A MODEL TEST FACILITY

FOR

SOIL-STRUCTURE INTERACTION

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by

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ABSTRACT

Aspects of the development and design of a large scale model test facility are presented. The test facility is capable of preparing relatively large beds of clay under variable controlled conditions, and of carrying out three-dimensional model footing tests under combined vertical and horizontal loading. Important features are the ability to produce an increase of strength with depth in the clay bed, and to accommodate large displacements and rotations of the footing during test.

The tests form part of an integrated research programme into soil-structure interaction in relation to offshore oil production gravity platforms in environments similar to that of the North Sea. They are on a scale intermediate between those of more usual soils laboratory, and full-scale prototype, testing.

The preliminary design of a scheme which would have allowed testing under plane strain conditions at working stress levels is described briefly. A review of the investigation of this approach centres on a study of the influence of the tank walls. This is found to be such that, for the scheme proposed, significant modification of plane strain behaviour could have been expected. It was partly because of this that the plane strain approach was discontinued and the three-dimensional scheme proposed.

In order to confirm design and performance objectives for the three-dimensional test facility, pilot footing tests were undertaken using beds of clay prepared in a 229 mm oedometer. These tests and their results are discussed, and lead to recommendations for modification to the approach to the development and operation of the large facility.

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CHAPTER 1

A FRAME OF REFERENCE

1.1 AN OVERVIEW

The work described in this thesis formed part of an integrated programme within the Structures and Soil Mechanics sections of the Civil Engineering Department at Imperial College. It involved the development of a test facility capable of preparing relatively large beds of clay under controlled conditions and of carrying out model footing tests under combined vertical and horizontal loading. Together with complementary analytical studies and basic soils testing, bed preparation and footing tests aim, in the first instance, to provide information about aspects of soil-structure interaction in relation to offshore oil production gravity platforms in environments similar to that of the North Sea.

The underlying geotechnical theory behind the design of the test facility was provided by those members of the Soil Mechanics section associated with the project, and a detailed presentation of it falls outside the scope of this thesis. Where it is important to the understanding of the work based on it, a skeletal presentation of geotechnical theory is given. Otherwise, reference is made to original work being carried out at the present time, or to appropriate established sources.

The work falls loosely into several main sections dealing with the design brief and its development, the computer studies of a proposed scheme, and the design of the adopted scheme, along with associated pilot tests to confirm the design specification.

1.2 BACKGROUND

Raft type footings for oil production gravity platforms currently being placed in the North Sea are of the order of 100 metres in diameter and 50-70 metres thick, and transfer large vertical and horizontal forces to the sea bed, Figure 1. The volume of foundation material mobilised in the support of such structures is large indeed. Even on land, conventional structures of this size would provide unusual design problems. These are compounded in the case of offshore structures by two additional factors.

The structures are subjected to repeated horizontal cyclic loads resulting from wave action, which are large in relation to the dead weight of the structures. These cause eccentric foundation loads and lateral shear forces, which, together with varying pressures on the sea bed around the structures, determine contact pressures, and foundation stability and deformation. Given typical periods of wave loading, conditions in the foundation under the bulk of the raft are undrained. Under these conditions, dense sand can lose strength and loose sand can liquefy and cause collapse of the structure (Seed and Idriss, 1971, Seed and Lee, 1967). In clays, changes in pore pressure may lead to permanent, possibly detrimental, modification of behaviour (Sangrey et al, 1968).

Additionally, the structures are complete prior to placement on the sea bed, and are thus subject to initial lack of fit between the structures and their foundations. Locally this is due to unavoidable unevenness of the sea bed, and to the provision of skirts and non-planar bases for stability and structural efficiency. On a larger

scale it is caused by a need to compensate for sloping of the sea bed.

The difficulties of predicting contact pressure distributions between raft and foundation, of predicting pore pressure build-up and dissipation, and of assessing the effect of cyclic change in applied stresses are increased in this case by the scale of the raft. Stresses and strains are induced in the foundation to a depth of perhaps twice the footing diameter, over which depth marked changes in foundation material and properties can reasonably be expected to occur. The situation is further complicated by the uncertainties involved in determining foundation properties, either by in-situ testing, or by taking of samples for laboratory testing.

The effects of these dissimilarities between offshore structures and those on land are not well understood. "Much pioneer work has to be done before one can reach a quality level in soil exploration, soil testing, and design procedures which is in proper proportion to the importance, cost, size, and risks involved in these structures" (Bjerrum, 1973, p.355).

1.3 SOIL-STRUCTURE INTERACTION PROJECT

It was to increase the degree of confidence with which design of offshore structures could be undertaken and their long-term viability assessed, that the research programme, of which the model footing testing forms part, was proposed and sponsored. The main elements of the programme are as follows.

A computer program (Hamza, 1976) based on the finite element technique of analysis, is being extended to allow analytical examination of both gravity (static) and cyclic wave (dynamic) loading of a foundation. The appropriate non-linear constitutive relationships for the foundation material to be used in the analyses are to be based on existing knowledge of North Sea bed soil types, supplemented where necessary by special tests. In this context, as a first step, a programme of cyclic testing of a typical clay is being carried out (Hight, 1977) as the determination of behaviour of soils under cyclic load involves the determination of the deformation characteristics at typical stress levels and stress changes, and under appropriate drainage conditions. More general investigation and assessment of the typical clay are also being carried out (El-Ghamrawy, 1977).

Existing laboratory techniques, using simple shear, triaxial and oedometer equipment, cannot fully reproduce complex conditions in the soil under a raft foundation. Therefore a hollow cylinder apparatus is being developed to provide deformation characteristics under more representative conditions.

The large scale model footing testing is to provide the link between the analytical studies and the basic soils testing. It will use as foundation material the clay tested in the soils laboratory, prepared under similar conditions. The results will provide a means of assessing the predictive capacity of the computer program incorporating constitutive relationships derived from soils laboratory tests performed with uniform stresses applied to the soil specimens.

The model footing test has the advantage that it can reproduce quite closely conditions within the soil under a raft. It can, therefore, by itself, provide useful information about behaviour for a given soil and a given loading history. It has the disadvantages that it is not easy to prepare a complex foundation, that results are influenced by the boundaries of the tank containing the soil bed, and that some types of loading (e.g. random dynamic) are not easily produced in the laboratory situation.

The footing testing is therefore seen as being able, for simple foundations and manageable loadings, to provide some basic information, and, more importantly, to aid in assessing the computer program, which can, once proven, readily cope with complex (and more realistic) foundation and load types.

1.4 THE MODEL TESTING PROGRAMME

Early consideration of the model footing testing eliminated a fully rigorous dimensional study (Rocha, 1953, Roscoe, 1968, Massey, 1971). For a linear scale of even one to one hundred of model to prototype, certain conditions of similitude would have meant fundamental differences in model behaviour compared with that of the prototype. In terms of permeability and particle size, a prototype foundation of sand would have meant a clay bed in the model. Prototype relative soil stiffness (clay or sand foundation under a concrete raft) would have required rock to be used in the model. A wave period of ten seconds in the field would have meant a prohibitively short period of one tenth of a second for model loading. In order for model tank boundaries to exert no influence, the tank would have had to have been extremely large.

It was therefore decided that it would be desirable to model closely conditions in the soil in the field, by using foundation materials representative of those occurring in the upper zones of the sea bed, and subjecting these to stress levels and stress changes similar to those under prototype structures. The speed of testing was to ensure that drainage conditions were reproduced.

The response of a prototype structure to wave action is amenable to solution once the foundation characteristics (and wave forces) have been established, or estimated with sufficient accuracy. A review of methods is given by Haggett (1974). Thus the experimental work was simplified by representing the structure by a simple plate of appropriate relative stiffness, and by subjecting this plate to vertical and eccentric horizontal loads. In the first instance the plate would be plane. Lack of fit, with only local effects on the foundation unlikely to affect ultimate behaviour (Clausen, 1973, Bishop and Vaughan, 1974) are far more critical for raft design. Skirts influence resistance to sliding by effectively founding the structure in a subsurface (and hopefully stiffer) stratum, and this increase in sliding stability can be allowed for in model soil bed preparation. It is planned that a later development in the test programme will include full instrumentation of the model footing, to allow detailed contact pressure distributions to be obtained. This will allow lack of fit and skirt penetration problems to be studied.

Two forms of model footing testing have been considered; plane strain (two-dimensional), in which the conditions under the central portion of the raft would be reproduced by a strip footing, and three-dimensional (3D). Both have merits and disadvantages. In the plane strain approach, the number of variables under consideration is reduced, present soils laboratory testing can provide satisfactory

non-linear constitutive relationships, computer studies are easier to formulate and more economical to run, and closed form solutions are available for many conditions (Poulos and Davis, 1974). The three-dimensional approach has the advantages that no idealisation of loading or extrapolation of results to a real situation is necessary, and that boundaries can be made to have less influence.

Preliminary computer and other investigations showed that for the two-dimensional, plane strain, testing the influence of the sides of the tank could be significant, and that development of a facility to fully utilise the potential of the more controlled conditions would be too costly and lengthy for the sponsored programme.

The three-dimensional testing is seen as a shorter-term compromise solution, which will provide much needed information about foundation behaviour in a reasonable time scale. It will provide basic data and a frame of reference for the development and interpretation of plane strain testing. For axisymmetric loading at least, radial stress non-linear computer studies can be checked. Then in a preliminary way, three-dimensional linear computer studies can provide insights into more complex behaviour than can be physically modelled. A planned modification of the tank by partitioning will allow pilot plane strain tests to be carried out.

The design specification for the three-dimensional footing testing and sample preparation was based on soils laboratory testing data. Many parameters were examined and confirmed during the design/fabrication stage using a (modified) 229 mm diameter oedometer apparatus available in the soils laboratory. Testing and development of both structural and soils related components of the tests were carried out on a small scale whenever possible prior to full scale fabrication or production.

CHAPTER 2

PLANE STRAIN MODEL TESTING PROPOSAL

2.1 INTRODUCTION

Early in the development of model testing proposals the decision was taken to evaluate a plane strain approach. The reduction of a real three-dimensional situation to a two-dimensional one allows a focussing on some conditions by the specific exclusion of others, such as the lateral contact stress distribution, the effects of sharp corners of non-circular footings, and the effect of lateral eccentricity of load due to arbitrary horizontal loading. Average conditions can be examined for later extrapolation and modification. In general, a narrow tank can facilitate either visual or other examination of the deformation within the soil or at its outer boundary, progressively, at pre-failure loading levels (e.g. X-ray, Roscoe, 1970; photographic exposure, Spencer, 1964, Butterfield and Andrawes, 1972).

2.2 OBJECTIVES

The tests were envisaged as concentrating primarily on working load conditions rather than ultimate loads, and were to be on a scale larger than that allowed by radiographic technique constraints (Roscoe, 1970). It was hoped to be able to measure or derive directly from the model test data, bearing and shear stress distributions at the underside of the footing, strain and pore pressure distributions and deformation fields within the soil, and boundary stresses. The influence of consolidation on these was also to be studied.

Rather than construct a model of the whole structure and apply vertical and horizontal loads to this, a stiff plate would be used to transmit loads to the soil. The loading levels were to be appropriate to the sensitivity of the measuring/recording systems, and to gravity platform loading levels.

The soil tested would initially, for ease of handling, be dry (sand) but later tests would incorporate a cohesive material (clay, Kaolin) with adequate provision for improved drainage during consolidation. In the case of sand, surcharge loading would be applied.

2.3 BASIC LOADING LEVELS

In order to produce soil beds similar in strength and deformation characteristics to those found in the North Sea bed (Hight, 1977), the following stress levels were adopted on a trial basis for the investigation of the proposal.

Overconsolidating pressure:	350 kPa
Long-term surcharge (sand):	100 kPa
Long-term surcharge (clay):	0

On a practical level, the overconsolidating pressure corresponds closely to the maximum pressure that can be applied through liquid-filled bags. It was proposed to use these rather than rigid plates (which could transmit higher stresses) in order to ensure uniformity of consolidating pressure rather than of surface settlement. The long-term surcharge of the sand is necessary to produce realistic behaviour under test conditions which is dependent on mean stress levels.

The average and peak fluctuating plate stresses given below correspond to those under typical platforms under working conditions (Clausen 1973, Eide, 1974).

Average vertical plate stress:	250 kPa
Average horizontal shear stress:	±100 kPa
Peak fluctuation of vertical stress:	±100 kPa

2.4 MODEL FOOTING

Given the proposed level of footing loading, a general arrangement was devised for the model strip footing incorporating a minimum of eight individual strips, in order to allow some determination of contact pressure distributions. Various methods were considered for measuring the load, and load distribution, on a strip, including those reviewed by Bransby (1973), and one incorporating pressure pads between strips (giving shears) and between strips and a rigid supporting plate (giving vertical loads). For preliminary study, an adaption of the Cambridge contact stress transducer was adopted, Figure 2. In order that strain levels in the strain bars should be reasonable (500-1000 microstrain at full load) without the bars becoming too slender, a strip size of 75 mm long by 500 mm wide was decided upon. The model footing size therefore became a minimum of 600 mm long by 500 mm wide, Figure 2.

2.5 MODEL TANK

Studies carried out by Davachi (1977) using the finite element program developed by Hamza (1976) had shown that for the end walls and the base of the tank to have insignificant influence on conditions in the region of the footing, these must be of the order of three times the footing length from the footing centre line. This dictated a tank

elevation of approximately 3.5 metres long by 2.0 metres high (allowing 250 mm freeboard to the soil) based on the footing length of 600 mm. The footing width of 500 mm had automatically set the tank width. A tank of this size holds over 3 cubic metres of soil, weighing in the region of 6 tonnes.

The walls of the tank would be subjected to lateral pressures from the soil. The levels of these are determined by two conditions; initial consolidation, giving a uniform distribution along the length of the tank at any depth, and footing test, when lateral pressures arising from vertical and horizontal footing loads are superimposed on initial stresses remaining after vertical consolidation pressure has been removed, or reduced.

Computer studies carried out to give information about sample behaviour under footing loads (see 2.6) gave lateral pressure distributions. As an example, those for the case of sand (including surcharge) are given in Figure 5.

For both conditions of lateral loading, clay gives more severe lateral pressures than sand, because of a higher value of the coefficient of earth pressure at rest, K_o. Table 1 summarises the average (or equivalent uniformly distributed) lateral pressures on the walls of the tank in the region of the footing. These were used for preliminary tank design.

A review of lateral displacements of tank walls in plane strain experiments on both model footings and retaining walls showed wide variation in accepted values, Table 2. For the purposes of preliminary design of the tank, two conditions again were recognised;

consolidation and footing loading. In view of the anticipated more critical role of lateral displacements during the footing tests, a stricter limit was adopted for this phase than during consolidation, Table 1.

When a tank design was prepared, governed by these load and deflection criteria, a rather massive structure resulted, Figure 3.

2.6 PLANE STRAIN COMPUTER STUDY

As part of the evaluation of the plane strain proposal, computer studies using the proposed tank geometry and loading levels were carried out to ascertain likely levels of stress, strain and deformation for both sand and clay. Footing plate thickness was determined on the basis of the non-dimensional coefficient for relative plate stiffness

$$K = \frac{E_p}{E_r} \left(\frac{1-\nu_{sr}^2}{1-\nu_p^2} \right) \left(\frac{t}{R} \right)^3$$ (Boswell and Scott, 1975),

with K initially set equal to 10, a value representative of prototype relative stiffness (Hight, 1977). E_p, ν_p are the elastic constants for the plate. t and R are the thickness and radius of an equivalent circular plate. E_r and ν_r are soil constants.

The computer program (Hamza, 1976) allowed non-linear finite element analysis under plane strain conditions. A relatively fine mesh was set up, Figure 4. Initial stresses, based on proposed consolidation sequence, were set at the start of computation. Loadings (surcharge and plate) were applied incrementally to allow closer modelling of the non-linear behaviour characteristics. Soil moduli and Poisson's ratio adopted are given in Table 3.

The program output gives, among other values, stresses and strains at the Gauss points and at the centre of elements. Displacements are given at the node points.

2.7 CONCLUSION

The plane strain approach to testing was investigated and a preliminary scheme was developed for model footing and tank. The computer studies undertaken were to have provided realistic estimates of footing and soil behaviour to facilitate design of the model footing, the tank and instrumentation. In the event, lateral pressure distributions were used for preliminary design of the tank. Surface deformation distributions, Figure 6, for sand, together with estimated lateral displacements of tank walls, were used in subsequent computer studies of the effects in three dimensions of lateral displacements and side friction of tank walls (see Chapter 3).

On the basis of these subsequent studies and other investigations the plane strain approach, concentrating on pre-failure conditions and relying on idealisation of loading and extrapolation of results to a real situation, was passed over in favour of a three-dimensional approach concerned primarily with ultimate conditions, for which loads, stresses, strains and deformation levels are conveniently higher. The plane strain test evaluation did, however, provide much useful information and a frame of reference for the design of the three-dimensional scheme.

CHAPTER 3

SIDE BOUNDARY LATERAL DISPLACEMENTS AND FRICTION IN PLANE STRAIN EXPERIMENTS

3.1 INTRODUCTION

In laboratory experiments where it is intended to reproduce conditions of plane strain, such as occur for example under central portions of long engineering structures, complications can arise in consequence of the inability of side boundaries to behave as would soil in a continuum. Under plane strain conditions there is between one section and the adjacent one a balancing of lateral pressures which precludes lateral deformation, and a compatibility of displacements which eliminates shearing stresses. In the test situation, side boundaries are neither totally free from lateral displacements in the presence of lateral pressure, nor perfectly frictionless.

The possible limitations of the laboratory situation have long been recognised, and various means have been adopted in the design of experiments to either minimise the effects of side boundaries or to measure them.

In order to make an independent assessment of the possible significance of side boundary lateral displacements and friction for the plane strain tests considered (see 2.2), a computer study using a program incorporating three-dimensional finite elements was carried out, and stress, strain and deformation fields were compared with those existing under plane strain conditions.

3.2 REVIEW OF PREVIOUS WORK AND ATTEMPTED SOLUTIONS

3.2.1 Introduction

A large part of the experimental work done under plane strain conditions has been concerned with the behaviour of retaining walls. Nevertheless, the problems encountered and the measures adopted to overcome them are also appropriate to a consideration of footing tests. There are some differences in conditions, the possible importance of which is not altogether clear. Walls are often rotated about fixed axes, whereas footings are allowed overall freedom of movement; movements of walls, especially in the active pressure mode, are small compared to those associated with footing failure.

The experimental work falls loosely into two categories; that in which forces and deformations of the wall or footing are of prime concern, and that in which conditions within the soil at pre-failure are of interest.

When a study of wall (or footing) behaviour has been the object of the experiment, a relatively simple solution has been to make the width (W) of the wall large in relation to its height (H), and thus make side boundaries remote. Typical of this approach is the experimental work on walls by Rowe and Peaker (1965) in which a wall 1.8 m long by 0.45 m high (W/H = 4) was used in the study of passive pressure with sand. Side boundaries were made quite stiff and precautions were made to reduce their friction. Nevertheless, the slip failure outcrop, quite straight in the central region, showed marked curvature in the outer 200 - 300 mm, demonstrating the modifying influence of the side boundaries. Rowe and Peaker also seem to have been the first (for a historical review see Kerisel, 1972)

to adopt the refinement of dividing the wall (or footing) into discrete sections, so that load distribution between central and outer sections could be identified and assessed. In this case, passive pressure readings in the central third of the wall were consistently found to be 25% less than those in the outer thirds, again demonstrating the influence of side friction in the outer regions adjacent to the boundaries.

This technique of dividing walls and footings into sections has now been refined to the stage where the wall or footing face becomes a mosaic of load transducers (Roscoe, 1970), so that a quite detailed contact stress distribution can be obtained. The results have not always been as conclusive as those of Rowe and Peaker, particularly in smaller models where smaller contact stress transducers allowed less averaging out of stresses. Non-uniformity of bed preparation has been identified in footing tests as the more likely cause of irregularities in stress pattern (Tennekoon, 1970). Generally, though, there has been a trend of lower stresses being recorded in central regions of walls and footings (James, 1965, Bransby, 1968 Dick, 1973, Milligan, 1974). For width to height ratios (W/H) for walls (width to length (W/L) for footings) of greater than 2 to 3, reasonably uniform behaviour has been found in a central region (Berry, 1965, Barden and Berry, 1965, Bransby, 1972).

It has not always been possible to design experiments on a scale in which side boundaries are remote (in terms of width to height ratio). This has been so in the case of the small scale model studies developed around the technique of x-ray photography of lead shot targets embedded in soil samples. Once a model for soil behaviour has been

formulated, their lead shot movements allow calculation of strain and hence stress fields in the soil at pre-failure and failure states (James, 1965, Roscoe, 1970). With this experimental technique, the overall width of the sample plus side boundaries becomes a significant factor in the design, in order to allow adequate resolution in the x-ray photos. The need to keep the side boundaries thin and reasonably transparent to x-rays, increases their flexibility and precludes use of some of the friction reducing techniques. The consequences of this potential increase in side boundary effects are compounded by the need to reduce the sample width. In order to maintain a high degree of resolution of vertical (for walls) or longitudinal (for footings) contact stress distributions, the height or length respectively, cannot be much decreased, and so W/H and W/L ratios are reduced (to 0.6 - 1.7 (Arthur and Roscoe, 1965, Roscoe, 1970)) with corresponding reduction in the region possibly unaffected by side boundary effects.

3.2.2 Lateral Displacements

The influence of side boundary lateral displacements, when not ignored, has been evaluated independently of side boundary friction, as the control of one does not necessarily reduce the other. At the inception of small scale model work at Cambridge, it was found in investigations where glass side wall thickness was varied from 6 mm to 16 mm for otherwise identical retaining wall experiments with sand, that " ... the effect on experimental results of varying the rigidity of the side plates ... [is] ... surprisingly large and suggests that the magnitude of the intermediate stress σ_2 has a considerable effect upon the behaviour of the soil" (Arthur and Roscoe, 1965, p.363). Subsequently, attempts

have been made to reduce side wall deflections to insignificant levels, both by increasing glass thickness and by decreasing span lengths as much as possible without physically interferring with the x-ray procedure. A summary of measured deflections is given in Table 2.

The influence of these displacements on the experiments has been impossible to evaluate, as comparison tests in the presence of walls of different stiffness were not carried out. The further reduction of displacements would in any case have made radiography impossible.

In the field of footing tests, Ko and Davidson (1973), aware of previous shortcomings of tanks for soil in terms of lateral stiffness, designed a braced tank which had very small lateral displacements. In each series of footing tests with sand, when some of the across-tank bracing was removed, lateral displacements increased twelve-fold and bearing capacities of the footings dropped to levels between 25 and 45% below the average values in similar braced tests, depending on whether footing faces were rough or smooth.

3.2.3 Side Friction

The attempt to reduce side boundary friction has been a feature of both large and small scale tests. Detailed investigations of the effect of friction have generally been associated with small scale tests where the significance or otherwise of the frictional stresses has been found to depend on which parameters are being measured. Bransby (1973) and Beasley (1970) have shown that for retaining wall tests with sand under passive conditions, side friction will increase the magnitude of the force on the wall, but will only slightly alter the relationships between wall normal and shear forces and the depth

of action of the total force. The slip outcrop is essentially straight, and shows no difference at the edges. Early work by Arthur and Roscoe (1965) showed only small differences in centre-line and edge displacements in a 100 mm wide sand bed behind a retaining wall. Lead shot were placed in a grid in the sand along the centre-line of the tank, and the movements of these recorded radiographically during the retaining wall test. Nylon hemispheres were placed along the glass face in the same grid as the lead shot, and their movements photographed. Negatives were made compatible with the x-ray photos. The movements at the face and internally were compared and found to show no significant variations. The conclusion drawn was that side friction has negligible effect.

While not disputing that there is insignificant variation in behaviour across the width of the tank in these experiments, Rowe (1972) and Kerisel (1972), among others, argue that there is no proof that the influence of side friction is not uniform across the narrow tanks, in the presence of the stiff displacement boundary that the retaining wall represents.

Burland and Roscoe (1969) report work by Hambly into the effect of side boundary friction in an apparatus for preparing a bed of normally consolidated clay for footing tests. Previous work with oedometers had shown that for diameter to height ratios of approximately 4 the effects of side friction could be minimised by smearing the walls with silicon grease. In this case, width to height was less than 0.6. Load applied by a piston to a clay in a tube was compared to the load applied by the clay to a load cell at the other end of the tube. Figure 7 shows the results of

tests with and without silicon grease treatment of walls. Quite significant changes in sample loading can result when a high proportion of load is carried by side wall friction.

A computational approach to the problem of side friction is presented by Bransby and Smith (1973). This approach utilises the method of characteristics attributed to Sokolovski, and was applied to retaining walls. The results of the study broadly confirm those of experimental studies in terms of the significance of width to height ratios, the influence on coefficients of active and passive pressure, and the small influence of side friction on the deformation field and the shape of the rupture planes. Although the method is not applied to footings, it can be modified to footing conditions. In this context, Bransby and Smith comment that side friction is likely to have a much greater effect in footing tests than in retaining wall tests, as the extent of the stress field is much larger.

The problem of (side) boundary friction is not unique to plane strain testing (end platten friction affects stress distributions in triaxial specimens (Bishop and Henkel, 1957)), and so, much evaluative study has been done in other areas. Some of the low friction systems or treatments developed for other situations do not lend themselves to utilisation in plane strain work however, as they could be sources of lateral displacements. A good example of this is the multiple neoprene rubber/grease layers found to practically eliminate end platten effects in triaxial tests (Green and Bishop, 1969, Atkinson, 1973). Even thick coatings of grease alone may allow significant lateral movements through being squeezed, especially if "significant" is defined in relation to soil particle size. Additionally, there may be a reduction in lubricating

efficiency with time under constant normal stress, especially in the presence of water (Burland and Roscoe, 1969, Atkinson, 1973).

Much of the investigative work has satisfied itself with establishing that (side) friction effects have been minimised, and actual values of coefficients of friction have not been determined in those cases. Table 4 gives some measured values of coefficients of friction under different conditions. Potentially low friction treatments, such as PTFE coatings, have been tried, but found to be easily scratched and thus rendered useless. Sand has been found to be able to seriously mark glass.

3.3 COMPUTER STUDIES

3.3.1 General

The previous studies into the influence of boundary effects had established that this should not be underestimated in the design of a test. Test geometry had been found to be an important factor.

As much of the work done to date had been concerned with the retaining wall situation, it was felt to be advisable to study side boundary lateral movement and friction effects for the proposed large scale plane strain footing test, and to do this using available computer programs. The preliminary model tank (i.e. soil) dimensions (see 2.5) were 3.5 m long by 1.75 m high by 0.5 m wide. Sample material was taken to be sand.

3.3.2 Plane Strain Study

The non-linear plane strain computer study carried out to provide basic information for the design of the proposed tests and systems (see 2.6) was utilised as a frame of reference for assessment of the three-dimensional computer study. The non-linear properties

for sand used in the program (Hamza, 1976) are appropriate to dense sands found in the North Sea bed (Hight, 1977) and are given in Table 3. The adopted finite element grid (two-dimensional) is shown in Figure 4. Initial stresses in the sand bed were modelled. Surface surcharge and plate loading were applied in small incremental steps; surcharge, 3 increments of 33 kPa, with additional plate loading in 3 increments of 30 kPa. Previous experience with the program had shown that these increments provided adequately close modelling of the non-linear response of the sand.

3.3.3 Three-dimensional Studies

To obtain a qualitative indication of levels of stress, strain and deformation produced by side boundary lateral displacements and friction, a three-dimensional finite element model was set up and studied using a program developed by Ward (1975). Sand properties adopted were linear elastic, Table 3, as realistic three-dimensional non-linear constitutive relationships were not available, and in any case, had they been, a non-linear analysis would have proved to be beyond the capacity of the available computer for all but the coarsest grid mesh. The results of these studies were seen as providing values of the right order of magnitude for comparison with the plane strain non-linear analysis results.

Prior to selection of the grid, Figure 8, a small scale study demonstrated the suitability of the three-dimensional eight node element, rather than the sixteen node one, for giving adequate resolution. Maintenance of a reasonably compact computational problem meant that the grid mesh was not as fine as that in the plane strain study, in consequence of the fact that the computer

model is four element blocks deep. The mesh was however fine enough in the region adjacent to the plate for reasonable comparisons to be made.

The effect of tank geometry was investigated by considering a tank (length 3.5 m) with widths of 0.5 m, 1.0 m and 2.0 m. Initially tank depth was taken as 1.75 m, for which the base of the tank has negligible effect on footing behaviour (Davachi, 1977). When these cases confirmed positively the sensitivity of plane strain conditions to tank width to depth ratio, the computer model depth was reduced to 1.0 m, and the analyses repeated. For these cases, the base of the tank normal and shear stress distributions became factors of interest.

Model footing plate thickness was based on a relative stiffness of plate to soil of greater than 10 (see 2.6).

3.3.3.1 Lateral Displacement Cases. Lateral displacements of tank side walls are complex, due to the complex variation of lateral pressure associated with footing loading and to the variable stiffness of wall construction. It was not possible to model them. It was decided that maximum likely levels of lateral displacement resulting from footing loading in the tank designed (see 2.5) would be applied uniformly along the whole length of the tank, thereby overestimating the influence of displacements in regions away from the footing.

In order to gain a feel for the likely significance of lateral displacements of the order allowed by tank design, a trial case considered a rectangular displacement distribution (not practically possible in view of end and base restraints) of the peak expected adjacent to and under the footing during test, Figure 9. This

trial case revealed the potential importance of lateral displacements, and so a modified quadrilateral distribution was used in the computer study proper, Figure 9. The effect of further reducing displacements (scarcely practical for the design evolved, Figure 3) was taken into account by considering the case where one-tenth of original displacements were applied. Reduced lateral displacements were applied in the case of the tank of reduced depth, Figure 10.

In the absence of any other loading or restraints, the imposition of side boundary lateral displacements resulted in the setting up of stress, strain and deformation fields within the model. These solely represented the response to the imposed boundary condition, and so, absolutely small values would correspond to minimal modification of (plane strain) behaviour.

3.3.3.2 Side Friction Cases. The side frictional shear forces will vary considerably along the tank walls and will depend on the lateral pressure distribution. In the present study, two extreme cases were considered; limiting friction which allows no relative displacement between soil and wall, and zero friction which models an ideal frictionless interface between soil and wall. This latter condition was considered to be a check on the plane strain case.

For the case of limiting frictional boundaries, the frictional shear forces are related to the level of deformation occurring (or tending to occur) within the soil away from the boundaries. The plane strain computer study had provided surface deformation profiles under plate and surcharge loading. These were imposed as surface boundary

constraints to the surface nodes of the central two elements for the three-dimensional study, Figures 9 and 10. Application of surcharge and plate loading to these central elements would have resulted in an underestimation of behaviour, and full-width application would have produced problems at the boundaries. Additionally, the linear elastic properties used in the three-dimensional model would have resulted in less realistic levels of deformation than had been obtained in the plane strain study.

The imposition as top surface boundary constraint of the surface deformations obtained in the plane strain study resulted in the setting up of stress, strain and deformation fields within the three-dimensional model. In the absence of modifying influence of side boundary friction, these fields would be similar to those obtained from the plane strain study.. Any differences would arise only because of different grid mesh and soil properties adopted in the two studies (see 3.3.3).

3.4 RESULTS

3.4.1 General

In the interests of coherence and clarity, only part of the results of the computer studies are considered and presented in detail. Longitudinal (x) direct stresses and deformations, lateral (y) direct stresses, and vertical (z) direct stresses and deformations are selected for investigation of variation, and for comparison purposes. Except for consideration of coefficients of limiting friction for the outer wall boundary nodes, Figures 30 to 32, attention is restricted to and results given for, the nodes in the central plane of the three-dimensional model, Figures 11 to 29.

These are least influenced by possible local effects resulting from imposed boundary conditions. In many of the figures, presentation of results is further restricted to the nodes under the edge of the plate, Figures 18 to 29, where conditions are most influenced by footing plate loading. In order to demonstrate general trends along the model tank (in both lateral displacement and side friction cases) a series of figures, Figures 11 to 17, gives full tabulation of results for all central plane nodes for one selected tank width. This facilitates assessment of and extrapolation to general behaviour away from the edge of the plate nodes, or to other tank width conditions.

The figures deal principally with the results for a tank of depth 1.75 m. For completeness, results for the tank of depth 1.0 m are given in compacter form in Tables 5 and 6.

Where appropriate, values of stress or deformation obtained from the plane strain study are given for comparison purposes.

3.4.2 Lateral Displacement Cases

3.4.2.1 Uniform Displacement, Figure 9. The results of this pilot study for a tank depth of 1.75 m and width of 0.5 m are given in Figures 11 and 12. Stresses and deformations are, in some places remote from the plate of the same order of magnitude or greater than those from the plane strain study, thus highlighting the need for a more refined definition of imposed lateral displacements. Actual tank design reduced the possible displacement at the top of the tank by clamping, and prevented it altogether at the base through bolting (or welding) to the base plate. Maximum displacement was likely to occur, under these circumstances, near the upper third

point of wall height.

3.4.2.2 Quadrilateral Displacement. The imposition of the more realistic lateral displacement distribution, Figure 9, results in a marked reduction in stress and deformation levels. Figures 13 to 17 show a complete comparison of results for a tank 1.0 m wide with corresponding results from the plane strain study. The imposition of the quadrilateral displacement (applicable principally only in the region of the plate) still results in overestimation of stresses and deformations in regions remote from the plate. In the region of the plate, however, they are of the right order of magnitude. This full tabulation shows that the nodes under the edge of the plate give more significant (i.e. absolutely larger) levels of stress and deformation when compared with plane strain values, than do the nodes under the interior of the plate. A concentration of attention on the nodes under the edge of the plate, therefore, will slightly overestimate the average significance of boundary effects.

For this three-dimensional case, both stress and deformation fields show the influence of boundary lateral displacements, Figures 13 to 17, varying from less than 1% to around 20% for stresses, and up to 15% for deformations, when compared with plane strain values. In the higher percentage levels, these three-dimensional changes, if allowed to occur, represent a significant and measurable departure of model conditions from plane strain, and would result in over- or underestimation (depending on sign) of behaviour.

The effect of varying the width of the tank and hence the width to depth ratio is shown in Figures 18 to 22. For x, y stresses and z

deformations the dramatic reductions which follow from increasing tank width from 0.5 m (B/D = 0.29) to 1.0 m (B/D = 0.57) taper off once tank width reaches 2.0 m (B/D = 1.14). In each case, the reduced values associated with a tank width of 1.0 m (B/D = 0.57) are already close to being insignificant. z stresses and x deformations are less regular in their pattern, but are, in any case, at levels which are not significant.

Precictably, the effect of reducing the absolute amount of lateral displacement reduces the levels of stress and deformation within the soil mass. Figures 23 and 24 compare results for the main displacement case with those which result when one-tenth of this displacement is imposed. At this lower level of displacement, stress and deformation fields would insignificantly modify plane strain behaviour.

The reduction of tank depth to 1.0 m also results in reduced significance of lateral displacements, Table 5. This arises because a smaller lateral displacement, Figure 10, can be associated with a shallower tank when construction remains unaltered, and because reducing the depth effectively increases width to depth ratios for the cases considered.

3.4.3 Side Friction Cases

Whereas in the three-dimensional lateral displacement study, insignificance of influence would lead to results that ~~there~~ were small in relation to the plane strain values, in the side friction study the opposite was the case. An absence of modifying influence of side friction would lead to values of the same magnitude as those obtained in the plane strain study.

In this context, the three-dimensional side friction case where boundary friction was set equal to zero is a check on how well the three-dimensional linear elastic, coarser mesh finite element study can reproduce the results of the non-linear elastic, finer mesh finite element plane strain study which corresponds more closely to real behaviour. For the critical nodes under the edge of the footing plate, the three-dimensional study values are an average of 89% of the plane strain values (variation 70 to 125%) for the 1.75 m deep tank, Figures 25 to 29, and 102% (variation 68 to 120%) for the 1.0 m deep tank, Table 6. This reasonable level of agreement between the two analyses establishes the validity of comparing values from the three-dimensional study with those of the plane strain study.

3.4.3.1 Effect of Tank Width. For the imposed condition of boundary limiting friction the general effect of increasing tank width (and hence width to depth ratio) is to bring stress and deformation fields closer to those existing under plane strain conditions. Stresses, Figures 25 to 27, Table 6, decrease for nodes near the surface of the tank, while for those near the base, they increase. This tends to occur also for x deformations, Figure 28, Table 6, while for z deformations, Figure 29, Table 6, there is a general increase in level of deformation at all depths. These results show a tendency in the soil to arch across the tank, which (tendency) is progressively reduced as the tank becomes wider.

3.4.3.2 Side Boundary Coefficients of Limiting Friction. In the modelling of the side friction condition, it was possible only to either fully restrain boundary modes, or to free them. The real situation is likely to fall generally between those two extremes,

with some relative movement between soil and wall being possible. Figures 30 to 32 show full tabulations of effective coefficients of limiting friction necessary to fully prevent any slip and the side boundaries in the tank of depth 1.75 m. The trend is for decreasing coefficients of friction with increasing tank width, which is a direct reflection of decreasing vertical shear associated with less tendency in the soil to arch as tank width increases. The significant effect of width to depth ratio is again highlighted by the comparison of necessary coefficients of limiting friction for a 1.5 m wide by 1.0 m deep (B/D = 1.5) tank and a 1.0 m wide by 1.75 m deep (B/D = 0.57) tank, Figure 34. For the wider tank an average coefficient of 1.03 is required; for the narrower one, 1.37. Both of these levels are high in relation to measured values of coefficient of friction of sand with typical tank wall materials or treatments, Table 4.

3.5 DISCUSSION

3.5.1 General

The computer studies of the influence of side wall lateral displacements and friction on plane strain conditions have confirmed that due attention must be given and that special care must be taken in the design of plane strain testing facilities to reduce their significance.

For the tank geometries, design, and loading levels considered, both lateral displacements and side friction were shown to be able to cause serious departures from plane strain conditions. The potential extent of their influence is greater than would have been expected from a study of previous plane strain testing. The reasons for this

may be that proposed footing loads gave low working stress levels with correspondingly small stress, strain and deformation fields, that the wish to make (bottom) boundaries remote led to low width-to-depth ratios, and that the material analysed was sand. Previous experiments have involved loading up to ultimate, for which surface penetrations especially of footings are large, have attempted to keep width-to-depth ratios high, and have often used clay which gives greater deformations for relatively lower loading levels.

3.5.2 Lateral Displacements

These can be essentially independent of loading and footing penetration levels through careful design of tank walls, or their support. The present studies show that even for the analysed conditions, lateral displacements can be made to have negligible effect by making the side walls sufficiently stiff, and by use of a wall surface treatment that reduces soil particle penetration. Their greatest influence was shown to be on longitudinal and lateral stress fields and on vertical displacements. For the tests proposed, bearing capacities would therefore have been essentially unaffected, although measured displacements of the footings would have been larger than for a real plane strain situation.

The present study confirms qualitatively the findings of Ko and Davidson (1973) (see 3.2.2). Their tank geometry is equivalent to a tank 0.6 m wide and 1.75 m deep in the present study. Their lateral displacements (when considered in terms of lateral strains) are equivalent to approximately 60% (top of tank clamped) and 800% (unclamped) of values imposed in the present study. Figure 20 shows that in the upper regions of the sample, for a tank 0.6 m wide, the sign of the vertical stress field is opposite to that obtained

in a plane strain situation under the imposed loading, so that the effect would be substractive. The increase in lateral displacements by a factor of 12 would substantially increase the amount by which measured bearing capacity was reduced. In the present study also,an eight-fold increase in imposed lateral displacements would have introduced marked modification of bearing capacity.

3.5.3 Side Friction

In considering the two extreme conditions of limiting friction and zero friction, the results of the present study bracket the real situation. No interface between soil and tank walls can be truly frictionless, and yet some movement will tend to occur in the presence of friction when shear forces become great enough. The tabulation of coefficients of limiting friction, Figures 30 to 32, and 34, when compared with measured values of coefficients, Table 4, shows that an efficient tank wall treatment ought to have a coefficient of friction of up to an order of magnitude smaller than that required to fully restrain the boundary layers of soil. Conditions in the tank are therefore likely to be intermediate between those considered, and in fact could be closer to the zero friction case.

When the results of the side friction case studies are considered with this allowance for likely real friction levels in mind, a marked reduction in their apparent significance is possible. Nevertheless, it is clear that the narrower tanks, representing low width-to-depth ratios, will cause greater departure from plane strain conditions than will wider ones. This confirms previous experimental studies.

Present computing techniques do not readily allow a modelling of restraint corresponding to a variable friction less than limiting, and so a quantitative analysis of this realistic condition is not possible. It is therefore impossible to assess previous studies and theories using the results of this present study. As this is the case, no attempt has been made to investigate across-the-tank changes in stress and deformation fields. It is a matter of problem definition in the present study that differences exist between outer boundaries and the centre line of the tank.

3.5.4 Effects of Varying Tank Depth

In an attempt to reduce the influence of side boundaries by effectively increasing width-to-depth ratios without increasing the already large volume of soil needed to make up the sample, a reduction in tank depth from 1.75 m to 1.0 m was considered and analysed.

It was found that for such a smaller tank, if loading levels were unaltered, levels of stress, strain and deformation were considerably lowered, making experimental measurements more difficult. As an example, the effect on surface deformation is shown in Figure 33. The depth reduction brings the base of the tank close enough to the footing for it to be necessary to measure stress distributions on the base to arrive at an overall picture of behaviour. Figure 33 shows that the changes in level of shear and especially normal stresses are such that absolutely small quantities, or the small difference between large quantities, would have to be measured. These complicating features of tank depth reduction are a distinct disadvantage, which may even overshadow its advantage in terms of reducing boundary effects.

3.6 CONCLUSION

It was found that significant influence on plane strain conditions could result from tank side wall lateral displacement and friction for the scheme considered. Design and detailing could reduce lateral displacements to insignificant levels. The effects of friction could not be effectively controlled, however, and could be significant.

These findings were considered in conjunction with other potential complicating features of the plane strain approach. These included the large volume of soil needed to make up the sample, the probable high cost of purchasing or developing instrumentation and logging equipment to fully monitor soil, footing and tank behaviour, and the length of time it could take to make the system operational. Together they came to be a basis for reservation about a plane strain testing facility of the type proposed. It was also becoming clear that a need existed for experimental work on ultimate bearing capacity and stability (rather than working stress level behaviour).

Consequently it was decided to abandon the exclusively plane strain testing facility in favour of a more general three-dimensional one, for which simpler objectives would be set, and which could be modified to allow testing under substantially plane strain conditions.

CHAPTER 4

PILOT STUDY: 229 mm OEDEMETER AND FOOTING TESTS

4.1 INTRODUCTION

Much of the performance specification for the consolidation phase of the large scale three-dimensional model testing (see 5.3, 5.4) was based on the results of soils laboratory experimental programmes. These involved investigation of the properties of the selected Norfolk clay (see 5.2) using 100 mm oedometer, simple (monotonic) triaxial and cyclic triaxial apparati in association with standard classification tests (El-Ghamrawy, 1977, Hight, 1977). Footing behaviour was predicted using the results of these experiments in association with classicial bearing capacity theory as summarised and extended by Vaughan et al (1976).

The availability of the 229 mm (9 inch) diameter oedometer apparatus for preparation of Norfolk clay samples for vertical load footing tests, made possible experimental verification on a reasonable scale of aspects of the proposed large scale experiments. It allowed pilot tests to confirm the suitability of proposed consolidating and loading sequences, and to verify specified limits for footing penetration. The study gave an opportunity of gaining experience with quantities of clay too large to be hand mixed. Possible problems in sample preparation, handling and quality control could be identified and solved prior to large scale model testing.

Four methods of sample consolidation were tried in the series of 14 tests, with moisture contents taken initially and after consolidation.

4.2 APPARATUS

The 229 mm oedometer apparatus is shown generally in Figure 35. The top surface is free draining, while bottom surface drainage can be controlled. In the standard arrangement, load is applied to the rigid top plate by the second-order lever system through the load yoke. The various knife edges which provide support or transfer load are arranged so that no change in verticality of loading results from the change in lever slope that follows from sample surface movement. It is possible to avoid running out of travel with the load lever by adjusting the fulcrum level with a manually operated hydraulic jack. The system is balanced so that no load, other than top plate (equivalent to approximately 2.5 kPa), comes onto the sample until weights are added to the weight carriers. A modification of the standard apparatus, Figure 35, allows the top surface of the sample to be loaded by air-pressure filled neoprene latex bags.

The bearing capacity tests, using a 63.5 mm diameter footing, were carried out on a modified 5 tonne triaxial test press. The footing, attached to a load cell (developed at Imperial College, giving readings on a Sangamo Strain Box), was held in the upper part of the press. The sample in the oedometer was driven up against the footing by a motor-driven screw jack incorporated in the lower part of the press. Screw jack extension rate was controlled through a reduction gear-box attached to the motor, and set at 2.47 mm/minute. Later work carried out by Sano (1977) on rate effects in (footing) penetration tests confirmed that a rate of 2.47 mm/minute gave essentially undrained conditions under the footing.

4.3 SAMPLE PREPARATION

The sample of Norfolk Clay* used in the tests was initially in blocks sealed with wax, as obtained from site. In the interests of uniformity with laboratory test carried out on other samples of the same material on which the procedure was carried out, the material was, after dewaxing, broken down, dried, ground and passed through a 3 mm sieve, in order to remove large stone and shell fragments. Particles eliminated in this way would not have presented serious problems in the 229 mm oedometer, but could have been significant in the smaller scale 100 mm oedometer and triaxial tests. Inevitably, some modification of the material grading must have resulted from grinding, through crushing and through dust loss. The grinding and sieving was only carried out once, as subsequently the oedometer samples were made up from moist material from the previous test. Any modification in grading (and hence, potentially, in properties) therefore occurred only once at the beginning, with no modification during the test programme. A typical grading curve for the material is given in Figure 36.

The choice of a target moisture content of 35%* was based on a balancing of two considerations. Handling of the material becomes progressively easier with increasing moisture content. However, El-Ghamrawy (1977) had shown that for moisture content much greater than 35%, segregation could occur with this material. Mixing the clay to a uniform consistency around the target moisture content proved to be difficult with the mixer available, both for the initially dry material, and for the moist material from one test being reconstituted for the next one. Once wetted, the very cohesive material tended to form a layer around dryer lumps of clay which were then resistent to the action of the mixer blades.

* Plastic and liquid limits are given in Table 9; grading curves in Figure 36; moisture content vs undrained shear strength curves in Figure 42.

Dryer than average material also tended to be pressed onto the side and the base of the mixing bowl beyond the reach of the blades and needed to be regularly removed manually. Even then, mixing time was such that it would have been prohibitively long for any large scale mixing.

Large (100 gm) samples were taken as the clay was placed in layers into the oedometer. This was done to determine the degree of uniformity of moisture content throughout the sample prior to consolidation.

4.4 SAMPLE CONSOLIDATION

4.4.1 General

As the testing programme proceeded, the approach to the consolidation of samples developed, based on growing experience of conditions. This led to four distinct methods of consolidation, in which the progression reflects an increasingly close reproduction of expected conditions in the large scale test programme. The purpose of the tests was to give broad verification of proposed design and procedures for the large scale testing and so they were not seen as forming a major self-contained study. With each test taking approximately 10 to 12 days to set up and complete, the number of tests in each group was kept purposefully low, the number of tests performed being governed by the need to be able to make adequate critical evaluation and to isolate problems.

Consolidation load sequences are set out below. Consolidation pressures are given in Table 8. The values of pressure given for Tests 1 to 8 are mean values of pressure, as it was not until these had been completed that it was discovered that the knife edge support for sample loading had been misplaced, and that load was therefore bypassing the knife edge.

Cell calibration showed that this resulted in a variable load magnification factor for the lever system, Figure 37. Back analysis of the test results gave the typical ranges of operation shown in Figure 37. For the fourth and last load increment, sample consolidation was generally such that at the end of consolidation, a load magnification factor was in effect which was close to that appropriate to normal operating conditions. The reasonably close agreement of moisture content distributions under the two conditions tends to confirm this, and so the results have been retained.

On completion of consolidation 16 mm diameter cores were taken, and cut into approximately 10 mm long segments so that final moisture content distribution could be assessed.

4.4.2 Consolidating Load Sequence 1: Tests 1 and 2

When an attempt was made to begin consolidating the first sample by applying direct top surface loading, clay squeezed out around the top plate. In an attempt to reduce this it was decided to try stiffening the sample by applying a partial vacuum to the underside of the sample prior to top surface loading. The top surface was sealed with polytene sheeting and kept moist.

Difficulty was encountered with maintaining a uniform level of vacuum. When a high proportion of total consolidation under vacuum had taken place, the sample, in the absence of a confining vertical top surface pressure, shrank away from the side of the cell, allowing surface water to drain to the lower porous stone, and be removed. The samples failed to remain fully saturated under these conditions, which also was unsatisfactory.

On subsequent application of top surface load, large initial bedding down was recorded, appropriate to the volume of material needed to fill the shrinkage gap at the side of the cell. A full top surface load sequence was applied. The results of the footing tests carried out on these samples are given in the interests of completeness only.

4.4.3 Consolidating Load Sequence 2: Tests 3, 4 and 12

Following the unsuccessful use of partial vacuum alone as the initial consolidating force, some squeezing out of the sample was accepted as probably inevitable after placing the top plate in position and applying the first load increment. Slight modification to the top plate (to facilitate handling), and improvement in the technique in placing it, helped to reduce the amount of squeezing out. This bedding down is reflected in the consolidation settlement versus time plot, Figure 38, by the apparent shift of the zero settlement point.

This method of consolidation, with no imposed pore water pressure gradient, was seen as the basic one against which others would be compared, both in terms of final moisture content distributions and of footing bearing capacities. It represents the simplest of oedometer conditions.

4.4.4 Consolidating Load Sequence 3: Tests 5 to 11

The application of a partial vacuum to the underside of the sample during consolidation was set as a major design feature of the large scale test facility. It was seen as a means of controlling moisture content, and hence strength, profile with depth (see 5.3.3.2). This series of tests represented an attempt to investigate the influence on the sample of applying a partial vacuum at the base simultaneously with the application of top surface loading.

The effect of applying partial vacuum is to increase the effective stress at the underside of the sample in relation to that at the top, through decreasing the pore water pressure. This has greater influence during the first load increment stage, when total stress imposed by the top plate loading is small. In consequence, there is an increase in the proportion of total consolidation that takes place during the first load increment. (Compare Figures 38 and 39). The effect on moisture contents and footing capacities is discussed in sections 4.5 and 4.6.

4.4.5 Consolidating Load Sequence 4: Tests 13 and 14

In the large scale tests, in order to impose a uniform surface pressure rather than uniform displacement (see 5.3.3.1), the clay was to be loaded by air pressure in a bag. A modification of the 229 mm oedometer allowed this condition to be tried as part of the overall programme. The modification of the cell allowed an early investigation of bag and filter making techniques on a manageable scale, prior to full scale production.

Additionally, the tests gave an opportunity to check whether significant differences in sample properties could be detected when load and partial vacuum levels were the same as previously in Tests 5 to 11.

4.5 MOISTURE CONTENTS

The scale of the testing programme was such that it warranted no special quality control procedures.

Moisture contents of samples taken during placing of the clay in the oedometer were used to provide basic information about the uniformity of the bed at the beginning of the consolidation process. Table 7

gives mean initial moisture contents and their variations. For the balances used to weigh samples and containers, a variation of approximately ±0.2% represents the confidence limit in readings taken, and so the clay beds, with local variations in moisture content ranging between -0.8% and +0.4% (in approximately 35%) can be said to have been reasonably uniform at time of placing.

The cores taken from the clay bed after consolidation provided an indication of moisture content profiles. El-Ghamrawy (1977) had shown that a generally reproducible relationship existed between moisture content and undrained shear strength for the Norfolk clay, Figure 42. Moisture content was therefore taken in a qualitative way to assess possible changes in strength profile resulting from variation in consolidating procedures. Mean levels of final moisture content and their variations from top to bottom of the clay bed are given in Table 7 with some profiles plotted in Figure 41. A complete presentation of profiles and other results is given in Herbertson, Hight and El-Ghamrawy (1977). For these moisture contents, a variation in value of ±0.1% (in approximately 15%) represents a likely level of confidence of readings, using more refined balances in conjunction with smaller samples.

Beds prepared for Tests 1 and 2 (with initial partial vacuum only) showed a reduction in moisture content from bottom to top of the bed, possibly indicating that the top region of the beds had failed to become resaturated (see 4.2.2). For Tests 3, 4 and 12 (no vacuum) a basically uniform profile was obtained. For Tests 5 and 6 no trend away from uniformity was found, although a partial vacuum of -35kPa had been applied. When the level of partial vacuum was increased to -70kPa, Tests 7 to 11, a trend towards decreasing moisture content with depth was identified. Test 14 showed a similar trend.

The cores were in all cases cut up into only 5 or 6 segments (so that reasonably accurate results could be obtained from the weighing) so an averaging of moisture contents will have taken place that makes a detailed analysis of the profiles inappropriate.

4.6 FOOTING TEST RESULTS

Load-deformation curves for the footing tests are given in Figures 40. ~~and 41~~. Initial lack of fit between footing and clay bed, and bedding down were probably responsible for the initial variable deformations which were recorded before basically linear behaviour began. The extent of linear behaviour, the slope of curves for these sections and the level of bearing stress at which behaviour became non-linear varied from test to test. Footing ultimate bearing capacity was, for uniformity, defined as the bearing capacity corresponding to a footing penetration of 10 mm, well beyond the end of linear behaviour in all cases. Values of these are given in Table 7. The tests were not sensitive enough to establish whether the differences which existed could be conclusively attributed to bed preparation techniques.

It seems likely that a refinement of the footing test procedures, both in terms of reducing initial lack of fit due to sample surface irregularities and of load cell data recording, would have resulted in better definition of load deformation curves, particularly of the point at which significant non-linearity begins. For the purposes of the present study, however, the footing tests provided the basic confirmation required of ultimate capacities. More importantly, allowing for scaling effects, quite reasonable indications of vertical penetration of the footing into the Norfolk Clay were established. Values of undrained shear strength back-calculated from footing ultimate

capacities gave close agreement with the work of El-Ghamrawy (1977) when considered in association with top-of-bed moisture contents, Figure 42.

4.7 DISCUSSION

The 229 mm oedometer consolidation programme and the associated footing tests, proved useful in the context of the large scale model testing facility in confirming the suitability of design and performance specifications, and in isolating areas of potential difficulty.

On the basis of the experience gained with the Norfolk clay, it was decided that a more efficient mixing system would have to be used. It would have to be able to reduce the time required to mix while at the same time increasing the degree to which it could break down clay lumps in the wet clay-matrix to an acceptably small size. The mixer would also have to be able to clear material from the side and base of the mixing bowl. In the interest of avoiding sample modification through loss of fines, dry mixing should be avoided.

Moisture content determination could still be used to confirm uniformity of the large clay beds before consolidation, but the experiments showed that another means would have to be found for obtaining post consolidation strength-with-depth profiles quickly and directly. Even if the relationship between moisture content and undrained shear strength had been a unique one, the practical obstacles and time consuming nature of sampling at many sections at close intervals argued against using this as the method of assessment. Therefore, an investigation was undertaken to establish rate dependency and scale effects, for probes driven into beds of the Norfolk clay. As a result of this a probe was

developed which will allow in-situ determination of strength-with-depth profiles (Sano, 1977).

The consolidation procedures showed that the clay is liable to squeeze out around the edge of the loading bag at low pressures at moisture contents which facilitate mixing and handling. This should produce only minor initial irregularities in the clay sample, which should be overcome by effects of later load increments. It seems that some further work must be done in the large scale facility with the strength measurement probe, in order to establish the optimum level of partial vacuum needed to provide a uniform increase of strength-with-depth of the order of magnitude required.

The footing tests confirmed that estimates of ultimate capacity and penetrations for the large scale tests were generous, and allow quite reasonable room for varying test conditions. From the viewpoint of footing capacities, a much stiffer clay bed can be coped with, or, alternatively, in terms of movements allowed for, a much less stiff bed can be used.

Difficulty of lowering the footing down onto the clay bed and ensuring that the clay surface is level and parallel to the underside of the footing was found in this study, and possibly contributed to differences in the load deformation curves. It is an aspect of the test procedure which needs improvement in the large scale testing.

CHAPTER 5

ASPECTS OF THE DESIGN OF THE LARGE SCALE THREE-DIMENSIONAL MODEL TESTING FACILITY

5.1 INTRODUCTION

The decision to undertake a three-dimensional footing test programme with clay as the bed material was taken after preliminary design and assessment of a plane strain approach. This had shown that plane strain conditions could be difficult to reproduce with sufficient accuracy in the laboratory situation and that development of the complex facility proposed could be too lengthy and costly for the sponsored programme. There was a growing awareness that a need existed for three-dimensional experimental work on bearing capacities and stability on a scale intermediate between normal laboratory footing test studies, (with footing diameters typically up to 75 mm) and full scale testing of footings (Muhs and Weiss, 1972) or prototype model testing as in the Ostflag 9, Christchurch Bay gravity structure (Penman and Gallagher, 1976).

A scheme was proposed incorporating a bed of Norfolk clay, which is similar in its properties and behaviour to some of the clays found on the North Sea bed (Hight, 1977). This clay has the practical advantage that samples are much more easily and economically obtained than is possible with actual North Sea bed material.

The clay bed would be formed in a circular tank, and would be loaded by either a circular or a square footing, initially. Relative proportions of bed and footing were to be such that the side boundaries at least would be remote. A footing diameter of 300 mm was chosen, based on which a clay bed diameter of approximately 1500 mm with depth of the order of 300 mm was selected. The plane strain computer studies

had shown that the base would, under these conditions, influence conditions within the clay under the footing during test. However, it falls outside the region of possible failure surfaces for both direct vertical loading and inclined eccentric vertical loading (the resultant of direct vertical and eccentric horizontal loads).

Interest would centre initially on ultimate behaviour under selected combinations of vertical and eccentric horizontal loads.

Allowance was to be made in the design of the experimental facility for conversion to a plane strain system of testing so that an experimental link could be made between the three-dimensional behaviour and plane strain behaviour, for which closed-form bearing capacity solutions exist.

5.2 THE NORFOLK CLAY

The clay chosen for the tests can be found in the base of cliffs along the North Sea beaches near Happisburgh in Norfolk. The clay deposit underlies depths of up to 60 metres of sand and organic soils, and is exposed by wave action (despite extensive sea defences) for heights of up to several metres above beach sand deposits. The outcrops thus formed show some variability from place to place along the coast and so a location was chosen close to where previous undisturbed samples had been taken for extensive soils laboratory testing (El-Ghamrawy, 1977, Hight,1977).

A preliminary sampling of the clay in three locations at and adjacent to the one chosen for the taking of the bulk quantity of clay required for the large scale testing, showed a reasonable degree of uniformity, Table 9. Samples taken from the 3 tonnes of clay actually excavated for use in testing again proved to be sensibly uniform, Table 9. This meant that no further mixing of the material was needed over and above

that which had occurred during excavation and packing of clay into polythene bags for transport.

Experience with mixing the clay to the required moisture content in batches too large to be hand worked (see 4.3) had identified the need for an efficient mixer with blades capable of cleaning material from the side and base of the bowl. The mixer available for large scale mixing(in the Concrete Laboratory) was able to do this, but was unable to break down lumps of drier clay which were protected by the wetted clay matrix which formed a coating around them. Trial mixing with varying moisture contents and sequence of water/clay addition were not able to overcome this problem. Times for even imperfect mixing were prohibitively long when it was considered that 10 mix batches would be needed to fill the large consolidation tank.

The clay lumps could be broken down or removed by sieving, and small scale work showed that vibration increased sieving efficiency markedly and eliminated much of the manual effort. A large size sieve (1200 mm by 600 mm) was fabricated incorporating a 9 mm mesh size over 5 mm main mesh. Vibration of the whole sieve unit with its frame on a large vibrating table failed to reproduce small scale results, probably through too high a damping effect of the frame. When the meshes were vibrated locally by a hand held vibrator, a high throughput was achieved, and so this method was adopted. Lumps (greater than 5 mm size) were returned to the mixing bowl for inclusion in the next mix for possible reduction. The distribution of lumps smaller than 5 mm throughout the clay bed was considered to be insignificant, particularly as their moisture content would be similar to that of the clay matrix after consolidation.

Small samples of the clay were taken from the bulk sample for further soils laboratory testing. Triaxial and hollow cylinder tests would be carried out following the same stress paths as are followed by some elements in the clay bed during consolidation and footing testing, to provide constitutive relationships for the companion computer program development and analysis.

5.3 CONSOLIDATION PHASE

5.3.1 Design Objectives

A system was to be devised to allow controlled consolidation of a clay bed of the order of 1500 mm diameter by 300 mm deep.

It was to be possible to apply top surface consolidating load in and to reduce it again under controlled conditions. This would allow swelling of the sample to produce an overconsolidated bed which would exhibit lower creep behaviour than a normally consolidated bed and more importantly, would simulate more closely North Sea beds. Top surface loading was to be in the form of a uniform pressure rather than a rigid uniform displacement. On final total removal of consolidating pressure, rapid exposure of the surface was required, to allow drying and sealing. This would minimise reabsorption into the clay of free water in the filter, which would result in a layer of clay of undesirably lower-than-average shear strength at the top surface of the clay.

Additionally, allowance was to be made for possibly variable partial vacuum to be applied to the underside of the clay bed. The partial vacuum in conjunction with top surface consolidating pressure would set up a pore pressure gradient within the clay bed which would ultimately lead to a decrease in moisture content with depth under equilibrium

conditions. This would result in an increase in (undrained) shear strength with depth in the clay bed, (Vaughan, 1976), a feature of prototype foundations which it was felt to be particularly important to model.

Top and bottom surface drainage was to be possible with or without back pressure. The top surface settlement of the clay bed was to be monitored during consolidation, and allowance was to be made for the possible inclusion of pore pressure measurement probes in the bed.

5.3.2 Consolidation Tanks

A general section through a consolidation tank is shown in Figure 43. Full details in the working drawings for fabrication are given in the Appendix to the Figures in sketches SK1 to SK11. The tank, lid and various sealing annuli are of welded mild steel construction. The tank has a horizontal stiffener near the top and twelve vertical stiffeners around the outer circumference. During consolidation radial displacement of the walls is calculated to be less than 0.00012 m, while maximum bending deflection between the top stiffener and the base plate (ignoring the influence of vertical stiffeners) is conservatively estimated at 0.00025 m. During footing tests, the level of displacement will be at least an order of magnitude smaller than during consolidation. Plane strain studies, applicable to simple radial stress conditions, and three-dimensional studies, had shown qualitatively (see 3.4.2) that this level of lateral displacement is insignificant for a bed of diameter to depth ratio of approximately 5. Side walls are covered by a thin grease film/rubber membrane system to reduce side friction effects, which would be low for this tank geometry, even in the presence of side wall limiting friction (see 3.4.3).

The parallel double stiffeners for the lid and base plate with the associated "quick release" bolts allow the tank to be self-contained with regard to consolidation operations. The four 600 kN "quick release" bolts obviate the need for elaborate bolting down of the top plate to the tank, and so allow rapid removal of the lid, loading bag and filter at the end of consolidation to expose the clay surface.

On the basis of experience with the 229 mm oedometer clay samples (with clay bed depths typically of the order of 75 mm) consolidation times for the proposed 300 mm clay beds were estimated to be of the order of ten weeks. To allow a speedier turn-round for the large scale tests, two complete consolidation tank units were fabricated.

5.3.3 Consolidation Loading

5.3.3.1 Top surface loading. During the development of the experimental scheme proposals it had been decided to adopt a maximum consolidating pressure of 350 kPa, and to apply this in the form of a uniform pressure to the surface of the bed in order to obtain uniform bed properties. It would have been essentially easier to apply a rigid surface load which, however, through the imposition of a uniform surface displacement, would have resulted in non-uniform properties, as contact pressure distributions under rigid plates are not uniform and side friction effects are not localised (Rowe and Barden, 1966).

Ideally, the pressure should be applied hydraulically using oil or water contained in a suitable bag. It can be seen on Figure 43 that for the design adopted the volume of fluid in a loading bag will be large (up to 400 litres) owing to the large sample consolidation settlements allowed for. Practical considerations associated with the need to be able to rapidly expose the bed surface after consolidation was complete, led to the adoption of compressed air to provide the required pressures.

While air pressure cannot be depleted as rapidly as can water or oil pressure, air presents no collection or disposal problem and any air remaining in the bag has negligible weight. Figures 44 and 45 show the basic scheme for providing loading and for its control and monitoring.

A high quality neoprene latex 0.8 mm thick, Table 10, was chosen for loading bag fabrication. It is normally used where resistance to weathering, ageing and oils is required, and has a reasonably high tensile strength associated with 880% elongation at rupture. The adhesive used in the fabrication of the bags, Table 10, had been specially developed by Dunlop for use in the manufacture of inflatable rubber life rafts for the Royal Navy.

Trial fabrication of bags of 100 mm and 229 mm diameter demonstrated the suitability of use of the adhesive with the neoprene latex, and allowed modification and refinement of original bag design. The bags are made up from three parts: base disc, side cylinder and top annulus. Although the neoprene latex has a high elongation under direct load,when made up into a bag it demonstrates a prefernece for adopting a balloon shape under pressure rather than for an extension of the side cylinder in order to maintain the shape required. The side cylinders of the large scale bags therefore incorporate sufficient material to allow the bags to follow clay bed settlement without any stretch. Initially excess side cylinder depth is reduced by folding. The outside of the cylinder is lubricated to reduce frictional resistance to sliding.

The top annulus of the bag is clamped between two steel annuli for sealing. Once this seal is successfully made it need not be broken except when the bag needs replacing. This facilitates assembly of the full lid/ annuli system which is already a lengthy feature of the preparation of

each clay bed for consolidation. Specially made hardened liquid-rubber O-rings are used in conjunction with neoprene latex gaskets to provide the seals necessary for maintenance of pressure. Full details are given in the Appendix to the Figures in sketches SK6, SK7 and GK1.

5.3.3.2 Underside partial vacuum. The capacity to produce an increase in strength with depth in the clay bed is central to the design of the consolidation system. For foundations under large raft footings the depth to which soil is mobilised for the support of the structure is such that significant increase in strength occurs with depth, due to the consolidating effect of the weight of the soil above. The increasing strength influences footing behaviour. In the laboratory situation, this increase in strength with depth has been reproduced in samples whose depth is insufficient to provide appropriate gravity effects, by subjecting the samples either to centrifugal forces (Avgherinos and Schofield, 1969, Rowe, 1975) or to a hydraulic gradient (Zelikson, 1969). Neither method was considered ideal for the proposed system, on the one hand because of its size and on the other because of the anticipated problems of providing a hydraulic gradient. The application of the partial vacuum to the underside of the sample during consolidation can lead to a setting up of an increasing strength with depth profile. This is achieved by increasing the effective stress at the bottom of the bed relative to the top, through modification of pore water pressure (Vaughan, 1976). Strength profile is measured by means of a specially developed probe (Sano, 1977).

In practice, the application of a partial vacuum to the underside of the clay bed is made simple by the more general need for underside drainage. Provision is made to connect the vacuum system to the

outlet of the drainage lines, which are so proportioned as to not restrict the level of vacuum.

A practical problem associated with applying vacuum to a clay bed is that of protecting the vacuum pump from the water which is drawn (and expelled) out of the clay. To overcome this, traps are installed in the vacuum system between the clay beds and the pump. Traps are spring-hung at one end so that their settlement during filling with water can be monitored with a linear displacement transducer and thus provide a measure of the weight of water drawn from the clay. To guard against overflow of the traps, a float switch in a sump provides further protection for the pump. Potentially, the volume of water in the traps is such that drying of the air in the vacuum system is not feasible. Even with the installation of a large capacity pump which can cope with large quantities of water vapour, frequent pump oil-changes are necessary to maintain pump efficiency by replacing emulsifying oil. Exhaust air from the pump is passed through an oil mist filter for cleaning.

Details of the vacuum system, control board and sump are given in Figures 44 and 45, and in the Appendix to the Figures in sketches SK100 and SK101. Details of components are given in Table 10.

5.3.4 Filters and Drainage

In smaller scale experimental work it is common to use porous stone discs to provide for sample drainage. This was not feasible on the scale of the present work, and so a filter, potentially efficient in transmission of water vertically away from the clay surface and horizontally to the drainage outlets, was designed.

A complicating feature of the top surface filter is that it must follow clay bed consolidation settlement. It therefore moves in relation to the fixed tank walls and tank lid/sealing annuli system. To overcome this, a drainage system was developed which incorporates a fixed part, attached to the sealing annuli and connected through the lid to drainage lines, and a moving part, bonded over holes in the neoprene latex loading bag, to collect water from the filter. The two are joined by a spiral nylon hose which can readily extend, Figure 43. Six drainage points are provided.

Bottom filter drainage is relatively simple, and is provided through pipes connected to four holes in the tank base plate. Details of both systems are given in the Appendix to the Figures in sketches SK101 to SK103.

The filters themselves are of sandwich construction. A sheet of highly efficient commercial filter material, Table 10, allows more than adequate drainage of water out of the clay without itself becoming clogged with clay or silt particles. This is backed by two layers of fibre-glass woven roving, the open weave of which allows water to flow horizontally to the drainage points, even under design-loading bag pressures. These three layers are enclosed in a fine nylon mesh, and the whole sewn together. Filters are cut to shape, and the loose edges so formed are impregnated with liquid rubber to form a stiff outer rim. Bottom filters are protected by a metal mesh so that they will not be damaged during removal of the clay after testing. Initially, top filters are being placed separately, but a later development in assembly technique could involve attaching them to the underside of the bag to shorten the time in which the clay bed can be exposed after consolidation is complete.

Provision is made in the tank design for consolidation under back pressure. An inflatable tube seals the gap between the lid/sealing annuli system and the side of the tank to prevent the escape of water (see Figure 43, and Appendix to the Figures sketch GK1), and back pressure can thus be maintained in the top drainage system. Either back pressure or partial vacuum can be applied to the underside of the clay bed.

An important design requirement was the minimisation of free water in the filter/top drainage system at the end of consolidation. Allowance has been made for water to be expelled from the fixed part of the top drainage system using low pressure compressed air. A layer (25 mm uncompressed) of foam rubber between the loading bag and the filter expands as bag pressure is reduced and draws up water from the filter and spiral hoses.

The layer formed between the loading bag and the clay surface is not totally flexible, but is still considered to be sufficiently flexible to allow essentially uniform pressure to be applied.

5.3.5 Surface Settlement Recording

As vertical displacement transducers are not guaranteed to function well in moist environments such as those associated with compressed air systems, the transducer, Table 10, for recording the vertical settlement of the clay surface during consolidation had to be mounted outside the tank. A weighted brass rod passing through a cap plate in the lid, Figure 43, rests on a 100 mm diameter perspex disc bonded to the loading bag, and travels down with the bag/filter system. The rod is not bonded directly to the disc in order to allow it to move freely in response to clay bed behaviour. The plate spreads the weight of the rod so that early during consolidation, when the clay

is virtually a slurry, no indentation occurs. Later, when high pressure has been built up in the bag, the weight reacts to the upward pressure on the rod and holds it down. The weight also acts to provide a force for overcoming friction between the rod and the O-ring seals in the cap plate.

5.4 MODEL FOOTING TESTS

5.4.1 Design Objectives

Three-dimensional footing tests were to be carried out using the previously prepared Norfolk clay beds as foundations. Relative stiffness of the model footing and clay were to be similar to that of prototype raft footings on the sea bed. It was to be possible to vary the shape of the model footing to reflect variation in raft design and to suit eventual modification of the tanks for plane strain testing.

Provision was to be made for subjecting the model footings to vertical and (eccentric) horizontal loading. In the first instance, this was to be unidirectional horizontal loading, but the test facility should allow for cyclic horizontal loading, of either simple or fully random variation. Load systems were also to allow for reasonable variation in loading levels, both for three-dimensional and later plane strain testing. Line of action of applied forces was to show insignificant sensitivity to 75 mm of vertical displacement, 40 mm horizontal displacement and 20 degrees tilt of the footing. Similarly, monitoring of footing displacements should not be influenced by their absolute magnitudes. Loading was to be to ultimate footing capacity, although pre-failure load-displacement behaviour was to be reasonably recorded. The tests were to be carried out in the absence of a general surcharge of the clay bed.

Initially the footings were to be plane without in-built instrumentation. Later modification would include instrumentation for monitoring contact stress distributions and would possibly incorporate non-planar contact surfaces.

5.4.2 Model Footings

In order to set the scale of the three-dimensional testing, a model footing dimension of 300 mm was selected. This falls intermediate between small scale test footing dimensions and full scale, and allows subsequent instrumentation of the underside of the footing to provide reasonable resolution of contact stress distributions. The basic footing was chosen to be circular, of diameter 300 mm, with a comparison square footing of the same area, with side length 265 mm. Comparison of results for the two footings under identical (simple) loading would allow the calculation of shape factors, and thus eliminate the need for full test programmes for both footing types.

Relative stiffness of prototype raft footings is of the order of 10 (Hight, 1977). Computer studies (Boswell and Scott, 1975) have shown that only minor differences in footing behaviour occur when relative stiffness is increased from 10 to "infinity". For convenience, therefore, the model footings were made practically rigid, Figure 46.

The footings are of bright steel, with machined plane surfaces (treated to prevent corrosion) and are connected to the main loading frame by a stiff stub column to which they are bolted. Footings can be readily interchanged. When, in later testing, an instrumented footing of different thickness is used, modification of the length of the stub column restores the level of the underside of the footing which acts as datum level for the loading system.

Initial potential lack of fit between the underside of the footing and a possibly non-planar clay bed surface is eliminated by coating the surface of the clay over the contact area with quick drying compound and founding the footing on this. Once set, the compound is quite stiff, and, because of the thinness of the applied layer, does not affect footing behaviour.

5.4.3 Estimated Model Footing and Load System Capacities

The undrained shear strength of the Norfolk clay at approximately the target moisture content (15%) for the proposed level of overconsolidation has been found to be of the order of 100 kPa (El-Ghamrawy, 1977). Based on this, and using the approach presented by Vaughan et al (1976), the following bearing capacities at failure (q_f:vertical, q_h:horizontal) can be calculated for the 300 mm diameter footing.

Vertical load only:	q_f = 515 kPa	≡ 36 kN
Eccentric, inclined load with angle of inclination of 21 degrees to the vertical (equivalent to vertical load plus eccentric horizontal load)	q_f = 255 kPa	≡ 18 kN
	q_h = 100 kPa	≡ 7 kN

The angle of inclination of 21 degrees represents the change-over point between sub-surface and surface failure. It was planned to limit loading conditions to those producing sub-surface failure.

It was decided that to allow sensible control of footing loading this would be strain (rate) controlled and be provided by hydraulic jacks. So that there would be a reasonable margin of jack capacities over estimated load requirements, vertical load would be provided by a 100 kN jack, unidirectional horizontal by a 25 kN jack. In order to

allow cycling of horizontal loads, one jack could be set at $P < 25$ kN while the other opposing jack would cycle between Q and S kN with $0 \leq Q < P$ and $2P \geq S > P$, resulting in a capacity to produce up to ±25 kN peak horizontal load.

The various elements of the loading system were designed to be able to withstand jack forces rather than estimated footing capacities with acceptable safety margins and deflection levels. Table 11 gives a summary of design loads and capacities provided in various critical elements.

5.4.4 Load System

The need to accommodate substantial footing displacements during testing (75 mm vertically, 40 mm horizontally and 20 degrees of tilt) had a fundamental influence on the development of the load system. In order to maintain the line of action of the applied loads in the presence of these displacements, at least two systems are possible. Either a complex compensating system must be developed to allow load jacks to move laterally, parallel to the orthogonal footing movement during the test, or fixed load application must be at such a distance from the footing that displacements cause insignificantly small angular changes in the line of action. Additionally, vertical load ought to be applied at the level of the underside of the footing.

In view of this and because tank design for the consolidation phase resulted in the top of the tank being up to 250 mm above the clay bed surface, thus severely restricting the possibility of direct horizontal loading of an extension of the footing plate, it was decided to apply the load to the footing through a load frame. This straddles the tank, allows application of load in the plane of the

underside of the footing and allows greater choice of lever arm for application of horizontal load. The load frame is connected to the load jacks by a series of links and beams. Link length is such that footing displacements cause negligible change in angle of load application. In order to keep all links in tension and to avoid using wire rope and pulley systems, the level of the tank during footing tests was raised, so that the vertical load system could be located below tank level. Figure 47 gives a schematic representation of the load system, and in the Appendix to the Figures, sketches SK14 to SK26 show full details for fabrication of the load frame, links and load beams. Horizontal jack support columns are shown in SK12, with tank support stand detailed in SK10 and SK11.

In order to minimise the effects of friction, link connections to the load frame and load beams are through dowels and PTFE impregnated bearings, Table 10, which, for the small dowell diameters used, give negligible resisting couples to rotating forces. Links have simple bolted couplings to facilitate installation and dismantling of the load system for each test. To accommodate variable final clay bed surface level, the vertical links incorporate in-line adjustment, while the level of horizontal load beams, jacks and their supports can be varied to satisfy both this situation and the more specific need to provide variable horizontal load eccentricity.

Vertical load link connection to the load frame lies in the plane of the underside of the footing plate. Horizontal load can be made to have an eccentricity in relation to the underside of the plate varying from zero (which in conjunction with centre vertical load produces the equivalent of centre inclined load) to 75 mm and then in 50 mm steps to 325 mm. Practical considerations made it impossible

to have infinitely variable horizontal eccentricities, but the seven locations provided give quite reasonable scope for investigating the behaviour of the footing under eccentric horizontal loading.

The design of the load frame is based on deflection criteria rather than strength. Computer analyses were carried out using a general purpose structural program (Lyons, 1977). These showed that for maximum vertical load (2 x 50 kN) the vertical displacement of the outer arms of the load frame would be a maximum of 3 mm, and that with maximum horizontal load (2 x 12.5 kN) applied at the level of the underside of the footing, horizontal displacement of this point would be a maximum of 4 mm, which would produce an eccentricity of vertical loading of 1.33% of footing diameter. The forces applied in this analysis represent jack capacities and are higher than the estimates of forces actually required to produce ultimate conditions for the footing (see 5.4.2). Therefore, under more likely conditions of loading, frame deformations are much smaller. It would not have been possible to make the load frame significantly stiffer, as even now, as fabricated, it weighs in the region of 2 kN, which can be a significant portion of vertical load at ultimate, under combined vertical and horizontal loading. So as not to reduce the stiffness of the load frame unnecessarily, load measurement is carried out in the links adjacent to the load frame. The only error which will result from this form of measurement is that due to link/load frame connection friction which has been minimised. Details of the load cells are given in Table 10.

Due to the solid design of the load frame, a significant proportion of footing vertical load capacity is provided by the self weight of the frame. While the prime object of the testing is the investigation of ultimate behaviour of the footing, the load-deformation behaviour at pre-failure levels of loading can also provide useful information. Two columns, with synchronised screw jacks for controlled lowering of the load frame and footing onto the clay bed surface, incorporate load cells so that the transfer of frame self weight from the columns to the clay can be accurately monitored. Once the clay is carrying all the frame self weight, loading continues with the jacks and link load cells monitor additional loading.

The decision to make load application points remote from the footing resulted in a rather spacious experimental facility, with a working platform constructed at the level of the underside of the tank. As a consequence of the spaciousness, the various column and platform bases could be positioned without undue congestion. Figure 49 gives a layout of the column and main platform base plates, and shows that even under present conditions, some overlapping of base plates occurs. Details are given in the Appendix to the Figures in sketches SK26 to SK29.

5.4.5 Footing Displacement Monitoring

The behaviour of the footing under load is monitored by a series of linear displacement transducers, Table 10. Readings of four vertical and four groups of two horizontal transducers, Figure 48, enable calculation of mean vertical and horizontal displacement, longitudinal and lateral tilts and axial rotation of the footing. However, for a uniform clay bed and symmetrical loading, lateral displacements and tilts, and axial rotation will be negligible. Under these circumstances, the transducer system contains sufficient units for

internal checking of results to be carried out. Additionally, "dummy" transducers, one for each type used in the monitoring system, identify any possible fluctuations due to power or temperature fluctuations.

As with the load system, transducers must be remote from the footing in order that footing displacements do not influence recorded values in orthogonal directions. The side of the tank projects up to 250 mm above the clay bed surface and so makes direct connection of the footing to horizontal displacement transducers impossible. Additionally, if transducer reference points had been located on the footing, these could, in the presence of large footing displacements, have been made inaccessable by the clay bed projecting above them. A reference plate, connected to the transducers by wires, is located at the top of the stub column to which the footing is attached. Computer analysis showed that this stub column is so stiff that it makes negligible contribution to recorded values through bending or axial shortening. Centre-lines of reference point on the reference plate are in a plane 475 mm above the underside of the footing. In the case of footing tilt, this eccentricity of reference points leads to a recorded horizontal displacement, the magnitude of which can readily be calculated by suitable manipulation of horizontal transducer readings.

To accommodate different initial clay bed surface levels, horizontal transducers can be raised or lowered in their frames at the support columns. In the case of vertical transducers, provision is made for this variation in surface level by varying the length of the wire connecting the transducer to the reference points. Adjustment to the length of the horizontal transducer wires is possible so that initial conditions can be set to best accommodate anticipated horizontal

displacements and tilts. The vertical transducers are attached to a gantry which can be swung on its support column to allow installation and removal of the clay bed tank. Transducer support column layout is given in Figure 49, with fabrication details shown in the Appendix to the Figures in sketches SK30 to SK33.

5.5 DATA LOGGING

The signals from the linear displacement transducers (monitoring clay bed consolidation settlement, vacuum system water trap filling, and model footing displacements during test) and from the load cells (monitoring load applied to the clay foundation by the model footing) are fed into a 30 channel automatic data logger, Table 10. The logger can scan all or a selection of channels, either on manual instruction or at preset time intervals. Scanning of channels can be done at up to 25 channels per second, at which rate, transducers can be read with an accuracy of ±0.005 mm and load cells to ±0.02 kN. The output from the data logger is punched onto tape for further computer processing to produce results in the form required, and is also printed out for visual presentation of unprocessed readings to allow immediate, on-the-spot evaluation.

CHAPTER 6

THE MODEL TESTING FACILITY IN A WIDER CONTEXT

6.1 PRESENT CAPABILITIES

The model testing facility fulfils the specified objectives for both clay bed consolidation and footing tests (see 5.3.1, 5.4.1). In so doing, it also has a certain built-in versatility which will allow it to be used for more varied work. The key to this lies mainly in the capacity to produce large beds of clay under controlled conditions. The extent of boundary effects is made small, so that a large volume of unaffected clay is produced. The only real limitations on preparing beds are the upper bound to consolidating pressure, the level of partial vacuum that can be applied for controlled swelling, and the range of depths of bed that can be produced.

Beds produced can be used to provide samples for soils laboratory testing. Provided that the sampling techniques are satisfactory, large numbers of identical small samples (e.g. for triaxial testing) can be taken from any one bed. Alternatively, the bed can provide one or more large samples (e.g. for the hollow cylinder apparatus; sample 250 mm high with 250 mm external diameter). Although stress history for some in-situ conditions cannot be reproduced, significant variation in bed preparation, and hence samples, can be achieved. Sampling from the prepared beds can remove possible ambiguity in test conditions arising when samples are taken from "undisturbed" material, or from laboratory remoulded material.

The beds of clay prepared allow development of in-situ testing and sampling techniques on a reasonable scale. Small scale pile and

footing tests can also be carried out. In all cases, many tests can be carried out on any one bed, thus eliminating the need to consider possible variations in bed conditions when analysing the test results.

The loading system is designed to allow testing to ultimate footing capacity with the Norfolk clay. Initially, testing is to be at a rate such that conditions in the clay under the footing are essentially undrained. This is representative of foundations of gravity structures in the sea when these are subjected to wave loading. It is under undrained conditions that pore pressure build-up resulting from cyclic loading can lead to possibly detrimental modification of foundation behaviour. In the long term, pore pressure dissipation in the prototype foundation must occur in the presence of the dead load of the structure. The test facility allows the investigation of the effect of this consolidation on footing behaviour.

6.2 FUTURE APPLICATIONS INVOLVING MODIFICATIONS

It has already been mentioned (see 5.4.1) that it is proposed to carry out some plane strain testing in a partitioned three-dimensional test tank using the present loading system. Partitioning and modification of the model footing are relatively simple. However, in order that sensible surface loading and drainage should be provided, a new loading bag with modified drainage outlets will have to be developed.

The fabrication of an annular loading bag (with its associated lid, supports and stiffeners) to fit around the circular footing will allow a surcharge to be applied to the bed during footing tests. Surcharge could also be provided by placing lead shot on the bed surface. In addition to allowing for variation in test conditions for clay beds,

this modification will permit the testing of sand beds at working loads. The loading system is not capable of inducing ultimate conditions with a footing on sand unless the footing diameter is reduced to approximately 100 mm. For the surcharge condition, the loading bag and lid assembly will have to be able to accommodate footing lateral movements and tilt. This will be a particular problem when the bed is of clay, as displacements are much larger than with sand.

A 25 mm thick plate is already provided as a false base plate inside each consolidation tank to support the bottom drainage filter. This plate is able to support the weight of the clay bed when lifted at four points around its circumference. This means that with the provision of a suitable lifting frame, the clay bed can be removed whole from the tank, making the bed freely accessible. If markers are placed inside the bed prior to consolidation these can then be carefully exposed after footing testing to provide information about internal deformations in the bed.

The present loading system is capable of providing regular cyclic loading of the footing. A control system for providing randomly varying dynamic loading is required to be added to the loading system for it to be able to reproduce more realistic loading conditions in the model. Under these conditions, particularly, it would be advantageous to have fully instrumented footings, so that stress distributions and their variation in response to loading could be recorded. Minor modification to the stub column to which the footings are attached would readily allow for footings of different proportions from those presently in use.

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NOTATION

B	Width of plane strain tank
D	Depth of plane strain tank
E	Modulus of elasticity
H	Height of a wall
K	Relative model footing plate stiffness
L	Length of a footing
P	Jack load
Q	Jack load
q	Footing capacity
R	Model footing plate radius
S	Jack load
t	Model footing plate thickness
W	Width of a wall or footing
δ	Deformation
ν	Poisson's ratio
σ	Stress

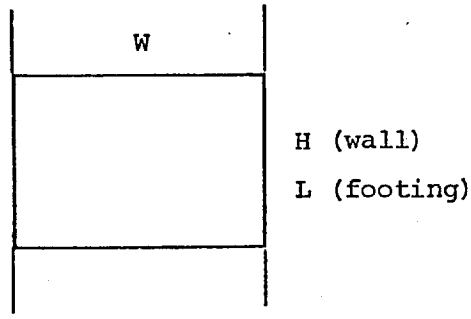


TABLE 1: PLANE STRAIN TEST TANK

SIDE WALL DESIGN PARAMETERS

Condition and Material (Loading: kPa)	Lateral pressure (approx. U.D.L.) (kPa)	Associated estimated lateral displacement (m)
2-phase consolidation to 350 kPa (vertical) (i) First load increment : 250 Instantaneous lateral: 250 Long-term lateral : 150 (ii) Second load increment: 100 Instantaneous lateral: 100 First increment long-term plus second increment instantaneous lateral pressure (150 + 100)	250*	1×10^{-3}
Footing test: sand (surcharge, 100; plate loading, 150)	50**	2×10^{-4}
Footing test: clay (no surcharge; plate loading, 250)	125**	4×10^{-4}

* : Based on soil mechanics theory (Hight, 1976)

** : Based on plane strain computer study results

TABLE 2: REVIEW OF LATERAL DISPLACEMENTS IN SOME PLANE STRAIN MODEL TESTS

Researcher/ Date	Material	Type	Max. lateral displacement (m)	Lateral strain (%)	Comments: tank width wall thickness
James (1965)	Sand	Wall	1.8×10^{-4}	0.19	190 mm width; 16 mm and 19 mm glass
Arthur and Roscoe (1965)	Sand	Wall	1.2×10^{-5}	0.017	150 mm width; 16 mm glass
Burland (1967)	Soft Clay	Footing	4×10^{-4}	0.57	140 mm width
Bransby (1968)	Sand	Wall	5×10^{-4}	0.53	190 mm width; 16 mm and 19 mm glass
Tennekoon (1970)	Sand	Footing	6.4×10^{-4}	0.67	190 mm width; 16 mm and 19 mm glass
Dick (1973)	Clay	Footing	4.3×10^{-4}	0.62	140 mm width; 38 mm armour plated glass
Ko and Davidson (1973)	Sand	Footing	5.0×10^{-5}	0.05	100 mm width; 12 mm glass
Milligan (1974)	Sand	Wall	9×10^{-5} (filling) 1.7×10^{-5} (failure)	0.09	200 mm width; 16 mm glass

TABLE 3: SOIL PROPERTIES FOR COMPUTER STUDIES (from Hight, 1976)

Study; Material	Properties
Plane strain; sand (non-linear)	Initial modulus: $E_i = 1500 \times 103.6 \times (\sigma_3/103.6)^{0.6}$ kPa Tangent modulus at any stress level: $E_t = E_i(1-0.28(\sigma_1-\sigma_3)/\sigma_3))^2$ kPa Poisson's ratio: 0.3 Angle of shearing resistance: $\phi' = 35^o$ Initial stresses: $\sigma'_H = 0.5\ \sigma'_V$
Three-dimensional; sand (linear)	Modulus: $E_i = 30000$ kPa Poisson's ratio: 0.3
Plane strain: clay (non-linear)	Initial modulus: $E_i = 5000$ kPa Tangent modulus at any stress level: $E_t = E_i(1-0.9153\ \sigma_r)/(1+2.591\ \sigma_r)$ kPa $\sigma_r = (\sigma_1-\sigma_3)/(\sigma_1-\sigma_3)$ at failure Poisson's ratio: 0.499 Initial stresses (assuming overconsolidated bed): $\sigma'_H = 2.0\ \sigma'_H$

TABLE 4: MEASURED VALUES OF

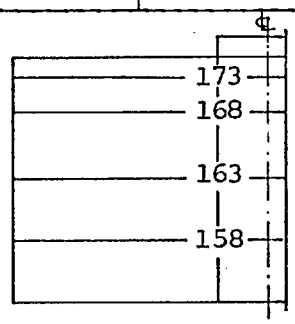
COEFFICIENT OF FRICTION

Researcher/ test	Materials	Method of Treatment	Coefficient of friction
Rowe and Barden (1964); triaxial test	sand	greased membrane	0.018
Arthur and Roscoe (1965); retaining wall, plane strain	Leighton Buzzard sand with (i) polished stainless steel (ii) polished duralumin (iii) glass	molybdenum disulphate paste cleaned with acetone	0.18 0.18 0.11
Green (1969); triaxial test	sand on rubber	greasing of rubber	0.01
Atkinson (1973); triaxial test	rubber membrane on polished brass or stainless steel	Releasil 7 grease, or molybdenum disulphate paste; or mixtures of these	0.045 or independent of vertical stress
Dick (1973); footing test, plane strain	clay on glass	grease/latex rubber membrane	0.05 increasing to 0.08 with time

TABLE 5: LATERAL DISPLACEMENT CASE -

SAND - 3D STUDY - TANK DEPTH 1.0 m

	Node	Three-dimensional study Tank width (m)				Plane strain study
		W = 0.5	W = 1.0	W = 1.5	W = 2.0	
σ_{xx}	173	1.65	0.73	0.40	-0.26	- 76
	168	2.4	1.1	0.67	0.49	- 69
	163	3.5	1.4	0.83	0.57	- 67
	158	2.6	1.3	0.78	0.55	- 66
σ_{yy}	173	4.4	2.1	1.2	0.7	- 88
	168	8.3	4.0	2.3	1.4	- 78
	163	13.0	5.5	2.9	1.6	- 73
	158	9.1	4.4	2.7	1.9	- 72
σ_{zz}	173	1.1	0.5	-0.75	-1.05	-219
	168	-0.4	-0.53	-0.60	-0.57	-190
	163	-1.30	-0.72	-0.34	-0.14	-175
	158	-0.32	-0.18	-0.19	-0.24	-172
δ_x	173	-0.04	-0.1	-0.25	-0.33	- 7
	168	-0.01	-0.05	-0.21	-0.33	- 15
	163			-0.11	-0.23	- 22
	158			-0.04	-0.12	- 19
δ_z	173	-9.5	-4.9	-3.6	-3.0	-438
	168	-8.7	-4.5	-3.1	-2.6	-363
	163	-5.3	-2.7	-2.1	-2.0	-228
	158	-1.7	-1.1	-1.3	-1.5	-154



Node Key

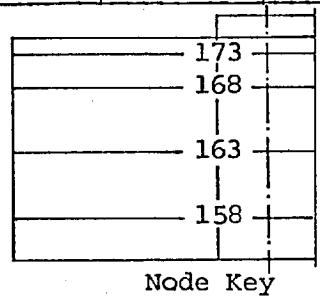
Units: Stress: kPa

Deformations: m x 10^{-5}

TABLE 6: SIDE FRICTION CASE - SAND -

3D STUDY - TANK DEPTH 1.0 m

	Node	Three-dimensional study Tank width (m)					Plane strain study
		Limiting friction cases				zero friction	
		W = 0.5	W = 1.0	W = 1.5	W = 2.0		
σ_{xx}	173	-162	-94.6	-79.6	-75.4	-71.2	- 76
	168	-108	-77.9	-69.2	-67.1	-61.0	- 69
	163	-64.0	-62.3	-60.1	-60.0	-51.8	- 67
	158	-30.0	-47.2	-52.1	-54.9	-45.0	- 66
σ_{yy}	173	-154	-106	-98.0	-95.9	-101	- 88
	168	-79.7	-70.7	-69.1	-69.7	-80.3	- 78
	163	-42.9	-46.7	-47.7	-49.5	-69.0	- 73
	158	-19.0	-28.1	-33.9	-38.9	-64.4	- 72
σ_{zz}	173	-475	-299	-259	-250	-268	-219
	168	-349	-257	-227	-218	-221	-190
	163	-199	-207	-201	-198	-190	-175
	158	-890	-157	-175	-181	-175	-172
δ_x	173	-11.7	-7.5	-6.5	-6.8	- 18	- 7
	168	-19.1	-19.0	-17	-16.9	- 29	- 15
	163	-12.9	-22.1	-23	-23.3	- 39	- 22
	158	-5.3	-16.0	-20	-20.6	- 42	- 19
δ_z	173	-398	-452	-463	-463	-456	-438
	168	-280	-376	-398	-404	-391	-363
	163	-128	-247	-281	-291	-283	-228
	158	- 56	-144	-174	-183	-183	-154



Node Key

Units: Stress: kPa

Deformations: m x 10^{-5}

TABLE 7: 229 mm OEDOMETER AND ASSOCIATED FOOTING TEST DETAILS AND RESULTS

I	II	IIIA	III	IIIB	IVA	IV	IVB	V	VI
1	1	-0.8	34.6	+0.2	16.2	16.5	17.0	455	420
2	1	-0.7	35.0	+0.2	15.5	16.3	17.0	261	450
3	2	-0.5	33.7	+0.2	15.7	15.6	15.6	410	420
4	2	-0.2	38.0	+0.2	14.7	15.0	15.3	307	390
5	3	-0.1	40.1	+0.3	15.8	15.9	15.8	237	360
6	3	-0.3	39.1	+0.2	15.7	15.7	15.5	261	430
7	3	-0.3	40.4	+0.3	15.7	15.6	15.4	215	435
8	3	-0.4	37.1	+0.4	15.6	15.2	14.8	216	415
9	3	-0.8	31.1	+0.3	15.9	15.3	14.9	216	450
10	3	-0.1	30.2	+0.3	15.5	14.3	13.5	212	470
11	3	-0.5	30.2	+0.5	15.5	14.3	13.3	214	485
12	2	-0.4	31.7	+0.1	15.9	16.0	16.3	357	375
13	4	-0.3	30.4	+0.1	15.7	16.1	16.5	218	300
14	4	-0.1	29.6	+0.1	16.0	15.2	14.8	288	466

KEY TO COLUMNS:

I : Test Number

II : Consolidation Load Sequence Type (see 4.4.2 to 4.4.5)

III : Average Initial Moisture Content (%)

IIIA : Variation to minimum moisture content (%)

IIIB : Variation to maximum moisture content (%)

IV : Average Final Moisture Content (%)

IVA : Mean top surface moisture content (%)

IVB : Mean underside of sample moisture content (%)

V : Total time for consolidation (hours)

VI : Footing Ultimate Bearing Capacity (kPa)

TABLE 8: 229 mm OEDOMETER TESTS:

CONSOLIDATING PRESSURES

Test Numbers	Load Increment 1	Load Increment 2	Load Increment 3	Load Increment 4
1 and 2	-33	80	160	320
3,4 and 12	13	36	120	325
5 and 6	13	36	120	325
	-35	-35	-35	-35
7 and 8	13	36	120	325
	-70	-70	-70	-70
9 to 11	13	36	120	325
	-70	-70	-70	-70
13 and 14	18	35	120	325
	-70	-70	-70	-70

NOTES: (i) Top surface pressures are shown positive. Levels of partial vacuum applied to the underside of the sample are shown negative.

(ii) Values for top surface pressures for Tests 1 to 8 are average values (see 4.4.1)

(iii) Units: kPa

TABLE 9: NORFOLK CLAY:LIQUID LIMITS AND PLASTIC LIMITS

(from Hight, 1977a)

Sample	LL (%)	PL (%)	Comments
A1	25.0	15.0	Preliminary samples taken from clay seam in area selected for bulk sampling
A2	23.6	12.8	
A3	24.9	13.6	
B1	25.4	12.4	Top layer of bulk sample
	25.3	13.2	
B2	25.1	12.9	Middle layer of bulk sample
	25.5	12.2	
B3	24.6	12.7	Bottom layer of bulk sample
	25.5	11.4	
B4	25.5	12.5	Mixed B1 to B3 types

TABLE 10: LARGE SCALE TESTIN G FACILITY : MAIN EQUIPMENT AND COMPONENT DETAILS

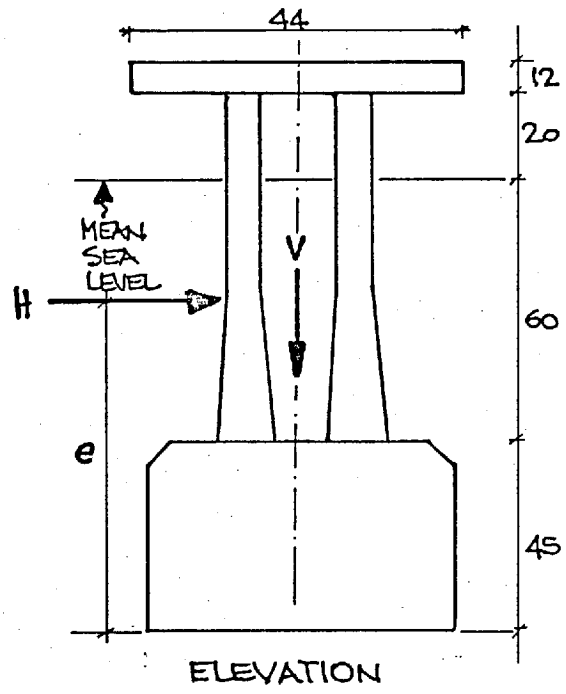
ITEM	DETAILS
Data logging	Solartron A210/214 Digital Voltmeter 3240A Data Transfer Unit 3325 Scanner Basic Assembly 3N° 10-Channel Modules Clary 3227A Printer Facit 4070 Punch
Load Cells	Sangamo Weston Controls D91/10000 5000 Kg D91/ 5000 2500 Kg
Linear Displacement Transducers	Sangamo Weston Controls NDLR/25 ± 25 mm travel NDLR/50 ± 50 mm travel NDLR/150 ±150 mm travel
Pressure/Vacuum transducers	Shape Instruments H215-5 J.P.B.
Load Bag: Neoprene Latex	Medcraft Bros. : 0.8 mm thick Elongation at break 880% Tensile strength 215 Kg/cm^2 Shore hardness 32/42
Adhesive	Dunlop Semtex : Two part cement S.2000
Filter: Main layer	Porvair Ltd.:Vyon filter: 0.75 mm thick 3.2 mm thick
Back-up layer	Strand glass:Woven roving: 830 gm/sq.m. (fibre glass)
Bearings:	SKF: maintenance-free spherical plain bearings, steel/PTFE fabric : thrust ball bearings : deep groove ball bearings
Vacuum system	Edwards: Vacuum pump : 1SC 450B Isolation valve : SV25 Oil mist filter : OMF 500 Vacuum switches : VSK 1B Speedivalves : SC (1½)
Pressure system	Monostat: Control values : 20-415 kPa G.X. Engineers: Reducing valve : B11-M3 35-1050 kPa

TABLE 11: LARGE SCALE TESTING FACILITY:

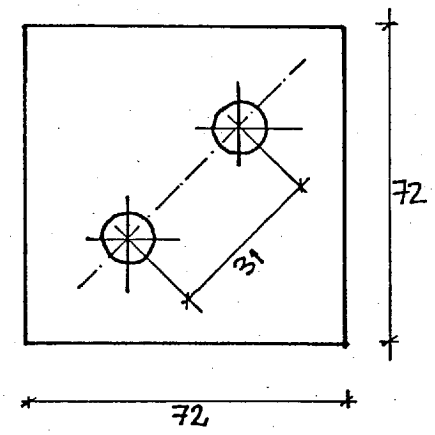
CAPACITIES OF LOADING SYSTEM COMPONENTS

Component	Maximum applied force (kN)	Component Working stress Capacity (kN)
Vertical load system:		
Links	50	125
Bolts	50	120
Bearings (GE25C)	50	129
Horizontal load system:		
Links	12.5	74
Bolts	12.5	72
Bearings (GE17C)	12.5	58

Note: Load frame and load transfer beams are designed on the basis of strict deflection criteria and have adequate strength.

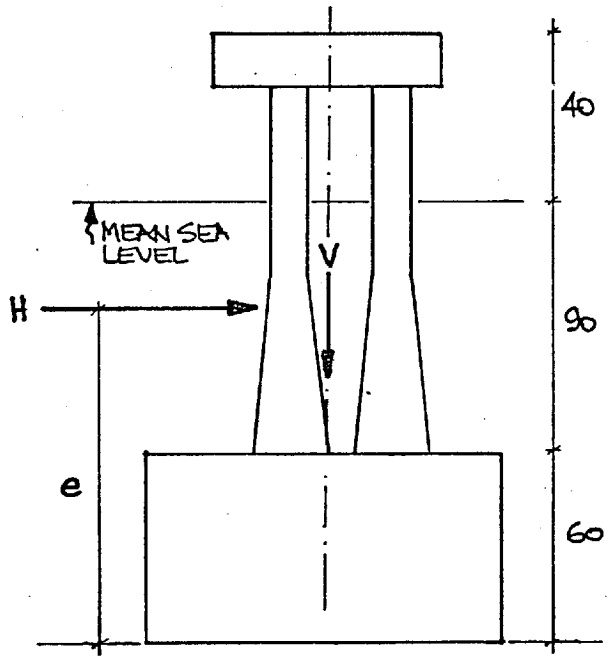


ELEVATION

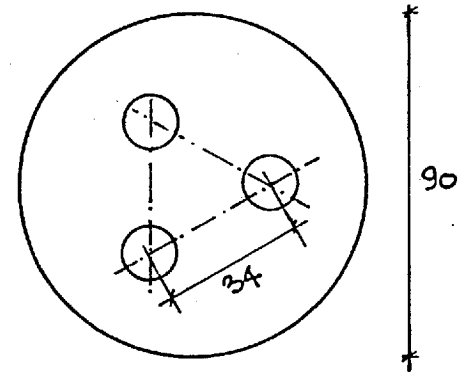


PLAN

TP 1 PLATFORM (FROM DERRINGTON, 1977)



ELEVATION



PLAN

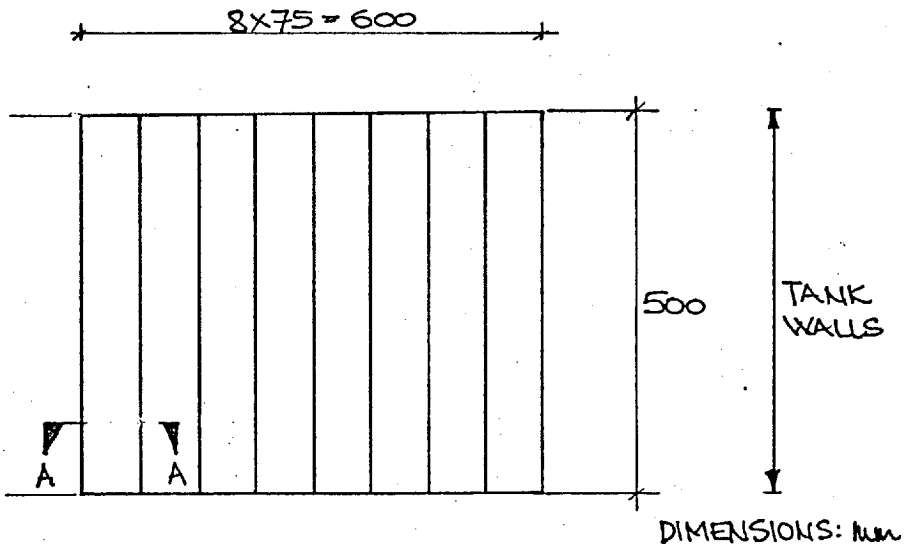
CONDEEP (FROM KIRK & NATARAJI, 1976)

TYPICAL VALUES: V_{max} = 200 000 t ± 10 000 t
V_{min} = 150 000 t
H = ± 75000 t
He = ± 1 000 000 tm
(FROM CLAUSEN, 1973; EIDE, 1974)

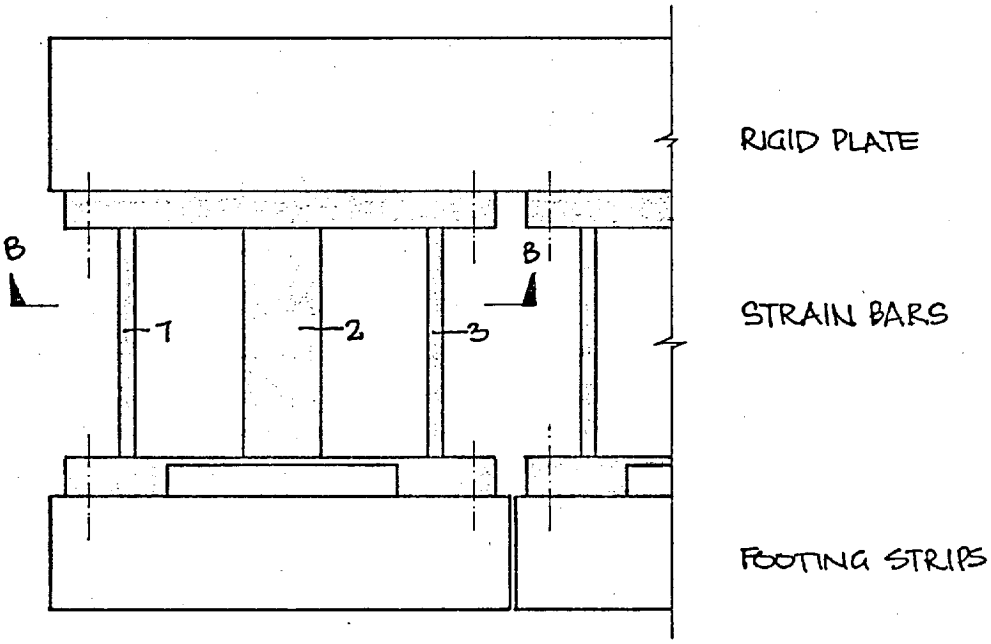
DIMENSIONS: METRES

TYPICAL GRAVITY PLATFORMS

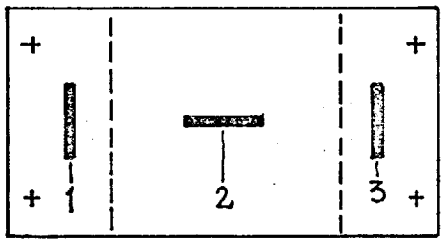
FIGURE 1



VIEW OF UNDERSIDE OF FOOTING



SECTION A-A – STRAIN BAR ARRANGEMENT

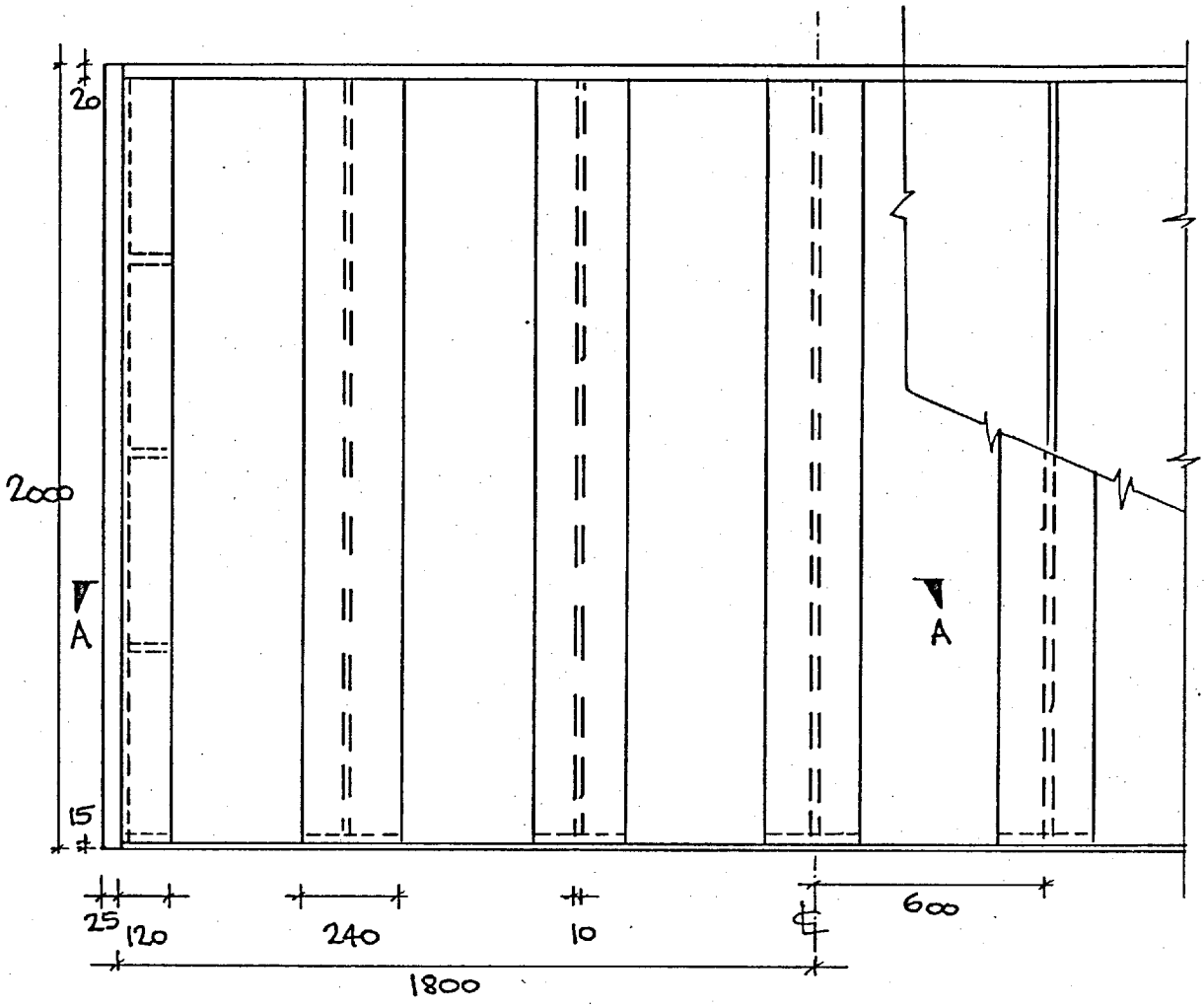


BARS: 1 & 3: VERTICAL LOAD & ECCENTRICITY
2: SHEAR FORCE
TWO SETS PER FOOTING STRIP

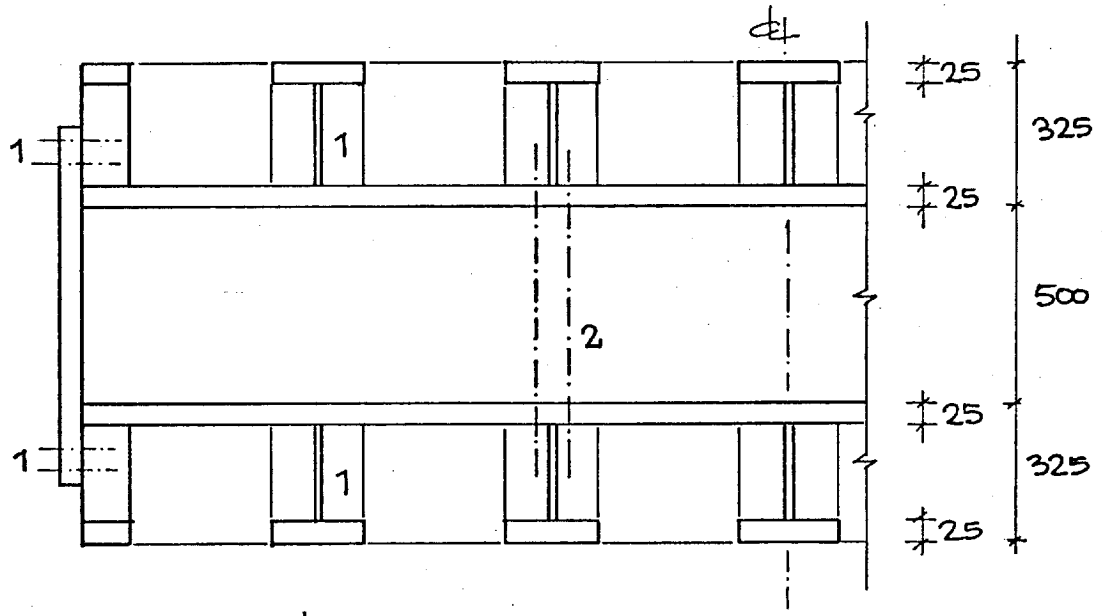
SECTION B-B PLAN OF STRAIN BARS

PRELIMINARY PLANE STRAIN FOOTING TEST DETAILS

FIGURE 2



ELEVATION



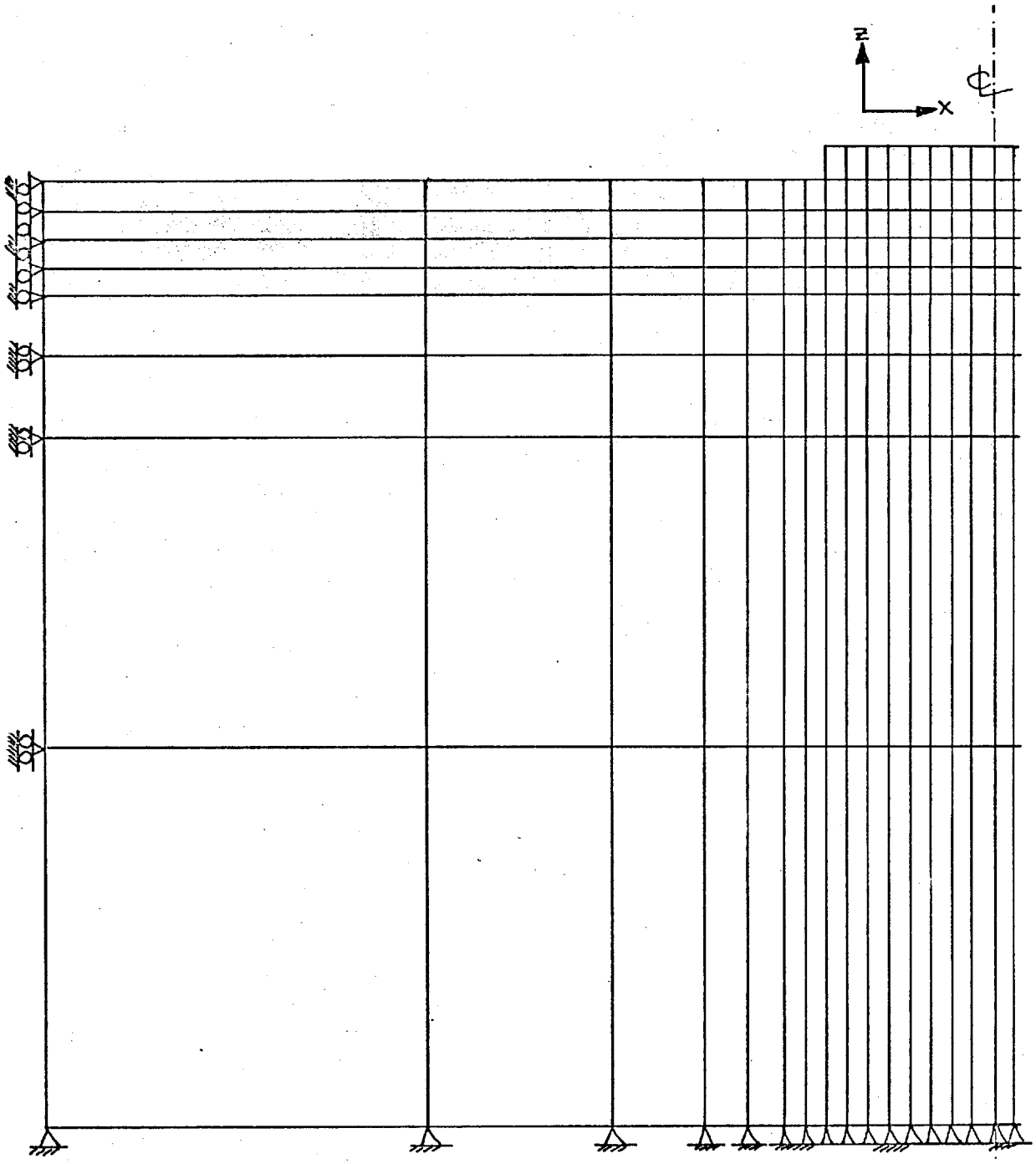
1. END PLATES & BASE OF STIFFENERS BOLTED TO FACILITATE SOIL SAMPLE REMOVAL
2. AT ℄ L 600 TANK BOLTED ACROSS TOP FOR RIGIDITY

SECTION A-A

DIMENSIONS : mm

PRELIMINARY PLANE STRAIN TEST TANK DETAILS

FIGURE 3



HALF FINITE ELEMENT MESH ONLY SHOWN -

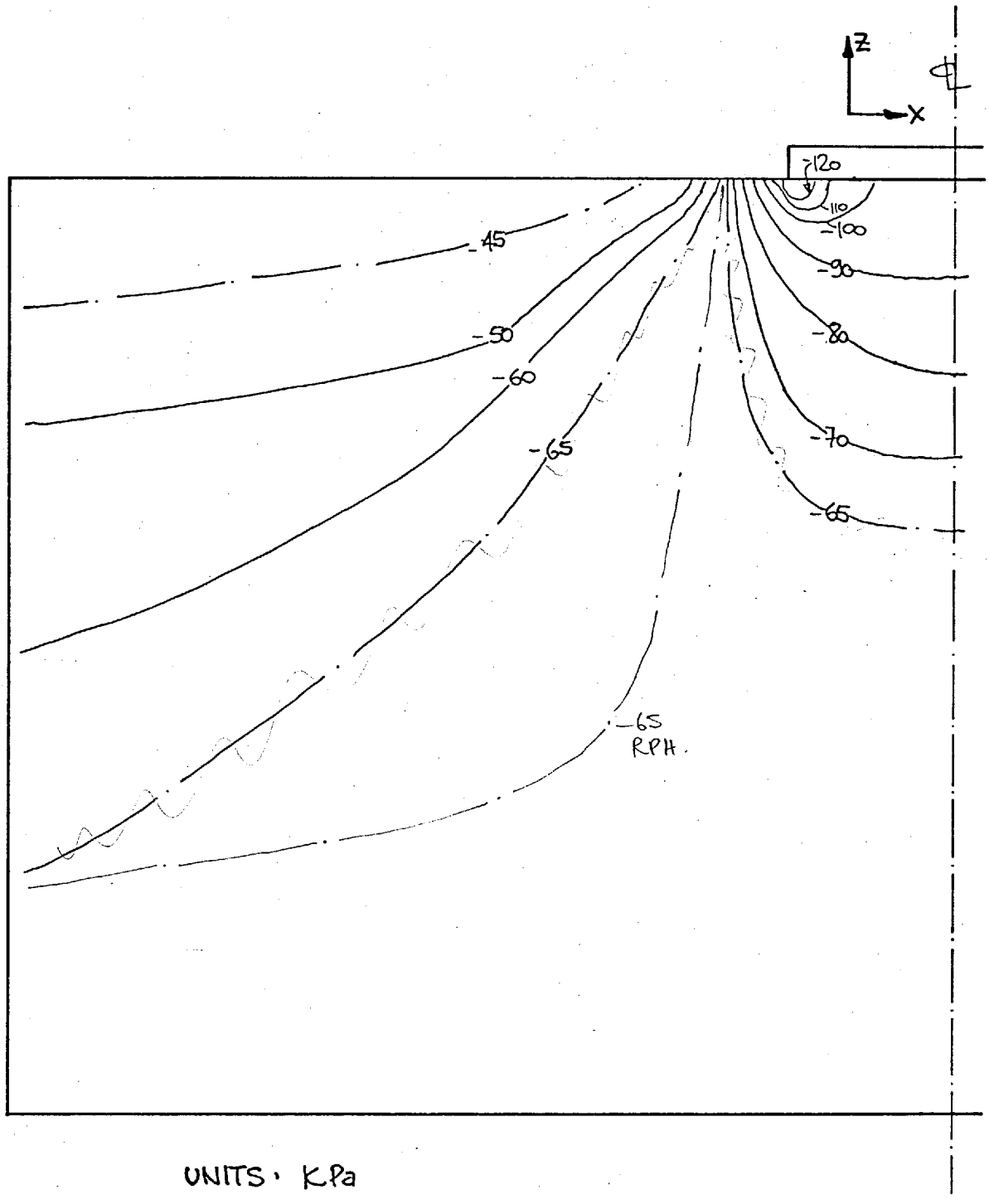
BOUNDARY CONDITIONS : FULLY RESTRAINED (X,Z)

RESTRAINED (X)
FREE (Z)

247 ELEMENTS (8 NODE); 818 NODES

PLANE STRAIN FINITE ELEMENT ANALYSIS MESH DETAILS

FIGURE 4

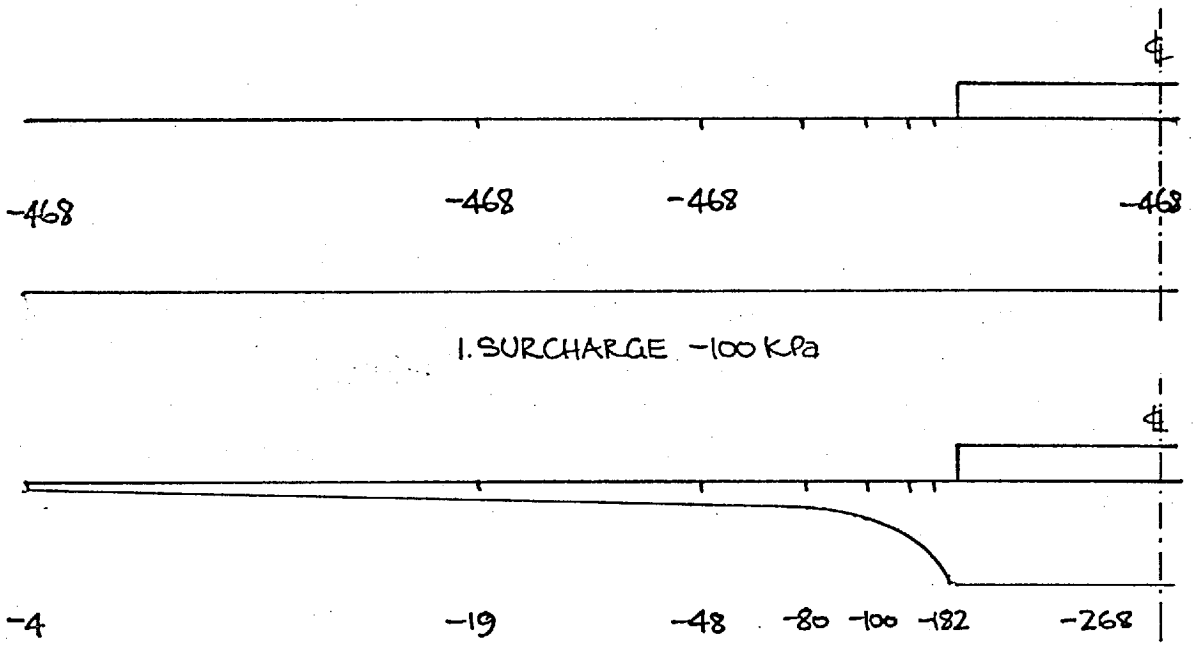


UNITS : K Pa

SAND UNDER SURCHARGE −100 K Pa
PLATE LOADING −250 K Pa

PLANE STRAIN STUDY LATERAL STRESS DISTRIBUTION

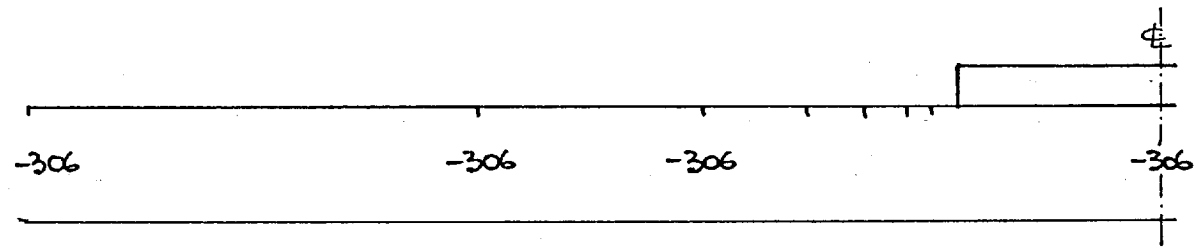
FIGURE 5



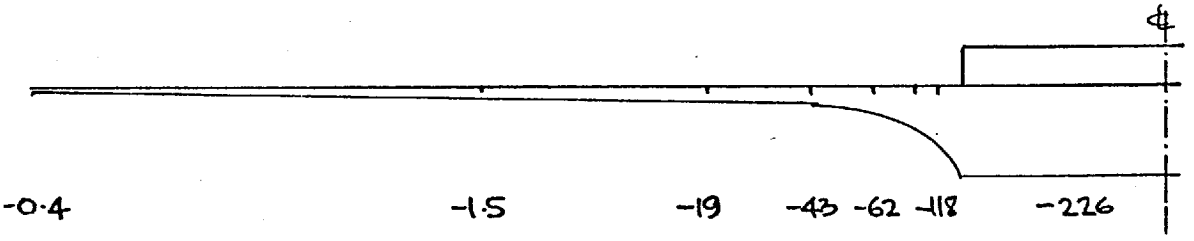
1. SURCHARGE -100 KPa

2. ADDITIONAL PLATE LOAD -150 KPa

RESULTS: TANK DEPTH 1.75m



1. SURCHARGE -100 KPa



2. ADDITIONAL PLATE LOAD -150 KPa

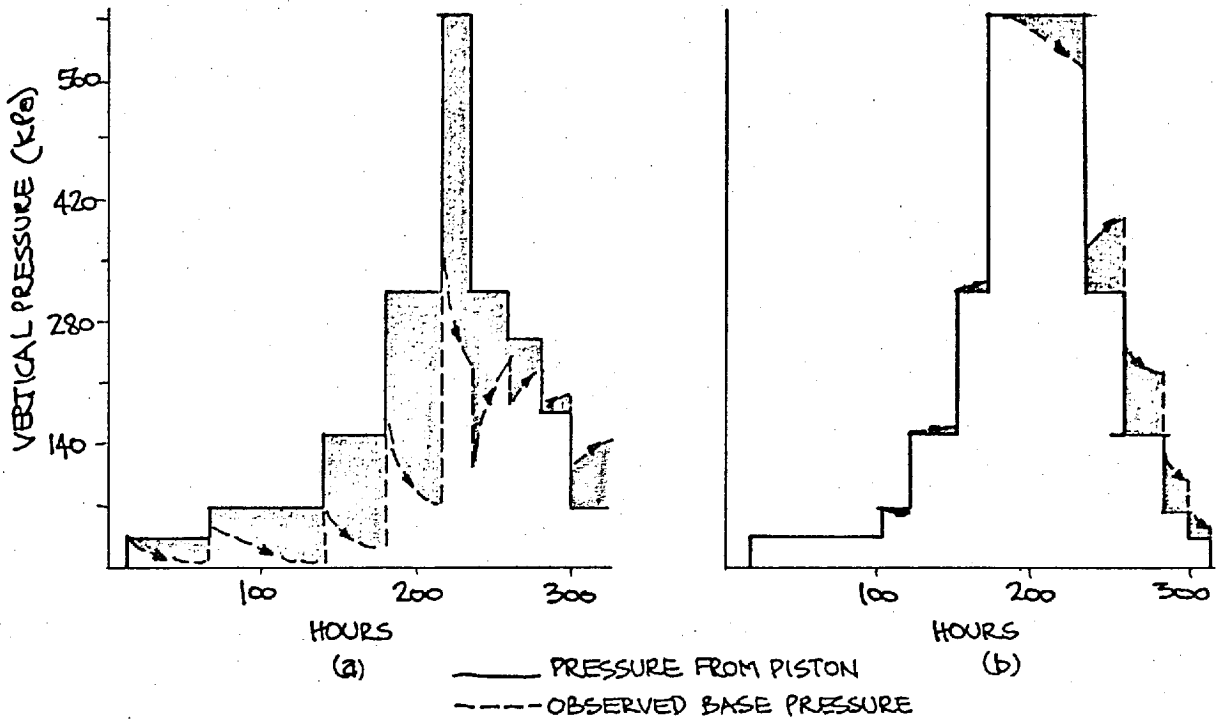
RESULTS: TANK DEPTH 1.0m

UNITS: $m \times 10^{-5}$

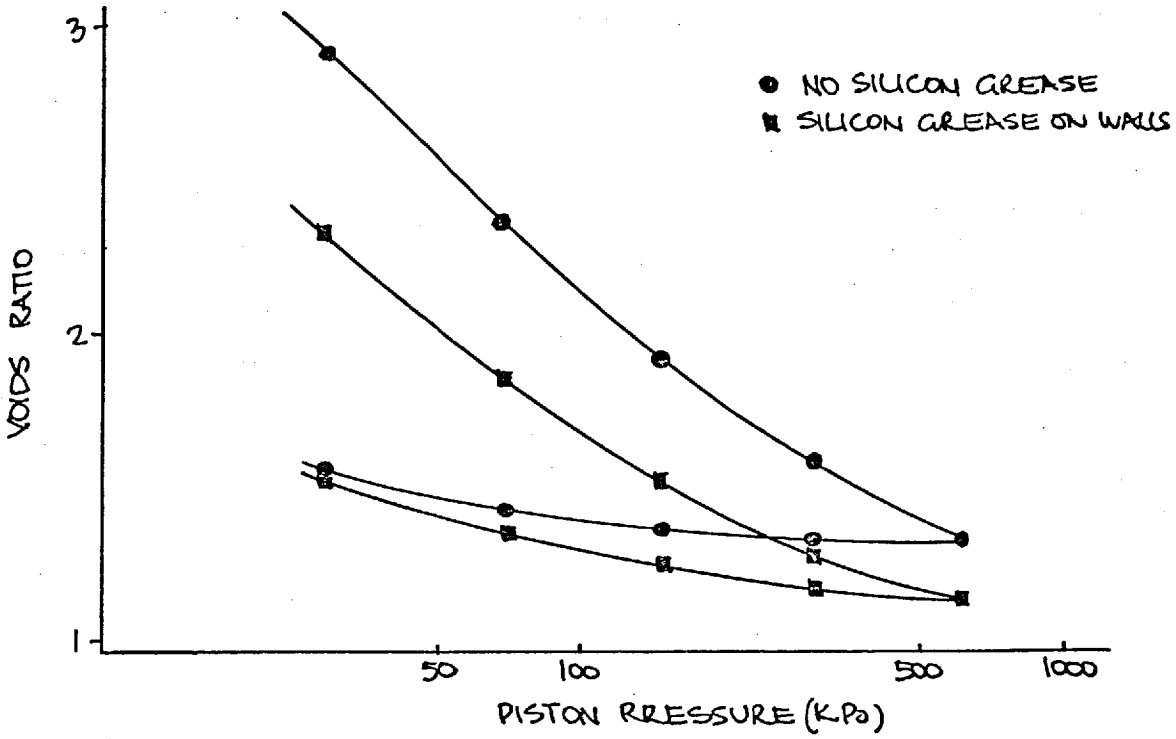
NOTE: PLATE LOAD VALUES TO BE SUPERIMPOSED ON SURCHARGE

PLANE STRAIN STUDY SURFACE DEFORMATIONS: SAND

FIGURE 6



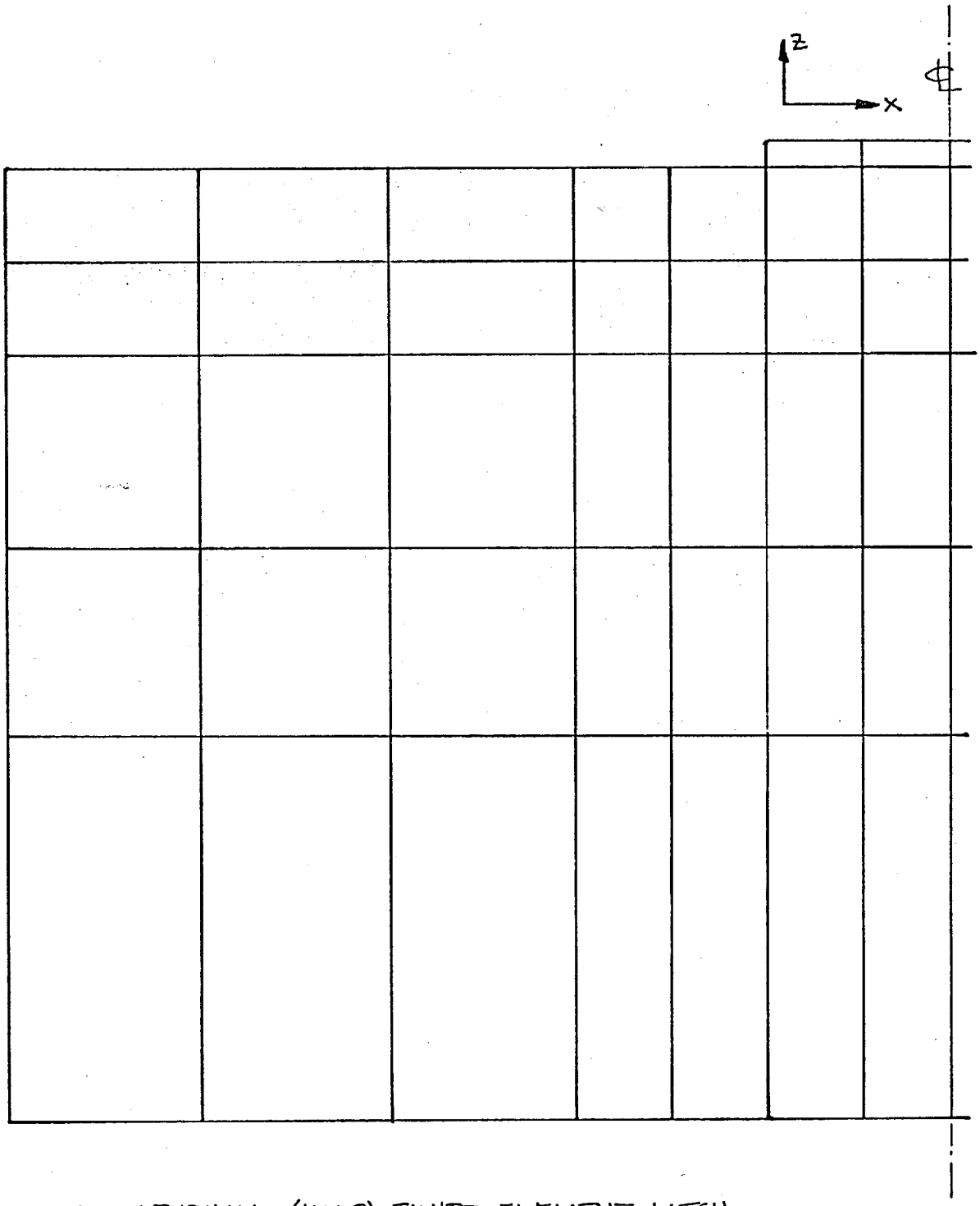
MAGNITUDE OF WALL FRICTION (a) WITHOUT AND (b) WITH SILICONE GREASE ON WALLS DURING CONSOLIDATION OF KAOLIN IN A TALL OEDOMETER OF HEIGHT/DIAMETER 7·2·5



INFLUENCE OF SILICON GREASE ON WALLS OF A TALL OEDOMETER ON SATURATED KAOLIN REMOULDED AT TWICE THE LIQUID LIMIT

WALL FRICTION TEST IN TALL OEDOMETER — (FROM BURLAND & ROSCOE 1969)

FIGURE 7

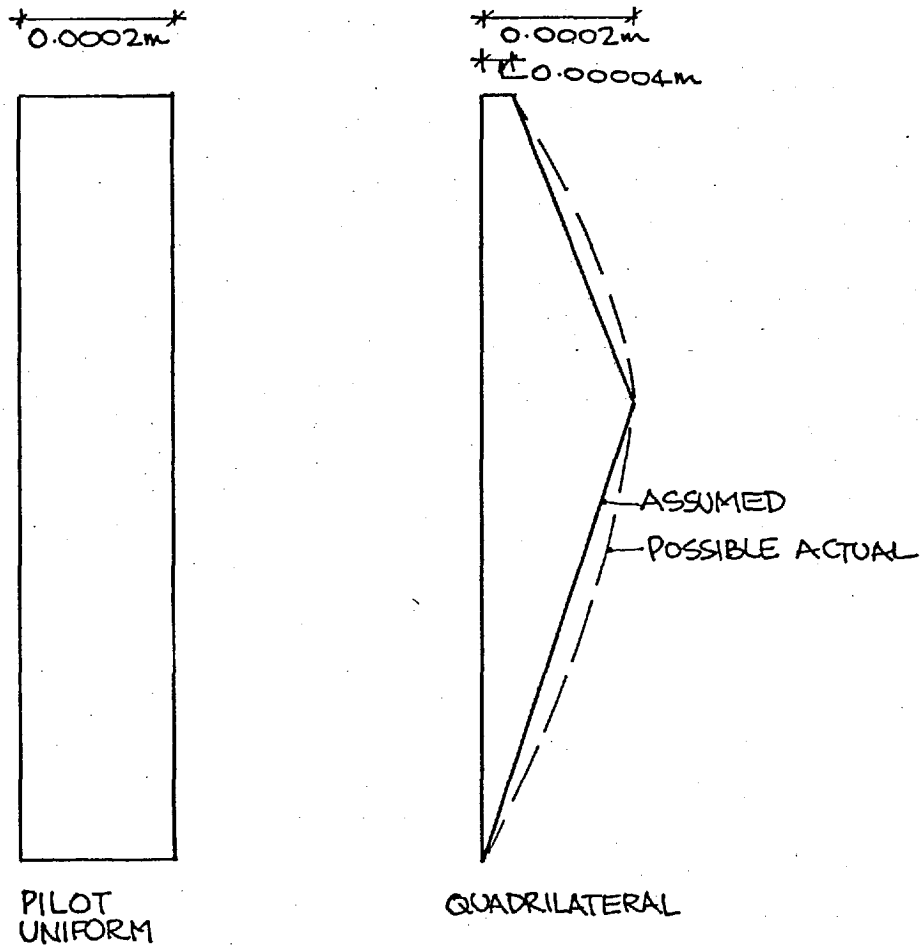


LONGITUDINAL (HALF) FINITE ELEMENT MESH —
FIVE PARALLEL PLANES MAKING MODEL FOUR 3D
ELEMENT BLOCKS DEEP — 296 ELEMENTS; 475 NODES

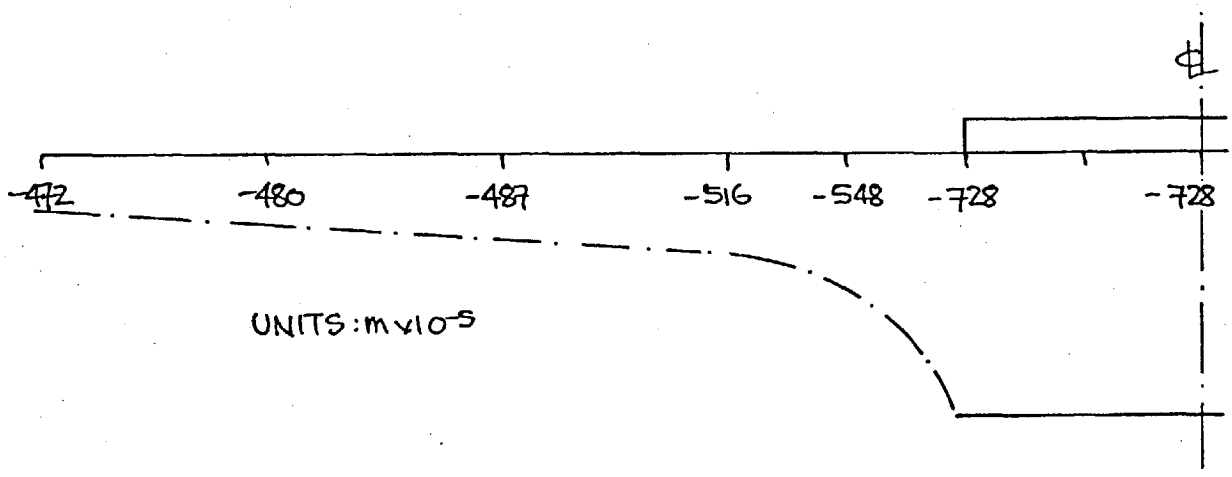
BOUNDARY CONDITIONS VARIED TO SUIT DIFFERENT STUDIES.

3D STUDY FINITE ELEMENT ANALYSIS MESH DETAILS

FIGURE 8



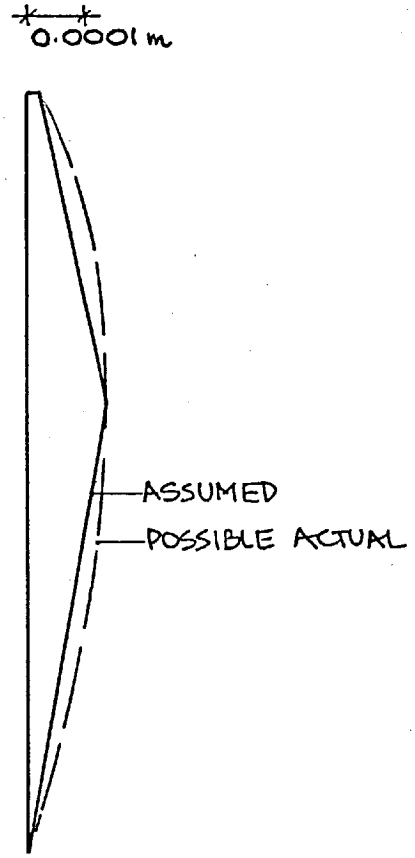
TANK WALL LATERAL DISPLACEMENT CASE



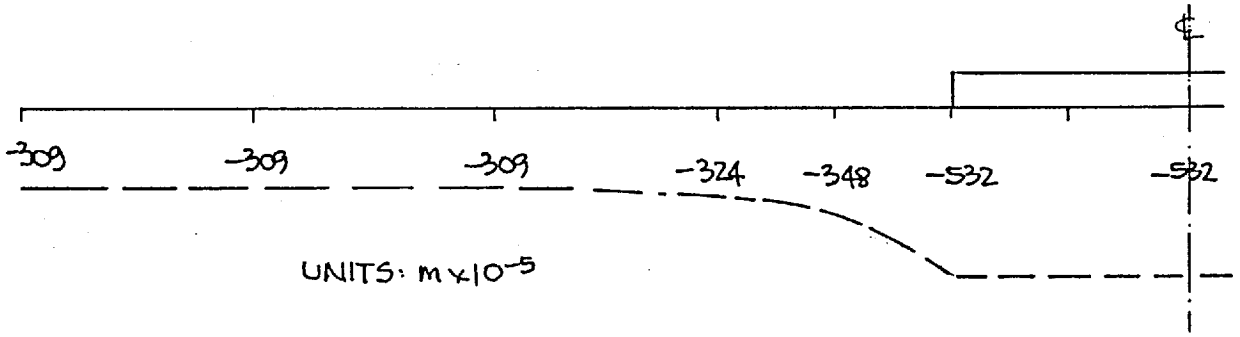
TANK WALL SIDE FRICTION CASE: SURFACE DEFORMATION

IMPOSED CONSTRAINTS FOR 3D STUDY, TANK DEPTH 1.75m

FIGURE 9



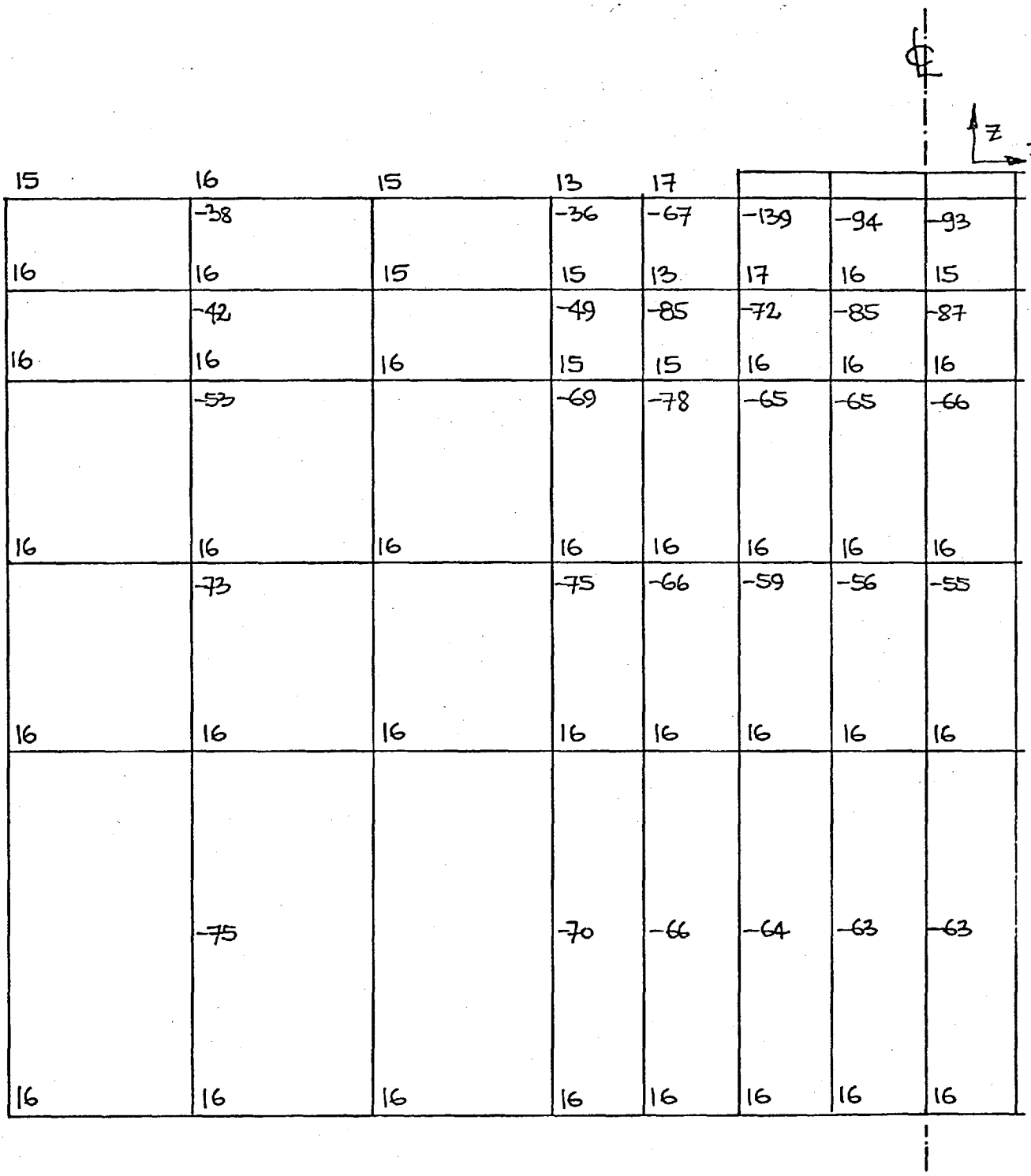
TANK WALL LATERAL DISPLACEMENT CASE



TANK WALL SIDE FRICTION CASE: SURFACE DEFORMATION

IMPOSED CONSTRAINTS FOR 3D STUDY: TANK DEPTH = 1.0m

FIGURE 10



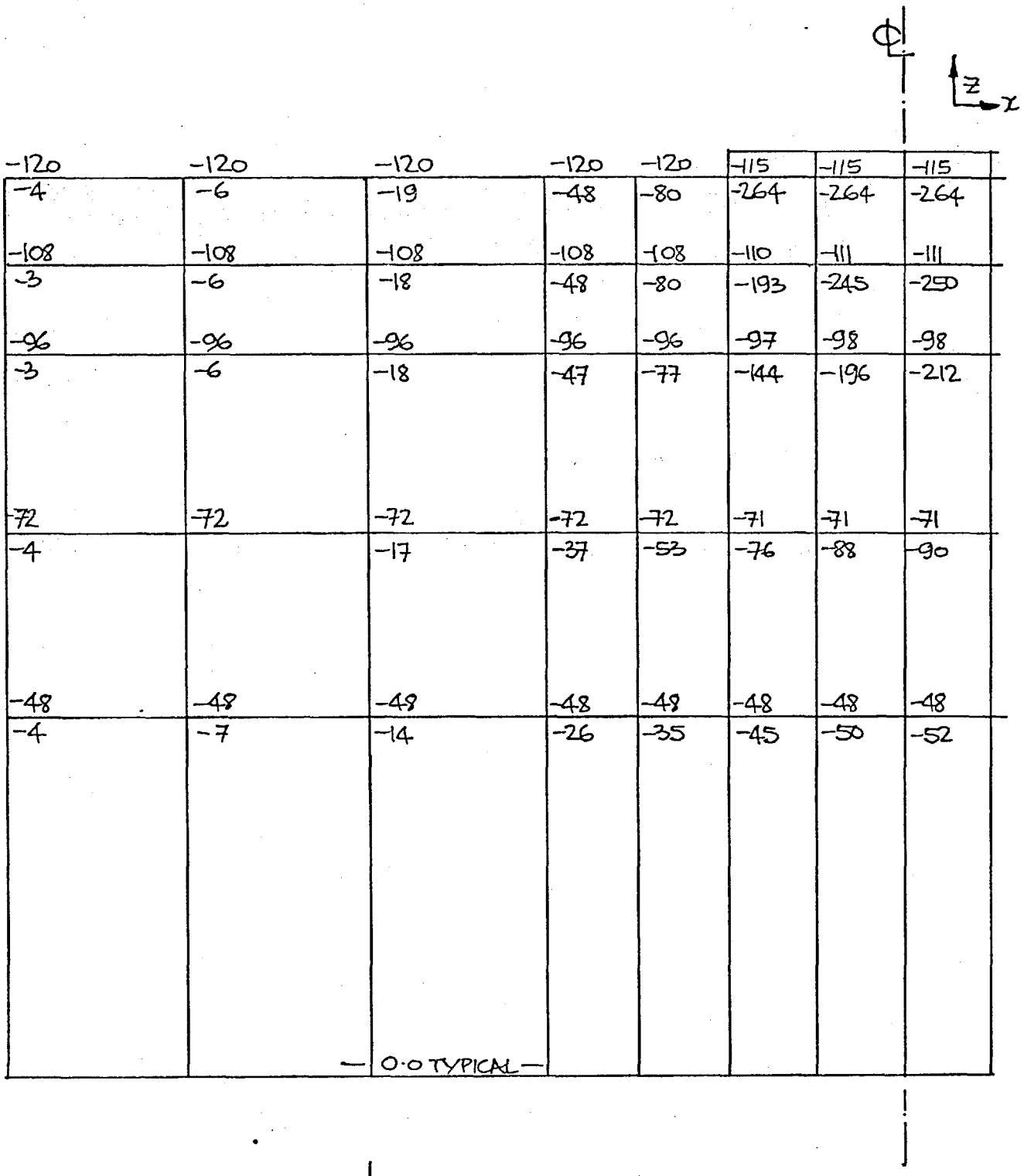
KEY : 3D STUDY VALUE / PLANE STRAIN STUDY VALUE

UNITS: KPa TANK WIDTH 0.5 m

LATERAL DISPLACEMENT CASE (UNIFORM) - SAND

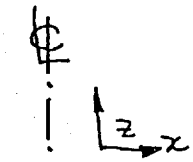
σ_{xx} COMPARISON

FIGURE 11



LATERAL DISPLACEMENT CASE (UNIFORM) - SAND
δ_z COMPARISON

FIGURE 12



0.8	0.7	0.6	0.5	1.2			
	-38		-36	-67	-139	-74	-93
1.5	1.5	1.5	1.5	1.4	1.6	1.6	1.6
	-42		-49	-85	-72	-85	-87
2.3	2.3	2.3	2.2	2.2	2.3	2.4	2.4
	-53		-69	-78	-65	-65	-66
3.4	3.4	3.4	3.4	3.4	3.3	3.3	3.3
	-73		-75	-66	-59	-56	-55
2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	-75		-70	-66	-64	-63	-63

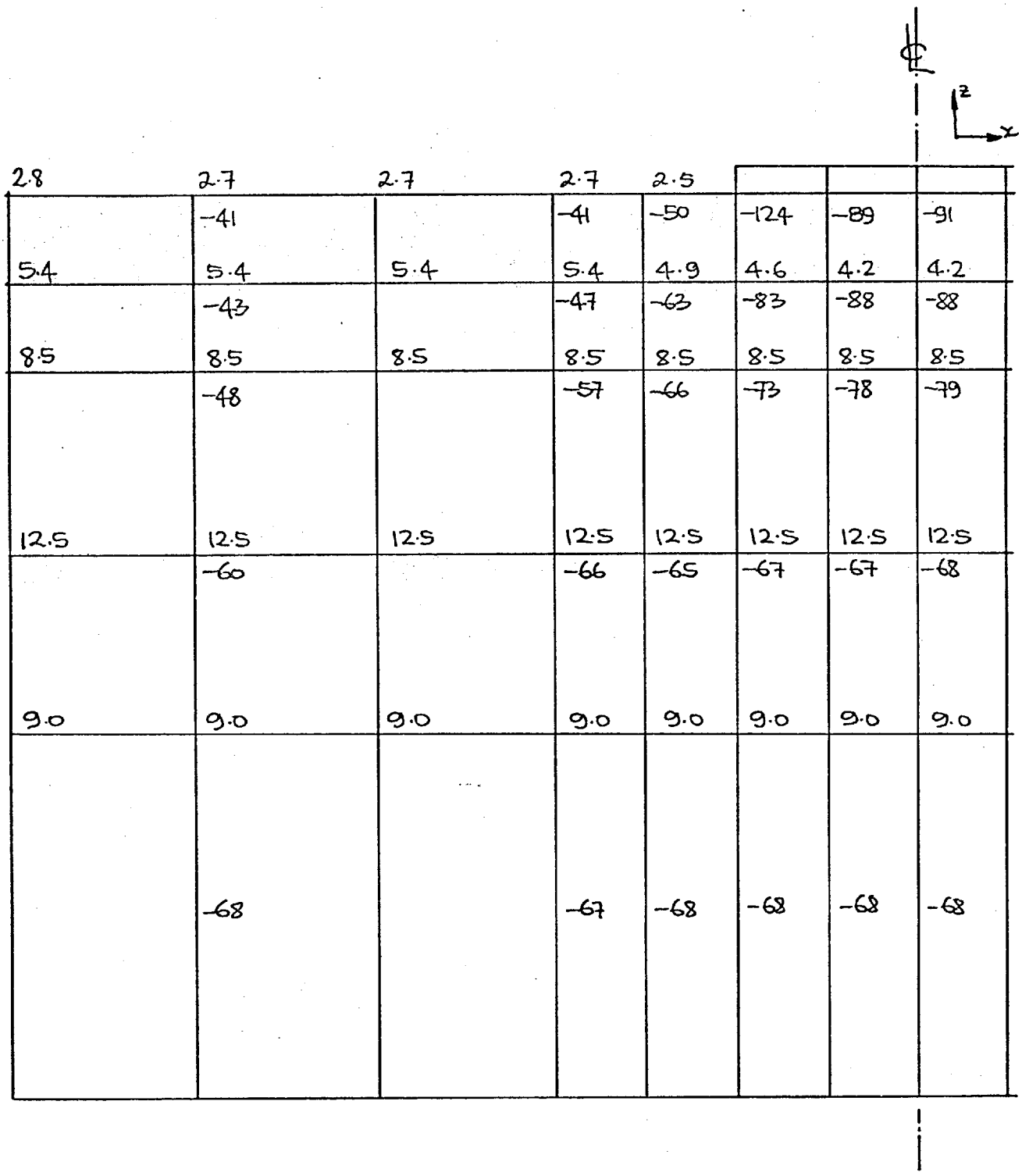
KEY : 3D STUDY VALUE / PLANE STRAIN STUDY VALUE

UNITS: kPa TANK WIDTH 1.0m

LATERAL DISPLACEMENT CASE – SAND

∇_{xx} COMPARISON

FIGURE 13



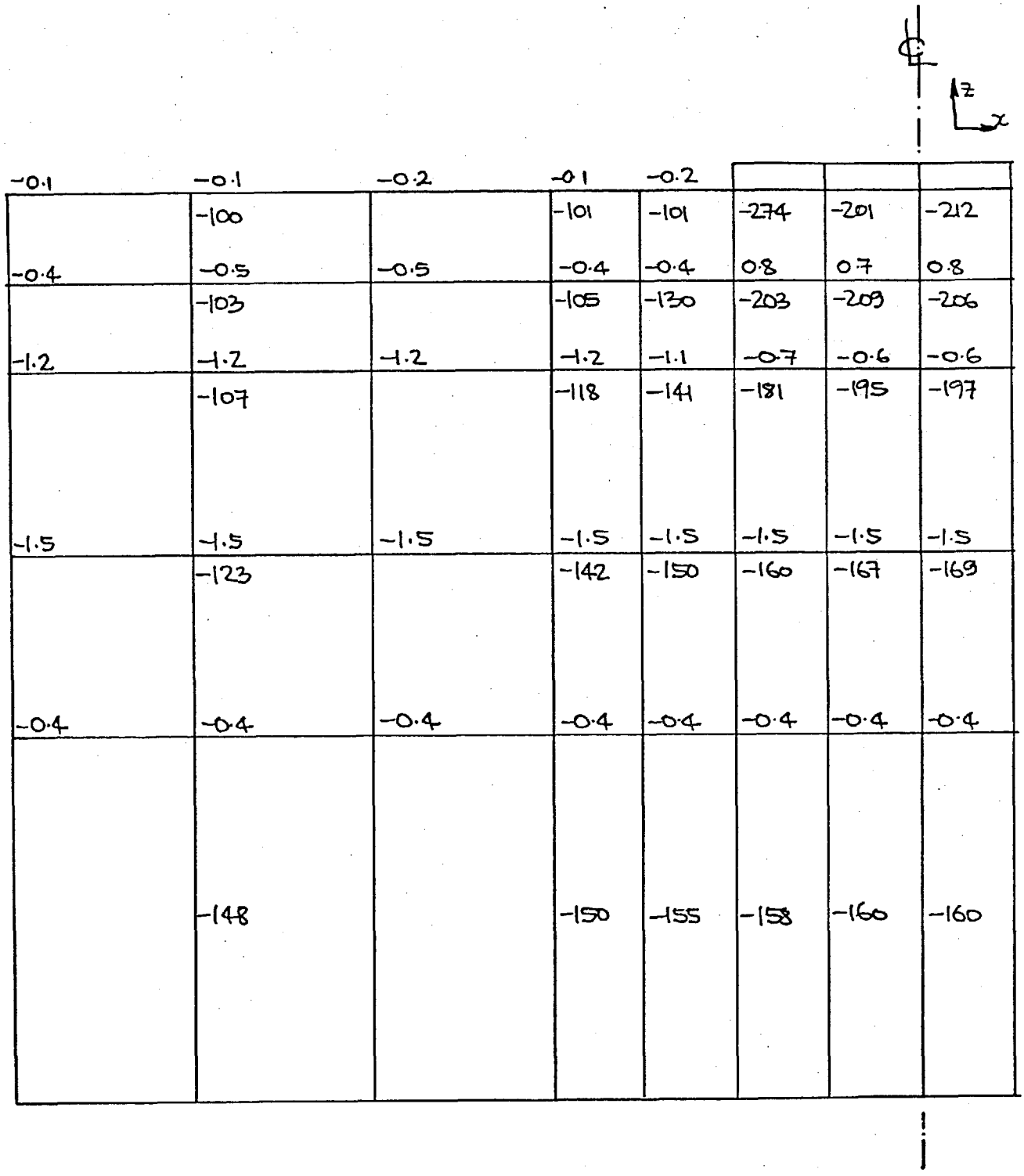
KEY : 3D STUDY VALUE / PLANE STRAIN STUDY VALUE

UNITS: kPa TANK WIDTH 1.0m

LATERAL DISPLACEMENT CASE – SAND

σ_{yy} COMPARISON

FIGURE 14



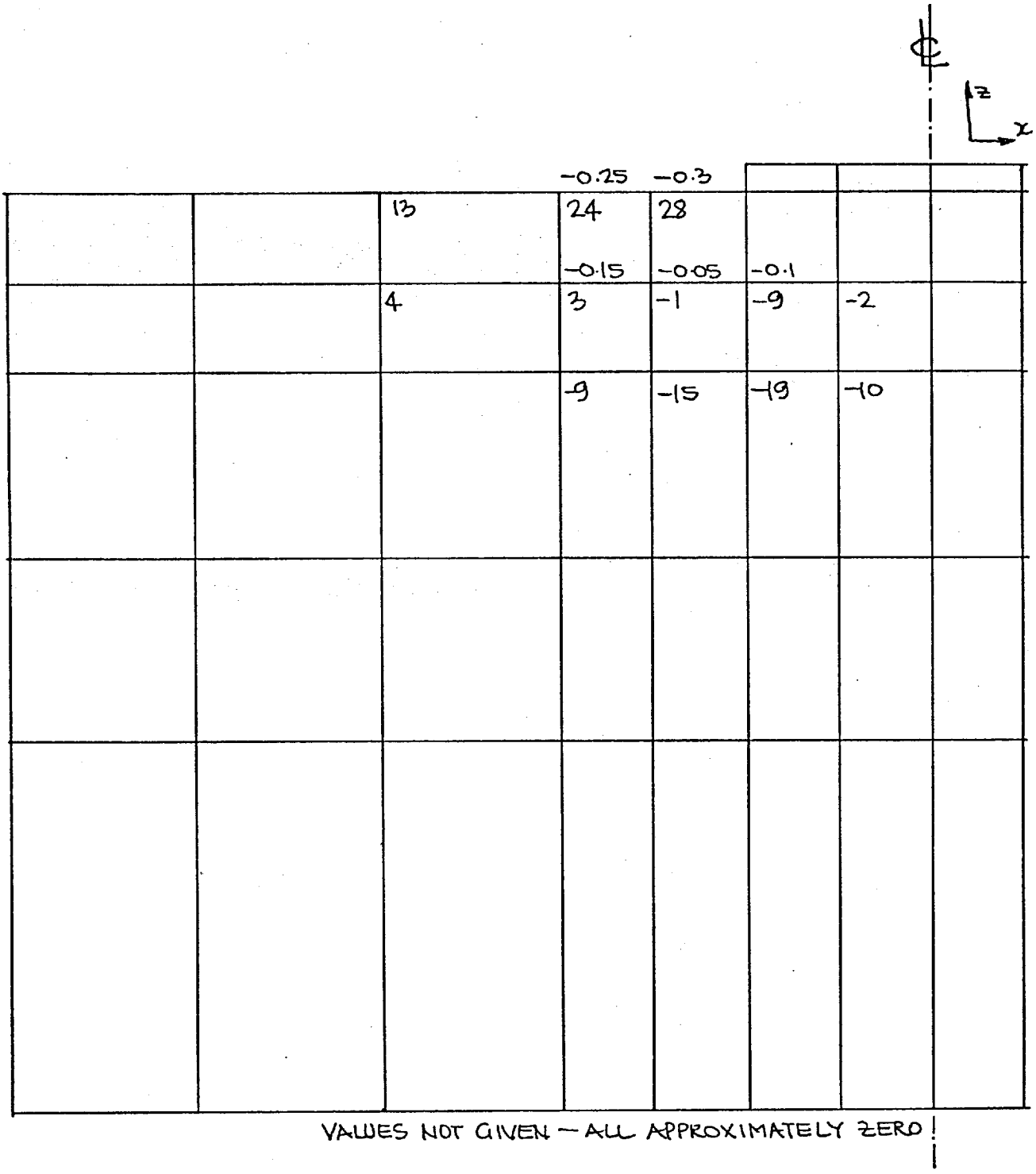
KEY : 3D STUDY VALUE / PLANE STRAIN STUDY VALUE

UNITS: kPa TANK WIDTH 1.0m

LATERAL DISPLACEMENT CASE – SAND

σ_{zz} COMPARISON

FIGURE 15



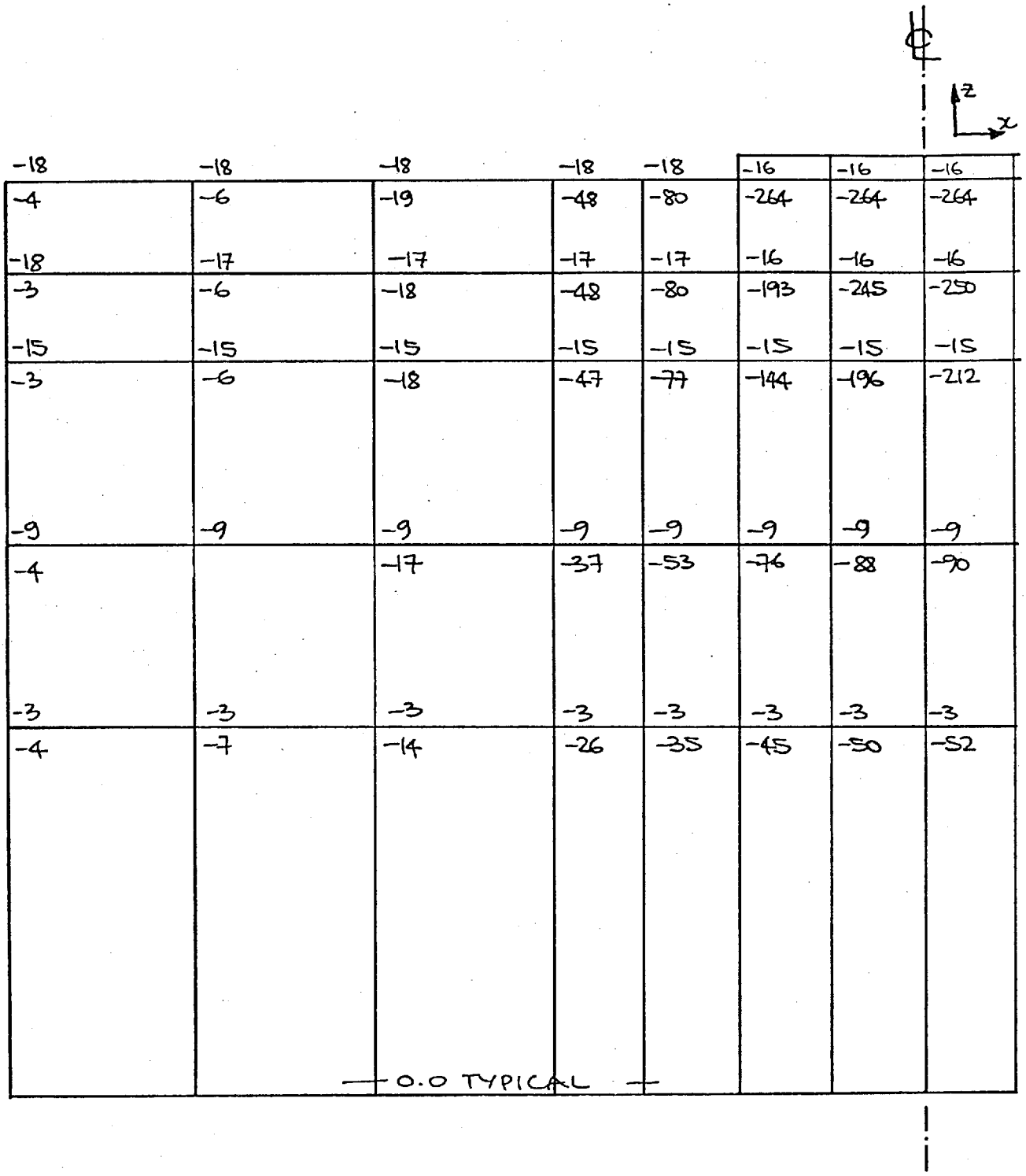
KEY: 3D STUDY VALUE / PLANE STRAIN STUDY VALUE

UNITS: $m \times 10^{-5}$ TANK WIDTH 1.0 m

LATERAL DISPLACEMENT CASE — SAND

δ_x COMPARISON

FIGURE 16



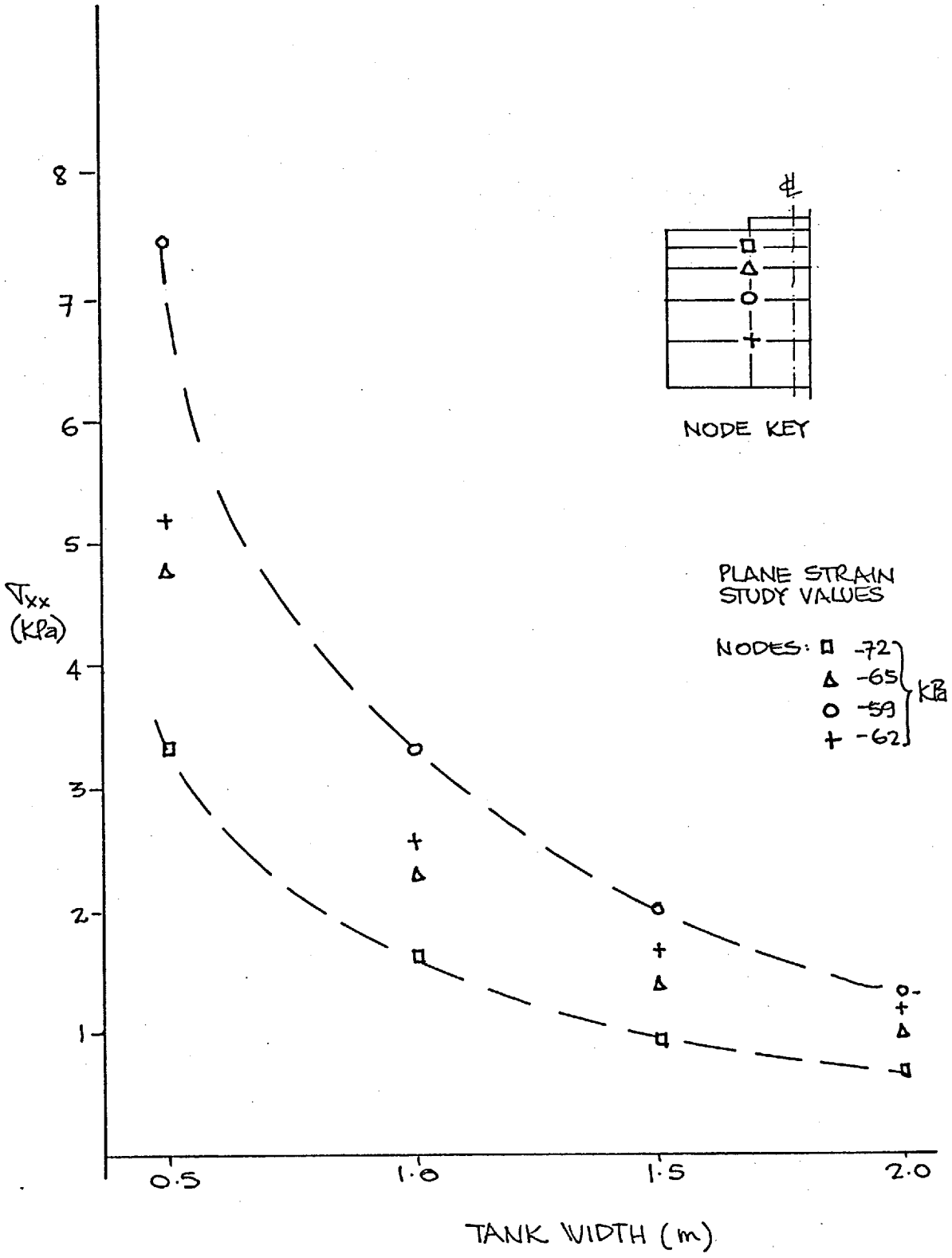
KEY : 3D STUDY VALUE / PLANE STRAIN STUDY VALUE

UNITS: $m \times 10^{-5}$ TANK WIDTH 1.0m

LATERAL DISPLACEMENT CASE – SAND

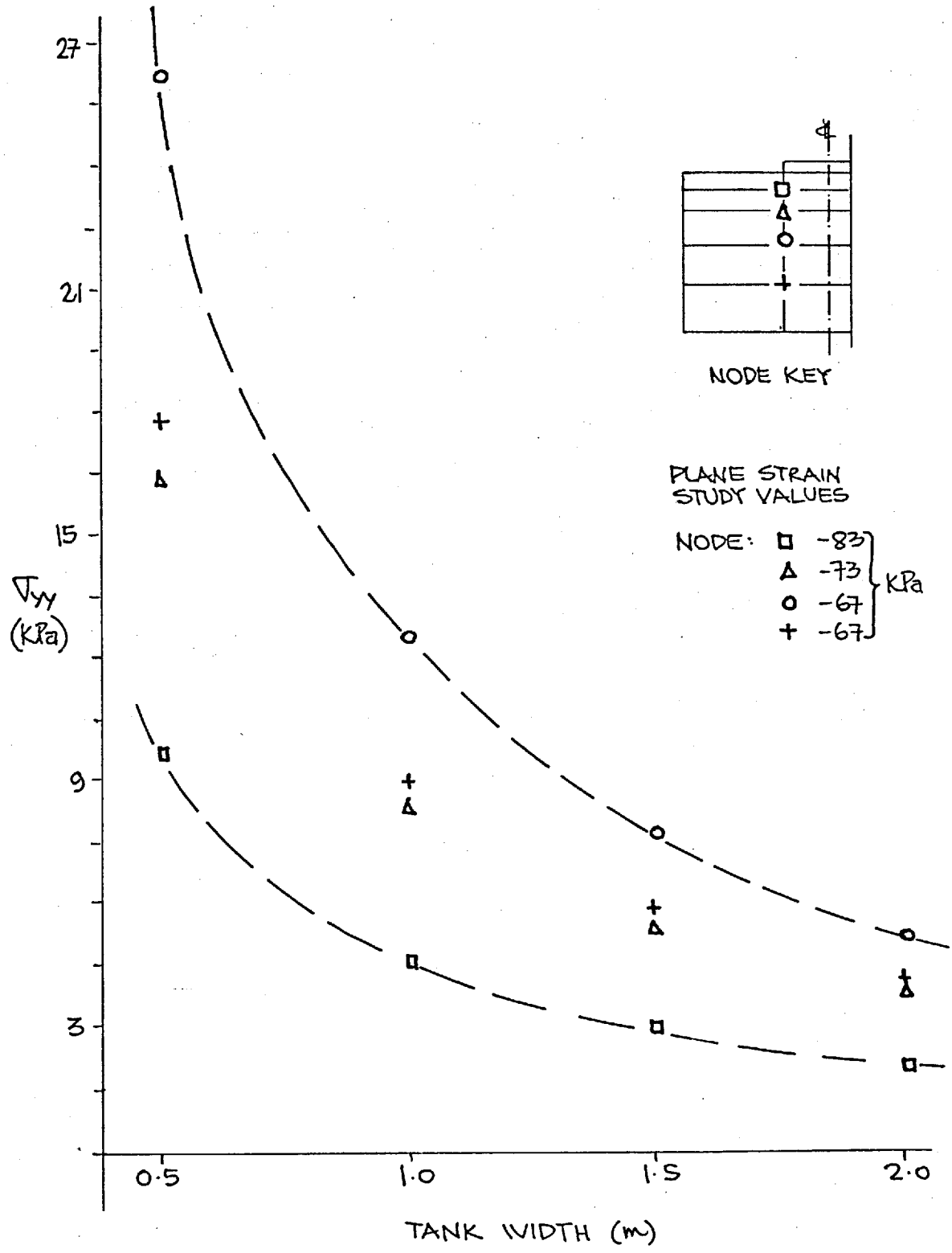
δ_z COMPARISON

FIGURE 17



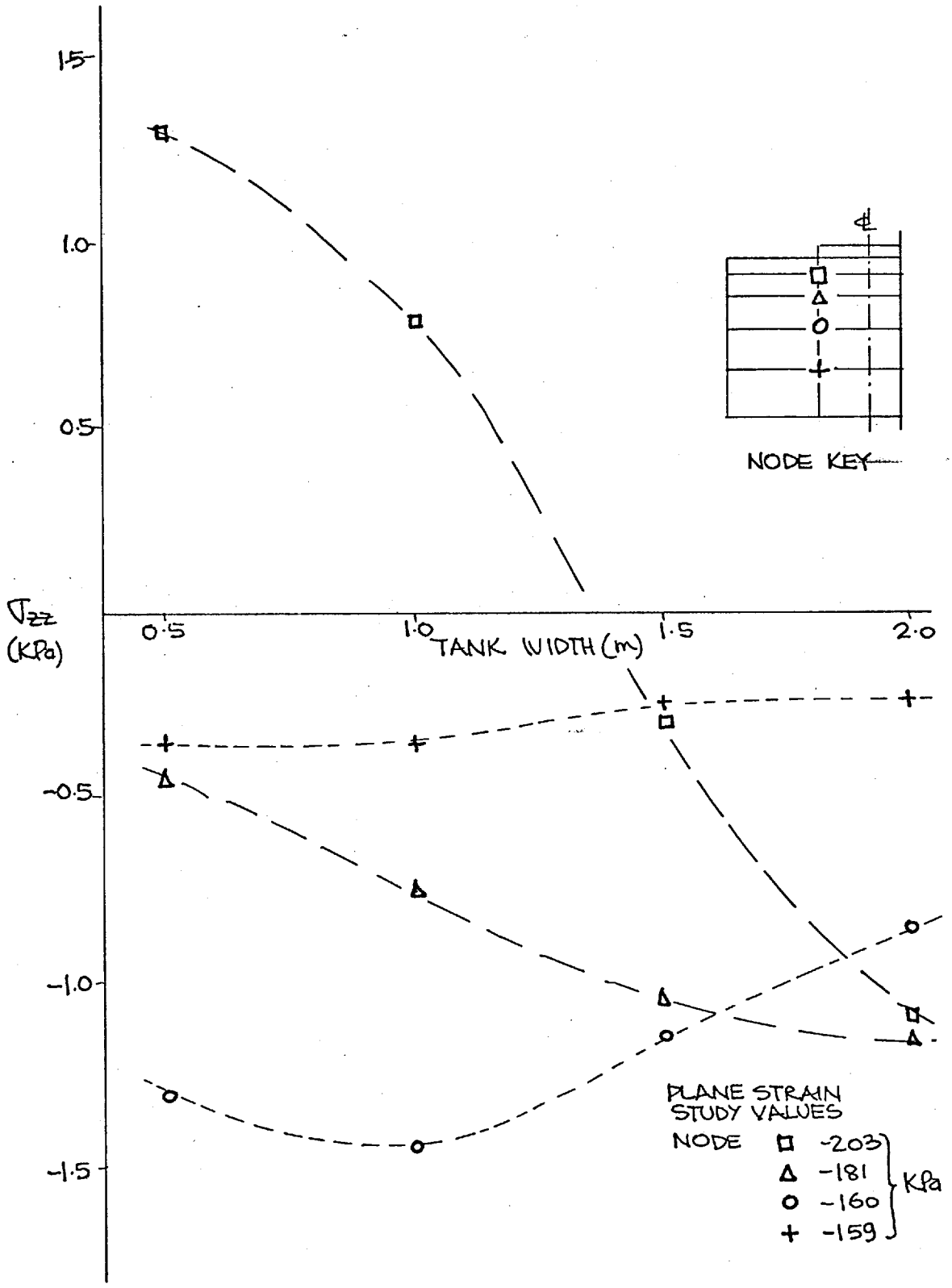
LATERAL DISPLACEMENT CASE - SAND - 3D STUDY
∇_{xx} VARIATION

FIGURE 18



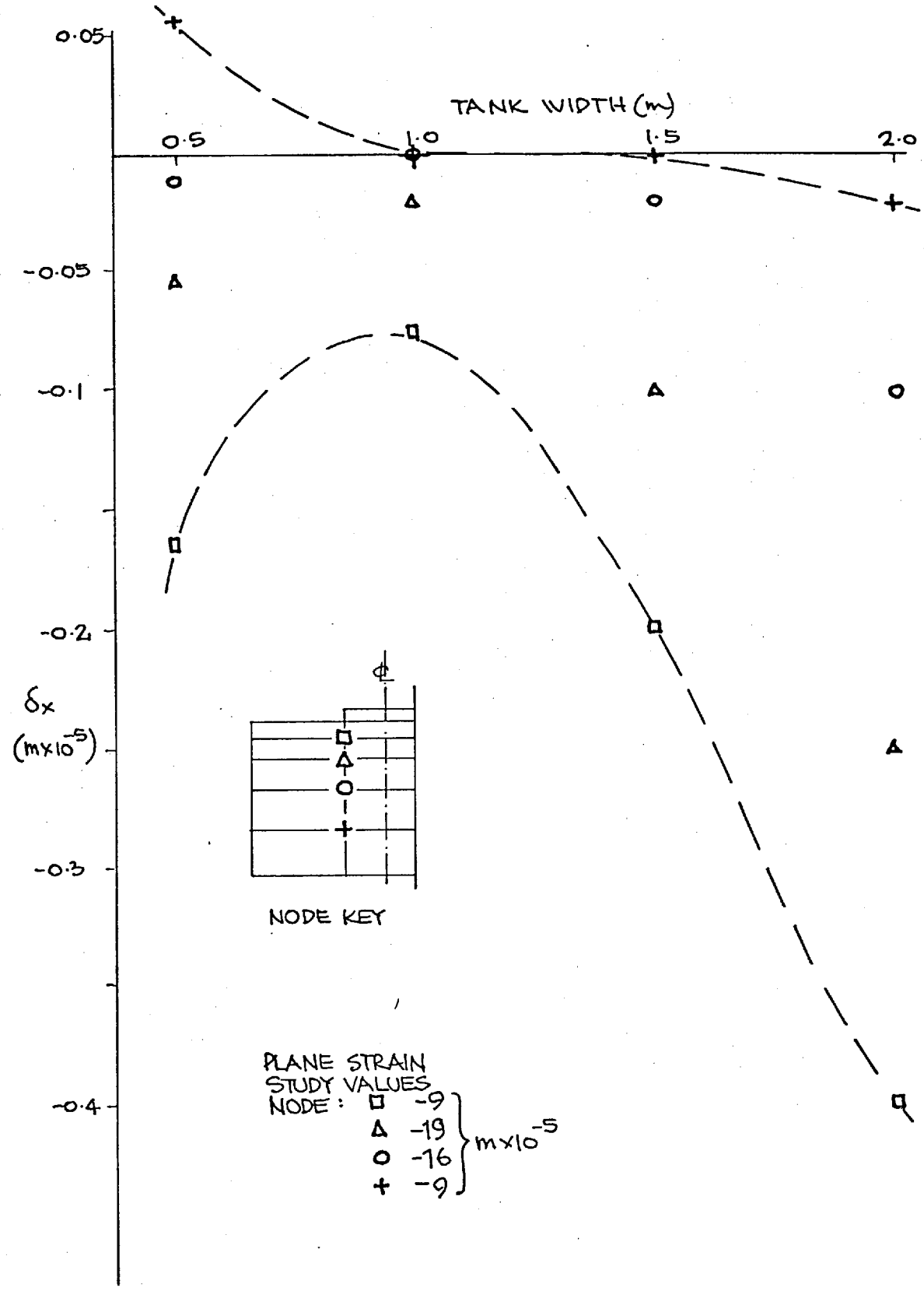
LATERAL DISPLACEMENT CASE – SAND – 3D STUDY
∇_{yy} VARIATION

FIGURE 19



LATERAL DISPLACEMENT CASE - SAND - 3D STUDY
σ_{zz} VARIATION

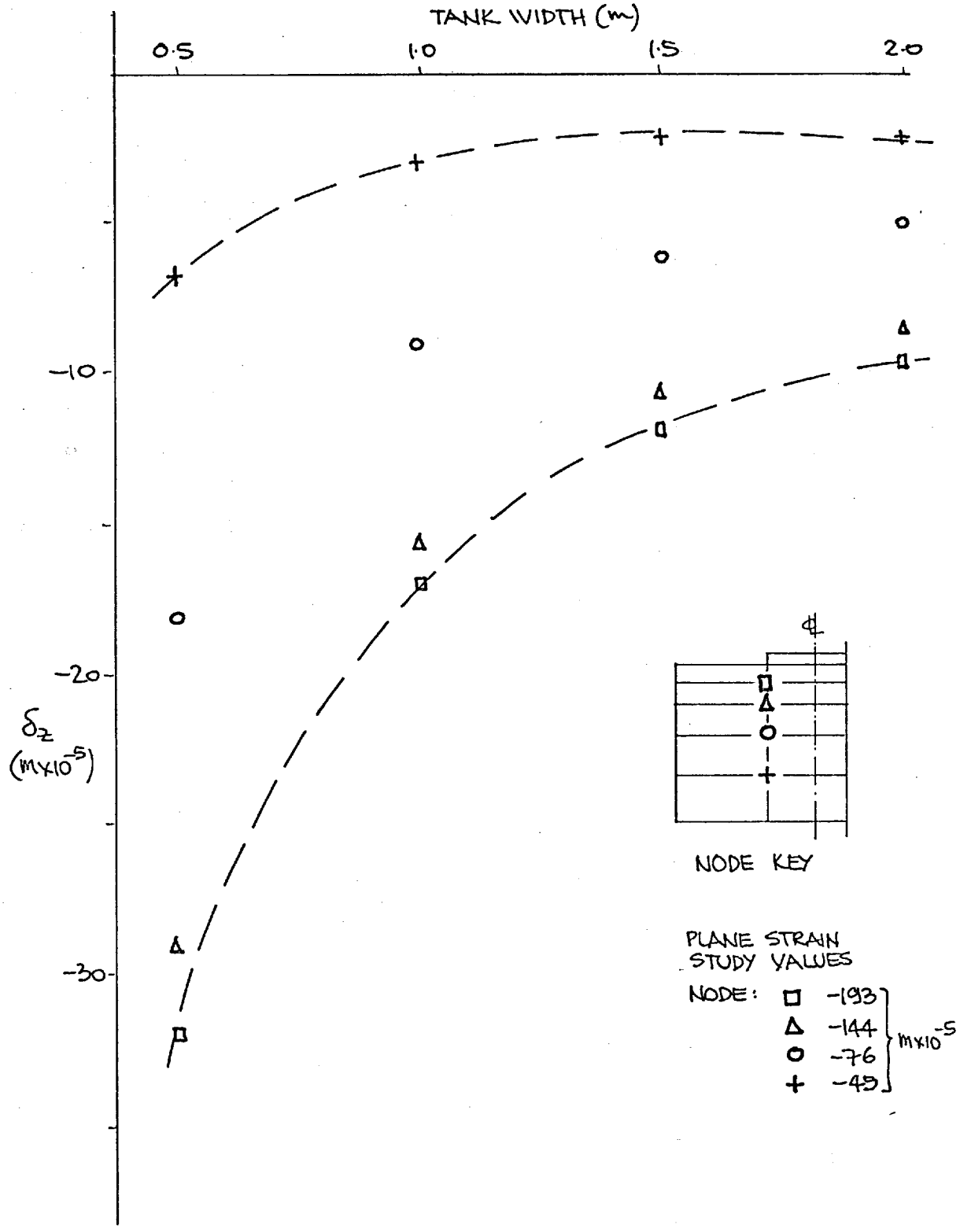
FIGURE 20



LATERAL DISPLACEMENT CASE - SAND - 3D STUDY

δ_x VARIATION

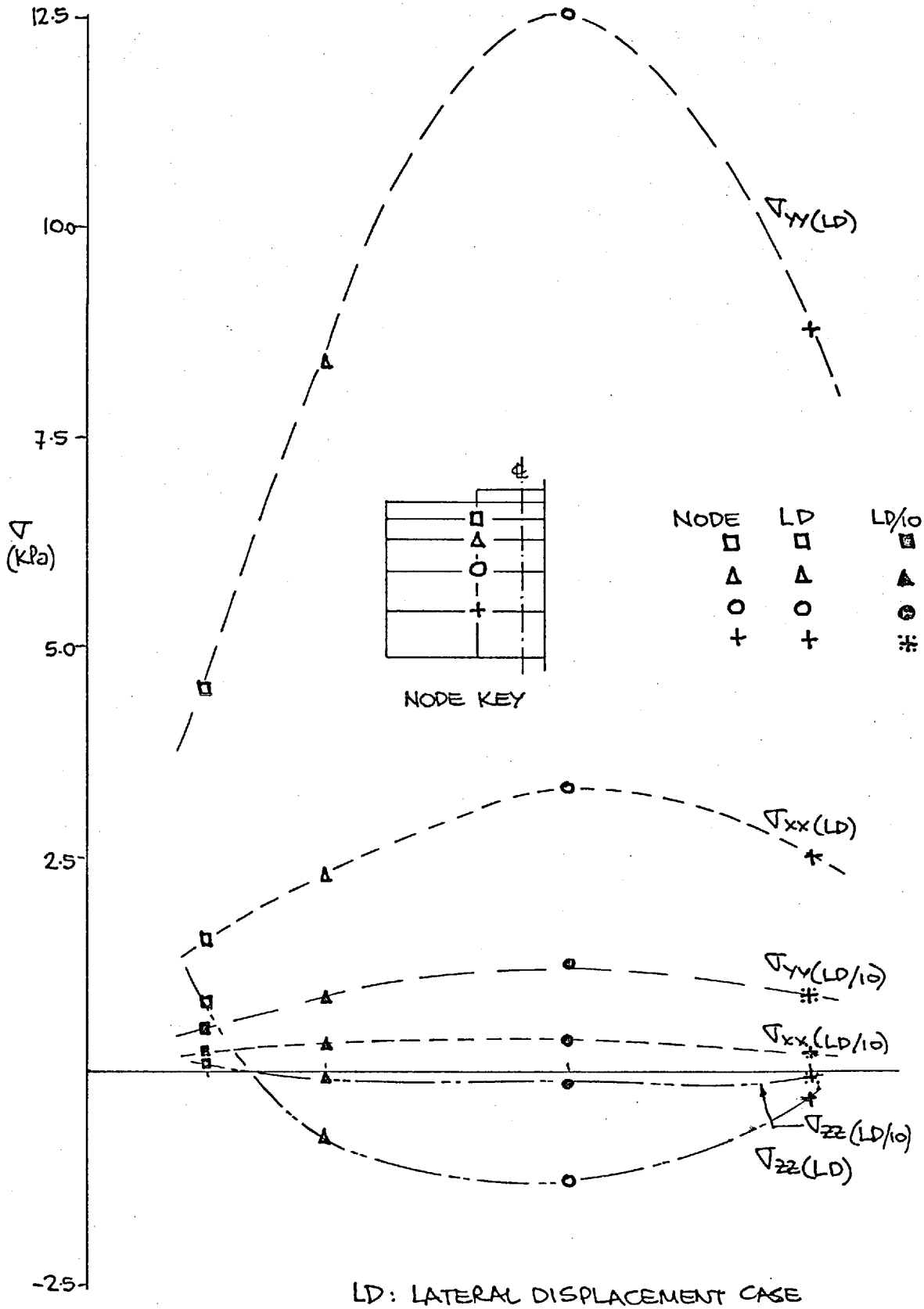
FIGURE 21



LATERAL DISPLACEMENT CASE - SAND - 3D STUDY

δ_z VARIATION

FIGURE 22



LD: LATERAL DISPLACEMENT CASE

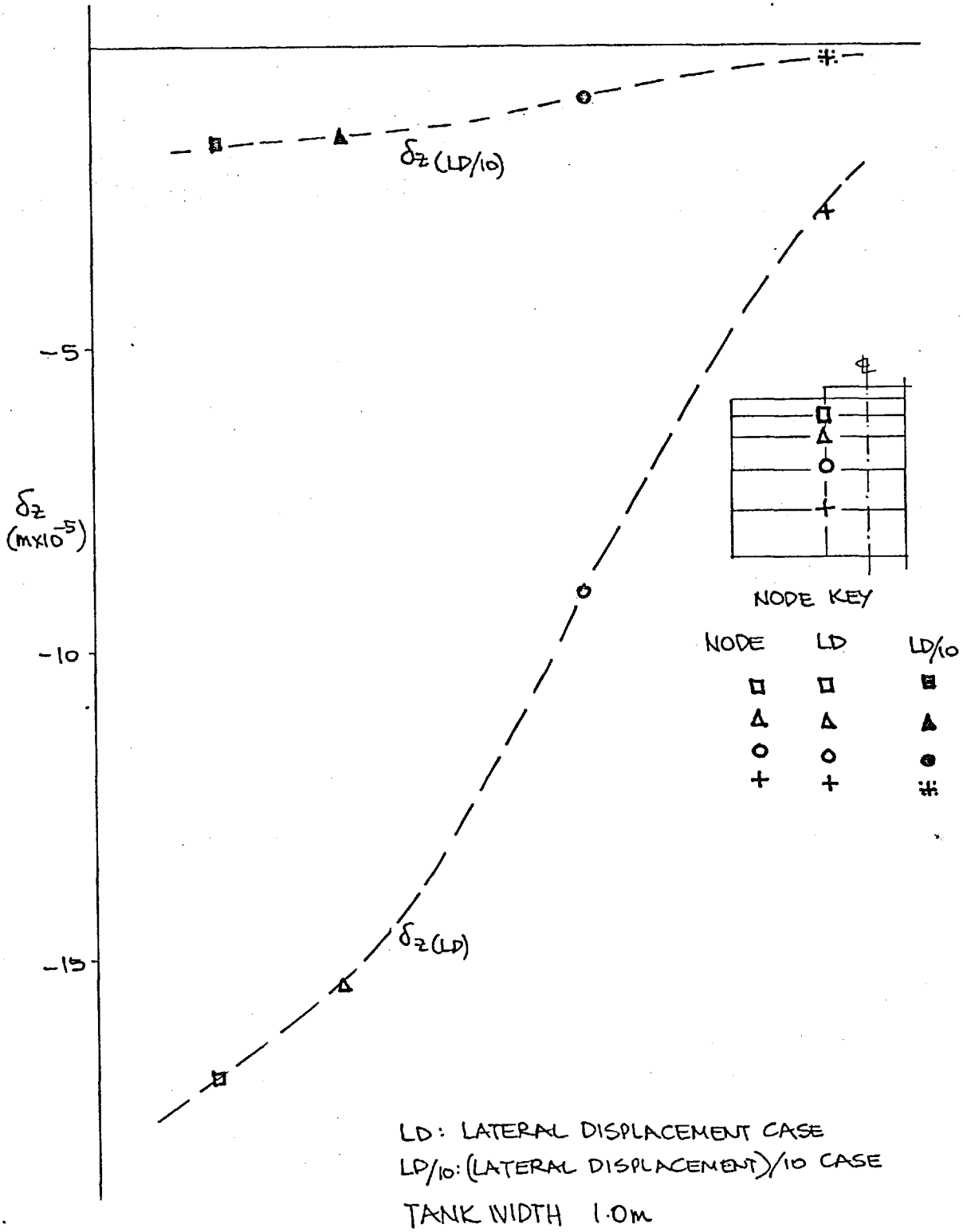
LD/10: (LATERAL DISPLACEMENT)/10 CASE

TANK WIDTH – 1.0 m

LATERAL DISPLACEMENT CASE – SAND – 3D STUDY

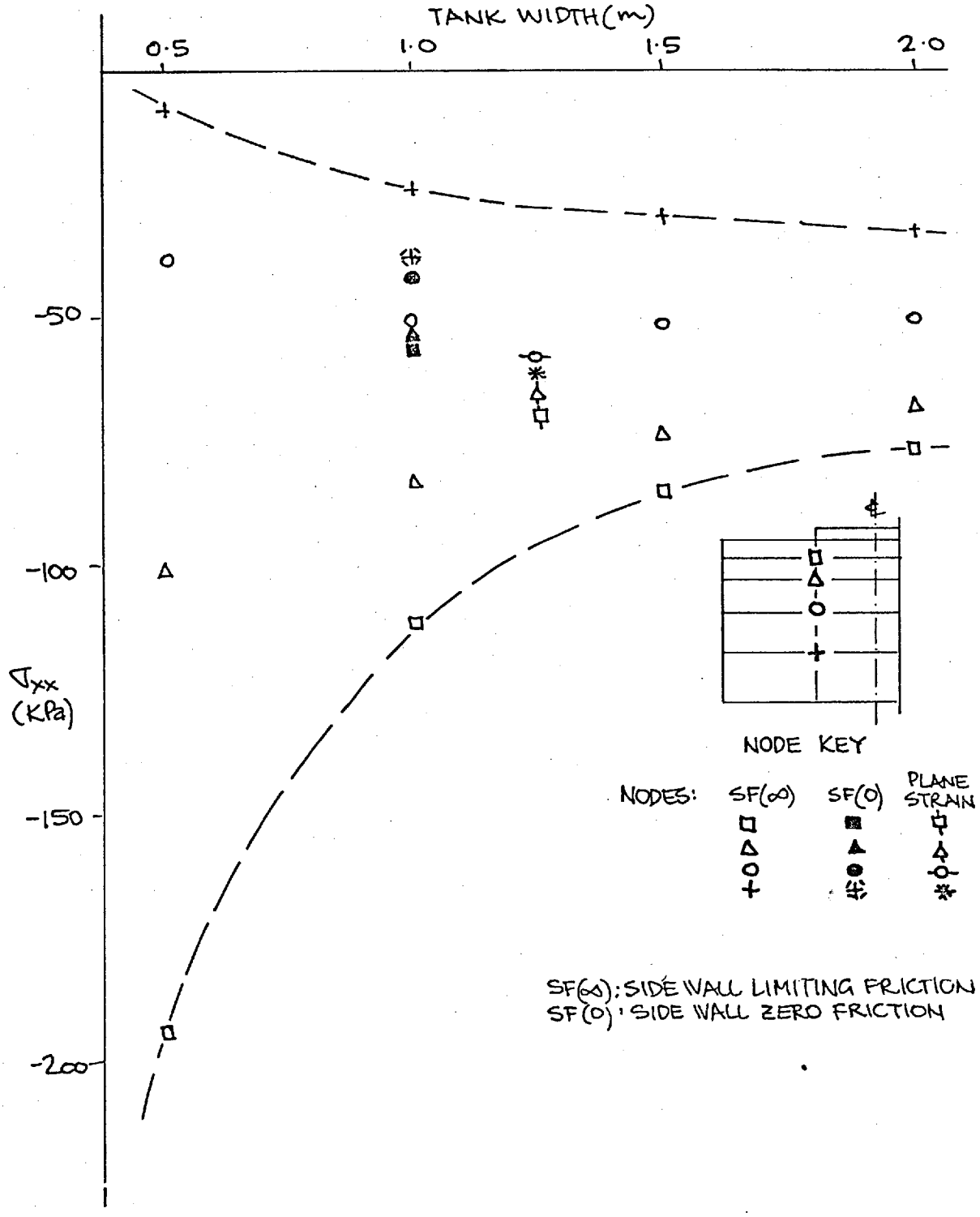
STRESS COMPARISONS

FIGURE 23



LATERAL DISPLACEMENT CASE - SAND-3D STUDY
δ COMPARISON

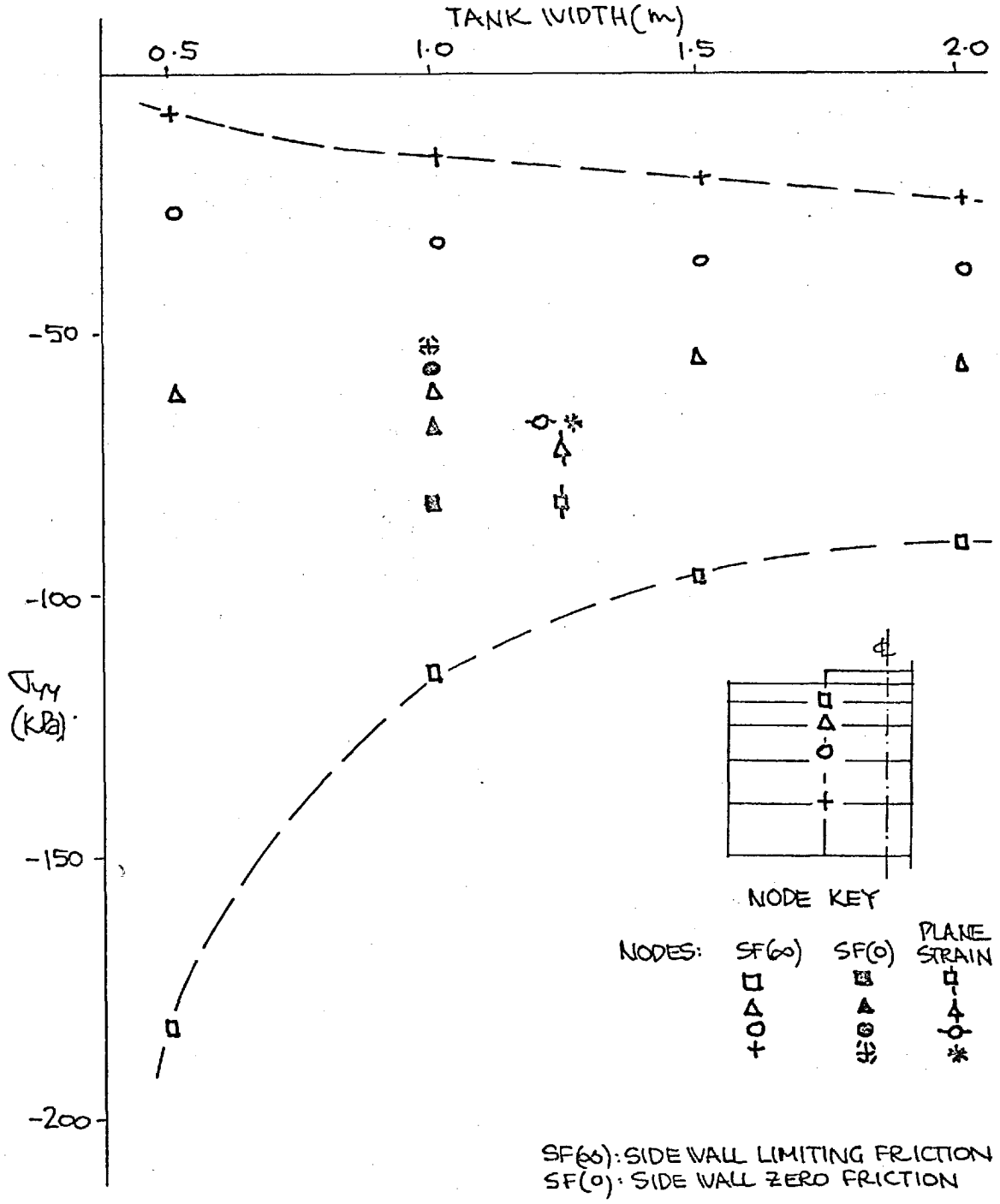
FIGURE 24



SIDE FRICTION CASE - SAND - 3D STUDY

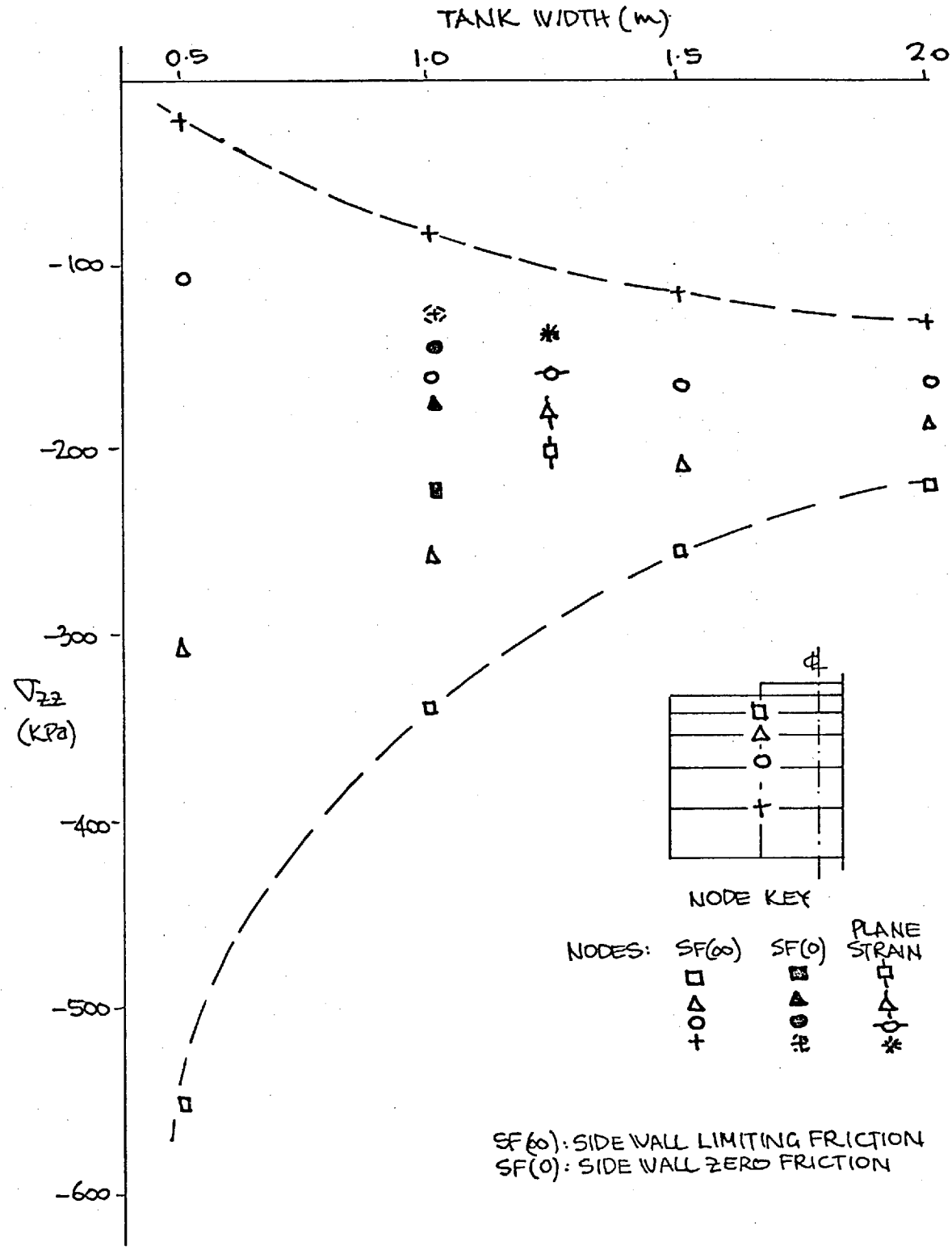
σ_{xx} VARIATION

FIGURE 25



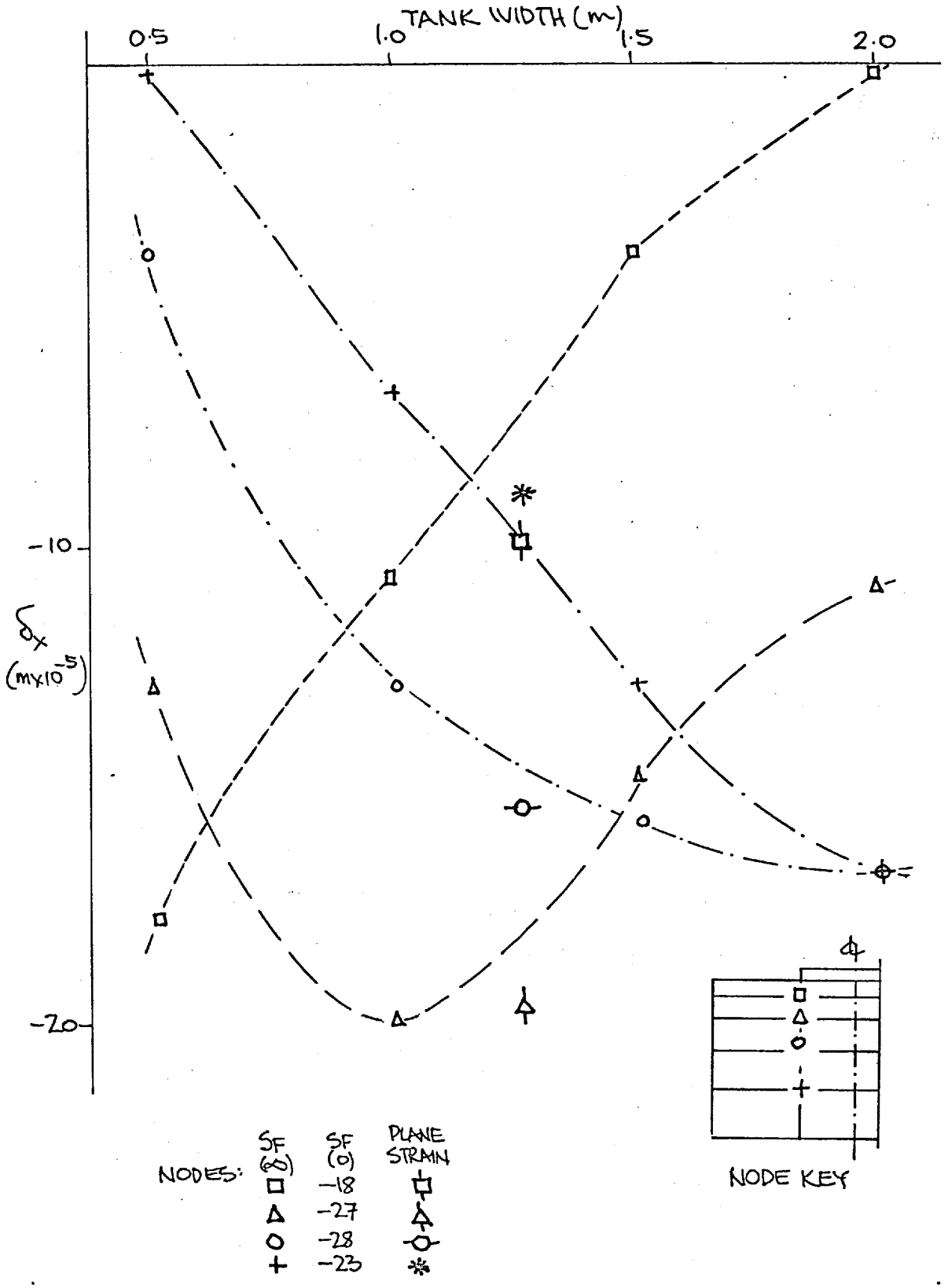
SIDE FRICTION CASE - SAND - 3D STUDY
σ_{yy} VARIATION

FIGURE 26



SIDE FRICTION CASE - SAND - 3D STUDY
∇_{zz} VARIATION

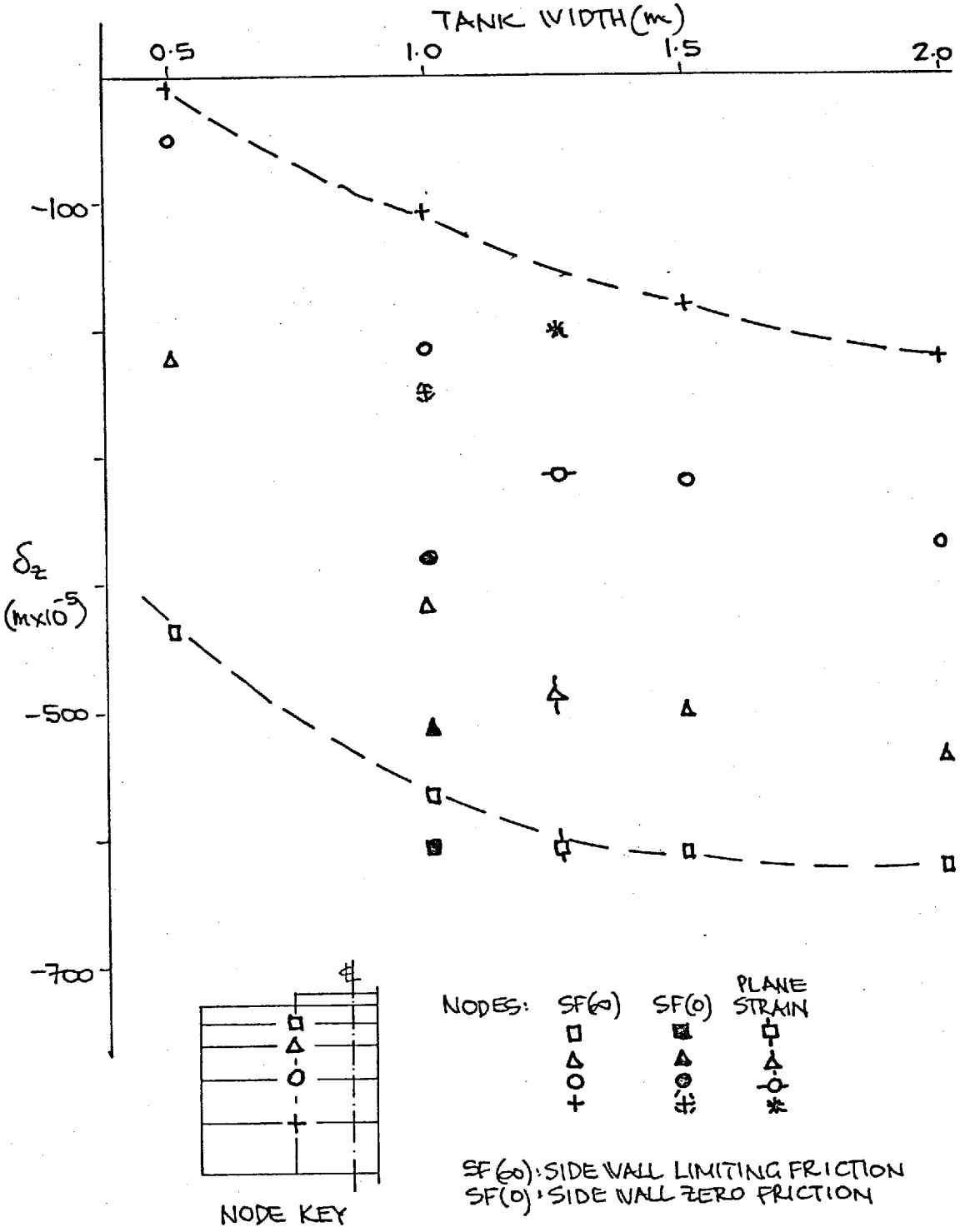
FIGURE 27



SF(∞): SIDEWALL LIMITING FRICTION
SF(0): SIDEWALL ZERO FRICTION

SIDE FRICTION CASE - SAND - 3D STUDY
δ_x VARIATION

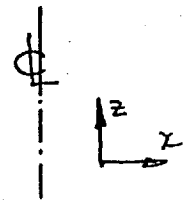
FIGURE 28



SIDE FRICTION CASE - SAND - 3D STUDY

δ_z VARIATION

FIGURE 29



	1.14	1.13	1.15	1.07	1.83	1.82	1.79
	1.76	1.52	1.45	1.52	1.43	1.42	1.42
	1.48	1.57	1.45	1.48	1.60	1.63	1.64
	1.92	1.95	1.88	1.89	1.88	1.92	1.92

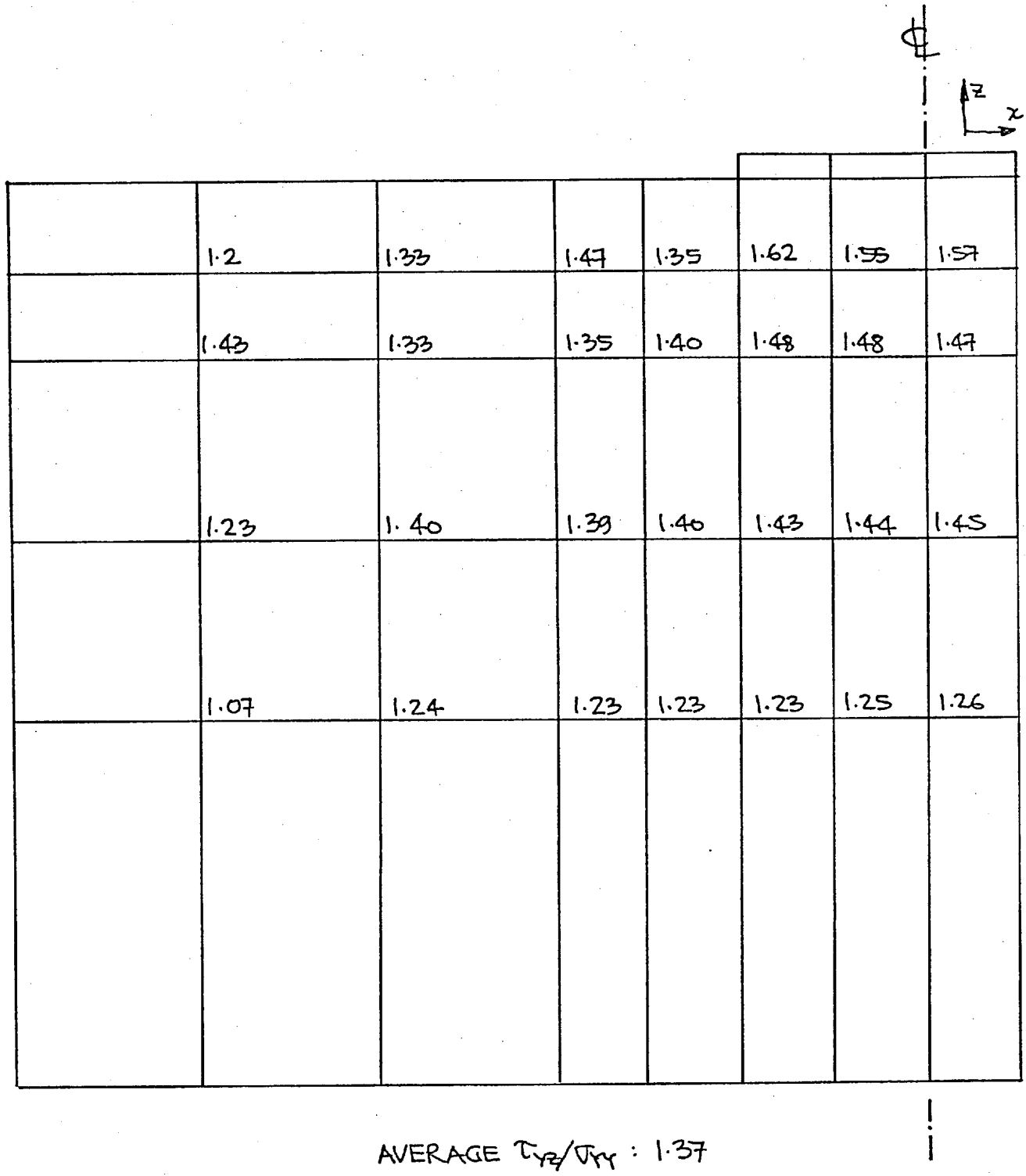
AVERAGE τ_{yz}/σ_{yy} : 1.59

τ_{yx}/σ_{yy} (DISTRIBUTION NOT SHOWN)
AVERAGE : 0.08

TANK WIDTH 0.5m

SIDE FRICTION CASE - SAND - 3D STUDY
COEFFICIENT τ_{yz}/σ_{yy}

FIGURE 30



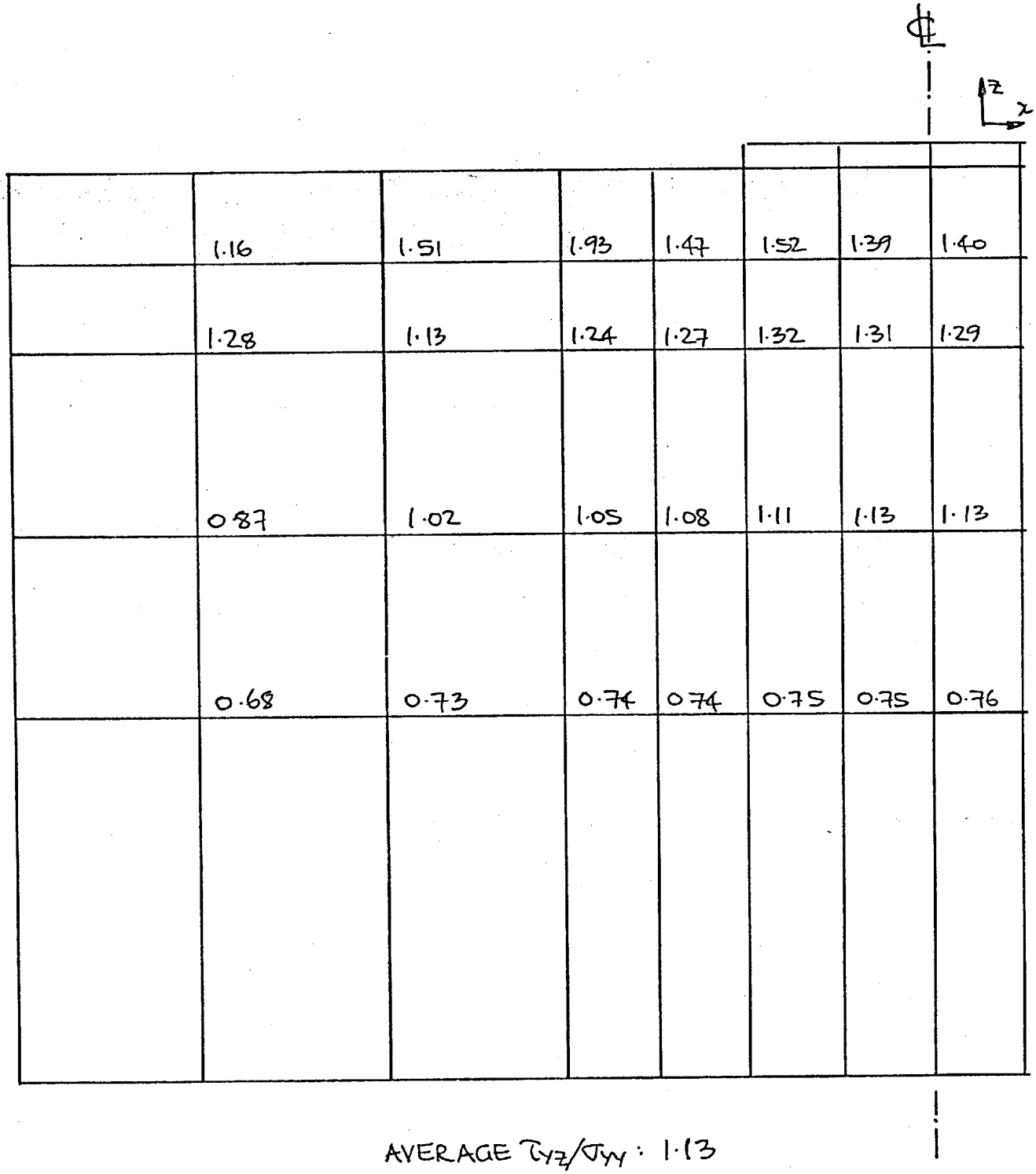
AVERAGE τ_{yz}/σ_{yy} : 1.37

τ_{yx}/σ_{yy} (DISTRIBUTION NOT SHOWN)
AVERAGE : 0.09

TANK WIDTH 1.0m

SIDE FRICTION CASE – SAND – 3D STUDY
COEFFICIENT τ_{yz}/σ_{yy}

FIGURE 31



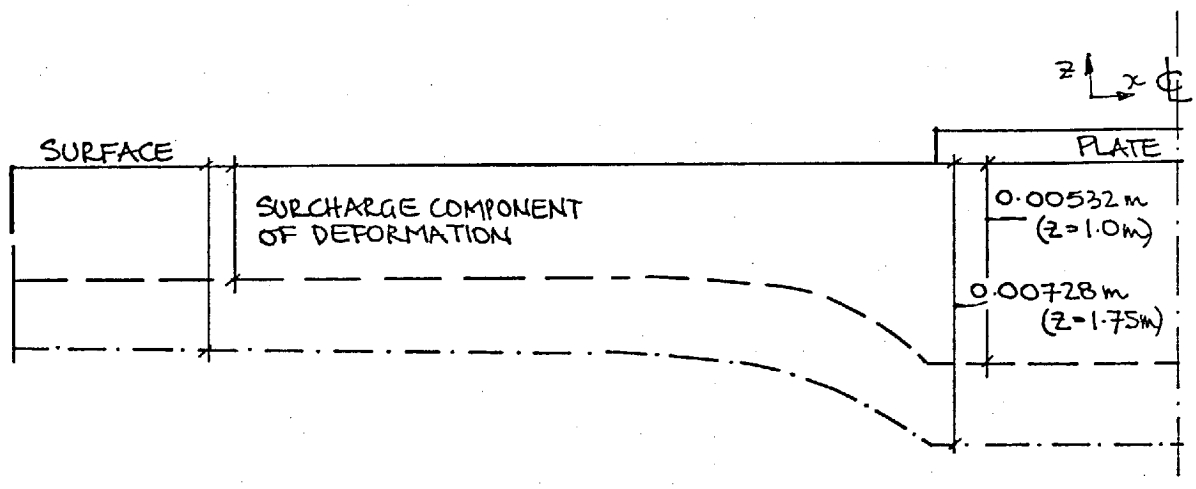
AVERAGE τ_{yz}/σ_{yy} : 1.13

τ_{yx}/σ_{yy} (DISTRIBUTION NOT SHOWN)
AVERAGE : 0.09

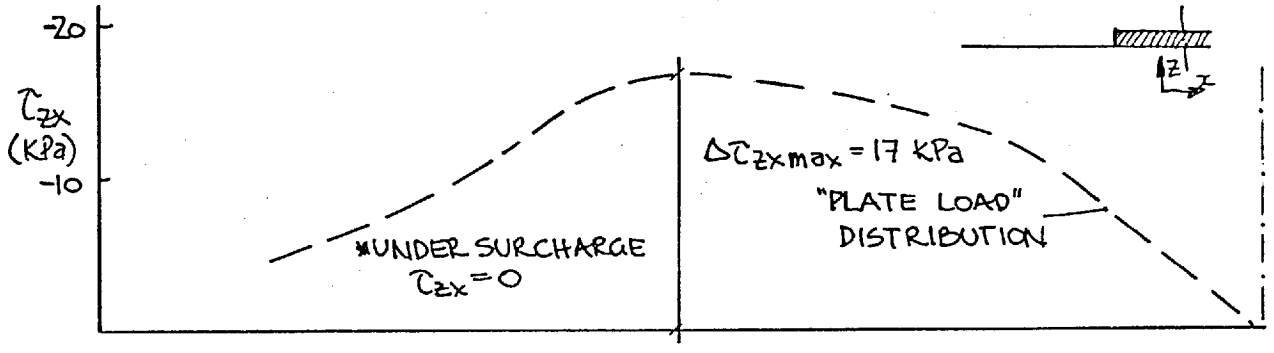
TANK WIDTH 2.0m

SIDE FRICTION CASE - SAND - 3D STUDY
COEFFICIENT τ_{yz}/σ_{yy}

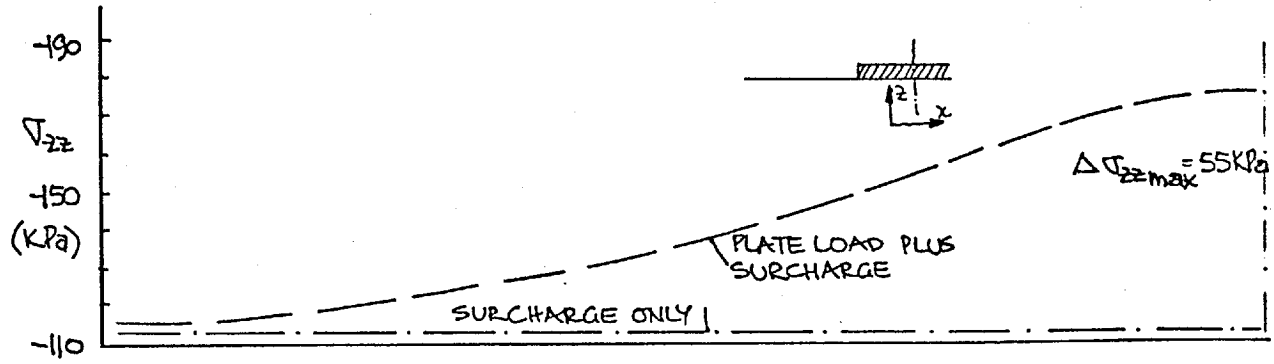
FIGURE 32



1. SURFACE DEFORMATION COMPARISON : DEPTHS (z) 1.75m/1.0m



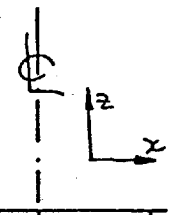
2. SHEAR STRESS NEAR BASE OF TANK : DEPTH (z) = 1.0m



3. NORMAL STRESS NEAR BASE OF TANK: DEPTH (z) = 1.0m

PLANE STRAIN STUDY – DEFORMATION & STRESS DETAILS

FIGURE 33



	1.17	0.69	2.50	1.50	1.40	1.26	1.30
	1.20 1.49	1.33 1.05	1.47 1.19	1.35 1.14	1.62 1.23	1.55 1.17	1.57 1.16
	1.43 0.94	1.33 0.86	1.35 0.83	1.40 0.88	1.45 0.95	1.48 0.97	1.47 0.97
	1.23 0.62	1.40 0.59	1.39 0.56	1.40 0.57	1.43 0.61	1.44 0.63	1.45 0.63
	1.07	1.24	1.23	1.23	1.23	1.25	1.26

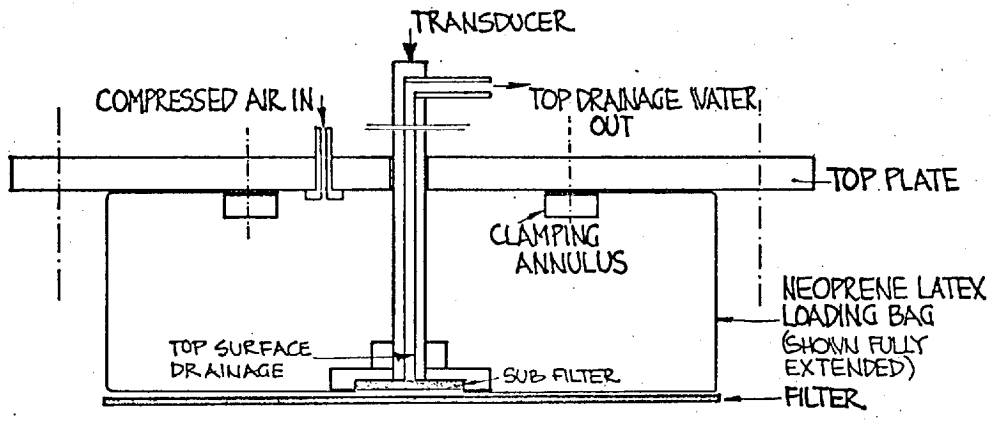
KEY :

3D STUDY τ_{yz}/σ_{yy} : TANK 3.5m x 1.0m DEEP x 1.5m WIDE

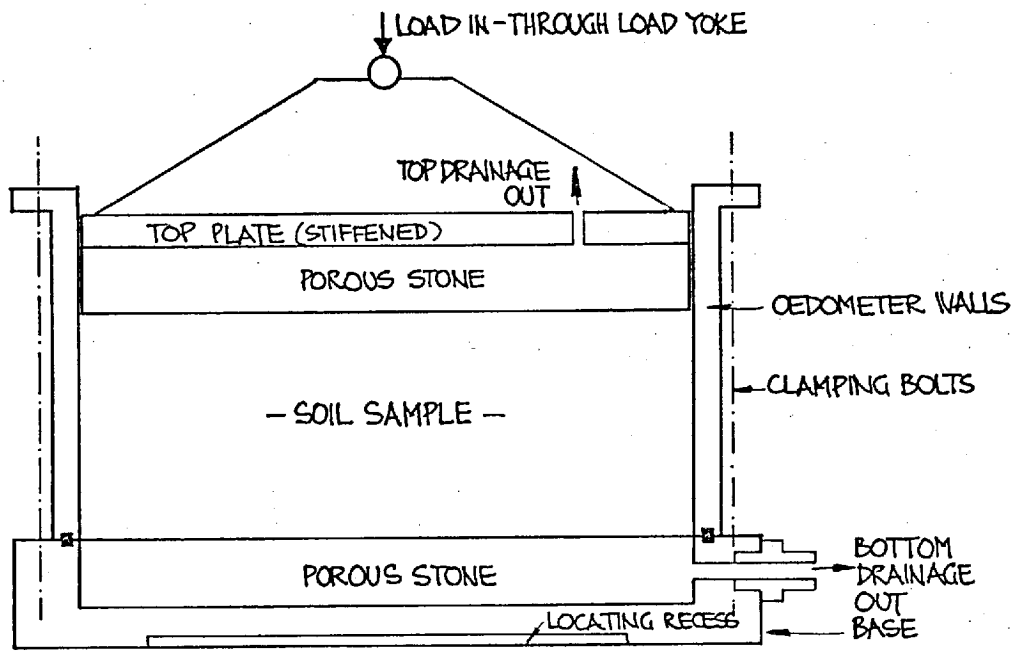
3D STUDY τ_{yz}/σ_{yy} : TANK 3.5m x 1.75m DEEP x 1.0m WIDE

SIDE FRICTION CASES - SAND - 3D STUDY
COMPARISON COEFFICIENTS τ_{yz}/σ_{yy}

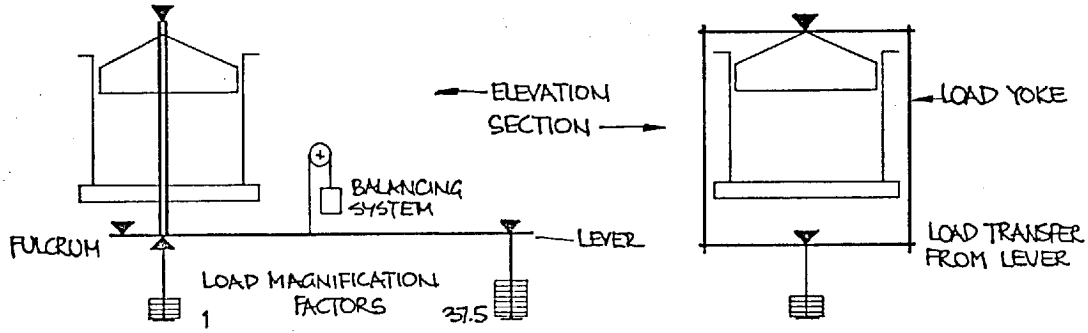
FIGURE 34



OEDOMETER MODIFICATION FOR PRESSURE LOADING



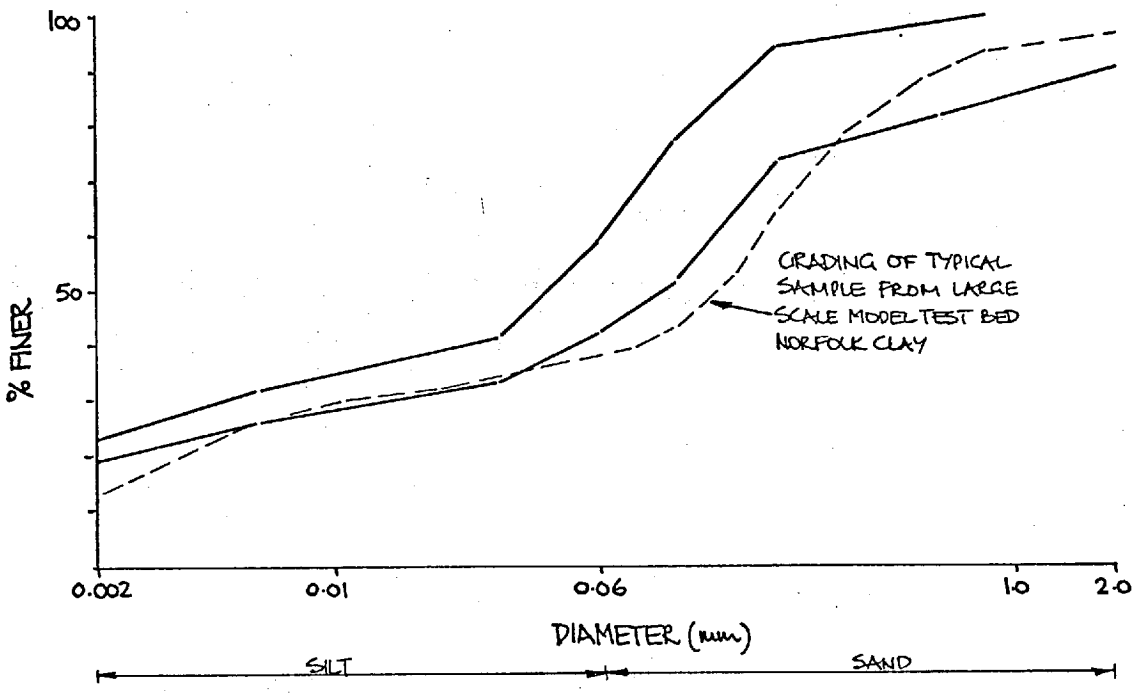
STANDARD OEDOMETER CONFIGURATION PLATE LOADING



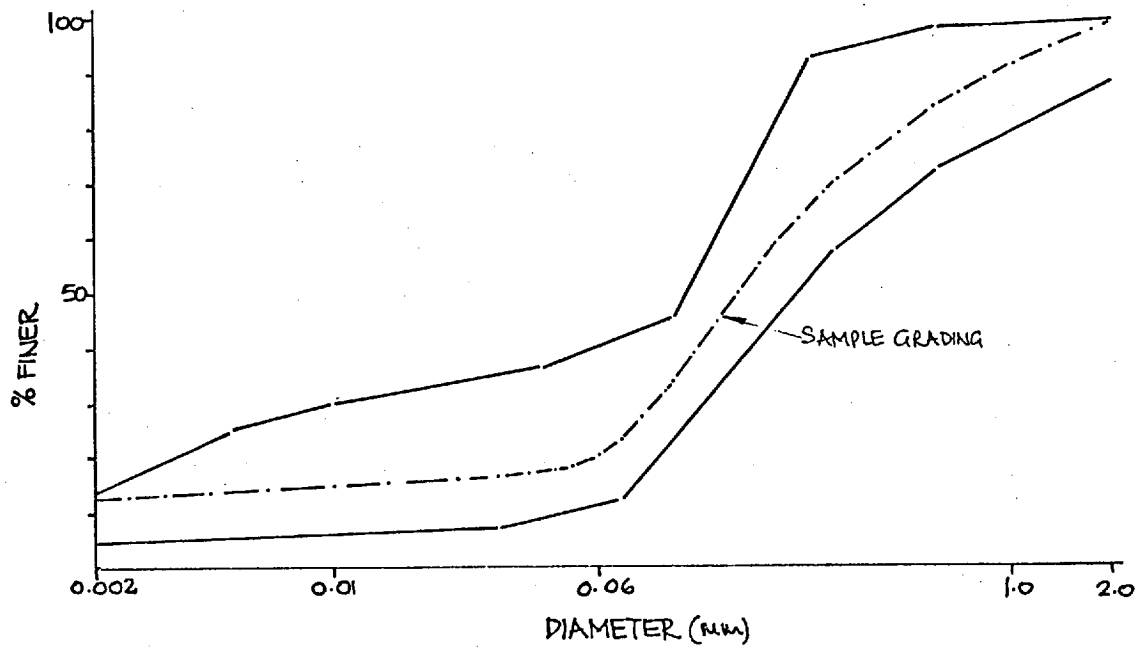
OEDOMETER LEVER LOAD APPLICATION SYSTEM (SCHEMATIC)

229 mm OEDOMETER DETAILS

FIGURE 35



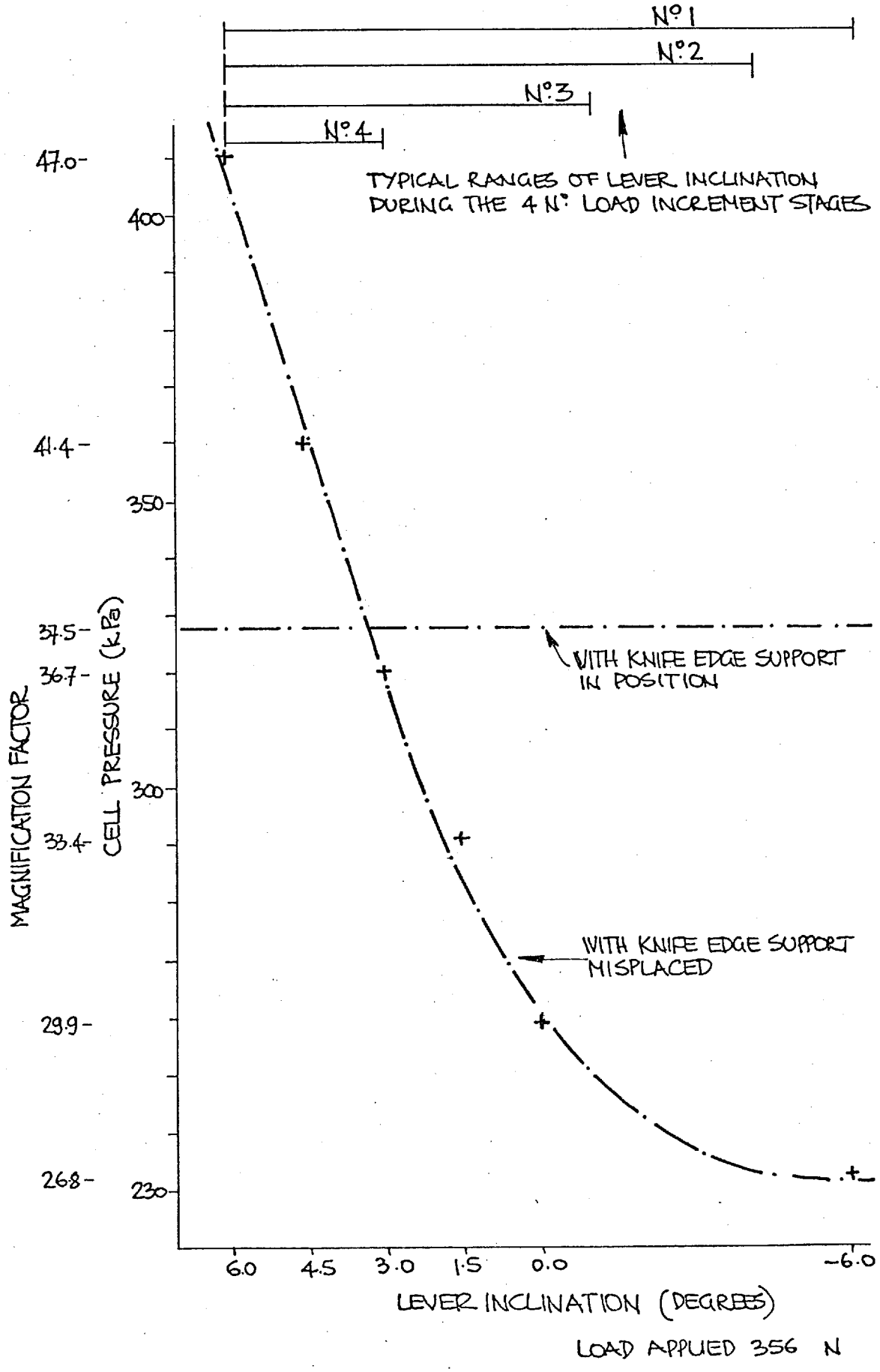
GRADING CURVE ENVELOPE FOR REPRESENTATIVE NORTH SEA BED SOILS
(from HIGHT, 1977)



GRADING CURVE ENVELOPE FOR NORFOLK CLAY SAMPLES (FROM HAPPISBURGH)
(from EL-GHAMRAWY, 1977)

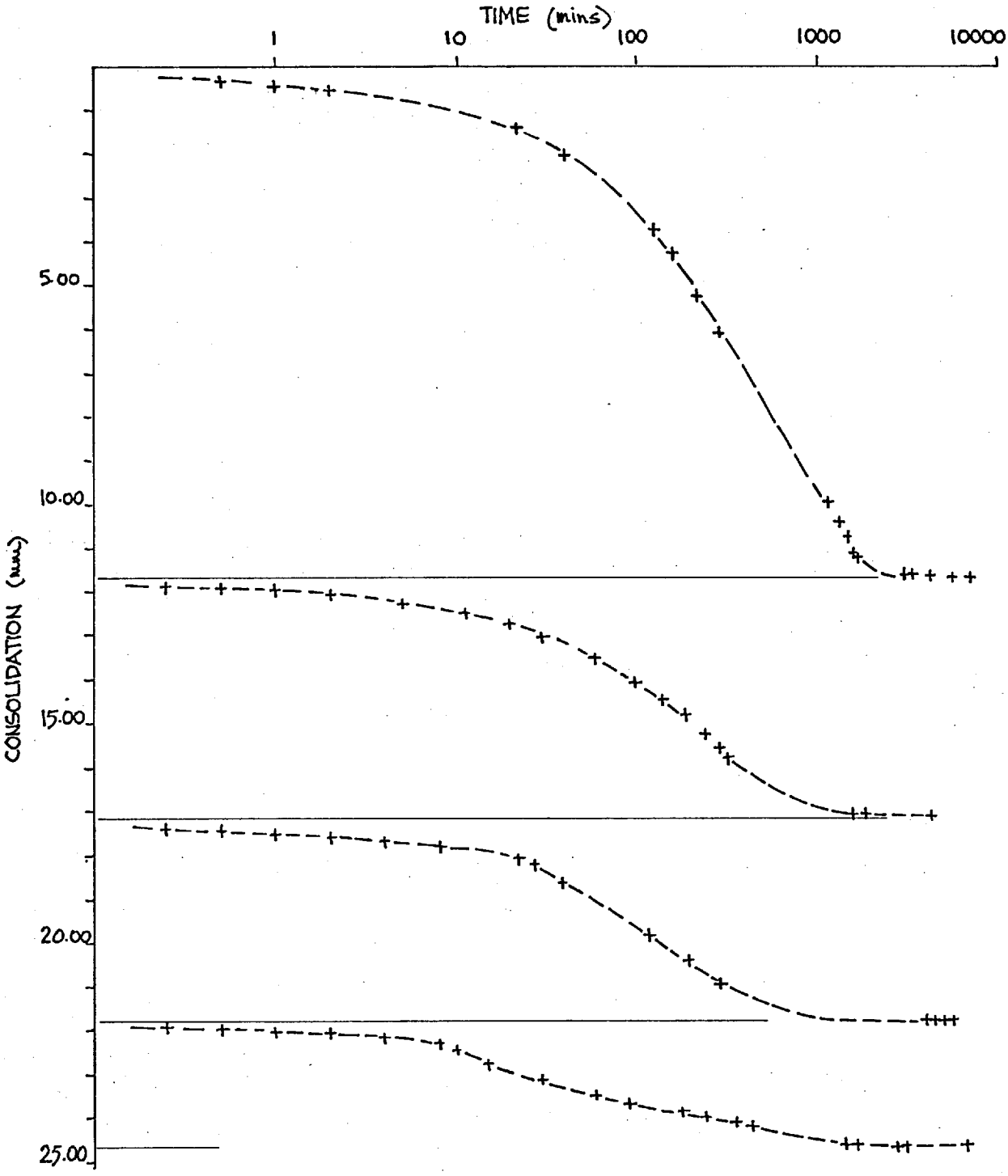
GRADING CURVES

FIGURE 36



CALIBRATION OF 229 mm OEDONETER

FIGURE 37



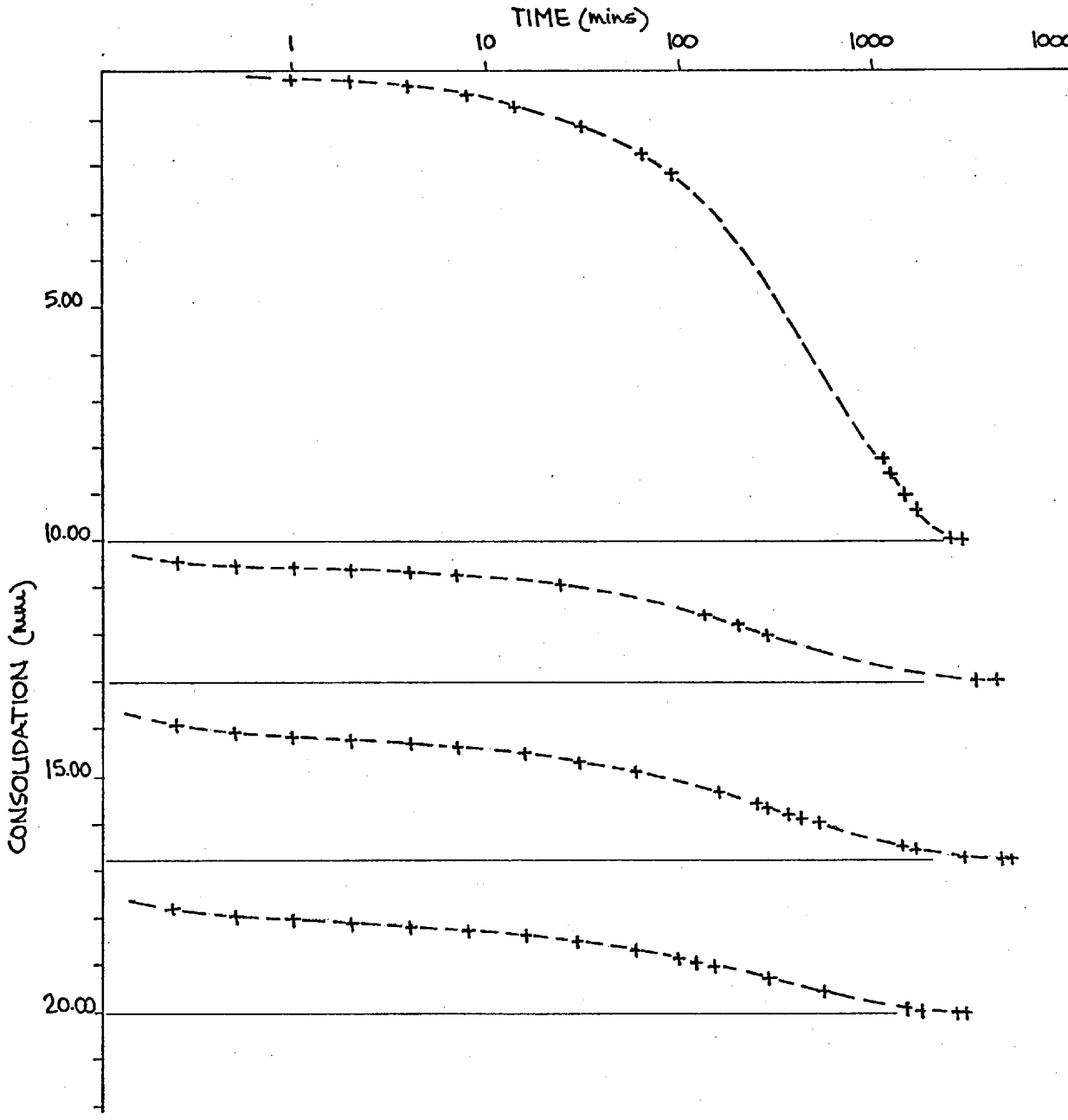
LOAD INCREMENTS: N°1 : 13 kPa
N°2 : 36 kPa
N°3 : 120 kPa
N°4 : 325 kPa

INITIAL MOISTURE CONTENT : 33.7%
FINAL MOISTURE CONTENT : 15.7%

TOTAL TIME : 409 HOURS

229 mm OEDOMETER : TEST 3 : CONSOLIDATION HISTORY

FIGURE 38



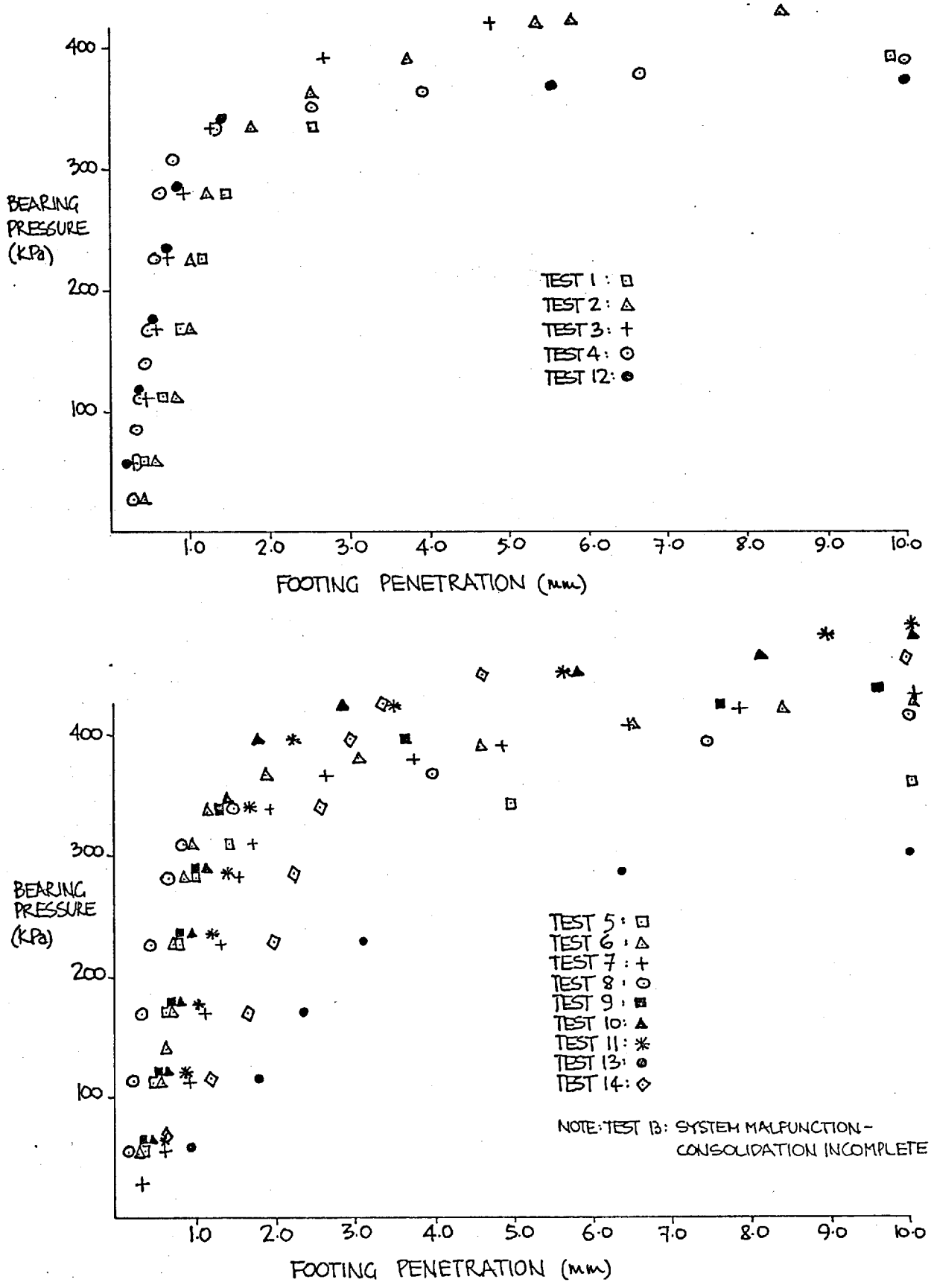
	TOTAL STRESS	BASE SUCTION
LOAD INCREMENT : N° 1:	13 kPa,	-60 kPa
N° 2:	36 kPa,	-62 kPa
N° 3:	120 kPa,	-65 kPa
N° 4:	325 kPa,	-68 kPa

INITIAL MOISTURE CONTENT : 30·2 %
FINAL (TOP) MOISTURE CONTENT : 15·5 %
FINAL (BTM) MOISTURE CONTENT : 13·3 %

TOTAL TIME : 216 hours

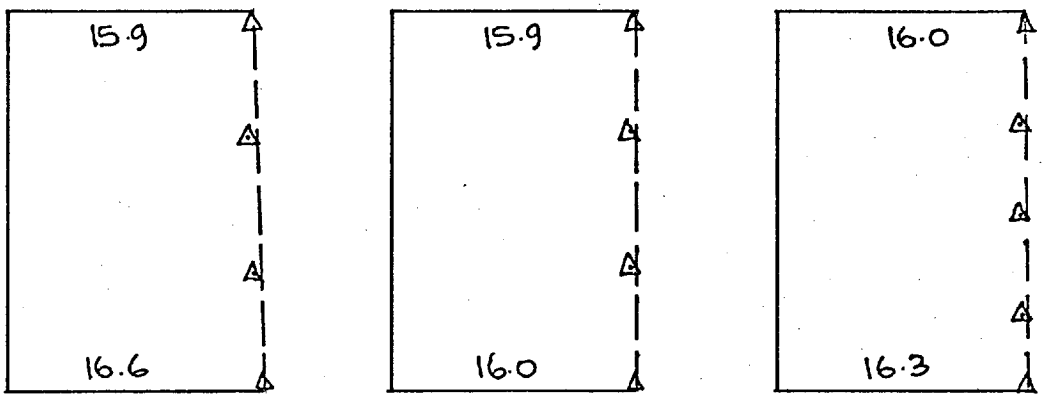
229 mm OEDOMETER : TEST 11 : CONSOLIDATION HISTORY

FIGURE 39

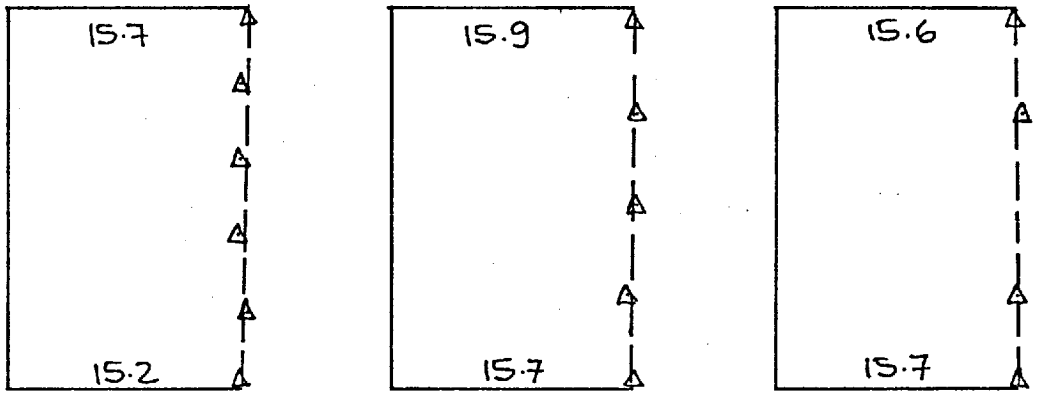


229 mm OEDOMETER TESTS : BEARING PRESSURE vs FOOTING PENETRATION

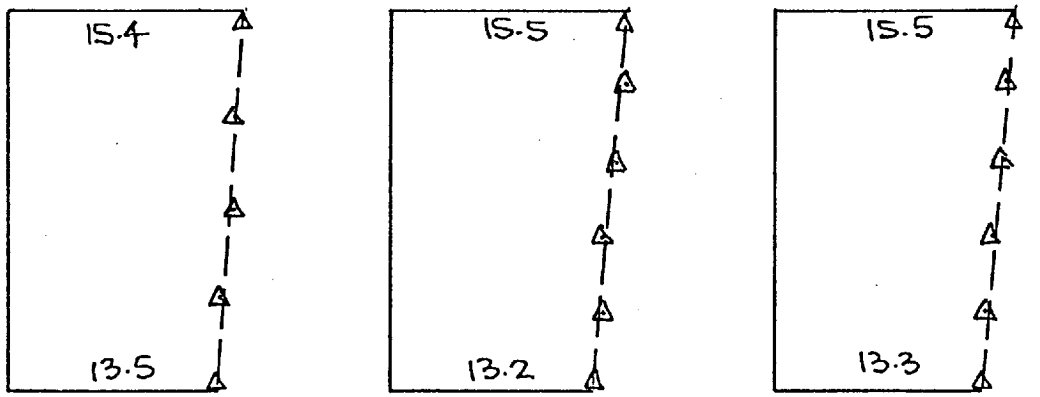
FIGURE 40



TEST 12 : NO PARTIAL VACUUM



TEST 6 : PARTIAL VACUUM -35KPa (approx)

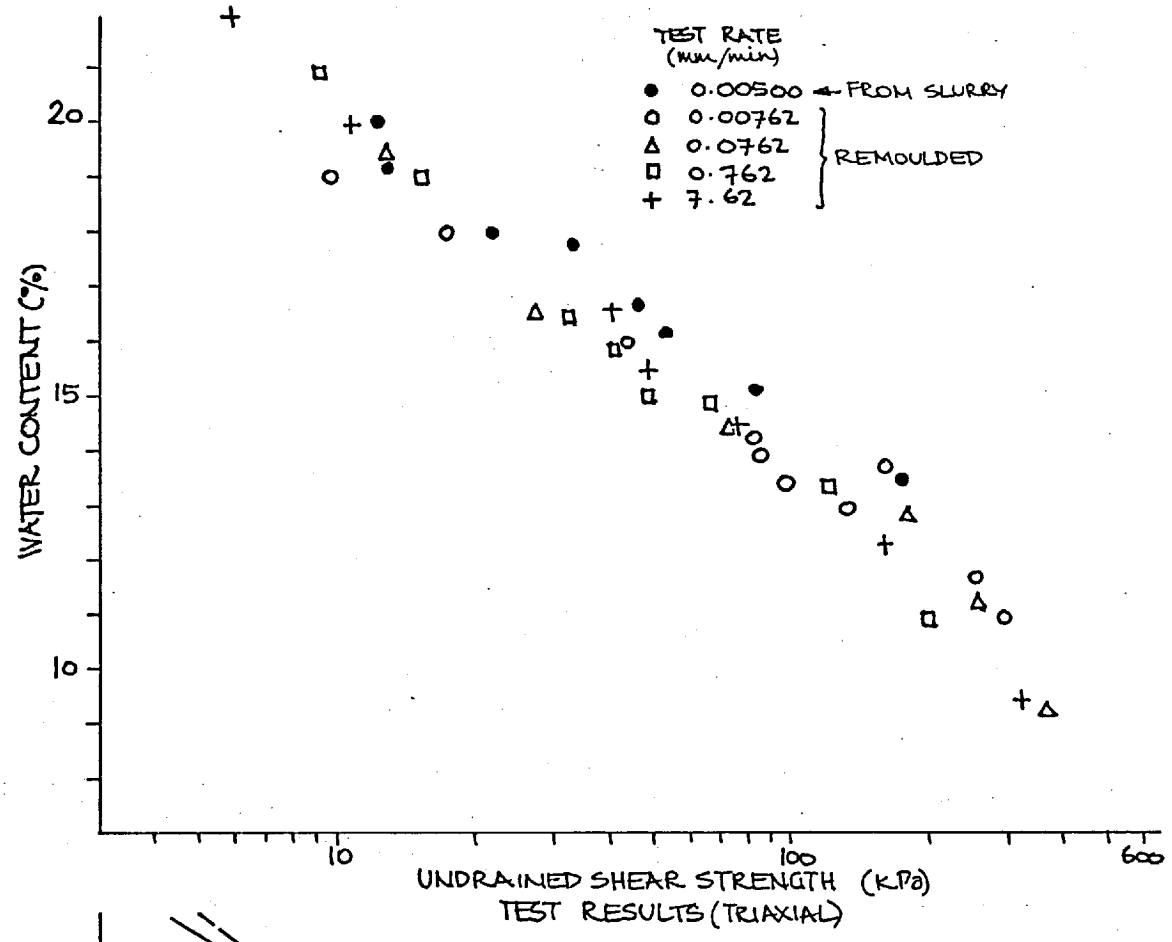


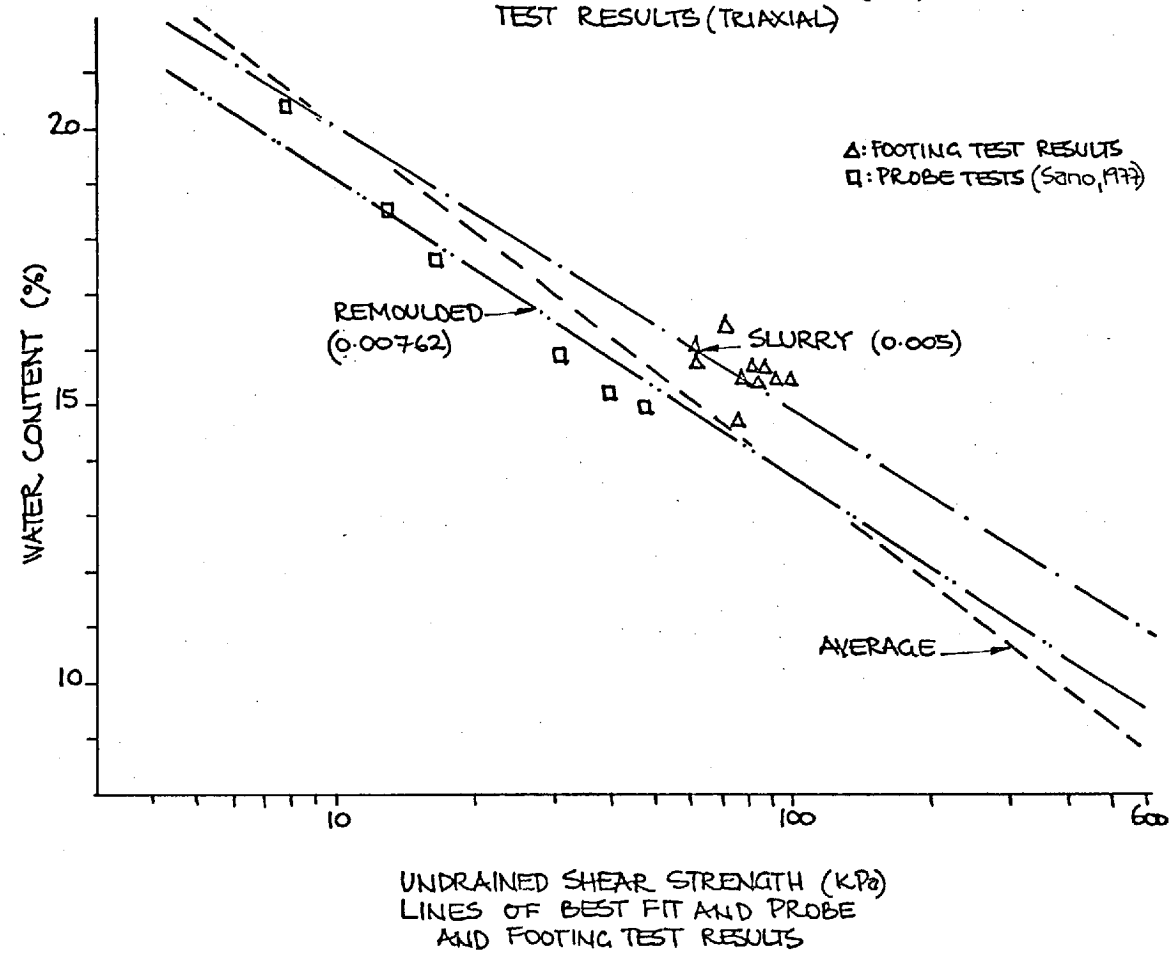
TEST 11 : PARTIAL VACUUM -70KPa (approx)

229mm OEDOMETER TESTS: FINAL MOISTURE CONTENTS:

SAMPLE PROFILES

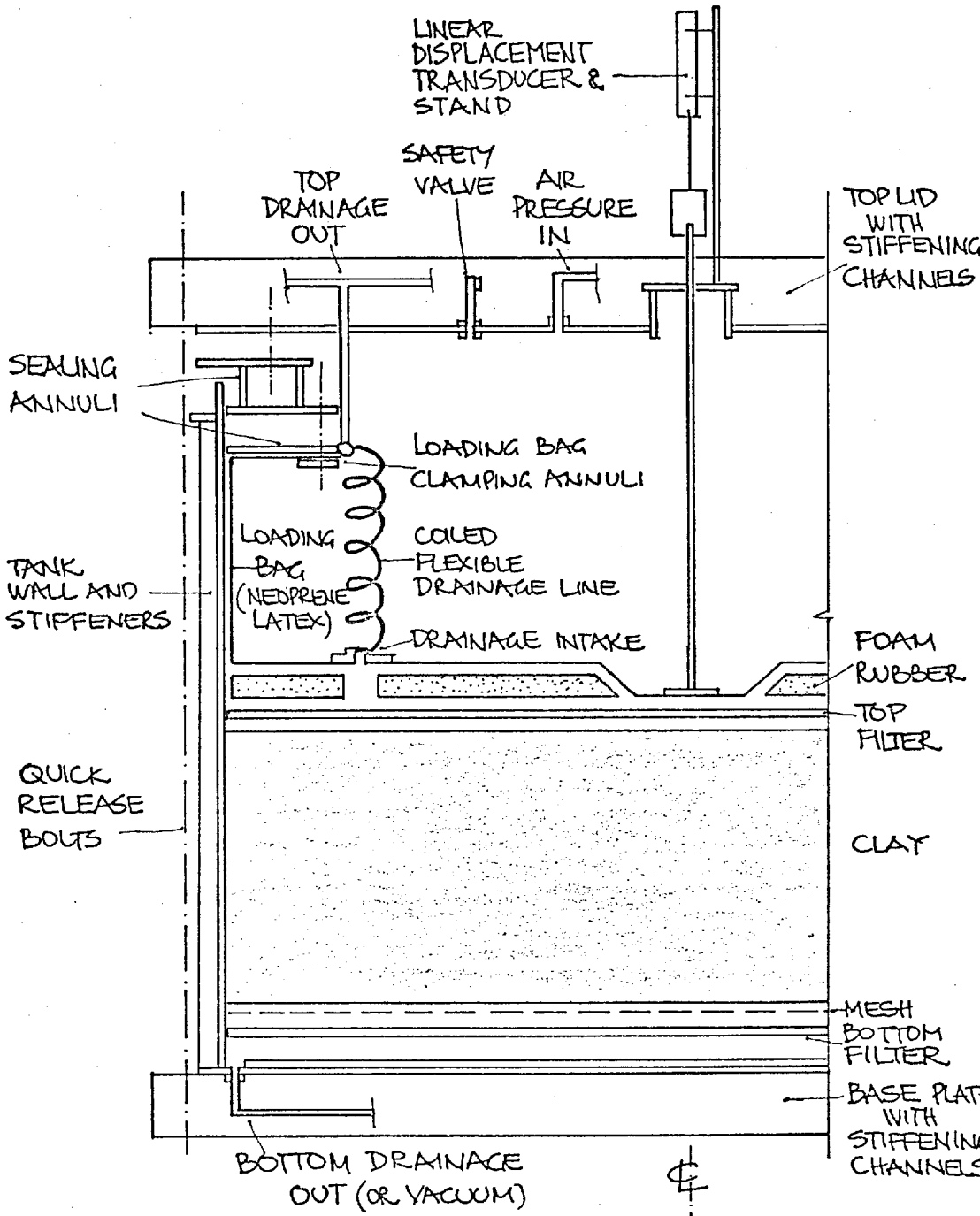
FIGURE 41





SHEAR STRENGTH vs MOISTURE CONTENT : NORFOLK CLAY
(after EL-GHAMRAWY, 1977)

FIGURE 42

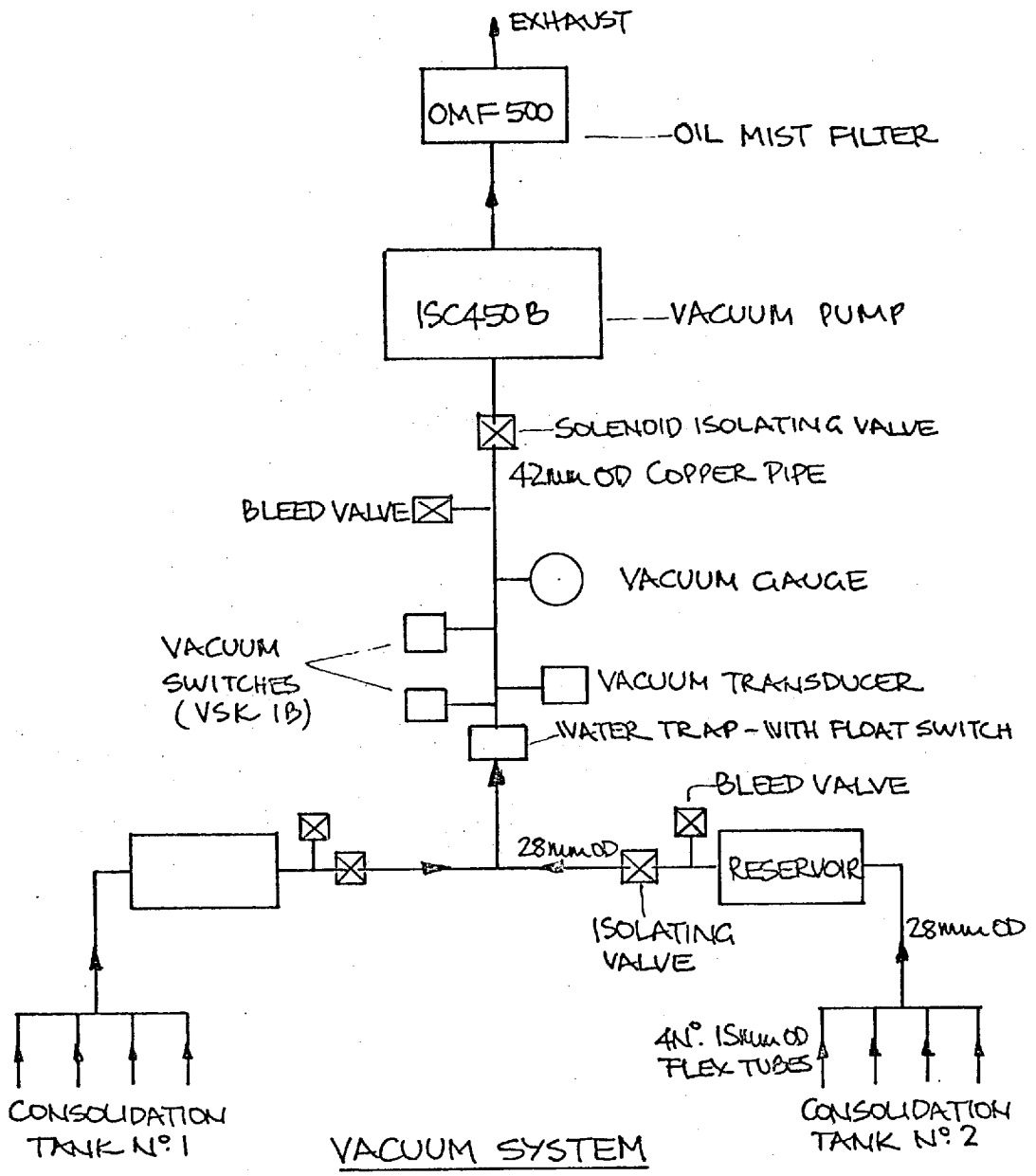


NOTE (i) FABRICATION & SEALING DETAILS GIVEN IN APPENDIX TO THE FIGURES IN SKETCHES SK1 to SK9, SK101 to SK103, GK1

(ii) DRAWING NOT TO SCALE.

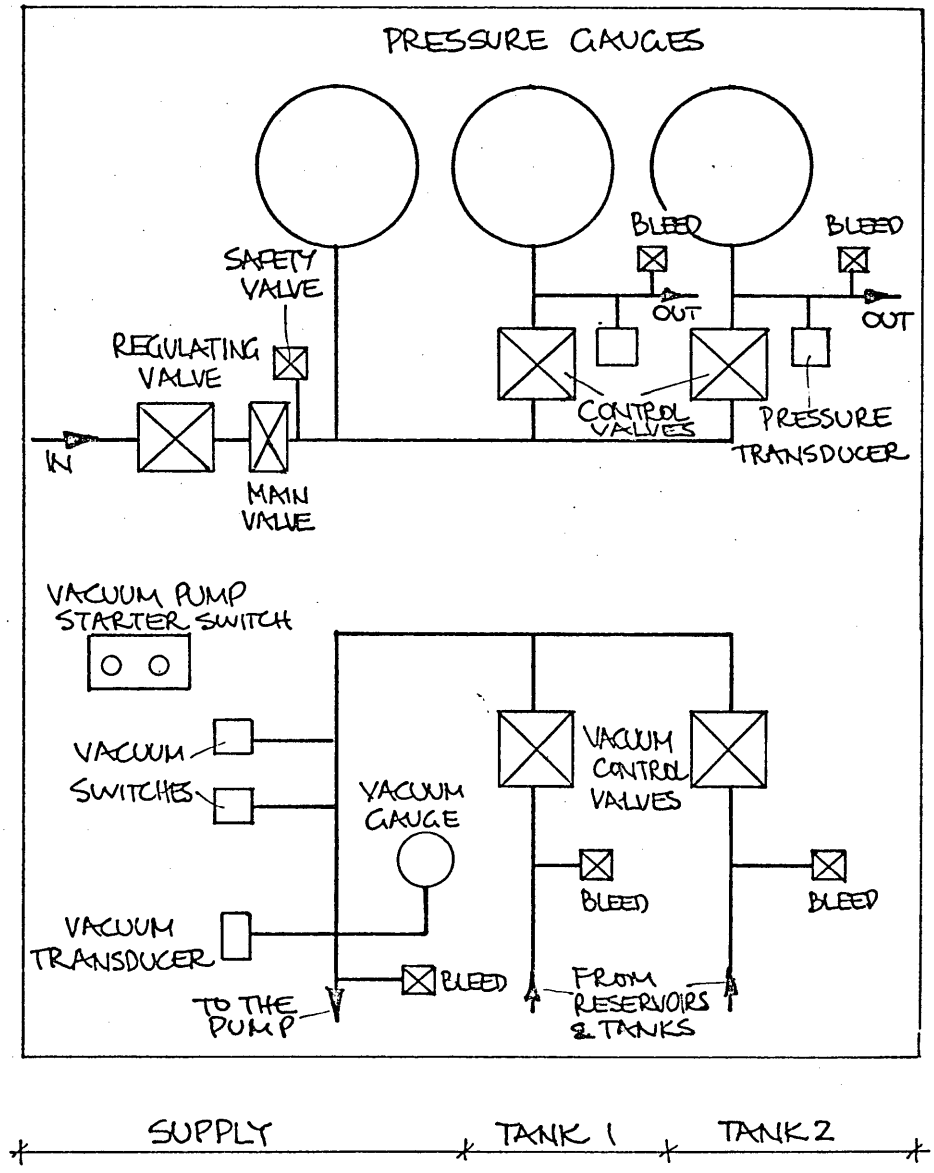
CONSOLIDATION TANK: (EXPANDED) SECTION

FIGURE 43



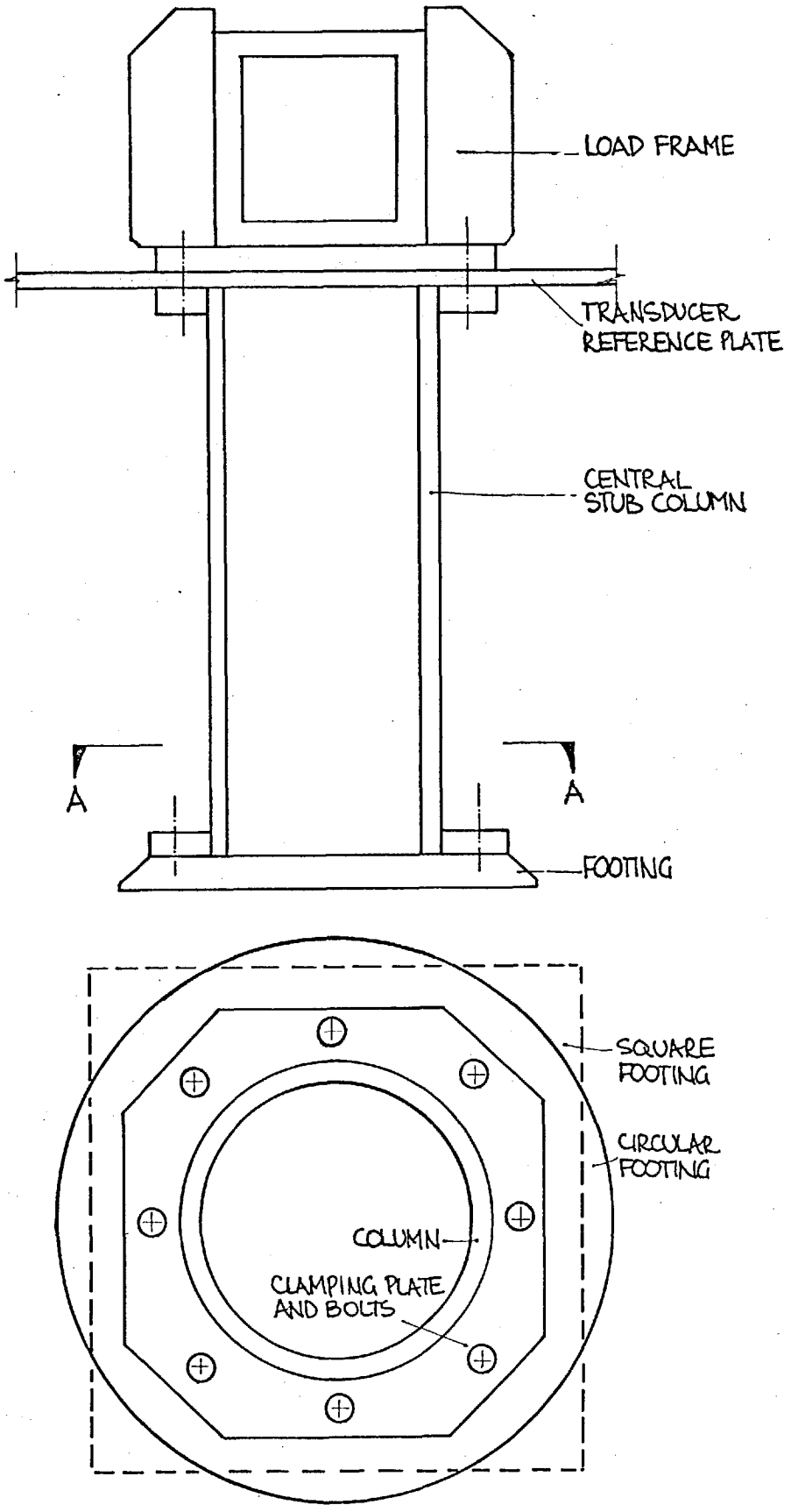
CLAY BED CONSOLIDATION - LOADING SYSTEMS

FIGURE 4.4



PRESSURE AND VACUUM SYSTEMS - CONTROL BOARD

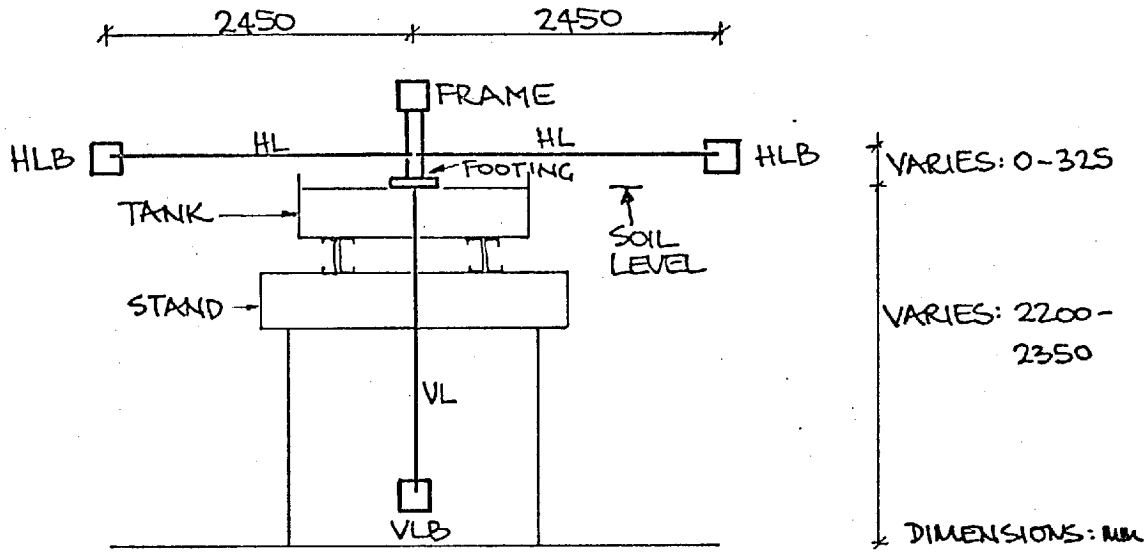
FIGURE 45



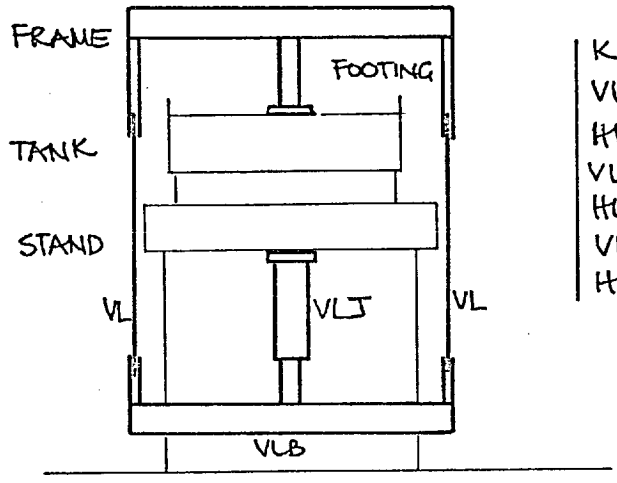
SECTION A-A (N.T.S.)

MODEL FOOTING & CENTRAL COLUMN (LOAD FRAME) SECTIONS

FIGURE 46

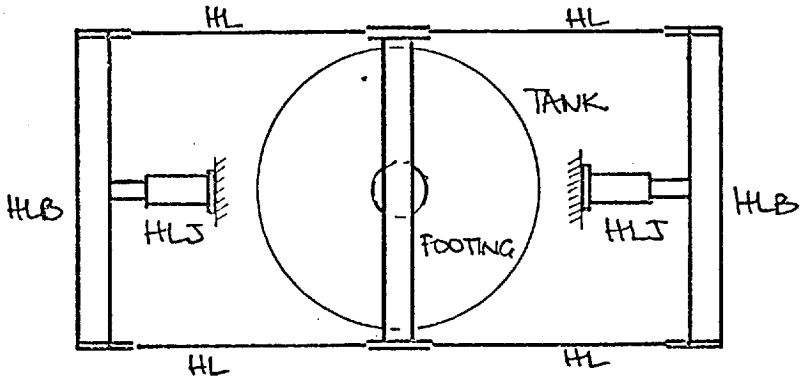


SIDE ELEVATION - GENERAL ARRANGEMENT



KEY:
VL : VERTICAL LOAD LINK
HL: HORIZONTAL LOAD LINK
VLB: VERTICAL LOAD BEAM
HLB: HORIZONTAL LOAD BEAM
VLJ: VERTICAL LOAD JACK
HLJ: HORIZONTAL LOAD JACK

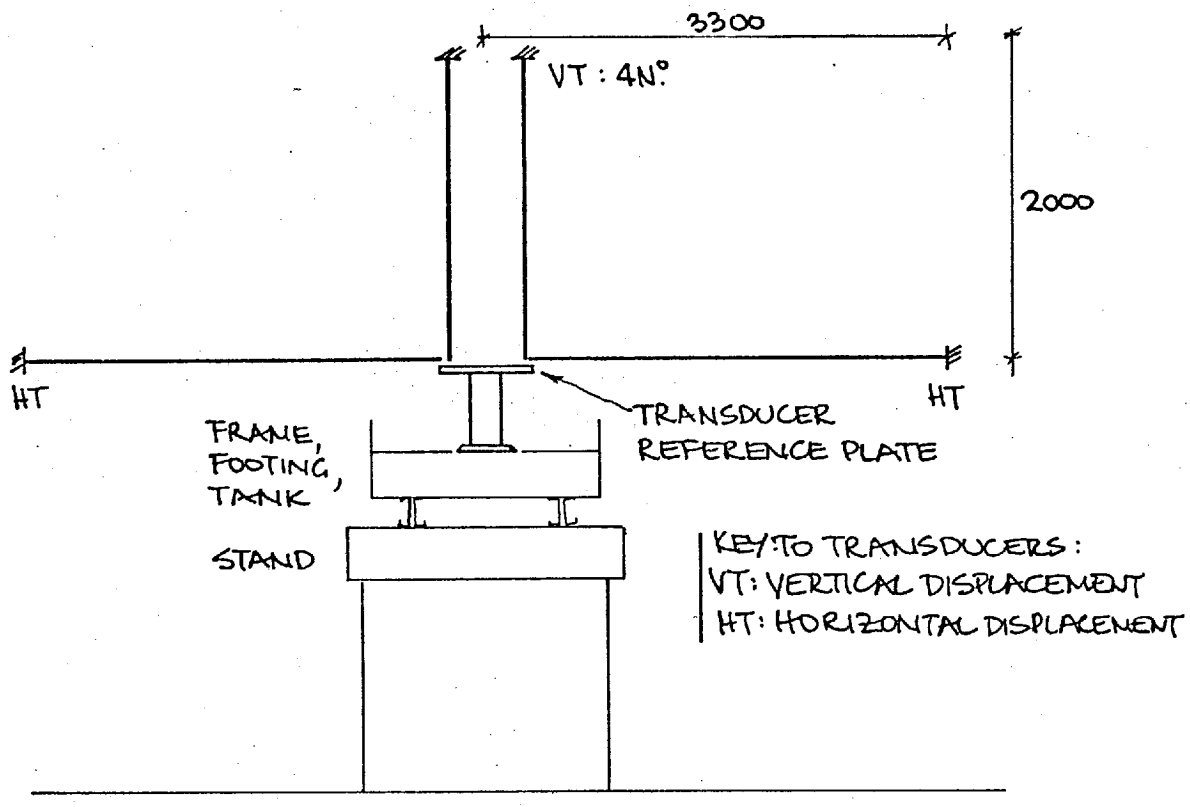
SECTION - VERTICAL LOAD SYSTEM



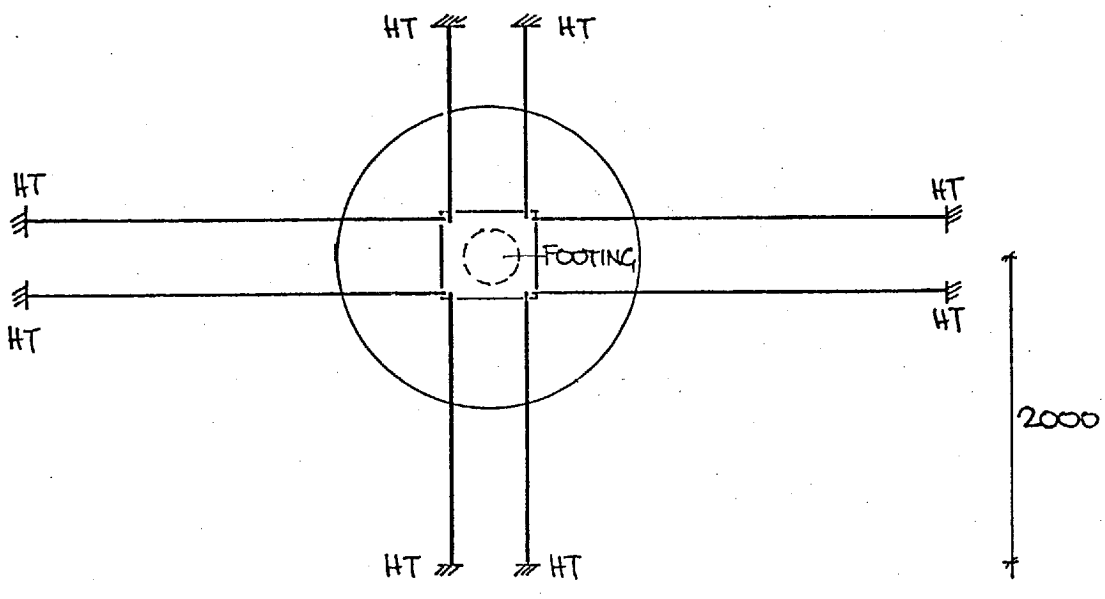
PLAN - HORIZONTAL LOAD SYSTEM

FOOTING TESTS - LOADING SYSTEMS

FIGURE 47



ELEVATION – VERTICAL & LONG^{L} HORIZONTAL TRANSDUCER LOCATIONS

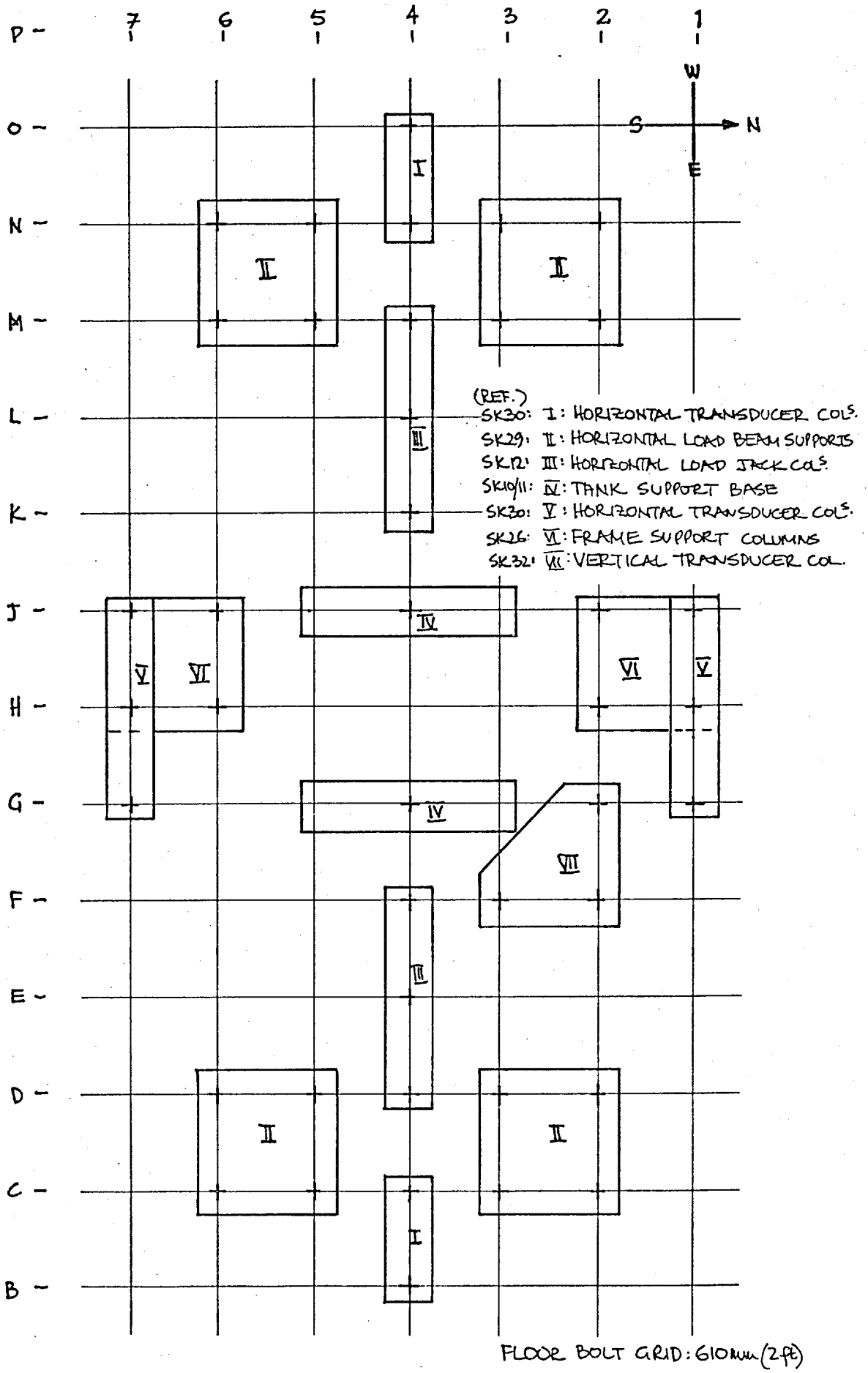


PLAN – HORIZONTAL TRANSDUCER LOCATIONS

DIMENSIONS: MM

FOOTING TESTS – TRANSDUCER LOCATIONS

FIGURE 48



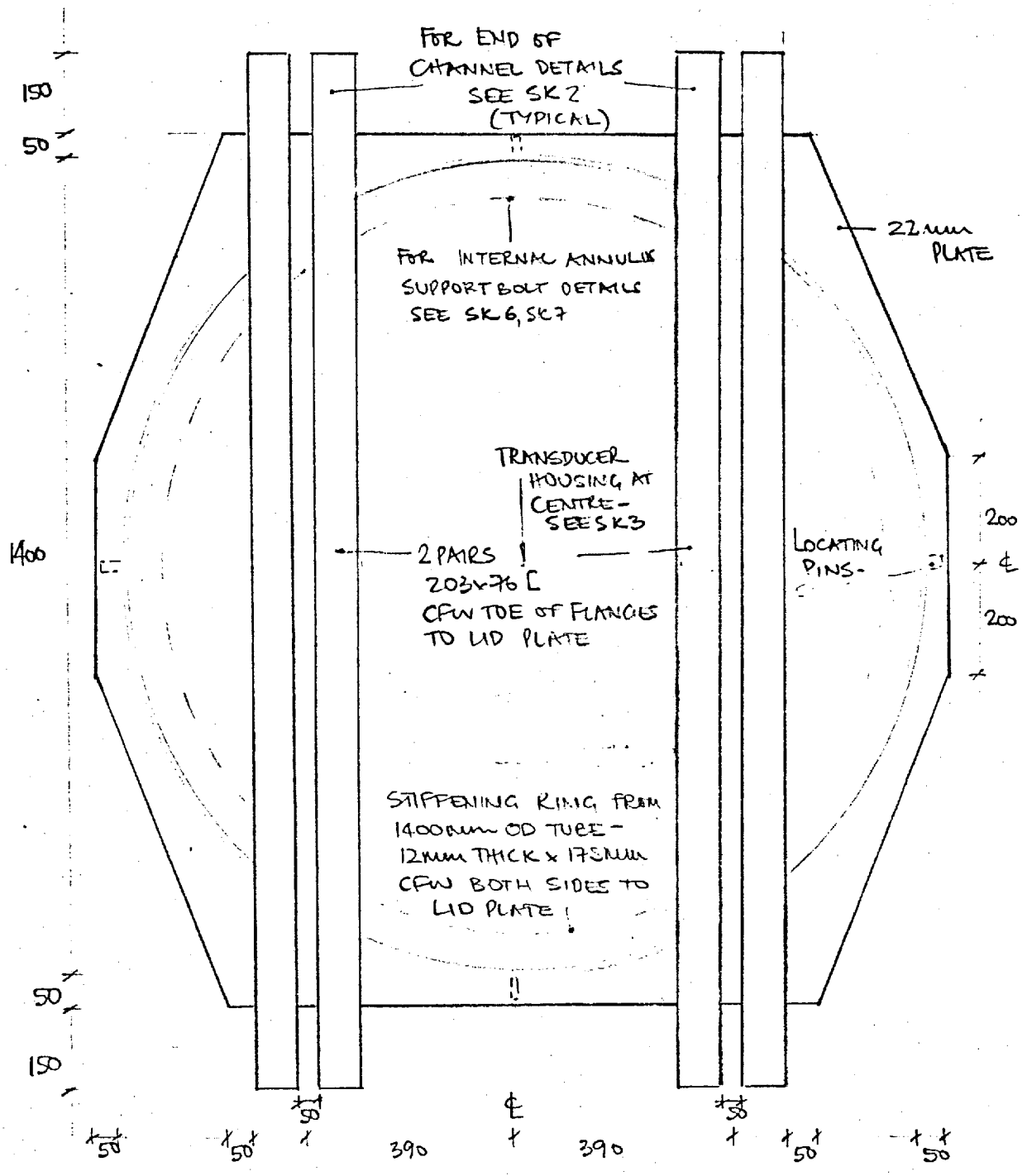
MODEL TESTING FACILITY: FOOTING LOADING SYSTEM
BASE PLATE LAYOUT

FIGURE 49

APPENDIX TO THE FIGURES

WORKING SKETCH DETAILS FOR FABRICATION

SK1 to SK11	:	Consolidation tanks
SK12, SK13, SK26 to SK33	:	Column details
SK14 to SK25, SK35	:	Loading system
SK34	:	Support frames
SK100 to SK104	:	Assembly details
SK1001	:	Probe carriage details
GK1	:	Consolidation tank sealing system

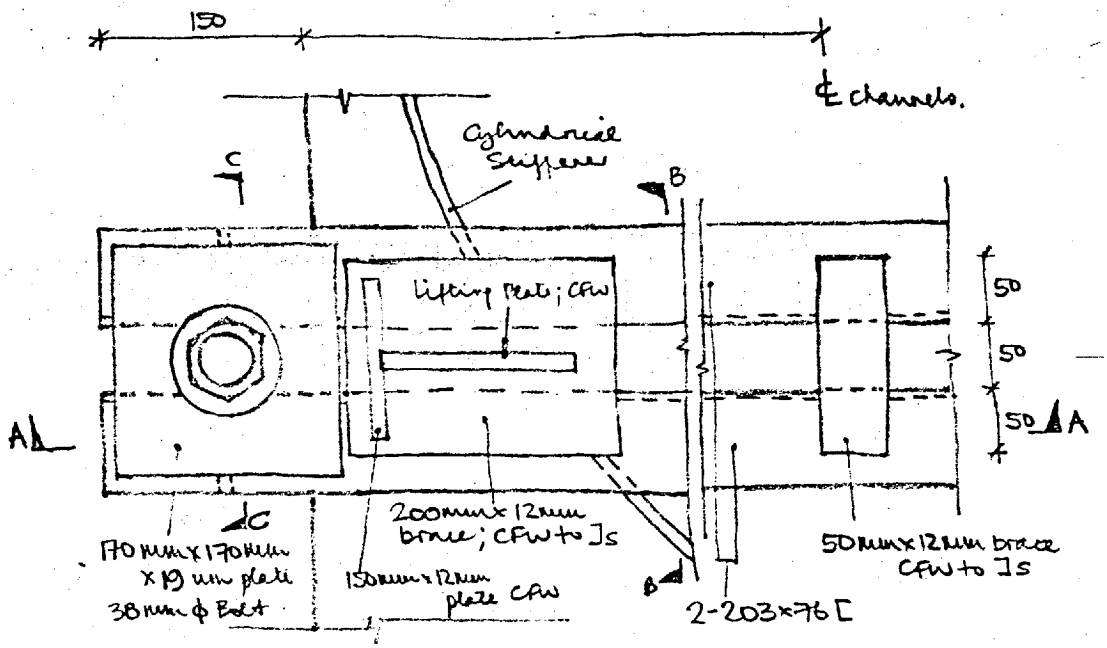


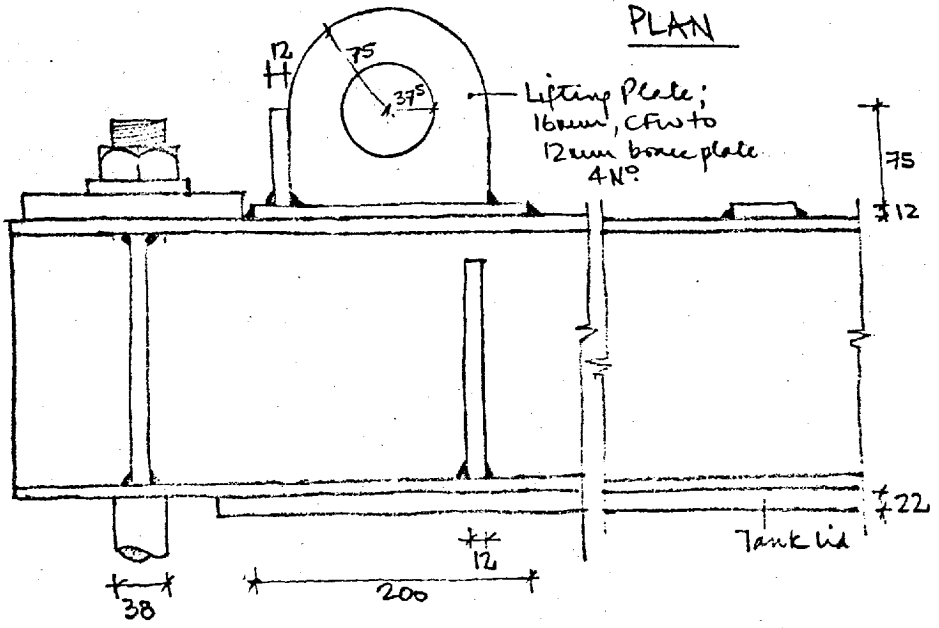
NOTES: FOR ALL SKETCHES : ALL DIMENSIONS : mm UNO

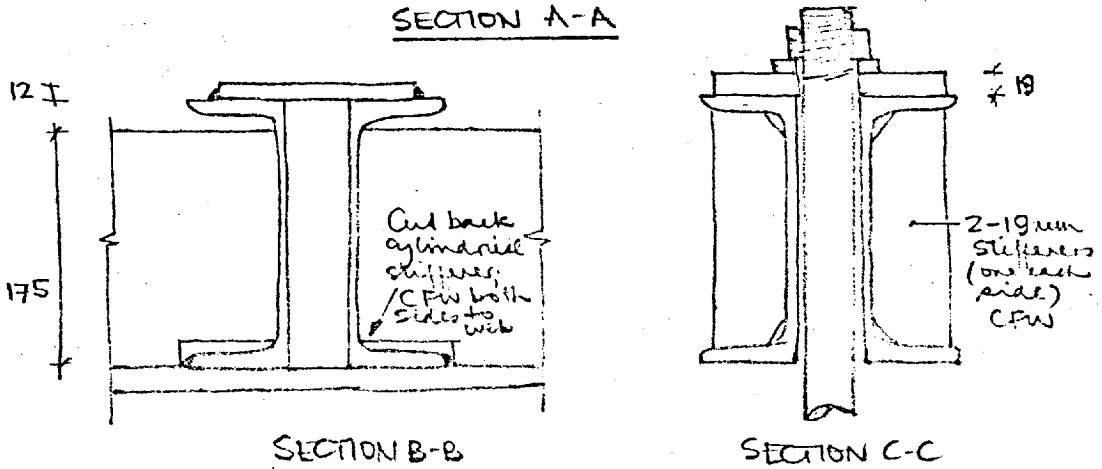
ALL FILLET WELDING. 6mm COMPLETE (CFW) UNO

RH
July 96

TANK LID - PLAN / GENERAL ARRANGEMENT : SK1

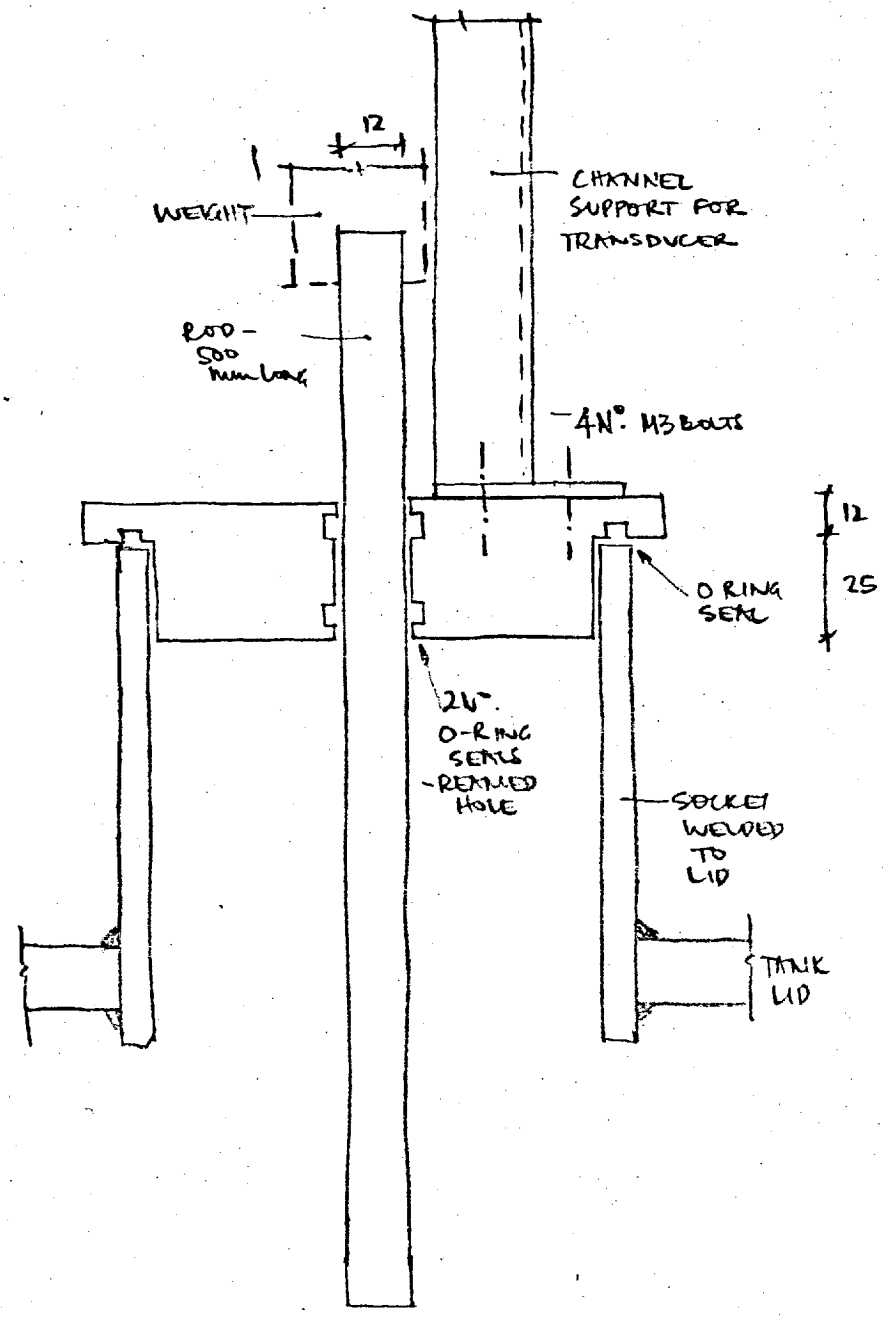






RH June 76

SK2: TANK LID – END OF CHANNEL DETAILS:

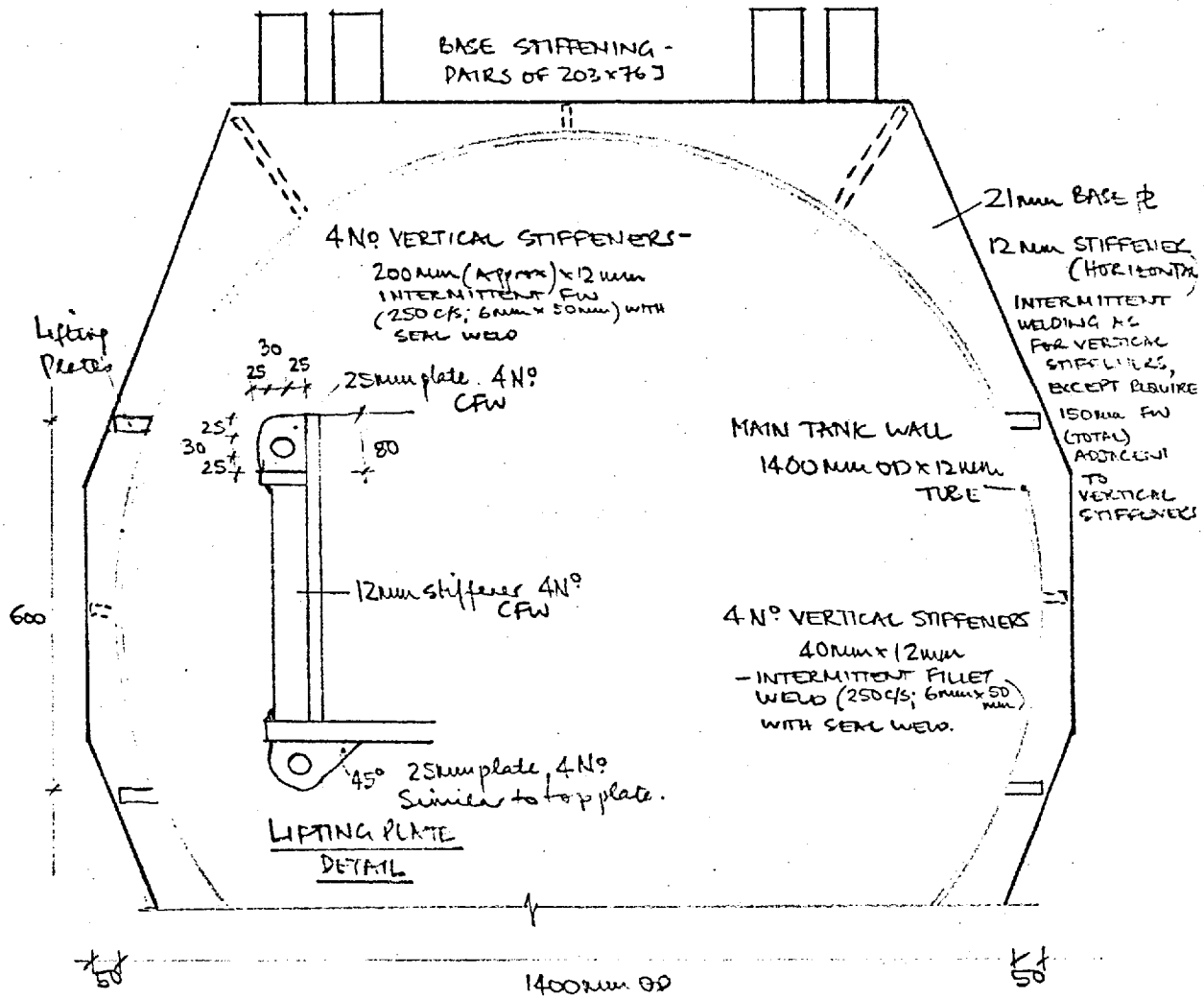


TRANSDUCER SUPPORT -
SAMPLE SURFACE DISPLACEMENT
MEASUREMENT.

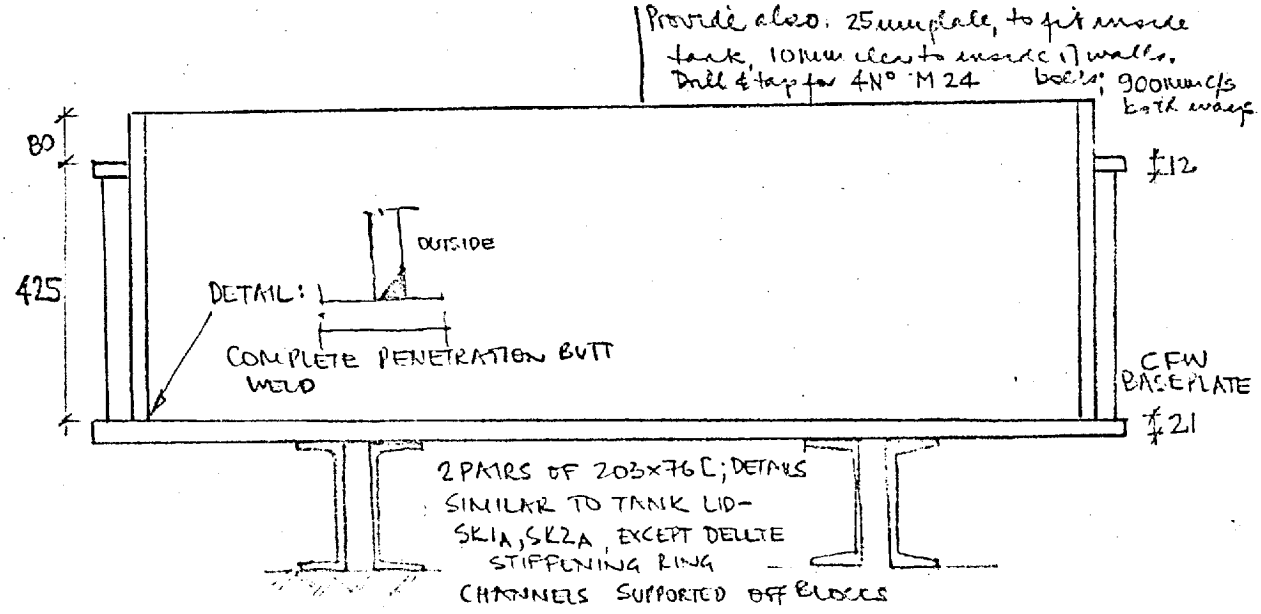
RA
10/17

TANK LID - DETAIL

SK3



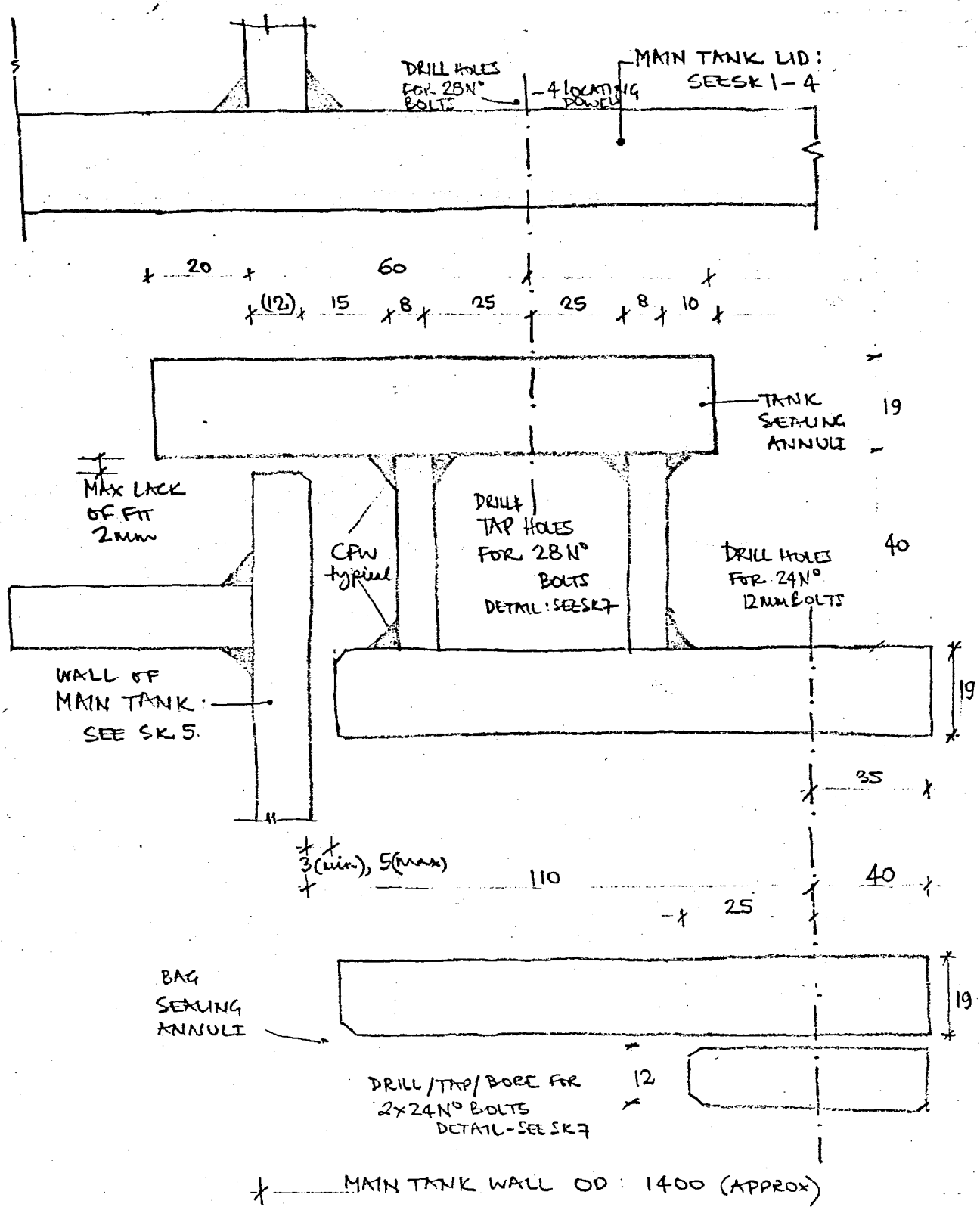
POSITION OF STIFFENING CHANNELS: AS FOR LID: SELSK 1A



RH July 76

MAIN TANK - PART PLAN / SECTION

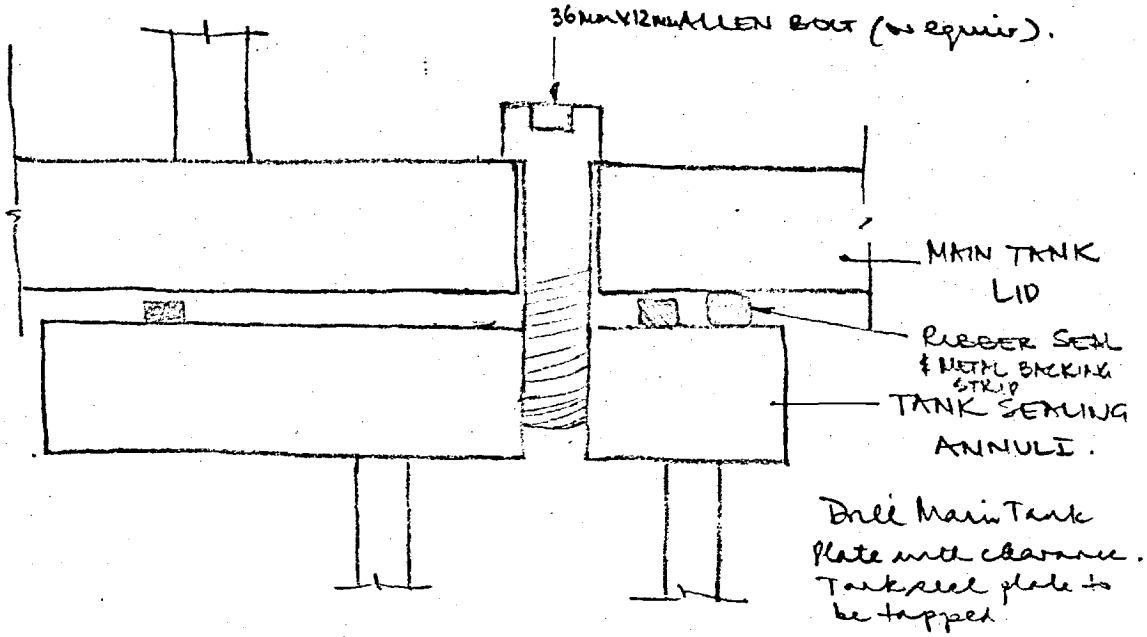
SK5c



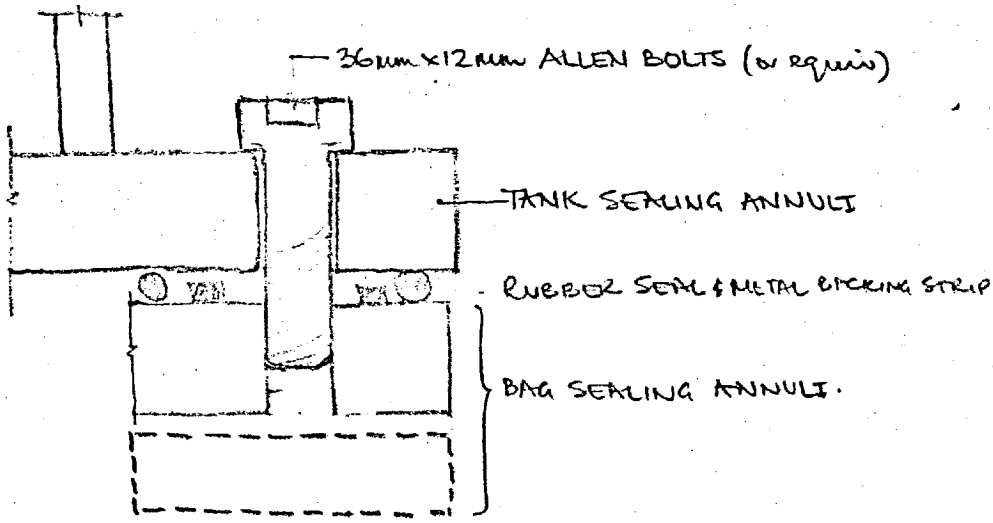
* CORNERS SHOWN CHAMFERED TO BE ROUNDED, FREE OF SHARP EDGES.

RH June 76

SK6: TANK SEALING – BAG SEALING SYSTEMS.

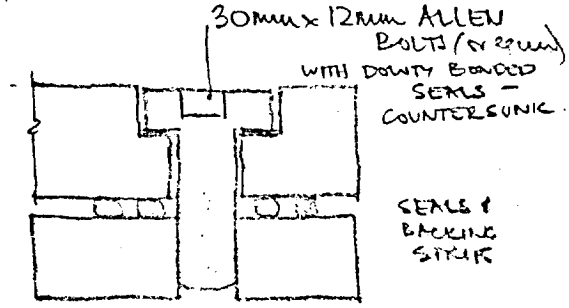


1. TANK LID / TANK SEALING ANNULUS CONNECTION



2. TANK SEALING / BAG SEALING ANNULI CONNECTION

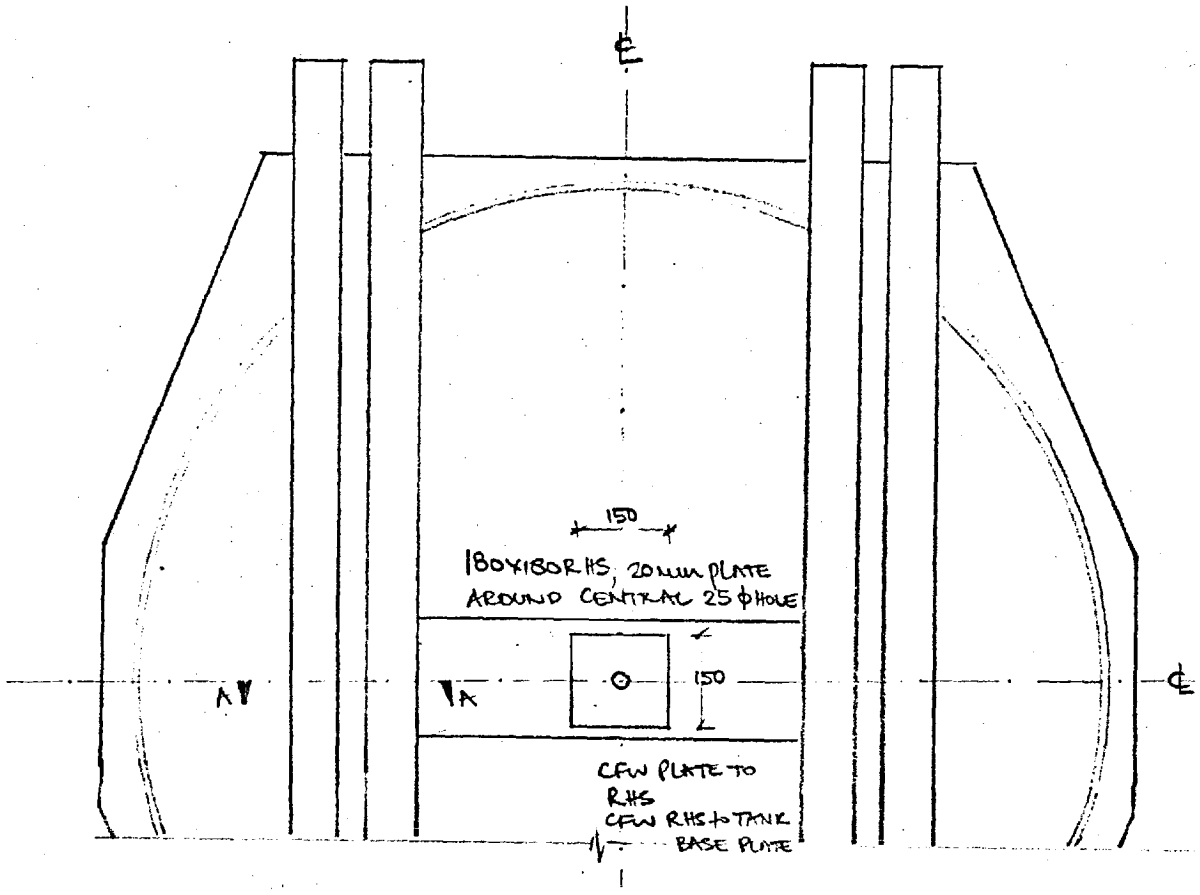
NB: CONNECTIONS IN 2 & 3 STAGGERED.



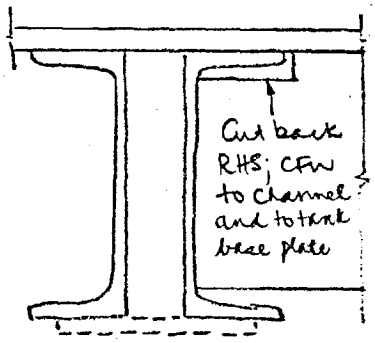
3. BAG SEALING ANNULI CONNECTION

RII
June 76

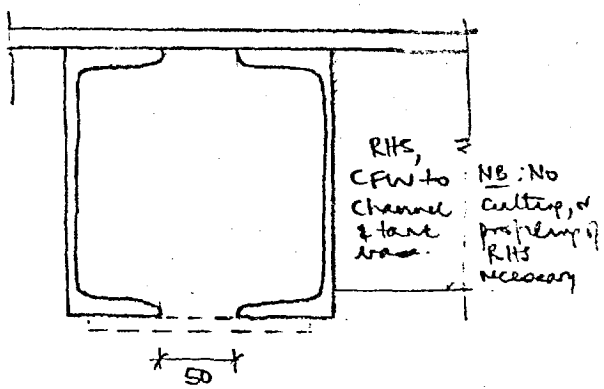
TANK LID / SEALING ANNULI CONNECTIONS: SK-7A



PART UNDERSIDE VIEW OF TANK –
SHOWING ADDITIONAL WORK.



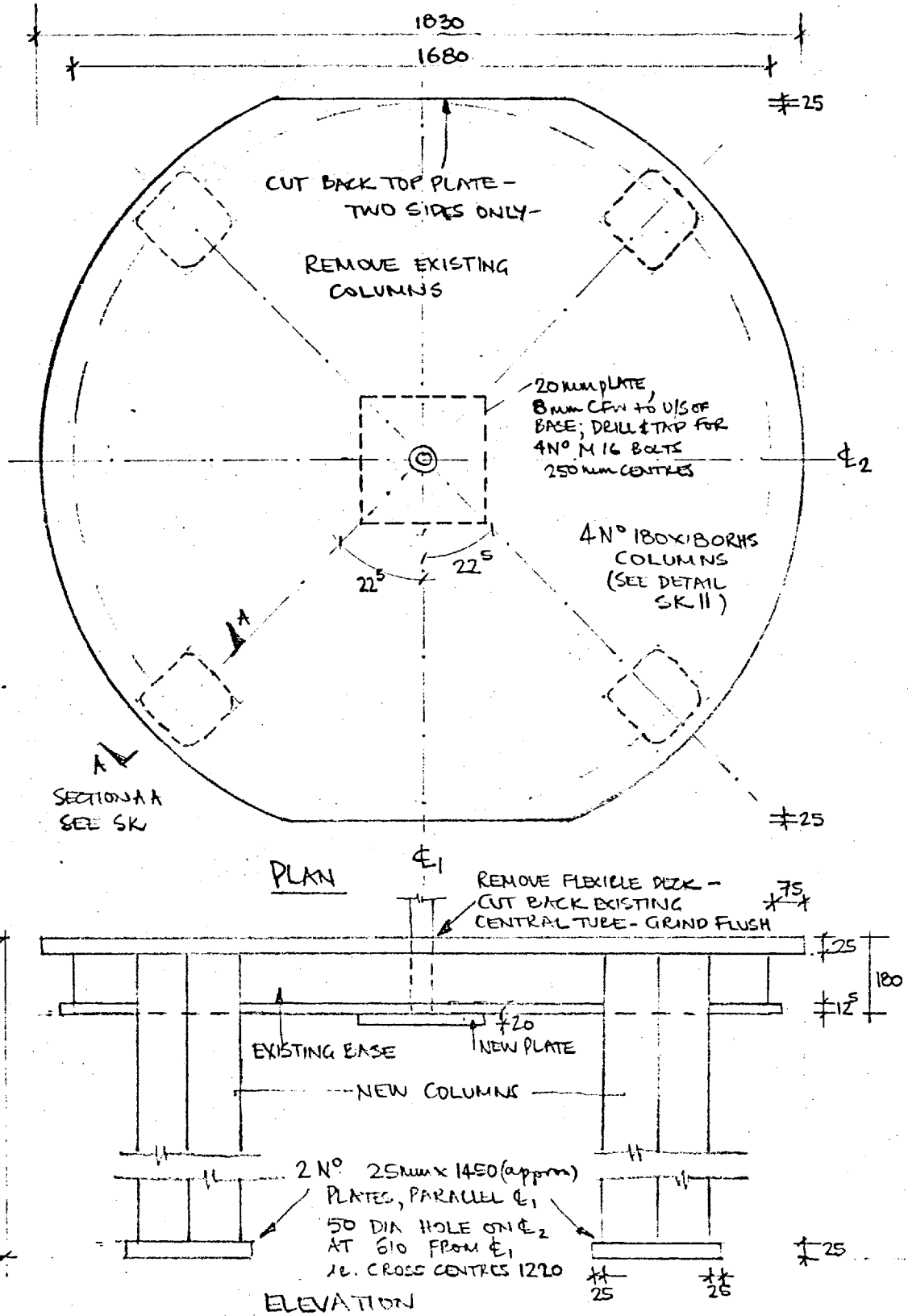
SECTION A-A
CONSISTENT WITH
PREVIOUS DETAILS –
(SK2A; SK5A)



ALTERNATIVE SECTION A-A
TO POSSIBLY FACILITATE
FABRICATION – OTHER DETAILS
ON SK2A, SK5A WOULD NEED
TO BE AMENDED TO CONFORM.

RH
July'96

ADDITIONAL WORK TO UNDERSIDE OF TANK SK9



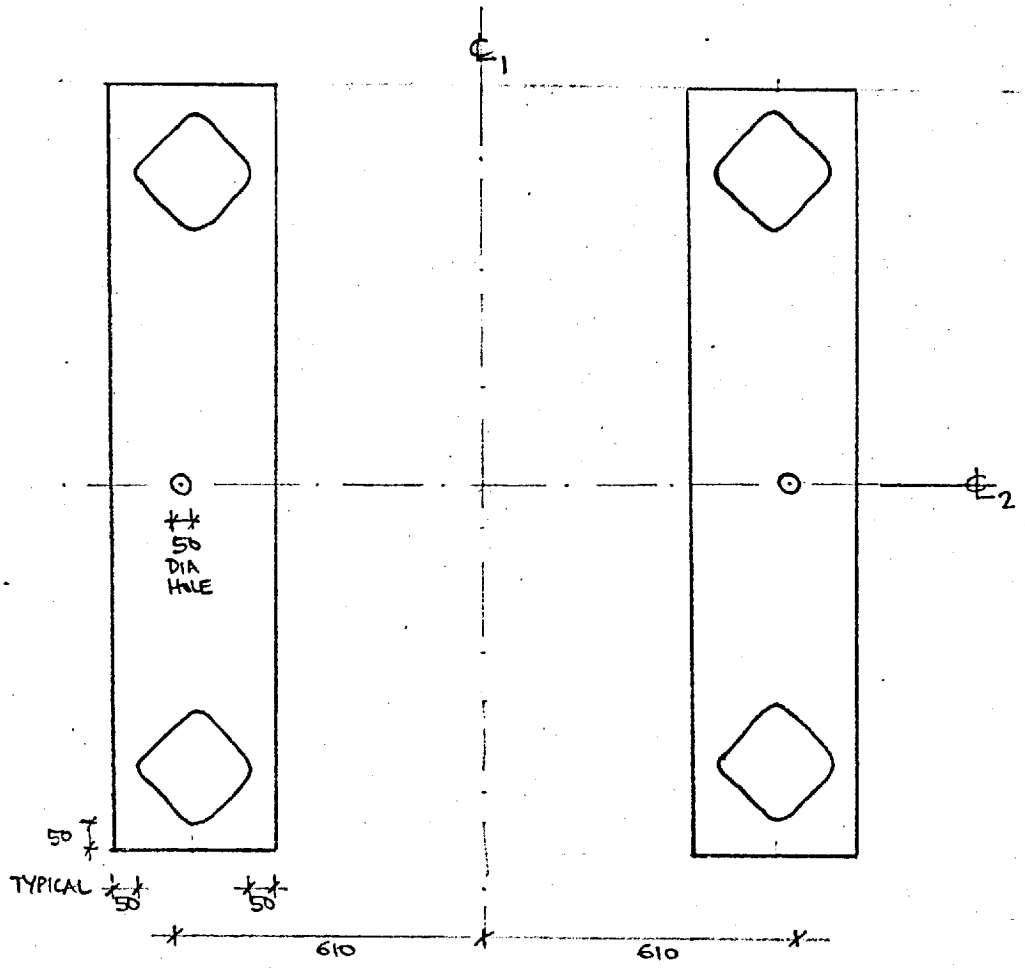
MODIFIED EXISTING "BP TANK" BASE.

CONFIRM DIMENSIONS ON SITE.

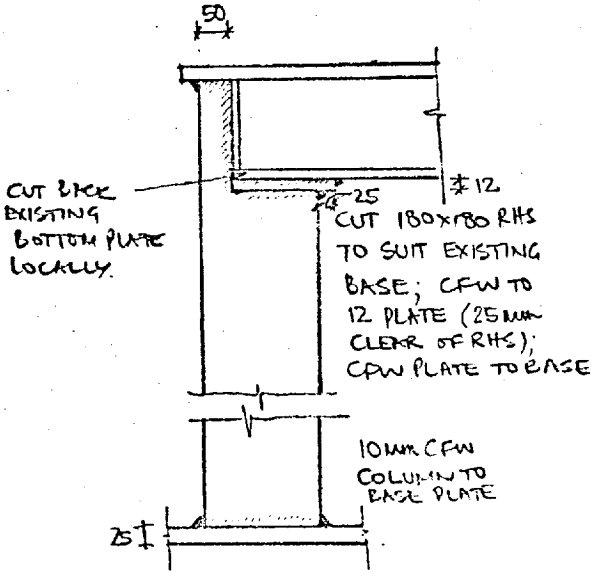
(DIMENSIONS: mm)

RH Aug 76

SK10: MODEL TANK SUPPORT BASE



PLAN OF BASE PLATES

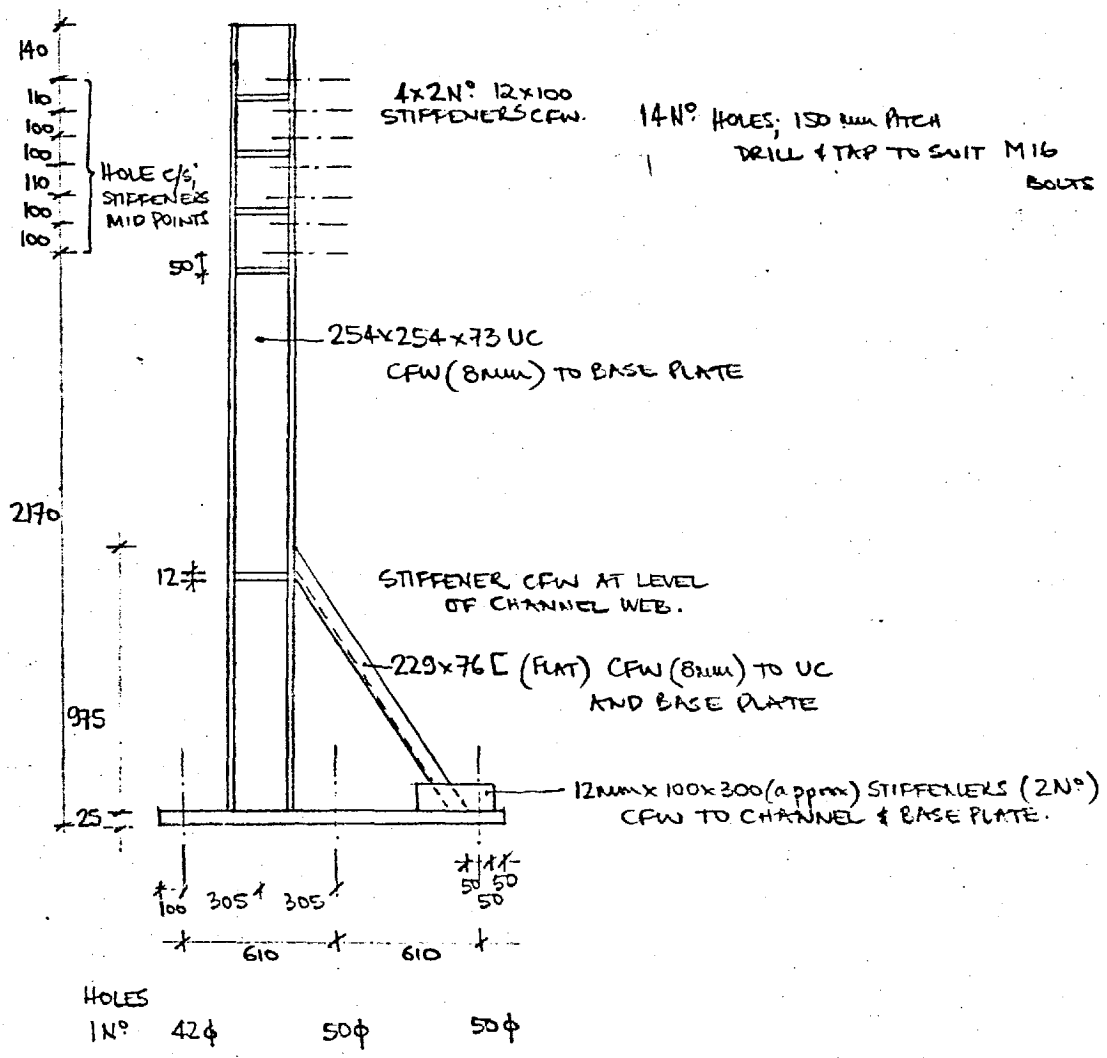


SECTION A-A

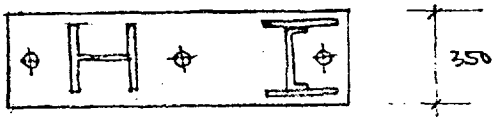
RH aug'%

MODEL TANK SUPPORT BASE DETAILS

SK11 A



ELEVATION

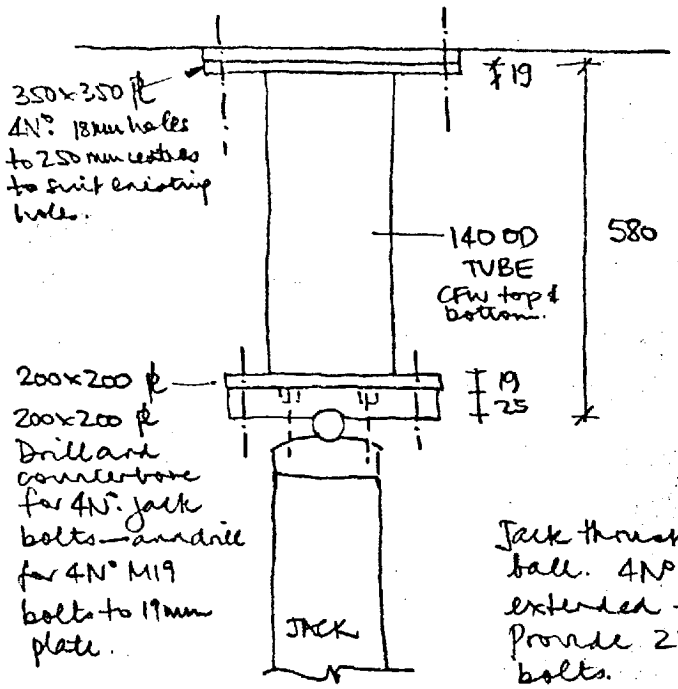


BASE PLATE PLAN

RH
Aug 11

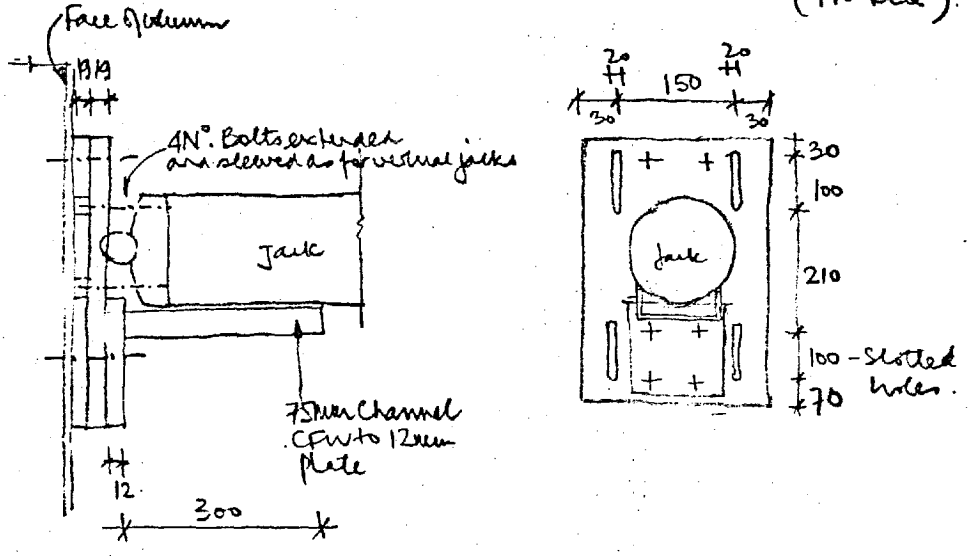
SK 12: HORIZONTAL JACK SUPPORT COLUMN

2 No. REQUIRED.



VERTICAL LOAD JACK SUPPORT DETAILS

(1N° REQ'D).



Elevation - Side

+ : location for bolts holding the two 19mm plates together.

Elevation - Front

RH May 77

HORIZONTAL LOAD JACK SUPPORT DETAILS

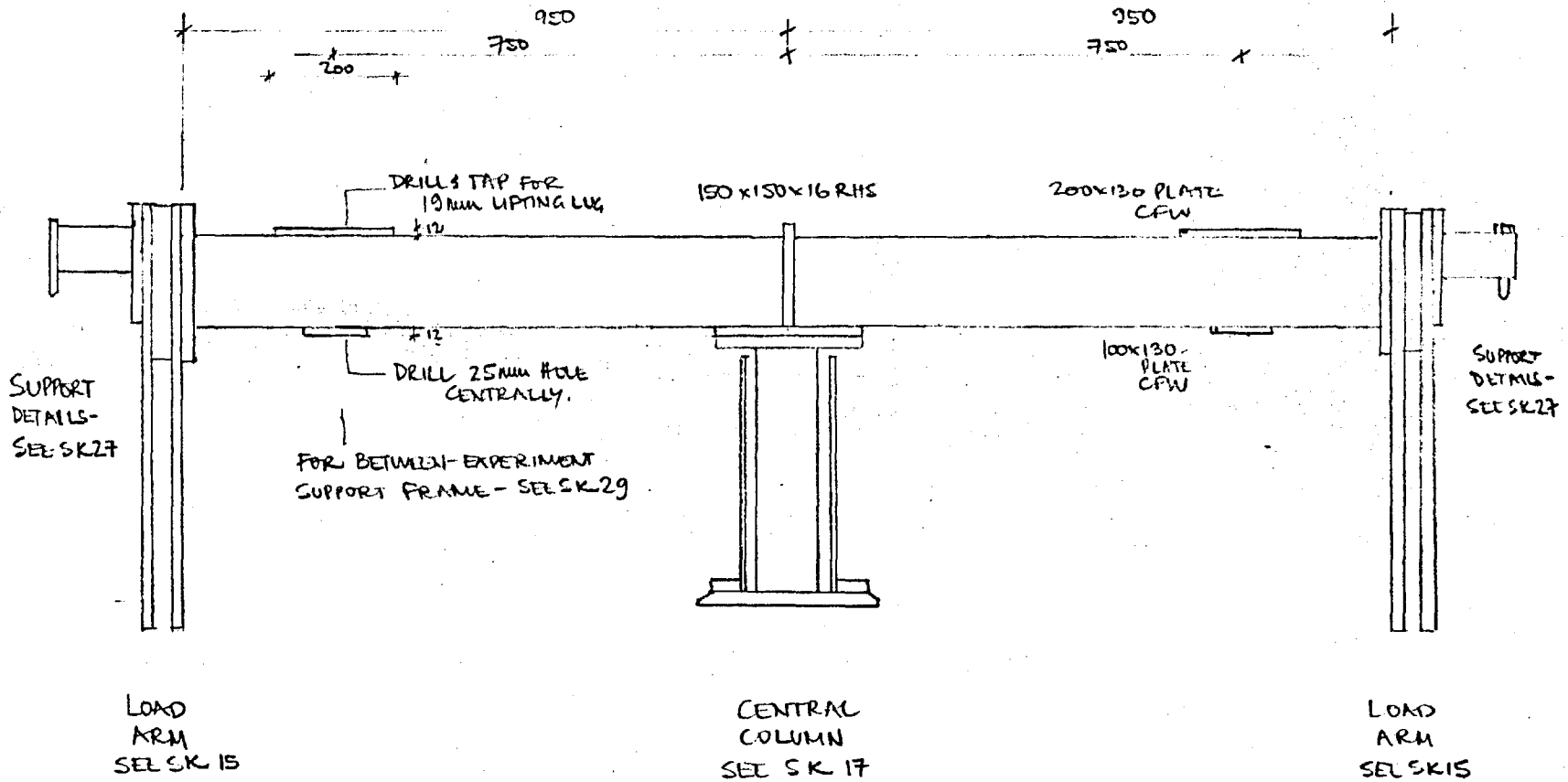
(2N° REQ'D)

LOAD JACK SUPPORT DETAILS

SK13

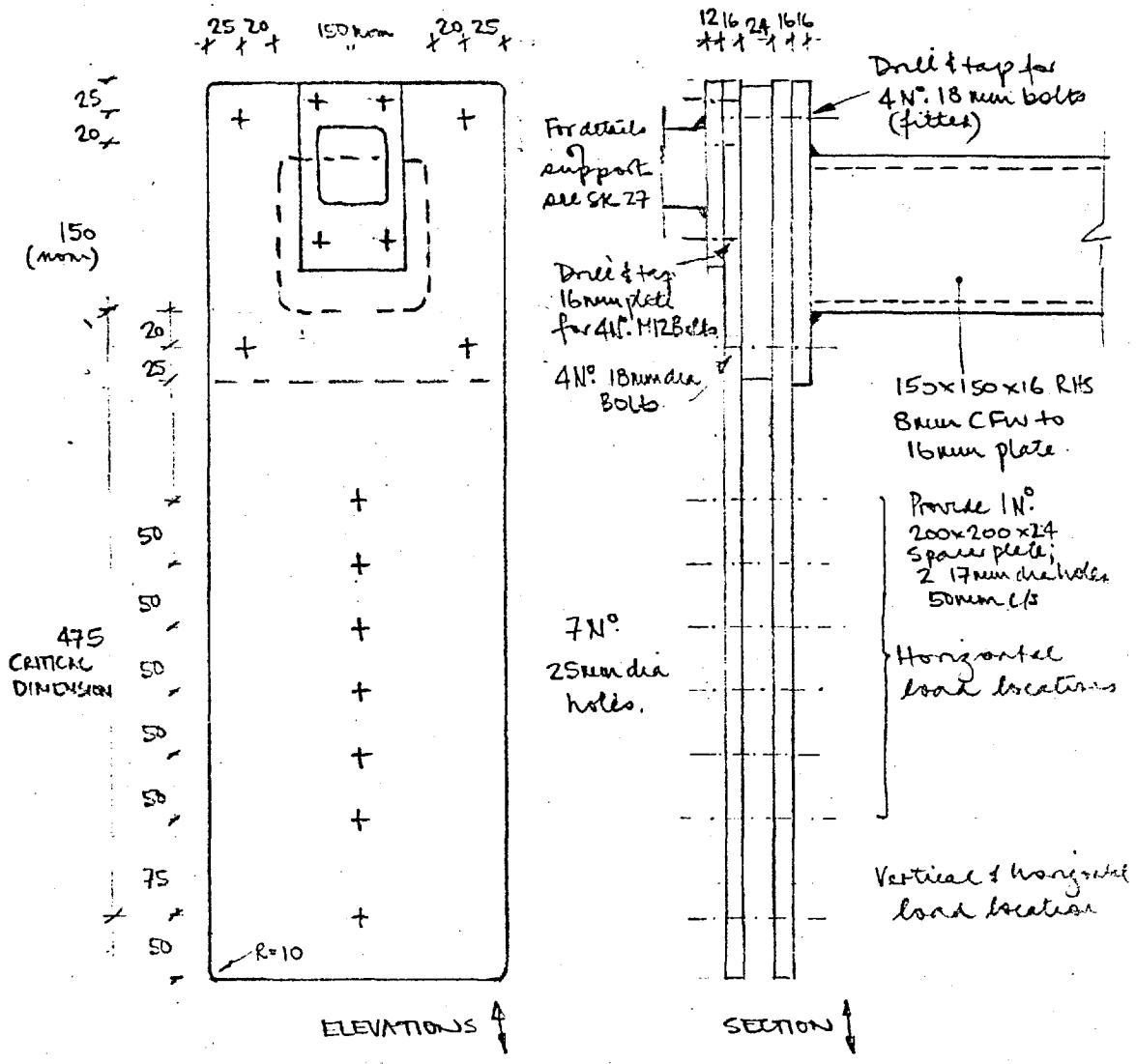
SK14:

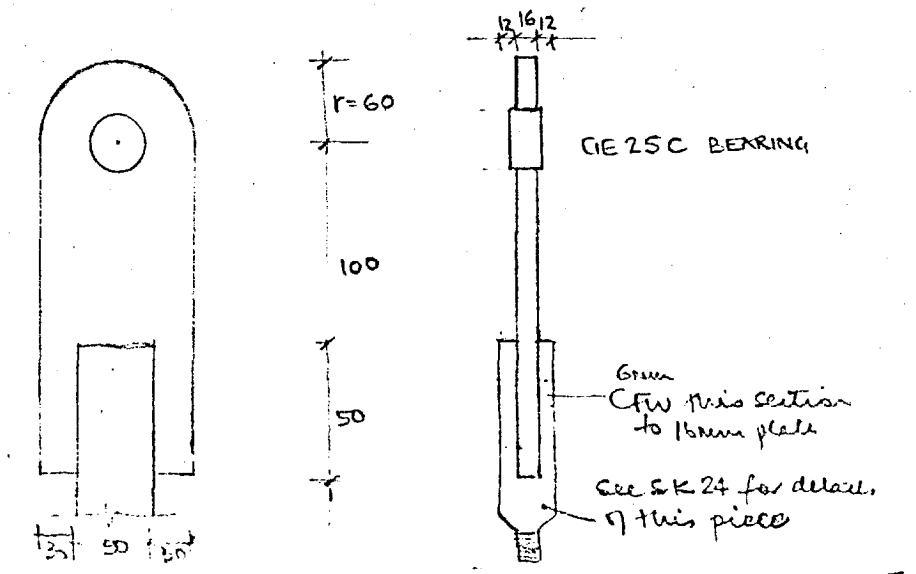
LOADING FRAME - MAIN ELEVATION
- LIFTING/SUPPORT DETAILS



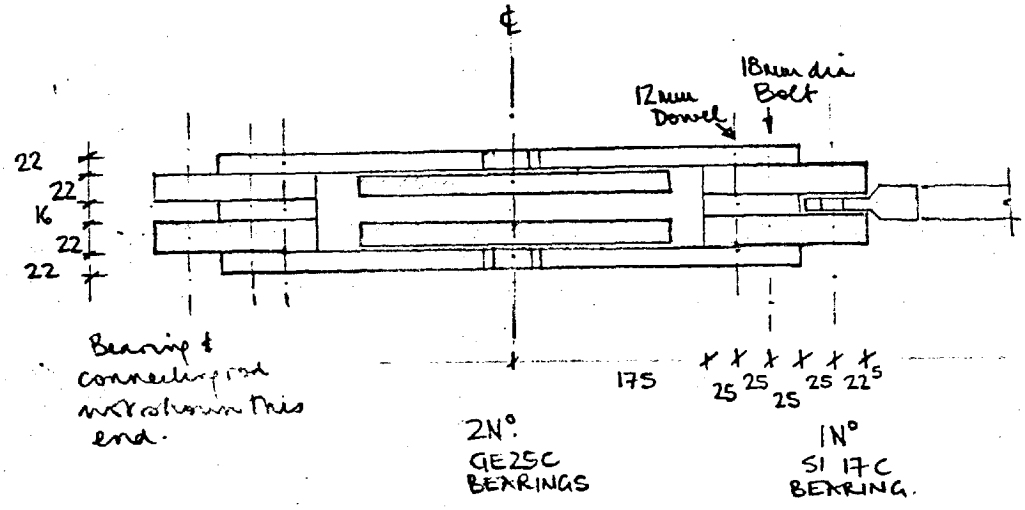
FOR DIMENSIONS NOT GIVEN HERE - SEE APPROPRIATE SKETCHES

RH Aug 7

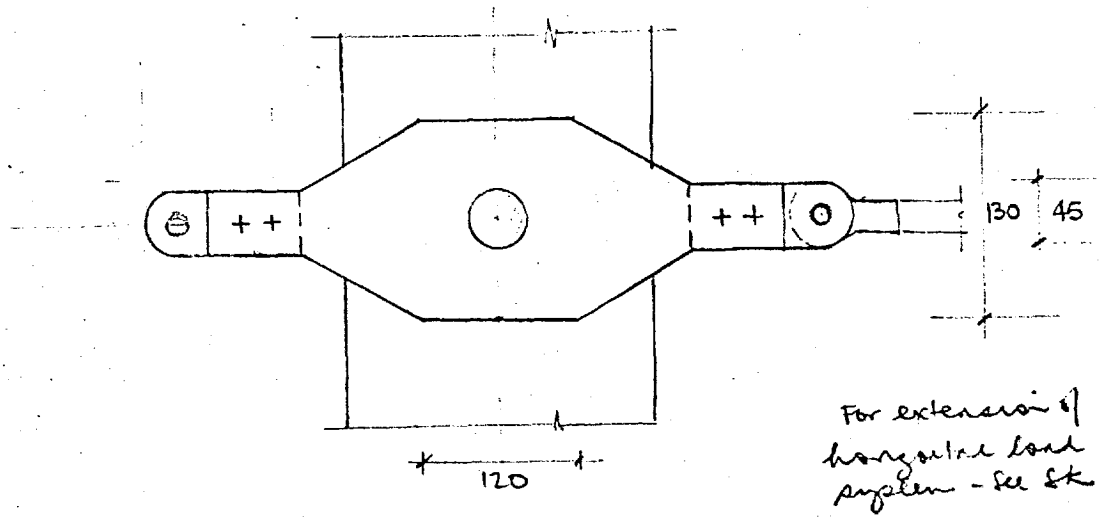




LOAD FRAME — LOADING ARMS/VERTICAL LOAD SYST. SK15 B



SECTION



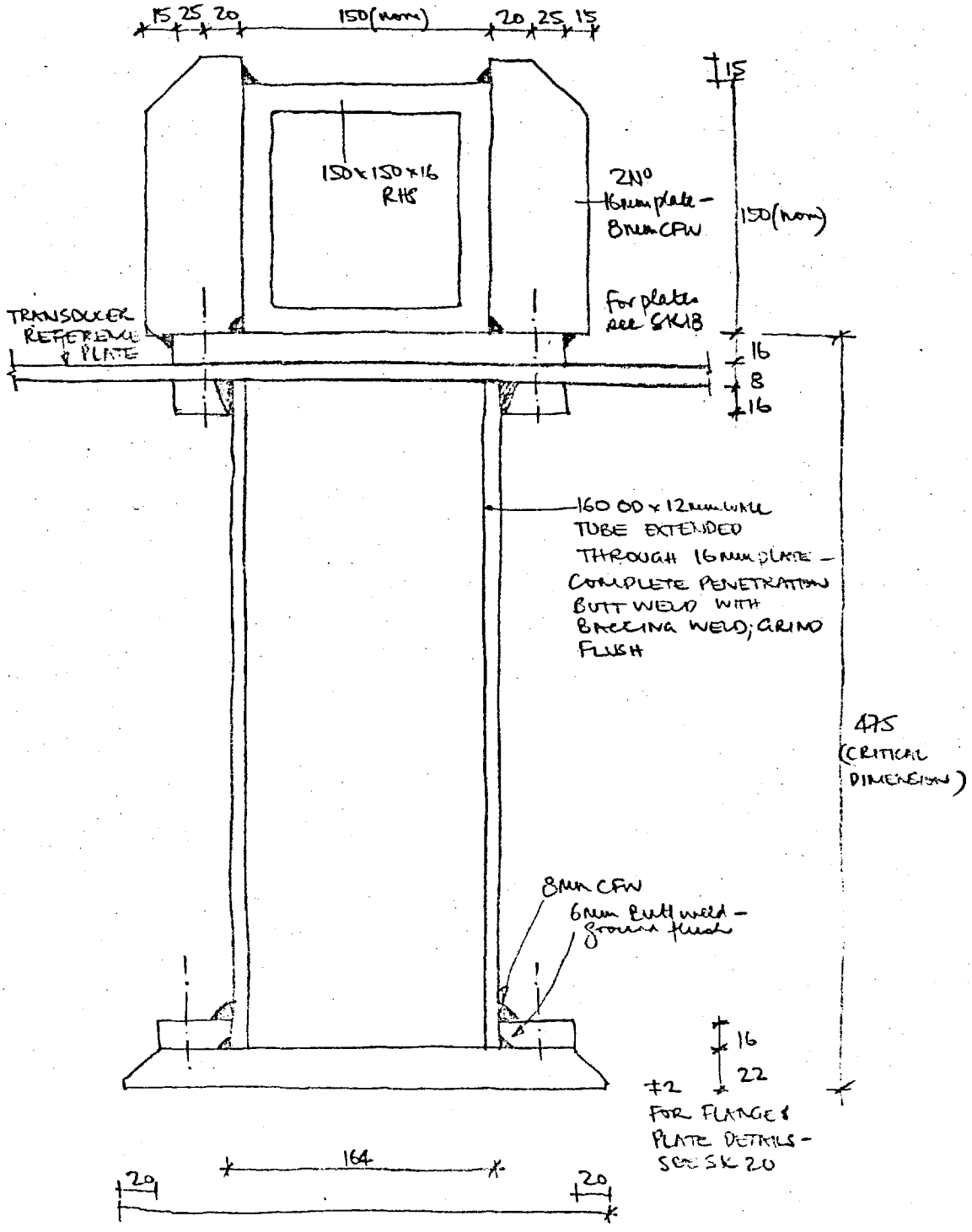
ELEVATION

For extension of horizontal load system - see SK

RH angle

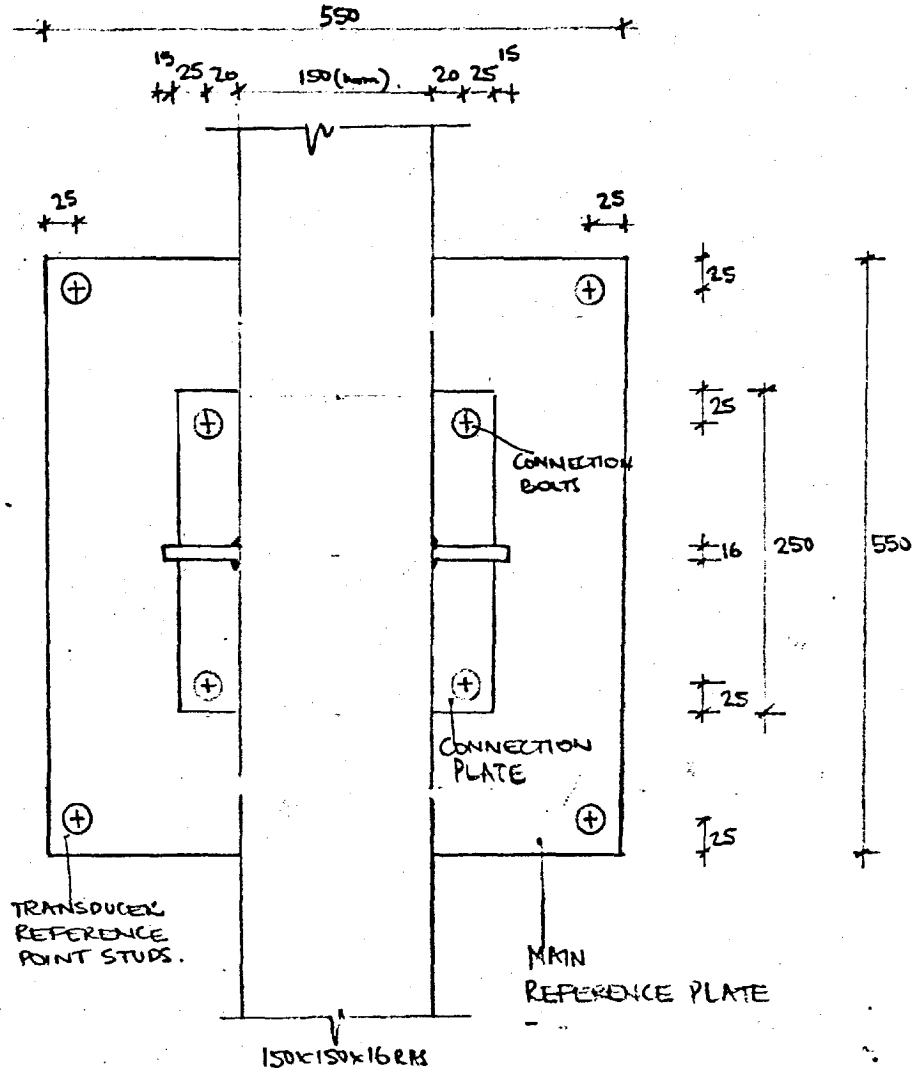
2N° REQUIRED.

SK16. LOAD FRAME – HORIZONTAL LOAD SYSTEM



RH
Sept 96

LOAD FRAME – CENTRAL COLUMN (SECTION) SK17
B



FOR CONNECTION: (SEE ALSO: SK 17)

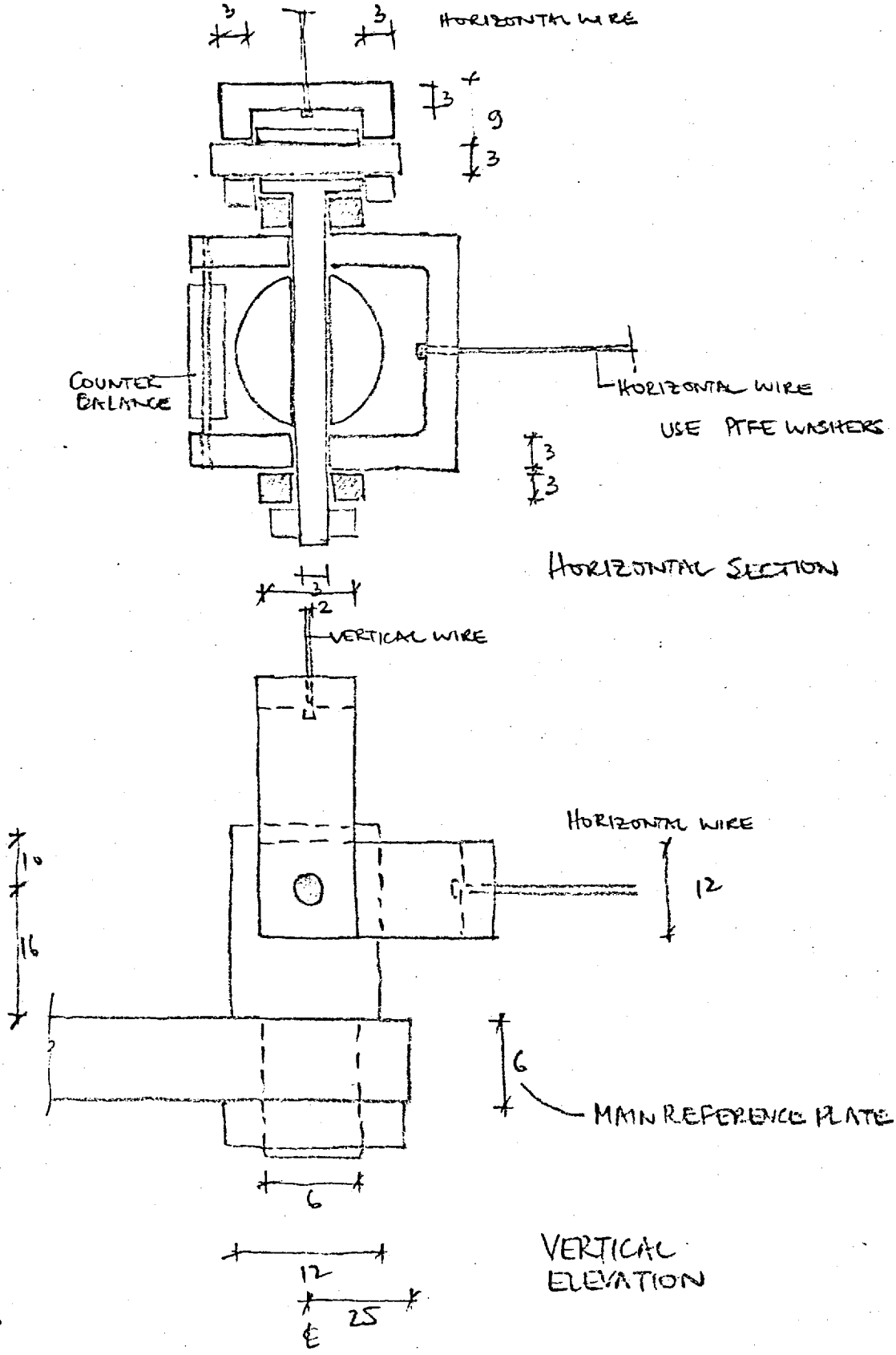
1. DRILL CONNECTION PLATE FOR 4N° 18mm dia BOLTS.
2. DRILL MAIN REFERENCE PLATE FOR 4N° 18 mm DIA BOLTS.

FOR TRANSDUCER REFERENCE POINT LOCATION:

1. DRILL MAIN REFERENCE PLATE FOR 4N° 8. mm DIA. STUDS. (SEE SK 19)

RH Aug 76

SK18: LOAD FRAME - CENTRAL COLUMN - TOP PLATES - PLAN



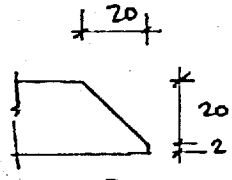
RH angle

DEFLECTION MEASUREMENT- REFERENCE POINTS

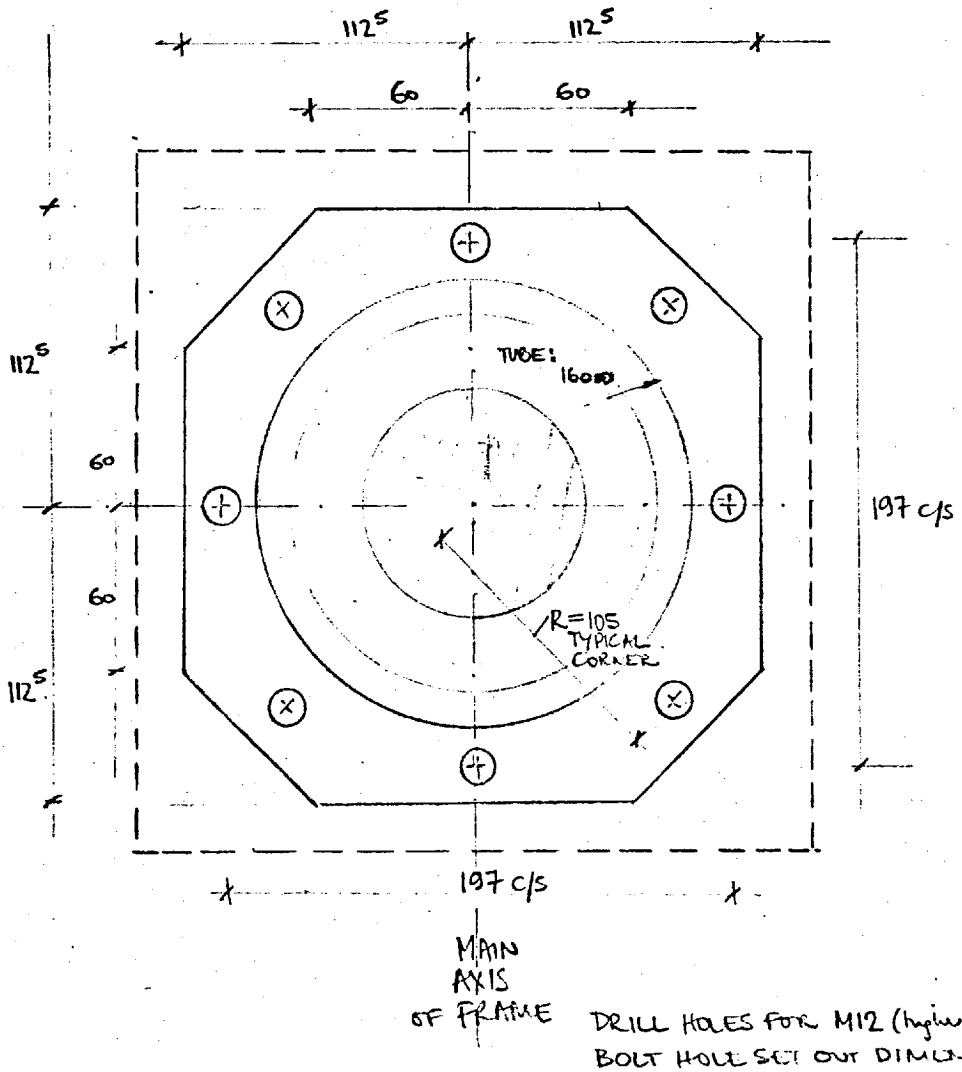
SK19

2 No RCR
PLUS 2 No HANDED.

FOOTING PLATES: 1 Nº 300 mm OD
1 Nº 265 mm SQ (SHOWN DOTTED)



EDGE OF
FOOTING PLATE DETAIL



DRILL HOLES FOR M12 (high strength) BOLTS
BOLT HOLE SET OUT DIMENSIONS
FOR FOOTING PLATES · ALSO AS ABOVE ·
DRILL & TAP FOOTING PLATES TO
WITHIN 5mm OF UNDERSIDE.

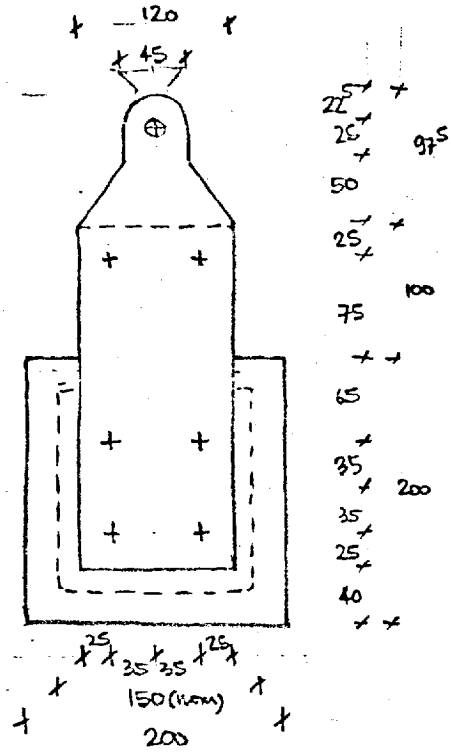
LOAD FRAME.

CENTRAL COLUMN BASE FLANGE -

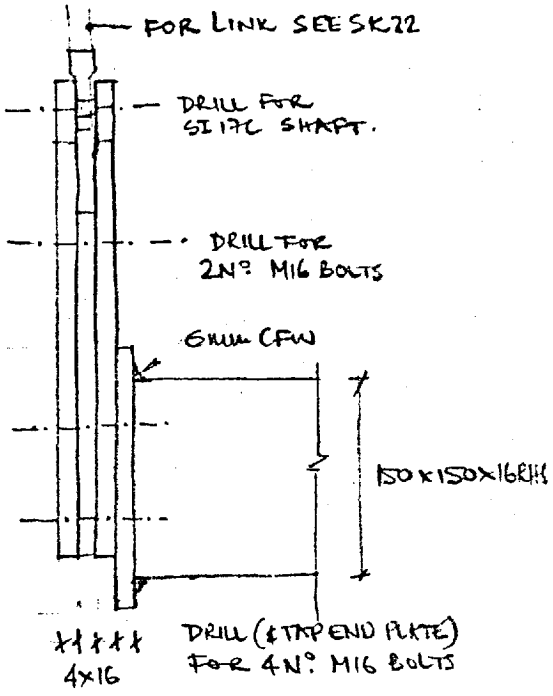
FOOTING PLATE DETAILS

RH
Aug 76

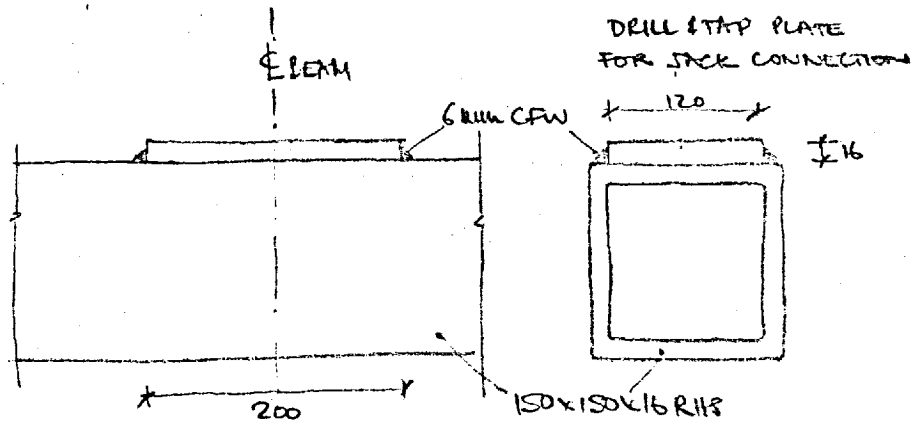
SK20



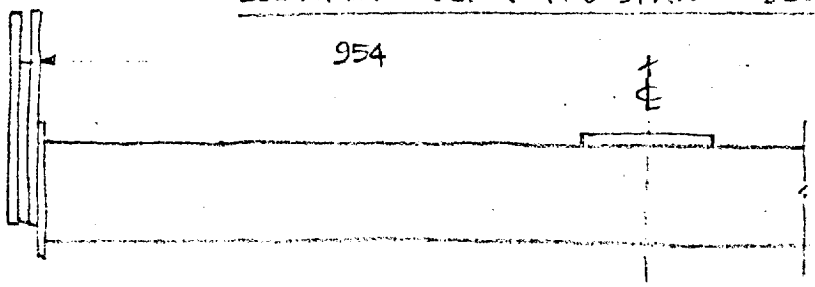
END ELEVATION



END PLAN.



ELEVATION – BEAM MID SPAN – SECTION

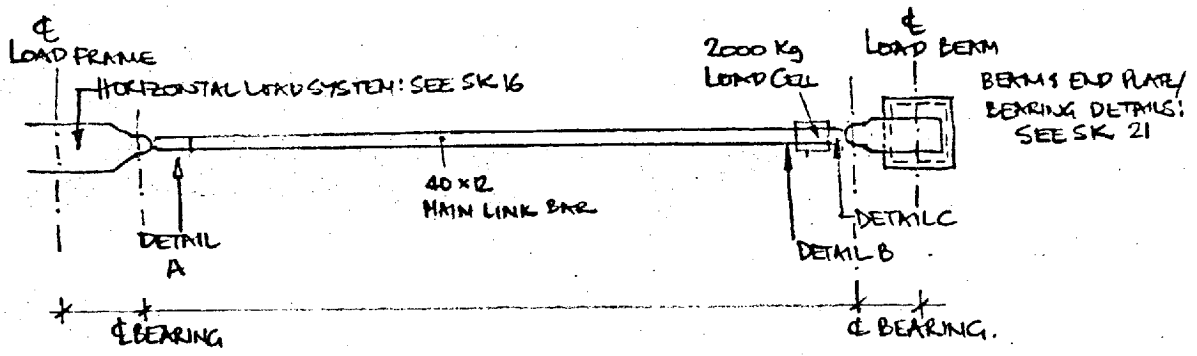


GENERAL ARRANGEMENT

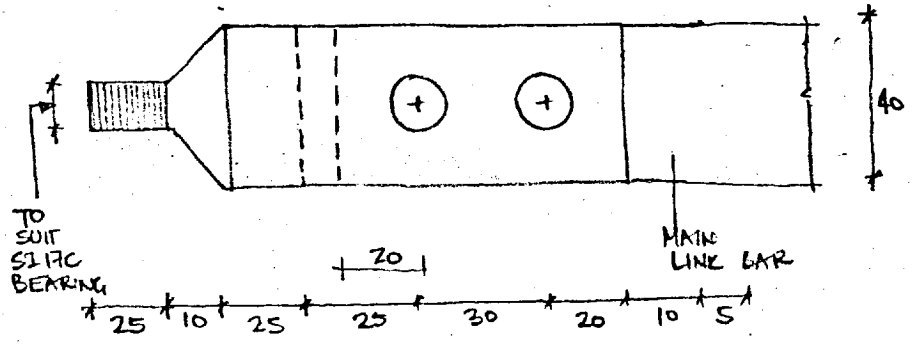
R# Sept 96

HORIZONTAL LOAD BEAM & DETAILS SK21

2Nº REQ.D (1Nº INITIALLY)

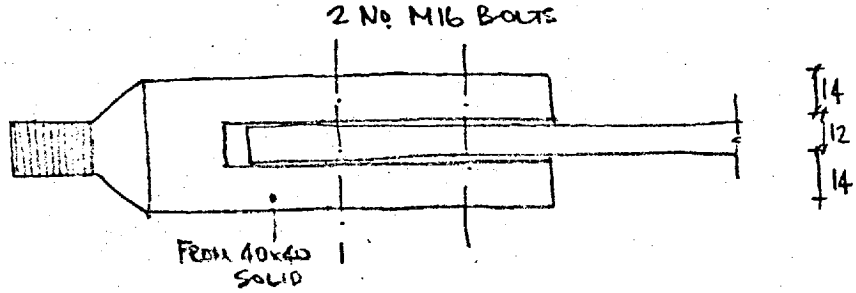


GENERAL ELEVATION

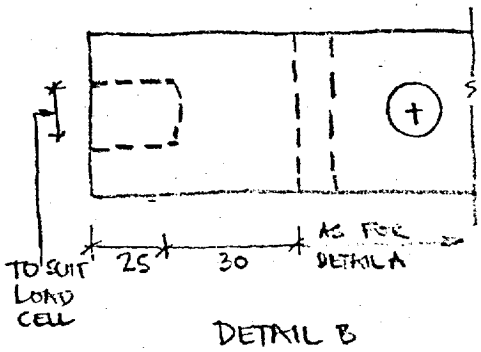


DETAIL A - ELEVATION

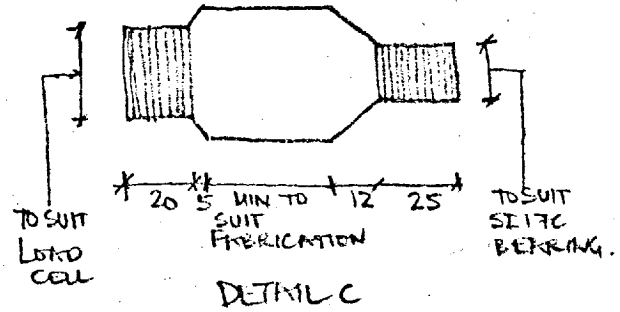
DETAIL B SIMILAR EXCEPT AS SHOWN BELOW



DETAIL A - PLAN



DETAIL B

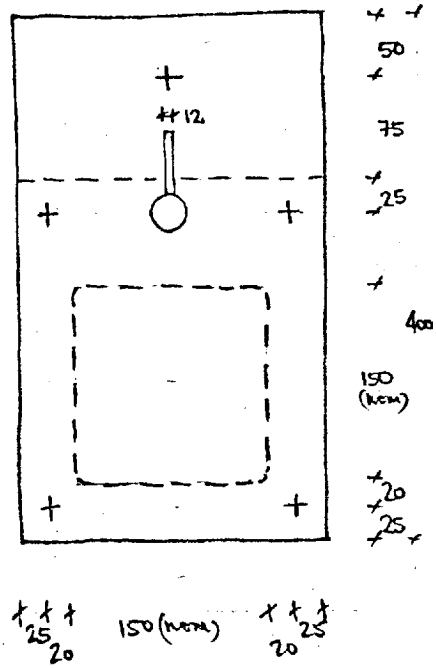


DETAIL C

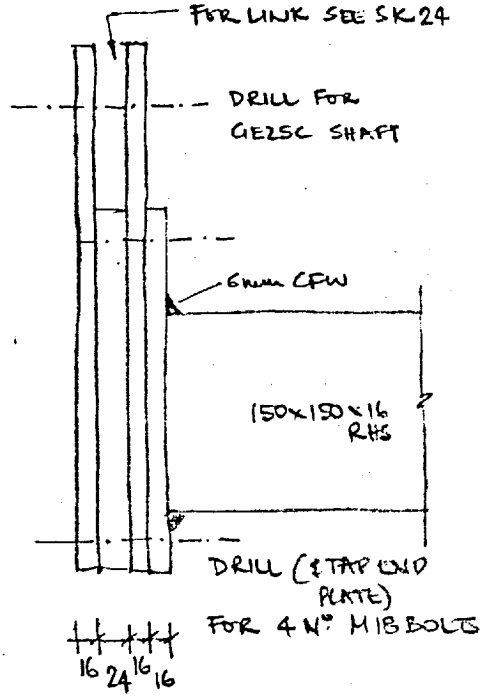
4 No. SETS REQD. (2 No. INITIALLY)

RH Sept 17

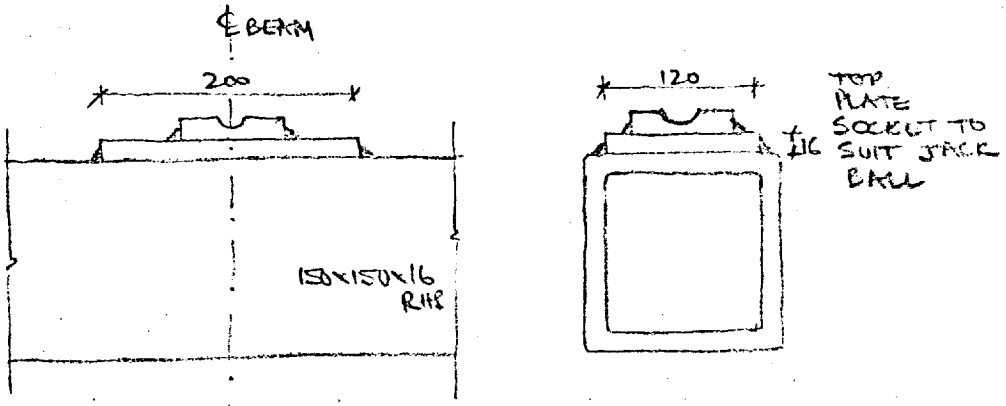
SK22: HORIZONTAL LOAD LINK & DETAILS



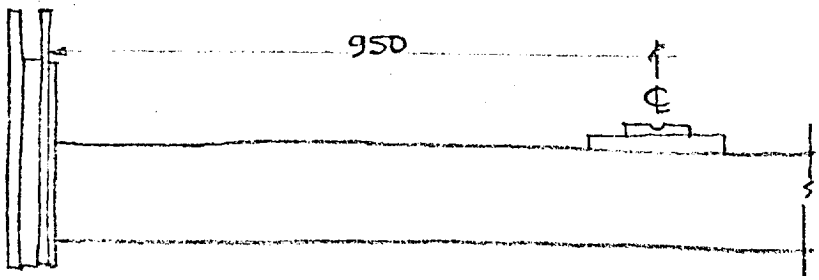
END ELEVATION



SIDE ELEVATION



ELEVATION – BEAM MID SPAN – SECTION

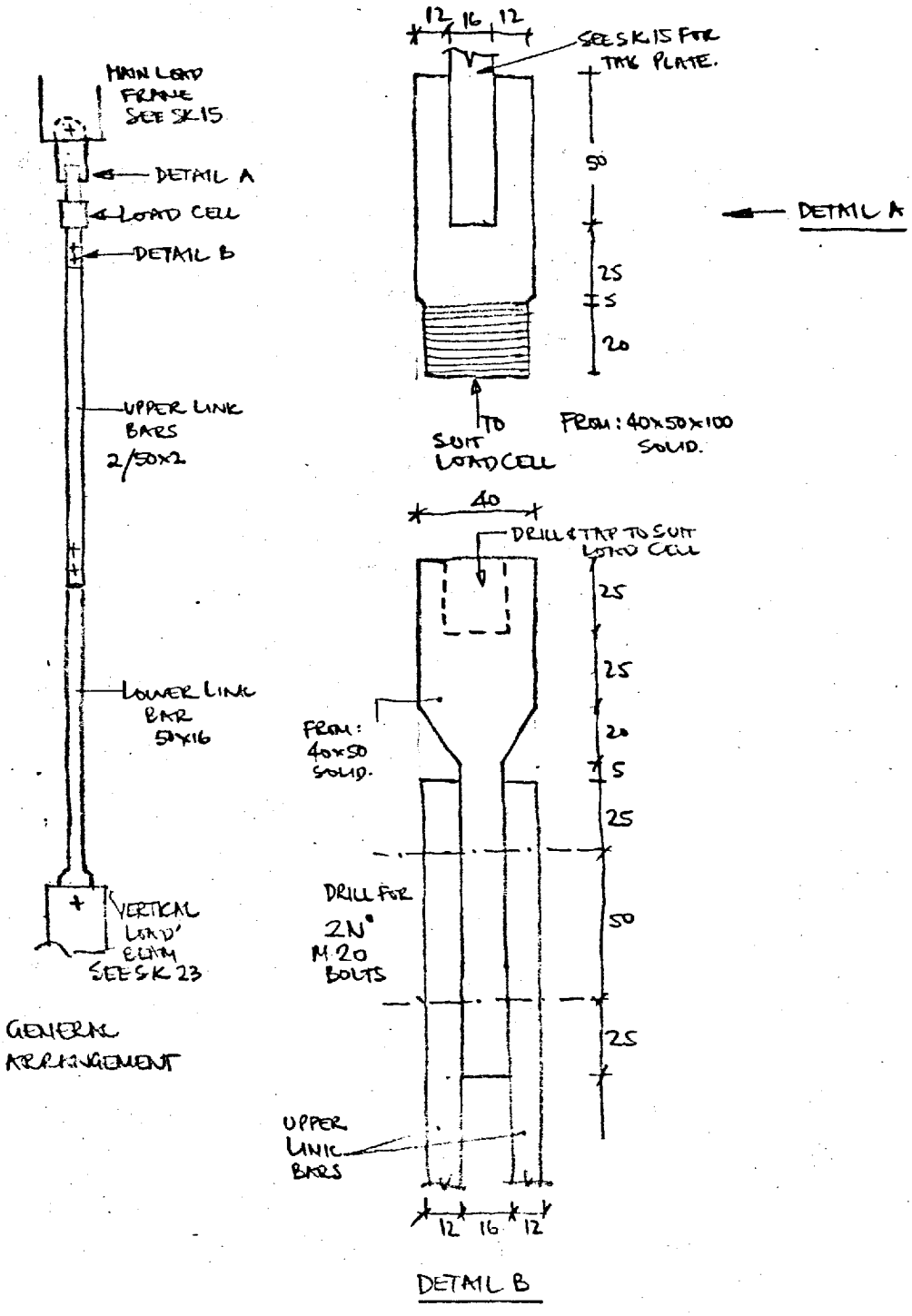


GENERAL ARRANGEMENT

RII
Sept 96

VERTICAL LOAD BEAM & DETAILS

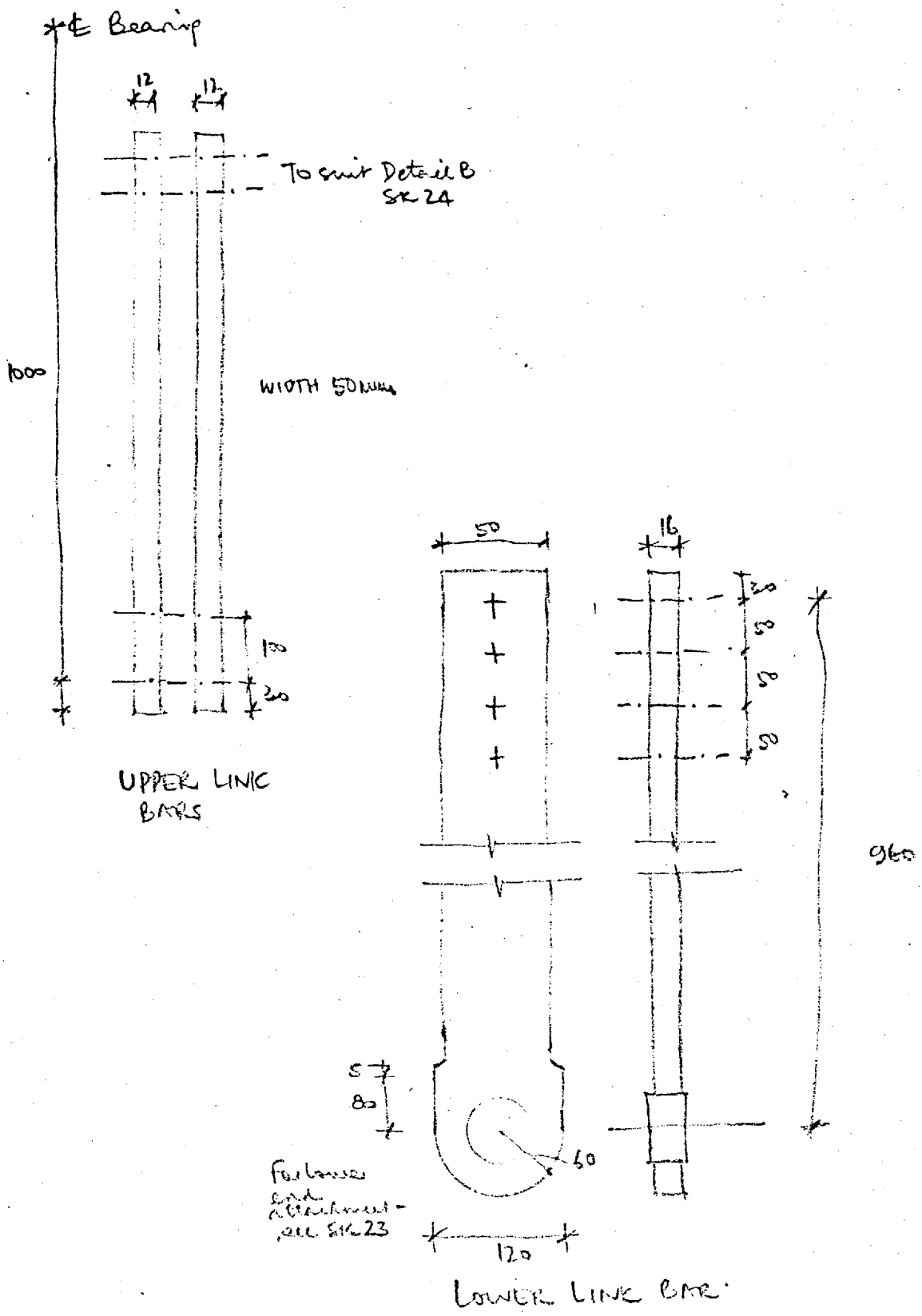
SK23



FOR LINK BARS & DETAILS SEE SK 25

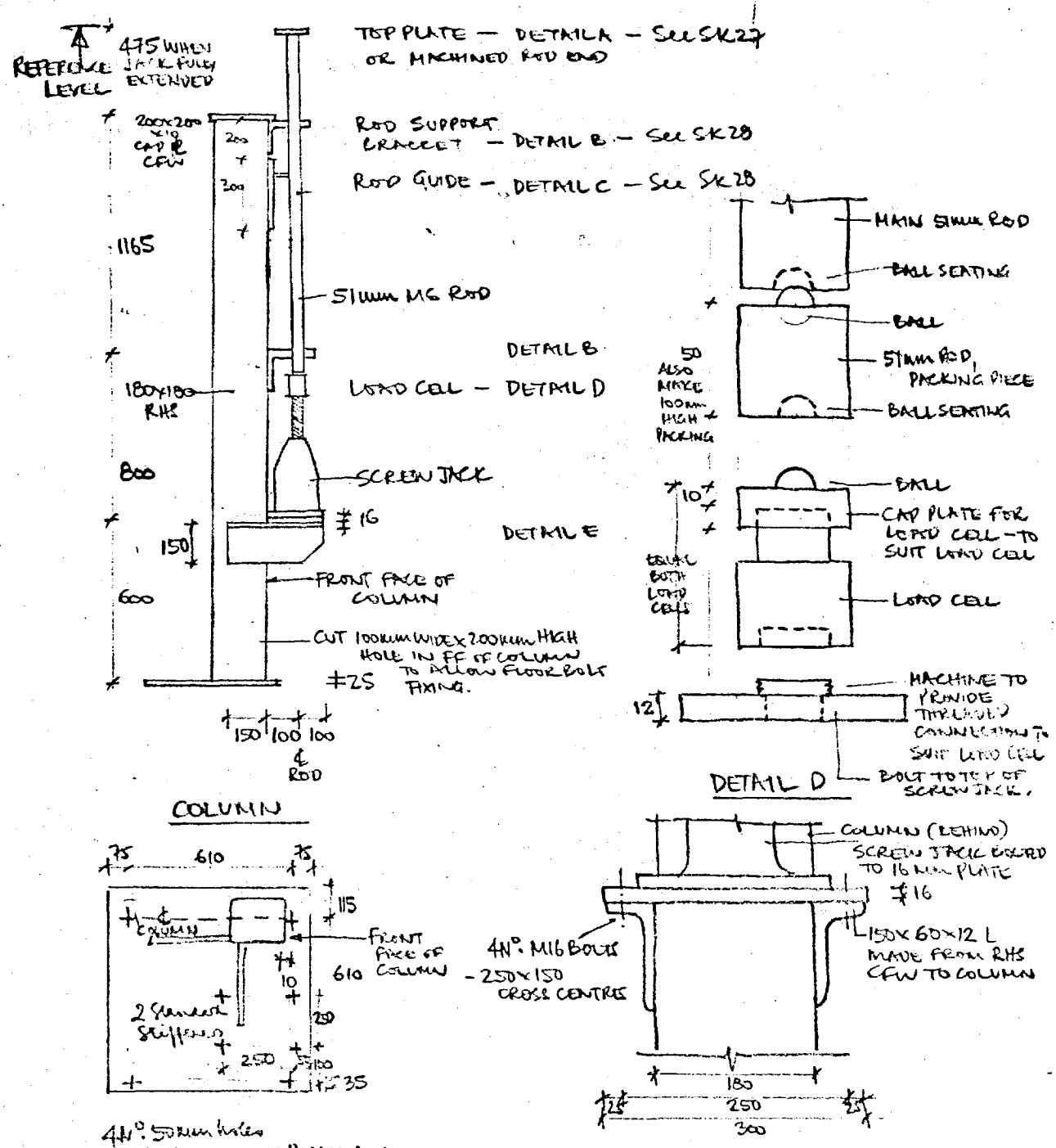
P.H
Sept 76

SK24: VERTICAL LOAD LINKAGE



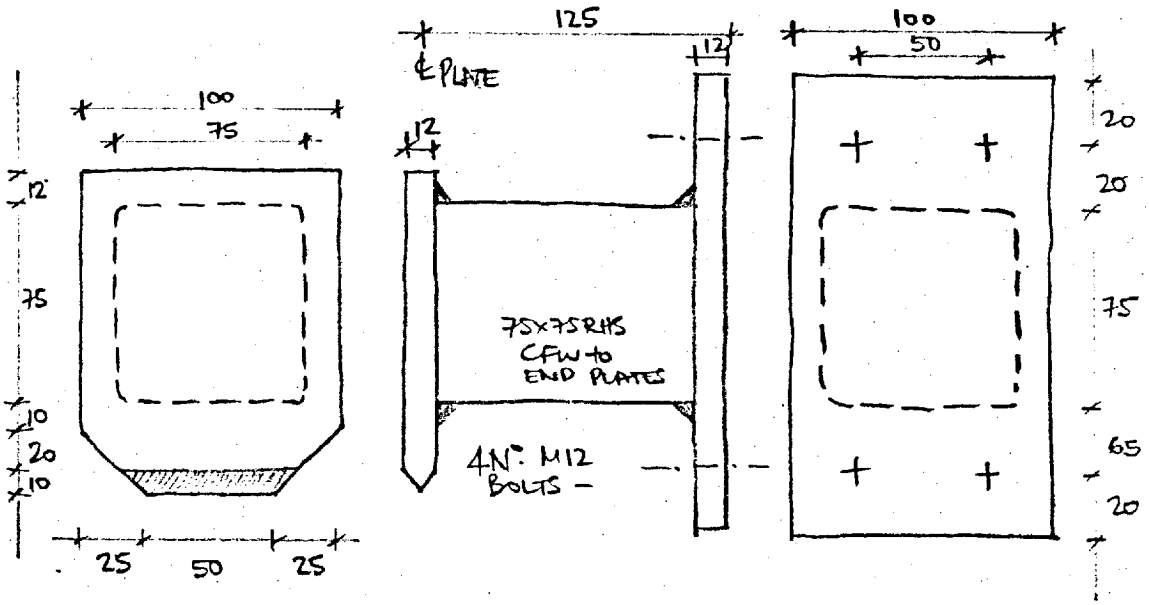
RH
Sept 76

LINK BARS (VERTICAL) & DETAILS SK25

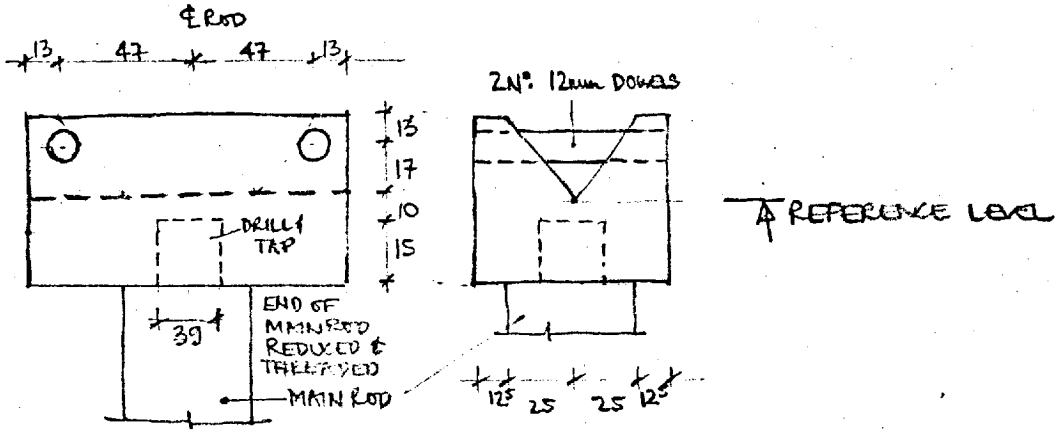


RH Nov 76

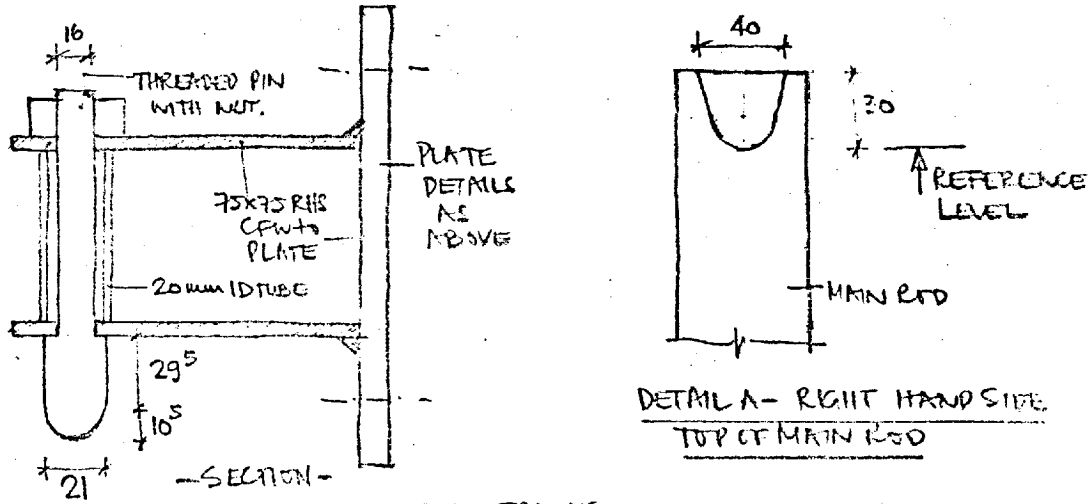
SK26: LOAD FRAME SUPPORT COLUMNS: 2N°. REQ'D



LEFT HAND SIDE - LOAD FRAME SUPPORT :



DETAIL A : LEFT HAND SIDE
TOP PLATE FOR MAIN ROD



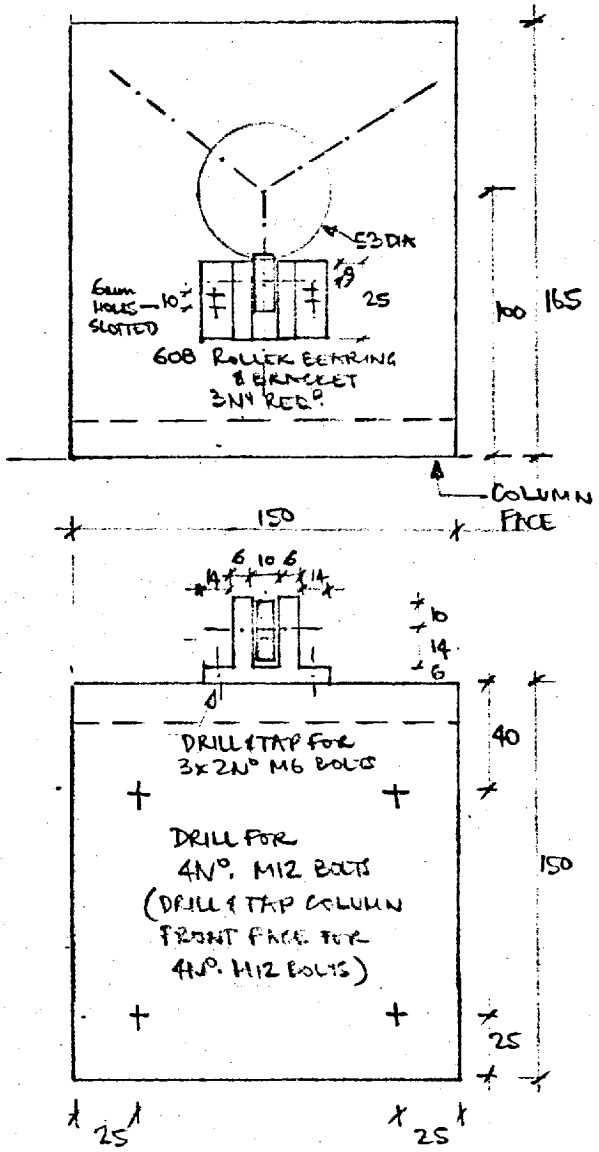
DETAIL A – RIGHT HAND SIDE
TOP OF MAIN ROD

RIGHT HAND SIDE - LOAD SUPPORT FRAME

RH
DEC 16

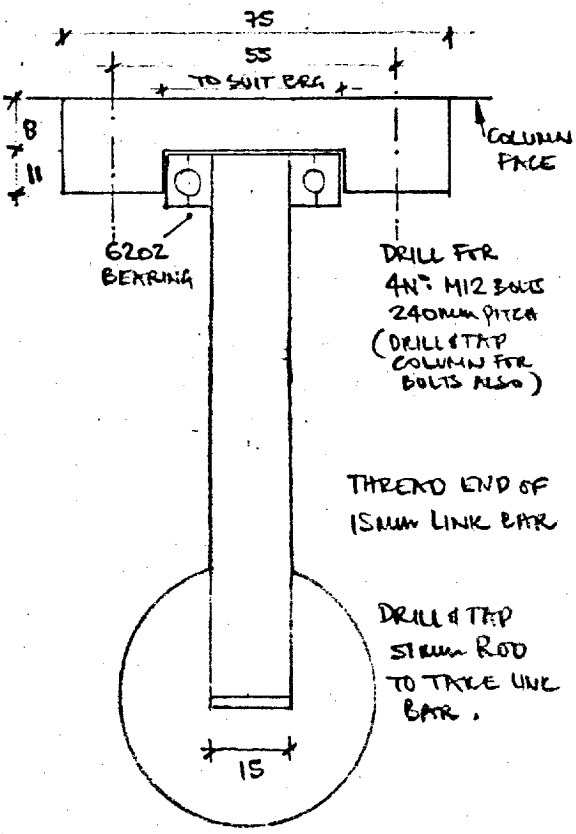
LOAD FRAME & FRAME SUPPORT DETAILS

SK27



DETAIL B: MAIN BRACKET

2Nº REQº PER COLUMN

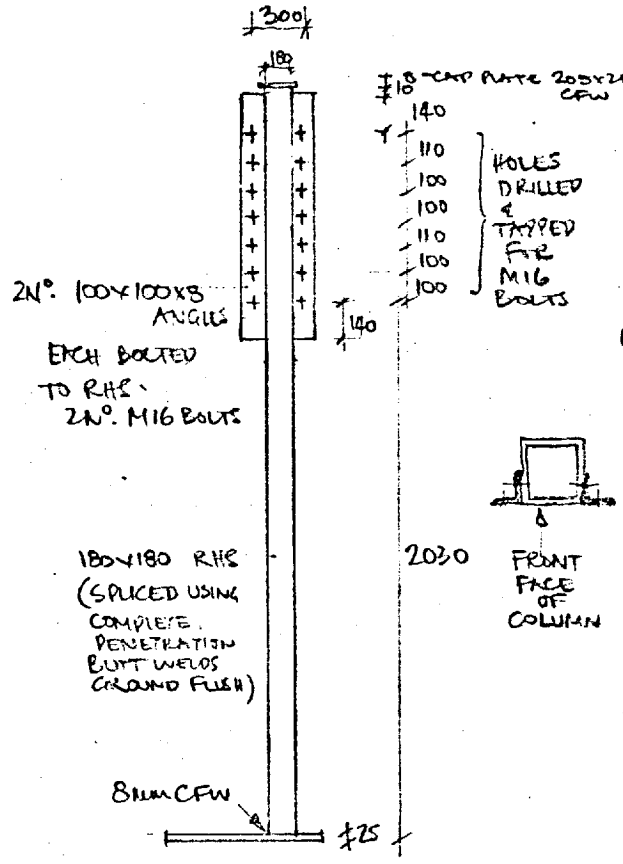


DETAIL C: ROD GUIDE

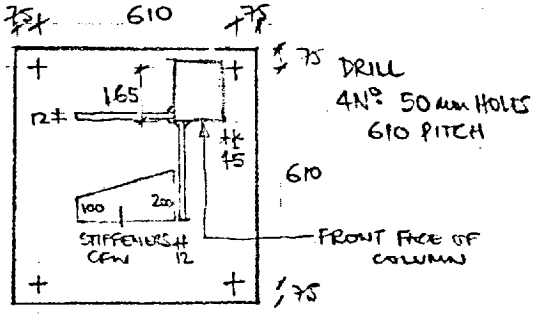
1Nº. REQº. PER COLUMN

R11
Dec 76

SK28: LOAD FRAME SUPPORT COLUMN DETAILS: 2

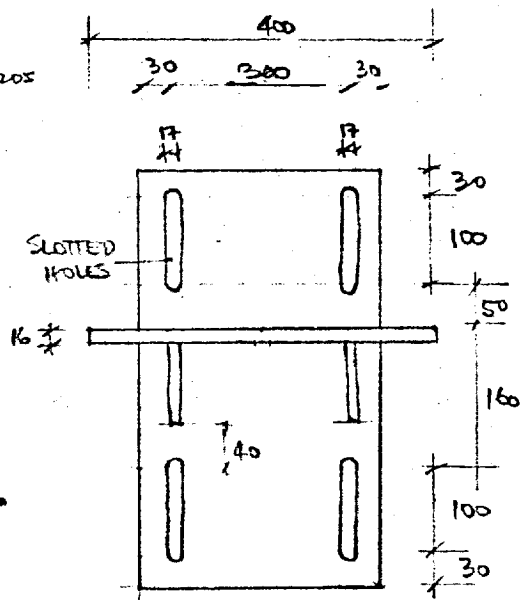


COLUMN

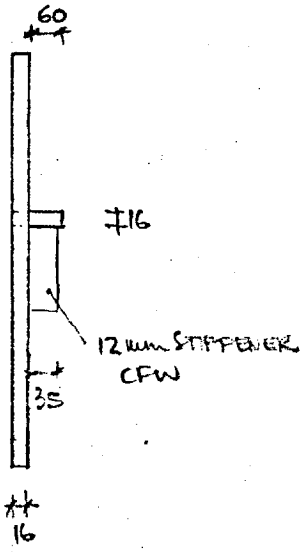


2N°. BASE PLATE DETAILS

2N°. BASE PLATE OPPOSITE HAND



FRONT ELEVATION



SIDE ELEVATION

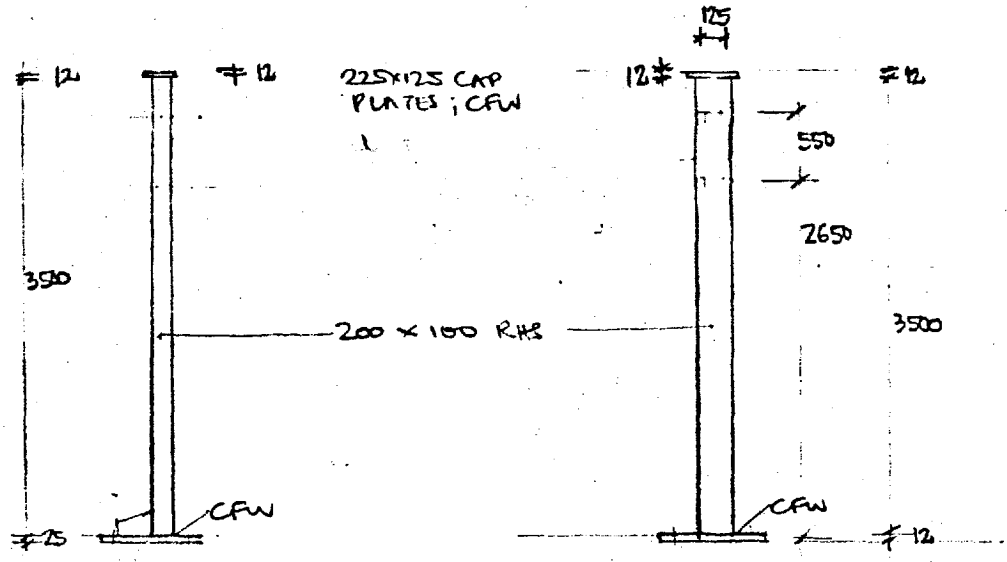
BEAM SUPPORT PLATE

* PROVIDE DRILLED & TAPPED HOLE IN CAP PLATE FOR ¾" LIFTING BOLT

RH
MAY 76

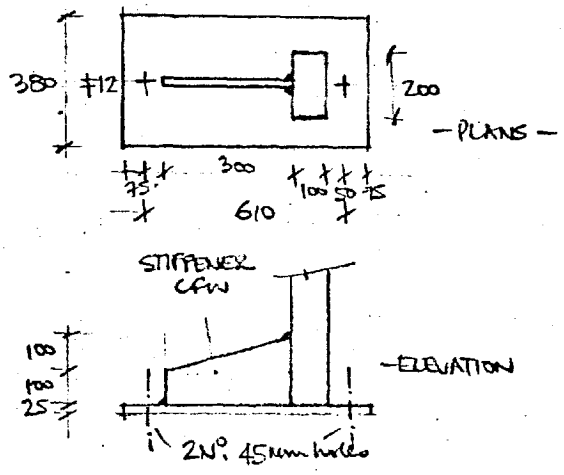
HORIZONTAL LOAD BEAM SUPPORT COLUMNS : 4N°. PR°.

SK 29



COLUMN TYPE 1 : 2N° REQ'D

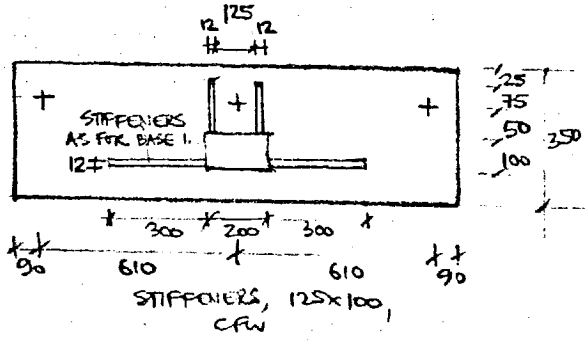
COLUMN TYPE 2 : 2N° REQ'D.



— PLANS —

— ELEVATION

BASE PLATE TYPE 1



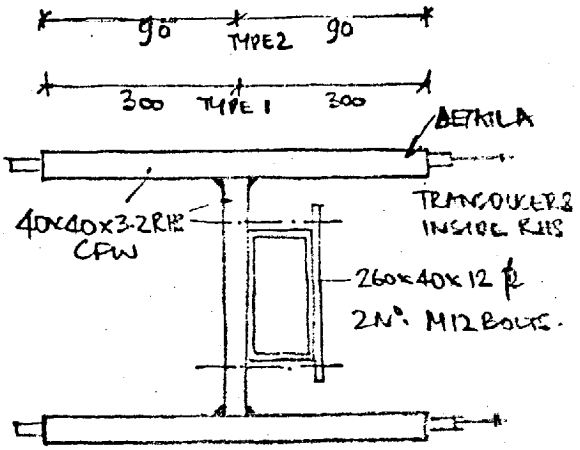
BASE PLATE TYPE 2

FOR HORIZONTAL TRANSDUCER SUPPORT DETAILS SEE SK 29

* PROVIDE DRILLED & TAPPED HOLE IN CAP PLATES FOR 3/4" LIFTING BOLT

RH
MAY 96

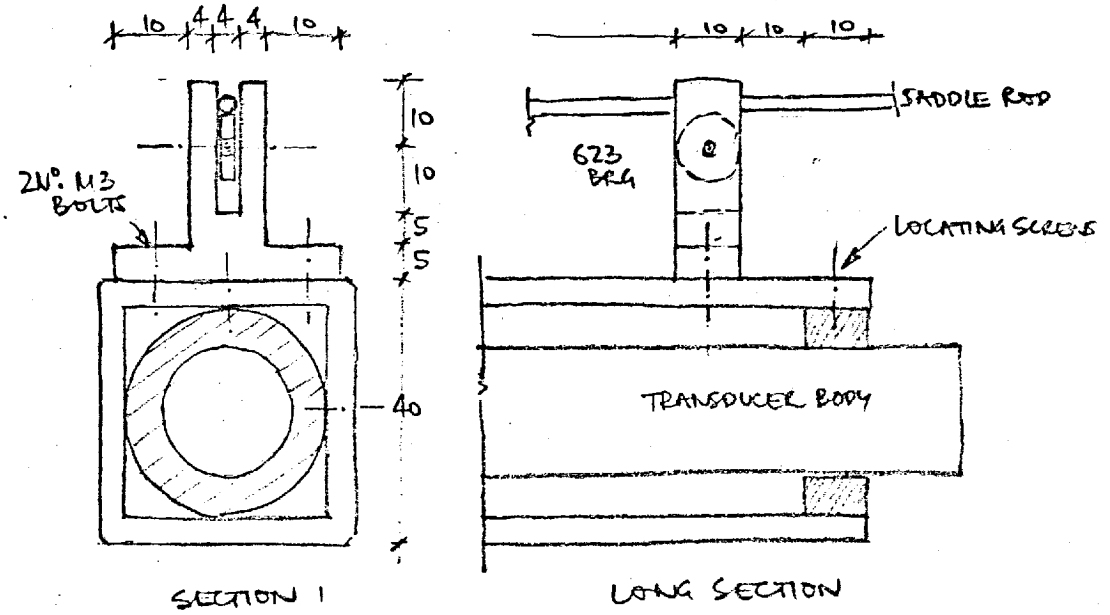
SK30: HORIZONTAL TRANSDUCER SUPPORT COLUMNS



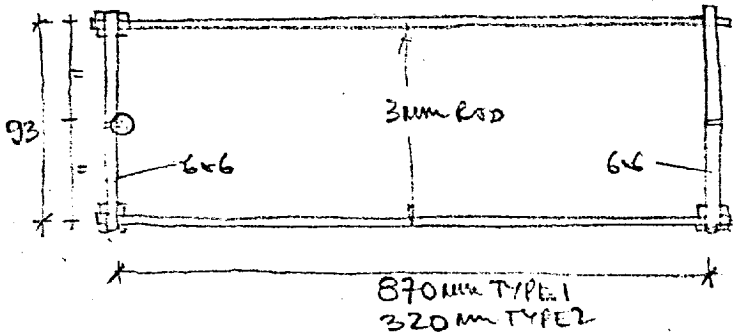
TRANSDUCER SUPPORTS-

2N°. REQ'D TYPE 1

2N°. REQ'D. TYPE 2



DETAIL A – TYPICAL EACH END (4N°. REQ'D EACH SUPPORT)



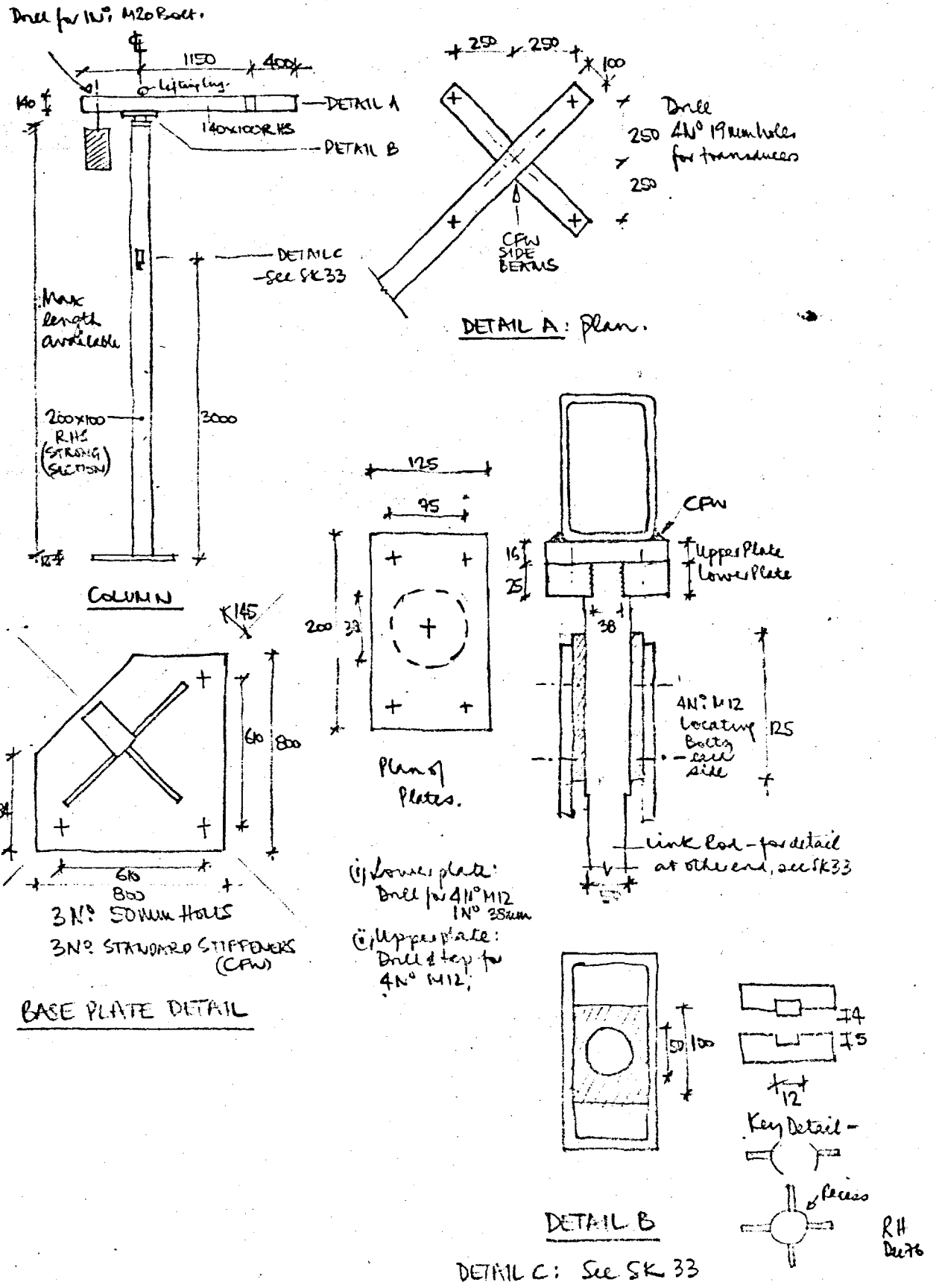
SADDLE ASSEMBLY

2N°. REQ'D EACH SUPPORT.

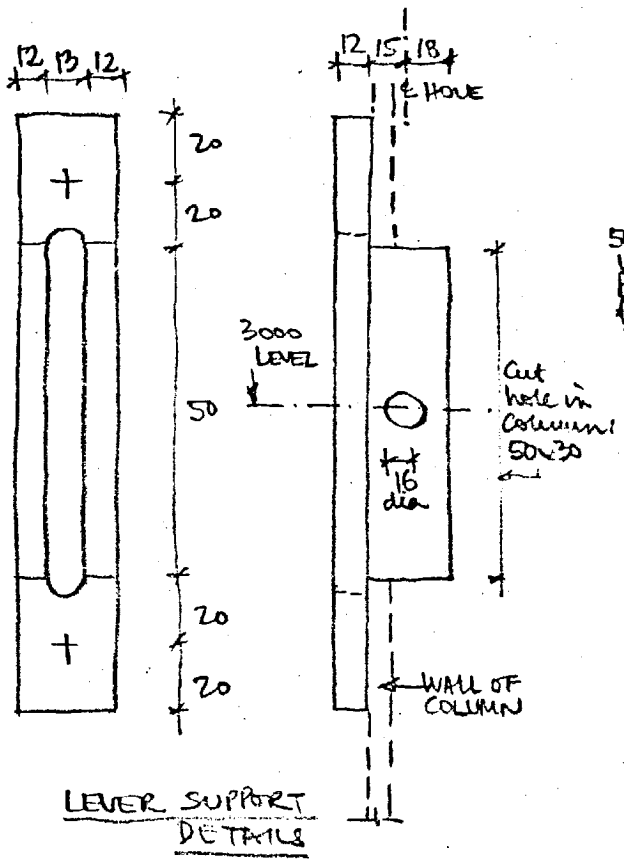
RA Jan 17

HORIZONTAL TRANSDUCER SUPPORT DETAILS

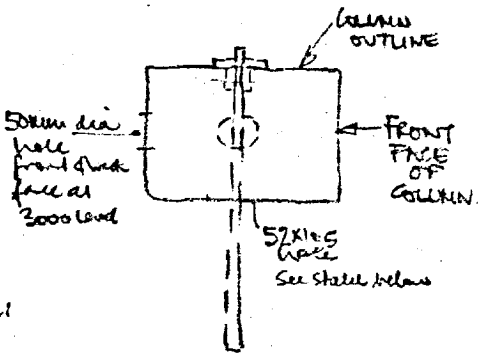
SK31



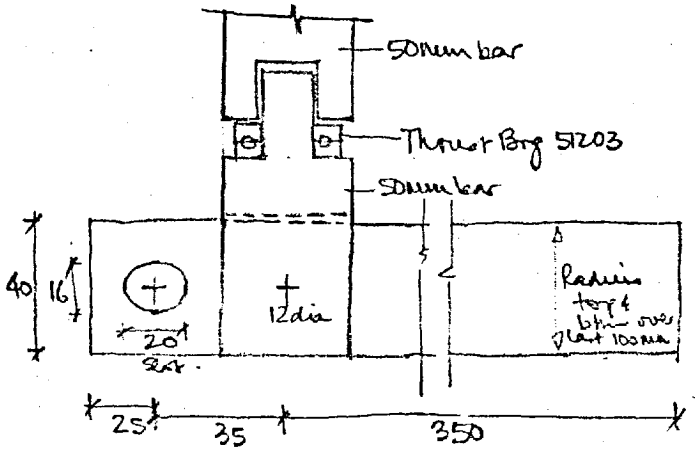
SK32: VERTICAL TRANSDUCER SUPPORT COLUMN

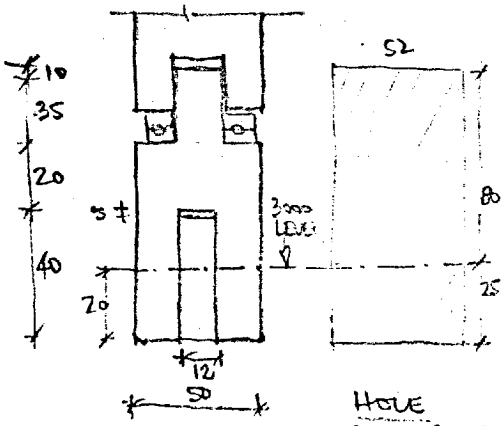


LEVER SUPPORT DETAILS



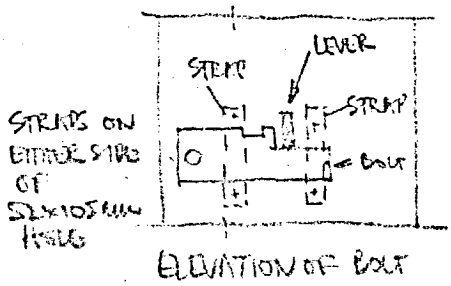
LEVER ORIENTATION



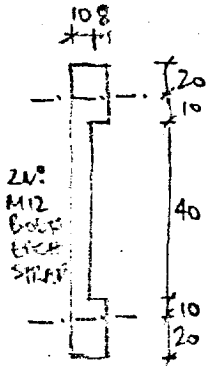


HOLE DIMENSIONS

LEVER DETAILS

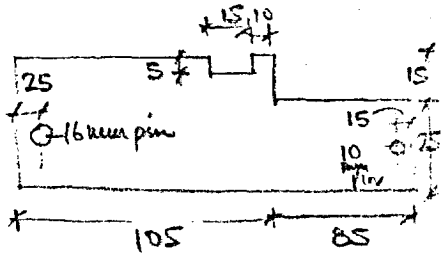


ELEVATION OF BOLT



STRAP : 2N° x 30 WIDE

SECTION

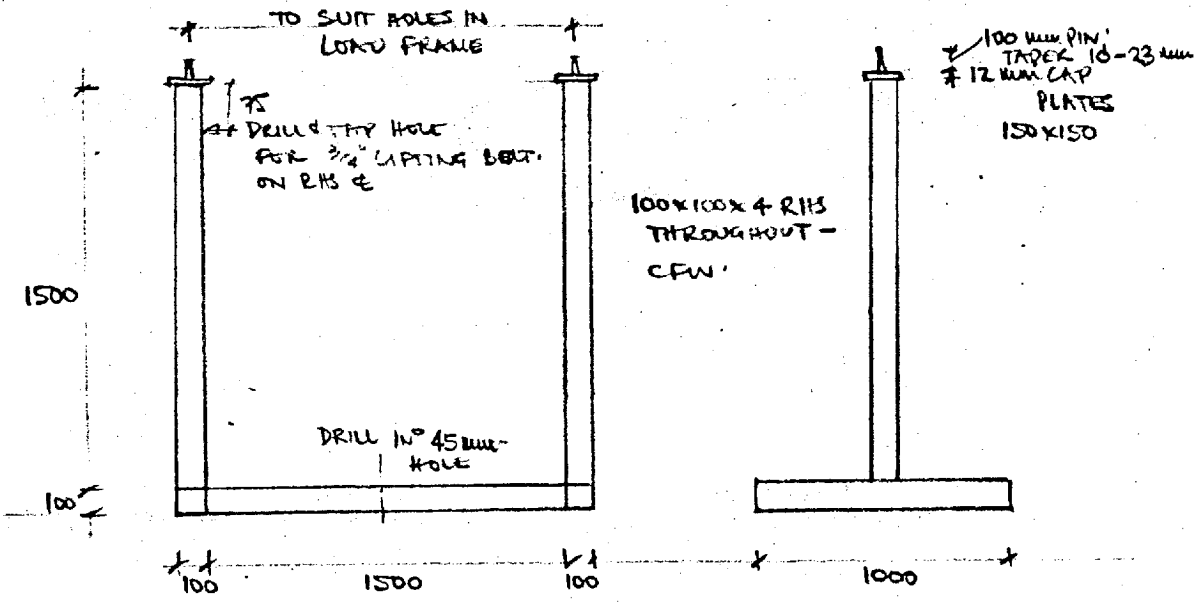


BOLT (8 mm THICK)

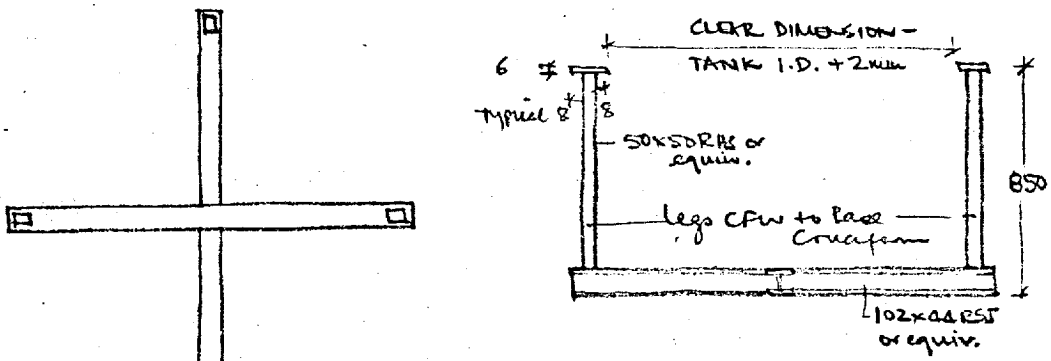
RH Dec 7

VERTICAL TRANSDUCER COLUMN – LEVER DETAILS

SK23

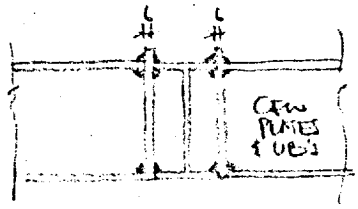


LOAD FRAME SUPPORT FRAME 1 N°. REQ°.



PLAN -

ELEVATION



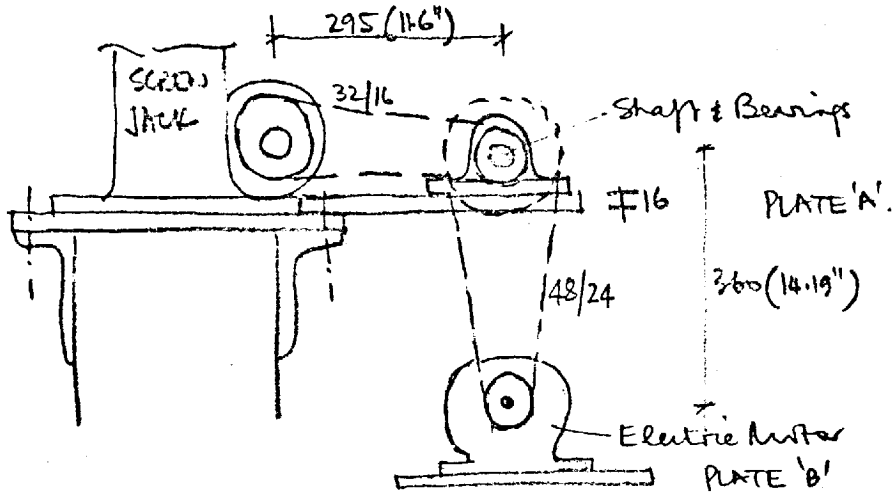
DETAIL AT CENTRE

NB : DRILL IN° HOLE TO SUIT M12 BOLT (CLOR) & WELD ON NUT, AT EACH END OF CRUCIFORM (FOR LEVELLING).

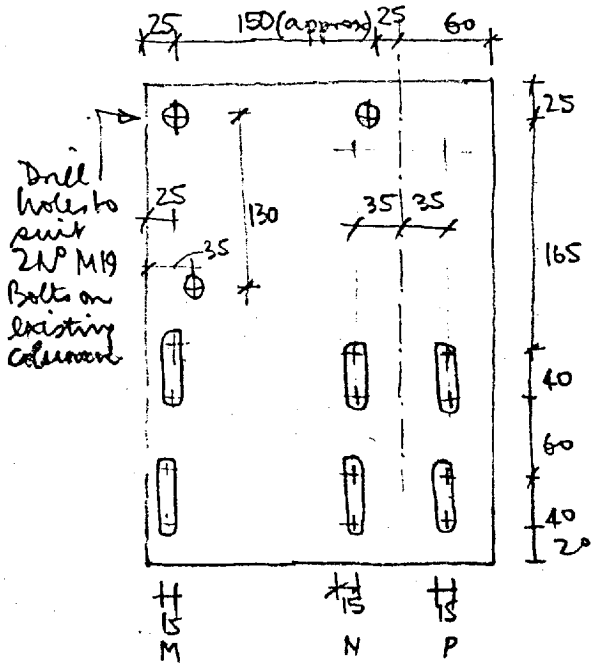
LID & ANNULUS SUPPORT FRAME : 2 N°. REQ°.

RH Nov 76

SK34: SUPPORT FRAMES

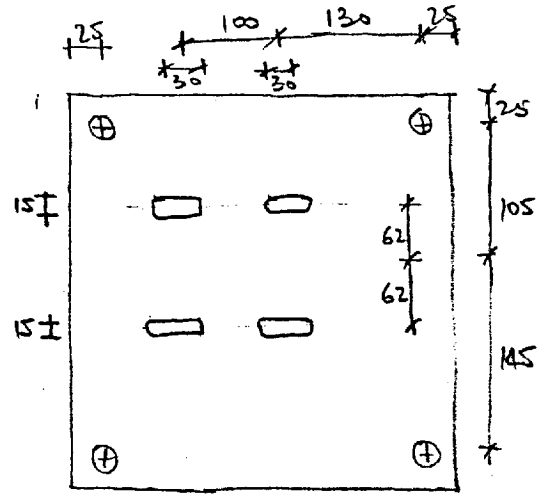


BASIC LAYOUT FOR SCREW JACK OPERATION:



One plate has slotted holes M & P
Other plate has slotted holes N & P

PLATE 'A' (2N° REQ°)



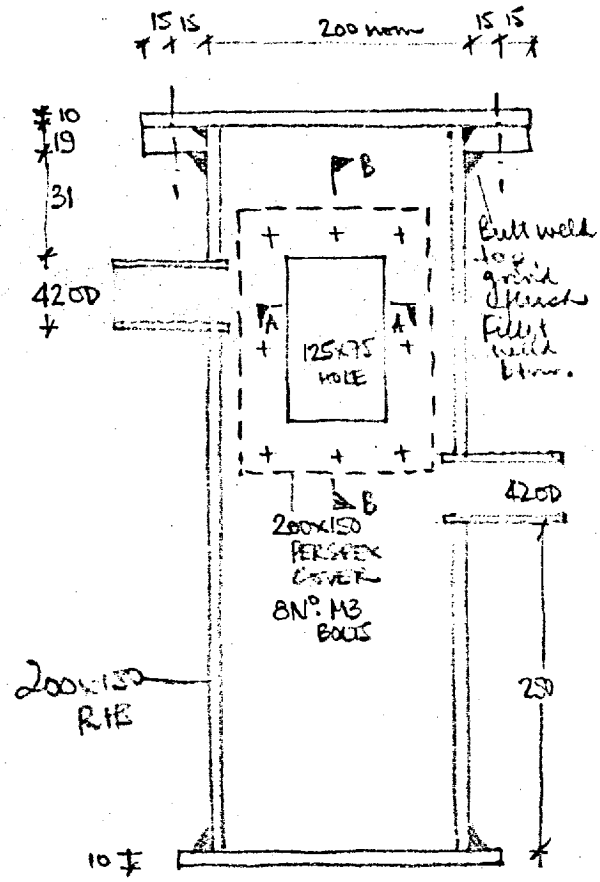
Provide also 4N° M12 studs for bolting to column base plate. Provide 4N° sleeve cut to length to establish correct height of motor.
Provide also sleeve to pack between plates A and B

PLATE 'B' (1N° REQ°)

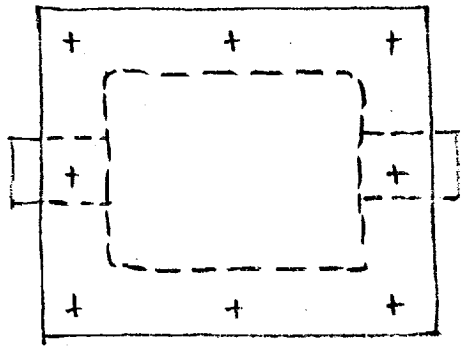
See also SK26

LOAD FRAME LOWERING CONTROL SYSTEM:

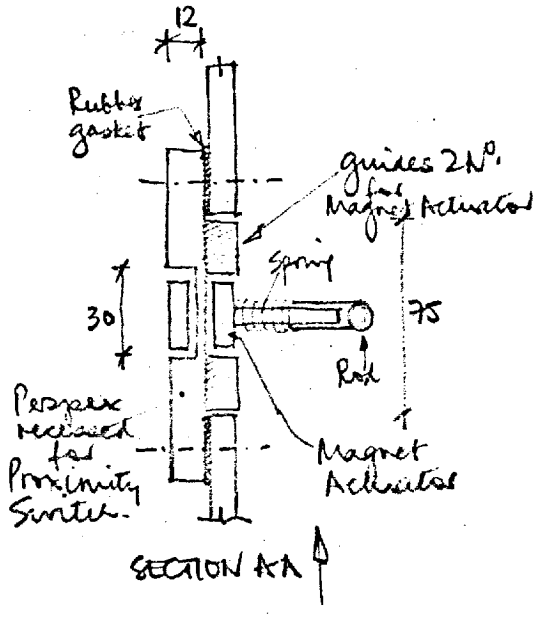
SK35

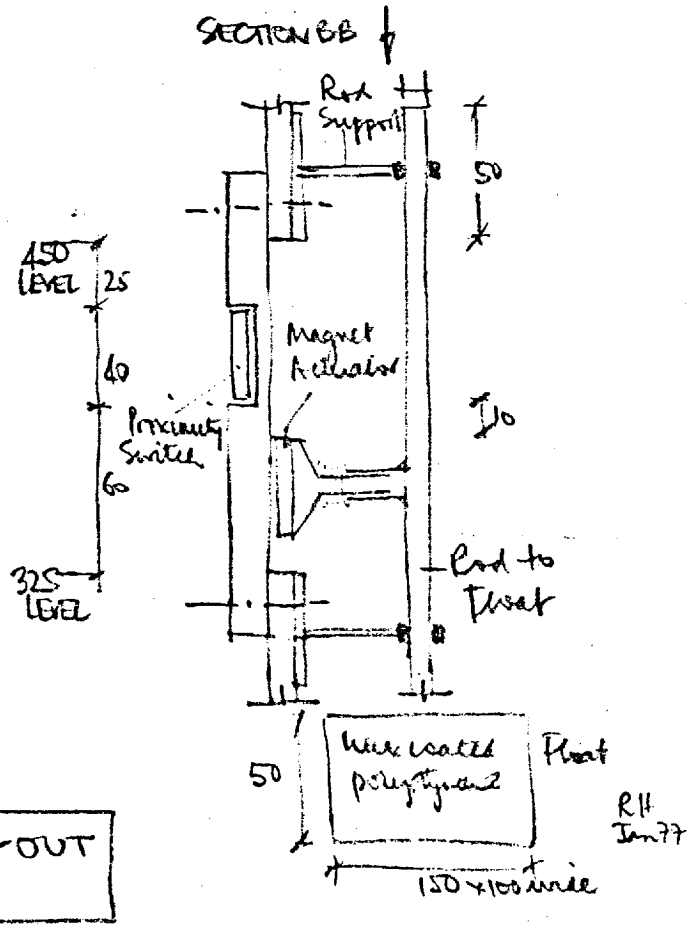


TANK SECTION



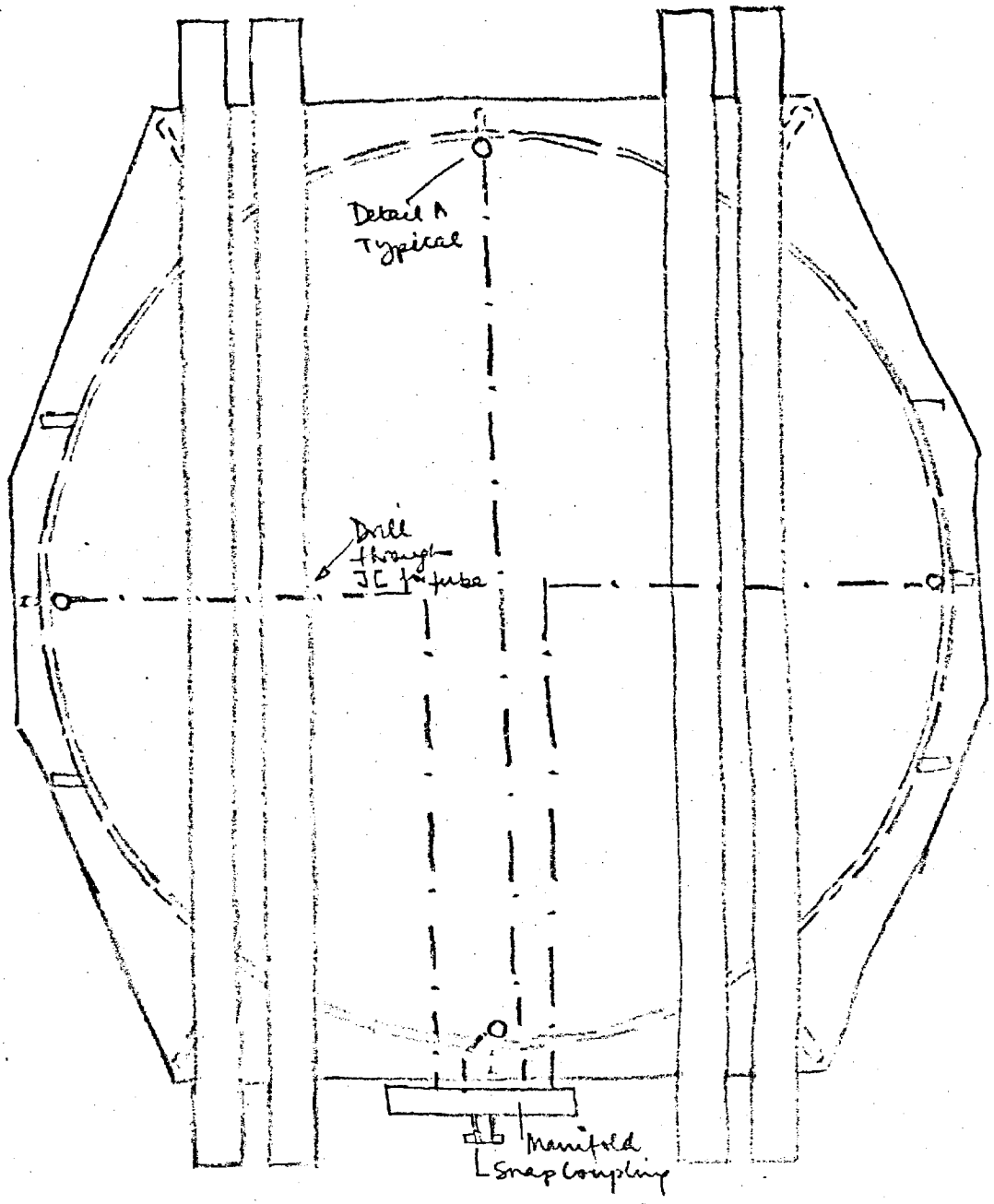
PLAN OF LID & FLANGE
8N°. M6 BOLTS





WATER TRAP FOR CUT-OUT OF VACUUM PUMP

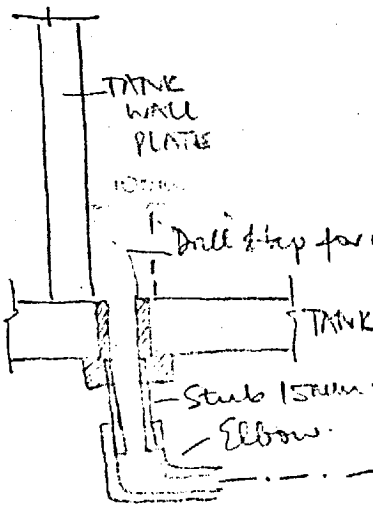
SK100



VIEW OF UNDERSIDE OF TANK

— . — . — 15mm OD TUBE

Note: Manifold tube at each a bend to ensure slight fall in all tubes

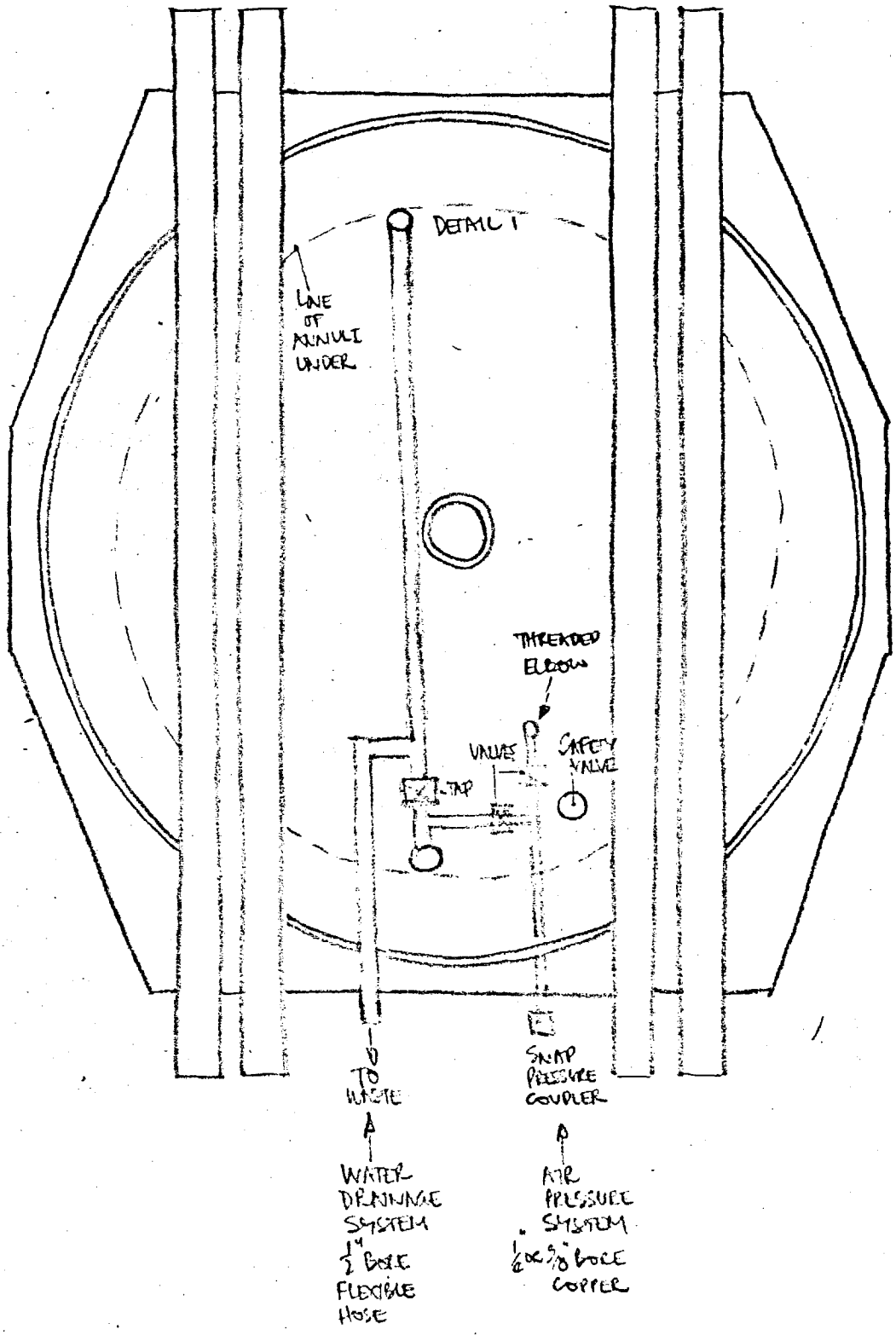


'DETAIL A'

TANK BOTTOM DRAINAGE

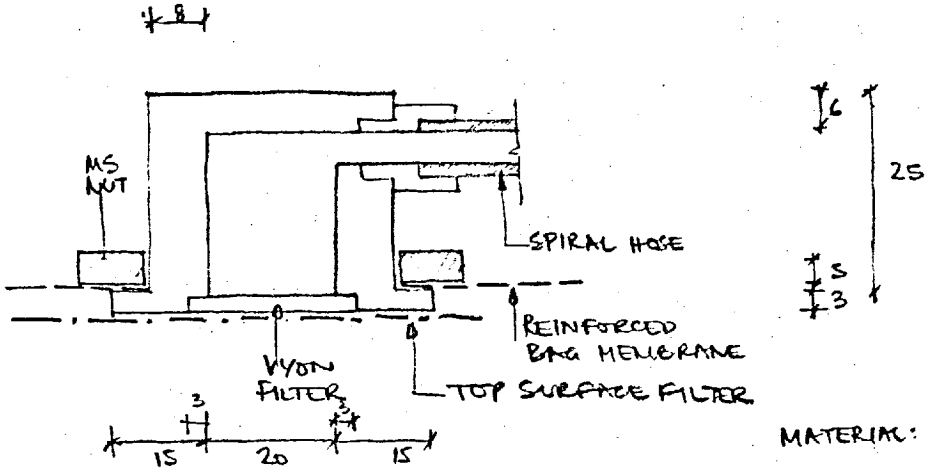
SK101

RH Jan 77



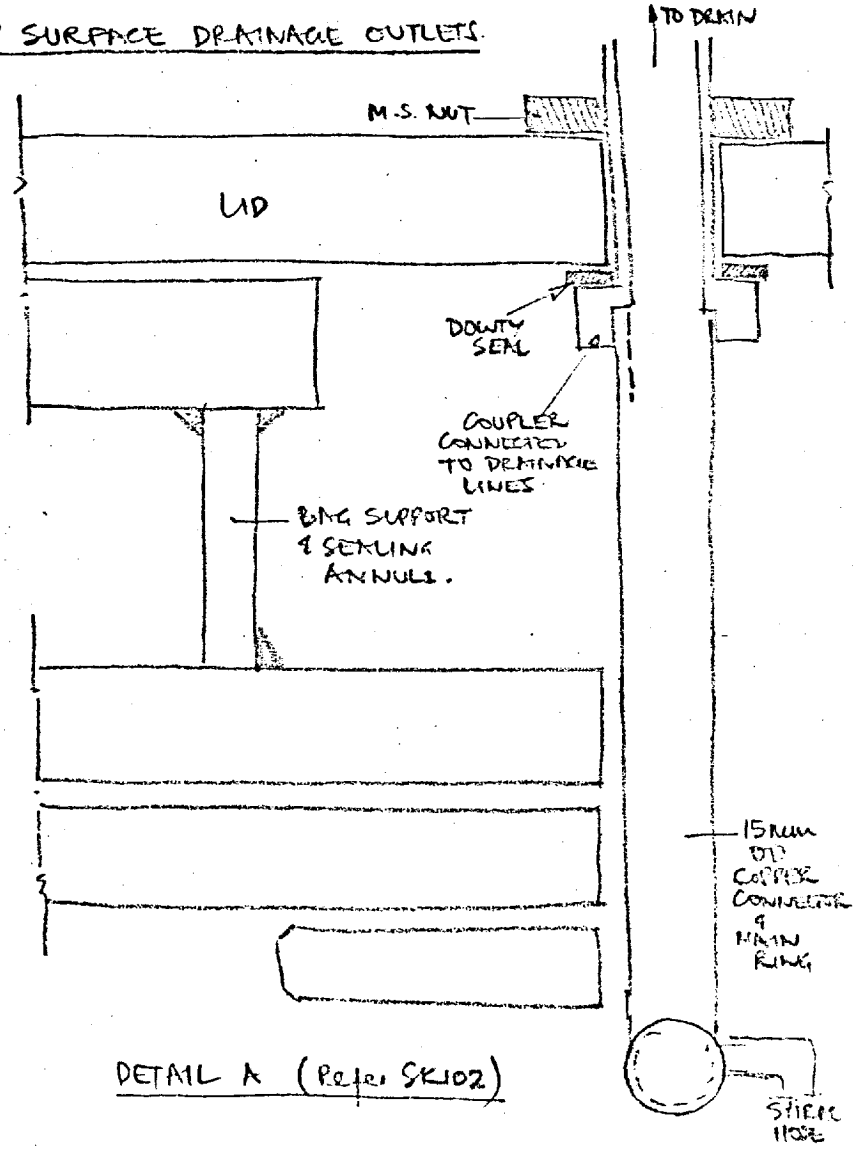
RH
Feb77

SK102: LID DETAILS; DRAINAGE, PRESSURE DETAILS



MATERIAL: BRASS UNO.

TOP SURFACE DRAINAGE OUTLETS.

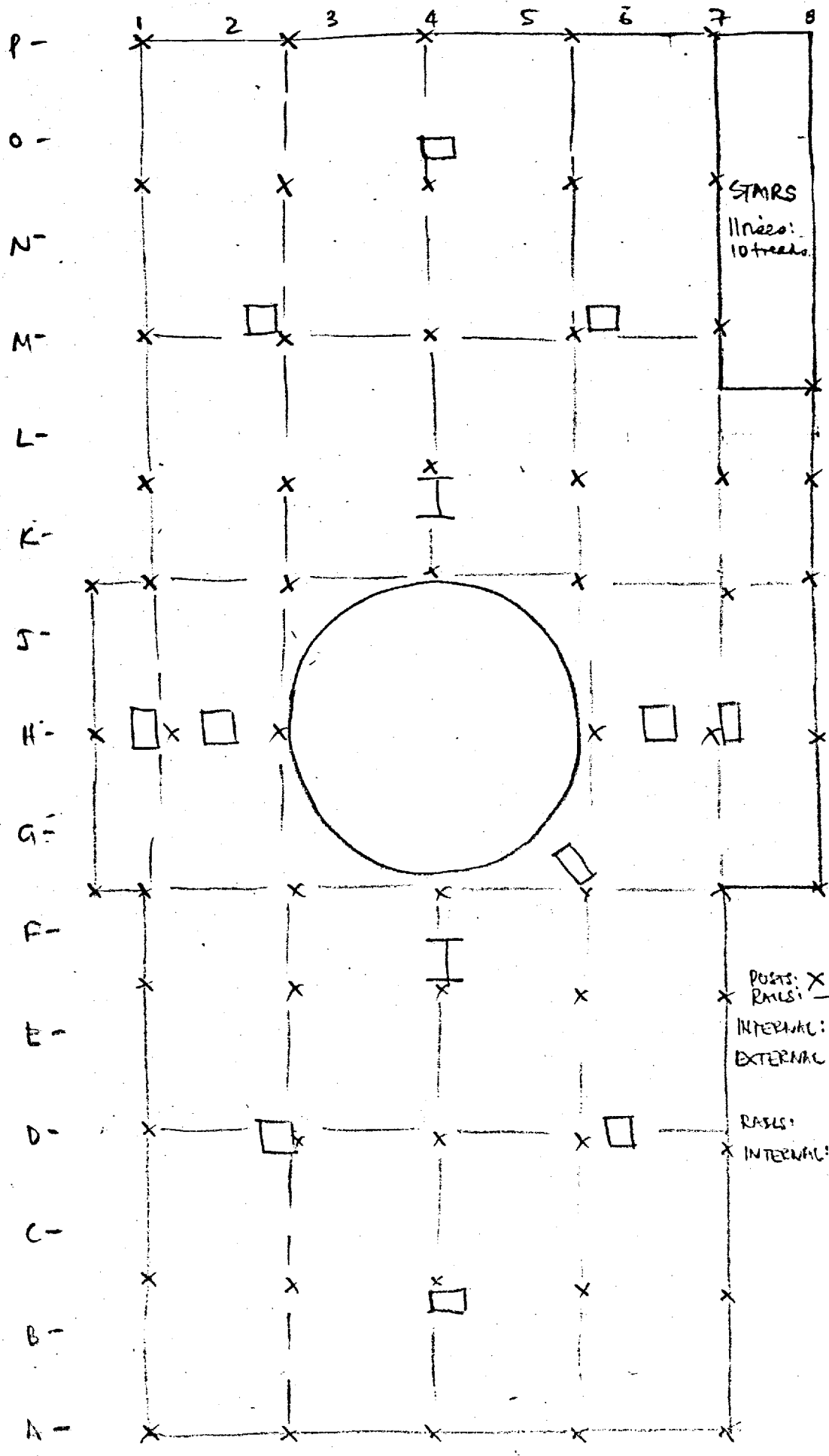


DETAIL A (Refer SK102)

RH
Mar 77

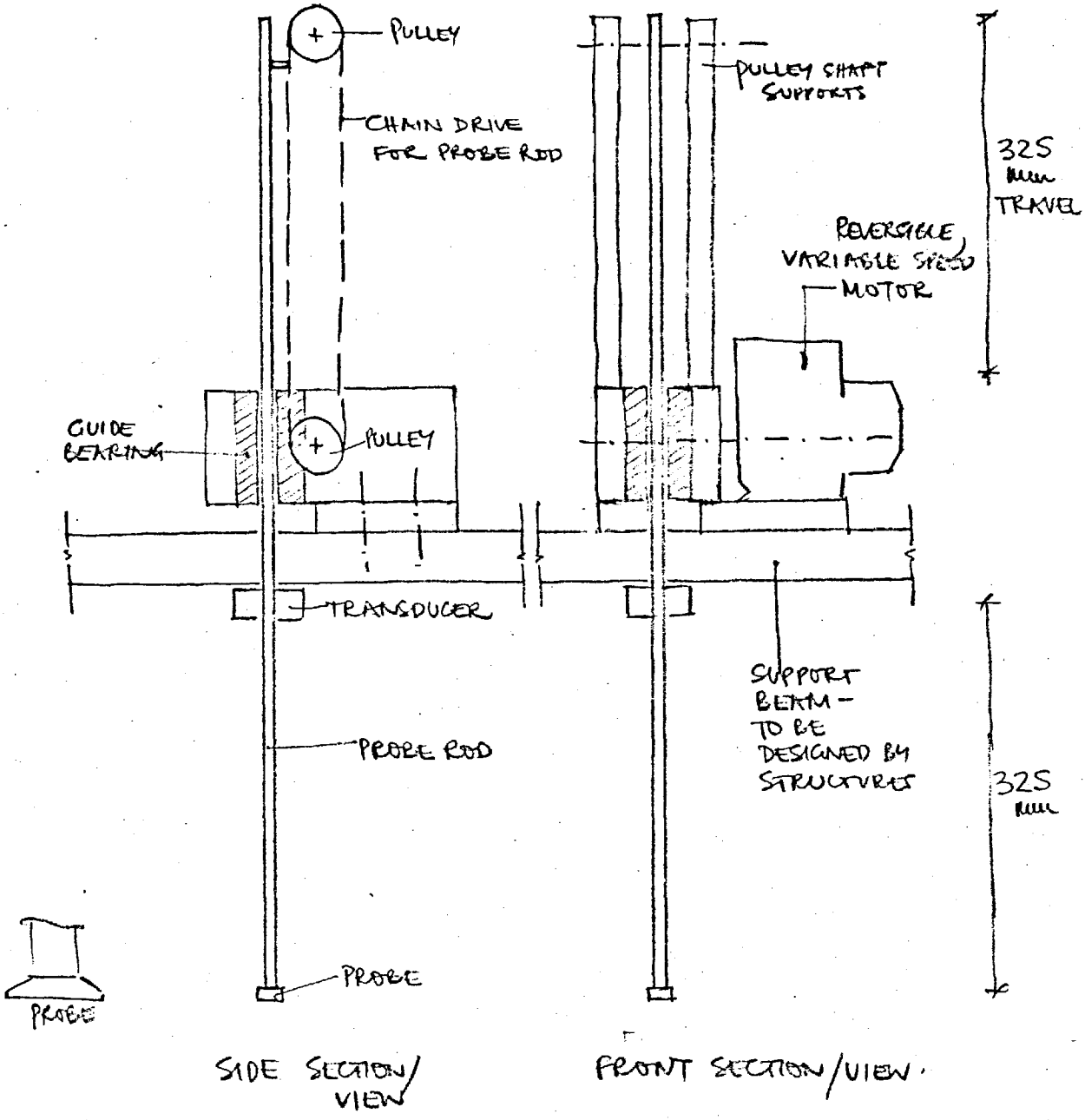
TOP SURFACE DRAINAGE DETAILS

SK103



RH
Jan 77

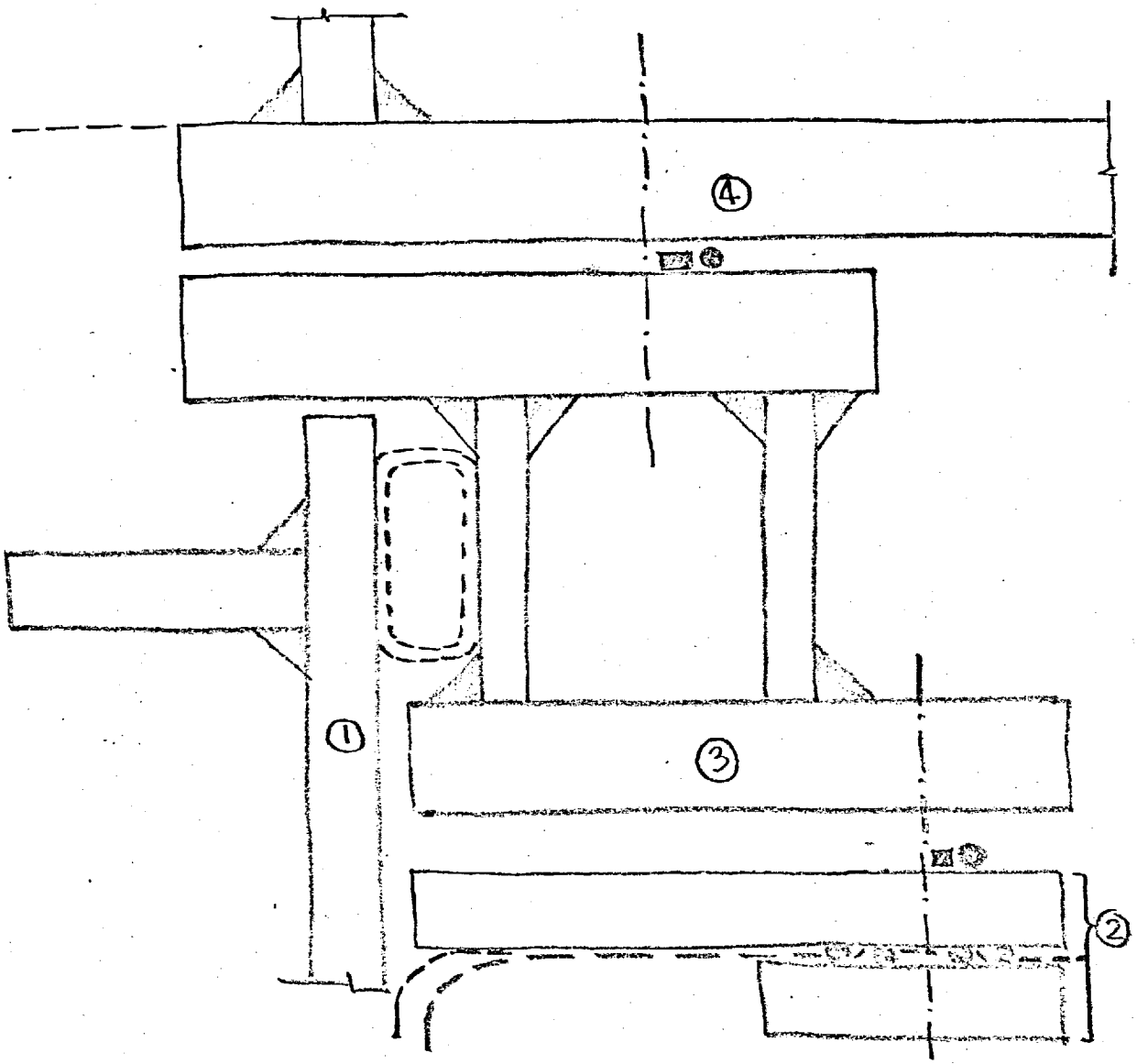
SK104: SCAFFOLD LAYOUT: HEIGHT 1.75m



PROBE & ROD SHOWN IN FULLY RETRACTED POSITION

RH JAN 77

SK/101



① MAIN TANK WALL

② BAG SEALING ANNULI

③ SIDE SEALING/BAG SUPPORT ANNULUS

④ STIFFENED LID

SCALE: APPROX 1:1

RH
June 76

CONSOLIDATION PHASE : TANK SEALING SYSTEM

GK1/A