THE STRESS DISTRIBUTION IN A SHEAR WALL

A thesis presented for
the degree of Doctor of Philosophy
in Civil Engineering
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by

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I.

This thesis deals mainly with a prototype test performed on an eight storey shear wall. The dynamic stress distribution, obtained from the prototype wall as the building was being shaken, is compared with results obtained both from a finite element analysis and from a photoelastic model study of the shear wall. The aim was to show the validity of a finite element analysis when compared with a photoelastic model, and to evaluate the accuracy of both it and the photoelastic study in the prediction of the elastic stress distribution in a prototype shear wall.

A large part of the thesis is devoted to an experimental program in which a portable dynamic strain pick-up instrument, and an associated Dynamic Strain Bridge was developed. This instrument was capable of recording small dynamic strains as low as .015 microstrain. Photoelastic modelling techniques are also discussed in some detail.

Comparisons between the results obtained from the prototype shear wall, the photoelastic model, and the finite element analysis were made mainly on the basis of the difference between the principal stresses and their orientation. The deflected shapes of the prototype and the finite element walls were also compared, and the theoretical frame/shear wall interaction was studied.

Floor slabs were shown to play a significant part in shear wall structures. The slabs stiffen the connecting beams in flexure, help prevent elastic instability, and probably aid in the ultimate strength of the shear wall by strengthening the spandrel beams and by acting as members in an effective truss within the wall.

Both the finite element and the photoelastic methods
were shown to predict the stress levels in the prototype generally to within 20%. The need for floor slabs to be included in the finite element analysis was shown.
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CHAPTER ONE

INTRODUCTION

1.1 PRELIMINARY

When an engineer designs a structure he must be confident that it will not fail under the maximum loading expected, and furthermore, if it does fail under extreme conditions, then the mode of failure should be such as to prevent a catastrophic collapse if possible.

The confidence an engineer has in his designs comes from two considerations. Firstly, there is the consideration of long experience in design and construction and perhaps the observation of some structural failures. However, this experience, although valuable, can be dangerous to an extent in that it may lull the designer into a false sense of security if his designs appear to be satisfactory.

The second, and most important way in which a designing engineer can gain a measure of confidence in his work and an insight into the actual behaviour of structures, is through reports on the testing of various structures. Model and prototype testing has now been carried out in the engineering field for a considerable period of time, and the actual behaviour of certain structures has been fairly well documented, both in the elastic and the post-elastic ranges.

With the application of the electronic computer to engineering problems in the last decade or so, many of the problems associated with the analysis of complex structures have been overcome. Now accurate calculations can be applied to members in complex structures where previously crude calculations, or rule of thumb methods were used. Consequently
more economical and efficient structures can be designed.

However, these more sophisticated methods of analysis also tend to give the designing engineer a false sense of security, as he is often inclined to accept the output from a computer as final and exact. Unfortunately, since the structures involved are complex, usually the only checks on the validity of the results that can be made are equilibrium calculations.

In order to have confidence in the computer analysis of a complex structure, a physical model of a typical structure can be made and tested to check the correlation between it and the mathematical solution. This is generally done in the development of a design method. Models for this purpose can be deflection type models, such as a Perspex model of a shell, or stress-type models, such as strain gauged or photoelastic models. Usually a sophisticated computer analysis can show a good correlation between the mathematical and the physical model of the prototype structure - this is particularly true if the boundary and loading conditions in the respective models are the same.

However, the correlation between the model method of analysis, be it a mathematical model or a physical model, and the prototype structure remains in doubt. This is due to two main factors:

1. The structure modelled is often an integral part of a larger and even more complex structure, and the interaction between the components of the whole structure is often difficult to estimate. This affects the modelling of the boundary conditions, and the loading on the structure under consideration.
2. The prototype structure is usually constructed from a different material than the model chosen to represent it. An example is that reinforced concrete structures are often modelled in Perspex or Araldite; these materials are essentially elastic, while reinforced concrete exhibits cracking with subsequent stiffness changes and plasticity near its ultimate strength. Even a micro-concrete model cannot accurately represent a prototype reinforced concrete structure in all respects because of the difficulties in modelling features such as the aggregate size and the bond strength of the concrete to the reinforcing steel.

To show the validity of a mathematical or a physical model it is necessary to relate the analytical results to experimental results obtained from a test of the prototype (full scale) structure, or a full scale component of the structure. If a good correlation between the mathematical analysis, the model analysis, and the prototype results can be shown to exist, then it is reasonable to expect that similar structures may be designed with confidence.

The aim of this research project was to find the behaviour of a prototype structure - namely a shear wall in an eight storey building, and to compare it with a finite element and a photoelastic analysis of the same structure.

1.2 SHEAR WALL DESIGN

In most multistorey buildings vertical walls run the entire height of the structure. These walls may form the lift shafts or the staircase wells in a building, or perhaps an exterior wall. Since they are required functionally, they are normally used as structural elements in such a building. These walls, as they are extremely efficient in resisting the lateral loads or shear forces due to wind and earthquakes, are known as shear walls. In high rise buildings
the structural requirements can supersede the functional requirements of these walls and they form the main lateral load resisting elements in the building.

Shear walls are usually pierced by holes for windows, doors, service ducts and other reasons. If these openings are regular and are in a vertical row, the shear wall is termed a "coupled shear wall". The coupling consists of a number of blocks, panels, or beams formed between the vertically arranged openings, which separate the entire wall into two distinct vertical walls.

The analysis of coupled shear walls was formulated by Beck (1) and expanded upon by Rosman (2, 3). In order to perform the analysis the discrete coupling beams in a coupled shear wall were represented by a series of elastic laminae capable of having flexural and shear deformations as well as resisting the axial forces which occurred if the two sides of the coupled shear wall had different sectional properties. An example of the elastic laminae can be seen in Fig. 8.5 on page 193. Axial and flexural deformations only were allowed in the two sides of the coupled shear wall.

Another method used to solve coupled shear wall structures uses an equivalent frame analysis - references 12-14 cover various aspects of this method.

Paulay (4, 5) gives an extremely comprehensive survey of the development of the elastic laminae method of analysis; works in the English, German and French speaking worlds being covered. He clearly formulates and critically examines the elastic laminae method and also takes into account the effect of cracking in the connecting beams and the subsequent local loss of strength and stiffness under an ultimate loading condition. He also provides various design criteria to ensure that the shear wall can actually attain the ultimate load resistance expected. Numerous references to coupled
shear walls can be found in Paulay's text (4).

Probably the most valuable integrated text on general shear wall behaviour to date has been compiled by Coull and Stafford Smith (6) from the Southampton Symposium on Tall Buildings in 1966. It is interesting to note that all the papers bar one in this text dealing with the mathematical analysis of shear walls refer to walls with vertically placed openings (either in a single row or a series of rows). However, Khan (7) discussed the effect of changes in the cross-section of shear walls and also the effect of regular but staggered openings in a shear wall. He proposed an analogous truss to deal with the problem, and in a discussion of Khan's paper, Rosman stated that his method using elastic laminae could be modified to deal with such a case.

Changes in the thickness of coupled shear walls cannot be directly taken into account in the normal method of laminar analysis, but Burns (8) has devised an approximate method using design charts to predict the shear forces across the connecting beams under a triangular loading distribution.

Although the analysis of coupled shear walls has been relatively well documented (4,6) the analysis of irregular shear walls has been avoided by most authors. Until the development of the finite element method of stress analysis (9,10) the only analytical methods available to solve irregular shear walls were those of the lattice analogy (11) and photoelasticity. However, the photoelastic method does not directly provide a complete stress analysis as explained in Section 5.2. Crude methods of analysis, such as taking sections across a shear wall and calculating the shear flow distribution for an unpierced section were probably used to solve irregular shear walls, but this method cannot take into account the effects of axial, flexural and shear deformations in such a wall.
1.3 VERIFICATION OF THE MATHEMATICAL ANALYSIS TECHNIQUES APPLIED TO COUPLED SHEAR WALLS

Rosman\(^{(15)}\), according to Paulay, undertook a photoelastic examination of a uniformly loaded, 10 storey symmetrical shear wall and found a very good agreement with the stresses derived from an elastic laminae analysis. Barnard and Schwaighofer\(^{(16)}\), using a strain gauged Araldite model, found that the maximum difference between their experimental results and the results provided by an elastic laminae analysis was only 5%.

However, as far as the author is aware, no correlation between the elastic laminae analysis and a prototype of a coupled shear wall has been produced.

1.4 THE PURPOSE OF THE ENSUING WORK

An irregular shear wall is a complex structure of the type described in Section 1.1 and in order to check the validity of analytical methods available, the stress distribution (in the elastic range) in a prototype shear wall was found. The shear wall concerned unfortunately had only eight storeys above a basement, and its overall height was just over twice its width - thus by world standards it was not a typical slender shear wall. Most of the shear walls discussed in the Southampton Symposium\(^{(6)}\) were at least three times as high as they were wide.

However, the prototype was available for testing, and it was felt that if a good correlation between the two analytical methods used (i.e. a finite element analysis and a photoelastic study) and the prototype structure could be found, then the use of these design methods could be justified. On the other hand, if the correlation was poor, modification of the analytical methods could be formulated on the prototype results.
The research project was divided into three separate phases: the finite element analysis, the photoelastic study, and the prototype test. The finite element and the photoelastic method of analysis have three main points in common:

(a) The material used in the structure (either the hypothetical material forming the finite elements or the Araldite in the photoelastic model) was homogeneous, isotropic, and perfectly elastic (neglecting creep in the Araldite).

(b) The structure modelled was the shear wall alone, loaded only with a lateral load applied at each floor.

(c) The boundary conditions were the same in that both the mathematical and the physical model were rigidly fixed at the base, and were constrained to two dimensional movement only.

The degree of correlation between the finite element analysis and the photoelastic study was of considerable importance. This is because the former is available to most designing engineers with the growth of computing facilities and program libraries. On the other hand the photoelastic method of analysis is restricted to designers with access to a photoelastic laboratory - such facilities are usually found only in research establishments.

This degree of correlation depends mainly on the layout of the mesh used to subdivide the mathematical model into finite elements. If the mesh is too coarse then the approximation of the mathematical model to the physical model may be rather crude. This is particularly true in the areas of high stress in and around the connecting beams in a shear wall. The type of finite elements used also has a bearing on the degree of approximation found in the mathematical model.

The three points discussed concerning the factors common to both the analytical methods can also be discussed with
respect to the reinforced concrete prototype shear wall.

(a) Reinforced concrete is non-homogeneous, anisotropic and is not perfectly elastic. It cracks, and changes in the sectional properties of the structure occur.

(b) The prototype wall was an integral part of the building, and because of this the actual loading on the shear wall was unknown, although the majority of the lateral load apparently came from the floor slab.

(c) The boundary conditions were unknown in that the action of the shear wall within the overall structure could not be accurately assessed. Also the degree of base fixity in the prototype wall was unknown.

Thus it can be seen that the three points discussed with respect to the two analytical methods are, in fact, assumptions required to enable the shear wall analysis to be made. The major part of this research project was to find the effect of these assumptions by comparing the two sets of analytical results, both with each other, and with the experimental results gained from the prototype. In this way the validity of the finite element and the photoelastic methods of analysis could be evaluated, and any necessary modifications made to them to provide a method by which irregular shear walls could be designed with confidence.

Furthermore, the general behaviour of the prototype shear wall and its performance with respect to the rest of the building could be studied. This would perhaps lead to a better understanding of shear walls in general: a necessary understanding as shear walls are playing an increasingly important part in the construction of modern high-rise buildings.

Broad details of the three separate phases of the research project are as follows:
1.4.1 The Finite Element Analysis. A mathematical model of the shear wall was formed by a mesh of finite elements using an existing computer program, as described in Chapter 6. The finite element method has been extensively upgraded since its conception. Clough\(^9\) has contributed much to its development, and in fact MacLeod\(^17\) has formulated a new rectangular finite element specifically for use in shear wall analyses. This analytical method lends itself to irregular shear walls as virtually any elastic structure can be solved using it, providing firstly that the structure can be adequately idealised by the finite element mesh, and secondly that the computer used is large and fast enough to handle the large number of matrix manipulations involved.

1.4.2 The Photoelastic Study. The use of photoelastic models to analyse shear wall structures is by no means new. Rosman\(^15\) used a model to verify his elastic laminae results, and Kokinopoulos\(^18\) studied the stress distribution and deflections of single storey walls with various cut-outs and different height to width ratios. Naumann and Walter\(^19\) also verified the elastic laminae results with a photoelastic model and Arcan\(^20\) used a photoelastic model to verify his own method of analysing a coupled shear wall. Elms\(^21\) studied a section of a coupled shear wall, placing the emphasis on the stress distribution in the whole structure rather than on the connecting beam shear forces.

The photoelastic model used in this particular study was \(1/60\) full size, and unlike the previous photoelastic tests mentioned above, the model had floor slabs attached. Since the photoelastic method of analysis only provides the difference between the principal stresses and their orientation directly, the comparisons between the two analytical methods of analysis and the prototype results were based on the photo-
elastic results. Chapter 5 deals with the photoelastic techniques used.

1.4.3 The Prototype Test. This third phase was the main feature of the research project, involving a considerable experimental program to prepare the apparatus for the stressing of the shear wall and the subsequent measurement of the dynamic strains involved. The equipment developed could equally well be used to perform similar tests on other prototype structures.

In order to find the stress distribution in the prototype shear wall, it had to be stressed. The only feasible way to do this was by shaking the building in one of its natural modes: thus the shear wall was loaded with the resulting lateral forces from the inertia of the building. A shaking machine was specifically designed to shake the 8 storey building.

As the building was only loaded laterally by the inertia load this meant that the static stresses in the wall due to the gravity load it was carrying was not recorded. However, these stresses are relatively predictable - they are also beneficial as they provide a prestressing action to the concrete at the base of the wall.

Once the building was shaking the dynamic strains were measured over a grid of points positioned on the wall at each level of the building. Chapter 2 deals with the development of the portable dynamic strain pick-up instrument and the electronics used to record the extremely small dynamic strains. The construction of the shaking machine is also covered in Chapter 2.

The strains were converted to stresses for comparison with the two analytical methods. Comparisons between the
deflected shapes derived from the finite element results and the prototype experimental results were also made. Chapters 7 and 8 deal with the comparisons drawn.

1.5 SHEAR WALL DESCRIPTION AND TERMINOLOGY

The shear wall tested was the end shear wall of an eight storey reinforced concrete building. The building is shown in Fig. 3.1 on page 100, and the dimensions and the positions of the openings in the shear wall tested are given in Fig. 3.6 on page 106.

By observing the latter figure one can see that the wall is essentially a coupled shear wall above level 2. However an elastic laminae analysis of the wall above this level would be inaccurate as the opening in level 5 is offset and the depths of the coupling beams vary. So too does the thickness of the wall.

This particular shear wall was similar to a steel channel in cross-section, as shown in Fig. 3.6. The terminology used to describe the parts of a channel is also used in conjunction with the shear wall, in that it has two flanges, and a web. The beams connecting the two sides of the wall (above level 2) are termed "spandrel beams".

1.6 THE FRAME/SHEAR WALL INTERACTION

Due to the incompatibility of the deflected shapes of frames and shear walls under the same loading distribution there are obviously "lack of fit" forces involved when a building with both frames and shear walls is subject to lateral loading. A computer analysis involving matrix manipulations was performed (as described in Section 7.16) to calculate the degree to which the frames in the building tested affected the lateral load on the shear wall. Khan and Sbarounis(22) provided a method of finding the frame/shear
wall interaction in a tall building which essentially used a slope deflection method, while Parme (23) has produced a manual method involving a stiffness approach to find the interaction. However, with advances being made in the computer field, the matrix manipulation performed on the flexibility matrices of the frames and shear walls involved will probably become a commonly used method of finding the frame/shear wall interaction. This interaction does play a significant part in frame/shear wall buildings with frames of substantial stiffness.

1.7 THE PROTOTYPE STRESS LEVELS

Since it was obviously impracticable to shake the relatively new and occupied building sufficiently hard to cause high stresses in the shear wall, the comparison of the results between the two analytical methods and the prototype was made strictly on the elastic behaviour. Also the wall was considered to be uncracked in order to form the comparison with the necessarily uncracked mathematical and photoelastic models. Consequently the high strains recorded across some of the cracks in the concrete around the spandrel beam areas were reduced, as described in Section 7.5. The maximum principal stresses calculated from the strains were in the order of 10 lb./sq.inch.

With the prediction of the post-elastic behaviour in structures gaining importance, the possible action of the shear wall under ultimate loading conditions is described in Section 9.7.

1.8 THE PRESENTATION OF THE RESULTS

It was impracticable to give all the results leading to the comparison between the two analytical methods and the prototype, but in order to show how the final comparisons of the principal stress difference were derived the complete
results for floor 3 are given. The stress distribution predicted by the finite element analysis was only given for floor 3 as this involved a mesh of finite elements for the entire wall and then forming a finer mesh to study one inter-storey area in detail.

The comparison of the stress results is presented in two ways - both as points on a graph as seen in Fig. 8.7 on page 202 and as contours of the principal stress difference, as seen in Fig. 8.19 on page 222. The details of the basis of the comparisons drawn are described in Section 5.2.
CHAPTER TWO

SECTION ONE

DEVELOPMENT OF THE DYNAMIC STRAIN PICK-UP

2.1.1 INTRODUCTION

The crux of the problem of obtaining a strain (or stress) pattern for the shear wall as the building was being shaken, was that of developing a portable dynamic strain pick-up. It was obviously impracticable to cover the entire wall with ordinary strain gauges because of:

1. The enormous number involved.
2. The uncertainty of strains measured over a short gauge length due to cracks or large aggregate particles.
3. The low sensitivity of a foil gauge.

From a previous research project*, which involved shaking the same building in order to find natural mode shapes and damping characteristics, the frequency of both the first translational and the first torsional mode was known to lie between 2.5 and 3.0 cycles/second. By numerically differentiating the deflected shape of the shear wall, the maximum bending strain was estimated at approximately one microstrain. From this information and from practical considerations, the rough specification for the dynamic strain pick-up was drawn up as:

1. It had to be completely portable.
2. The gauge length had to be such as to minimise local effects in the reinforced concrete, yet not average out stress concentrations where they occurred legitimately.

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3. It had to have a mechanical strain magnification to amplify the low strains expected.
4. It had to be robust.
5. It had to be temperature stable.
6. It had to be fatigue resistant.
7. Small zero drifts could be tolerated as only the dynamic peak to peak strain signal was required.

2.1.2 THE PRINCIPLE OF THE DYNAMIC STRAIN PICKUP

The principle of operation of the dynamic strain pick-up was similar to that of the "Demec" gauge commonly used to measure deformations in concrete (24). The latter consists of an Invar main beam with two conical gauge points, one fixed at one end and the other pivoting on a knife edge. The pivoting movement is transferred to a 0.0001 inch dial gauge suitably mounted on the beam. The gauge points are located by small drilled stainless steel pads cemented to the structure.

The principle of operation of the dynamic strain pick-up instrument is shown in Fig. 2.1.1. The knife edge in the Demec gauge was replaced by a sliding device, and an electronic displacement transducer replaced the dial gauge. Basically it provided a method for transferring the total movement between two gauge points to the transducer, and in this way the average strain between the two points could be calculated.

In future sections the dynamic strain pick-up instrument will be simply called the "instrument" where the meaning is clear.
FIG. 2.1.1. DIAGRAMMATIC VIEWS OF THE INSTRUMENTS SHOWING THE PRINCIPLE OF OPERATION
2.1.3 THE DISPLACEMENT TRANSDUCER

At this stage the problem of finding a suitable displacement transducer to incorporate in the instrument was met. It had to fulfill the rough specifications where applicable, and also to have a high gain preferably with a linear output. After briefly considering inductive, capacitive and piezoelectric displacement transducers, it was decided to use two semiconductor (piezoresistive) strain gauges mounted on a thin piece of material. This was to be subjected to axial forces when embodied in the instrument. (However, ordinary strain gauges were used during the instrument development stages.)

The gap the displacement transducer spanned - in other words its effective length - was maintained at half an inch throughout the instrument development program. This meant that if the body of the instrument was infinitely rigid, then the mechanical strain magnification was 2 x the gauge length in inches.

2.1.4 THE INSTRUMENT MAGNIFICATION FACTOR

The strain value read from the strain gauges in the displacement transducer was always measured in terms of a gauge factor of 2.00, regardless of the actual factor. (Section 2.2.3 deals with gauge factors). Hence the magnification factor of the instrument was defined as:

\[ M = \frac{\text{Strain value in transducer at } F = 2.0}{\text{Strain level in material being measured by the instrument}} \]

This magnification factor will be used in the next few sections.

2.1.5 THE FIRST DYNAMIC STRAIN PICK-UP

The first instrument was made from mild steel, had a 12" gauge length, and consisted of a rectangular body (2" x 1\(\frac{1}{8}\)"
THE HAND HOLDING FRAME

AND THE CRUDE SUCTION HOLDING DEVICE
with a $\frac{3}{4}$" dia. piston sliding inside it. A displacement 
transducer consisting of two strain gauges on a piece of 30g. 
aluminium, $\frac{1}{4}$" wide, spanned the $\frac{1}{2}$" gap between the body and 
the piston.

Fig. 2.1.2 shows two views of the first instrument. 
The frame with four legs and a handle attached to the instru­
ment as shown in the top photograph was a result of the first 
main feature discovered during the instrument development pro­
gram. This was that the instrument could not be held on the 
gauge pads by hand as the strains due to the operator's hand 
movements were far in excess of any dynamic strains present. 
The purpose of the frame was to provide a steady holding device. 

This instrument proved unsatisfactory as a dynamic 
strain pick-up, but it was useful in that it showed the way 
for future development. It was too heavy, the transducer 
was too stiff, and the piston could rotate slightly in the 
circular cylinder - these were a few of its faults.

The holding frame, although it reduced the effect of 
operator hand movements was far from satisfactory, and a method 
of holding such an instrument on a smooth wall was devised us­
ing a vacuum pad device. A crude vacuum arrangement was made, 
which seemed to work satisfactorily; the vacuum was supplied by 
a pump. This arrangement is shown in the bottom photograph in 
Fig. 2.1.2.

2.1.6 THE SECOND DYNAMIC STRAIN PICK-UP

The second instrument was designed in the light of the 
experience gained from the first one; this time the piston was 
of $\frac{1}{2}$" square steel, sliding inside a body fabricated from 1" x 
$\frac{3}{4}$" aluminium alloy. It was much lighter than the first instru­
ment and the gauge length was reduced to 10". Fig. 2.1.3 
shows the instrument with a new suction holding frame attached.
FIG. 2.1.3. DETAILS OF THE SECOND INSTRUMENT
As soon as the instrument was made, two more problems presented themselves, the first being the unknown frictional and possible damping effects in the oiled piston sliding inside the body. The second problem was more important, however - no provision had been made for small adjustments in the gauge length. If the gauge pads were not accurately positioned, even though a jig were to be used, extremely large strains would be produced in the transducer as the instrument was forced into the gauge pads. To overcome this problem an elongated hole was cut in one end of the Perspex transducer, which allowed the instrument to be fitted on the gauge pads before that end of the transducer was clamped to the body.

The suction holding device had four suction pads, two connected at each end of the instrument by tension springs running to a cross-arm as shown in Fig. 2,1,3. By having springs it meant that even if the surface were not perfectly flat, the feet would still make contact, and since the springs had negligible lateral stiffness it ensured the only movement reaching the displacement transducer was coming from the gauge point via the gauge pad.

In order to make initial contact with the surface, the suction feet were moved down by pressing on two small handles on the instrument. Once suction had been established the handles were released so that only the springs connected the suction feet to the instrument.

The suction device worked well as long as it was stuck to a horizontal surface, but when the instrument was placed on a vertical surface with its body horizontal, the instrument sagged sideways until the springs took up a new position of equilibrium. This problem was overcome in a subsequent suction holding device, by running a light steel cable inside a compression spring. In this way the spring could not elongate
yet the lateral stiffness was not greatly affected.

2.1.7 THE THIRD DYNAMIC STRAIN PICK-UP

A new concept was used in the manufacture of the third instrument. Instead of using a piston to restrict the direction of the movement between the two halves of the instrument, a linkage system was devised. Basically it was as shown in the bottom view of Fig. 2.1.1, with lengths of spring steel glued (with Araldite) rigidly to each half, forming the links. Because of the extremely small strains involved, the vertical movement between the two transducer supports was negligible.

The displacement transducer was modified to allow for small changes in the gauge length. It was a Perspex strip \( \frac{1}{4}'' \times \frac{1}{16}'' \) in section spanning the \( \frac{1}{2}'' \) between two \( \frac{3}{16}'' \times \frac{3}{4}'' \) square end blocks; the Perspex being cemented together with I.C.I. Tensol 7. One end was clamped permanently to the instrument body, while the other end had an elongated hole and an adjustable clamp. A leaf spring device allowed a small bolt to clamp a metal pad over the end block without allowing it to rotate. The third instrument is shown in the top photograph in Fig. 2.1.4, while the bottom two photographs show the transducer and the transducer clamp.

The reason for the large end blocks was to provide a gauge length adjustment without significantly altering the properties of the transducer.

The previously made suction device was transferred to this instrument and tests showed that the magnification factor had dropped in relation to the second instrument. This suggested the body was too flexible, so stiffening plates were cemented to the sides with epoxy resin. Fig. 2.1.4 shows the stiffeners in place on one side, and also an extra leaf spring in the centre in an attempt to stiffen the body against bending.
FIG. 2.1.4. THE THIRD INSTRUMENT
Stiffening the sides increased the magnification factor, and by increasing the base area of the conical gauge points a further increase was obtained. (See Section 2.3.2.) However changes in the clamping position of the transducer affected the magnification factor.

With the experience gained in the manufacture of the three instruments, and by trying small changes within each one, a comprehensive appreciation of the problem had been obtained. With both the initial specifications and the subsequent experience in mind, the fourth and final instrument was designed.

2.1.8 THE FULLY DEVELOPED DYNAMIC STRAIN PICK-UP

2.1.8.1 The Final Design. Due to developments in the electronics for measuring the strain in the displacement transducer, four semiconductor strain gauges were required to be incorporated in the dynamic strain pick-up. This led to the decision to have two displacement transducers in the instrument. The two stiffened halves, still connected with leaf springs, were modified to take the two transducers in such a way that the axis of the strain gauge bearing transducer strip was lying along the centre line of the stiffened body. This meant that there was no strain loss due to the shearing effect that had to transfer strain to the transducer in the previous instrument.

The other major change from the previous design was that small differences in the gauge length were provided for by having an adjustable gauge point; this is discussed in detail in Section 2.3.3. This allowed both ends of the transducers to be rigidly fixed to the body of the instrument.

Fig. 2.1.5 is the working drawing of the final instrument, and Fig. 2.1.6 shows it in its completed form.
Fig. 2.1.5. The working drawing of the final instrument
2.1.8.2 The Suction Holding Device. A new suction holding device was fabricated, having three suction feet as shown in Fig. 2.1.6. The two end feet were connected to the instrument by the previously used tension spring method, while the central foot on the opposite side had the stranded wire inside a compression spring as explained in Section 2.1.6. This longitudinally rigid system prevented any sagging when the instrument was on a vertical surface.

The two end suction feet were pushed down individually to make contact with the surface of the structure under test. To do this a rod with a button on the top and a "crow's foot" beneath was pushed down to force the suction foot against the surface. Three locating pips in the crow's foot fitted into corresponding depressions in the top of the suction foot in order to keep it correctly oriented. When suction had developed under the foot the button was released, allowing a light compression spring to return the rod, thus preventing any strain pick-up via the suction foot. The \( \frac{1}{4} \)" dia. P.V.C. suction tubing leading to the feet, which was looped, was considered to have negligible lateral stiffness. The operation of the suction feet can be seen in the right hand photograph in Fig. 2.1.6.

Some initial trouble was experienced with the suction holding frame in that it was impossible to get the feet to stick to the wall unless all three were held in contact simultaneously. The reason was that too much air was passed through the tubes from the feet not in contact, thus preventing a vacuum building up in the feet in contact. To overcome this, the suction splitter on top of the holding frame (as seen in Fig. 2.1.7) was modified by filling the spigots leading to the two end suction feet with solder and re-drilling them with a No.64 (0.036" dia.) drill. The spigot leading to the central pad (which was the first to make contact)
had a $\frac{3}{32}$" dia. hole. With the leakage thus restricted, the Edwards RB.4 vacuum pump was able to maintain 22 inches of mercury vacuum, thus allowing the two end feet to be pushed down at leisure.

With the suction holding device working satisfactorily, the vacuum system details were consolidated. As previously mentioned, the pump was an Edwards RB.4, a high volume rotary cylinder vacuum pump driven by a $\frac{1}{2}$ H.P. electric motor, capable of 28.2 inches of mercury vacuum with no leakage.

The first detail considered was that of a power or pump motor failure causing loss of vacuum, and allowing the instrument to fall off the wall. To guard against this, a non-return valve was incorporated in the system, together with a $\frac{1}{3}$ cubic foot reservoir; this gave a time lapse of a minute between the pump stopping and the instrument falling off the wall.

Secondly, a method had to be devised to enable the instrument to be released while keeping a vacuum at the pump to ensure it kept working. A Schrader 4 port - 2

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**FIG. 2.1.7. DIAGRAMMATIC LAYOUT OF THE SUCTION CONTROLS**
position air valve was used to do this - the entire vacuum circuit is shown in Fig. 2.1.7.

The only way in which the instrument could fall off the wall was if the P.V.C. hose between the pump and the instrument was accidently trodden on. Precautions against this are described in Section 4.11.

The single suction foot shown attached to the P.V.C. vacuum line in Fig. 2.1.6 and diagrammatically in Fig. 2.1.7 was designed to anchor the lead wires and vacuum line to the wall, thus taking the load off the instrument and providing an extra safeguard against the lead wire or the vacuum line being accidentally pulled.

2.1.8.3 The Trial Instrument. An instrument was constructed along the lines of the final model except that ordinary foil gauges were used in the transducers. These were cemented with Eastman 910 strain gauge cement to two Araldite strips, 1½" long, with a section .25" x .030". These strips were fixed in place by filling the cavities in the body, as shown in the top photograph in Fig. 2.1.9, with Araldite D casting resin; a dam of cellulose tape was provided to retain the Araldite during the casting. One end was cast at a time.

The strain gauges were wired up with fine shielded cable as A.C. (mains) interference had been a problem in the past, despite filtering. The end blocks shown in Fig. 2.1.5 containing the gauge points and carrying the suction holding device were attached to the body of the instrument, and it was ready for testing.

Testing was carried out on the dynamic calibration device which is described in Section 2.1.9.3 and the instrument performed better than expected from previous models. No performance figures have been quoted for the various instruments at different stages of development because:
1. The magnification factor depended on the gauge pads used as well as the instrument itself, and as the former were also being developed, no true comparisons can be drawn.

2. The calibration devices were different in some cases - they will be discussed in Section 2.1.9.

3. The electronic strain measuring system was still being developed. Although direct electrical calibration overcame any comparison difficulties, the early measurements were not as sensitive as later ones.

2.1.8.4 The Construction of the Displacement Transducers.

With the design of the instrument consolidated using foil gauges, it was at last considered prudent to construct transducers containing semiconductor strain gauges. ( Semiconductor gauges are discussed in Section 2.2.6; at this stage it is sufficient to say that the gauges used were manufactured by Baldwin-Lima-Hamilton, and had Bakelite backing with silver lead wires.)

The semiconductor strain gauges were approximately .25" x .40" in size. Because of the difficulty of cementing these small, delicate, and almost irreplaceable strain gauges on to a small strip of Araldite, and then cementing the strip in the body of the instrument, it was decided to try casting the gauges in the strip. This would also overcome waterproofing problems.

A solution seemed to be to make a mould from silicone rubber, position the strain gauges in it, and then fill the mould with Araldite D casting resin. To start with, however, a mould for the mould had to be made, as the silicone rubber used (Dow R.T.V.803) was a viscous liquid which set in about 4 hours when mixed with an accelerator. The first mould was milled from a block of 1" thick Perspex, and the liquid rubber was poured in after being de-aerated with the vacuum pump. A trial casting in the second mould showed that there were no
FIG. 2.1.8. SEQUENCES IN THE CASTING OF A DISPLACEMENT TRANSUDER

TERMINALS IN POSITION

DETAIL OF TERMINALS

DETAIL OF SEMICONDUCTOR STRAIN GAUGE

TOP GAUGE POSITIONED

BOTTOM GAUGE POSITIONED

THE COMPLETED MOULD
problems in casting the transducer, except that in order to
get a good seal around the mould when the Perspex top (see
Fig. 2,1.8) was placed in position, the thickness of the
strip was decreased. Another Perspex mould was made which
produced a rim 3 thou. high around the outside of the transducer in the rubber mould, giving a good seal.

Perspex was used as a seal on top of the silicone rubber
mould as it forms a poor bond with Araldite D.

The actual transducer was cast in the following manner;
this is also shown sequentially in Fig. 2,1.8.

1. The silicone rubber mould was cleaned with soapy
water, and dried.

2. Brass punchings, 1/8" dia. were soldered to pin heads
as shown, and were positioned in the bottom of the mould to
form terminals.

3. The gauges were trimmed to fit in the transducer
strip, and the first gauge was wedged in the bottom of the
mould. The lead wires were soldered to the terminals. The
second gauge was soldered to the terminals and positioned
slightly proud of the silicone rubber mould.

4. The cleaned Perspex top was placed on top of the
rubber mould so that the top gauge was bearing against it,
being held in the correct position in the transducer strip.

5. A 10:1 Araldite D mix was prepared (see Appendix
C) and this was poured into the mould through a funnel
drilled in the 1/2" thick Perspex top. The Araldite slowly
filled the mould, care being taken to ensure no air bubbles
were trapped in the corners. As the liquid ran through
the transducer strip it could be seen to form a thin film
over the topmost gauge; after stripping a similar effect
was found on the bottom gauge.
6. Once the Araldite entered the exhaust vent, the casting was left to cure for 24 hours at room temperature.

7. The mould was disassembled, and the cast transducer was gently pulled from the rubber mould complete with the pins soldered to the terminals. The small nibs in the corner of each end block resulting from the pouring funnel and the exhaust vent were cut with a blade, and the casting was carefully prised from the Perspex. Two days more curing at room temperature were allowed.

8. The pins were unsoldered by applying heat to their shanks, thus subjecting the Araldite casting to as little heat as possible.

2.1.8.5 The Stiffness of the Transducers. The cross section of each transducer was 0.225" x 0.030" with a 1/16" radius fillet in each corner. The strip was half an inch long.

If a strain of one microstrain in the structure is taken to cause 20 microstrains in the transducer through mechanical magnification (which assumes no losses in the instrument or gauge pads) then the force necessary to cause this latter strain in both transducers is 0.135 pounds. (This assumes an elastic modulus of .5 x 10^6 p.s.i. for Araldite.) Since the bonding strength of gauge pads to the wall (see Section 2.3.5) was greater than 30 lb, the stiffness in the transducers was negligible.

2.1.8.6 Assembly of the Transducers. After a minimum curing time of a week at room temperature, the two transducers were cemented in place in the machined cavities in the instrument body. The aluminium alloy was roughened, and degreased, while the transducer end blocks were just cleaned. Where bond was to occur, both contact surfaces were wetted with Araldite D before the transducer was fitted.
FIG 2.1.9. THE INSTRUMENT ASSEMBLY
in place. It was lightly clamped in position by placing a small weight on each end. A small fillet of Araldite was run across the join at the end of the strain gauge carrying strip to ensure no serious stress concentrations occurred and also to shorten the effective length of the transducer, thus increasing the magnification factor. A photoelastic test had been performed on a trial casting of a transducer - this showed that at the centre of the strip, where the semiconductor or strain gauge filaments were located, the stress was uniform. Also no stress concentrations were obvious in the corners due to the \( \frac{1}{16}'' \) radius fillet provided in the casting.

The wiring of the strain gauges was done with fine shielded cable as before, and the cables were held in place on the instrument body by running a bead of Araldite along them. Particular attention was paid to earthing the shield of each cable to the aluminium body to guard against earth loops, and once this was satisfactory the soldered terminal connections were covered with Araldite to provide both mechanical protection and insulation. The output wires were run to a small terminal block on the body of the instrument to provide an easy connection for further leads.

Fig. 2.1.9 shows the transducer in place and the wiring. (The quality of the wiring was improved upon in a subsequent instrument.)

2.1.8.7 The Instrument Body Covers. When the instrument with the semiconductor gauges incorporated in it was initially tested, severe zero drift problems were encountered. This was obviously due to draughts blowing over the transducers, (see Section 2.2.8.10) and to overcome the problem two rectangular covers were fabricated out of light tinned steel sheet; steel was used to provide good electromagnetic shielding as well as mechanical protection and draught exclusion. The covers were attached to the instrument, as shown
in Fig. 2.1.9, and the gap between them covered with plastic adhesive tape. A gap of \( \frac{3}{32} \)" between the inside of the cover and the body of the instrument provided adequate clearance for the wiring, and the five strain gauge leads were passed through holes in one cover to a terminal block on the suction holding device.

2.1.8.8 **Semiconductor Strain Gauge Failure.** Tests on the dynamic calibration device showed the instrument worked well, until one semiconductor strain gauge failed by open-circuiting. The transducer containing this gauge had been the first cast, and it had been cured in an oven at 140°F for a day; this was a possible reason for the failure. Subsequent transducers were cured at room temperature as described.

Another instrument was constructed: using the experience gained from the first attempt a higher standard of construction was attained.

2.1.9 **THE INSTRUMENT CALIBRATION DEVICES**

2.1.9.1 **Preliminary.** In the early instruments crude strain calibration devices were used in which a set deflection was applied to a simply supported beam or cantilever in order to produce small static strains. These calibration devices led to a simple yet effective static calibration beam being constructed.

2.1.9.2 **The Static Strain Calibration Beam.** This was a 6" x 3" x \( \frac{1}{4} \)" R.H.S., 36" long, with a \( \frac{1}{4} \)" dia. steel rod running centrally inside it. On one end of the rod was a thrust bearing; this was set in one of the \( \frac{1}{2} \)" thick steel blocks welded in the ends of the R.H.S. The other end of the rod was attached to a \( \frac{5}{8} \)" dia. screw with 40 threads per inch, with a knurled knob graduated with 250 peripheral divisions. By screwing out this knob the steel rod was put in tension and the steel beam in compression; subsequent cali-
bration of the calibration beam with ordinary strain gauges showed that 10 divisions rotation of the knurled knob produced 315 microstrain.

This particular static calibration device was mainly used during the development work. Its disadvantage was that, due to its static nature, zero drifts in the strain gauges could not be filtered out as they would be with a dynamic calibration device (see Section 2.2.8.5 on filtering).

2.1.9.3 The Dynamic Strain Calibration Beam. Because of the zero drifts and also because the dynamic properties of the instrument had to be evaluated, a dynamic calibration beam was made. Basically this was a 24" length of 6" x 6" x ½" R.H.S., once again with ½" end blocks welded in place. This beam was strained longitudinally with a sinusoidal pull; this pull came from an 8'-10" length of ½" dia. spring steel wire anchored to the centre of one end block. The sinusoidal pull came from the other end of the wire being anchored to a ball race on an eccentric rotating on a shaft. The eccentricity of the centre of the ball race could be adjusted in order to provide a range of strains in the beam. Fig. 2.1.10 shows the eccentric shaft and the wire anchor.

To resist the wire's pull, the other end of the calibration beam was attached to a ½ ton load cell, hence the longitudinal force on the roller mounted beam could be measured. This is also shown in Fig. 2.1.10.

The eccentric shaft, which had a flywheel attached to reduce the accelerations due to the wire being pulled and then pulling on the eccentric during one revolution, was driven by a ½ H.P. electric motor. A coarse speed control was provided by a belt-pulley system, while the exact speed could be obtained by adjusting the frequency of the power supply to the motor. The Civil Engineering Department had a built-in variable voltage and frequency power supply system provided
FIG. 2.1.10 THE DYNAMIC CALIBRATION APPARATUS
for cases such as this, and it was found that 220 volts at 45 Hz, gave the motor speed required to drive the dynamic calibration device at the set frequency. The driving motor is shown in the bottom photograph in Fig. 2,1,10.

The period of the crank rotation was measured by a photocell unit which, when its light beam was broken by a pointer on the flywheel, sent a 7 volt pulse to an Advance Timer-Counter, TC1-A. Since a test on the building had shown that the required shaking frequency was 2.71 cycles per second, the dynamic calibration was done at this frequency.

The photographs and description of the dynamic calibration beam refer to its final form. Like most experimental development it had initial faults. For instance, a Carter Hydraulic Gear was used to provide the correct frequency, but the chain drive on this apparatus produced high frequency vibrations in the wire. The calibration beam and the eccentric shaft were initially mounted on a 12" x 5" U.B., but this proved too flexible. To overcome this problem a separate support for both the shaft and the 6" x 6" beam were made, and these were bolted to a concrete floor to provide complete rigidity.

For good calibration results it was necessary to have some tensile force in the wire even when the crank was closest to the beam. A 1" length of thread was provided on the bolt shank running through the load cell to give an easy method of tightening the wire.

When calibrating the instrument on the dynamic calibration beam, the load cell was connected to a Budd P.350 static strain bridge, and the output of this bridge was led directly to another channel on the pen recorder used to measure the output from the dynamic strain pick-up. The sinusoidal load cell trace was calibrated to give a wire tension value and from this the strain in the calibration
beam could be calculated.

The dynamic calibration beam was calibrated by using the third instrument, whose magnification factor was found on the static calibration beam and calculations showed that a theoretical solution based on the load cell force gave the strain exactly.
Consolidation of the design of the displacement transducer as discussed in the first section of this chapter showed the need for semiconductor (piezoresistive) strain gauges to be incorporated in the Araldite strip of the transducer. This section discusses the development of the electronic systems necessary to measure the dynamic strain output from these strain gauges, and also other electronic systems associated with the testing such as frequency counters and displacement and velocity meters.

2.2.2 THE VARIABLE RESISTANCE STRAIN GAUGE

The variable resistance strain gauge is defined as a device whose electrical resistance changes in proportion to the strain to which it is subjected. In the case of the bonded resistance type strain gauge, the resistive element consists of a length of fine metal wire, a metal foil, or a whisker of semiconductor material. To reduce the length of the gauge, while its sensitivity is retained (by maintaining a relatively large resistance), the wire or foil is usually formed into a grid pattern. The resistive element is fixed to a suitable base, usually paper, plastic, or ceramic.

The term "variable resistance strain gauge" will, in this text, be replaced by "strain gauge", or, where the meaning is clear, simply "gauge". The type of strain gauge will be specified, where necessary, by calling it a semiconductor strain gauge or an ordinary (foil or wire) strain gauge.
2.2.3 THE ORDINARY (FOIL OR WIRE) STRAIN GAUGE

This section describes the properties of the ordinary strain gauge so that ensuing references to them may be understood. The properties of the semiconductor strain gauge will be discussed in Section 2.2.6.

The essential property of the strain gauge is that its resistance varies with strain. This property is defined in terms of the gauge factor. By definition:

\[ F = \frac{dR/R}{s} \]  \hspace{1cm} (2.2.1)

where \( R \) is the gauge resistance and \( s \) the strain.

Thus the gauge factor is essentially a measure of gauge sensitivity.

Another important property of strain gauges is that of temperature compensation. Briefly, this means that if a material expands or contracts with temperature (resulting in no stress change) then a temperature compensated strain gauge can usually be manufactured for that particular material. The strain due to the temperature change in the material to which the gauge is fixed is counteracted by a resistance change in the compensated gauge. Different alloys are used in the grid manufacture to produce the desired changes.

2.2.4 THE WHEATSTONE BRIDGE

The Wheatstone bridge configuration of resistances lends itself to the measurement of small resistance changes, and this is the basis of most strain measurements, although the balancing methods vary.

If a constant voltage power supply of voltage \( E \) (assuming D.C. for the purpose of the following calculations) is applied between terminals \( a \) and \( c \) of a Wheatstone bridge,
as shown in Fig. 2.2.1, the bridge is balanced when $E_{bd} = 0$. It can be shown that in the balanced state:

$$\frac{R_1}{R_2} = \frac{R_4}{R_3} \quad (2.2.2)$$

If a small resistance change $dR_1$ is made to $R_1$, by calculating the potential drops in the circuit it can be shown that:

$$E_{bd} = \frac{-R_2 dR_1}{(R_2 + R_1)^2} E \quad (2.2.3)$$

This equation can be written for small changes in all the resistances, and by summing the expressions we get:

$$E_{bd} = \frac{-R_2 dR_1 + R_1 dR_2}{(R_2 + R_1)^2} + \frac{-R_4 dR_3 + R_3 dR_4}{(R_3 + R_4)^2} E \quad (2.2.4)$$

By substituting equation 2.2.1 into this equation, and if $R_1 = R_2 = R_3 = R_4 = R$, as it is in most strain gauge configurations, it can be shown that:

$$E_{bd} = \frac{F E}{4} \left( -e_1 + e_2 - e_3 + e_4 \right) \quad (2.2.5)$$

Hence the conclusion is that the imbalance of the Wheatstone bridge, $E_{bd}$, is proportional to the sum of the strain (or resistance) changes in opposite arms and proportional to the difference of the strain (or resistance) changes in the adjacent arms.

The preceding algebra refers to the situation where the bridge is balanced or near to balance. However, the Wheatstone bridge configuration gives a nonlinear output of $E_{bd}$ if the strain changes are high. It can be shown that for a strain change of 100 microstrain and a gauge factor of 100 (such as is found in a semiconductor strain gauge) the deviation from linearity is only 1%. Since this strain
FIG. 2.2.1. GENERAL WHEATSTONE BRIDGE

FIG. 2.2.2. ACTUAL STRAIN GAUGE CONFIGURATION

$R_p$ — B.L.H. SPB2-12-12
\[ G_F = 113.9 \quad C'_4 = 3500 \]

$R_N$ — B.L.H. SNB2-06-12-S13
\[ G_F = -1063 \quad C'_2 = 13400 \]
range was much higher than that encountered in the experimental work, the non-linearity of an unbalanced Wheatstone bridge was ignored.

2.2.5 ACTUAL STRAIN GAUGE CONFIGURATION

Although semiconductor strain gauges have not yet been discussed in detail it will be sufficient to say that they have both positive and negative gauge factors. (Strictly speaking semiconductor strain gauges do not have gauge factors, although the parallel drawn will suffice for the following explanation).

In the instrument both transducers have the same state and level of strain (tension or compression) at the same time, and if four ordinary gauges were incorporated in the two transducers, equation 2.2.5 shows that, regardless of the strain, no output can be expected. However, by using positive gauge factor semiconductor gauges on two opposite arms of the Wheatstone bridge and negative gauges on the other two arms equation 2.2.5 performs as if two gauges are in compression and two in tension. This effectively increases the sensitivity of the Wheatstone bridge to four times its value for a single operational strain gauge.

Since the displacement transducers were supposed to operate by producing measurable axial strains it was necessary to cancel out any strains produced by flexural (bending) stresses in the strip of the transducer. (Flexural stresses may have been present due to an eccentric axial loading on the strip, or rotations of the ends of the transducer due to flexure in the two halves of the body of the instrument). To cancel out any flexural stresses (or to average the strain level recorded by each of the two gauges in one transducer) the pair of gauges used in each transducer were of the same type. As previously described, the semiconductor
gauges were embedded in the Araldite strip just below the surface, one on each face. However the two gauges from each strip were wired to the opposite sides of the Wheatstone bridge, thus averaging the strain level. Fig. 2.2.2 shows the actual configuration of the semiconductor strain gauges in the Wheatstone bridge.

2.2.6 SEMICONDUCTOR STRAIN GAUGES

The piezoresistive effect is the name given to the change in electrical resistivity that occurs with the application of stress; the piezoresistive effect of semiconductors is unusually large. The presence of a large "shear coefficient of piezoresistance" \(^{(25)}\) yields high gauge factors and makes these materials useful for strain gauge and transducer applications.

The actual semiconductor strain gauges selected for the transducers were from the Baldwin-Lima-Hamilton \(^{(28)}\) range and were designated:

\[
\begin{align*}
B.L.H. & : SPB2 - 12 - 12 \\
B.L.H. & : SNB2 - 06 - 12 - S13
\end{align*}
\]

the latter being partly temperature compensated (S13). For more adequate temperature compensation in Araldite the gauges would have to be classified as S40, but no such gauges existed in this particular range.

The second letter in the B.L.H. classification refers to the strain sensitivity (positive or negative); the reason for using gauges of both types was discussed in the previous section.

Also in that section it was stated that semiconductor gauges do not have a gauge factor as defined in Section 2.2.3. This is because their resistance changes are a...
non-linear function of strain. Theoretical and experimental work has formulated the expression for the change in absolute resistance as:

\[
\frac{dR}{R_0} = \left[ \frac{T_0}{T} \right] x G_x F'_x x \epsilon + \left[ \frac{T_0}{T} \right]^2 x C'_2 x \epsilon^2 \tag{2.2.6}
\]

where \( G_x F'_x \) and \( C'_2 \) are constants supplied for a particular gauge. The actual values for the gauges used are given in Fig. 2.2.2.

In order to discuss semiconductor strain gauges in terms of a gauge factor, equation 2.2.6 can be rewritten as:

\[
F = \frac{dR}{R} = \left[ \frac{T_0}{T} \right] x G_x F'_x + \left[ \frac{T_0}{T} \right]^2 x C'_2 x \epsilon \tag{2.2.7}
\]

which describes a family of straight lines when \( F \) is plotted against strain, at different temperatures.

### 2.2.7 ELECTRICAL OUTPUT PROPERTIES OF THE TRANSDUCER

By referring to Fig. 2.2.2, equation 2.2.4 can be rewritten as:

\[
E_{bd} = \left[ \frac{R_P \frac{dR_N}{RN}}{(R_P + R_N)^2} + \frac{R_N \frac{dR_P}{R_P}}{(R_P + R_N)^2} \right] x 2E \tag{2.2.8}
\]

Since the decrease in \( R_N \) and increase in \( R_P \) due to a tension strain is approximately equal, then \( (R_P + R_N) \) is essentially constant. For the same reason \( R_N, R_P \) are relatively constant.

Let \( (R_N + R_P)^2 = 4R^2 \)

and \( R_N R_P = R^2 \)

then from equation 2.2.8

\[
\frac{E_{bd}}{E} \times \frac{(R_P + R_N)^2}{2} = -R_P \frac{dR_N}{RN} + R_N \frac{dR_P}{R_P} = R_P R_N \left[ \frac{-dR_N}{R_N} + \frac{dR_P}{R_P} \right] \tag{2.2.9}
\]

and if \( C_N = \frac{R_N}{R_0 N} \)
and \( C_p = \frac{R_{0p}}{R_p} \) where subscript 0 refers to the unstrained resistance, we can write

\[
\frac{2 \cdot E_{bd}}{E} = \frac{-dR_N}{R_{0N}} C_N + \frac{dR_P}{R_{0P}} C_P
\]

(2.2.10)

Since \( F = \frac{dR}{R_0} \epsilon \)

then \( E_{bd} = (-F_N C_N + F_P C_P) \times \frac{\epsilon}{2} \times E \)

(2.2.11)

Equation 2.2.5 reduces to \( E_{bd} = F \cdot E \cdot \epsilon \) since the sign of the strain is counteracted by the positive and negative gauge factors. Hence we can find the representative gauge factor, \( F_R \), for the Wheatstone bridge formed by the four semiconductor strain gauges by equating \( E_{bd} \) as expressed above to equation 2.2.11,

Hence \( F_R = \frac{-F_N C_N + F_P C_P}{2} \)

(2.2.12)

Equation 2.2.7 showed that \( F \) was a linear function of strain for semiconductor strain gauges, and consequently \( F_R \) is a linear function of strain.

Fig. 2.2.3 shows the functions \( F_N, F_P \) and \( F_R \).

It is evident from the plot of \( F_R \) in this figure that it was essential to have no initial stress in the transducer when the instrument was positioned on the gauge pads on the wall, and the adjustable gauge point was tightened. A strain change from 0 to 1000 microstrains is shown to produce a 4% change in gauge factor from the graph of \( F_R \) and only 5.6 lb. axial force on the instrument was necessary to produce this strain; poor positioning of the instrument on the gauge pads and careless clamping of the adjustable gauge point could have produced this. The effect of initial stress in the transducer (obvious by the position
FIG. 2.2.3. SEMICONDUCTOR GAUGE FACTORS

FIG. 2.2.4. PRELIMINARY DYNAMIC STRAIN BRIDGE CIRCUIT
of the coarse bridge balancing control) is discussed fully in Section 2.5.4.

2.2.8 DEVELOPMENT OF A DYNAMIC STRAIN BRIDGE

2.2.8.1 Preliminary. Various electronic firms partly or fully specialising in strain gauge work have developed and marketed static and dynamic strain bridges; most modern static strain bridges can be used as dynamic strain bridges up to a certain frequency limit.

An early decision made in the research project was to amplify the dynamic strain signal from a balanced Wheatstone bridge and feed the resulting signal to a pen recorder (a Philips Oscilloscript type PT2104). This decision was adhered to although an attempt was made to digitise the dynamic strain gauge signal by using a diode clamp amplifier and a digital voltmeter which was linked to a paper tape punch. The electronics associated with this attempt will be discussed in Section 2.2.9.

Since there was no commercial strain bridge available to perform all the functions necessary in the case of this particular research, it was decided to develop a Dynamic Strain Bridge specifically for the purpose of reading small, low frequency, sinusoidal strains. It is shown in its completed form in Fig. A.2 on page A2.

2.2.8.2 Dynamic Strain Bridge Specifications. The specification for a Dynamic Strain Bridge to perform the job of reading many different pairs of gauge pads with a single dynamic strain pick-up instrument was drawn up as both electronic and instrument development progressed and was finalised as:

1. The gain should be high enough to enable very small strains to be measured even when semiconductor strain gauges were used.
2. The output should be virtually free of electrical noise.
3. It should be completely portable.
4. It should have built in strain calibration.
5. Any slow zero drifts during a dynamic strain measurement should be able to be filtered out.
6. It should be linear in amplification.
7. It should be able to be quickly balanced.
8. It should have a variable gain control.

The existing strain bridges in the Civil Engineering Department did not have all of these features and did not lend themselves to easy adaption.

2.2.8.3 Design of the Dynamic Strain Bridge. Although a constant current Wheatstone bridge circuit is recommended for semiconductor strain gauges (25), it was decided to retain the more simple constant voltage method of Wheatstone bridge measurement because of three factors. Firstly, gauge drift did not matter, as it could be filtered out, secondly the strains involved were very low, which meant that the effective gauge factor of the gauges remained constant, and thirdly the output strain signal could be calibrated directly.

The initial circuit* of the Dynamic Strain Bridge is shown in Fig. 2.2.4. From this initial design the development of the Dynamic Strain Bridge progressed with additional circuitry being designed to bring the bridge up to the final specification. The initial development was done using ordinary strain gauges on the instrument transducers, but the final conversion to semiconductor gauges made some

* Provided by Dr. J. Bargh, University of Canterbury.
difference, as described in Section 2.2.8.10.

The final circuit is shown in Fig. 2.2.9 and the following sections deal with different features of the dynamic strain bridge.

2.2.8.4 The Wheatstone Bridge Balancing Circuit

The actual balancing control is shown in both Figs. 2.2.9 and 2.2.5, and Fig. 2.2.5 shows the value of $R_b$, the total balancing resistance in the Wheatstone bridge for different positions of the coarse balance control.

The following calculations show the method used to find the strain range that the bridge balancing control could cope with, and the sensitivity of the balancing control for different strain levels.

Equation 2.2.2 (for the bridge balance) can be re-written, by referring to Fig. 2.2.2 as:

$$\frac{R + dR_N + dR_b}{R + dR_P + R_b - dR_b} = \frac{R + dR_P}{R + dR_N} \quad (2.2.14)$$

where $R$ is the backed resistance of the semi-conductor gauges (126 $\Omega$) and $dR_N$ and $dR_P$ are the resistance changes in the negative and positive gauges respectively for a set strain.

Now $dR_N = R_{0N} N$ and $dR_P = R_{0P} P$ where $N$ and $P$ are the values inside the square brackets in equation 2.2.6 when applied to the negative and positive gauges respectively, and $R_0$ is the unbacked gauge resistance. If $R$ is used in place of $R_0$ then equation 2.2.14 can be re-written as:

$$\frac{R(1 + N) + dR_b}{R(1 + P) + R_b - R_b} = \frac{(1 + P)}{(1 + N)} \quad (2.2.15)$$
FIG. 2.2.5. BALANCE CONTROL POSITION V. TOTAL BALANCE RESISTANCE

FIG. 2.2.6. STRAIN RANGE COVERED BY BALANCE CONTROL
or \[ \frac{R(N_1) + dR_b}{R(P_1) + R_b - dR_b} = \frac{P_1}{N_1} \]  

which reduces to

\[ \frac{dR_b}{R_b} = \frac{R}{R_b} \left( \frac{P_1 - N_1}{P_1 + N_1} \right) \]

From the graph of \( \frac{dR_b}{R_b} \) and \( R_b \) (Fig. 2.2.5) a trial and error process gave the position of the coarse balancing control for different static strain levels in the Araldite strip of the transducer as shown in Fig. 2.2.6.

2.2.8.5 The Amplifier Circuit. The basic component of the Dynamic Strain Bridge was a "Nexus S.Q.10a" all silicon solid state operational amplifier. The general description of the module (29) said it had a high gain and high stability. The input circuitry of the module was fully protected against damage due to accidental connection of the input terminals across the power supply, and the output circuitry was also protected against short circuits to ground. The amplifier was primarily designed for closed loop operation with external negative feedback.

The gain of the operational amplifier was set by the ratio of the feedback resistance (\( R_2 \)) to the input resistance (\( R_1 \)) as seen in Fig. 2.2.4. Basing calculations on this ratio, the gain produced by the circuitry used in the Dynamic Strain Bridge could be varied between 100 and 1100.

The two capacitors in the circuit shown in Fig. 2.2.4 acted as filters. The 0.01 \( \mu \)F capacitor in parallel with \( R_2 \) was the high pass filter designed to chop frequencies above 10Hz. (at minimum gain). The capacitor effectively decreased the total impedance of the feedback path for higher frequency voltages and hence they were not amplified with the same gain. Thus any noise or electrical inter-
ference from the semiconductor strain gauges or the amplifier was eliminated.

The 1.0 μF capacitor in series with the output of the amplifier was the low pass filter which only allowed frequencies higher than 1 Hz, to pass through to the pen recorder. In this way slow strain gauge zero drifts were eliminated during a recording. When the strain signal was being calibrated the low pass filter was by-passed, as shown in Fig. 2.2.4.

The designed performance of the filtering system was experimentally evaluated by feeding the input of the amplifier with a sinusoidal voltage from a signal generator and a graph of voltage output versus frequency was obtained — see Fig. 2.2.7. From the graph it was apparent that the filtering system worked as anticipated, the most important feature being the performance of the low pass filter as severe zero drifts were encountered with the semiconductor strain gauges. The reasons for the high drifts are discussed in Section 2.2.8.10.

2.2.8.6 Characteristics of the Nexus Amplifier.

In the light of the previous discussion it is possible to list some of the relevant characteristics of the Nexus operational amplifier (29) and comment on their significance.

1. The rated output of the module was 11.5 volts maximum, with a maximum current output of 50 mA. This was ample to power the pen recorder, and no problems of the amplifier limiting were met.

2. The open loop gain of the amplifier was 100,000, but 1000 was never exceeded.

3. The noise voltage was 1 micro-volt peak to peak at the maximum amplification and bandwidth used. This was negligible.
4. The input impedance of the module was 3M Ω. However the input impedance as seen by the Wheatstone bridge was 1000 Ω (R_i). It was necessary to maintain this value to prevent a current leak of any magnitude from the Wheatstone bridge.

5. The output impedance of the amplifier was 50 Ω, which was of no consequence as the input impedance of the pen recorder was 1M Ω.

6. Power requirements for the module were ±15 volts with the common terminal earthed. Under full output the maximum current drawn was 15 mA. Dry cells were used.

7. The amplifier had to be trimmed. This meant that with the input earthed, there must be no output voltage. A 50 K Ω variable resistance was connected across the appropriate terminals; this took the form of a 5K Ω potentiometer and a 50K Ω preset potentiometer, the former acting as a fine control.

2.2.8.7 The Meter. To enable small output voltages to be detected when trimming the amplifier or when balancing the Wheatstone bridge, a sensitive meter was incorporated in the output circuit. A 0 - 100 μA meter was modified by adjusting its helical springs to read ±50 μA. As the meter's internal resistance was 1K Ω it effectively read ± 50 mV.

The sensitivity of the meter could be reduced by a variable resistance in series with it, and meter protection against high currents was afforded by crossing its terminals with two 0A,91 diodes (in opposite polarity). The diodes needed a small voltage before they would conduct, cutting in at meter full scale deflection.
2.2.8.8 Calibration of the Wheatstone Bridge

A compressive strain in a strain gauge results in a decrease in its resistance. Such an effect can be simulated by switching in a large resistance in parallel with the strain gauge. The resulting small decrease in resistance across the gauge can be measured on a static strain bridge in order to relate it to a strain change, and then used to calibrate subsequent strain measurements where there is no calibrated meter or digital readout to provide a direct measurement.

The principle can be expanded to include a range of calibrating resistances, (and hence a number of different strains) by using a rotary switch, as shown in the calibration circuit in Fig. 2.2.9.

By using the formula for resistances in parallel it can be shown that for a gauge factor of 2.0 and a gauge resistance of 120 \( \Omega \), the value of the calibration resistance to simulate a certain strain is:

\[
R_{\text{cal}} = \frac{60}{\varepsilon} \text{ ohms}
\]

(2.2.18)

In this way a series of resistors to cover the strain range anticipated was selected and a trial run on a normal static strain bridge calibrated the eleven calibration resistors (at a gauge factor of 2.00). The effective strains ranged between 70 and 2125 microstrains.

The calibration circuitry in the Dynamic Strain bridge took the form of a 12 position rotary switch and a push-button switch. A suitable calibration strain was selected on the rotary switch and the circuit was completed by pressing the button. Hence the calibration signal was maintained as long as the button was pressed, and in this way a series of square waves could be applied to the paper trace - their height being the value of the preset calibra-
tion strain. A typical calibration trace is shown in Fig. 7.2 on page 158. This calibration method meant that, as long as no amplification settings were changed between the dynamic strain reading and the calibration run, the dynamic strain signal could be expressed directly as a strain by comparing the respective signal heights on the paper trace. This made the recording of actual amplification settings on the Bridge and pen recorder unnecessary.

2.2.8.9 The Constant Voltage Supply. The constant voltage supply (E in previous calculations) was initially supplied by a 1.5V dry cell across terminals 2 and 3 (see Fig. 2.2.2). During an instrument calibration test it was noticed that the square waves, which resulted when the calibration circuit was operated, had their normally sharp leading edges rounded off. This was traced back to the fact that once the dry cell was partly drained, its voltage varied with small changes in its current output. The absolute voltage level was not critical as the direct calibration method was independent of the amplification of the whole system, but the bridge voltage varying during a dynamic strain or calibration reading could not be tolerated. (Initially it was thought that the small resistance changes involved in the Wheatstone bridge would not affect the dry cell's voltage).

To overcome the problem a regulated power supply to provide 2.0V D.C. from a mains/6.3V transformer was designed*. Two power supplies were constructed, (the reason for two is discussed in the next section) and it was found that the transformer, even though electromagnetically shielded, induced too much A.C. hum into the circuitry. The transformers were replaced by 6V batteries

* Mr. M. Maginness, University of Canterbury.
(positive and negative) and the output of both regulated power supplies was regulated at 2.25 volts. The increase in bridge voltage effectively increased the sensitivity of the system in the ratio of 2.25/1.5.

2.2.8.10 The Use of the Two Regulated Power Supplies

When the Dynamic Strain Bridge had been partly developed it was found that the four semiconductor strain gauges produced severe zero drifts, particularly when the voltage was first applied across the Wheatstone bridge. This appeared to be due to the heating effect of a current running through a resistance (strain gauge). Once a thermal equilibrium had been established, the drift was much less severe and could easily be controlled by the low pass filter. The semiconductor strain gauges were particularly susceptible to drift because firstly, they were not temperature compensated for application on Araldite, and secondly the Araldite provided a very poor heat sink. This was aggravated by the fact that the gauges were embedded in the transducer. Very small temperature changes were involved to cause this drift, as a slight draught blowing over the transducer gave uncontrollable drifts.

Since the calibration circuit was a separate Wheatstone bridge of 4×120 Ω wire wound resistances it was necessary to switch the constant voltage bridge supply (E) from strain gauge circuit to the calibration circuit so that exactly the same bridge voltage was caused in the calibration. This meant that during the time calibration was being carried out, no current was flowing through the semiconductor gauges, resulting in a relative cooling down. In order to retain a relatively stable thermal equilibrium in the transducer during calibration (and trimming) the second regulated power supply was run through the Wheatstone bridge formed by the gauges - the principle
FIG. 2.2.7 VERIFICATION OF FILTER PERFORMANCE

FIG. 2.2.8 METHOD OF MAINTAINING THERMAL EQUILIBRIUM IN THE BRIDGES
is shown in Fig. 2.2.8. This was accomplished by using a 6 pole, 3 position rotary switch in the record-calibrate-trim circuit, as shown in Fig. 2.2.9.

2.2.9 THE DIGITAL READOUT TRIAL

The Civil Engineering Department possesses an EDAC Data Logger, which is essentially a digital voltmeter linked to an electric typewriter and paper tape punch, plus having logic for sequential readings. The device was primarily designed for reading strain gauges at one every 1.3 seconds, but provision was made for direct connection to the digital voltmeter.

The output voltage from the Dynamic Bridge during a strain reading, known to be sinusoidal at 2.71 c.p.s., could not be fed directly into the data logger as it had no means of picking the peak voltage. A diode clamp amplifier was designed and constructed, once again using a Nexus operational amplifier. Essentially the circuitry stored the peak amplitude of a voltage in a capacitor - hence the digital voltmeter was able to measure the apparently static voltage across the capacitor. To store the peak voltage, the amplified signal from the Dynamic Strain Bridge was passed through a diode to a 1000 µF electrolytic capacitor. Once several data logger readings across the capacitor terminals had been made the capacitor was earthed, and the process repeated for the opposite polarity. In this way the two data logger readings gave the total amplitude of the sine wave, without an accurate bridge balance being necessary.

The system worked extremely well when the sinusoidal input to the diode clamp amplifier was supplied by a signal generator. However the drift due to the semiconductor strain gauges could not be coped with by the low pass filter
in the Dynamic Bridge as the input impedance of the diode clamp amplifier was too low. As it was impossible to accurately record the amplitude of the drifting sine wave the project was abandoned, although an active filtering system could have eliminated the drift. This main reason for ceasing work on a digital strain read-out system was reinforced by the facts that:

1. The separate recordings and calibration signals could not be easily identified on the paper tape.

2. The sign of the strain could not be found from one reading. It would have been necessary to have a pen recorder to compare the strain reading with the photocell signal from the shaking machine to find its sign.

3. The digital method provided no way of visually checking the performance of the system, whereas any peculiarities could be easily seen on a paper trace.

4. The data logger was very cumbersome, being 8'-0" long and 6'-0" high.

The lesson learnt from this exercise was that more sophisticated data recording systems in experimental research are not necessarily an advantage unless there is some fast, independent way of checking each reading.

2.2.10 VELOCITY AND DISPLACEMENT METERS

Since it was necessary to have a reference to ensure the shear wall was being strained at the same overall level during the test, it was decided to continuously monitor the transverse displacement and velocity at the top of the building during the test.

The horizontal displacement meter had been designed specifically for a previous building shaking project. It was essentially an inverted pendulum inside a can filled
with silicone oil, as shown in the centre of Fig. 2.2.10. The displacement between the can body and the pendulum was recorded by a Philips PR9310 Inductive Pick-up. The natural frequency of the pendulum was 3.0 c.p.s. at 60% critical damping. The output of the horizontal displacement meter was temperature dependent due to viscosity changes in the oil, and the phase angle was unreliable because of the small difference between the shaking frequency and the meter's natural frequency. The output signal from this meter was amplified by a Philips PR9304 Dynamic Strain Bridge, and then fed into one of the pen recorder channels.

To supplement the horizontal displacement meter, the signal from a Willmore velocity meter was fed into another pen recorder channel. Since the frequency remained constant the velocity could be taken directly as an amplitude measurement. This recording was considered much more reliable than the former, and it was used to correct the strain readings for changing building amplitude. The Willmore velocity meter is also shown in Fig. 2.2.10.

Both meters were used, however, in case of malfunction of one of them, and to provide an easy way of checking the sign of the strain signal.

2.2.11 FREQUENCY MEASUREMENT

A photocell unit, which provided a 7 volt pulse every time its light beam was broken, was fed into an Advance Timer Counter, type TC1A, and this measured the period of the shaking machine. This equipment was mentioned in Section 2.1.9.3, and the Timer Counter is shown in Fig. 2.2.10.

The pulse from the photocell unit was also fed
FIG. 2.2.10. ANCILLARY ELECTRONICS USED

- Horizontal displacement meter
- "Willmore" velocity meter
- "Budd" static strain bridge
- Philips dynamic strain bridge
- "Advance" timer - counter
directly into a channel of the pen recorder to provide a phase reference. The sign of the dynamic strains recorded was calculated from this signal on the paper trace, which can be seen on the left hand trace in Fig. 7,2 on page 158.

2.2.12 THE PEN RECORDER

This piece of equipment was a Philips PT2104 Oscilloscript shown on the trolley in Fig. 4.2 on page 114. It had four separate recording channels equipped with D.C. amplifiers which allowed both static and dynamic processes to be recorded. The input impedance of each channel was 1 MΩ and the maximum pen sensitivity was 3 millivolts (peak to peak) per millimeter. A built in vibration circuit enabled a 400 Hz signal to be applied to the pens to overcome the paper friction when measuring slowly changing voltages.

The trace on the paper was formed by a steel pen forcing the moving paper against a more slowly moving carbon paper giving a dry, instantly visible trace. A synchronous motor provided the correct paper speed; this speed could be regulated by inserting different gear wheels in a drive chain.

With the particular pens used the width of the recording was 40 mm (+20 mm) and the frequency range was 0 to 160 Hz. The linearity of each channel was checked by applying a set voltage increment at different positions across the channel and no abnormalities could be found.

In general, this pen recorder was a very versatile piece of equipment. The light beam type recorders investigated had an advantage in that they provided a wider trace, but they had a low input impedance and the output trace needed developing. The Oscilloscript was a remarkably sturdy instrument, and gave little trouble even with
the prolonged use during the instrument and electronic development, and the actual test.

2.2.13 **THE FINAL RECORDING SET-UP**

To enable fast readings to be taken during the prototype test, a frame was constructed to support the Dynamic Strain Bridge on top of the pen recorder. The control panel of one channel was lifted out of the pen recorder, placed in a cabinet, and was positioned in the frame alongside the Bridge; this arrangement can be seen in Fig. 2.2.11.

The output of the Dynamic Strain Bridge was plugged directly into this pen recorder channel, with the pen centering and amplification controls alongside the Dynamic Strain Bridge controls. The paper feed control on the pen recorder was just below the channel controls, and with this arrangement little operator movement was required. This was an important point as far as error-free recordings over a length of time were concerned. In addition the Dynamic Strain Bridge controls had been laid out with this point in mind.
FIG. 2.2.11. THE ARRANGEMENT OF THE DYNAMIC STRAIN BRIDGE AND THE PEN RECORDER.
CHAPTER TWO

SECTION THREE

GAUGE PAD DEVELOPMENT AND MASS PRODUCTION

2.3.1 INTRODUCTION

The principle of operation of the gauge pads and the associated gauge points, as defined in Section 2.1.2, is to transfer the relative movement between a pair of gauge pads to the body of the instrument. In this section the development of the gauge points and the gauge pads is traced, and the technique used to mass produce the developed gauge pads is described. The method used to attach the pads to the shear wall is also discussed.

2.3.2 THE EARLY DEVELOPMENTS

The first instrument, as described in Section 2.1.5, had conical gauge points with a 60° taper which fitted into a $\frac{1}{8}$" dia. hole drilled in a half inch square of steel, $\frac{1}{8}$" thick. However this particular system was very susceptible to any rocking of the instrument on the gauge pads, which resulted in large strains in the transducer completely unconnected with any strains present in the structure under test.

Hanson and Kurvits (30) found that by attaching $\frac{1}{8}$" dia. steel balls to the tips of the conical points of Demec gauges a marked improvement in the consistency of reading was obtained. Consequently $\frac{1}{8}$" dia. ball bearings were soldered to recessed ends of the gauge points as shown in Fig. 2.1.2 on page 18, and this solved the problem of the instrument rocking sideways on the pads. The $\frac{1}{8}$" dia. balls fitted into $\frac{7}{64}$" dia. holes in the pads.
Instrument development was rapidly proceeding so the next major change in the design of the gauge points occurred in the third instrument. As mentioned in Section 2.1.7 the base area of the conical gauge point was increased by using a 45° cone which resulted in a stiffer gauge point and a higher magnification factor. An even broader base was constructed to hold the gauge pads, and a slight increase in sensitivity was noticed, which amplified the fact that gauge points should be as stiff as possible to provide as little eccentricity of loading to the body of the instrument as possible.

When the third instrument had been fully developed, a variety of different gauge pads were tried. Some types used an interference fit principle where the \( \frac{1}{8}\) in. ball was forced into a slightly smaller hole in a soft material. The materials used were Perspex, where the complete gauge pad was made from it, and solder, where a \( \frac{3}{16}\) in. insert was melted into a hole in an original square steel pad. Other types made use of the circular ball sitting on the rim of a circular hole a little less than its own diameter or sitting inside a tapered hole; these trials used steel gauge pads. Magnetic gauge points on steel pads were even tried.

The highest magnification factor was obtained from the \( 0.125\) in. balls on the gauge points being pushed into \( 0.120\) in. holes in solder inserts in steel pads. However, repeated use of the gauge pads permanently deformed the solder, which altered the magnification factor.

2.3.3 THE FINAL DESIGN OF THE GAUGE PADS AND POINTS

In Section 2.1.8 it was described how small changes in the gauge length of the instrument were provided for by having an adjustable gauge point. The final design of the gauge point is shown in Fig. 2.1.5 on page 25. Basically it was a very flat cone carrying a \( \frac{3}{16}\) in. ball bearing soldered in its centre. The rim of the cone was clamped
hard against the surface ground base of the end block of the instrument when the knurled screw, as shown in Fig. 2.1.6 on page 26 was tightened. Because the cone was flat, and only touched the ground steel surface at its periphery it was very rigid. By using a 3/16" dia. ball a larger hole could be drilled more accurately (as far as diameter was concerned) in the gauge pad, and by having a larger bearing area the local deformations formerly due to high contact stresses between the ball and the hole were reduced.

The final design of the gauge pads used a principle that had features of both the interference type fit, and the ball sitting on the rim of a hole.

A steel punching, 1/8" thick and 1/2" dia. was drilled centrally with an 11/64" dia. drill almost completely through. This was followed by a No.13 drill (0.185" dia. or 2.5 thou. less than the diameter of the ball), and then a 3/16" dia. ball was placed on the rim of the hole and given a light tap with a hammer. This effectively formed the rim of the hole into a curved circular seat to receive the ball on the gauge point. The advantage of forming the seat in this way was twofold. Firstly, it removed any small burrs or irregularities on the rim of the hole, and secondly, the elastic deformation caused the formed seat to be slightly smaller than the ball on the gauge point, so that a slight interference fit was obtained.

However, even without the interference fit effect, the fact that the diameter of the hole was only 1.3% smaller than the ball meant that there was a high angle of incidence (80°) between the ball and the rim of the hole. Thus the ball was effectively wedged in the hole.

It was most important to ensure the hole was never oversize; this was the reason for the two drillings. Since ordinary twist drills tend to cut oversize the hole was
drilled to a nominal 0.172" dia. (\(\frac{11}{64}\)" dia.), and when the No.13 drill was used it acted more as a reamer, only cutting away up to 6.5 thou of steel off the sides of the hole. In this way it was unlikely to cut oversize, and it had the added advantage of cutting away the small burrs invariably left on the rim of the hole by the first drilling. A check against an oversize hole was made when the seat was formed with the \(\frac{3}{16}\)" dia. ball; it was easy to see if it fitted in too far. Tests showed that this design of gauge pad gave magnification factors in excess of the infilled solder pads, and it was decided to put them into mass production for the impending test on the building.

2.3.4 THE MASS PRODUCTION OF THE GAUGE PADS

Fig. 2.3.1 diagrammatically shows the steps taken in the production of a gauge pad, and also shows some of the equipment used in the different processes.

2.3.4.1 The Grinding Jig. About 3000 \(\frac{1}{2}\)" dia. mild steel punchings were purchased and since they were punched from \(\frac{1}{8}\)" plate they were slightly convex with a rough base. In order to get a flat rim on the base to enable accurate holding of the pad during the drilling and reaming process, and also to give good adhesion to the plastered surface of the shear wall, each gauge pad was ground. This was done by holding the gauge pad in a jig which was able to slide on runners. These runners were mounted on a grinding machine parallel to the axis of rotation of the wheel; in this way the base was skimmed by coming into contact with the flat side of the carborundum wheel.

The pad was held in the jig by a vacuum system, using the Edwards RB.4 Pump. When the jig was pulled away from the wheel along its runners a spring loaded pin on the back of the jig hit a stop, and forced the ground gauge pad out. The top of the convex surface was also ground flat to prevent
FIG. 2.3.1 STAGES IN THE MASS PRODUCTION OF GAUGE PADS
the $\frac{11}{64}$" dia. drill wandering when the gauge pads were drilled.

The grinding jig is shown in Fig. 2.3.1.

2.3.4.2 Tumbling. Sharp edges and burrs on the gauge pads tended to cause the feed of the drilling jig (as described in the next section) to block, and also caused poor seating in the locating holes in the jig. To remove these, about 300 gauge pads at a time were tumbled in a rotating drum with some coarse sand for 18 hours.

2.3.4.3 The Drilling Jig. Two general views of the jig are shown in Fig. 2.3.2 and a more detailed photograph in Fig. 2.3.3.

This jig was designed with a rotary table, and its operation can best be described by referring to Fig. 2.3.3. The rotary table, a 6" dia. plate cut from $\frac{1}{2}$" thick steel, was mounted on a 6" x 3" channel 12" long, which could be clamped to the table of a drill press. Four recesses were milled in the rotating table, $\frac{1}{2}$" dia. by $\frac{3}{32}$" deep, and in the centre of each recess a $\frac{3}{8}$" dia. hole went right through the table. The table could be locked in position by a $\frac{1}{8}$" dia. spring loaded pin set beneath the gauge pad drilling position (as specified by the No.2 position in Fig. 2.3.3). When the table was required to be rotated anticlockwise through 90° the Schrader push button air valve was operated, and a compressed air piston withdrew the spring loaded locating pin to enable the knurled wheel set above the table to be turned. By having four positions available four separate operations could be carried out, as follows:

Position 1. This is the position at the rear of the photograph. The vertical tube with a hole in its side near the base was filled with 70 gauge pads, and was clamped in position in its holder. This effectively held a pile of gauge pads over the path of the $\frac{1}{2}$" dia. recesses in the
FIG. 2.3.2. GENERAL VIEWS OF THE GAUGE PAD DRILLING JIG
FIG. 2.3.3. DETAILS OF THE DRILLING JIG
rotary table, and as the table was rotated each recess was filled by a gauge pad.

Position 2. The locating pin held the recess, and hence the gauge pad centrally beneath the drill bit, and by operating the Schrader two position-four port valve, the compressed air piston and lever system seen at the left of Fig. 2.3.3 forced the steel ring down to clamp the gauge pad to await drilling. The depth of the drilled hole was controlled by a stop on the drill press, and an oil-water lubricant was automatically fed to the bit, every time it was brought down, from a tank mounted on the drill press. This tank can be seen in the central photograph in Fig. 2.3.1.

Position 3. The air supply to the gauge pad clamping system was released by operating the two position Schrader air valve, which was then moved into the air blast cleaning and eject position. (The table was not rotated at this stage.) A previously drilled gauge pad, now in position 3, was ejected by an air blast which came up through the $\frac{3}{8}$" dia. hole beneath the $\frac{1}{2}$" dia. recess. The pad was blown up into, and along the 1" dia. tube seen in the front right of Fig. 2.3.3, and from here it dropped into a container. A portion of the air blast was led away by the curved tube in the centre of the figure to blow any swarf away from the drilled gauge pad in position 2. Remaining swarf was ejected with the gauge pad into the 1" tube.

Position 4. With the air blast cleaning and ejecting going on, the U-shaped pipe in position 4 was blowing the empty recess clear of any remaining swarf and lubricant prior to it receiving a fresh gauge pad in the next position.

Once the pad was ejected and the swarf cleared the Schrader valve was returned to neutral and the table was rotated.
An air pressure of 80 psi was used to operate the drilling jig.

The drilling operation itself was perfected by a trial and error process. It was found that even when the top of the convex gauge pad was ground flat (as described in Section 2.3.4.1) the drill still tended to wander as it was impracticable to centre punch each gauge pad. By fitting a $\frac{3}{4}$" dia. steel collar over the drill, leaving only $\frac{1}{4}$" of the bit showing, the drill was stiffened sufficiently to drill a central hole in the pad, providing the initial cut in the pad was made by holding the drill (rotating at about 1000 r.p.m.) lightly on the surface of the pad. The stiffened drill can be seen lying on the drill press table in Fig. 2.3.1.

Once the gauge pads had been initially drilled, the reaming out to 0.185" dia. was done with the No.13 drill held with only about $\frac{1}{4}$" of its shank in the chuck. This provided a very flexible drill which allowed it to position itself centrally in the $\frac{1}{16}$" dia. hole, even if the latter was slightly off centre.

Strict attention was paid to the sharpening of the drills; after each sharpening about a dozen trial holes were drilled in a piece of mild steel to check that the size of the steel spiral emerging from each flute of the drill was the same. This indicated that the point of the drill was in the centre of the tip.

2.3.4.4 The Seat Forming Process. In order to tap the seat forming $\frac{3}{16}$" steel ball into the 0.185" dia. hole in the pads with a constant force, a compressed air operated piston was made up, as shown in Fig. 2.3.1. The piston had air leakage paths cut in its sides, so that if a constant air supply was held on the piston it reached an equilibrium position on its returning springs without remaining in contact with the rod carrying the steel ball.
However, when a foot operated valve was pressed the initial momentum of the piston provided enough travel to give the end of the rod a sharp tap and by regulating the compressed air supply to 65 psi, the desired effect was obtained.

The seat forming process was as follows. The rod carrying the \( \frac{3}{16} \)" ball was positioned in the hole in the gauge pad as shown in Fig. 2.3.1, and the foot operated button was pushed once. The ball, which was by now wedged in the pad, was removed by grasping the washer welded to the rod and twisting the pad free. An experienced operator could easily detect any defective gauge pads by the effort required to remove the ball bearing from the pad after the seat forming process.

2.3.5 THE GAUGE POINT FIXING TECHNIQUE

Various methods were tried to cement the gauge pads to the plastered, painted surface of the shear wall without harming its finish, but none were satisfactory. Finally it was decided to use sealing wax and to repair the finish on the wall after the test.

After the seat forming process, the gauge pads were heated and a small drop of sealing wax was applied to the base on each pad. When the time came to attach the pads to the wall a square inch of paint was scraped off at each pad position, and a smear of wax was melted on to the bare plaster using a portable butane gas torch to provide the heat, as shown in Fig. 2.3.4. The apex pad in a triangle of pads was fixed to the plaster by holding the pad in place with a light rod in its central hole and applying heat to the edges of the pad. Once the wax had melted, and the base of the pad could be felt to be in contact with the plaster, the heat was removed, and the wax allowed to set.
THE TRIANGULAR JIG

THE PORTABLE GAS TORCH

FIG. 2.3.4. THE APPARATUS FOR THE APPLICATION OF THE GAUGE PADS
The two other pads of the triangle were wedged on the \( \frac{3}{16} \)" ball bearings on the locating points of the triangular jig as seen in Fig. 2.3.4. (These balls also provided a cross-check on any defective gauge pads.) Once the level bubble on the triangular jig, which ensured the correct orientation of the triangles, was central, the heat was applied and the two pads completing the triangle were fixed to the wall.

2.3.6 THE REASON FOR THE TRIANGLES

In order to calculate the principal strains and their orientation at a point at least three strain measurements must be taken at that point. Rosette strain gauge configurations (24) usually use the equiangular, or delta rosette, or two grids at 90° with an intermediate grid at 45°.

The triangle of gauge pads represented an equiangular rosette strain gauge and the calculations to convert the three strains to stresses are given in Fig. 7.6 on page 173. However, the main reason for using triangles of gauge points was to enable the clusters of triangles to be formed (by simply building on one initial triangle) around the highly stressed areas of the shear wall. An example of the clusters of gauge pads can be seen in Fig. 4.4 on page 121.
CHAPTER TWO

SECTION FOUR

THE SHAKING MACHINE

2.4.1 INTRODUCTION

In Section 1.4.3, the need for a machine to excite the building in order to laterally load the shear wall with the inertia of the building was discussed. This section briefly describes the design and construction of the shaking machine and the operation of its variable speed control.

2.4.2 THE CALIFORNIA INSTITUTE OF TECHNOLOGY SHAKING MACHINE

The basic design of the shaking machine used in this research project was based on the machine designed at the California Institute of Technology. Hudson (31), in his description of that shaking machine, gave a comprehensive review of previous machines and the simple theory behind the production of a force from a rotating eccentric weight.

The Californian machine produced a uni-directional sinusoidally varying force of about 1000 lb. at 1 cycle per second with a maximum force limit of 5000 lb. To achieve this, the machine had two rotors, counter-rotating on a vertical shaft; these rotors had recesses in which up to four layers of lead segments, three per layer, could be fitted in order to provide a range of shaking forces at the same frequency. The rotors were chain driven by a variable speed electric motor.

This machine was considered particularly suitable for the proposed test, but it was not feasible to construct an identical machine at the University of Canterbury because
of the lack of certain machining facilities, and also because of the difficulty in obtaining all the necessary materials. Hence a machine was designed, incorporating the features of the Californian machine yet compatible with the materials, facilities, and finance available.

Fig. 2.4.1 shows a photograph of the complete shaking machine, while Fig. 2.4.2 shows the working drawing of the machine.

2.4.3 THE DESIGN OF THE ROTORS

Hudson (31) showed that the optimum sector angle for a rotating mass formed by a segment of a circle was 120°; the rotors were designed to this angle. They were constructed from mild steel and cut away to provide as little eccentric weight as possible for high frequency operation yet they were strong enough to carry the lead weights. A rotor can be seen in Fig. 2.4.2 with a 5" length of 5" dia. Asab bar forming the hub of the rotor which carried the Timken 39412 tapered roller bearing cups.

Steel was used for the rotors rather than the aluminium alloy as in the Californian machine, as no casting facilities were readily available. Also, as only two rotors were required, this did not warrant making a pattern. After the rotors had been fabricated they were annealed to relieve any welding stresses, and the final cut of the bearing housing in the hub was then made.

Two 5\(\frac{7}{8}\)" dia. threaded holes were cut in the back of the rotor hubs to enable counter-weights to be attached, should rotor speeds greater than 10 cycles per second be required. The empty rotors gave the maximum force of 5000 lb. at that speed.

The vertical shaft carrying the rotors was almost identical to the shaft in the Californian machine.
FIG. 2.4.1. THE SHAKING MACHINE
FIG. 2.4.2. THE SHAKING MACHINE — WORKING DRAWING
2.4.4 THE DESIGN OF THE FRAME

Although the rotors were similar to those in the Californian machine the frame of the shaking machine was entirely different. It was constructed from 4" x 3" x 0.192" rectangular hollow section (R.H.S.) and the design stress in the steel was limited to 10,000 lb/sq.inch to guard against a fatigue failure. All the welded joints in the frame were reinforced with plates.

The frame was Tee shaped in plan, as shown in Figures 2.4.1 and 2.4.2. The top of the Tee was designed to resist the unidirectional horizontal force that occurred when the rotors were aligned, while the stalk of the Tee was designed to resist the torque produced by the rotors when they were horizontally opposed, as they had lines of action 5.25" apart. The removable block shown in Fig. 2.4.2 was necessary to enable the rotors, when assembled on the shaft, to be fitted into the frame, (as shown in Fig. B1 on page B.2.) After the block was pressed on to the top of the vertical shaft, three 5/8" dia. high tensile steel tie rods were screwed into threaded anchors in the block. By torquing the nuts on the ends of the rods to 175 lb.ft. an estimated tensile force of 3000 lb. was put into each rod. This had a prestressing action on the top members of the 4" x 3" frame, and effectively held the gap between the frame and the removable block closed.

2.4.5 THE DRIVING MECHANISM

The Californian machine used chains to drive the rotors, and to provide a means of obtaining counter-rotation from the same drive shaft rather than gears, as gears tend to produce high frequency vibrations which could be detrimental to some building tests.
By referring to the top right photograph in Fig. B1 on page B2 the principle of driving one rotor off the back of a sprocket to obtain the counter-rotation can be seen. An idler sprocket was used to hold the chain in position. The shaft rotated in sealed deep groove ball races in housings machined to hold them in the sub-frame. However, a simpler method of holding the shaft would have been to use plummer blocks bolted to the frame as the only vertical load on the bearings was the weight of the shaft and the sprockets keyed to it.

Tension in the chains to the rotors was adjusted in two ways. Firstly, the sub-frame holding the drive shaft could be moved sideways on elongated holes. Secondly, the idler sprocket had a half inch adjustment.

2.4.6 THE DRIVING MOTOR

In order to provide a variable speed drive a direct current, shunt wound, electric motor was used. This 7 H.P. motor (much more powerful than necessary) weighed about 200 lb. and was held on the machine by another sub-frame (also shown in Fig. B1 on page B2). The tension in the chain between the motor and the drive shaft was adjusted by moving the motor on elongated holes in its sub-frame. The motor and its control unit is discussed in Section 2.4.11.

2.4.7 THE HOLDING DOWN FRAME

The holding down frame is shown in different views in Fig. B1. It was constructed from 4" x 3" x 0.192" R.H.S. and had \( \frac{3}{4} \)" square sections welded across its base at 12" centres to provide a good key to the structure when a mortar mix was forced between the frame and the concrete roof slab. Initially the frame took the form of a symmetrical cross in plan, but as sufficient space was available it was ext-
ended by adding the end triangular pieces to provide a restraint against lateral movement to the top member of the shaking machine frame. The modified frame had provision for eight holding down bolts.

The shaking machine was held in the holding down frame by U-bolts and the lateral movement was prevented by tightening $\frac{1}{4}$" dia. bolts in the holding down frame against the shaking machine; these bolts were centrally in line with the top and bottom members of the machine's frame. At the base they forced against $\frac{1}{2}$" steel plates while at the top they forced against extension pieces screwed to the ends of the tie rods. These bolts and the U-bolts can also be seen in Fig. B1.

2.4.8 THE CASTING OF THE ROTOR WEIGHTS

The 1" thick lead segments were cast in a steel mould. The larger central segments weighed 33 lb. each, while the smaller triangular side segments weighed 23 lbs. A half inch threaded hole was drilled at their centroids to enable a lifting eye to be screwed in for easy handling.

2.4.9 THE FORCE OUTPUT OF THE MACHINE

After the rotors had been fitted on their shaft, the eccentricity of their weight was measured by balancing them with a known weight, the main shaft being held horizontally. The small difference between them was corrected by screwing on a compensating weight to one rotor.

The eccentricity of the lead weights was measured from the centre of the rotor hub when they were in position.

Fig. 2.4.3 shows the various force-frequency curves for different combinations of weights in the rotors. The combinations of rotor weights shown were chosen because
\[ f = \sqrt{\frac{\text{Force} \times g}{\text{Wr} \times (2\pi)^2}} \]

**ECCENTRICITY (W.r)**

- EMPTY BASKETS: 39.3 ft.lb.
- 1 LAYER - SIDE WEIGHTS: 60.3 ft.lb. (1S)
- 1 LAYER - CENTRE WEIGHTS: 55.0 ft.lb. (1C)

**FIG. 2.4.3. SHAKING MACHINE PERFORMANCE**
they gave the most regular spacing to the curves. Also, in this figure, the eccentric weights of the individual rotating components are given (these values are for both rotors).

2.4.10 PRECAUTIONS AGAINST OVERLOADING THE MACHINE

Since it was not easy to incorporate a mechanical or electronic speed limiting device, a series of large cards was compiled giving the maximum frequency permissible for each combination of weights, and the machine operators were trained to keep within the limits imposed. Also, the motor was geared so that at its maximum speed a machine speed of 7 cycles per second could not be exceeded. An emergency stop switch was also built into the motor control circuitry, as described in Section 2.4.11.

2.4.11 THE MOTOR AND ITS CONTROL UNIT

The direct current motor, as described in Section 2.4.6, was originally part of a textile knitting machine, and as it had shunt characteristics it tended to remain at constant speed over a wide load range.

The varying voltage supplied to the motor, which regulated its speed, was provided by a D.C. generator. This was driven by a three phase mains powered electric motor, and the varying voltage output of the generator was controlled by varying the current flowing through the generator field. A constant D.C. potential was provided by four SQ.18 diodes (which replaced a full wave gas diode rectifier) and the potential across the generator shunt field was controlled by five rheostats in series. Fig. 2.4.4 shows a schematic diagram of the circuit.
The control unit from the textile machine incorporating the motor/generator and the circuitry to provide the D.C. potential to the generator field, was extensively modified to suit its new function. The five rheostats, originally in parallel, were wired in series, and a switch was incorporated to operate the contactor shown in Fig. 2.4.4. This started and stopped the D.C. motor. A small remote control box was constructed, containing one rheostat (the one with the lowest resistance) as a fine speed control, a 0 to 15 amp. meter to record the current in the motor armature circuit, and a 0 to 1 amp. meter to record the current flowing through the diodes. An emergency off switch was also incorporated in the remote control box; when this was operated the contactor was opened, allowing the driving motor to stop under the
friction in the shaking machine and the wind resistance of the rotors.

Fig. 2.4.5 shows the remote control box, and two views of the control unit.

The operation of the control unit is described in Appendix B.

2.4.12 THE PORTABILITY OF THE SHAKING MACHINE

The entire shaking machine could be handled by one person when broken down into its component parts, the heaviest being the frame which weighed about 130 lb. However the drive motor and the control unit were both very heavy, the latter needing four men to lift it. Fortunately, no trouble was experienced in transporting the parts to the Chemistry Block fan room for assembly.

2.4.13 THE ASSEMBLY OF THE SHAKING MACHINE

The assembly of the shaking machine is described in detail in Appendix B, while the fixing of the holding down frame to the concrete floor slab is discussed in Section 4.2.
FIG. 2.4.5. THE SHAKING MACHINE CONTROLS
2.5.1 INTRODUCTION

The final dynamic strain pick-up was put through a series of exhaustive tests before the building test to evaluate the performance of both it and the associated Dynamic Strain Bridge.

2.5.2 THE LINEARITY OF THE OUTPUT

In this, the first and most fundamental test of the series, the dynamic strain reading from the instrument was compared with the dynamic strain in the calibration beam; this was measured as described in Section 2.1.9.3. The readings were performed over a range of strains and the results are shown in Fig. 2.5.1. From this figure it can be seen that the instrument did give a linear strain output over the range considered, and also from this graph the magnification factor was derived from the slope of the line. It was set at \( M = 2000 \), as defined in Section 2.1.4. The pair of gauge pads used during the test was picked at random from the pads being manufactured at the time, and were attached to the steel beam with sealing wax.

There is a slight scatter in the points through which the straight line has been drawn. This was probably due to an accumulative error caused by the reading of four trace heights from the pen recorder paper. These were necessary to plot each point, and each trace height could normally be read only to within 2\%. 

2.5.3 THE OFF-ON TEST

The effect of slight differences in the placing of the instrument on the gauge pads was evaluated: during this test the dynamic strain in the calibration beam, the frequency, the ambient temperature and the gauge pads remained unchanged. The instrument was taken off and replaced on the calibration beam twenty successive times, care being taken not to put any prestress in the Araldite transducer strips. The position of the coarse balancing control was also recorded in terms of the percentage of its total available clockwise travel, (as in Fig. 2.2.5 on page 53).

The results showed a total variation in the height of the strain output trace of 0.6 mm in 34 mm, or less than 2%. Special care was taken in the reading of the trace heights to ensure accuracy.

2.5.4 THE EFFECT OF PRESTRESS IN THE DISPLACEMENT TRANSDUCERS

With the strain linearity and off-on tests showing an unexpected reliability of dynamic strain readings, a test was performed to evaluate the effect of the changing gauge factor of the semiconductor strain gauges. In section 2.2.7 the theoretical output properties of the transducer were discussed, and it was shown how the gauge factor of a representative gauge in the transducer varied with strain level.

In this test the dynamic strain in the calibration beam, the ambient temperature, the frequency and the gauge pads were unchanged. Varying amounts of deliberate prestress were put into the transducers by pushing or pulling on the instrument before the adjustable gauge point was clamped.

As in the previous test, the position of the coarse
FIG. 2.5.1 PERFORMANCE OF THE DYNAMIC STRAIN PICK-UP

FIG. 2.5.2. THE EFFECT OF PRESTRESS IN THE DISPLACEMENT TRANSDUCERS
balance control was noted, and the height of the sinusoidal trace was recorded.

Fig. 2.5.2 shows the results obtained, the vertical scale being in terms of the sine wave height in millimeters. The curve shown in this plot shows no similarity to the shape of the curve in Fig. 2.2.6 on page 53, which gives the theoretical static strain levels, (and hence a measure of the relative gauge factor) for different balance control positions.

However, this plot of points showed how important it was to ensure that as little as possible prestress was put into the transducers during each reading. In fact, during the actual test, if a balance could not be obtained with less than $\pm 10\%$ rotation of the coarse balance control from the unstrained position, then the instrument was taken off and repositioned on the gauge pads. From the curve in Fig. 2.5.2 it is shown that the 20% balancing zone allowed gave a maximum strain output difference of 6%.

2.5.5 TEMPERATURE EFFECTS

This test to determine the effect of temperature on the performance of the instrument involved taking a series of readings on the dynamic calibration beam as the ambient temperature was varied from 50°F to 70°F during the course of a day. No change in the magnification factor could be detected.

2.5.6 GAUGE PAD REPEATABILITY

This particular test was not performed until after the building shaking test, but excellent results were obtained. A pair of used gauge pads from the shear wall were attached to the surface of the steel dynamic calibration beam with sealing wax (using the jig to maintain a
constant gauge length). The frequency, dynamic strain level and the temperature were kept constant during the test.

After each dynamic strain reading each alternate gauge pad was replaced, so that in all thirty different pairs of pads were tested. By only replacing one pad at a time it was feasible that two consecutive low readings would indicate a faulty gauge pad. However, all the readings were within \( \pm 4.4\% \), and the average deviation from the mean was 1.93\%. The coefficient of variation \((32)\) was 2.25\%.

2.5.7 THE OVERALL PERFORMANCE OF THE INSTRUMENT

From the preceding tests it is obvious that the two main reasons for errors occurring during the dynamic strain measurements were due to prestress in the transducers and slight discrepancies between different pairs of gauge pads. The former gave an error of up to \( \pm 3\% \), while the error due to the latter was as high as \( \pm 4.4\% \). Hence the maximum error likely to be encountered during a normal strain measurement could be as high as \( \pm 7.4\% \). However the probability of this maximum error occurring was not very high due to three factors. Firstly, the bridge was usually balanced within the range of the balance control allowed on Fig. 7.5.2, and secondly the average error \((32)\) due to different gauge pads was \( \pm 2.25\% \). Thirdly, the possibility of one factor giving an increase in the strain reading while the other gave a decrease meant that the total error was actually decreased below the magnitude of the individual errors due to each of the two factors.
CHAPTER THREE

THE STRUCTURE

3.1 INTRODUCTION

The building tested was the Chemistry/Physics Building of the University of Canterbury's Science Complex, shown in the top photograph in Fig. 3.1.

3.2 SUBDIVISION OF THE BUILDING

By referring to the typical floor plan of the building, shown in Fig. 3.2 it can be seen that the 3" seismic gaps separating the Chemistry and Physics Blocks from the Tower Block effectively divided the building into three separate structures. The symmetrical Tower Block was entirely a shear wall structure, which contained four lifts, two staircases and ablutions.

The lateral loading on both the Chemistry and Physics Blocks was resisted by both frames and shear walls in the transverse direction and by frame action alone in the longitudinal direction (although the infilled frame panels could be considered as a number of slender coupled shear walls). Above foundation level both Blocks were structurally similar except that the Chemistry Block was longer.

3.3 THE DESIGN AND CONSTRUCTION OF THE BUILDING

The building was designed by the New Zealand Ministry of Works and was constructed by a New Zealand contractor, being completed late in 1966. The entire structure was made of cast insitu reinforced concrete requiring no orthodox construction techniques. The only precast concrete units used were the non-structural exterior wall panels between and below the windows as shown in the bottom left
FIG. 3.1 THE BUILDING TESTED
photograph in Fig. 3.1; and the decorative vertical beams shown in the right hand photograph.

The foundations under the Chemistry and Physics Blocks were of a simple bearing pad type, while the Tower Block was supported by a deeper cassion type foundation. The main reason for this was to give emergency lift over-run.

Although the building was subject to a lot of criticism due to the lack of aesthetic and functional imagination used in its conception, it was ideal for the particular experimental test performed on it. Its regular structural nature made a theoretical analysis relatively easy and the fact that one side of a complete shear wall was almost completely exposed and accessible made it possible to obtain a stress distribution. Since the building was owned by the University it was possible for permission to test the building to be obtained; not many owners of multi-storey buildings would be willing to face the inconvenience of such a structural test.

The building had eight floor levels; these are shown clearly in Fig. 3.3. In the text the terms "level X" and "floor X" both refer to the interstorey area at level X.

3.4 THE STRUCTURAL DETAILS

3.4.1 General. The eight storey building had an overall width of 49’-0" and an interstorey height of 12’-6" giving a total height of 100’-0" measured from the ground floor. (This height does not include the penthouse seen in Fig. 3.1. This penthouse, known as the "fan room" contained the ventilation fans servicing the numerous laboratories and fume cupboards in the Chemistry Block. This steel framed structure had no effect on the main structure as far as lateral load resistance was concerned.)
FIG. 3.2. INTERSTOREY PLAN OF BUILDING

FIG. 3.3. FRAME DETAILS

FIG. 3.4. EFFECTIVE STRUCTURE

BUILDING PIVOTS ABOUT SEISMIC GAP

ASSUMED DEFLECTED SHAPE (ALONG BUILDING)

TOTAL LOAD = \((105 + 22 + 23 + 24 + 25 + 26) / 26\)

LOAD ON ONE FRAME = 4.03 x LOAD ON ONE FRAME

EFFECTIVE STIFFNESS MATRIX FOR FRAMES

= \((105 + 22 + 23 + 24 + 25 + 27) / 26\)

= 5.05 x THAT OF ONE FRAME
The Chemistry Block had a basement 6'-6" in height containing various pumps and air conditioning units for the combined laboratory services, while the Physics Block had approximately 3'-6" of gravel between the foundation pad and the ground floor.

The building frames were at 9'-0" centres: the Chemistry Block had 27 bays and four shear walls and the Physics Block had 21 bays and three shear walls. The shear walls all had various corridor opening sizes and positions, and at least one opening per level, but they all had the same thicknesses - 12" in the basement, 10" for the first four levels, and 8" for the top four.

Fig. 3.2 shows the layout of the frames and shear walls in the building.

3.4.2 The Frames. The building frames were regular for their entire height and the frame details are shown in Fig. 3.3. The main reason for the massive, closely spaced frames was that the entire gravity load was carried by the exterior columns (in the absence of a shear wall) and the load was mainly due to the weight of the 5½" monolithic reinforced concrete floor although the building was designed for a live load of 60 lb./sq.ft. on the floor slabs (33). An interior system of two way beams and columns would have given a more efficient vertical load carrying structure, but the columns would have been detrimental in the large laboratory areas in the lower levels.

3.4.3 The Seismic Gap. The two 3" seismic gaps separating the Tower from the Chemistry and Physics Blocks were covered by a 6" wide brass strip screwed to the wooden tiles on the floor. Before the test the screws on one side of the strip covering the gap between the Chemistry
Block and the Tower were removed to minimise the transfer­ence of movement to the remainder of the building. The only other direct links across this gap were gas and water pipes which had been installed with flexible couplings. The function of the seismic gaps was to allow the individual structures with their different dynamic response characteristics to move freely under an earthquake loading.

During the test it was discovered that substantial coupling occurred across the seismic gap; this was made evident by the top of the end of the Physics Block moving about half as much as the Chemistry building. Direct coupling across the gap could have been due to friction between the brass strips and the floor and to the various pipes. Since the shaking force was only 2,400 lb. these reasons are quite valid. However, another reason for the continued movement in the Physics Block was the fact that a seismic gap could not exist in the soil mass forming the building's foundations; hence the movement could be transferred through the soil.

Although the seismic gap seemed to have almost no effect on preventing coupling between the three structures during the test this would not be the case under earthquake loading. The frictional coupling effect would have a constant level; this would become almost negligible in a severe earthquake, and the non-linearities in the load-deflection properties of the soil would tend to reduce the coupling effect.

3.4.4 Shear Wall Details. The shear wall tested was Wall 1 (see Fig. 3.2), the end wall in the Chemistry Block. The area between the wall and the end of the building (which was formed by a glass curtain wall with vertical decorative concrete columns outside) took
FIG. 3.5. THE WORKING DRAWING OF THE WALL
OPENING SIZES

LARGE — 8.56 ft. high x 5.28 ft. wide.
SMALL — 6.85 ft. high x 5.28 ft. wide.
BASEMENT — 5.33 ft. square.

FIG. 36 KEY
DIMENSIONS OF
THE SHEAR WALL

SECTIONAL PLAN OF WALL
the form of a large landing servicing a staircase to the right of the door, and miscellaneous rooms to the left of the door. The wall was fully accessible on one side except for the areas covered by the false ceiling, the partition walls forming the rooms, and a services duct on the left flange of the wall. Above the doorways (level 3 to level 8) the panels forming the false ceilings were removed and the bare concrete was plastered smooth to enable strain readings to be taken in the critical spandrel beam areas.

The shear wall is shown in the right hand photograph in Fig. 3.1 and the Ministry of Works working drawing of the wall is presented in Fig. 3.5. The key dimensions of the wall are given in Fig. 3.6 and the inaccessible areas on a typical interstorey area are shown in Fig. 4.4 on page 121.

Very few buildings would have internal shear walls with the accessibility afforded by this particular one and this feature itself proved to be of considerable worth when testing the structure. Even the staircase offered little obstruction as it was not connected to the wall itself, but was supported between the landing and the shear wall flange on a reinforced concrete beam; a gap of 2" existed between the wall and the treads. Another important feature of the wall was that it had approximately \( \frac{1}{2} \)" of smooth plaster on each face which provided the necessary flat surface for the suction feet of the instrument.

### 3.5 FRAME ANALYSIS

Since the theoretical shear wall-frame interaction under the calculated inertia loading on the building was required it was necessary to calculate the flexibility matrix for the eight storey frame. The plane frame computer program, described in Section 6.3 was used.
The idealised frame was considered completely fixed at level one, the distance between the columns was 49'0" and the interstorey height was 12'6". The sectional properties of the members were calculated for an uncracked concrete section, taking the full 9'0" width of the bay as being effective. A load of 0.5 kips was applied to each joint at every level, one level at a time and the flexibility matrix was thus compiled.

3.6 LOADING ON THE BUILDING

To calculate the inertia load on the building, as mentioned in the previous section, it was necessary to estimate the mass at each floor level. The dead load or self weight of the structure was estimated by totalling the volume of concrete and taking 150 lb./cu.ft. as its density. This was done for the shear wall and for a representative frame which was considered to incorporate a 9'0" wide section of the building. The effective live loading due to the contents of the building was taken from the N.Z. Standard Building Code as 20 lb. per square foot of floor area. (A quick survey of the building suggested that 15 lb./sq.ft. would have been more accurate, but if this theoretical load analysis had to simulate a section of a design procedure, it was necessary to use the first figure.)

The shear wall zone of the building was not considered to carry any live load as the staircase area was completely bare and the rooms in this area contained very little furniture.

The inertia load on the building was calculated for the shear wall and for a representative frame using the lateral accelerations found from the experimentally
derived deflected shape. The inertia load on each floor level was then doubled because in the prototype test the peak to peak dynamic strain readings were taken.

The inertia loads on the shear wall and a frame are shown in Table 7.10 on page 183.

3.7 THE EFFECTIVE STRUCTURE

From the previous test on the building it was found that in the first torsional mode the entire building swung about the central tower block; very little transverse movement was recorded at the Chemistry Block seismic gap. If a linearly increasing movement along the building from the gap is considered, then the inertia load on the structure can be scaled accordingly.

By referring to Fig. 3.2 it can be seen that there were 10 bays between walls 1 and 2. Hence, within the shaded region in Fig. 3.2 there were 5½ frames (including the end frame (27)) and there were 4½ frames loaded. (The loading on the half bay between frame 25 and wall 1 was included directly in the shear wall loading.) The frame loading is shown in detail in Fig. 3.4.

If the reference deflection, which was measured at the shear wall, is factored for the different frames then it is shown, in Fig. 3.4, that 4.03 frames were effectively loaded with the full inertia load. By a similar process the effective stiffness matrix of the 5½ frames was shown to actually apply to 5.05 frames.

This means that two effective structures existed. By considering walls 1 and 2 to have a similar stiffness the physical effective structure was one wall plus 5½ frames, as shown in an isometric view in Fig. 3.7. However, by allowing for the decreasing amplitude along the
FIG. 3.7  THE PHYSICAL EFFECTIVE STRUCTURE
building due to the torsional mode then the mathematically effective structure was one wall plus 5.05 frames, 4.03 of which were loaded. This mathematically effective structure is used in the matrix manipulations to find the theoretical frame-shear wall interaction under the semi-theoretical inertia load. This process is described in Section 7.16.
CHAPTER FOUR

THE PROTOTYPE TEST

4.1 INTRODUCTION

This chapter deals with the preparations leading up to the building test and the ensuing strain measurements on the shear wall while the building was being shaken in its first torsional mode.

4.2 INSTALLATION OF THE SHAKING MACHINE

After the initial test of the shaking machine it was completely dismantled, checked for any signs of malfunctioning (none were found) and the parts were taken to the corner in the fan room directly above the shear wall. The concrete floor slab in this area was examined with an ex-army mine detector to find the position of the reinforcing steel and the positions of the holding down bolts were marked out. Terrier bolts (self fixing anchorages for use in concrete) were installed and a stiff sand/cement mortar mix was prepared. The holding down frame was lightly held in position by screwing 5/8" dia. bolts into the anchorages and the mortar was forced into the gap between the sub-frame and the concrete slab, the excess being used to form a 1" fillet around the frame. A week later the bolts were tightened.

The shaking machine was assembled in the holding down frame and given a trial run with one layer of weights in the matrices - everything was found to work satisfactorily. Fig. 4.1 shows the machine in position on the building.
FIG. 4.1 THE SHAKING MACHINE IN POSITION IN THE FAN ROOM
4.3 **COMMUNICATIONS**

The existing internal telephone system was used, with extension 'phones being run to the shaking machine operator and the testing personnel.

4.4 **PORTABILITY OF EQUIPMENT**

A four wheeled trolley with leaf spring suspension was built to carry the Philips pen recorder and a cabinet containing the vacuum pump, its reservoir, and the Dynamic Strain Bridge. A locker in the cabinet housed the dynamic strain pick-up. Other equipment used during the test was carried on a smaller trolley. Fig. 4.2 shows the pen recorder and the cabinet on the trolley.

4.5 **EARLY TESTING**

A trial test was carried out with about a dozen triangular groups of gauge pads cemented to the wall on level 1. The rotors on the shaking machine contained two layers of weights each. A graph of strain level versus shaking machine frequency was plotted, the strain being measured on a pair of gauge pads positioned close to the shear wall flange on the staircase side of the building in the vertical direction. The frequency range was varied from 2.5 to 3.7 cycles per second, and the resulting curve is shown in Fig. 4.3. The frequency of the first torsional mode was found to be 2.71 cycles per second, and the frequency range allowed during the shaking machine operation is shown in Fig. 4.3.

This initial test gave an idea of the magnitude of the strains involved, and showed some of the difficulties likely to be encountered. The need for steady, portable scaffolding was apparent especially in the staircase area, the development of a methodical system for taking the read-
FIG. 4.3. DYNAMIC RESPONSE CHARACTERISTICS OF BUILDING
ings was necessary, and it was obvious that the test would progress much more smoothly if both the instrument operator and the pen recorder—Dynamic Bridge operator were comfortable and had to change position as little as possible.

Shortly after this test some glass chemistry apparatus was found to be cracked—apparently due to the shaking. As a result of subsequent discussions with the Department of Chemistry it was decided that the test had to be completely carried out in the space of one week.

This new time factor substantially changed the testing routine which was to have been a process of consolidating a testing technique through trial runs, and steadily working up the building on successive nights. Now complete preparations had to be made prior to the testing date, with most of the equipment duplicated should any unit fail. Any overlooked factor was capable of stopping the test, and with the short time available, such a delay may have proved disastrous to the project.

One advantage to the continuous test was that conditions would remain more constant, such as the internal weight distribution in the building and the ground water level.

4.6 THE SCAFFOLDING

The scaffolding was specifically designed as a four plank platform which was suspended from the staircase beam. Trestles were made from slotted steel angle to enable the areas on the wall above the floor slab to be reached using two of the existing planks.

4.7 ANCILLARY ELECTRONICS

The horizontal displacement meter and the Willmore velocity meter were both positioned in the opposite corner
of the fan room to the shaking machine (still directly above the shear wall). The signals from these two meters, and also the 7 volt pulse from the photocell unit on the shaking machine were carried to the pen recorder through shielded cables hung down the staircase well alongside the shear wall.

4.8 THE SHAKING MODE AND FORCE

Previous tests on the building had shown that the first torsional mode gave the largest displacement to the shear wall under test, and also had the lowest frequency. The shaking machine was positioned above the end shear wall in order to excite this mode predominantly.

Fig. 4.3, as described in Section 4.5, shows the local increase in strain level due to the first translational mode at 3.02 cycles per second; the strain level is 73% of the level due to the first torsional mode.

Higher modes were not tried for the following reasons:

1. The shaking machine was limited to 10 cycles per second without counterweights.

2. The filtering system in the Dynamic Strain Bridge would have had to have been modified – this would mean that high frequency electrical noise would be harder to control.

3. The inertia of the instrument during higher frequency readings might have had an effect.

4. The most important reason was that of the actual deflected shape of the shear wall. If there were one or more nodes in the deflected shape then there would be one or more points of contraflexure. This would, from simple statics, result in the bending moment changing sign at these points, and also lead the maximum and minimum points
on the bending moment diagram. These would give areas on
the shear wall in which there was no shear and would in
fact reduce the total shear force acting on the wall, as
it would be distributed as both a positive and a negative
shear force on the structure.

Since the shear wall under test was only twice as
high as it was wide, its shear force resistance was more
important than its bending moment resistance, particularly
as it had substantial flanges, so the mode which gave the
maximum shear force was chosen - the first torsional mode,
obvious from Fig. 4.3.

The shaking force was limited not by the output of
the machine, but by the amplitude of vibration, the Chemis-
try Department staff were prepared to tolerate. This was
2400 lb., due to two complete layers of weights in the
machine's rotors, as shown in Fig. 2.4.3 on page 89.

4.9 GAUGE PAD APPLICATION

A technician and the author began fixing the gauge
pads to the shear wall about two weeks before the test
started; the gauge pads were being drilled and punched by
another technician to provide a constant supply. This was
done in order to allow as little time as possible to elapse
between their manufacture and the actual test to prevent
any accumulation of rust or dirt on the critical surfaces.
(However, test pads on the wall had shown no ill effect
over a two month period.)

The positions of the triangular groups of gauge pads
forming the regular pattern over the majority of the wall
were marked out using a cloth tape and a chalked string.
Measurements were taken from the floor slab and the inside
face of the shear wall flange next to the staircase. The
apex pad of each triangle was fixed in place with wax and
the other two positioned with the jig. This technique was described in Section 2.3.5.

In the case of the clusters of gauge pads around the openings in the shear wall, the first triangle in a row of pads was fixed in place and the cluster was built up, one pad at a time. The Perspex plate attached to the jig as seen in Fig. 2.3.4 on page 80 had the positions of the instrument suction feet and body scribed on it so that the gauge pads could be placed in order to miss local obstructions, such as light switches.

The bare concrete above the false ceiling in the spandrel beam areas was plastered to provide a smooth surface for gauge pad application and also for the suction feet of the instrument.

The time taken to apply approximately 280 gauge pads to each storey was about seven hours.

As each floor was completed, the individual gauge pads were identified by consecutive numbers written on adhesive paper labels. In the case of the triangular groups of pads, each group was also given a number. Lists of gauge pads were prepared to ensure that all the readings possible were taken during the test, and these lists were also used by a research assistant during the test to ensure the numbers recorded on the paper trace of each strain reading were actually the numbers referring to the pair of gauge pads read.

Fig. 4.4 shows the positions of the gauge pads for floor 3. The important feature to note in this figure is the deviation from the horizontal rows of triangles (groups) across the wall when the staircase area is met. In order to keep a regular vertical spacing in this area, the relative positions of the triangles along a vertical line had
FIG. 4.4. THE POSITIONS OF THE GAUGE PADS AND THE NAMING OF THE VARIOUS ROWS OF PADS ON A TYPICAL FLOOR.
to be staggered to enable obstructions such as the staircase beam and the handrail to be missed. However the full triangles shown in this figure represent the positions of stress levels interpolated from the stress distribution curves up the vertical lines concerned.

Unfortunately only one side of the shear wall could be tested. The other side was mainly concealed with permanent fixtures such as laboratory benches, shelves and cupboards. This meant that any stress differences between the two faces at a corresponding position due to possible bending within the web of the wall could not be measured. This bending effect was probably not very large due to the lateral stiffening effect of the flanges and the floor slabs on the web of the wall, but measurements on two faces would have provided more reliable strain readings.

4.10 AMENITIES DURING THE TEST

Particular care was taken with the scaffolding to ensure it was at a comfortable height for the instrument operator to work from; and for reading gauge pads close to the floor he was provided with a small platform on castors on which to sit. Provision for frequent rests from the arduous task of taking continuous readings was made to avoid errors being made due to tiredness and lack of concentration.

4.11 DUPLICATION AND PROTECTION OF EQUIPMENT

All of the electronic equipment used during the test was duplicated so that the short time available for the test could not be jeopardised by breakdowns. The Dynamic Strain Bridge was not duplicated, but a full range of spare parts was available. A spare transducer for the Dynamic Strain Pick-up instrument was cast and cured should both semiconductor strain gauges in a transducer fail and
hence need replacement.

If one gauge failed, a calibration run would have to be performed on the instrument which would be effectively working on \( \frac{3}{4} \) sensitivity. In fact, if all but one gauge failed the instrument could still be used, providing it was recalibrated each time. Failure of the semiconductor strain gauges was considered to be a serious possibility as a previous instrument had two of the four gauges fail after they had been subjected to a relatively short period on the dynamic calibration beam. Although it was stated(28) that "B.L.H. semiconductor strain gauges have a superior fatigue life" no figures were given to qualify this statement. However, as discussed in Section 2.1.8.8 the failure was probably due to too high a curing temperature being used for the Araldite transducer.

Fortunately the semiconductor strain gauges gave no trouble during the test.

The only electronic equipment to fail was the recently acquired Advance Timer-Counter (TC.5) which did so just before the test began. The previously used TC.1A was substituted and gave no trouble.

Precautions against the instrument falling off the wall were also taken. In the case of pump stoppage the precautions are detailed in Section 2.1.8.2. However an accidental loss of vacuum due to the Schrader vacuum control knob being inadvertently operated or the vacuum hose being trodden on resulted in an instant release of the suction feet. A cord was run across each storey just below the false ceiling, and a light chain was hung from a jockey wheel running on the cord. A cord attached to the independent suction foot on the vacuum hose was hooked on to the chain, so that if the instrument did fall off the wall it was supported by the chain. This cord is shown in Fig. 2.1.6 on page 26. This precaution saved the instrument
from possible damage on three separate occasions.

4.12 PERSONNEL INVOLVED DURING THE TEST

The author operated the Dynamic Strain Bridge and the pen recorder while a technician* operated the Dynamic Strain Pick-up instrument; the hours worked were from 8.00 a.m. to 2.00 a.m., with four shifts of research assistants, each of three people. One controlled the shaking machine in the fan room, one operated the Schrader vacuum control and checked the gauge pad numbers during each reading, while the third assisted with scaffolding and carried out miscellaneous tasks.

4.13 THE ACTUAL TEST

Testing began on the evening of the 2nd August 1968, with a verification of the response of the building (see Fig. 4.3); no change was found. Readings began in the basement, and progressed up the building. The experimental procedure was consolidated fairly quickly and testing proceeded as planned, the only interruptions being a few gauge pads knocked off - until the shaking machine seized.

This was found to be due to a bearing housing on the vertical sprocket drive shaft (as shown in Fig. B1 on page B2) cold welding to the shaft. A locating key had not been tightened and was rubbing on the housing, causing steel particles to fall into the housing. Fortunately it was quickly repaired and testing was resumed. No other stoppages of any consequence occurred.

An hour and a half was the maximum period that could be worked without operator mistakes being made, although on an uninterrupted run, a rhythm of readings could be established. The research assistants who were found to have good powers of concentration over a long period were given

* Mr. P. Robinson
the key job of operating the vacuum control as the three times the instrument fell from the wall were mainly the fault of the vacuum control operator.

4.14 MEASUREMENT OF THE DEFLECTED SHAPE

Strain measurements were completed in the afternoon of 7th August and the Willmore velocity meter was placed on each level in turn, positioned near the foot of the staircase alongside the wall. A reading was taken in order to determine the total amplitude of the shear wall at each level. The horizontal displacement meter was left untouched to provide a reference displacement at the top of the building, and the period of the shaking machine was checked at 0.369 second before each velocity reading was taken.

Horizontal readings were taken right down to basement level, and the Willmore meter was converted to read vertical velocities. A reading was taken at a position 1'-0" from each shear wall flange next to the wall in the basement in order to estimate the foundation rotation.

Calibration of the Willmore readings was done in two parts. A shaking table with provision for both horizontal and vertical movement was used - this was moved sinusoidally by an eccentrically mounted bearing, and was driven by the variable speed motor and pulley system used for the dynamic strain calibration device. The total throw of the eccentric was 0.078". The Willmore meter was positioned on the table and connected to the previously used pen recorder channel with the same length of lead wire. A calibration signal at a period of 0.369 seconds was obtained, using the same amplification setting on the pen recorder that was used in the main test.

The second part of the calibration was to calibrate the different amplification settings on the pen recorder, as the amplification was increased when the velocity signal
decreased due to the smaller amplitude near the bottom of the building. The voltage required to cause a millimeter of pen movement was specified on the amplification control of each channel: by feeding various calibration signals into the channel from the Dynamic Strain Bridge the ratio of amplification values between each setting was obtained. It was found that the experimentally derived ratio differed by 8% from the ratio calculated from the specified gain at each setting.

The accuracy of the Willmore velocity (or more correctly, displacement) measurements was within 5% after correction for the amplification changes. This figure of 5% is a measure of the accuracy of the reading and correcting the paper traces.

4.15 THE VARIATIONS IN THE TOTAL AMPLITUDE

The amplitude of the building, as recorded by the Willmore velocity meter, varied by up to 10% during the test, and the general form of variation was a slow increase during an individual shaking run which may have lasted up to 3 hours. Had the time of each individual reading been recorded the variation could have been better documented but this characteristic of the structure was only discovered after the test.

One possible reason for the trend of a slow increase could be the local migration of soil water under the continuous cyclic loading, causing the soil mass under the building to become more flexible. This may have affected the natural frequency of the first torsional mode but another response curve was not found at the end of a prolonged shaking run.

Random changes in amplitude also occurred, but these were generally less than 4%.
Soon after the test the entire shear wall was photographed in detail. The resulting photographs were scaled to $\frac{1}{12}$ full size and the mosaic for each level was mounted on a sheet of hardboard. The enlargement was sufficient to allow easy reading of the numbers defining the gauge pads. Coordinates of the key gauge pads were measured before the pads were removed from the wall and the positions of intermediate pads were scaled from the photographs.

These mosaics provided an easily interpreted record of gauge pad positions and numbering and also showed in detail any obstructions causing a missed reading.

In order to take the many photographs a light steel frame was constructed which was attached to the glass wall 9'-0" away from the shear wall with suction feet. A 35mm reflex camera, with a wide angle lens was mounted on this frame, and a series of photographs was taken in order to build up the mosaic of each interstorey area. The problem of ensuring that the line of sight of the camera was perpendicular to the wall was solved by constructing a flat 3-legged frame (spanning 2'-6") which carried a 6" square mirror in its centre. This was held against the wall in the centre of the proposed photograph and the camera was correctly aligned when the image of its lens could be seen in the centre of the view finder.
CHAPTER FIVE

PHOTOELASTIC TECHNIQUES

5.1 INTRODUCTION

This chapter deals with the construction of a photoelastic model of the prototype shear wall, and its subsequent testing. A description of basic photoelastic techniques and the theory of photoelasticity can be found in references 24, 26, 27, 34, 35, 36 and 37.

5.2 THE BASIS OF THE PHOTOELASTIC TEST

The normal method of photoelasticity gives isochromatics and isoclinics for a structure. The isochromatics (or fringes) represent contours of the principal stress difference, or the shear stress. The isoclinics are lines along which the principal stresses have the same orientation, and these enable stress trajectories to be drawn. They also enable the horizontal (or vertical) shear stress at a point to be calculated from the isochromatics.

The method of photoelastic analysis particularly lends itself to the examination of shear walls (in the elastic range) as it directly provides the shear stress distribution. Unless the wall is very slender the axial stresses due to bending are not as critical as the shear stresses, and they are generally more predictable, especially if the shear wall has flanges. The normal method of photoelasticity alone cannot provide the complete stress pattern for a structure as only two of the three unknowns required to completely solve the structure can be found. If the individual principal stress values are required, the isopachics (contours of the sum of the principal stresses) can be obtained by various methods such as oblique incidence photoelasticity (26, 27), the conducting paper
analogy \((24, 27)\), and finite difference techniques \((24)\).

Since the principal stress difference and orientation was given by the photoelastic test it was decided to base the stress comparisons between the prototype, the photoelastic model and the finite element analysis directly on these values.

5.3 SHEET CASTING TECHNIQUES

A good deal of effort went into developing a method of casting sheets of Araldite D (an epoxy casting resin with photoelastic properties) for model construction. The castings were made in Perspex moulds. Appendix C contains the details of mixing and casting the two part epoxy resin, and general points on model construction.

5.4 MODEL PLANNING IN GENERAL

One of the main features of a model for photoelastic analysis is that it must be dimensionally identical to the prototype. However, as a shear wall is essentially a two dimensional structure, the relative thickness of the wall may be increased to provide a stronger model and more isochromatics. Similarly the thickness of the flanges can be varied (within reason) as long as the flange area remains constant.

The less obvious feature in the modelling is concerned with the loading method. The loading must represent as closely as practicable the actual case, especially in its distribution. Care must be taken, however, not to stiffen the model with the loading device, which includes the method of holding the model in the loading frame.

Specific points concerning the making and testing of models are covered in Appendix C.
5.5 THE POLARISCOPE

The Civil Engineering Department’s polariscope had 18" dia. circular plates and was constructed in two halves, as shown in Fig. 5.1. On one half was the light source, the polariser and the first quarter wave plate, while the second half carried the second quarter wave plate and the analyser. Each half was essentially a table on castors with a frame to carry the plates mounted on top, and since the two halves could be moved apart, large models could be analysed by placing them in the resulting gap. The light source provided white or monochromatic light from two banks of fluorescent tubes (coated and uncoated).

Early testing showed the need to be able to rotate all the plates simultaneously, and a rotational coupling was fabricated using light chains and sprockets as shown in Fig. 5.1.

5.6 PHOTOELASTIC PHOTOGRAPHY

In order to obtain high contrast black and white photographs of the isochromatics, Ilford F.P.3 panchromatic film was used. This was exposed normally, but the development time was increased by 50%. The printing was done on Ilford Ilfobrom 5 high contrast paper. The philosophy adopted was to obtain all the necessary detail on the medium contrast negative and then selectively print the isochromatics on the high contrast paper.

The camera used was a Kodak Specialist plate camera with a Wray 21" lens. By using a lens with a long focal length, the rays of light entering the lens from the model were much less divergent than for an ordinary lens.

The camera is also shown in Fig. 5.1.

5.7 CONSTRUCTION OF THE ACTUAL SHEAR WALL MODEL

5.7.1 Preliminary Model Design. From the experience
FIG. 5.1  THE SHEAR WALL MODEL IN THE POLARISCOPE
gained from previous models, as briefly described in Appendix C, it was decided to cast the web of the Chemistry Block shear wall model in one piece, complete with openings instead of cutting them. This decision was made on the grounds that more than one model would probably be required, and it also overcame the problem of accurately butt-joining sheets of different thicknesses together. Another important fact was that the model thicknesses would be more uniform as the rectangular Perspex opening formers would act as intermediate spacers. The importance of having a uniform thickness is due to two factors. Firstly, the physical properties of the model depend on the thickness, and secondly, the difference between the principal stresses is given by the formula:

$$\text{Principal Stress Difference} = \frac{\text{Constant} \times \text{Fringe Order}}{\text{Model thickness}}$$ (5.1)

A scale of \(\frac{1}{60}\) full size was used for the planar dimensions of the model and a thickness scale of \(\frac{1}{40}\) full size was chosen. This gave the model dimensions of approximately 21" x 9\(\frac{1}{2}\)" with thicknesses .30", .25", and .20". The thickness was increased mainly to provide resistance against possible local buckling or web crippling.

5.7.2 Construction of the Mould. The side spacers and the opening formers were milled from \(\frac{1}{2}\)" thick Perspex and the four corners were filed off the rectangular opening formers to prevent severe stress concentrations in the model. The mould was assembled in three sections, one for each web thickness, and the three sections were glued together with I.C.I. Tensol 7. Bolts at 1" centres held the side spacers in place, while the opening formers were held with four similar bolts. Fig. 5.2 shows the completed mould.
The mould was disassembled and cleaned and silicone grease was smeared lightly over the opening formers. Some trouble was expected in removing them from the casting as Araldite D shrinks by about 3% on curing. The mould was assembled and the small gaps between the ends of the spacers at the thickness changes were plugged with modelling clay.

The nozzle of the pouring device as seen in Fig. 5.3, was not attached to the mould at this stage, but was pushed into the tube leading from the mixing beaker as explained in Section 1, Appendix C.

5.7.3 Pouring the Mix. As soon as the mix of Araldite D and HY951 Hardener was ready, about 25 grams was run to waste through the pouring nozzle. This nozzle was then attached to the mould. The mould was held vertically, but tilted about a bottom corner at 30° while pouring took place, the air being vented out a small hole in the top corner of the mould opposite the pouring nozzle. Once the Araldite level reached the vent, it was plugged with modelling clay, and the tube was removed from the mixing beaker. A small expendable container with a nozzle at its base was connected to the tube, and filled with Araldite. The completely enclosed mould was laid on its side with the container supported a few inches above it, as shown in Fig. 5.4. This provided an extra supply of Araldite to prevent bubbles forming as shrinkage took place during the initial curing. By lying the mould on its side a small but uniform hydrostatic pressure was provided by the full container which kept distortions due to pressure in the Perspex mould to a minimum.

Several castings were made before the casting technique was perfected. Some trouble during pouring was experienced with air bubbles separating from the sharp corners inside the pouring lead at the top of the mould.
In Fig. 5.3 (a detail of Fig. 5.2) the wedge shaped area shown additional to the rectangular model was provided to allow any air bubbles due to shrinkage to form in it rather than in the model - a bubble can be seen near the top. This happened when the Araldite in the tube leading from the reservoir to the mould became too viscous to run under gravity, but shrinkage was still occurring.

5.7.4 Stripping the Mould. Twenty hours after pouring the mould was stripped as described in Section 3, Appendix C. Once the sheet was free, the opening formers were worked out. Care was taken not to contaminate any other part of the casting with silicone grease, and the inside edges of the openings were wiped as clean as possible. The model was laid on a paper covered stepped glass support for further curing.

5.7.5 Fabrication of the Model. Reference lines were scribed on the model. These were drawn along the lines represented by the rows of gauge pads as described in Section 4.9 and shown in Fig. 4.4 on page 121. Hence stress measurements could be made at the same points for comparison purposes.

Representative floor slabs were cut from 1/8" Araldite sheet and were glued perpendicularly to the wall. The floor slabs were 1/16" wider than required to allow for final machining.

Flanges were cut 1/8" wider than required from 1/4" thick Araldite sheet and were glued in place. Fig. 5.5 shows the gluing procedure for a model without floor slabs using the steel surface table and magnetic clamps to hold the model during the gluing of a flange.

Once the model had been assembled, a holding base
FIG. 5.5. ASSEMBLY OF THE ARALDITE MODEL
was cast on it. This was a 1" wide by 1\(\frac{1}{4}\)" deep rectangular section in which the base of the model was embedded by half an inch. A Perspex mould for the base was made to fit between the flanges of the model at the base of the wall - the flanges had been extended beneath the base of the model by one inch, and the base was actually cast in place. Half inch fillets were also cast in the corners of the flange-base junction.

By casting the base on the model any weakness due to a glued joint was overcome.

Once the model was complete it was held in a stepped jig on the table of a milling machine and the edges of the flanges and floor slabs were fly-cut to give a flat surface on each side and to reduce the flange width to the scaled dimension of 2.18". This width was scaled to 1/40 full size (the thickness scale) and the area of the prototype flange considered included the 24" return on the end of the flange as shown in Fig. 3a6 on page 106. An I-beam type cross-section was used for the model rather than the channel type cross-section of the prototype to provide better lateral stability and to provide a lateral support for the floor slabs on both sides. This change does not theoretically affect the stress distribution in the web.

Two \(\frac{3}{8}\)" dia. holes were drilled in each floor slab to receive the steel blocks of the loading device.

Fig. 5.6 shows the complete shear wall model, with the slotted blocks fixed to the floor slabs. Note also the grid lines representing the lines of triangles of gauge pads on the prototype wall.

5.7.6 Annealing the Model. With the model fully constructed, apart from the bolt holes drilled in the base, annealing was carried out. Eight steel supports
FIG. 5.6 THE PHOTOELASTIC MODEL
were machined to support the interstorey parts of the model when it was laid on one side on a glass plate in the oven, (see Appendix B, Section 6). The base was also supported.

Although the base, the web and the flanges of the wall did not deform during annealing, some of the floor slabs buckled slightly but this had no detrimental effect on the model.

5.8 **THE LOADING FRAME**

The steel base holder was made from \( \frac{1}{8} \)" x 2" flat as shown in Fig. 5.7, an overall view of the loading frame. Six \( \frac{3}{8} \)" dia. steel pins were pushed through reamed holes in the Araldite base and the nuts on the pins were tightened to provide both a bearing and frictional resistance against the applied moment and shear at the base of the model wall.

The loading frame itself was mounted on a table on castors, the centre of the model being at the same height as the centre of the polariscope plates.

The shear wall was loaded through the floor slabs as in the case of an actual wall. To do this, slotted blocks were pushed over the floor slabs and fixed in position with \( \frac{1}{8} \)" dia. pins pushed through the previously drilled holes. Rods, \( \frac{3}{4} \)" dia. were screwed into tapped holes in the ends of the slotted blocks and were connected to the lever system which is shown in general and in detail in Fig. 5.7.

The distribution of the lateral load on the shear wall model could be altered by changing the position of the various links between the horizontal arms; the distribution used was that obtained by integrating the horizontal shear stress across the shear wall at each interstorey level using the prototype results - this is described in
FIG. 5.7 DETAILS OF THE PHOTOELASTIC MODEL'S LOADING FRAME
Section 7.8. Hence the photoelastic model of the shear wall was loaded with the same lateral loading distribution as the prototype shear wall in order to enable a valid stress comparison to be made.

The loading frame was also designed to enable sections of the wall to be tested, although none were.

The loading lever was designed to come in contact with a wooden block if the model should fail - this can be seen in Fig. 5.7. However, this "fail-safe" system was not fully effective as there was a lot of elastic deformation in the lever system and the model itself, which, when considered in terms of strain energy, might have been sufficient to severely fracture the model if an initial crack developed, even though the full load would be removed by the lever hitting the stop.

Fortunately the model remained intact.

The gap between the lever and the block was regulated by the turnbuckle shown in Fig. 5.7. This figure also shows the high mechanical advantage of the lever arm, which meant a relatively small load could be used to stress the model. This simplified moving the loaded model during photography when it was in the polariscope and provided an easy release of the load.

5.9 THE PHOTOELASTIC MODEL TEST

The model was gradually loaded while examined in the polariscope until sufficient fringes for measurement were formed without overstressing the model. The loaded model was photographed under monochromatic light and from the isochromatic photographs, computer data sheets were prepared with the coordinates of each point and the approximate fringe order being entered.
The orientation of the principal stresses at each grid point was recorded by finding the isoclinic running through that point. When the isoclinic was indistinct, oscillating the coupled analyser and polariser slightly showed the trend of its movement and the isoclinic angle could be repeated better than \( \pm 2^\circ \).

The principle of Tardy Compensation \(^{(26,27)}\) was used to find fractional fringe orders. The four polarscope plates were set up for dark field isochromatic viewing and they were coupled together and rotated until the analyser was set at the isoclinic angle for the particular point in question. The analyser was then released to allow independent rotation, and the angle at which it was set to cause the adjacent fringe of the lower order to pass through the point recorded. If a series of points had principal stress orientations within 6° of each other a mean value was chosen, and the series was read without changing the orientation of all four plates. It had been found that it was not necessary to align the plates exactly for each reading; clear fringes still resulted. This can be verified by considering the Poincaré sphere \(^{(38)}\) analysis of the principle of photoelasticity, as the deviation from linear polarization is a function of the sine of 90° - 2α where α was up to 4° in this case.

The computed fractional fringe orders were checked against the estimated fringe orders, and any discrepancies were re-checked on the model. An example of the computed results for floor 3 can be seen in Table 7.7 on page 176.

The total lateral load on the shear wall model was 620 lb, as calculated from the 115 lb on the lever system. By finding the average resolved fringe order across each interstorey section, as described in Section 7.11, and knowing the shear force across each section, the fringe
constant for the Araldite was calculated at 66 lb. per inch - which is slightly higher than the representative figure for this material. Usually the fringe constant is about 61 lb. per inch.

5.10 PRODUCTION OF THE HALF FRINGE ORDER PHOTOGRAPHS

The model was photographed under both light and dark field isochromatic viewing conditions in order to record both full and half order fringes. Each floor level was photographed with the film and equipment described in Section 5.6.

Printing was done by double-exposing the resulting negatives (dark and light field for each level) on a sheet of relatively high contrast photographic paper (Ilfochrom 4) and in this way an isochromatic photograph of both whole and half order fringes was produced. An example of these photographs can be seen in Fig. 8.19 on page 222. The grid can be seen to be slightly out of alignment in some places - this appeared to be due to the light being slightly refracted in the analyser, thus influencing the position of the grid on the negative as the analyser was rotated through 90°.

5.11 THE EFFECT OF POISSON'S RATIO

Dally and Riley (27) showed that in a photoelastic study of a two-dimensional model with elastic constants (E, the elastic modulus, and μ Poisson's ratio) which differ from the prototype, the difference between the E values has no effect on the stress distribution. They also show that the effect of a different value of μ had no effect on the stress distribution providing:

1. There are no body forces.
2. There is a uniform body-force field, i.e. gravitational.

3. There is a linear body-force field in x and y.

There are, however, two exceptions to the general similarity of the stress distribution in two dimensional parts.

Firstly, if the two dimensional photoelastic model is multiply connected the effect of a different Poisson's ratio becomes apparent. Since a shear wall model is of this type, then some deviation between the stress distributions in the model and the prototype could be expected. Fortunately, in specific examples of multiply connected bodies where the effect of Poisson's ratio has been evaluated it's influence on the maximum principal stress is usually less than 7%.

The second exception to the laws of similitude governing the stress distribution is where the photoelastic model undergoes a severe distortion. However, in the shear wall test performed the photoelastic model did not deform sufficiently to change the geometry of the structure.
CHAPTER SIX

THE COMPUTER PROGRAMS

6.1 INTRODUCTION

This chapter deals with the computing facilities available, and with the computer programs used. The finite element and the contour plotting programs are described in some detail.

6.2 COMPUTING FACILITIES

The University of Canterbury possessed an I.B.M. 360/44 computer with 128K bytes of core storage and also an I.B.M. 1620 computer. An I.B.M. 1627 Autoplotter was linked to the smaller 1620 machine. This could plot graphs up to 11" by 100" with ball point or wet ink pens.

6.3 COMPUTER PROGRAMS AVAILABLE

At the time the computation phase of the research was carried out, the Civil Engineering Department was building up its own computer program library under the direction of Dr. A. J. Carr. Particular programs used from this library were:

1. A contour plotting program for use on the 1627 plotter. This program was upgraded and modified by the author to enable it to plot isochromatics.

2. A finite element program to solve shell structures. This program could handle up to 100 nodal points and 180 triangular elements with 5 loading cases.

3. The "Symbolic Matrix Interpretive System"; a program designed to manipulate matrices.

4. The "Plane Stress Program"; a finite element program used to solve two dimensional structures. It could
handle up to 200 nodal points and 180 quadrilateral or triangular elements with 5 different loading cases.

5. The "Plane Frame Program", a frame analysis program which could handle two dimensional frames with infilled panels if required.

6.4  **FINITE ELEMENT THEORY**

A history and description of the theory and use of finite elements is unwarranted in this text. However the reference most readily available is probably Clough's paper in the text by Zienkiewicz and Holister(10). The actual type of finite element used was the quadratic strain triangle, proposed by Felippa(39) and developed by Carr(40). The quadratic strain triangle is more complicated than the more commonly used linear or constant strain triangles. However its use can be justified in that it gives a much better representation of the action of the structure and a coarser element mesh can be used.

Cross reference concerning the finite element technique can be found in Carr's text(40).

6.5  **THE PLANE STRESS PROGRAM**

This program used the quadratic strain triangle with three nodal points as its primary element. These elements, as the nodes were restrained to two dimensional movement only, had no bending stiffness about the planar axes of the element, in contrast with the three dimensional shell program.

The program had nine subroutines which were called by a short main-line program and the operation of the subroutines is as follows:

1. **SETUP.** The input data was read and checked where possible. The triangular mesh was formed from the
quadrilateral elements and the element stiffnesses were formed.

2. **FORMK.** The stiffness matrix of the elements was formed.

3. **LOAD.** The loads on the elements (in terms of nodal point loads or element edge loads) were read in and checked where possible. Boundary conditions were applied.

4. **BIGRED.** The stiffness matrix was reduced.

5. **BIGSOL.** The load vector was reduced and the displacements of the nodes were found.

6. **MESH.** A six nodal point mesh was formed on the triangular elements and these nodal point displacements and strains were calculated for use in the line printer contour plotting.

7. **FORCE.** The nodal point displacements and strains were calculated and printed (for the original nodal points). The stresses were also calculated and printed.

8. **PLOTA.** The graph size and contour spacing for the line printer contour plots was calculated.

9. **PLOTB.** The contour plots of the listed stresses were printed (the contours being in terms of numbers) on the line printer.

6.6 **USE OF THE PLANE STRESS PROGRAM**

This program was used to obtain the stress distribution in one interstorey area of the shear wall in order to compare the prototype and photoelastic results with a computer analysis of the same structure. A theoretical flexibility matrix was also calculated for the shear wall using this program.

Data was fed into the computer in four main segments as follows:
1. The control deck, which provided the various parameters such as the Elastic Modulus, Poisson's ratio, and the number of nodal points and elements.

2. The nodal point deck, which specified the coordinates of each node and the thickness of the structure at the nodes.

3. The element deck, which described the triangular or quadrilateral element by its nodal point numbers.

4. The loading deck(s), which gave the nodal point, or element edge loads.

A feature of the program was that it could interpolate regular nodal points or elements - only the first and last items in a regular run were specified.

A maximum number of 200 nodal points and 180 elements was allowed in the primary mesh and the maximum numerical difference between the nodal point numbers describing any one element was 14.

The output of the program came in five stages, as follows:

1. The input data was interpolated and echoed on the line printer.

2. The nodal point displacements in the X and Y directions, and their four derivatives (i.e., strains) were printed. This stage was controlled by subroutine FORCE.

3. The next table, also produced by subroutine FORCE gave the nodal point stress values and the principal stress orientation. The actual stresses printed were the direct stress in both the X and Y direction, the shear stress in the X (and Y) direction, the principal stresses and the maximum shear stress.
4. Six contour plots were produced (if required), contours being printed for the stress values specified above. These contour plots gave an excellent visual picture of the tables of results and also clearly showed any stress concentrations that may have been due to input data errors.

5. The stress values could also be punched on to cards if further work was to be done on the results, such as contour plotting on the L.B.M. Autoplotter.

6.7 THE DATA CHECKING PROGRAM

A program to check the input data for the plane stress program was developed by the author. Its major part was the section of subroutine SETUP which read, interpreted, and checked the input data where possible and echoed the input data on the line printer.

In addition, this checking program drew out the element mesh on the 1627 plotter and numbered the nodes and elements. This was found to be of great benefit as even scrupulous checking of the initial data often missed a wrong nodal point coordinate, or wrongly numbered element - these faults were clearly visible when the mesh was drawn.

The checking program also had optional features. It could print out each card as it was read to pinpoint punching errors, and it had the facility to punch a full nodal point coordinate and triangular element deck if contour plotting of the plane stress program results was to follow.

6.8 THEORETICAL SHEAR WALL ANALYSIS

The only operational finite element program when the theoretical analysis phase of the research began was the 100 nodal point shell program. This was used to find a flexibility matrix for the wall by applying 1 Kip to
each floor level at a time. However, the limitation on the number of nodal points meant that a coarse mesh had to be used.

Shortly after the shell program was used the new 200 nodal point plane stress program became available. A finer mesh was devised, and the flexibility matrix was recalculated. The results obtained were suspect because of the differences in the deflected shape between the shell program and the new plane stress program. Some research showed that the representation of the flanges of the shear wall in the essentially two-dimensional plane stress program was not valid. They had been formed by thin, wedge-shaped elements. The program had to be modified to allow flanges to be properly represented; this was done by formulating a beam element with an axial stiffness but no bending or shear stiffness, which could be connected between any two nodal points, providing the beam element lay in the Y direction.

This modification also allowed more nodal points to be used in the web of the shear wall as only two against the previous four nodal points were required to describe the flanges. A finer mesh was devised and the flexibility matrix was recalculated.

The new flexibility matrix had values approximately 15% lower than predicted by the shell program. A slight decrease in stiffness was predicted by Carr(4) due to a finer element mesh, but the stiffer wall probably resulted from the fact that no shear lag was possible in the beam elements forming the flange, whereas in the shell program the shear wall flanges were represented more exactly.

For analysis purposes the shear wall was considered rigidly fixed at the base of level 1. Fig. 6.1 shows the finite element mesh of the entire shear wall: it is
151.

**FIG. 6.1 THE FINITE ELEMENT MESHES USED**
actually the mesh drawn by the graph plotter as part of the data checking program.

In section 7.8 the calculation of the actual lateral load on the prototype shear wall is described, and the derived prototype loading is shown in Table 7.10 on page 183. This loading was applied to the finite element shear wall in the form of a linearly distributed load across each level at the position of the floor slab.

A part of the shear wall was taken for a more detailed analysis; that between the top of the opening in level 2 and the top of the opening in level 4. This mesh is also shown in Fig. 6.19, and it was designed to examine floor 3 in detail.

The nodal point stresses along the top and the bottom of the part of the shear wall considered were plotted from the results obtained from the coarser mesh of the entire wall, and from these curves the nodal point forces were calculated for the finer mesh. The section of the wall formed by the finer mesh was fixed against movement in the X and Y direction at one bottom corner, and against movement in the Y direction at the other to restrain rigid body motion. The floor slab loads were also applied to the structure: once again they were uniformly distributed over the width of the wall.

The resulting nodal point stresses were punched out for contour plotting.

6.9 THE CONTOUR PLOTTING PROGRAM

In order to show a comparison between the prototype, the photoelastic model, and the plane stress program results, the contours of the difference between the principal stresses were plotted, except for the
FLOOR 3

Fig. 6.2. A TYPICAL CONTOUR PLOT
photoelastic case as the isochromatic photographs gave the contours directly.

To do this, the area over which the contours were to be plotted was triangulated. The coordinates of each nodal point and its stress value was fed into the computer as well as the element deck.

The actual contour plotting part of the computer program worked by linearly interpolating the positions of the incidence of the contours along each side of the triangular elements. The corresponding positions were joined by straight lines and the contour plot was thus built up. Plotting over any element could be skipped by specifying this in the element deck.

The contour spacing was read into the computer at the beginning of the plotting data. The program was also modified to give each nodal point stress value a specified stress increment - the reason for this will be discussed in Section 7.10.

Plot size on the 1627 Autoplotter was limited to 9" x 25" for this particular program. Fig. 6.2 shows a typical plot and if this is compared with the shaded-in plot of the floor 3 contours in Fig. 8.19 on page 222 it can be seen how the rather jagged contours have been smoothed by hand to give a more realistic plot.

6.10 THE SYMBOLIC MATRIX INTERPRETIVE SYSTEM

This program which had user oriented language data preparation was very easy to use to manipulate matrices. It was used to compute the theoretical frame/shear wall interaction. The basic items of information fed into the program were the flexibility matrices for the shear wall and a typical frame and the lateral load
on the structure. Details of the actual matrix operations performed are described in Section 7.16. The flexibility matrix of the frame was obtained from the Plane Frame program.

6.11 OTHER PROGRAMS USED

A series of relatively simple programs were written to reduce and operate on the experimental data from both the prototype and the photoelastic tests. Many of these programs were concerned with plotting results on the 1627 Autoplotter.

In Table 7.1, on page 166, the listing of the Fortran IV program for computing the principal stresses and other values from the prototype experimental data is shown. The output from the program is shown in Fig. 7.2 which is one of a series of typical results prepared in full for floor 3.
CHAPTER SEVEN

DATA REDUCTION AND PRESENTATION

7.1 INTRODUCTION

This chapter deals with the reduction of the prototype dynamic strain data to provide the principal stresses and their orientation. It also discusses the basis of the comparisons drawn between the prototype, the photoelastic model and the finite element analysis, and describes the method used to find the frame/shear wall interaction.

7.2 THE CALCULATION OF THE CONCRETE PROPERTIES

Directly after the building test an ultrasonic concrete tester was used to try to determine the modulus of elasticity of the concrete in the shear wall. However, as a reflection technique had to be used, little useful information could be obtained, apart from showing that the velocity of the impulse through the concrete was fairly constant, which meant the concrete properties were fairly constant.

A Schmidt hammer was used, and this gave an average ultimate strength of 5,000 lb./sq. inch for the concrete. Test cylinders taken during the construction of the building had shown a 28 day strength of at least 4,500 lb./sq. in. with a density of 150 lb./cu.ft. Using the factors of 5000 lb./sq. in. and 150 lb./cu.ft., the value of $E = 4.43 \times 10^6$ lb./sq. in. was calculated. (41) This value of $E$ was assumed to remain constant both because of the small stresses involved and because of their dynamic nature.

Poisson's ratio was taken as .15 for concrete as suggested by Neville (42).
7.3 THE METHOD OF READING THE PAPER TRACES

A device was made to assist in reading the long rolls of pen recorder paper. It wound the roll from one spindle to another, presenting a 10" length of the trace flat on a small platform, as shown in Fig. 7.1. The winding mechanism was powered by a treadle operated electric sewing machine motor.

A Perspex parallel rule was modified to enable the peak to peak height of the sinusoidal strain or calibration trace to be read. This involved fitting a protractor to one radius arm of the parallel rule which gave the distance between two lines, one scribed on each half of the rule, in terms of an angle. This reading device is also shown in Fig. 7.1.

FIG. 7.1 THE PAPER READING DEVICE

Fig. 7.2 shows a typical trace from a dynamic strain reading, and also shows the calculations used to reduce the sine wave to a strain level. Fig. 7.3 shows some unusual readings taken during the test. The strain readings from
Fig. 72 Typical Dynamic Strain Reading

TOTAL DYNAMIC STRAIN = 2000 µ
INSTRUMENT MAGNIFICATION = 2000
CAL. 2 = 2.93 microns
CALIBRATION SIGNAL
STRAIN WAVEFORM

PAPER SPEED 50 m./min.

WILLIAMS VELOCITY
STRAIN READING AND CALIBRATION
HORIZONTAL DISPLACEMENT
PHOTOCELL (7 kHz)
FLOOR 4 AREA ABOVE DOOR — NO VISIBLE CRACK

FLOOR 5 HORIZONTAL READING ON SHEAR WALL FLANGE

FLOOR 7 ACROSS VISIBLE CONSTRUCTION JOINT

FIG. 7.3 PECULIAR READINGS DURING THE TEST
floors 4 and 5 indicate that the gauge pads concerned were spanning a crack which was opening and closing, and somehow bouncing slightly each time. Across the horizontal construction joint in floor 7 it seemed as if there was a grating action in the crack possibly due to small shearing moments along it.

The experimental traces were written on computer coding sheets, following the gauge pad numbers, as trace heights in terms of the protractor angle. The computed trace heights in millimeters, together with other relevant information was double checked against the experimental results. Strain levels were expressed in terms of the calibration strain which was based on a gauge factor of 2.0 (see Section 2.2.8.8). The actual strain level was later obtained by dividing by the magnification factor, as described in Section 2.1.4.

The strain levels were also adjusted during the computation stage for the changes in building amplitude that occurred during the test. Each strain reading from the paper trace was accompanied by a reading of the height of the Willmore velocity meter trace; its average value was found, to enable strain reading adjustments to be made. Since the shear wall was loaded by the inertia of the building, and this inertia loading was proportional to the amplitude of the building, the above adjustment can be justified. (See Section 4.15 on the magnitude of amplitude change.)

7.4 INITIAL STRESS CALCULATIONS

A trial plotting of the principal stress difference calculated from the strains showed a scatter and it was decided to plot, and correct if necessary, every strain reading. The wall was analysed as an uncracked concrete section, which meant that high strains due to a pair of gauge pads spanning a crack had to be reduced in most cases.
The few strains lower than expected could have been due to loose or dirty gauge pads, or perhaps due to the instrument operator neglecting to tighten the adjustable gauge point. A scatter was inevitable due to the experimental nature of the test, but any obviously wrong strains could be detected.

7.5 THE STRAIN CHECKING AND CORRECTING METHOD

7.5.1 Correction of the Group Strains. The checking and correcting of the strains read from the groups, or individual triangles covering the majority of the shear wall, was done in the following way.

The strain levels were plotted for each floor across the building; these plots revealed a significant pattern as seen in the bottom plot in Fig. 7.4. When the strains along a vertical line up the building were plotted a strong pattern emerged; this was due to the fact that the building was more symmetrical in this direction. The plots were drawn in ink on transparent graph paper and when each strain reading was checked it was located on both the plot of the points up the building and the plot across the building. If the particular value considered deviated from the other corresponding values on one graph its proposed corrected value was plotted in pencil on the other graph to check that the trend of correction was correct. Fig. 7.4 shows an example of the two sets of curves plotted to draw a strain comparison and to provide the basis for corrections.

7.5.2 Correction of the Cluster Strains. The cluster strains were also checked by drawing at least two, and sometime three sets of curves to check that the trend of correction of a reading was correct. The first set of curves was plotted by considering a row of triangles of gauge pads as found, for example, in the bottom row of the spandrel in floor 3. Each strain reading
GROUP STRAINS - 15'-0" FROM LEFT FLANGE — ALL FLOORS

FIG. 7.4 CORRECTION OF GROUP STRAINS
STRAIN LEVELS - BOTTOM ROW IN SPANDREL FLOOR 3

COMPARISON OF STRAIN LEVELS - BOTTOM ROWS IN SPANDRELS

FIG. 7.5 CORRECTION OF CLUSTER STRAINS
along the row was plotted in ink on transparent paper - an example is shown in the top plot in Fig. 7.5. A relatively smooth curve was expected through each strain reading with the same orientation, as the readings were only 10" apart.

The second set of curves was obtained by plotting all the strain readings with the same orientation in the same area on each level: the bottom plot in Fig. 7.5 shows an example of this. Comparisons were made mainly on the basis of these two sets of curves, but for the 3 (or 5) rows of gauge pads above the openings another set of curves was drawn. The strain levels with the same orientation for each row of triangles were plotted similar to Fig. 7.5, but instead all the curves for a single spandrel were plotted. This gave the trend of the strain readings from the top to the bottom of the entire spandrel beam, and this third set of curves was used to cross-check any corrections made on the basis of the first two sets of curves.

From the strain curves it was easy to estimate strain levels at points not measured because of obstructions. Also the position of a crack running across a row of triangles could be traced, as shown in the first plot in Fig. 7.5.

It was found that more strain corrections were necessary for the clusters of gauge pads rather than for the groups, mainly because there were some cracks around the more highly stressed doorway area.

The corrected strains for both the groups and the clusters were punched on computer cards alongside the original strains, and after punching the corrections were verified by checking back to the original graphs. An example of the strain corrections made on floor 3 can be seen in Tables 7.2 to 7.5.

7.6. **Final Computation of the Stresses**

The three strain values for each equilateral triangle
of gauge pads were solved (43) to give the principal stresses and their orientation, as shown in Fig. 7.6.

The corrected strains, together with the coordinates and orientation of each triangle of pads, were programmed into the computer, and a table of the original and corrected strains, together with the stresses were printed out on the line printer and also punched on cards for further use. Table 7.1 shows the program used to reduce the cluster strains to stresses, and the computed cluster results are shown for floor 3 (as an example) in Tables 7.2 to 7.4. Table 7.5 gives the strains for the groups of gauge pads for floor 3, and Table 7.6 provides the stress values.

An additional column in Table 7.6 gives the phase angle that the maximum strain lagged behind the maximum force of the shaking machine; these values were checked to ensure the strains all had approximately the same phase angle.

7.7 PRESENTATION OF RESULTS USING THE AUTOPLOTTERT

In Section 5.2 it was stated that the comparison of the stress results between the prototype, the photoelastic model, and the finite element analysis would be made on the basis of the principal stress difference and the principal stress orientation. Since grid lines on the photoelastic model corresponded to the lines of triangles of gauge pads on the prototype wall, the finite element mesh could be suitably designed, it was decided to present the stress values in two ways.

Firstly, using the IBM 1627 Autoplotter, points were plotted along each line on the wall common to the different methods of analysis. For example, in Fig. 8.8 on page 203, which is the set of points showing the comparison of the principal stress differences along a line 6.44 above the floor slab, the results from all three methods can be seen in the plot for floor 3.
TABLE 7.1 STRAIN REDUCTION PROGRAM

...
### Table 7.2 Floor 3 Prototype Results — Cluster

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**TABLE 7.5 FLOOR 3 PROTOTYPE RESULTS — GROUPS**
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**TABLE 7.6 FLOOR 3 PROTOTYPE RESULTS—GROUPS**
Secondly, contours of the principal stress difference were compiled to supplement the plots of principal stress difference. These contours, in the case of the photoelastic model, were provided directly by the isochromatic photographs. The contours for the prototype and the finite element results were compiled using the plotting program described in Section 6.9.

7.8 PLOTS OF THE PROTOTYPE VALUES ACROSS THE BUILDING

Plots across the building were produced by the Auto-plotter at the four levels formed by the rows of triangles (9.29, 6.44, 360 and 0.77 ft. above the floor slab as shown in Fig. 4.4 on page 121). These were made for both the principal stress difference and the resolved shear stress. Before these plots were made, the plots of the principal stress difference and the resolved shear stress along the vertical lines formed by the staggered gauge pads in the staircase area were plotted, and from the resulting curves the stress values at the positions of the full triangles shown in Fig. 4.4 on page 121 were interpolated. This enabled the values to be plotted along the same line right across the building.

By integrating the plots of the resolved shear stress across the building at the four different heights above the floor, the lateral load resisted by the web of each inter-storey area was calculated. An attempt to estimate the lateral load carried by the flanges was made by finding a theoretical relationship between the shear stress at the flange-web junction and the shear force carried by the flange. Hence the lateral load carried by the flanges was found from the prototype shear stress values at the flange-web junction.

These results are shown in graphical form in Fig. 8.1 on page 187 and the experimentally derived lateral load on the prototype shear wall is shown in the last two columns of
A = \frac{(\varepsilon_1 + \varepsilon_2 + \varepsilon_3)}{3}

B = \frac{(\varepsilon_1 - \varepsilon_3)}{\sqrt{2}}

C = (\varepsilon_3 - A)

\text{PRINCIPAL STRAINS :}
\epsilon_{pq} = A \pm \sqrt{B^2 + C^2}

\text{PRINCIPAL STRESSES :}
\sigma_p = \frac{E(\varepsilon_p + \mu \varepsilon_q)}{1 - \mu^2}
\sigma_q = \frac{E(\varepsilon_q + \mu \varepsilon_p)}{1 - \mu^2}

\text{PRINCIPAL STRESS ORIENTATION :}
\theta = \frac{1}{2} \tan^{-1} \left( \frac{B}{C} \right) \text{ clockwise}

\text{FIG. 7.6 DATA REDUCTION CALCULATIONS}

\text{FIG. 7.7 A TYPICAL CLUSTER RESULT}
Table 7.10 on page 183. This lateral load is used several times later in this text.

7.9 PLOTS OF THE PROTOTYPE VALUES AROUND THE OPENINGS

An example of a plot of the principal stress difference along a row of triangles is given in Fig. 7.7. A slight zig-zag effect can be seen and the continuous line shows the stress level interpolated from the results. The reason for the zig-zag, or, more correctly, the two distinct curves is that there was a strain gradient over the width of a row of triangles. As there was 8.6" between the two parallel rows of gauge pads the strain difference between the top and bottom strain readings in the row slightly affected the resulting stress values.

A curve was drawn through all the plots of principal stress difference for the cluster results and the interpolated values were punched on to cards for further use.

7.10 THE CONTOUR PLOTTING OF THE PROTOTYPE RESULTS

A triangular mesh was devised for each interstorey area so that away from the doorways the nodal points of the mesh were those provided by the individual triangles of gauge pads (see Section 6.9 on the contour plotting program). Around the openings, lines were drawn along the rows of triangles forming the clusters, and nodal points at 10" centres were chosen to correspond to positions of stress measurement.

The principal stress difference at each nodal point was programmed into the computer. In the case of the groups, the stress values were taken directly from the computed results, but in the case of the clusters, the interpolated stress values were used, as described in the last section.

In Section 7.13 the method of finding the ratio of the stress level in the prototype to the photoelastic fringe order is discussed. By using this ratio, the contour spacing
of the prototype principal stress difference values was set to give four contours to each full order photoelastic fringe, and the prototype stress values were all incremented by half the value of the contour spacing. (This was done by using the built-in facility of the contour plotting program.) This meant that if each alternate pair of contours was used as the boundary of a shaded area, the effective centreline of the shaded area was the position of an un-incremented contour. In this way the shaded contour plots of the prototype principal stress difference were equivalent to half fringe order photoelastic isochromatics.

Fig. 6.2 on page 153 showed the contours plotted from the prototype principal stress difference values for floor 3, and Fig. 8.18 on page 222 shows the shaded contour plot compared with an isochromatic photograph for floor 3. By comparing these two figures it can be seen how the contours had to be smoothed to remove the jagged effect caused by the straight line interpolation of the stress values across each triangular element as described in Section 6.9.

7.11 THE PHOTOELASTIC DATA

The isochromatic photographs of half fringe order were prepared for each floor as described in Section 5.10, and in Tables 7.7 and 7.8, examples of the computed photoelastic results are given for floor 3. The derivation of these results was given in Section 5.9.

In addition to the photoelastic results being printed out on the line printer, the coordinates of each point, the fringe order and the isoclinic angle were punched on to cards for further use on the Autoplotter.

The values of the resolved fringe order were plotted across the shear wall at each of the four levels for each storey and from these plots the distribution of the lateral
**TABLE 7.7 FLOOR 3 PHOTOELASTIC RESULTS — GROUPS**

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### Table 7.8 Floor 3 Photoelastic Results — Cluster
load on the model shear wall was verified as being the same as the experimentally derived distribution on the prototype wall.

7.12 THE ORIENTATION OF THE PRINCIPAL STRESSES

The orientation of the principal stresses was presented graphically for the entire wall for both the prototype and the photoelastic results. By using the Autoplotter a small cross was made at the position of each stress measurement: the cross was oriented in the direction of the principal stresses. Figs. 8.26 and 8.27 on pages 230 and 231 show these two plots. In Fig. 8.27, which shows the photoelastic values, the positions of observed isotropic points are marked with a circle, or a semicircle if on a doorway: isotropic points are points through which all the isoclinics pass and hence are necessarily points of zero shear stress.

From these plots, the stress trajectories were drawn for the prototype, photoelastic and finite element results for floor 3. These are shown in Fig. 8.28 on page 232.

7.13 THE FRINGE ORDER - PRINCIPAL STRESS DIFFERENCE RATIO

In order to compare the fringe order, which is effectively a measure of the principal stress difference in the photoelastic model, with the principal stress difference obtained from the prototype test, the ratio between the two was found. Since the fringe order in a photoelastic model is proportional to both the principal stress difference and the thickness of the model (see equation 5.1) two ratios were actually found; one for the 10" thick first four storeys and a second for the 8" thick top four floors.

The ratio of the prototype principal stress difference to the photoelastic fringe order was found in the following way.

In Section 7.7 it was explained how stress values
were plotted along lines on the wall common to both the
prototype and the photoelastic model. By numerically integrat-
ing each respective curve the average principal stress
difference or fringe order was found. The average ratio
for each floor was obtained by weighting each individual
ratio from each area of stress readings with the number of
readings. The average ratios for all floors with the same
thickness were then themselves averaged to give the final
ratio. Table 8.1 on page 198 shows the value of the ratios
found in each location on each interstorey area, together
with the weighted averages and the final two ratios.

This table will be considered in detail during the
discussion in the next chapter. However, the key figures
to note are that 1 fringe is equivalent to 2.14 lb./sq.inch
in floors 1 to 4 and to 2.94 lb./sq.inch in floors 5 to 8.

The fringe constant of 66.0 lb./inch as described in
Section 5.9 could have been used to provide the above ratios-
however this figure was based on a calculated load from a
theoretically frictionless lever system and the assumption
that the flanges in the photoelastic model carried no shear.

7.14 THE PLANE STRESS PROGRAM RESULTS

The method of analysis of the shear wall loaded with
the experimentally derived lateral load using the plane
stress program was described in Section 6.8, and the results
were output both on the line printer and on punched cards.

The results for floor 3, the only one examined in
detail, were presented in the same form as the prototype
results except that the contour plot of the principal
stress difference was not smoothed.

Unfortunately the floor slab could not be represented
in the plane stress program as the beam elements (see Section
6.8) designed to represent the flanges could only be speci-
fied running in the Y direction. Had beam elements as floor slabs been available, the stress trajectories, as seen in Fig. 8.28 on page 232 and the symmetrical shape of the contour plot in the spandrel area, as shown in Fig. 8.25 on page 228 would have been modified along the top of the interstorey area.

7.15 THE SHEAR STRESS DISTRIBUTION AND SHEAR FORCES ACROSS THE SPANDREL BEAMS

An important feature of shear wall analysis is the prediction of the vertical shear force resisted by each spandrel beam. The comparison of the prototype, photo-elastic and, where applicable, the finite element results is shown in Fig. 8.3 on page 191, which shows the shear stress distribution along various vertical sections across each of the six spandrel beams. The vertical shear force across each spandrel beam was calculated from the prototype results by averaging the areas under each curve in Fig. 8.3. These results are shown in Fig. 8.4 on page 192.

By calculating the second moment of area of the shear wall, (neglecting the openings) and applying the experimentally derived lateral loading, (see Table 7.10) the shear flow at the centre of the web for each storey was found. This shear flow was converted to a shear force, necessarily acting across the spandrel beam alone in each interstorey area, and these results are also presented in Fig. 8.4.

7.16 FRAME/SHEAR WALL INTERACTION

An analysis was carried out to find the theoretical lateral load on the shear wall \(^{44}\). The lateral load on the effective structure was the inertia load calculated by finding the self-weight of a typical frame and the shear wall at each level and multiplying it by the experimentally derived acceleration of the building at the appropriate level. This process was described in detail in Section 3.6.
The flexibility matrices for the frame and the shear wall were compiled as described in Sections 3.5 and 6.8 respectively - these are shown in Table 7.9.

The analysis of the mathematical effective structure which consisted of 5.05 frames plus one shear wall, with a lateral loading due to 4.03 frames and one shear wall (see Section 3.7 for details of the effective structure) was carried out in the following way, using the Symbolic Matrix Interpretive System. This computer program was discussed in Section 6.10.

1. The flexibility matrices of both one frame and the shear wall were loaded into the computer.

2. The frame flexibility matrix was divided by 5.05.

3. Both flexibility matrices were inverted to give stiffness matrices and were added together to give the total stiffness matrix for the effective structure.

4. The lateral load (inertia loading) vector both for a typical frame and the shear wall were loaded into the computer. The frame load vector was scaled by 4.03 and the two vectors were added to give the total load on the effective structure.

5. The flexibility matrix for the effective structure was formed by inverting the stiffness matrix, and when multiplied by the total lateral load vector the deflected shape of the structure was given as a vector.

6. The deflected shape vector, when multiplied by the stiffness matrices of both the 5.05 frames and the shear wall provided the load vectors for the two interconnected structures - hence the frame-shear wall interaction was found.

The inertia loads acting on the effective structure, as well as the final results of the frame/shear wall inter-
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<td>1.959</td>
<td>-0.056</td>
<td>2.315</td>
<td>8.73</td>
<td>3.10</td>
</tr>
<tr>
<td>8</td>
<td>0.664</td>
<td>0.379</td>
<td>2.176</td>
<td>1.764</td>
<td>0.411</td>
<td>2.65</td>
<td>9.65</td>
<td>3.37</td>
</tr>
</tbody>
</table>

TABLE 7.10 FRAME / SHEAR WALL INTERACTION & THE EXPERIMENTALLY DERIVED LATERAL LOAD ON THE SHEAR WALL
action are given in Table 7.10. These values for the wall are also presented in a graphical form in Fig. 8.1 on page 187, the values being represented by full lines.

In order to clearly show the effect of the frames on the shear wall, a triangularly distributed load was applied to the effective structure and the proportion of the load carried by the shear wall is shown in Fig. 7.9. The reduction in the load carried by the wall at the top of the building was due to the severe incompatibility at the top of the building of the deflected shapes of the frames and the wall under the same loading distribution.
Movement due to foundation rotation and translation.

Fig. 7.8 Deflected shape of shear wall.

Total swing of prototype wall (thou.)

Fig. 7.9 Frame - shear wall interaction example.

Arbitrary load

load carried by shear wall

triangular lateral load applied to effective structure.
CHAPTER EIGHT

COMPARISON OF THE RESULTS AND DISCUSSION

8.1 INTRODUCTION

In the previous chapter the methods involved in the reduction of the prototype data and the presentation of the results were discussed. This chapter deals with the possible reasons behind the difference between the values found for the prototype, the photoelastic model and the finite element analysis.

8.2 COMPARISON OF THE INERTIA AND ACTUAL LOADS ON THE WALL

The actual load was calculated by integrating the plots of shear stress across the building, as described in Section 7.8, and the inertia load on the shear wall was calculated from the masses and accelerations involved, and from the frame/shear wall interaction as discussed in Section 3.6.

In the central graph in Fig. 8.1 the accumulated lateral load due to the inertia loading (the heavy line) is compared with the experimental results. The latter are given in two forms - the crosses refer to the shear force resisted by the web alone and the dots refer to the estimated lateral load carried by both the web and the flanges - the basis of this estimation was also described in Section 7.8.

The general pattern of the inertia and actual prototype loading is the same but at the base of the wall the experimental lateral load is about 40% higher than the calculated inertia load. This indicates that the value of the modulus of elasticity used ($4.43 \times 10^6$ lb./sq.inch) may have been too high; a value of $3.15 \times 10^6$ would bring the results
FIG. 8.1 SHEAR WALL LOADING

- Prototype results
- Experimental load applied to finite element wall
- Deflected shape of wall (expt. results corrected for foundation rotation and translation)
- Accumulated lateral load on shear wall
- Lateral load increments (plotted at mid storey)

FULL LINES Inertia loading.
+ Lateral load carried by web.
- Flanges included.
almost in line. However, although the mass of the building could be estimated to within 10% the end shear wall may have been resisting more of the lateral load of the building than the 5½ bays attributed to it (as discussed in Section 3.7). Shear wall 2, the next in the building as shown in Fig. 3.2 on page 102 had several openings per floor in some cases, and some of the openings were 6'-1" wide compared with the 5'-3½" wide openings in wall 1. (The layout of the two walls is shown in Fig. 8.2.) This would lead to more flexible spandrel beams. Hence the second shear wall was probably more flexible, meaning that the end wall probably carried more of the load than actually attributed to it. An analysis of the whole structure would have given the load on the end shear wall more accurately, but time did not permit a finite element analysis of more than one wall.

The discrepancy between the deflected shape of the building under inertia loading and the experimentally derived deflected shape (shown by the dots) in the left hand graph of Fig. 8.1 also shows that the experimental value is 36% higher at the top of the building. This ties up with the discrepancy between the accumulated lateral loads.

Also shown in this figure is the deflected shape given by the finite element analysis of the shear wall under the experimental loading. These deflections are all slightly higher than the experimental deflected shape (which has been corrected for foundation rotation and translation, as shown in Fig. 7.8 on page 185), the average discrepancy being 13%. A possible reason for the more flexible finite element wall is that floor slabs could not be incorporated in the finite element mesh. In Sections 8.5 and 8.7 they are shown to have a stiffening effect on the spandrel beams.
FIG. 8.2. COMPARISON OF SHEAR WALL OPENINGS
The values that show the largest discrepancies between the inertia load and the experimentally derived prototype load are plotted on the right hand graph in Fig. 8.1. These are the lateral load increments derived from the accumulated lateral load on the shear wall, and they have been plotted at mid-storey for each level. The largest increase in lateral load occurs at level 5. This means that the largest individual lateral load is provided by the floor slab between levels 5 and 6. An initial assumption was that a large mass was situated on this floor slab, but a survey of the building's interior showed that the floor loading at levels 6, 7 and 8 was approximately the same, mainly due to offices and laboratory equipment.

A reason for this high load was probably due to local stiffness differences between the two shear walls shown in Fig. 8.2. The interstorey areas at levels 4 and 5 in wall 2 both have two openings per level, hence the lateral load supplied by the floor slab at level 6 would tend to be carried more by the locally stiffer end shear wall.

8.3 THE SPANDREL BEAM SHEAR FORCES

As described in Section 7.15, the areas under the curves in Fig. 8.3 were used to find the shear forces across the spandrel beams in the prototype wall, and these shear forces are presented in a numerical and a graphical form in Fig. 8.4. Also in Fig. 8.4 the values of the spandrel shear forces obtained by applying the experimentally derived lateral loading to a channel section of the same dimensions as the wall and calculating the mid-section shear flow are shown. This process is also described in Section 7.15. The object of this presentation was to find out how accurately the shear forces across the spandrel beams could be predicted by a simple theoretical method.
FIG. 8.3 VERTICAL SHEAR STRESS DISTRIBUTION ACROSS SPANDREL BEAMS
FIG. 8.4. SHEAR FORCE ACROSS SPANDREL BEAMS DERIVED FROM CALCULATING THE SHEAR FLOW AT THE MID-SECTION OF THE SHEAR WALL

SHEAR FORCE ACROSS SPANDREL BEAMS DERIVED FROM CALCULATING THE SHEAR FLOW AT THE MID-SECTION OF THE SHEAR WALL

Shear force in kips.

580 lb.
1150 lb.
1500 lb.
2500 lb.
3650 lb.
2350 lb.
7700 lb.
0
1
2
3
4

FIG. 8.4. SHEAR FORCE ACROSS SPANDREL BEAMS
Since there was no operational computer program available to perform an elastic laminae analysis on the shear wall, the spandrel shear forces calculated by the simple method in Section 7.1.5 only were considered. (This method assumes that the spandrel beams are infinitely rigid.) Had an elastic laminae analysis been performed, the wall would have been taken as a coupled shear wall for the top 6 storeys as there is no effective spandrel beam in level 2.

Fig. 8.5 shows the result of a typical elastic laminae analysis (4), and if the general shape of this curve is compared with the experimental curve provided by
the prototype spandrel shear forces the two curves can be seen to differ in shape in two places. Firstly, there is a large increase in the shear force carried by the prototype spandrel beam in level 4 and to a lesser extent in level 5. Since the opening in level 5 is offset by over half the opening width this means that the shorter spandrel beams are a lot stiffer and hence attract more load. Moreover, the spandrel beam in level 4 is thicker (10" versus 8") and deeper (5.65 ft. versus 3.94 ft.) than the spandrel beams in the top four storeys. An elastic laminae analysis would not detect this increase in shear force as the laminae are normally considered to have a uniform stiffness up the entire wall, although a computer program of this nature could probably be modified to include stiffness changes such as these.

The second difference in the shape between the curves in Fig. 8.4 and 8.5 occurs at the top of the building. The laminae at the top of the building show a tendency to carry an increasing shear force - this is due to the increasing shearing movement between the two sides of the coupled wall. However, the prototype results show a relative decrease in the shear force across the top spandrel beam and this was probably due to the fact that there was a 15" square hole approximately in the centre of the spandrel beam in level 8 (which passed a steam pipe). This hole would make the prototype spandrel less stiff in shear and in bending, hence causing it to accept less load.

The crude prediction of the spandrel shears as described in Section 7.15, (the results being presented in Fig. 8.4) appears to be fairly accurate. This suggests that the spandrel beams in this particular shear wall are fairly rigid. They are probably proportionally deeper than spandrel beams in many other shear walls as the height of the doorways remains fairly constant in most
buildings but the interstorey height is often less than 12'-6".

8.4 THE SHEAR STRESS DISTRIBUTION IN A FLOOR SLAB

Shear walls are loaded mainly by the floor slabs accumulating the lateral load and transferring it, through shear, to the shear wall. If the floor slab is considered to be a beam then the theoretical shear stress distribution can be found. For instance, the shear stress distribution across a rectangular beam takes the form of a parabola, and in the case of an I-beam or channel section the shear stress at the web-flange junction is dependent on the flange size.

In the Chemistry Block the floor slab was effectively a channel section, the web of the channel being formed by the actual slab, while the flanges were formed by the infilled panels and the upstands beneath the windows: these can be seen in Fig. 3.7 on page 110. By calculating the shear stress distribution across the floor slab, a flat parabola was obtained. The value of the shear stress at the extremities of the slab was only 8% less than the mid-section value.

Although this 8% decrease existed, the shear forces carried by the effective flanges on the floor slab would tend to increase the shear force applied to the edges of the wall, and because of this the distribution of the lateral load across the shear wall due to the floor slab was considered to be uniform. An exact prediction of the distribution of the lateral load could not be made as local stiffness changes in the wall, and also large masses situated close to the wall would affect it.

Thus the prototype shear wall loading distribution could only be estimated while the finite element shear
wall was loaded with uniformly distributed loads. However, in the photoelastic model, the lateral load was applied to the outside edge of the floor slab at the centre of each floor; this can be seen in Fig. 5.7 on page 140. This method of applying the load was used to avoid stiffening the model with the loading device, as discussed in Section 5.4. The shear stress distribution along the shear wall-floor slab junction in the photoelastic model would have a maximum at the centre of the web, decreasing towards the shear wall flange.

8.5 THE POSSIBLE EFFECTS OF THE FLOOR SLAB ON THE SHEAR WALL

Without considering any of the prototype or photoelastic results, the possible effects of the floor slab on the shear wall are as follows.

1. Probably the most important contribution the floor slab makes to the shear wall is in the way in which it stiffens the spandrel beams in flexure. The spandrel beam is effectively a Tee beam when the floor slab is in compression, and hence the section modulus of the spandrel beam is increased. This factor is not usually taken into account in shear wall analyses. A Tee beam is also capable of carrying a higher shear force (at least in the elastic range) because the parabolic shear stress distribution of the shear stress across a rectangular section (uncracked) is modified by the effect of a flange on the top of the beam. This enables the curve of the shear stress distribution across the beam to cover a larger area for the same maximum stress level and the same beam depth.

2. In the ultimate case a Tee beam would probably have a higher shear resistance than a rectangular beam of the same dimensions. There would be a larger area
of concrete to provide aggregate interlock\(^{(45)}\), although the contribution of aggregate interlock after a few stress reversals in a cyclic loading system as found in spandrel beams under earthquake conditions is probably fairly small. There would also be an additional shear strength provided by the dowel action\(^{(45)}\) of the reinforcing bars in the floor slab. In addition, as the floor slab usually extends a distance perpendicularly away from the wall, even if severe cracking occurred in the slab near the wall, it would still have some strength in both flexure and shear further out. Another feature of the slab is that it would limit the size of the cracks in the top of the spandrel beam as the slab would contain reinforcing steel placed parallel to the wall.

3. As most shear walls have thin webs in relation to their width, the floor slabs as well as the flanges provide a restraint against web crippling. Also the web-flange junction is braced by the floor slab, which is normally attached to both.

8.6 THE FRINGE ORDER - PRINCIPAL STRESS DIFFERENCE RATIO

The derivation of the fringe order - principal stress difference ratio was described in Section 7.13 and the ratios for different areas in the shear wall are shown in Table 8.1. Since the ratios provide a basis for stress comparisons they are presented in Table 8.2 in different form. The ratios are expressed as percentage differences when compared with the overall ratio for the thickness involved. Also, for each location on the shear wall the differences have been averaged - these values are shown in the right hand column of Table 8.2. These percentages will be considered in detail during the discussion of the graph comparisons.
<table>
<thead>
<tr>
<th>LOCATION ON SHEAR WALL</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.29 ft. across building</td>
<td>2.34</td>
<td>2.18</td>
<td>2.75</td>
<td>2.39</td>
<td>3.22</td>
<td>3.00</td>
<td>3.58</td>
<td>3.46</td>
</tr>
<tr>
<td>6.44 ft. across building</td>
<td>1.99</td>
<td>2.17</td>
<td>2.31</td>
<td>2.07</td>
<td>2.72</td>
<td>2.79</td>
<td>2.77</td>
<td>2.76</td>
</tr>
<tr>
<td>3.60 ft. across building</td>
<td>2.08</td>
<td>2.07</td>
<td>2.28</td>
<td>2.19</td>
<td>3.08</td>
<td>3.05</td>
<td>3.01</td>
<td>3.05</td>
</tr>
<tr>
<td>0.77 ft. across building</td>
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<td>2.00</td>
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<td>2.81</td>
<td>2.45</td>
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<td>2.64</td>
</tr>
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<td>1.65</td>
<td>1.22</td>
<td>1.79</td>
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<td>1.84</td>
<td>2.92</td>
<td>1.49</td>
<td>2.02</td>
</tr>
<tr>
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<td>1.36</td>
<td>2.07</td>
<td>1.98</td>
<td>2.51</td>
<td>2.22</td>
<td>2.29</td>
<td>2.38</td>
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<tr>
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<td>2.27</td>
<td>1.72</td>
<td>2.97</td>
<td>2.09</td>
<td>3.20</td>
<td>3.45</td>
<td>3.85</td>
<td>2.27</td>
</tr>
<tr>
<td>Second row above door</td>
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<td>1.67</td>
<td>2.30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Third row above door</td>
<td>2.15</td>
<td>1.78</td>
<td>2.54</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top row in spandrel</td>
<td>2.14</td>
<td>3.19</td>
<td>4.67</td>
<td>3.78</td>
<td>3.17</td>
<td>3.15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom row in spandrel</td>
<td>2.14</td>
<td>2.60</td>
<td>3.26</td>
<td>3.32</td>
<td>2.69</td>
<td>2.94</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WEIGHTED AVERAGE</td>
<td>2.07</td>
<td>1.90</td>
<td>2.27</td>
<td>2.30</td>
<td>3.01</td>
<td>3.03</td>
<td>2.89</td>
<td>2.83</td>
</tr>
<tr>
<td>TOTAL AVERAGE</td>
<td>2.14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.94</td>
</tr>
</tbody>
</table>

**TABLE 8.1** VALUES OF THE PRINCIPAL STRESS DIFFERENCE/FRINGE ORDER RATIO
<table>
<thead>
<tr>
<th>LOCATION ON SHEAR WALL</th>
<th>FLOOR NUMBER</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>AVERAGE DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.29 ft. across building</td>
<td>1</td>
<td>+9</td>
<td>+2</td>
<td>+28</td>
<td>+11</td>
<td>+9</td>
<td>+2</td>
<td>+22</td>
<td>+18</td>
<td>+12.6</td>
</tr>
<tr>
<td>6.44 ft. across building</td>
<td>-7</td>
<td>+2</td>
<td>+8</td>
<td>-3</td>
<td>-7</td>
<td>-5</td>
<td>-5</td>
<td>-6</td>
<td>-2.9</td>
<td></td>
</tr>
<tr>
<td>3.60 ft. across building</td>
<td>-3</td>
<td>-3</td>
<td>+7</td>
<td>+2</td>
<td>+5</td>
<td>+4</td>
<td>+3</td>
<td>+4</td>
<td>+2.4</td>
<td></td>
</tr>
<tr>
<td>0.77 ft. across building</td>
<td>+5</td>
<td>-5</td>
<td>-7</td>
<td>-9</td>
<td>-4</td>
<td>-16</td>
<td>-8</td>
<td>-10</td>
<td>-6.7</td>
<td></td>
</tr>
<tr>
<td>Left side of door</td>
<td>-23</td>
<td>-43</td>
<td>-16</td>
<td>-8</td>
<td>-37</td>
<td>-1</td>
<td>-49</td>
<td>-31</td>
<td>-26.0</td>
<td></td>
</tr>
<tr>
<td>Right side of door</td>
<td>-12</td>
<td>-37</td>
<td>-3</td>
<td>-7</td>
<td>-15</td>
<td>-24</td>
<td>-22</td>
<td>-19</td>
<td>-17.4</td>
<td></td>
</tr>
<tr>
<td>First row above door</td>
<td>+6</td>
<td>-19</td>
<td>+40</td>
<td>-2</td>
<td>+9</td>
<td>+17</td>
<td>+32</td>
<td>-22</td>
<td>+7.5</td>
<td></td>
</tr>
<tr>
<td>Second row above door</td>
<td>-10</td>
<td>-22</td>
<td>+8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-8.0</td>
<td></td>
</tr>
<tr>
<td>Third row above door</td>
<td>+1</td>
<td>-17</td>
<td>+19</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>+1.0</td>
<td></td>
</tr>
<tr>
<td>Top row in spandrel</td>
<td>0</td>
<td>+49</td>
<td>+59</td>
<td>+29</td>
<td>+7</td>
<td>+7</td>
<td>+7</td>
<td>+25.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom row in spandrel</td>
<td>0</td>
<td>+22</td>
<td>+11</td>
<td>+13</td>
<td>-12</td>
<td>0</td>
<td></td>
<td>+5.7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 8.2**  PERCENTAGE DIFFERENCES BETWEEN THE INDIVIDUAL AND OVERALL RATIOS.
8.7 COMPARISON OF THE PRINCIPAL STRESS DIFFERENCES

8.7.1 Preliminary. The main comparison between the prototype, the photoelastic model and the finite element results is based on the values of the principal stress difference, as described in Section 5.2. The results are presented graphically in two ways, firstly as a series of plots along common lines of measurement on the shear wall, and secondly as isochromatics or contour plots of the principal stress difference. They are also presented in a tabular form as described in the preceding section.

Fig. 8.6 is a key to the following plots of the principal stress difference, shown in Figs. 8.7 to 8.16. The circles representing the finite element results are only found on some of the graphs for floor 3 as only this floor was examined in detail by this method. The description of each plot (in the lower left corner) can be linked with the positions shown in Fig. 4.4 on page 121.

The plots of the principal stress difference are in terms of the fringe order - this is the vertical ordinate in each graph. (All plots have the same boundary height to enable a quick comparison to be drawn). The position of the opening in the shear wall is shown in Figs. 8.7 to 8.10, while the single line in Figs. 8.11 and 8.12 marks the top of the openings. Figs. 8.13 to 8.16 have the opening shown; also the vertical boundaries of these plots have the same position on the shear wall.

8.7.2 The Plots Across the Building. An overall comment can be made regarding Figs. 8.7 to 8.10, the plots of the principal stress difference across the building. The values from the prototype and the photoelastic results are mostly within 20%. This also applies to the values from the finite element analysis of floor 3.
SEE FIG. 4.4 ON PAGE 121 TO FIND THE KEY TO THE GRAPH TITLES

- PROTOTYPE
- PHOTOELASTIC MODEL
- FINITE ELEMENT ANALYSIS

SCALING FACTORS:

Floors 1 to 4 –
1 fringe = 2.14 lb./in.²

Floors 5 to 8 –
1 fringe = 2.94 lb./in.²

FIG. 8.6. KEY TO THE FOLLOWING GRAPHS OF PRINCIPAL STRESS DIFFERENCE
FIG. 8.7 9.29 ft. ACROSS THE BUILDING
FIG. 8.8. 6.44 ft. ACROSS THE BUILDING
FIG. 8.9. 3.60 ft. ACROSS THE BUILDING
FIG. 8.10. 0.77 ft. ACROSS THE BUILDING
FIG. 8.11  LEFT SIDE OF THE DOOR
FIG. 8.12 RIGHT SIDE OF THE DOOR
FIG. 8.13 FIRST ROW ABOVE THE DOOR
FIG. 8.14 SECOND AND THIRD ROWS ABOVE THE DOOR
FIG. 8.15 TOP ROW IN SPANDREL
FIG. 8.16 BOTTOM ROW IN SPANDREL
shown in Figs. 8.7 to 8.9; the finite element results can be seen to favour the prototype results more than the photoelastic results.

These plots are mainly concerned with the stress distribution in the areas away from the opening and probably the major differences between the results is due to the distribution of the lateral load across the shear wall. The distributions involved in the prototype wall, the photoelastic model and the finite element analysis were discussed in Section 8.4.

8.7.3 The Plots Up the Sides of the Openings.

Figs. 8.11 and 8.12 show the values of the principal stress difference up a line on the right and left sides of the door opening respectively. In both these series of plots the photoelastic analysis usually overestimates the stresses at the bottom of the opening and underestimates near the top corner of the doorway. This fact indicates that the floor slab is much stiffer in the prototype than in the photoelastic model: the width of slab modelled in the latter was 5'-0" each side of the wall. Hence much more than 5'-0" of the floor slab must act with the prototype shear wall.

Apart from this discrepancy the stress values were normally within 25% of each other except in the case of floor 2 where the photoelastic results overestimated the prototype results by over 100% in places.

8.7.4 The Plots in the Spandrel Beam Area. The plots of the principal stress difference above the doorway (first, second and third rows above the door, and top and bottom rows in the spandrel) are shown in Figs. 8.13 to 8.16.

The shape of the curves formed by the two sets of points in each graph is similar, but in general the photo-
elastic analysis underestimated the prototype values. This is particularly true in the local areas directly above the corners of the openings; in some cases the photoelastic results are less than 50% of the prototype results, as shown in Fig. 8.13.

In the third row above the door on floor 3 (Fig. 8.14), the finite element results can be seen to compare well with the photoelastic results.

8.7.5 The Discussion of the Tabulated Results.

The derivation of the values in Table 8.2 was discussed in Section 8.6, and this table can be used to put values on the comparison trends that have just been discussed. Two main points emerge, as follows:

1. The sections taken across the shear wall 9.29 ft. above the floor slab show an increase of 12.6% in the principal stress difference in the prototype when compared with the photoelastic model. Also from Table 8.2 the corresponding value for the section 0.77 ft. above the floor slab is -6.7%. The total difference of 19.3% can be regarded as the difference in the average horizontal shear stress across the web of the wall at the two different heights above the floor slab. This is because the horizontal shear stress is found by modifying the principal stress difference by \( \frac{1}{2} \sin 2\theta \) where \( \theta \) is the angle of the orientation of the principal stresses. (The orientation of the principal stress along the sections involved is approximately the same in both the prototype and the photoelastic model as seen in Figs. 8.26 and 8.27.)

This difference between the two sections is also shown in the central plot in Fig. 8.1. If the crosses are considered it can be seen that the values at 9.29 ft. above the floor slab are generally higher than the values close to the slab.

The obvious reason for the decrease in the shear stress and hence the lateral load resisted by the web of
the wall as the height of the section above the floor slab decreases is that the flanges of the prototype wall are carrying more of the lateral load. This increase in the shear force carrying capacity of the flanges is probably due to two factors.

(a) The floor slab prevents shear deformations in the flanges in the direction of the web of the wall - hence they are stiffer and can carry more shear.

(b) The shear wall flange area is increased above each floor slab by the upstand beneath the window openings in the sides of the building. (See Fig. 3.7 on page 110.) This upstand, which is also stiffened against shear deformations by the floor slab, enables an extra proportion of the lateral load to be carried in the flange.

The reason that an increase in the shear force carrying capacity of the flanges near the floor slabs was not obvious in the photoelastic model, was probably due to the fact that the model flanges were idealised as described in Section 5.7.5. Since they were thicker, and not as wide as the prototype flanges (and had a constant width) their shear force carrying capacity would be relatively constant throughout each interstorey area. Thus, although the actual dimensions of the flanges can be theoretically shown to have little effect on the shear stresses in the web, the flanges should be modelled exactly to take into account the effects found in the prototype.

An attempt was made to calculate the lateral load carried by the prototype shear wall flanges, as described in Section 7.8. The results of these calculations are shown in Fig. 8.1 as dots in the central plot, and the vertical distribution of the lateral load within each storey is slightly more even. This crude calculation showed that an average 7% of the lateral load was carried by the flanges.

Another reason for the decreasing lateral load
carried by the shear wall web could be offered by the possible truss action in the shear wall, as discussed in Section 8.11.

2. The second point to emerge from the tabular comparison was that the photoelastic analysis overestimated the stress levels up the sides of the openings by an average of 22% and underestimated the stress levels in the spandrel beam area by an overall average of 8.5%. The largest deviation between the two sets of results was in the top row in the spandrel where an average difference of 25.1% was recorded.

These stress differences could have been partly due to the fact that there was a difference between Poisson's ratio between the reinforced concrete prototype and the Araldite photoelastic model. However, as discussed in Section 5.11 the model-prototype comparisons should be within 7%.

The proportionally higher stresses in the prototype must be mainly due to the fact that the spandrel beams in the prototype are proportionally stiffer than their photoelastic counterparts. One would normally assume that the more highly stressed concrete prototype spandrel beams were less stiff than their modelled counterparts as they were cracked in places and hence their section modulus was lower.

The only way in which the spandrel beams could be proportionally stiffer in the prototype shear wall would be for them to have a wider flange formed by the floor slab than the 10'-0" width modelled in the Araldite shear wall. The possible effect of the floor slab was discussed in Section 8.5, and it appears, from the prototype results, to have a greater effect than previously thought on the stiffness of the spandrel beams.

The effect of the reinforcing steel in the spandrel beams was shown to stiffen them by less than 8%. This was done by calculating the second moment of area of the deep
prototype spandrel beam (uncracked) with an assumed 5'-0" of floor slab acting on both sides of it. The second moment of area was calculated both for a pure concrete section and for a transformed section, using a modular ratio of 9.

The floor slab similarly affects the principal stress values up the sides of the openings, as discussed in Section 8.7.3 and the trend is reinforced by the fact that an overall difference of 22% exists between the prototype and the photoelastic results in these areas.

All of the differences discussed in this section can be seen by referring to the isochromatic comparisons, Figs. 8.17 to 8.24. These will be discussed in detail in Section 8.9.

8.8 THE SHEAR STRESS DISTRIBUTION ACROSS THE SPANDREL BEAMS

The effect of the floor slab on the spandrel beams in the prototype can be seen by referring to Fig. 8.3, the plots of the vertical shear stress across the spandrel beams.

In most cases the shape of the curves drawn through the prototype results was different from the curves traced out by the photoelastic results in that the former values were lower near the bottom edge of the spandrel beam and higher at the junction of the beam and the floor slab. If a Tee beam (uncracked) is analysed to plot the shear stress distribution across it, then a wider flange (or floor slab) draws the peak of the shear stress distribution curve towards it - exactly the trend of the prototype results.

In floor 3 in Fig. 8.3 the finite element (plane stress) results tie in well with the photoelastic values except at the top of the spandrel beam. This is because there was no floor slab acting as a flange in the finite element analysis.
8.9 DISCUSSION OF THE ISOCHROMATICS

8.9.1 Preliminary. Figs. 8.17 to 8.24 show the contour plots of the prototype principal stress differences compared with the half fringe order isochromatic photographs of the photoelastic model, one for each inter-storey area. Fig. 8.25 shows the principal stress difference contour plot of the finite element results compared with the photoelastic results - this plot has not been smoothed like the prototype results.

The prototype contour plots cannot cover the whole area as measurements could not be taken outside the boundaries of the plots as discussed in Section 7.10. The boundaries of the prototype contour plots can be seen on the photoelastic photographs by observing the appropriate grid lines. The grid lines, of course, follow the positions of the lines of gauge pad triangles on the prototype.

Due to the printing process used, the numbers on the plots giving the value of the fringe orders are a little difficult to read, but these values can be determined if necessary by referring to the plots of the principal stress differences in Figs. 8.7 to 8.16.

8.9.2 Floor 1. (Fig. 8.17). In this figure the contours differ somewhat in position, although the fringe orders are compatible. This is probably due mainly to the heavy, and consequently stiff, holding base that was cast on the Araldite model as discussed in Section 5.7.5. Also, in the prototype plot, no contours were drawn above the opening as only a single row of gauge pad triangles were measured; this meant that this area could not be triangulated.

8.9.3 Floor 2. (Fig. 8.18). In general the contour patterns show a good correlation, except to the left
side of the opening. The photoelastic fringe order is higher in this area than the prototype results; this is probably due to the relatively stiffer flanges in the model, as discussed in Section 8.10, locally stiffening the area and attracting more load. The contour plot shows particularly good correlation near the centre of the wall; the high stresses being due to the openings directly above and below the interstorey area.

8.9.4 **Floor 3 (Fig. 8.19).** These two plots show a good correlation. It is interesting to note the lowly stressed areas 45° diagonally up and away from the top corners of the opening in the photoelastic photograph. Should it be necessary to pierce the wall for service ducts then holes in these areas would have little effect on the structure.

8.9.5 **Floor 4 (Fig. 8.20).** Once again, the general correlation between the contour patterns is good, except that the fringe order is higher in the spandrel beam area in the prototype - this was discussed in Section 8.7.5. However, a local contour peak exists in the prototype spandrel beam above the left corner of the opening; this may have been due to a strain reading across a crack in the prototype not being corrected sufficiently.

A lowly stressed area also exists in the photoelastic model, but this time it is further along the diagonal line mentioned in the previous section.

8.9.6 **Floor 5 (Fig. 8.21).** In the prototype plot another locally high stress can be seen to the lower left side of the door - no obvious reason exists for this peak. As discussed in Section 8.7.5 the spandrel stresses are higher in the prototype but in general, the pattern in the spandrel area is the same. Once again an area with a fringe order of less than \( \frac{1}{2} \) exists in the
photoelastic model 45° away from the top right corner of the opening, and the prototype contours also show this trend.

8.9.7 **Floors 6 and 7 (Fig. 8,22 and 8,23).** The correlation between the contour patterns is a little harder to define as, with the decreasing stress, the contour spacing is larger. Probably the most important point to note in these two figures is the presence of areas with a fringe order of zero in the now familiar zone, 45° up and away from the corners of the opening, in the photoelastic model. Of course, this means that the principal stresses have the same value, and the area is known as an isotropic area (or point). These isotropic points were discussed in Section 7,12.

8.9.8 **Floor 8 (Fig. 8,24).** This area carried little overall stress and in addition there was a gap in the measurements in the prototype spandrel beam as a steam pipe was run through a 15" square hole. No isotropic points existed, although there were the usual lowly stressed areas.

8.9.9 **Floor 3 (Fig. 8,25).** This figure shows the comparison between the finite element and the photoelastic results for floor 3. However, as discussed in Section 6,8, the plane stress program had no provision for floor slabs. This resulted in the symmetrical pattern in the spandrel beam area having its maximum fringe order at the centre of the beam rather than closer to the floor slab as in the case of the photoelastic mode.

Apart from this difference, the general correlation between the two sets of contours was good, both in the shape of the pattern and the fringe order.
FIG. 8.18. FLOOR 2 ISOCHROMATICS
FIG. 8.22  FLOOR 6  ISOCROMATICS
FIG. 8.23  FLOOR 7  ISOCHROMATICS
FIG. 8.25  FLOOR 3  ISOCROMATICS  
PHOTOELASTIC AND FINITE ELEMENT RESULTS
8.10 **THE STRESS TRAJECTORIES**

Figs. 8.26 and 8.27 show the orientation of the principal stresses as crosses representing their direction for both the prototype and the photoelastic model respectively. In addition these figures show the position of each reading taken.

From these plots, (done by the IBM 1627 Autoplotter) and a similar plot from the finite element analysis, the stress trajectory curves were constructed for floor 3. These are shown in Fig. 8.28 as a comparison between the prototype, the photoelastic model and the plane stress program results.

The three orthogonal sets of curves show a remarkable similarity around the opening, but in the areas close to the shear wall flanges, the photoelastic principal stress orientation is influenced more by the direct stresses due to the bending moment resisted by the shear wall at floor 3. This results in the orientation of the principal stresses being less inclined to the vertical than in the other two plots.

The difference between the various values of \( \mu \), Poisson's ratio, which were assumed to be 0.15 in the prototype and the plane stress analysis, and which was approximately 0.35 in the Araldite model, is the probable cause of the differences found in these areas. The wall would be relatively stiffer in shear in the former two cases as the shear modulus \( G \) is relatively higher. This is shown by the expression

\[
G = \frac{E}{2(1+\mu)}
\]

However, the direction of the principal stresses in the critical spandrel area of the prototype shear wall
FIG. 8.26  PRINCIPAL STRESS ORIENTATION  PROTOTYPE RESULTS
FIG. 8.27  PRINCIPAL STRESS ORIENTATION  PHOTOELASTIC RESULTS
FIG. 8.28. STRESS TRAJECTORIES  FLOOR 3
is shown to be accurately predicted by either the photoelastic method or the plane stress analysis.

8.11 THE TRUSS ACTION IN A SHEAR WALL

In Section 8.7.5 the reason for an increased lateral load carried by the shear wall flanges close to the floor slab was discussed, and in Figs. 8.9 and 8.10 a sharp increase in the shear stress in the wall close to the right hand flange can be seen. It was not possible to take stress measurements close to the left hand flange because of interference by a service duct.

From the plot of the principal stress difference up a vertical line formed by the gauge pad groups 0.49 ft. from the right hand flange, as shown in Fig. 8.29, peaks can be seen to occur just above each floor slab. These peaks, coupled with the plot of the prototype principal stress orientation indicates that there may be a truss action in the shear wall resisting lateral loads. The horizontal and vertical members of the truss are formed by the floor slabs and shear wall flanges respectively, while the diagonal members of the truss are effectively formed by bands of concrete in the web of the wall—these struts and ties must pass through the spandrel beams. Fig. 8.30 shows a possible truss structure within the shear wall.

During the test on the shear wall, diagonal cracks in the web at the extreme corners of the wall were noticed. These cracks ran from points up to 10'-0" away from where the roof and the basement floor slabs met the shear wall flanges, and they indicated tension cracks due to the effective ties in the web of the wall being stressed too highly, particularly where they were caused to concentrate near the flange-floor slab junction.
FIG. 8.29 EXPERIMENTAL PRINCIPAL STRESS DIFFERENCE IN WEB 0.49 FT. FROM FLANGE

FIG. 8.30 POSSIBLE TRUSS ACTION
This truss action, however, would not become significant as far as lateral load resistance is concerned until the shear wall had been severely cracked. This is because a truss is more flexible than an uncracked diaphragm of the same dimensions and the load carried by each action must be distributed according to the stiffness. In Fig. 8.30 two sets of diagonal members have been drawn, but after cracking the diagonal members would only act efficiently in compression.

To enable this truss action to take place after cracking the tension forces due to the effective strut meeting the flange-floor slab junction must be able to be resisted. The tension forces in the flange would be catered for by the vertical steel designed to resist shear wall bending. However, a floor slab is comparatively lightly reinforced in the direction of its support (the wall). Also the bars can only be anchored in the shear wall flange, which can be in tension, thus providing a poor anchorage. Another problem is that the effective compression strut in the web of the wall meets the flange-floor slab junction at a relatively local point, hence the tension forces in the latter two members are very concentrated.

Careful design is needed therefore, to enable the potential reserve strength of a shear wall due to the truss action to be realised.
CHAPTER NINE

CONCLUSIONS AND FUTURE CONSIDERATIONS

9.1 INTRODUCTION

This chapter discusses the conclusions drawn from the comparisons made in the preceding two chapters, and outlines possible fields of research which could be continued following the trend laid down in this text.

9.2 CONCLUSIONS DRAWN FROM THE STRESS COMPARISONS

In the preceding chapter the comparison of the principal stress difference between the prototype, the photoelastic model and, for floor 3, the finite element analysis showed that, in general, an accurate assessment of the elastic stress distribution in an irregular shear wall can be made by using either of the latter two methods. It must be remembered, however, that the photoelastic technique is not regarded as a commonly used method for the design of irregular shear walls, whereas the finite element approach is.

Two main points have to be considered when attempting an analysis of this type.

The first, and less important point is that the distribution of the lateral load across the shear wall due to the floor slab cannot be accurately predicted. In Section 8.4 a reason justifying the use of a uniformly distributed load was put forward, and in the case of a floor slab without any effective flanges a parabolically distributed load could be justified in a similar manner. This loading distribution can be easily controlled in a finite element analysis, but in the case of a photoelastic model the loading method has to be designed so that the model is not locally strengthened by the loading device.
However, the actual distribution of the lateral load across the shear wall mainly affects the shear stress distribution away from the openings, and this is not usually critical as far as the shear strength of the wall is concerned.

The second and major point to emerge from these stress comparisons is the effect the floor slab has on the shear wall. It stiffens the spandrel beams, mainly in flexure, causing them to attract more vertical shear, it reduces the stress level at the bottom of the openings, and stiffens the web-flange junction of the shear wall, enabling the flanges to carry a higher proportion of the lateral load. The floor slabs may also act as horizontal members in a truss, thus stiffening the whole shear wall, and possibly providing a reserve of strength in the ultimate condition.

9.3 **THE EFFECTIVE WIDTH OF THE FLOOR SLAB**

From the comparison of the prototype and the photoelastic results in the spandrel beam area it was apparent that the width of the floor slab was underestimated in the photoelastic model. A crude method of predicting the effective width of slab to be used in subsequent analyses or models would be to double the width of the floor slab used in the photoelastic model. This would mean the width of the effective floor slab would be 20 times the depth of the slab, or two times the width of the opening, running each side of the wall where applicable.

Barndard and Schwaighofer (16) in an elastic model study of a coupled shear wall with horizontal floor slabs alone acting as spandrels, found that the whole width of the bay between the shear walls (in the structure they considered) should be used when calculating the spandrel stiffnesses. This does not necessarily apply to Tee
beam type spandrel beams, but it does indicate that a large effective slab width must be used.

A computer analysis using the plane stress program modified to accept horizontal beam elements as floor slabs may provide a method for finding the effective width of the floor slab in this particular structure. Comparisons with the results of the prototype tests could be made in two ways. Firstly, the deflected shape of the finite element wall, (loaded with the experimentally derived prototype loading) could be compared with the deflected shape of the prototype shear wall, as shown in Fig. 8.1 on page 187. Secondly, the vertical shear stress distribution across the spandrel beams provided by the finite element analysis could be compared with the prototype results shown in Fig. 8.3 on page 191.

9.4 THE COMPARISON OF THE DEFLECTED SHAPES

As discussed in the previous section, the absence of the floor slabs in the finite element analysis of the shear wall probably accounted for it being more flexible than the prototype. However, a feature of the experimentally derived deflected shape, as shown in Fig. 8.1 on page 187, warrants discussion.

There is a relative reduction in slope at level 5. This can be explained by observing the high shear force resisted by the spandrel in floor 4 as shown in Fig. 8.4 on page 192. This high shear force would result in a large reverse moment applied to each side of the shear wall at this level, resulting in a decrease in the slope of the wall due to bending. This effect is not obvious in the finite element solution of the structure but the addition of floor slabs may produce it.
9.5 **THE ULTIMATE LOAD RESISTANCE OF A SHEAR WALL**

Paulay\(^{(4)}\), as mentioned in Section 1.2, has completed an extensive review of shear walls and has carried out an experimental program to find the ultimate strength of spandrel beams (rectangular) in coupled shear walls; since the spandrel beams are normally short and deep, normal methods of reinforced concrete post-elastic analysis cannot be used.

Paulay expanded his experimental findings to provide a design method for coupled shear walls under an ultimate loading condition, but this method is limited in that only regular, vertically in line openings can be considered. However, his basic philosophy that the spandrel beams in a coupled shear wall should crack and fail first, thus allowing any extra load to be resisted by the two vertical cantilevers of the shear wall is very sound. This should be applied to irregular shear walls if possible.

A photoelastic model cannot give the ultimate load stress conditions in a structure, but it certainly can indicate the probable method of failure. Similarly the plane stress program, which is also a purely elastic method of analysis can only indicate the probable failure. However, finite element techniques have been developed to deal with cracking\(^{(46)}\) in concrete, and plastic behaviour\(^{(39)}\).

9.6 **THE EFFECT OF FLOOR SLABS IN THE ULTIMATE CASE**

In Section 8.11 the possible truss action of the shear wall was discussed and obviously, if the floor slabs are designed against a tension failure close to the shear wall flanges, they can contribute to the ultimate lateral load resistance of the shear wall. As far as the spandrel beams are concerned the floor slabs can have a two-fold effect.
According to Paulay's experimental work, the floor slabs will have little direct effect on the ultimate shear resistance of the spandrel beams. This is because both the top and the bottom faces of the spandrel beam are in tension under the ultimate load - hence only the floor slab reinforcement would be operational due to the severely cracked concrete. However, a higher shear force across the spandrel beam may be required to produce this severely cracked state. Aggregate interlock would probably be negligible, as discussed in Section 8.5, and also the contribution of the dowel action in the floor slab would be relatively small because the shear force causing the deformation of the slab reinforcement would have to flow from the web of the spandrel beam into the flange formed by the floor slab. This flow has been shown to be dependent on the action of the vertical stirrups in the spandrel beam in the ultimate case (4). Also the slab reinforcement would be relatively light.

However, the floor slab has an indirect action that would increase the ultimate shear force resistance of the spandrel beams as follows.

The tensile forces in the top and bottom faces of the spandrel beams under an ultimate load are caused by an extension in the length of the beam (4). A floor slab would be relatively stiff in tension due to its large effective width, and hence the spread of the spandrel beam would be restrained. This would lend a type of longitudinal prestressing action to the spandrel beam under ultimate conditions. Hence the crack sizes would be limited and perhaps some of the concrete in the vital shear resisting areas may be retained in compression, thus providing a higher ultimate shear force resistance.
Again the usually neglected floor slab can provide an extra measure of strength to the shear wall under ultimate load conditions.

9.7 THE PROBABLE MODE OF FAILURE OF THE PROTOTYPE WALL

The probable action of the shear wall under ultimate loading conditions can best be described if the intensity of the lateral loading is considered to increase slowly, allowing the various components of the wall to fail sequentially.

From the stress levels involved in the spandrel beams it is obvious that the first beam to fail would be that in floor 5, closely followed by the spandrel beam in floor 4. The latter would follow closely as it would have to absorb some of the shear force normally carried by the spandrel in floor 5 as well as sustain the previous high load it was carrying. This mode of failure is also predicted by the shape of the shear flow curve in Fig. 8.5 on page 193 if the prototype wall is considered to be coupled above level 2.

Failure of the spandrels would probably continue in the following order under an increasing load. Spandrel 3 would be followed by spandrels 6 and 7. However the spandrel beam in floor 8 would probably fail at about the same time as the zone in floor 2 between the openings in floors 1 and 3 became severely fractured with diagonal cracks.

At this stage the shear wall would begin to act as two individual walls coupled by the fractured spandrel beams much reduced in strength. The two sides of the complete shear wall would probably suffer diagonal cracking with tension cracks due to the bending moment applied to each individual wall occurring at the sides of the openings. The substantial flanges on the shear wall
would probably limit the tension cracks in that zone.

The possible truss action of the shear wall, described in Section 8.11, would also become more apparent.

However, this particular building would stand a fair chance of falling on its side before major damage was sustained due to the lateral loading under severe earthquake. Fig. 7.8 on page 185 shows that during the prototype test 67% of the movement at the top of the building was due to foundation rotation and translation. If this is an indication of the behaviour of the building under earthquake conditions it is possible that a slip circle failure could occur in the clay beneath the building as it was rocking on one side of its foundations.

9.8 FUTURE SHEAR WALL RESEARCH

As far as irregular shear walls are concerned, a micro-concrete model of the Chemistry Block shear wall tested to its ultimate load condition would provide a valuable correlation with the purely elastic case considered in this text. A series of models with different widths of floor slabs would provide information on the effect of the slabs on the ultimate strength. The micro-concrete models could be compared with analyses performed on the structure using a finite element program designed to cope with ductility in the reinforced concrete.

The vertical prestressing of shear walls could be investigated. From an analytical approach it would appear that vertical prestressing could increase the ultimate strength of a shear wall by providing a greater resistance to cracking caused by tension stresses (due to both bending and shear) in the majority of the web of the wall. By retaining reinforced concrete spandrel beams the ductility afforded by these beams would reduce
the bending moment applied to each half of a coupled shear wall and at the same time they would absorb some of the energy during an earthquake loading.

Different placement of the reinforcing steel could be investigated. The stress trajectories shown in Fig. 8.28 on page 232 provide a guide to the direction of the reinforcing bars to resist both the bending and the shear stresses encountered. However, the practical problems of placing diagonal reinforcing in a shear wall would have to be investigated. The inclusion of some heavy diagonal bars in the spandrel beams may do much to enhance their ultimate strength.

The curtailment of the reinforcing bars in the spandrel beams should be given some thought. They should be stopped in areas of low stress; this is because the anchorage of reinforcing bars is severely reduced in an area with high concrete tensile stresses (4). Diagonal bars through the spandrel beams could be curtailed in the comparatively lower stressed areas at the sides of the openings. In the case of the normal horizontal bars through the spandrel beams, they should be carried through the entire width of the wall—the extra cost involved would be negligible compared with the cost of the shear wall. By having continuous bars the anchorage problem would not exist and also some heavy steel would be available to resist the diagonal tension near the flange-wall junction due to the possible truss action in the shear wall.

Probably the most important investigation in shear wall design that could be carried out using photoelastic models would be to find the effect of different shaped openings. In most shear walls the web is pierced by corridor or lift access doorways which are often necess-
arily rectangular. However, an octagonal, or even circular opening in the concrete could be framed to provide a rectangular opening after construction, while the actual structural shape is optimised to provide a more even stress distribution and possibly a greater ultimate strength.

The casting of a stiffening beam around the openings could also be investigated by using a photoelastic model. A return of this nature, particularly above the opening, may do much to strengthen the spandrel beam as it would then become an I-beam. With the extra reinforcing steel possible due to the lower flange in the beam the crack widths would be reduced and the tendency of the spandrel beam to elongate under ultimate conditions would also be restricted further by this steel. Furthermore, such a return around the openings would cover the highly stressed areas at the corners of the openings.

However, according to Paulay (4) the shear resistance of the spandrel beam should not govern the strength. A shear failure is a "brittle" failure while a flexural failure affords the most ductility - a feature desirable in shear walls under an ultimate load. Thus the benefits of a return around the opening would have to be critically studied with the ultimate behaviour of the spandrel beam in mind.

9.9 CONCLUSION

Although the ultimate strength behaviour of a structure such as a shear wall is of prime importance, the elastic analysis is by no means a feature of the past. The elastic analysis provides the key to the behaviour of a structure under an ultimate load and, in many cases, the ultimate load analysis simply cannot be performed due to a lack of knowledge regarding the ultimate behaviour.
Even if such a method of analysis exists, the ordinary designer is often forced to use existing and well tried methods because of the lack of computing facilities or the lack of his specialist knowledge.

As far as shear walls are concerned, the frame/shear wall interaction cannot be reasonably assessed other than for the elastic case, and also a knowledge of the working load behaviour is required to enable deflections and cracking to be controlled.

In conclusion, it can be said that the correlation between the finite element analysis and the photoelastic study was generally good. Two main differences existed, however. The first was that the absence of a floor slab in the finite element mesh affected the stress distribution in the spandrel area. The second difference was in the orientation of the principal stresses near the shear wall flanges; this was probably due to the difference between the values of Poisson's ratio. Both the finite element analysis and the photoelastic study in general showed a good correlation with the prototype shear wall.

Thus, as a standard method for the elastic design of irregular shear walls, the finite element method of analysis can be used with confidence - providing suitable finite elements can be incorporated in the mesh to enable the floor slabs to be mathematically modelled.
APPENDIX A

THE OPERATION OF THE DYNAMIC STRAIN BRIDGE
AND THE PEN RECORDER

A.1 INTRODUCTION
The operation of the Dynamic Strain Bridge to record and calibrate dynamic strains with a frequency close to 2.71 cycles per second is discussed in this Appendix.

A.2 THE GENERAL PROPERTIES OF THE DYNAMIC STRAIN BRIDGE
Although the Dynamic Strain Bridge is primarily designed for measuring dynamic strains from four active strain gauges, it has provision for two different configurations of two external strain gauges if required, with dummy 120 Ω resistances inside the Bridge to complete the Wheatstone bridge. The connections for different bridge configurations are shown in Fig. A1. Also there is provision to use the Dynamic Strain Bridge as a Direct Current Amplifier - this can be done by plugging the input signal into the 'phone jack in the centre of the input terminals as seen in Fig. A2 and in Fig. 2.2.9, the circuit diagram, on page 61.

Although the filters are specifically designed to enable strain frequencies of 2.71 cycles per second to be recorded, the band width can be altered to suit other requirements by changing the capacitors C1 and C2 as seen in Fig. 2.2.9. The gain and input impedance of the amplifier can also be varied by changing the values of the resistances associated with the Nexus Operational Amplifier if required.

A.3 THE PREPARATION OF THE DYNAMIC STRAIN BRIDGE
The Bridge is prepared to take dynamic strain
FIG A1. EXTERNAL STRAIN GAUGE CONNECTIONS FOR DIFFERENT MOD
SWITCH POSITIONS

FIG. A2. THE CONTROLS OF THE DYNAMIC STRAIN BRIDGE
readings in the following way: the controls referred to can be seen in Fig. A2.

1. Position the Run-Calibrate-Trim switch (R.C.T.) in the "Trim" position and the output control in the "unfiltered meter" position.

2. Switch the Bridge on.

3. Check the battery voltages. This is done by turning the lower meter control clockwise through three positions; the meter needle must swing at least 40 divisions from the centre. The battery checks in order are:
   a. +6V to the regulated power supply.
   b. +15V to the amplifier.
   c. +6V to the regulated power supply.

Replace any batteries if necessary. Return the switch to its left hand (run) position.

4. After 5 minutes warm up time, trim the amplifier. With the gain fully up (clockwise) and the meter sensitivity on high (the top control in the meter controls fully clockwise), adjust the trim control until the meter needle is central. If the meter cannot be zeroed adjust the preset potentiometer inside the Bridge. Return the meter sensitivity to its least sensitive position.

5. Attach the strain gauges to the terminals as shown in Fig. A1 and position the mode switch accordingly.

6. Turn the R.C.T. switch to "Calibrate" and check that the bridge can be balanced by using both the fine and coarse balance controls.

7. Turn the R.C.T. switch to "Run" and also check that a balance can be obtained.

8. Check the amplifier trim again, and do so every 30 minutes during the readings.
9. Return the R.C.T. switch to "Calibrate" and balance the bridge.

A.4 THE PREPARATION OF THE PHILIPS OSCILLOSCRIPT

With the Dynamic Strain Bridge thus prepared, the pen recorder can be readied for strain readings as follows:

1. Turn the recorder on and allow at least ten minutes warm up time.

2. Plug the output of the Dynamic Strain Bridge into the appropriate recorder channel.

3. Select the correct paper speed (50 m.m. per second for a dynamic strain frequency of 2.71 cycles per second).

4. Centre the needle on the paper trace - this can be seen as a shadow as there is a light source beneath the paper.

5. Turn the pen vibration on.

6. The pen recorder channel has two gain controls. Turn the continuous control (the small left hand knob) to minimum gain - and set the graduated control to maximum \(3 \times 10^3\) Vpp/m.m.).

7. With the Bridge still balanced in the "Calibrate" position, set the preset calibration control at position 6. Switch the output control to the "unfiltered terminals" position. Operate the press button calibration control while slowly increasing the channel gain, until a square wave of approximately 20 m.m. high is obtained. Set the pen recorder needle to the right hand side of the trace and check that the signal is the same height. (The linearity of the channel can be checked in detail in this way by using smaller increments).
8. If the resulting square wave has a regular height and sharp edges then strain readings may commence.

A.5 THE METHOD OF RECORDING DYNAMIC STRAINS

With the Dynamic Strain Bridge and the pen recorder thus prepared, with the controls arranged as in Fig. 2.2,11 on page 68, the dynamic strain readings are taken and calibrated in the following way.

1. Re-trim the amplifier and turn the Bridge gain to maximum.

2. Re-centre the pen recorder needle.

3. Switch the output control to the "unfiltered meter" position, and switch the R.C.T switch to "Run". Balance the Bridge so that the needle swings equally each side of centre due to the dynamic strain from the instrument.

4. Turn back the continuous gain control of the recorder channel.

5. Switch the output control to the "filtered terminals" position.

6. Increase the channel gain control until the pen can be seen to sweep at least 30 m.m. across the paper.

7. Operate the paper control to give 6" to 8" of trace, and record the gauge pad numbers.

8. Turn the output control back to the "unfiltered meter" position.

9. Set the R.C.T. switch to "Calibrate" and balance the Bridge.

10. Set the output control to "unfiltered terminals". DO NOT TOUCH ANY GAIN CONTROLS.
11. By operating the balance controls set the pen recorder needle to the right hand side of the trace.

12. Select a suitable preset calibration value by trial and error - the travel of the needle can be observed by watching its shadow when the push button is operated. Try to select a value that gives a calibration trace height roughly the same as the preceding sine wave.

13. Start the paper moving and apply about 4 complete square waves to the paper. Record the preset calibration number on the trace.

With the strain reading thus recorded and calibrated, the sequences from 3 to 13 can be repeated, but every half hour or so go through the sequence from 1 to 13.

A typical dynamic strain trace can be seen in Fig. 7.2 on page 158.
THE ASSEMBLY AND OPERATION OF THE SHAKING MACHINE

B.1  THE ASSEMBLY OF THE SHAKING MACHINE

The assembly of the shaking machine is shown in Fig. B1 and is as follows:

1. The rotors are fitted to the main shaft and the tapered roller bearings are pre-loaded by applying 5 lb.ft. torque to the locking nut. The hubs of the rotors are filled with light grease (Shell Retanix A).

2. The shaking machine frame is positioned on blocks in the holding down frame, as shown in Fig. B1. The bottom part of the main shaft is fitted into the hole in the machine’s frame, and the \( \frac{1}{2} \)" dia. bolt is tightened in the bottom end of the shaft.

3. The removable block is placed on the top part of the main shaft and is tightened in place by screwing down the \( 1\frac{1}{2} \)" dia. top nut, as shown in Fig. B1. (Both ends of the main shaft are a light press fit in the frame of the shaking machine.)

4. The tie rods are fitted through the reamed holes in the top member of the shaking machine frame, and once the rods are screwed into the anchors in the removable block, the end nuts are torqued to 150 lb.ft. The two extension pieces are then screwed on to the tie rods.

5. The wooden blocks are removed and the U-bolts holding the machine to the holding down frame are positioned and tightened. The \( \frac{3}{4} \)" dia. bolts in the holding down frame are tightened to prevent lateral movement.
FIG. B.1. STAGES DURING THE ASSEMBLY OF THE SHAKING MACHINE
6. The vertical drive shaft assembly is lightly fixed to the frame of the shaking machine, and the chains are positioned on the sprockets. The chain tensions are adjusted as described in Section 2.4.7 and the bolts holding the drive shaft assembly are tightened.

7. The electric motor sub-frame is bolted to the shaking machine and the motor is bolted to it. The chain between the motor and the drive shaft is positioned, and its tension is adjusted as described in Section 2.4.6. The motor holding bolts are tightened.

8. The photocell unit is bolted to its holder on the frame and a pointer is cemented on the top rotor to break the light beam each time the rotors are aligned.

9. By referring to Fig. 2.4.3 on page 89, the rotor weights necessary to produce the required force are calculated and the lead segments are placed in the rotors.

B.2 THE OPERATION OF THE SHAKING MACHINE

The machine is checked, started and stopped by the following sequence. The various controls referred to can be seen in Fig. 2.4.5 on page 93.

1. Determine the number of weights in each rotor (there must be the same number) and find the maximum allowable frequency. Do not exceed this value.

2. Rotate the rotors by hand to ensure they are free to rotate, that the lead weights are securely positioned, and that no obstruction can fall into the machine.

3. Plug the lead to the shaking machine motor into the side of the control unit.

4. Ensure that the main control switch at the bottom left of the control unit below the rheostat controls is off.
5. Turn on the motor/generator unit and check the direction of rotation. Swap two of the three phase leads to the main switch if necessary.

6. Ensure that the four rheostats in the control unit are turned fully back (anticlockwise) and set the fine control rheostat in the remote control box to a central position. Check that the emergency stop switch is in its normal position (a red light on the remote control box indicates otherwise.)

7. Start the machine using the main control switch. The rotors should be revolved slowly.

8. Operate the emergency stop to check its operation.

9. Turn off the main control switch and return the emergency stop switch.

10. Restart the machine and gradually increase the speed by turning up the rheostats in the control unit, starting with the top left and working down until the desired speed is reached. During this operation check that the motor armature current does not exceed 5 amps and that the diode current remains below 1 amp.

11. Maintain the shaking machine at the desired speed by operating the fine control. Very few adjustments should be necessary to maintain a constant speed.

12. To stop the machine in the normal manner, slowly turn back the rheostats in the reverse order, then turn off the main control switch.

13. In an emergency, operate the emergency stop switch. The machine will stop under its own frictional and wind resistance loading.
APPENDIX C

THE CASTING OF ARALDITE SHEETS AND GENERAL POINTS ON PHOTOELASTIC MODEL CONSTRUCTION AND TESTING

C.1 ARALDITE MIXING

CIBA Araldite D was the epoxy casting resin used throughout the research program. The mix used was 100 parts of Araldite D (CY230) resin to 8 parts of Hardener (HY951) by weight. This was the minimum ratio for full strength and it was never raised above this level to reduce the exothermic reaction involved in the initial curing stage. For quantities over 500 grams mixed in a glass beaker, it was found that the pot life was as little as one hour.

The actual mixing was done in a glass beaker with an electric stirrer as shown in Fig. C1. The stirring motor had a variable speed control. Before the Araldite was added to the beaker it was heated to 100°C to rid the normally viscous Araldite of any trapped air bubbles and moisture. The stirring propellor was clamped in the chuck while the Araldite was still hot to prevent it trapping any air bubbles in the liquid. The Araldite was cooled to 70°F in a water bath for mixes over 500 gms.

When the author began his research, the method of adding the hardener to the mix was to fill a 50 c.c. syringe and slowly inject it to the bottom of the beaker through a clear plastic tube. The mix was stirred during injection so that the less dense hardener, rising through the mix, was caught by the stirring propellor, but the drawback to this method was that air bubbles were sometimes introduced into the mix. This happened in two ways: firstly, air was carried into the mix as bubbles tended to stick to the plastic tube, and secondly, when the
FIG. C1. THE ARALDITE MIXING APPARATUS
hardener was sucked into the syringe it tended to become aerated.

A new method of injecting the hardener was devised, as shown in Fig. C1. The correct quantity of hardener was poured into the glass funnel shown held in a retort clamp after the Araldite had been poured into the main beaker. It was left to stand to allow any air bubbles to float to the surface. When stirring began the clamp on the tube joining the funnel to the spigot on the main beaker was released slightly, thus allowing a slow gravity flow of hardener into the bottom of the mix. The incoming hardener could be easily seen and the flow was regulated to ensure no hardener floated up past the propeller. When the pressure head between the two liquids became small a bung was set in the mouth of the funnel and it was gently pressurised until the hardener level almost reached the clamp. The clamp was retightened, and stirring continued for half an hour.

In earlier castings, flow lines were a problem. These were bands of discoloured Araldite which followed the direction the Araldite took as it flowed into the mould. They were thought to be due to moisture condensing on the surface of the mix (when water is mixed with Araldite D a white liquid results). To try to overcome this problem it was decided to try pouring the mix from the bottom of the beaker, thus leaving the surface behind as about 10% excess Araldite was normally mixed. This is the reason for the larger glass spigot at the base of the beaker as seen in Fig. C1 with a clamped hose attached to it.

When stirring had finished the hose clamp on the funnel/beaker connecting tube was released in order to force the unmixed hardener back up the tube with mixed
Araldite. The large tube was then unclamped and about 25 grams was run to waste to clear any air and unmixed Araldite from the tube.

Even this pouring method produced some flow lines but they were negligible. However air bubble-free mixes could be easily made using this method of hardener injection.

C.2 THE PERSPEX MOULDS

Araldite D does not bond very well to clean, smooth Perspex, so this material was used to make the moulds for the sheets. Also Perspex has the advantage of producing highly polished surfaces in the Araldite castings. Normally the moulds were about 12" square (inside) constructed from ¼" thick Perspex. After cleaning, the sides were bolted together at 1" centres through Perspex spacers, which both regulated the thickness of the cast sheet and provided an Araldite tight exterior.

The mixed Araldite was poured into the vertically held mould; initially it was allowed to run down one side. Care was taken to prevent any air bubbles forming in the corners of the mould, and once it was full it was left to cure in the vertical position.

The cast sheets were cured in the mould for about 48 hours (though after 20 hours the bolts through the spacers were removed). After this initial curing one side of the mould was prised off and the three spacers were removed. An hour in a refrigerator stiffened the Araldite sheet sufficiently for it to part from the Perspex when the latter was flexed.

The main disadvantage of casting sheets in this way (apart from their relatively small size) was that their thickness was non-uniform. This was because the hydro-
static head due to the liquid Araldite in the vertically held mould tended to bow the sides of the mould outwards.

C.3 THE NEW MOULD STRIPPING TECHNIQUE

When Araldite sets it first becomes very viscous, then it hardens until it resembles stiff rubber without any resilience. This stage is reached at about 20 hours at 70°F after mixing. An attempt was made to strip the Perspex mould at this stage and it was found that by using a long bladed circular shanked screwdriver, (about 3/16" dia.), the flexible Araldite could be prised away from the Perspex with no ill effect on the casting. Any dents made in the surface of the Araldite with the screwdriver slowly disappeared. The flexible sheet was then laid on a paper covered glass sheet to allow a further two days curing at room temperature.

If the Araldite was not sufficiently cured before this form of mould stripping took place, it was found that the internal bond strength in the Araldite sheet was less than the Araldite-Perspex bond, which resulted in the sheet disintegrating.

A parting agent, QZ.11 (a wax) was tried on the Perspex but that had negligible effect. Silicone based parting agents were not used as the silicone film is difficult to remove and this might have given trouble with future Araldite joints.

C.4 CURING

Araldite sheets were cured for at least 24 hours at 140°F before any further work was done on them.

C.5 MODEL CONSTRUCTION

The proposed dimensions and a reference grid were lightly scribed on the surface of the Araldite sheet, and
the sheet was cut on a small, high speed jigsaw using a steel straight-edge as a guide. It was found that the saw produced negligible stresses in the Araldite. Cutouts in the model, such as shear wall corridor openings, were also made with the saw, but a \( \frac{1}{8} \)" dia. hole was drilled in each corner to prevent a severe stress concentration due to a sharp corner and also to allow the blade to be turned.

The various pieces were glued together with Araldite D after the mating surfaces were roughened and cleaned. The model was either held in a jig specifically made for the purpose, or more commonly was held by magnetic clamps on a steel surface table. If a right angled joint was to be made, a scriber was run along each side of the joint to form an even fillet of glue. If a long butt joint was being made the two components were placed in position and a strip of cellulose tape was run along the joint. The two connected components were turned over, and hinged to open up the joint. The glue was run along the joint, the components were laid flat, and the excess Araldite on the surface was wiped off.

C.6  **Annealing**

When the model was complete it was annealed at 240°F. The Civil Engineering Department possessed an oven designed specifically for curing, annealing and stress freezing photoelastic models. The oven temperature was controlled by a rotating cam - the time taken to complete one cycle of the cam could be set at 12 or 24 hours. For annealing the shear wall Araldite D models the 12 hour cycle was used, and the cam provided a linear increase from room temperature to 240°F in 3 hours, 6 hours at 240°F and then a further 3 hours dropping linearly back to room temperature.
This particular temperature cycle completed the curing of the joints as well as relieving any stresses induced during assembly.

At 240°F Araldite D becomes very flexible and consequently the models being annealed had to be fully supported to prevent them sagging under their own weight. Flat models were simply laid on paper covered glass, while flat models with different thicknesses had a stepped glass support made.

C.7 MODEL STRENGTH

Araldite D has an ultimate strength of 4000 lb./sq.in. in tension and butt joints were found to have an equal strength as long as they were carefully cleaned prior to gluing. However it was most important to avoid stress concentrations, so that where possible joints, section changes, and sharp corners were all reinforced with fillets or gussets of Araldite, providing the model's basic properties were not altered.

Broken models could be repaired by simply gluing the crack together, or by replacing an entire section of the model.

An important point to consider when designing models is to ensure that the most highly stressed area in the model will not fail before measurable stresses are obtained elsewhere in the model. Sometimes local strengthening at the expense of dimensional accuracy is required to achieve this.

C.8 LOADING DIFFICULTIES

The major difficulty in loading any model is in representing the loading system as found in the prototype, - the usual trouble is that loads applied to a model
take the form of point loads which should be distributed over a larger area. This loading method is satisfactory for displacement models but not for stress distribution models.

Other problems of a more practical nature occur, the major one being the lateral stability of the model. Since it is only a part of the whole structure that is often modelled, the model lacks the inherent lateral bracing of the prototype; this must be provided to prevent premature failure.

Loading frames should be constructed to provide a versatile loading system, with the possibility of different sized models in mind. Another important feature of loading frames is that they should incorporate a "fail safe" system to avoid complete destruction of the model in the case of a crack developing. An example of this is a lever system of loading with limited travel available.