
14
15
16
17
18
-0.5
-1.0
-1.5
-2.0
-2.5
STRAIN - MM/M
0.5
1.0
1.5
2.0
DISPLACEMENT -
DISPLACEMENT
STRAIN
GROUND.
WALL.

FIG 4.13.



DISPLACEMENT - IN.
1.5
1.0
0.5
0
18
17
16
15
14
13
12
11
10
9
STRAIN
DISPLACEMENT
STATIONS
1.5
1.0
0.5
0
STRAIN MM/M

FIG 4.14

It is suggested that the faces, to be worked under Newton Church, be spaced at intervals of 200 ft.

The taping technique proved a satisfactory means of recording movements in the wall surrounding the school. In general, extensional movements of the ground and wall were of comparable magnitude. Compressional movement resulted in relative movement of the ground and the wall.

4.6.2. General.

A summary of the findings of the field investigations is given in Table 4.2.

Because of the very nature of the problem any analysis, of the factors influencing the transmission of ground movements to structures, must be made on a statistical basis. The results obtained, from the work described in the previous sections, are insufficient in themselves as a basis for any such analysis but are offered as a contribution towards the establishment of the necessary mass of data.

It is suggested that notes on the incidence of damage to structures be kept and amplified where possible by details of

a) the magnitude and direction of ground movement,

b) the magnitude and distribution of structure movement,

c) the type structure,

d) the type of foundation,

e) the type of soil.

TABLE 4.2.

Site	Type of Structure	Type of Foundation	Type of Soil	Depth of Working	Thickness of Seam	Maximum Ground Strain	Corresponding Structure Strain	% Strain Transmitted to Structure	Classification of Damage
DOUGLAS WATER	CHURCH Stone built	Strip	Sandy	550 ft	26 in.	- 3 mm/m	Not of Measurable Magnitude	-	Apprec:iable
NEWTON VILLAGE	HOUSES Brick Dimensions 70 ft x 30 ft and 55 ft x 30 ft	Strip	Clay	400 ft	36 in.	+ 2 mm/m - 3 mm/m	Not Measured	-	None to Slight
NEWTON VILLAGE	BRICK WALL	Strip	Clay	400 ft	36 in.	- 2 mm/m	- 0.7 mm/m	30% -100%	Apprec:iable

Section 5. TESTS WITH SAND MODELS

5.1. Introduction.

A series of tests were carried out with sand models in an attempt to determine if the simple soil mechanics theory discussed in Section 1.3.2 is applicable in the case of sand subject to lateral extension.

5.2. Apparatus.

The sand model used in the tests is shown in Plate 5.1. Two sheets of plate glass 'A', ¼ in. thick, are carried by the wooden end frames 'B'. The wooden section 'C' fits between the glass plates to form an open topped box of dimensions 18 in. x 12 in. x 2 in. One half of the section 'C' is fixed to an end frame 'B', the other can rotate about the hinge 'D'. Rotation is controlled by means of the adjusting screw mechanism 'E'. Self adhesive plastic foam strip was attached to all surfaces of section 'C' in contact with the glass to act as a seal. The whole is fitted to base board 'F' which is clamped to a table top during tests.

Fixed to the glass on the front of the box is a sheet of perspex scribed with a grid of lines. The grid consists of horizontal lines at 1 in. intervals and vertical lines at intervals of 0.02 feet. The grid is used in measuring the horizontal and vertical movement of sand in the apparatus when the base of the box section is rotated.

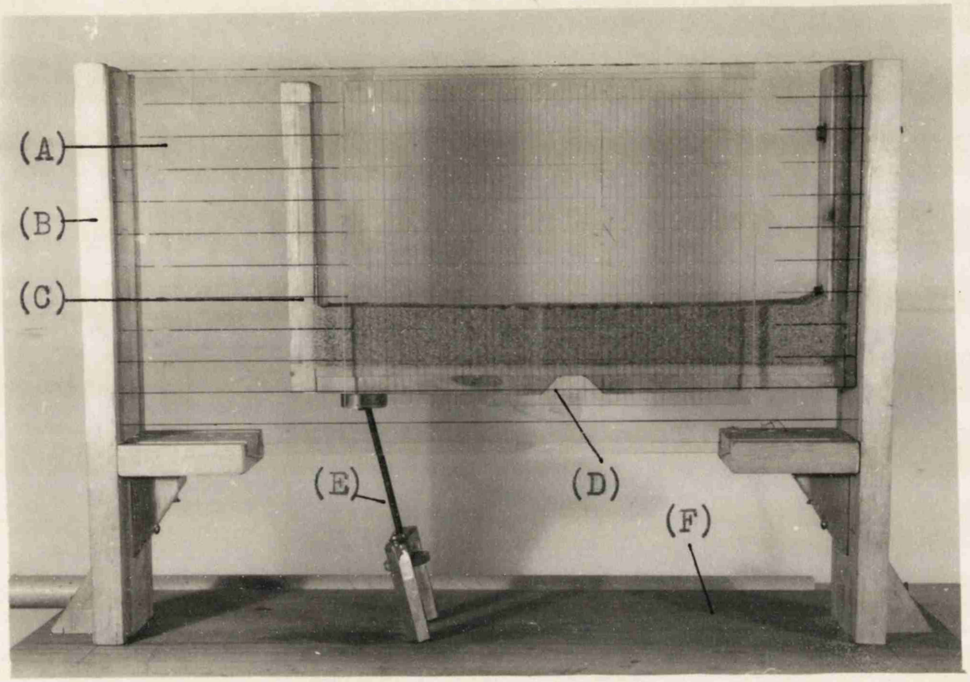


PLATE 5.1. MODEL AT START OF TEST.

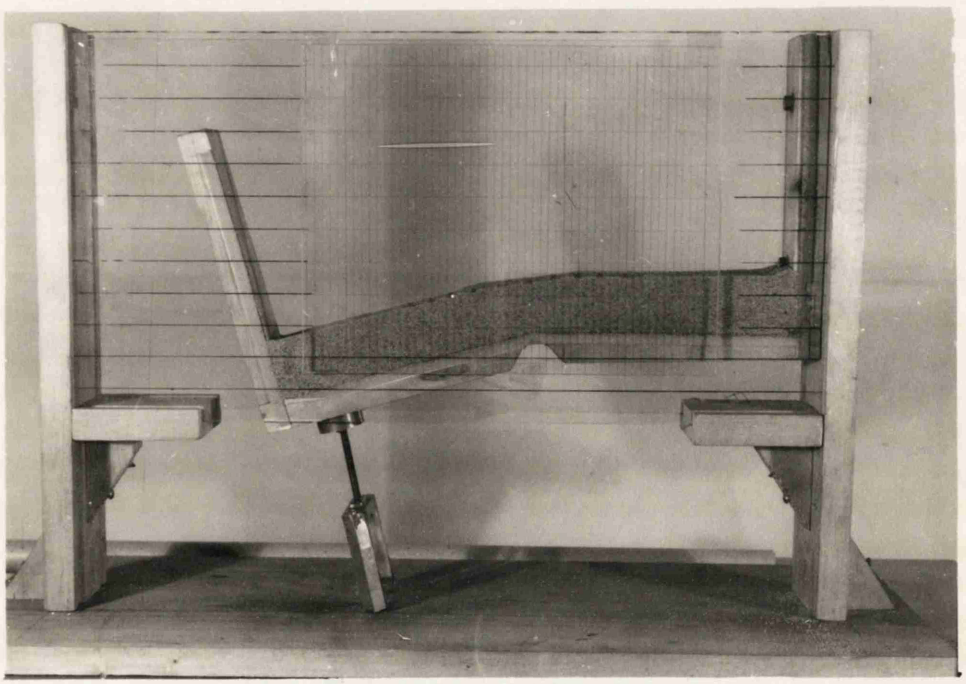


PLATE 5.2. MODEL AFTER ROTATION OF BASE.

5.3. Measurement of Sand Movement.

Reference points within the sand mass are necessary for measurement of movement in the mass. It was obvious from earlier tests with sand models [5.1] that the use of coloured sand grains as reference points was not satisfactory due to the fact that such points may become obscured when movement of the sand takes place. Reference markers were therefore constructed from small pieces of steel strip $\frac{1}{8}$ in. wide by $\frac{1}{4}$ in. long which were bent at the centre to an 'L' shape. A cross was scribed on one of the outside faces of the L. These markers were used set in the sand with the cross against the glass side of the box.

The movement of these reference points was measured by means of a Cooke, Troughton & Simms S.500 Geodetic Level, with parallel plate micrometer attachment, which was used in conjunction with the grid lines already described. The parallel plate micrometer is designed to measure differences in level and is normally used with an invar levelling staff graduated in $\frac{1}{50}$ ths ft. Rotation of the micrometer drum causes an apparent traverse of the instrument cross hairs, one rotation of the drum, equivalent to 20 drum divisions, resulting in an apparent movement of $\frac{1}{50}$ ft, the distance between graduations on the staff; one drum division is therefore equivalent to an apparent movement of the cross hairs of $\frac{1}{1000}$ ft. To make use of the parallel plate micrometer for the measurement of the horizontal movement of the reference points the level had to be rotated about its horizontal axis through 90° and held in that position. Measurements were made relative to the vertical grid lines which were spaced at intervals of $\frac{1}{50}$ ft. Movements of less than $\frac{1}{100}$ in. could be detected.

The position of a reference point relative to the horizontal grid lines was found by measuring the angle subtended at the instrument by the reference point and the grid line vertically above it, and comparing it with the angle subtended by the horizontal grid lines above and below the point. Since the distance between grid lines was known (1 in.) the distance of the point from a line could be calculated. This procedure enabled vertical movements of less than $\frac{1}{100}$ in. to be detected.

5.4. Test Programme.

Tests were carried out using a coarse grained quartz sand. The sand was of uniform grain size and was used dry. Tests on layers $1\frac{1}{2}$ in., 3 in., and 6 in. thick were carried out at two different density states - a) with maximum compaction, b) placed as loosely as possible. These two states were the only ones which could be reproduced with any degree of consistency.

5.5. Test Procedure.

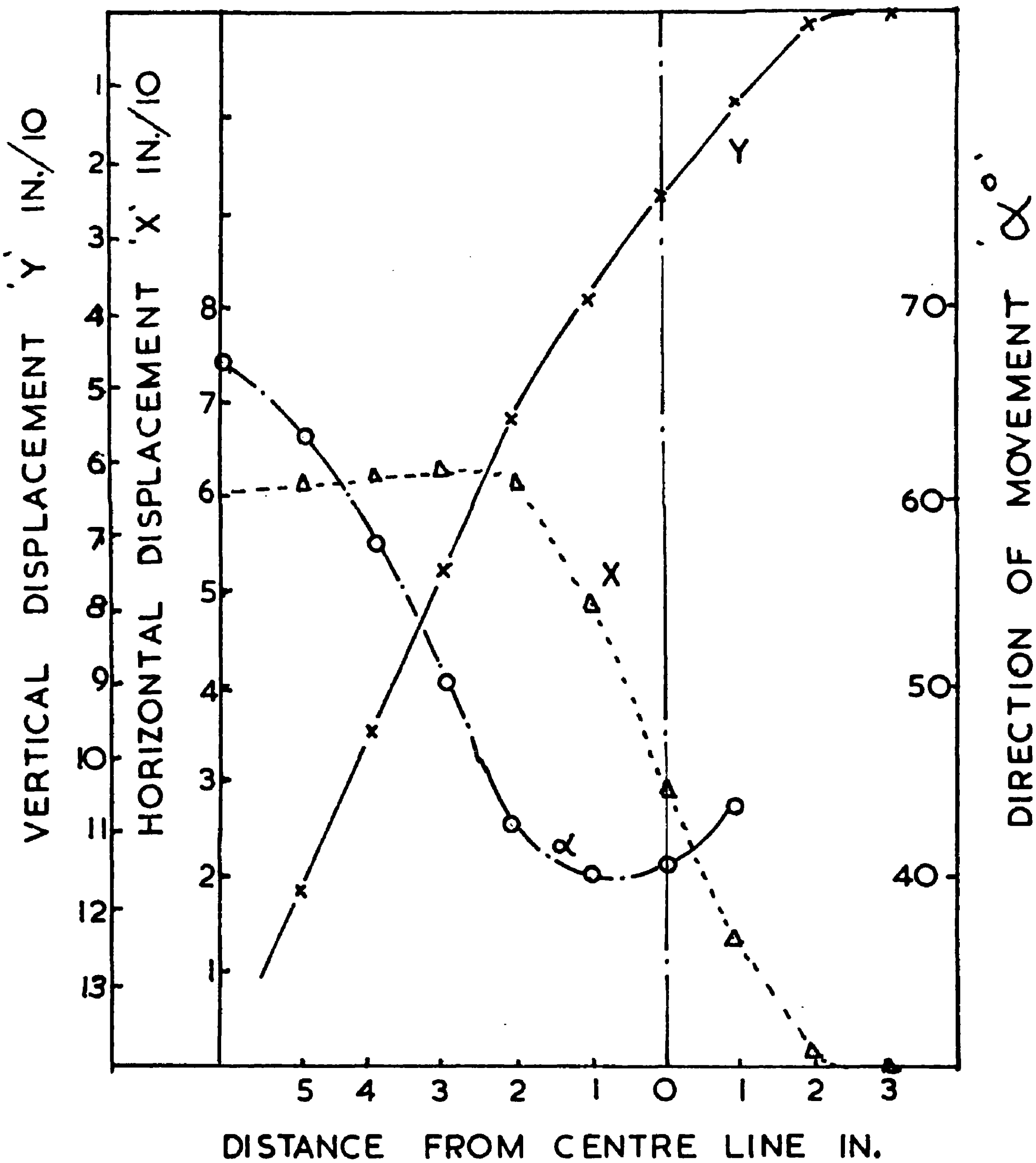
In all tests the sand box was securely clamped to a heavy wooden table after being set level and plumb. The geodetic level was set up about 10 feet from the front face of the box so that its line of sight was central with, and perpendicular to, this face.

In the case of tests with sand at maximum compaction, the sand was placed in $\frac{1}{2}$ in. layers to the required depth. Each layer was thoroughly tamped before the next layer was filled in. The metal reference points were spaced in a horizontal line at intervals of approximately 0.06 feet, i.e. opposite every third vertical grid line near the top of the final sand layer. A thin layer of sand (about $\frac{1}{8}$ in.) was placed on top to make the sand bed up to the required depth and to ensure that the reference points were imbedded in sand.

For the tests on loose sand the layer was built up by delivering sand to the bottom of the box section through a glass tube which had a small baffle plate attached at its lower end; this prevented compaction of lower layers of sand by the impact of upper layers. No tamping was used. The reference points were included as before.

Section 5.6. RESULTS

The horizontal and vertical movements measured in a typical test are shown in Fig. 5.1.; points to the left of the centre line represent the movement of sand over that part of the base which was rotated. Within a distance of approximately 2 in. on either side of the centre line lateral extension of the sand took place and was accompanied by a reduction in the depth of the sand layer. At distances greater than 2 in. to the right of the centre line no movement was recorded. Within the zone of extension the direction of movement of reference points was more or less uniform and was inclined to the horizontal at approximately



MOVEMENTS RESULTING FROM A 13° ROTATION OF BASE. – COMPACTED SAND – DEPTH 3·1 IN.

FIG 5.1

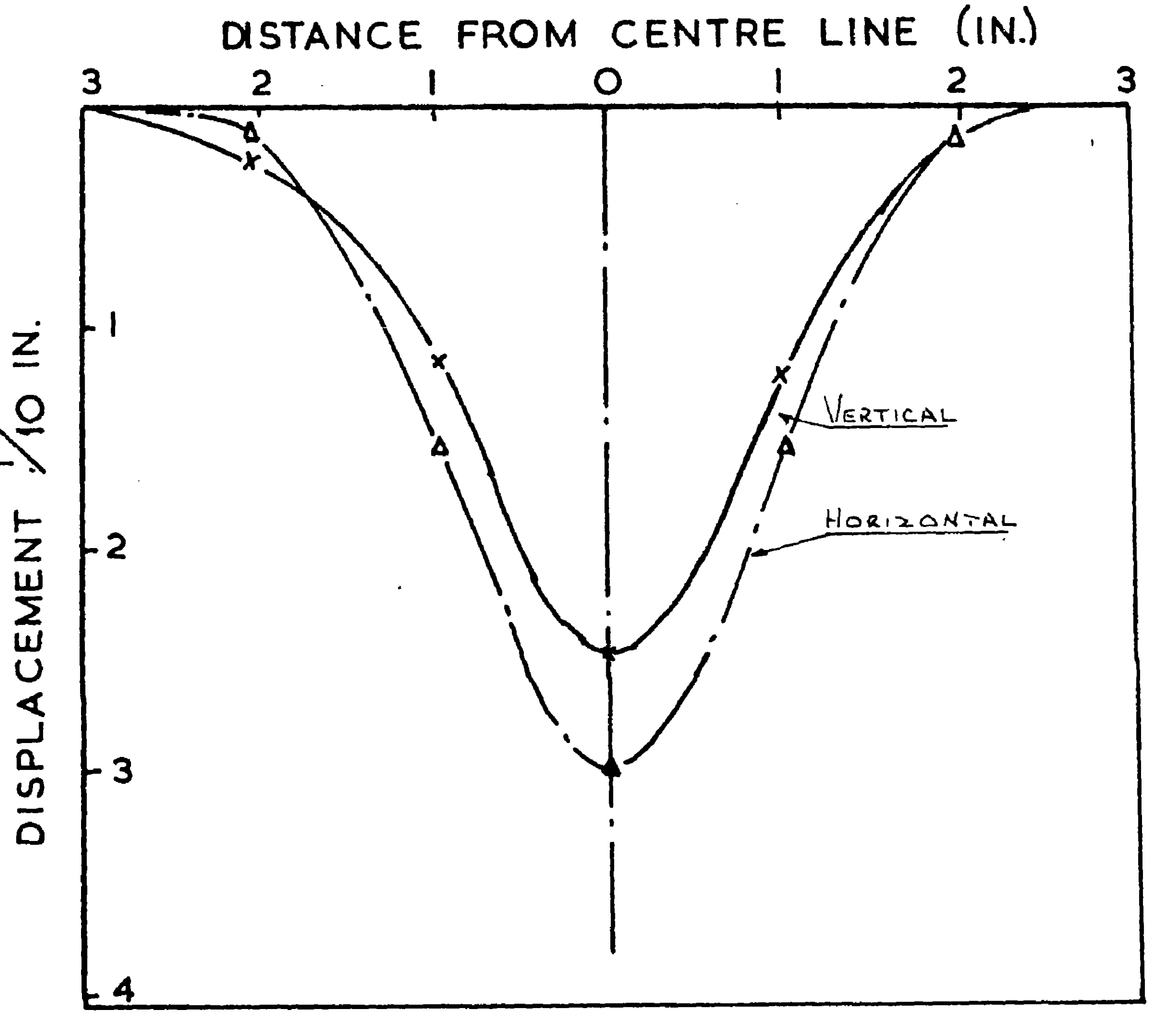
$40^o - 45^o$.

Allowance was made for movements due to rotation of the sand mass in constructing Fig. 5.2. The vertical movement of each reference point relative to the base of the model, and its horizontal movement relative to a line perpendicular to the base through the initial position of the point are shown in this figure. It will be seen that horizontal and vertical movements are uniformly distributed on either side of the centre line of the model.

The results obtained from tests carried out to determine the effect of varying the degree of rotation of the base are shown in Fig. 5.3. The relationship between rotation and maximum horizontal and vertical movement, which occurred on the centre line of the model, was found to be linear.

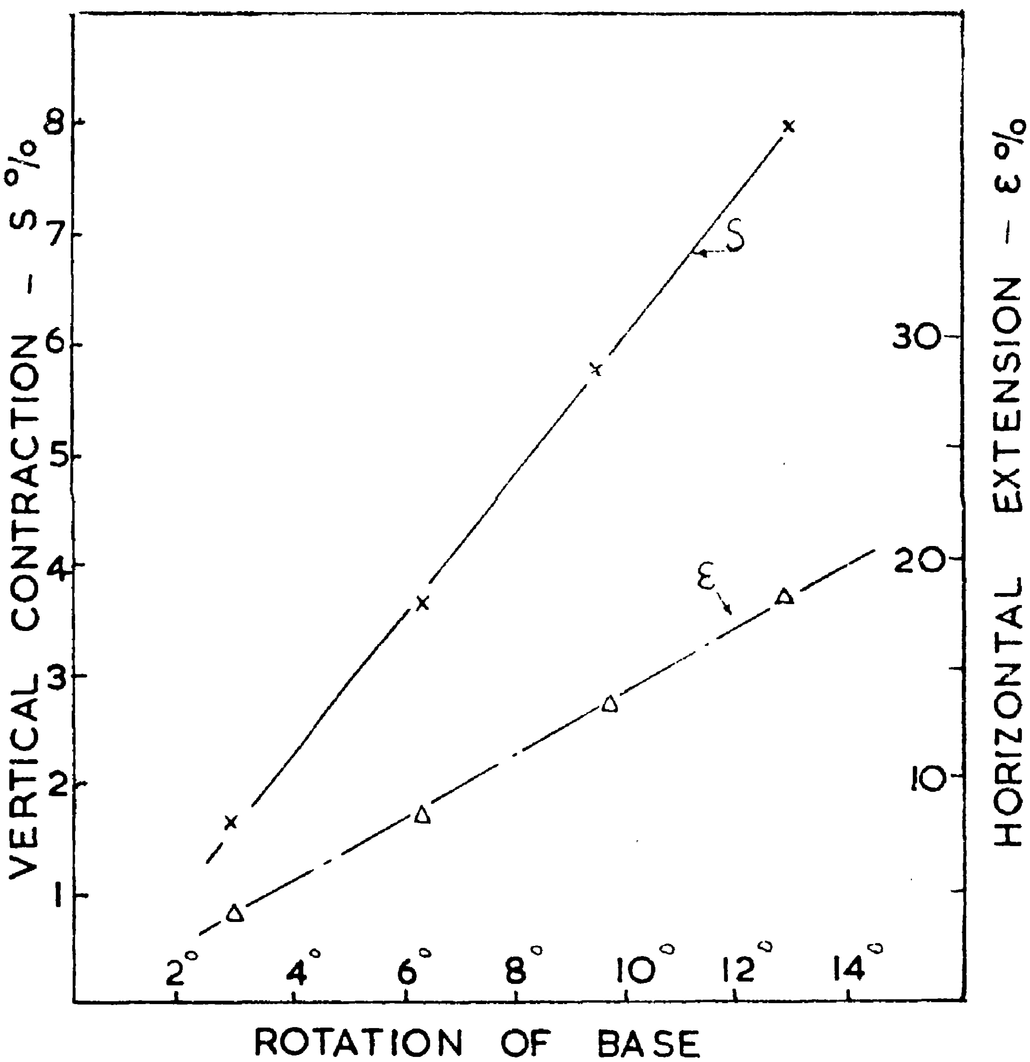
It was decided that the rotation of the base of the model in further tests be standardised at 13^o.

The variation of horizontal and vertical movement with depth and with the degree of compaction of the sand is shown in Fig. 5.4. The relationship between horizontal movement (extension) and depth is seen to be linear. The effect of the degree of compaction of the sand seems to have little effect on the magnitude of horizontal movement. This is not the case with vertical movements, which are greater in magnitude for loose than for compacted sand. There is an indication that the rate of increase of vertical movement with depth tends to diminish as the depth increases. It is suggested that this is due to arching in the upper layers of deeper sand masses.



HORIZONTAL & VERTICAL DISPLACEMENT
COMPACTED SAND — DEPTH 3·1 IN.

FIG. 5.2



VARIATION OF MOVEMENT WITH ROTATION

COMPACTED SAND — DEPTH 3·1 IN.

FIG 5.3

VARIATION OF MOVEMENT WITH DEPTH.

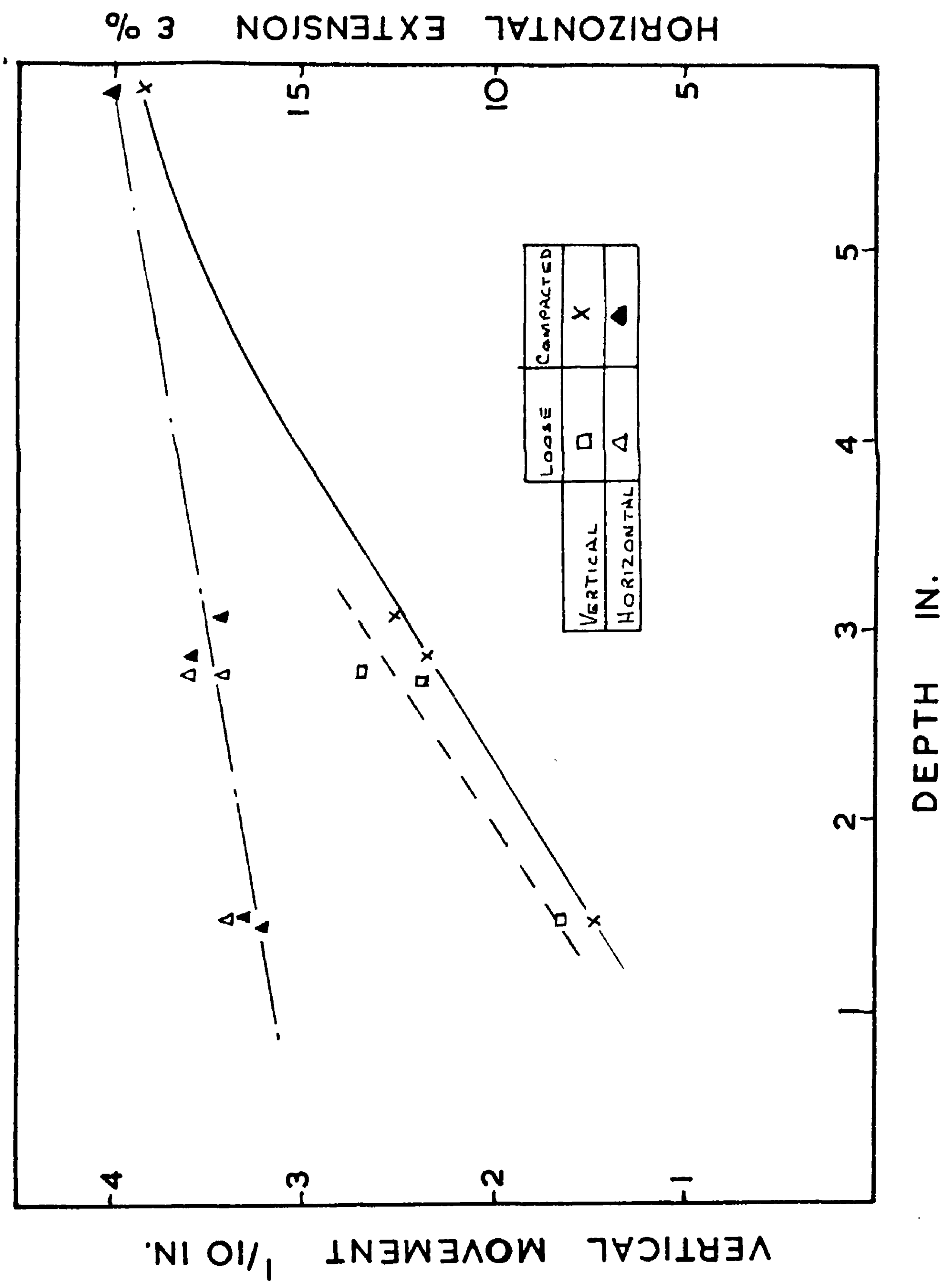
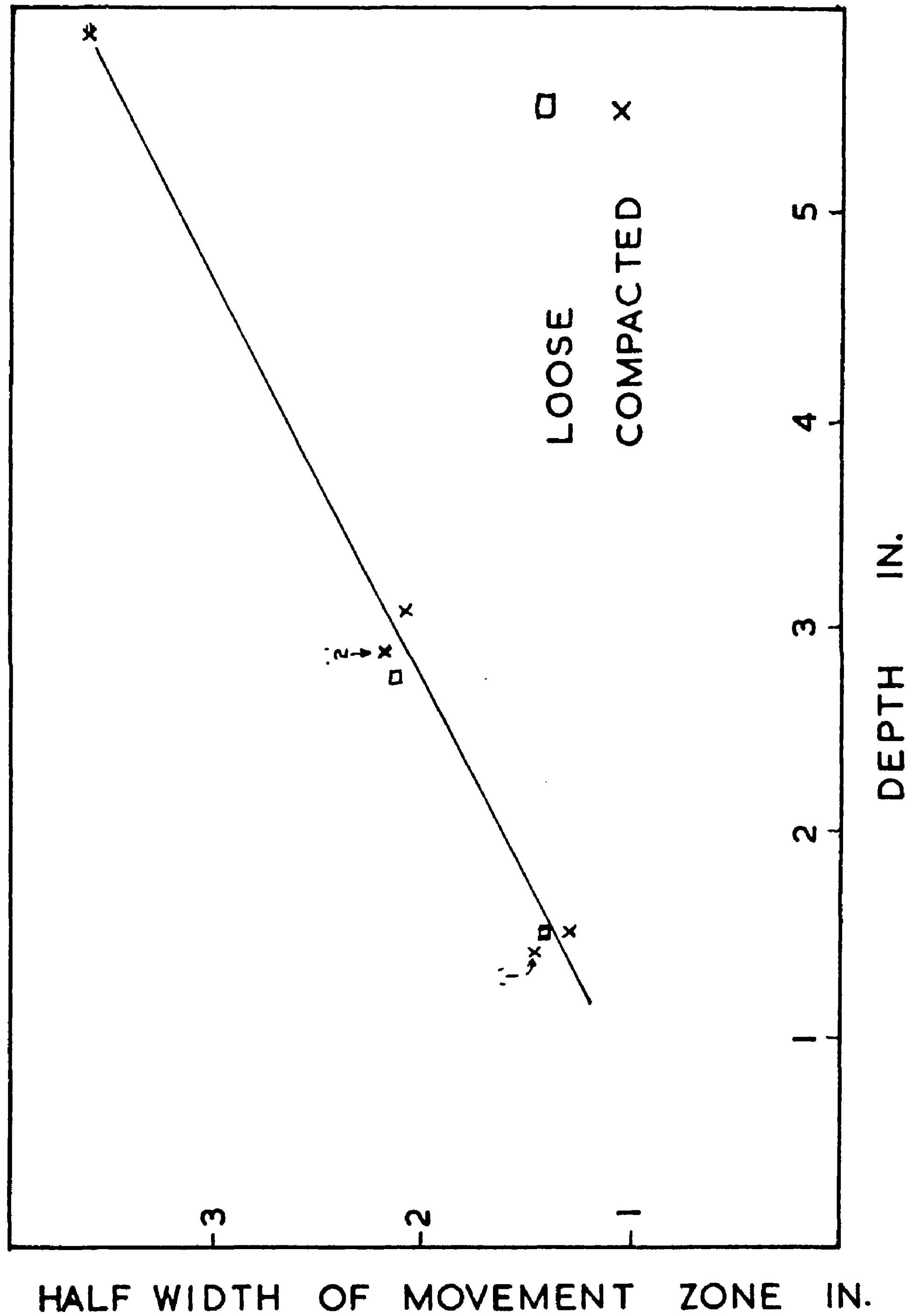


FIG 5.4

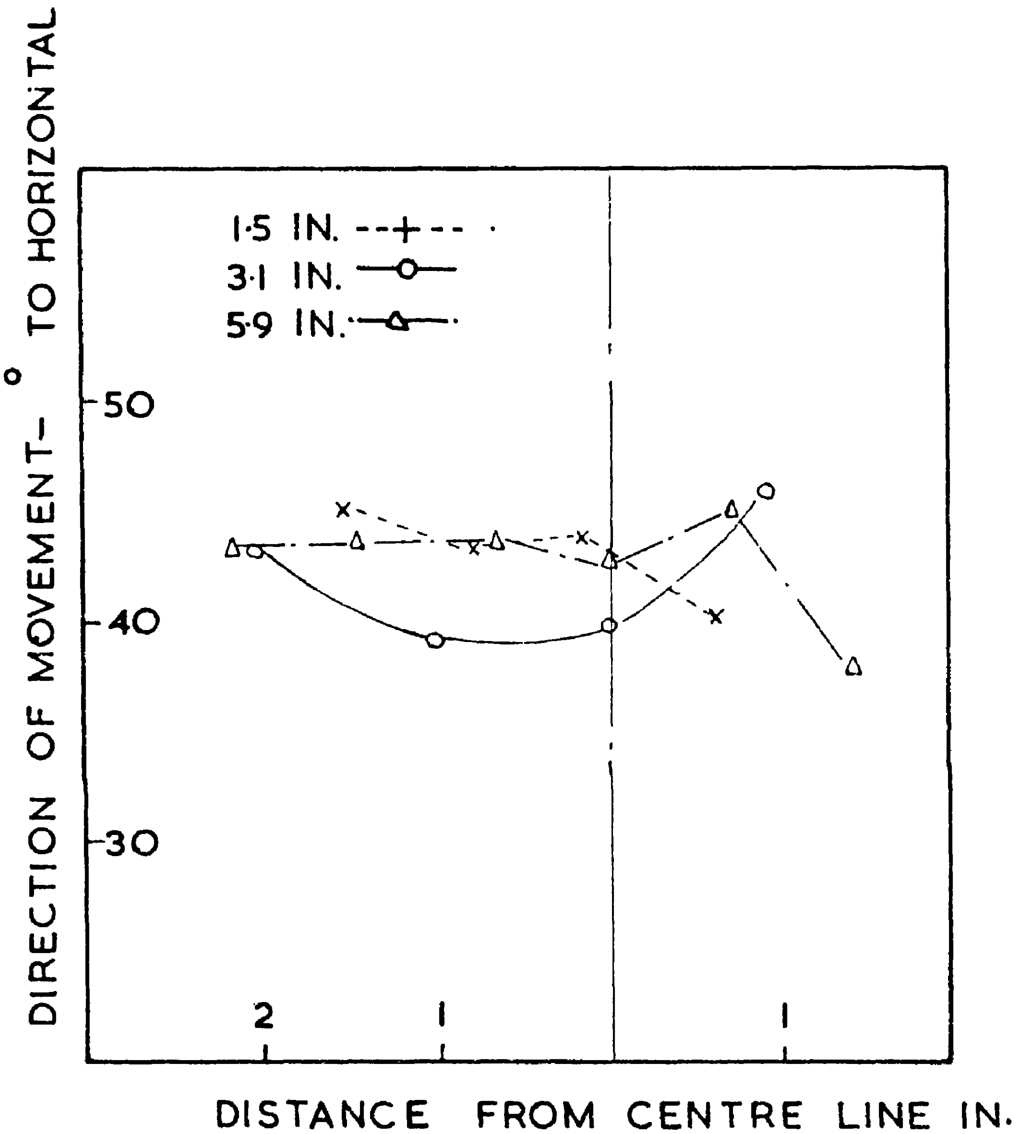
Fig. 5.5 was constructed to show the variation of the width of the zone of extension with depth of sand. It will be noticed that there is an approximately linear relationship. Points '1' and '2' relate to measurements in the same layer of sand; '2' is for movements at the surface and '1' for movements midway between the base of the model and the surface. The limit of movement '1' within the mass of sand is not markedly different from that at the surface of a sand layer at a comparable distance above the base. The curve shown in Fig. 5.5 will thus approximate to the boundary of the region within which elongation takes place, the depth axis representing the centre line.

In all tests it was noted that the measured direction of movement of the sand over the part of the base which was rotated, and outwith the zone where elongation of the sand took place, was comparable with the direction of movement that would be expected from consideration of the magnitude of the rotation of the base. (See Fig. 5.1). Within the zone of elongation, however, movement in all cases took place in a direction at 40° to 45° to the horizontal (Figs. 5.6, 5.7, and 5.8). From the results obtained there seems to be no definite relationship between the direction of movement and depth or degree of compaction of the sand. As indicated in Section 1.3.2 movement would be expected to take place at an angle of $45 - \frac{\phi}{2}$ to the vertical, where ϕ is the angle of internal friction of the material. For the material used ϕ is of the order of 35°. It is obvious, therefore, that movement in the model took place in a direction inclined to the vertical at an angle greater than would have been predicted by the application



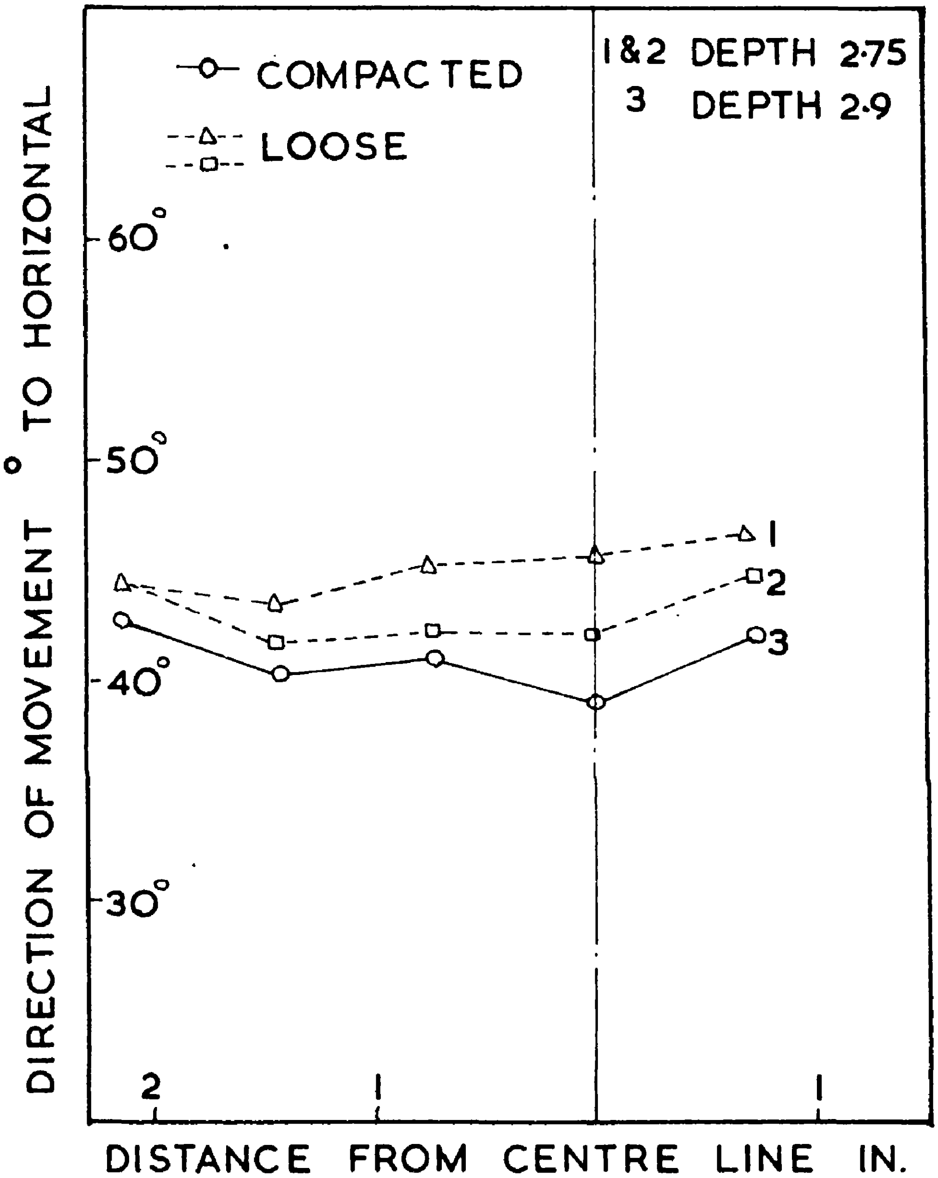
EFFECT OF DEPTH ON THE WIDTH OF THE ZONE OF MOVEMENT.

FIG 5.5



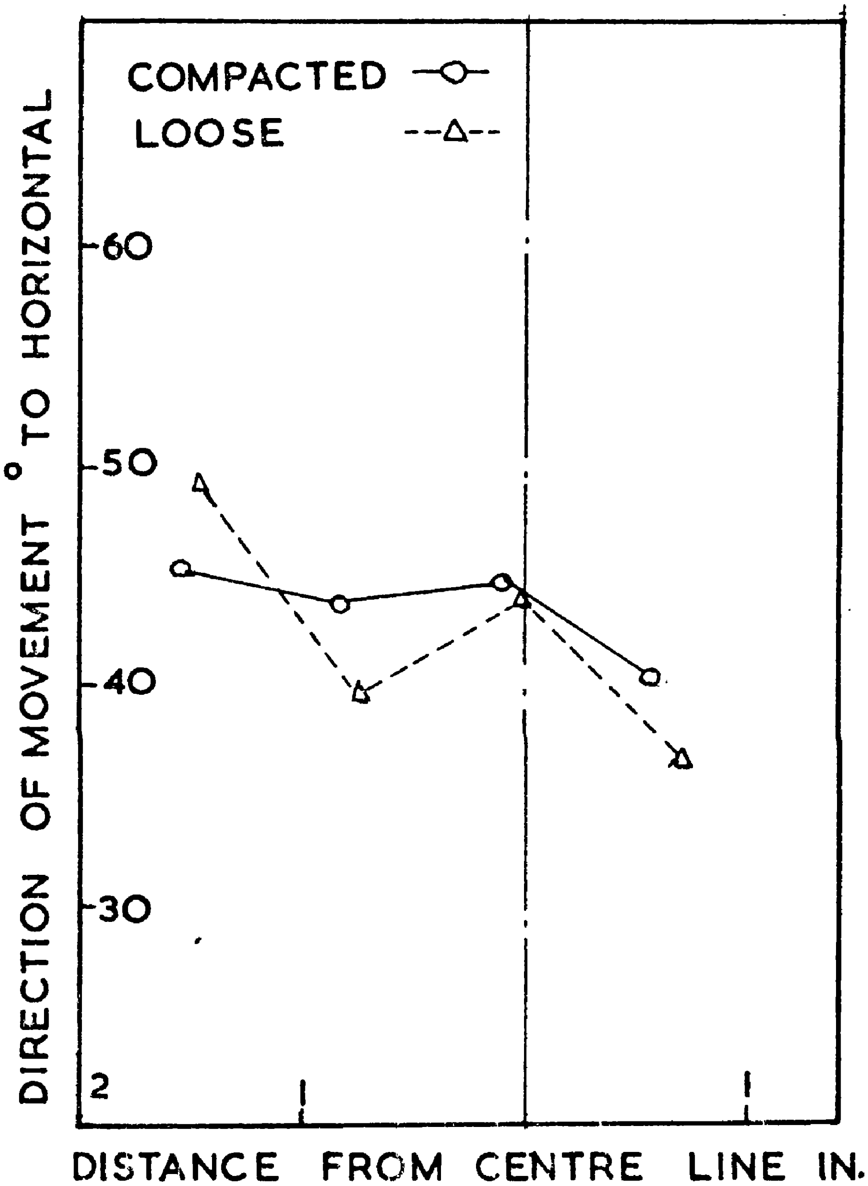
EFFECT OF DEPTH ON DIRECTION OF MOVEMENT — COMPACTED SAND

FIG 5.6.



EFFECT OF DENSITY ON DIRECTION OF MOVEMENT.

FIG. 5.7



EFFECT OF DENSITY ON DIRECTION OF MOVEMENT — DEPTH 1·5 IN.

FIG. 5.8

of the theory mentioned above. It is suggested that this discrepancy was due to the effect of arching in the sand mass which caused the magnitude of the vertical movements to be reduced. The effect of friction between the sand and glass can be neglected as horizontal and vertical movements were of comparable magnitude.

Section 5.7. CONCLUSIONS

The results obtained suggest that the simple soil mechanics theory is not applicable to the prediction of the movement pattern in a sand mass subject to extension under the conditions occurring in the model; this is due to the effect of arching in the soil. As the narrowness of the zone of extension may, in some measure, have contributed to the setting up of the arching effect, further tests, in which extension is induced over a wider region, are necessary.

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SECTION 3

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A P P E N D I X   I

The Measurement of Strain in Ground and Structures

Measurement of Ground Movement

The majority of workers in subsidence research have employed taping techniques for the measurement of horizontal ground movement [A.1.1], some have used tacheometric techniques [A.1.2] and more recently an invar measuring rod with dial gauge attachment has been used [A.1.3]. Stations are generally installed in lines parallel or perpendicular to the direction of face advance. To obtain more information about the magnitude and direction of horizontal ground movements, investigators at Leeds University [A.1.4] have made use of stations set in a triangular pattern.

It is claimed [A.1.5] that using a steel tape with a standard measuring procedure the measurement of the distance between two stations can be repeated to give results within 1 m.m. of each other. This measuring procedure has been used in obtaining a large amount of data relating to horizontal ground movements, and has proved invaluable in the study of ground movement phenomena. However, in using such a technique there is a liability to include an appreciable error in the application of a temperature correction due to difficulty in determining accurately the true temperature of the tape. Experiments [A.1.6] have shown that the length of a steel tape, when suspended in sunlight in a light breeze, can fluctuate rapidly over a range equivalent to a temperature change of 18° F. To minimise error arising from an inability to measure the tape temperature accurately, it is advisable to use a tape made from a material with a low coefficient of thermal expansion. (e.g. Invar).

It has been customary in the past to make measurements with a tape stretched along the ground under standard tension. Such a procedure is liable to result in error if the tape is deflected by obstacles in its way, and therefore precludes the use of such a technique on grass-land. Because of this, it is considered that measurement in catenary is a more satisfactory procedure. However, it is essential when measuring in catenary to have the measuring points above ground level, or to provide some means of extension on stations set below ground level. When extensions are used it is essential that they be kept as short as possible to minimise error.

For a given error in reading the ends of the tape, the fractional error of the measuring technique will be less for a long than for a short tape. On the other hand, long tapes are more liable to be affected by wind and by snagging on obstacles, when suspended in catenary close to the ground. Also where the magnitude of the horizontal ground movement changes rapidly with distance it is desirable to keep the stations as close as possible to obtain a good picture of the variation of movement. This is most important when measuring over shallow seams and in such cases would be the decisive factor in choosing the station spacing.

On balance it is felt that the advantage lies in using a fairly short station spacing, this being reflected in the author's decision to use a 25 ft invar tape and to concentrate on the development of a measuring technique designed to minimise the error in reading the end of the tape.

Horizontal Movement of a Structure

A structure subject to ground movement will in the initial stages be able to accommodate the movement by elastic deformation. However, a stage will be reached where further movement will cause cracking. The cracks will open and develop with further increase in ground movement.

During the initial elastic phase of structural movement several methods of measurement may be applied, including mechanical strain gauges, electrical resistance strain gauges and acoustic, or vibrating wire, strain gauges [A.1.7].

The magnitude of movement which occurs in a structure will vary from section to section but, in general, will be distributed in a more or less regular pattern. The distribution of movement at a particular section in a given structure will be governed by the homogeneity of the material used in its construction. Thus the distribution of movement in the steel frame of a building will be more uniform than the distribution of movement in the brickwork of the same building. Thus while a short gauge length device, such as an acoustic gauge, may give useful results when used on the steel frame work of a building they would be quite unsuitable for use on the brickwork. A measuring device with gauge length several times that of the bricks should be used to offset the effect of individual bricks. Further, when cracking of the wall occurs, movement tends to be concentrated at the cracks so that an essential requirement of a measuring device for use on brickwork is an ability to accommodate such concentrations of movement without sustaining damage in the process. As taping fulfills the foregoing requirements, wall movements were measured using a technique essentially similar to that used in the measurement of ground movement.

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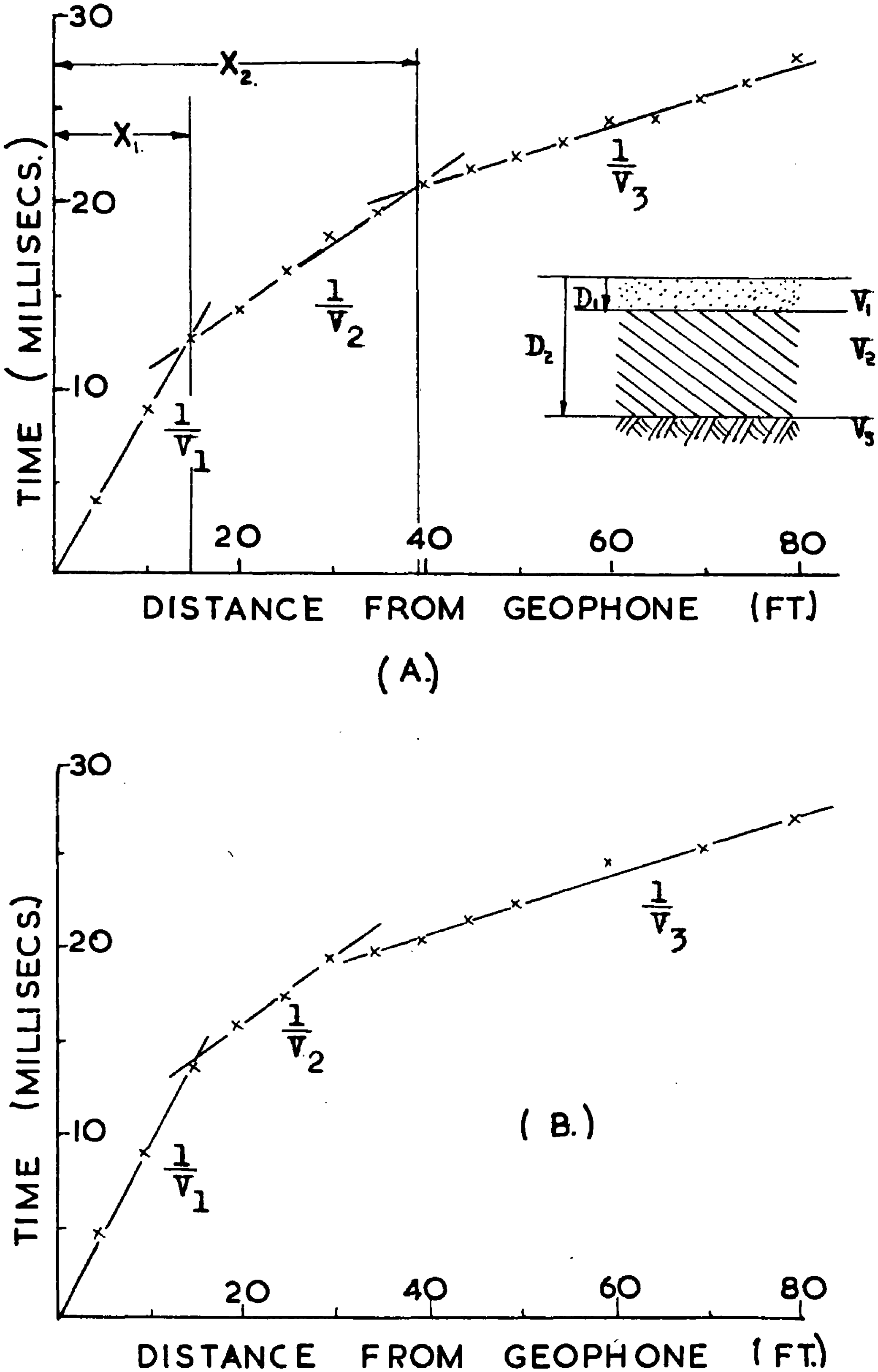
APPENDIX 2

The Determination of the Thickness of Unconsolidated Material at Newton Village

The technique of seismic refraction has been widely used for the exploration of sub-surface geological formations [A.2.1]. Generally the method used in the mapping of deep formations is the measurement of the time taken by shock waves, generated at the 'shot point' by the detonation of an explosive charge, to reach each of a series of geophones set at varying distances from the shot point. In this case a single geophone was used with multiple 'shot points'. Shock waves were produced by striking a steel plate with a heavy hammer; the hammer blow actuated a switch which set an electronic timing device in operation; the timing device was switched off by the first arrival of the shock waves at the geophone. At the Newton Village site a series of shot points, at 5 ft intervals, were used on two lines at right angles to each other. The results obtained are shown in Figs. A.2.1, A and B. The results shown relate to a three layer problem. If the base of the first layer is at a depth of D_1 ft and of the second at D_2 ft, and if the velocity of sound in the layers is V_1, V_2 and V_3 ft/sec. as shown in Fig. A.2.1, then it can be shown that

$$D_1 = \frac{X_1}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$



30
20
10
TIME (MILLISECS.)
X_2
X_1
$\frac{1}{V_1}$
$\frac{1}{V_2}$
$\frac{1}{V_3}$
D_1
D_2
V_1
V_2
V_3
20
40
60
80
DISTANCE FROM GEOPHONE (FT)
(A.)
30
20
10
TIME (MILLISECS.)
$\frac{1}{V_1}$
$\frac{1}{V_2}$
$\frac{1}{V_3}$
(B.)
20
40
60
80
DISTANCE FROM GEOPHONE (FT.)

FIG. A2.1.

$$D_2 = D_1 + \sqrt{\frac{V_3 - V_2}{V_3 + V_2}}$$

Using the values of V_1, V_2, V_3, X_1 and X_2 found from the graphs, values of D_1 and D_2 were calculated for each of the lines.

Line	V_1 ft/sec.	V_2 ft/sec.	V_3 ft/sec.	D_1 ft	D_2 ft
A	1176	2777	6110	4.6	15.1
B	1132	2778	6000	5.2	13.7

From the values of V_1 and V_2 obtained it is obvious that layers 1 and 2 comprise unconsolidated sediments, probably boulder clay [A.2.2]. The presence of boulder clay was confirmed by exposures in an adjacent ploughed field.

APPENDIX 3

Photo-Elastic Experiments

A.3.1. Introduction

It was shown in some of the field tests (Section 3.7) that particular structural features contributed to the nature and magnitude of the damage caused by ground movement. It was felt that photo-elastic analysis was a suitable means to study the effect of the main structural features, e.g. doors, windows, floors, roof etc., on the distribution of stress in a structure subjected to ground movement.

Damage resulting from differential vertical ground movement is due to deformation of the structure under the action of gravity. If this effect has to be studied by photo-elastic means a material is required which will deform, and show stress fringes, under gravity loading. Such a material is gelatin which has been used in the past [A.3.1 to A.3.7] for studies of the deformation, under gravity loading, of structures and earth works.

Gelatin models are only suitable for the study of problems in two dimensions. Three dimensional models are desirable, however, in studying the effect of horizontal ground movements on buildings. This is particularly the case in considering the effect of interaction of walls, floors, etc., and of the orientation of the structure to the direction of the maximum ground movement, on the distribution of stress in the structure.

The photo-elastic examination of three dimensional models is best accomplished by the use of the stress freezing technique [A.3.8 and A.3.9]. Materials for use in stress freezing (e.g. 'Araldite B' resin) are not sufficiently sensitive for use where gravity loads only are acting. A compromise has, therefore, to be made, the effect of gravity loading being studied using two dimensional gelatin models, while three dimensional studies of composite models are made using the stress freezing technique.

Preliminary test with gelatin and 'Araldite B' are described in the ensuing sections.

A.3.2. The Photo-Elastic Effect.

For normal incidence on a flat plate of photo-elastic material subjected to plane stress the transmission of light is determined by the following laws -

1) The light is polarized in the directions of the principal stress axes and is transmitted only on the planes of principal stress.
2) The phase difference between the two light components is proportional to the difference between the principal stresses.

This relative retardation depends also on model thickness and the sensitivity of the material to the photo-elastic effect

$$R = C t (P - Q) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (A.3.2.1)$$

where R = relative retardation in inches.

C = constant for material (Stress optical coefficient)

t = thickness of model in inches

P and Q = principal stresses (lb/in^2)

The relative retardation between the two phases can be studied in a polariscope where it is measured in terms of the wave length of light by counting the fringes produced by optical interference. If the order of the interference fringe produced is N then

$$N\lambda = R$$

where λ = wave length of light

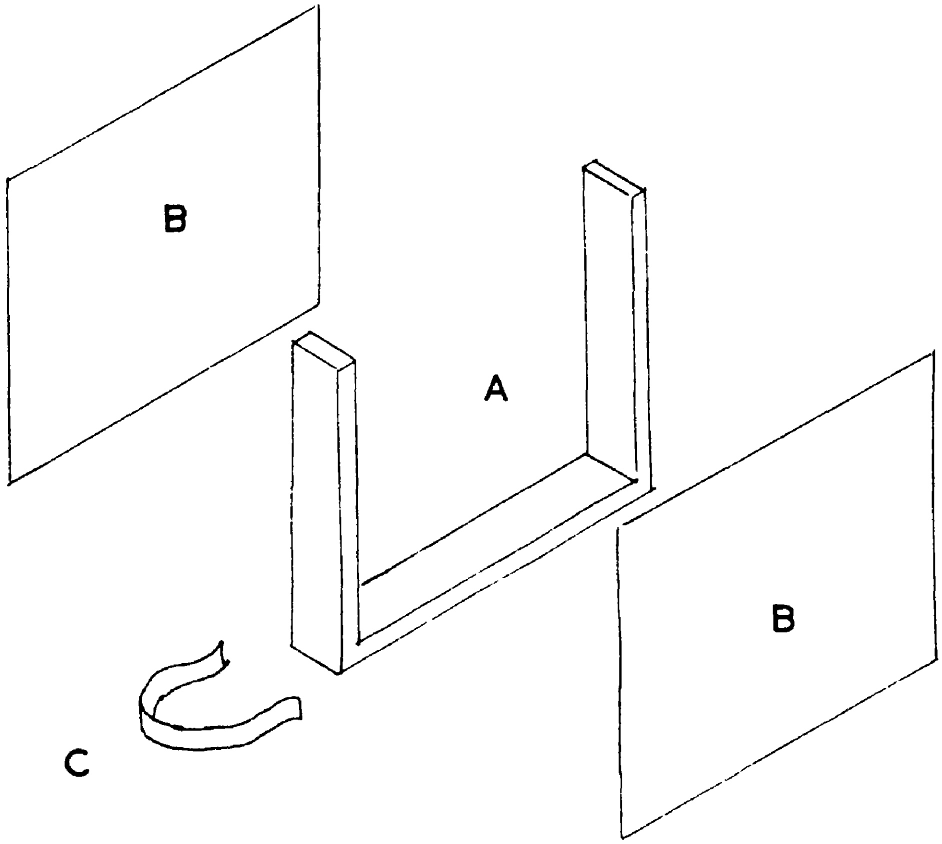
and $$F = \frac{(P - Q)t}{N} \qquad \text{.................} \quad (A.3.2.2)$$

where $F = \dfrac{\lambda}{C}$ = fringe stress coefficient

A.3.3. Gelatin Models.

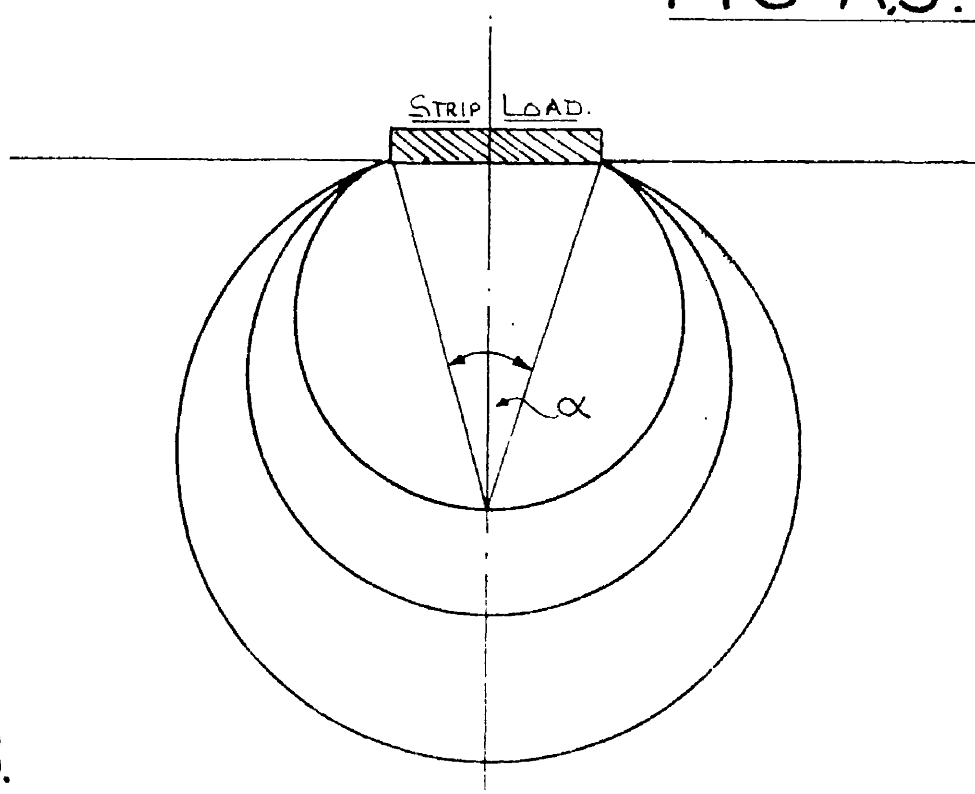
A series of tests was carried out to develop a suitable casting technique for the production of models and to study the effect of variation in gelatin concentration on its optical and photo-elastic properties.

A.3.3.1. Moulds were constructed as shown in Fig. A.3.1. The dimensions of the mould were determined by the size of polariscope available. The edges of the 'perspex' frame 'A' were machined to ensure a good fit when the glass plates 'B' were fitted. Clips 'C'



A.

FIG A.3.1



B.

were used to hold the assembly and water-tightness was assured by sealing the joints of the mould with molten paraffin wax. This method of sealing proved satisfactory provided the temperature of the gelatin solution, when poured, was not above 45° C.

A.3.3.2. Preparation of the Solution and Casting Technique.

The required quantity of gelatin powder was weighed into a beaker. Approximately 20% of the volume of water necessary to make a solution of the required concentration was added to the powdered gelatin and the mixture left to soak. Water at 90° C was added in sufficient volume to make a solution of the required strength. The mixture was stirred until all the gelatin had dissolved, then allowed to stand to let entrained bubbles of air come to the surface. These bubbles formed a scum on the surface of the solution which was carefully removed before pouring. The solution was allowed to cool to about 45° C and then siphoned into the mould to the required depth.

The mould had previously been prepared as follows. The glass plates were washed in a detergent solution, dried and polished using a clean dry duster; the plates were fitted to the 'perspex' frame by means of clips and the mould sealed by the application of molten paraffin wax to the outside joints; the mould was then tested for leaks by filling with water which was poured out just before the gelatin solution was siphoned.

Some 24 hours after casting the glass plates were carefully removed. If the cast tended to stick to a glass plate, the plate was warmed on the outside by the application of a cloth dipped in warm water. The plates were wetted and replaced.

A.3.3.3. The Testing Procedure was designed to study the effect of variation in gelatin concentration on

a) the optical density of the cast

and b) the photo-elastic properties of the cast.

An S.E.I. photometer was used to measure the brightness of an illuminated matt surface (clean white blotting paper) viewed through the mould empty and then containing the gelatin solution. By comparing these two results a measure of the optical density of the gelatin cast was obtained.

To determine the fringe stress coefficient the procedure described by Farquharson and Hennes [A.3.1] was used. A strip of metal of width 0.5 in. which was a sliding fit between the glass plates was placed on the surface of the gelatin. The strip was loaded by means of a sliding rod topped with a load pan. The model was examined in a crossed circular polariscope used with monochromatic light. Fig. A.3.1.B shows a typical fringe pattern obtained. If the pressure exerted on the gelatin by the strip is 'B' lb/in^2 then the maximum shearing resistance, '$\tau_{max.}$', at a point vertically below the strip is

$$\tau_{max.} = \frac{B \sin \alpha}{\pi} \qquad \ldots\ldots\ldots\ldots\ldots \text{(A.3.2.3)}$$

where α = angle subtended by the strip at the point whence $F = \dfrac{2tB \sin\alpha}{\pi N.}$

where N = fringe order at the point.

A.3.3.4. The Results obtained are summarised on Table A.3.1.

It was found that solutions containing 40% by weight of gelatin powder were very opaque due to the presence of trapped air bubbles which could not rise to the surface due to the high viscosity. Solutions at this strength stuck firmly to the glass plates which could not be removed without damage to the cast.

Solutions containing 5% and 10% by weight of gelatin powder proved unsuitable due to difficulty in removal of the glass plates. This resulted from the relatively low mechanical strength of these gels and from the fact that they stuck tenaciously, at their free upper surface, to the glass plates.

The glass plates were easily removed from gel casts containing 20% and 30% by weight of gelatin. There was little difference in the fringe stress coefficient in the two cases.

A.3.3.3. Conclusions.

Due to the low percentage of light transmitted through the 30% strength gel, it is considered that the 20% gel is the more suitable and it is suggested that this strength be used for further tests.

T A B L E A.3.1.

Gel Strength	%age Light Transmitted	Fringe Stress Coef. lb/in. Fringe	Remarks
10%	30	-	Plates could not be removed from Mould
20%	20	0.33	
30%	8	0.30	

SUMMARY OF RESULTS - GELATIN CASTS

A.3.4. Tests with 'Araldite B'.

'Araldite B' is a synthetic resin suitable for photo-elastic work using the stress freezing technique.

Tests were carried out to develop suitable techniques for

a) the casting of 'Araldite B' into sheets,

b) the cutting and preparation of models from 'Araldite B' sheets,

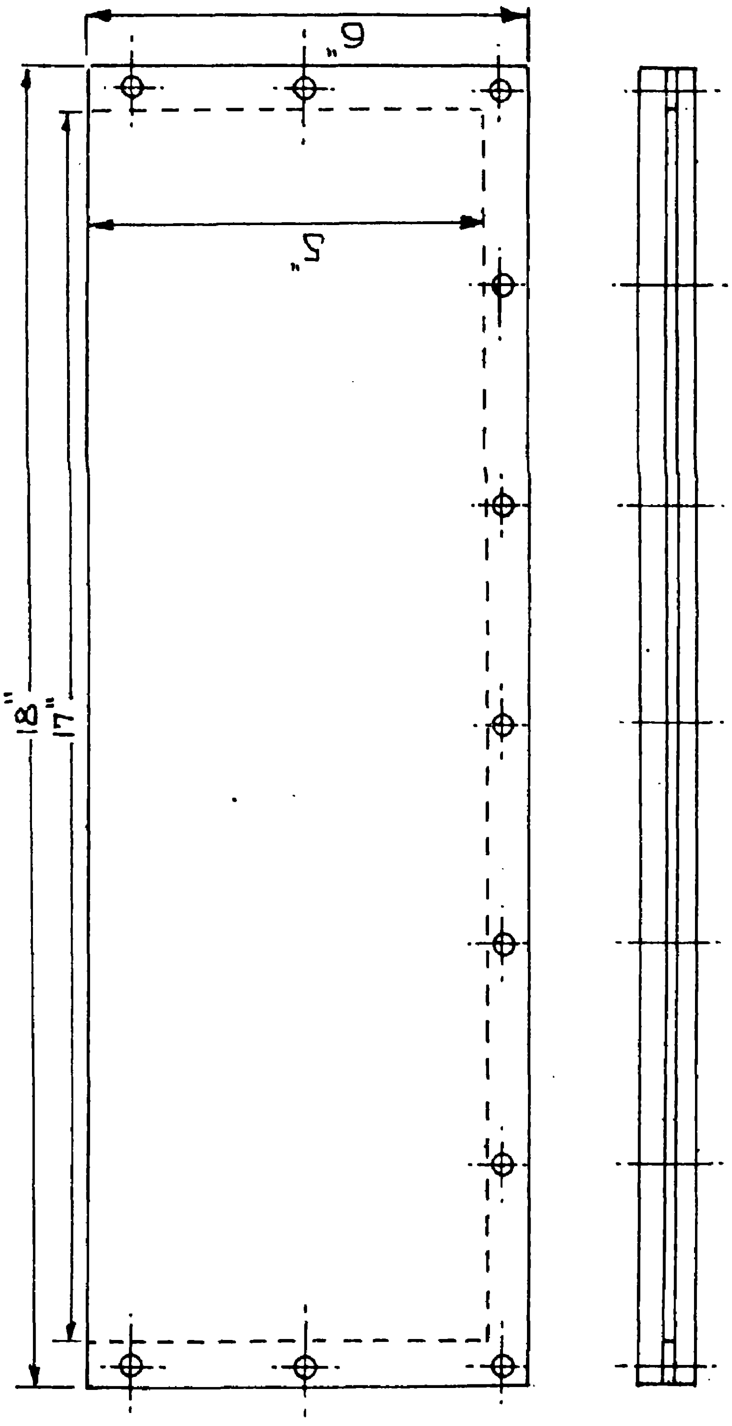
c) joining 'Araldite B' sheets,

as a preliminary to the construction and testing of three dimensional models.

A.3.4.1. The Moulds used for the preparation of 'Araldite B' in sheet form were made to the design shown in Fig. A.3.2. The mould is constructed from ground flat steel gauge strip. The thickness of sheet produced was varied by using spacing frames of different thickness. The inside surface of each side plate was polished using successively finer grades of emery cloth and finally metal polish.

A.3.4.2. Curing Oven.

The construction and operation of the oven have been described by Alan [A.3.9]. The oven is electrically heated and the temperature can be controlled automatically by means of a Kelvin-Hughes Controller at a fixed value or in accordance with a pre-set time temperature cycle.



MOULD FOR CASTING ARALDITE SHEETS

FIG. A3.II

A.3.4.3. Casting Technique.

To prevent the 'Araldite B' sticking to the steel, the inside surfaces of the mould were coated with 'Releasil 14' silicone grease. The mould was assembled, using paper gaskets smeared with silicone grease, placed in the oven and clamped in a vertical position; it was allowed to heat up with the oven to a temperature of 280° C.

Requisite quantities of 'Araldite B' and 'Araldite' Hardener 901 (30 parts Hardener to 100 parts 'Araldite B') were weighed into separate beakers and placed in the oven to melt. When melted the Hardener was poured into the 'Araldite B' and the mixture thoroughly stirred; the mixture was then poured into the mould. This was achieved, without the removal of the mould or the beakers from the oven, by the use of wooden tongs to handle the beakers.

The temperature was allowed to fall to 220° C and maintained at that value for 24 hours to cure the resin. The time-temperature cycle mechanism was then switched on to cool the oven slowly to room temperature over a period of 24 hours. When cold the mould was dismantled and the 'Araldite' sheet removed.

In initial trials the 'Releasil 14' was applied to the inside surfaces of the mould, dissolved in carbon tetrachloride. Araldite sheets could easily be removed from moulds prepared in this way but several of the sheets obtained were useless due to 'locked-in' stress fringes. It is thought that these were caused by areas of the Araldite sheet adhering to the mould in the initial stages of setting.

A more effective lubricating procedure was evolved to prevent the occurrence of these 'locked-in' stresses. The 'Releasil 14' was applied to the surfaces of the side plates as before; the side plates were placed in the oven and maintained at a temperature of 280° C for approximately 1 hour; the plates were then removed, allowed to cool and polished with a clean dry duster before the mould was built up. This procedure proved effective and enabled sheets to be produced which were stress free, except in limited areas at the edges of the sheet in contact with the mould.

Attempts to remove the 'locked-in' stresses by annealing proved useless; the stresses were reduced in intensity but spread over greater areas of the sheet.

The sheets produced were of uniform thickness; the surfaces were smooth but not highly polished.

A.3.4.4. Preparation of Models.

Two techniques were used for the preparation of models.

The first procedure was to cut models accurately to size by means of a pantograph engraving machine. A sharp tool is essential. Light cuts and a slow steady feed were used. Care was necessary at corners to prevent machinery stresses, which occurred if the tool was allowed to 'dwell'.

Alternatively the model could be cut $\frac{1}{4}$ in. oversize using a hacksaw and finished to size in a milling machine. An end-milling cutter was used; light cuts and a slow steady feed are essential.

The first technique is the more generally applicable of the two.

It can be used to machine any shape by the use of a suitable templet. Sharper internal corners can be machined due to the small diameter of the cutter (0.1 in.).

A.3.4.5. Testing Procedure.

Strips of Araldite were tested in pure bending and examined in a crossed circular polariscope to determine the fringe stress coefficient. Fringe stress coefficients varying from 52 lb/in. fringe to 59 lb/in. fringe were obtained.

A test was carried out to study the effect of joining two pieces of 'Araldite B' sheet with 'Araldite' Adhesive. A strip of 'Araldite B' sheet was carefully cut in two using a hacksaw. Both pieces were examined in a polariscope to check that stresses had not been induced by the cutting. The mating edges were cleaned with carbon tetrachloride; other surfaces were coated with 'Releasil 14'. 'Araldite' Adhesive was mixed and applied to the joint faces. The joint was made and the job supported on a sheet of plate glass coated with 'Releasil 14'. The joint was left to cure for 4 days after which the Araldite sheet was cleaned of 'Releasil 14' and excess adhesive, and examined in a polariscope. The joint showed up black. No stresses had been set up by the jointing procedure. On test under load there was no discontinuity in the fringes across the joint.

A.3.4.6. Conclusions.

Stress free sheets of 'Araldite B' can be produced by the method described in previous sections.

The most generally useful method of cutting models to size is the use of a pantograph engraving machine.

'Araldite' Adhesive is a suitable material for making joints in the construction of composite models.

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