BEHAVIOUR OF LOW DENSITY AUTOCLAVED AERATED CONCRETE MASONRY UNDER CONCENTRATED LOADS

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Abstract

Autoclaved Aerated Concrete (Aircrete) (AAC) is the lightest form of concrete masonry. The material was introduced into the UK in the 1950's. It has been used extensively since that time to form block walls especially in the construction of dwellings. The current product is very different from that produced in earlier years having become progressively lighter. At the same time the ratio of the compressive strength to the density has been increased. Improvements in production techniques have made the present day material properties more consistent. Quality control criteria have become much more stringent and third party supervision has been introduced for manufacture and construction. Raw materials and process are carefully controlled to give consistent output.

As Aircrete has become progressively lighter, new methods of assembly have recently been introduced which raise questions about the performance of the new material. The two principal drivers for lowering the density have been improved manufacturing economy by reduced raw material consumption and improvements in the thermal insulation properties of the material to meet today's Energy requirements. Reducing density tends to have the effect of producing lower strengths and reducing robustness, durability and resistance to chemical attack.

When undertaking the structural design of masonry walls, the stresses induced by concentrated loads can be more critical than those from the general run of uniformly distributed loads on walls. For masonry materials at the lower end of the strength range, their resistance to concentrated loads is central to their suitability for economic application in construction. The current rules and regulations regarding the ability of walls to support concentrated loads were developed on the basis of the strength and behavioural properties of masonry materials material which are stronger and denser than the lighter forms of Aircrete.

In this research, the effects on the behaviour of low density Autoclaved Aerated Concrete blockwork of different forms of concentrated loading were examined using physical testing and mathematical modelling and the behaviour categorised mathematically. The research builds on an EPSRC research project at Kingston University and previous research undertaken by the author (MPhil). The results will enable structural design for concentrated loads on low density Aircrete to be undertaken with greater confidence. This will enable further economy in the use of the material and thereby further improve its economic viability. The conclusions and recommendations will influence national and European masonry structural design codes and standards used by structural engineers.

Acknowledgements

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Chapter 1. Introduction

1.1 Summary of the Research

The research was divided into seven series comprising different forms of physical testing of low density Aircrete blockwork under the effects of concentrated loading. Six of the series of tests were on Aircrete blockwork walls and wallettes built and tested in the laboratory and involved a total of 52 tests and 4020 Demec gauge readings and the other series was on mortar prisms. The wall and wallette tests were supported by tests of the properties of the Aircrete blocks and the thin layer mortar.

1.1.1 The tests in Series 1 were on three Wall Type 1 half storey height Aircrete blockwork walls, each of which was subjected to a different concentrated load and the strains across the full face of the walls were measured under progressively increasing loads in the elastic range.

1.1.2 The tests in Series 2 were on two Wall Type 2 half storey height Aircrete blockwork walls, which were similar to two of the walls in Series 1 except that they had a small cluster of 50 mm long demec gauges under each concentrated load in order to measure strains immediately beneath the loads.

1.1.3 The tests in Series 3 were on a set of six Aircrete blockwork wallettes which were three courses high and built in general purpose mortar. The effect on the ultimate load of replacing the top course of blocks with three courses of brick sized units was measured.

1.1.4 The tests in Series 4 were measurements of Aircrete block properties and the compressive and flexural strengths of thin layer mortar.

1.1.5 The tests in Series 5 were proof loads of lintels bearing on Aircrete blockwork wallettes three courses high.

1.1.6 The tests in Series 6 were concentrated loads, taken to destruction, on nine half storey height Aircrete blockwork walls.

1.1.7 The tests in Series 7 were proof load tests of joist hangers bearing on Aircrete blockwork wallettes.

Two dimensional Finite Element models were made of walls similar to those tested and FE analyses carried out for end and central concentrated loads. Using this approach it was possible to mathematically categorise vertical and horizontal strains across the faces of the walls.

A range of masonry design codes were compared to see the different ways concentrated loads were treated. Calculations comparing test loads and calculated loads were made, conclusions drawn and recommendations made for adoption of revised rules for the codes and for further work.

The results will lead to structural concentrated load design for low density Aircrete blockwork to be undertaken with greater confidence and lead to further economy in the use of the materials. The conclusions will influence masonry Codes and Standards used by practitioners in structural design.

The research builds on an EPSRC research project at Kingston University and the previous research undertaken by the author (MPhil).

Chapter 2. Literature Survey

2.1 Masonry

In the Middle Ages, stone masonry was the principal form of construction for most major buildings. Masons were the leading craftsmen in an alliance of craftsmen on a construction project. Master masons combined the role of architect and builder(1).

Owing to the difficulties and expense of transport, the stone for the masonry was usually quarried from a nearby, often small, quarry except when a particular stone was required for an important building(1).

The first mortars used for laying masonry were made from mud or clay. These materials were used because of availability and low cost. The Egyptians utilized gypsum mortars to lubricate the beds of large stones when they were being moved into position. However, these materials did not perform well in the presence of high levels of humidity and water(2). Lime was used from very early times and Vitruvius, who began as an architect and engineer under Julius Caesar, included a formulation for lime mortar (i.e. lime mixed with fine aggregate and additives) in his publications. Lime mortar joints were considerably weaker than the stone masonry units used in Roman buildings(3). The lime mortar was produced by first burning chalk or limestone in kilns. Quicklime was then formed by slaking the burnt chalk or limestone. Sand (fine aggregate) was added usually in the ratio of one part of lime to two parts of sand and the mortar was thoroughly mixed by hand or in mixers.

Although stone masonry is still used extensively in some areas of the UK, brickwork is the most commonly used form of external masonry in the majority of highly populated areas. Initially external walls were usually solid of a thickness arrived at by rule of thumb. In exposed areas they were often built with common bricks and rendered externally to improve the resistance to rain penetration. External masonry will generally be maintained in a drier condition by a moderately porous uncracked render(4). In more sheltered areas the exposed exterior was in facing bricks with attractive and precise bonding patterns. The Royal Institute of British Architects in consultation with the Brick Makers Association and the Institution of Civil Engineers agreed on "The RIBA Standard Size of Brick" which came into force on 1st May 1904(5). There were a complicated set of conditions which defined the size but they gave an average brick length of 9 inches (225 mm), thickness of 4 $^{5}/_{8}$ inches (115 mm) and height of 2 $^{11}/_{16}$ inches (67 mm). The current 215 mm long by 102.5 mm thick by 65 mm high brick(6) is only slightly smaller than that standard set over 100 years ago. Brick cavity walls of two leaves of brickwork became the most popular form between the two world wars. Following the Second World War the inner leaf progressively became blockwork and by the late 1980's the volume of blockwork sold exceeded that of brickwork. The most common concrete block face size was 450mm long x 225mm high which is equivalent to six bricks laid in mortar. Since the late 1980's there has been a further reduction in the volume of bricks produced and common bricks have virtually disappeared from production. In modern general purpose mortar for masonry, cement has replaced quicklime as the principal binder. This makes them stronger, but more brittle, than the lime mortars they replaced. Mix proportions such as 1:1:6 (cement:lime:sand) are recommended(4) as they are still weaker in compression than most clay bricks and aggregate concrete blocks. General purpose mortars are now defined in BS EN 998(7) as "mortars without special characteristics". They are usually between 8mm and 15mm thick and are used for the vast majority of masonry construction. If sand and cement alone were to be used without hydrated lime and mixed into water, the workability of a general purpose mortar would be unsatisfactory. To overcome this, usually, when hydrated lime is not used in general purpose mortars, a plasticiser is used with the cement to obtain satisfactory workability in the mortar. The plasticiser may be in the form of a proprietary liquid.

The meaning of the word masonry has broadened over the years. Originally a masonry wall meant a wall constructed of blocks of natural stone. Since then the term masonry wall has come to mean any wall constructed of bricks or blocks of any material e.g. clay, concrete, calcium silicate, Aircrete etc, as well as natural stone masonry. In this thesis, "masonry" is used in the present day broad sense to mean an assemblage of masonry units laid in horizontal courses in any type of mortar unless otherwise indicated. A "masonry unit" is a brick or a block. The first British Standard for concrete blocks BS 2028(8) was superseded by BS 2028 : 1364(9) in 1968 and by BS 6073 in 1981(10). The structural design of masonry has traditionally been undertaken by using empirical rules which specified sizes for specific conditions. These empirical rules (currently known as "simple rules") predate the development of structural engineering as a mathematically based profession. When methods of calculation were introduced for steel and reinforced concrete structures in the early part of the 20th Century, masonry continued to be designed using empirical rules. It was not until the 1930's that interest developed in calculation methods for masonry. Even today the majority of masonry is designed by empirical rules which have merely been extrapolated from the earlier rules based on experience essentially of brickwork. Brickwork generally has unused reserves of strength which enable rules of thumb and simple prescriptive design rules to be used with confidence. These avoid the necessity of making complex calculations. Despite the difficulty of analysing masonry behaviour reliably, the designers of modern masonry walls have nevertheless taken advantage of these reserves of strength and walls are now more slender than those of former times. The same reserves do not necessarily apply in the case of low density Aircrete (350kg/m^3) studied in this research which is the weakest of the modern masonry materials. The previously lightest Aircrete in the UK was 475 kg/m3 and described at the time as Ultra lightweight. For the latest low density material it is necessary to obtain a better understanding of its behaviour under load to be able to use it safely and economically.

Low strength masonry has also been used extensively from earliest time, particularly in the form of unfired earth masonry. Unfired earth walling is still used today across the world in many countries where there is no ready access to factory produced masonry(11) and the performance continues to be researched(12). But earth walls are heavy and have to be protected from rain falling or wind driven on to them in any quantity. In wet climates the roof is recommended to overhang by ⅓rd the wall height. The recommended minimum length of lintel bearing is given as 200 mm(13). The compressive strength of rammed earth is similar (2.0 N/mm^2) to the compressive strength of the grade of Aircrete being used it this research.

2.2 Aircrete Masonry

Autoclaved Aerated Concrete (Aircrete) is the most widely available modern material used to produce lower strength masonry units. It can only be produced under factory conditions. This material has many advantages over heavier forms of masonry provided that it has adequate compressive strength and dimensional stability.

The advantages of Aircrete include having the best thermal resistance properties of any type of solid masonry, being non-combustible and having high fire resistance. Its lightness provides easy working and laying properties.

Since the early 1940's, the UK building industry has realised that lightweight concrete blocks could be laid considerably faster than clay bricks. It was highlighted in the trade magazine, "The Builder" in 1944 and subsequently research was undertaken at the Building Research Station, "A Work Study in Blocklaying" which was published in 1948(14) showed that when comparing equivalent volumes of walling, 18 inch by 9inch by 9 inch (450 by 225 by 215 mm) lightweight concrete blocks could be laid in about a quarter of the time that it would take using (clay) bricks. Three inch (75mm) thick lightweight concrete blocks could be laid in about half the time it took to lay the equivalent area of walling using bricks.

In the UK, since the 1950's, an increasing proportion of masonry has been autoclaved aerated concrete blockwork now known as Aircrete. Aircrete masonry, when constructed using thin layer mortar in joints between 1mm and 3mm thick, is intrinsically stronger than Aircrete masonry built using general purpose mortar(15). Thin layer mortar has superior wet and dry adhesion to masonry units compared with that of the recommended mixes of general purpose mortar. Thin layer mortar is supplied as a premixed powder in bags. It is mixed with water in the proportions of a 25kg bag of powder with 5 - 5.5 litres of clean water until an even mix is obtained on the construction site in a tub using a mechanical mixer in a manner similar to, but on a smaller scale than, lime mortar mixing where the powder is always added to water(16).

Thin layer mortar is applied to the Aircrete blocks using specially designed scoops or trowels which enable the mortar to be quickly and accurately applied in the correct thickness. The thin layer mortar sets more quickly than general purpose mortar and has a more rapid strength gain. Instead of the traditional limitation of only building a wall to half storey height in a day in dwellings, thin joint blockwork can be built to the full storey height or more in a day. General purpose mortar joints are notionally taken to be 10mm thick but are frequently thicker and virtually never thinner. Thin joints on the other hand are only 1-3mm thick. Thus the use of thin layer mortar enhances the thermal performance of Aircrete masonry including the air tightness. The area of cold bridging provided by the thin mortar joints is reduced by say 80% compared with general purpose mortar joints. Both types of mortar cause cold bridging through the Aircrete due to their density being much greater than that of the Aircrete. Using thin joint Aircrete masonry (Figure 1) results in faster build times, increased productivity on site, improved thermal performance, better air-tightness, less waste and improved lateral load capacity(17).

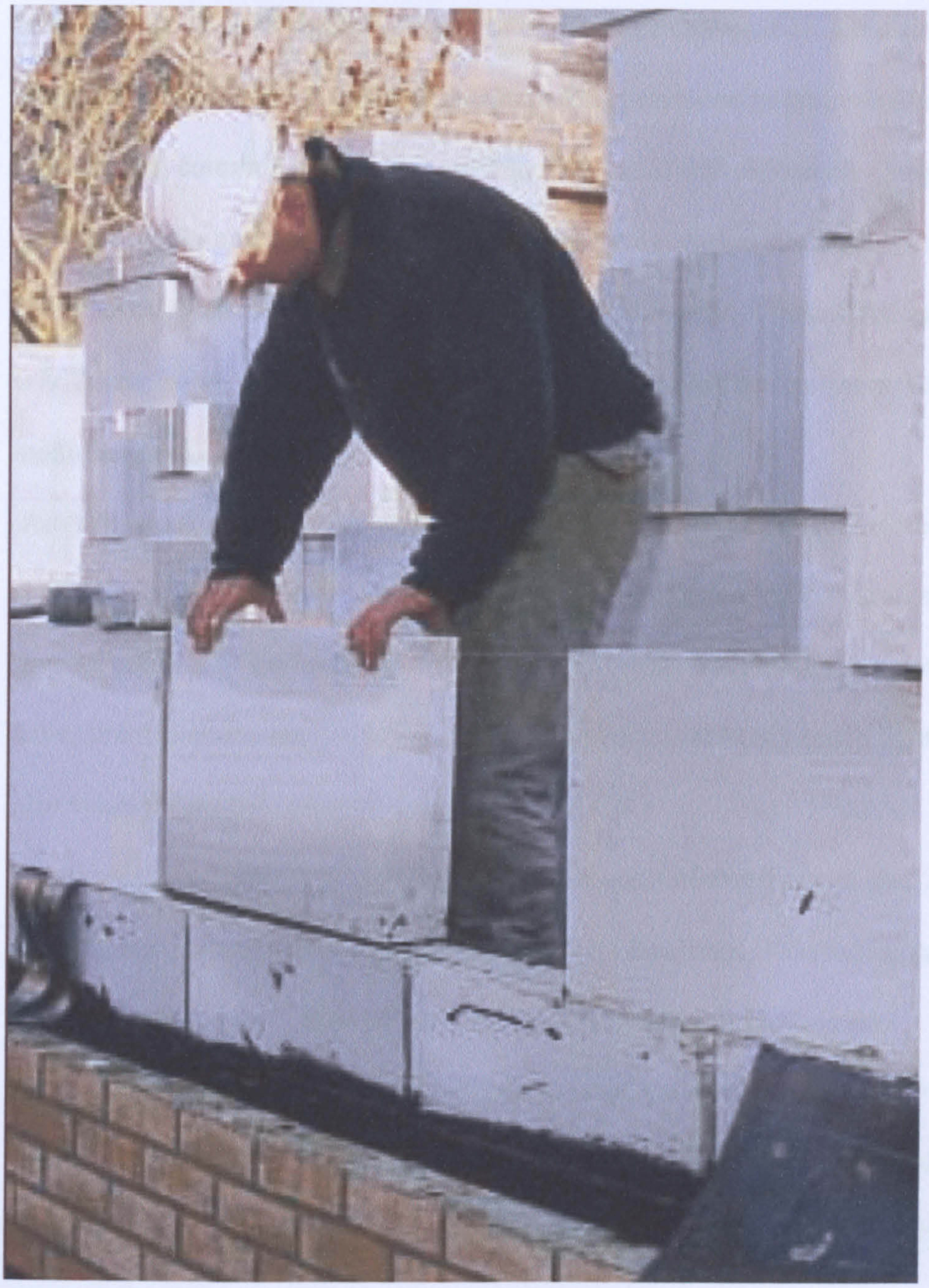


Figure 1: Laying Aircrete blocks in thin layer mortar

As the types of masonry mortar have evolved over the years to use more modern materials, so the materials for masonry units have also evolved. Initially clay bricks replaced stone units for a large proportion of masonry and became the main material used for UK external house wall construction according to a report on the Building brick industry Department of Energy and Department of

Industry(18). In 1938 more than 80% of bricks were common bricks according to West(19). Subsequently, concrete blocks partially replaced bricks, eventually replacing all common bricks. Aircrete blocks have in turn replaced aggregate concrete blocks and now represent 35% of the total concrete block market in the UK. Currently an estimated some 30 million m^3 of Aircrete is produced annually worldwide. More than 3 million m^3, on the basis of submitted statistics from the manufacturers, is produced in the UK.

Aircrete blocks have become progressively lighter, taking a steadily increasing share of total walling in the UK year on year. Although this trend is unlikely to reverse because of the combination of properties referred to earlier, we now have, as a major masonry material, a material which is not only much lighter but also weaker than the earlier masonry materials.

The most common form of construction for the walls of dwellings in the UK is loadbearing masonry. In the vast majority of new dwellings, Aircrete is used for the loadbearing inner leaves of the external cavity walls and for internal blockwork generally. Aircrete is also used to some extent in solid external walls and the outer leaves of cavity walls finished with an external render. It is also used below ground level damp proof course showing perhaps a surprisingly good resistance to frost and ground water chemicals.

Unlike other structural materials, masonry is assumed to be a composite non-homogeneous material whose mechanical properties depend on the properties of and interaction between the masonry unit and mortar including their volume ratio and the properties of their bond.

Autoclaved Aerated Concrete (Aircrete) is the lightest form of concrete masonry. It is a development from sand-lime bricks which were patented in the UK in

1866(20). The first UK patent for Autoclaved Aerated Concrete (using pulverised fuel ash) was granted to P E Starnes in 1951(21). The material, which is of necessity factory made, was introduced into the UK commercially at that time, more than 50 years ago. Its popularity has continued to grow and it is now used extensively in the form of blocks to build the walls of dwellings. It was, however, manufactured in Scandinavia much earlier i.e. in 1920 (using ground sand). By 1968 Autoclaved Aerated Concrete was a construction material of world significance(22). The use of Aircrete has continued to spread and it is now manufactured in more than 40 countries in all climates from very cold to very hot and in seismic regions. Anomalously, although Aircrete is well known in the USA where it was initially termed "Cellular Concrete" and was the subject of two very thorough papers (23), despite various attempts it has failed to get into commercial production until very recently and now American Society for Testing and Materials (ASTM) (24) Standards have been published and the term AAC has replaced Cellular Concrete.

In the United Kingdom, Aerated concrete commenced experimental production in the form we now know it i.e. autoclaved, in the latter part of the 1940's when the late Sir John Laing recognised its great potential. The first British Standard for "Aerated Concrete Building Blocks" was published in 1947(25). It was for dimensions only to prevent the development of a proliferation of different sizes. The preparation of such a standard showed considerable foresight in view of the way the importance of the material grew.

The first factory came into commercial production in I951 using the process developed independently in the United Kingdom with cement as the binder, pulverised fuel ash as the fine aggregate and aluminium powder as the aeration agent. That company produced about 10,000m^3 of aerated concrete blocks in I951 from one factory and about 1,000,000m^3 in 1977 from ten factories. With rationalisation and by replacing labour-intensive small factories with larger more efficient ones, the company now has only three large factories which produce in excess of a million m^3 annually.

Aircrete, when it was originally introduced into the UK in 1951, had a density of 750 kg/m^3 or more and the material has become progressively lighter over the ensuing period of over half a century. In 1978 a lower density of 500kg/m^3 was added to the range which quickly evolved down to around 450 kg/m^3. In the mid 1980's the National House-Building Council still required autoclaved aerated concrete blocks to have independent test certification as they were afraid they would be fragile(26) In recent years, an even lower density of 350kg/m^3 and new methods of assembly have been introduced raising questions about the performance of the material. Two principal drivers to lowering the density are reduced raw material consumption and improvement in the thermal insulation energy conserving properties to meet building regulation requirements as they become progressively more stringent(27). The current product is very different from that produced in earlier times in a number of ways. Many of the changes result in the material being even more reliable than in the past. In particular, quality control criteria have become much more stringent and third party supervision has been introduced for manufacture and construction. The raw materials and process are more carefully controlled resulting in a more consistent output.

Everything else being equal, reduction in density leads towards reduced strength, robustness, durability, fire resistance and resistance to chemical attack. It also raises questions about moisture and thermal movement and whether compatibility with other building materials is adversely affected etc.

The most severely loaded positions in most masonry buildings are usually where the masonry supports concentrated loads. The safe and economic use of Aircrete blockwork depends on its ability to support concentrated loads such as the reaction from a beam spanning over an opening in the plane of the wall e.g. lintel bearings and where the masonry supports the reaction of a beam spanning perpendicular to the wall including beams or joists supported on joist hangers.

As far as the compressive strength of Aircrete blockwork is concerned, the critical question is the resistance to concentrated loads(28). This property is central to the suitability for economic application in the principal uses of forms of masonry with low compressive strength. Concentrated loads occur in all masonry buildings where the masonry has to support loads from lintel bearings, beam bearings, roof trusses, joist hangers etc. Current rules and regulations for concentrated loading were developed on the basis of the strength and behavioural properties of other forms of masonry(29). Measurements of the performance of Aircrete are limited to work on the earlier stronger and denser Aircrete material(30)(31).

An investigation of the behaviour of any masonry under concentrated loading needs to start with consideration of the behaviour of masonry walls under uniformly distributed loading(32).

The relationship between calculated wall strengths for Aircrete masonry according to EC6 and measured wall strengths is shown in Figure 2(33).

Earlier findings by Edgell(34) indicated that walls built from different size Aircrete units all exhibited similar strength and this was confirmed by subsequent work at the British Cement Association (BCA)(35) on Aircrete wallettes of various thicknesses built in two types of general purpose mortar. These findings are contrary to the assumptions for masonry generally in BS 5628 Part 1 and EC6 Part 1-1. The BCA studies on Aircrete wallettes concluded that the two codes overestimated the strength of low strength block masonry.

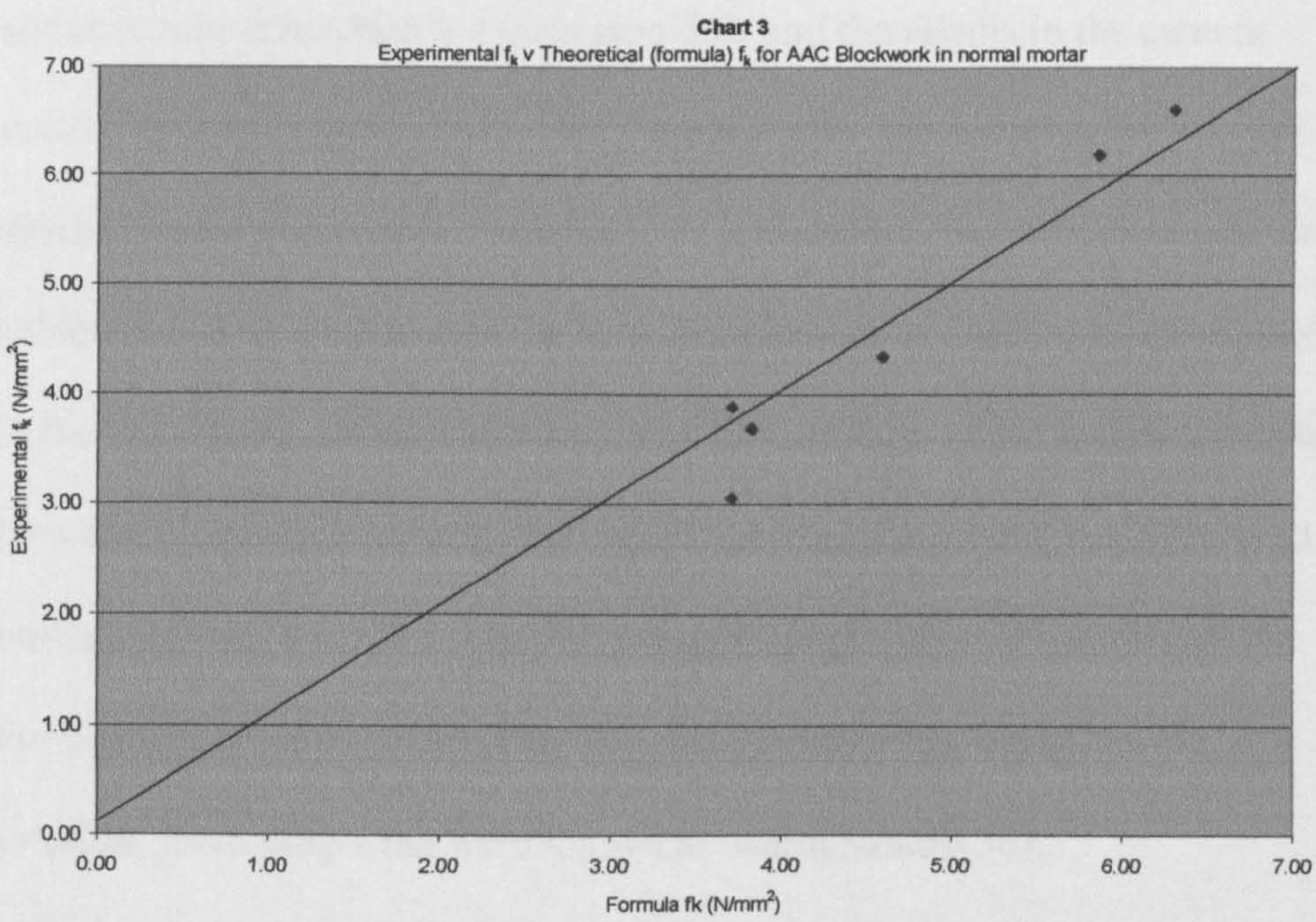


Figure 2: Experimental vs Theoretical characteristic compressive strength of Aircrete masonry

2.3 Aircrete Manufacture

The first method of producing aerated concrete was patented by E Hoffman in 1889. The aeration was produced by carbon dioxide generated in the reaction between hydrochloric acid and limestone.

In 1917, a Dutch patent was registered using yeast as an aerating agent. Later patents involved reaction between zinc dust and the alkalis in the cement mixture, hydrogen peroxide and air foaming. The first documented attempt at autoclaving aerated concrete was in 1927 in Sweden. The discovery was almost accidental in that a lecturer at the Royal Institute of Technology in Stockholm, Dr Axel Eriksson, decided to autoclave a porous mass of burnt shale limestone, water and aluminium powder. The result was that the porous mass survived autoclaving with a much increased strength. This discovery soon led to the development of an autoclaved aerated concrete industry using local raw materials, producing what were known as 'warm stones'(36).

At the present time in large factories, large batches of finely ground silica sand and/or pulverised fuel ash (pfa), are mixed with cement and/or lime and water to form a slurry of low viscosity. No coarse aggregate is used. The slurry is mixed with a large paddle mixer for three to four minutes. Small quantities of aluminium powder(0.05 to 0.1%) and dispersion agent are added to the slurry 10-20 seconds before it is discharged to part fill large steel moulds(36). The aluminium powder reacts with the alkaline environment of the mix to generate tiny bubbles of hydrogen gas (maximum diameter 2mm) within the slurry ($2Al + 3Ca(OH)_2 \rightarrow 3CaO.Al_2O_3 + 3H_2$), which cause the mass to rise and stabilize to form the Aircrete cellular structure (Figure 3). The matrix forms narrow bridges around the cells. The lighter the material, the thinner are the solid bridges. The slurry rises to form large "cakes" typically 3-4m^3 which fill the moulds. The hydrogen dissipates and is replaced by air. The moulds are previously oiled to aid removal of the Aircrete material without damage prior to cutting. After about 20 minutes the Aircrete sets to a firm but relatively soft consistency suitable for cutting into 300 to 400 blocks of say 100mm thickness (or a proportionally smaller number if blocks of greater thickness are cut) using reciprocating steel wires after demoulding. Aircrete blocks are produced in a range of thickness from 60mm to 500mm with a range of face dimensions depending on the country of use.

After cutting, curing by steam in giant autoclaves at ten times atmospheric pressure with temperatures above 200K enables the formation of the calcium silicate hydrate tobermorite and the chemical conversion into a stable and durable material. This is essential to complete the manufacturing process and comprises purging to remove the air from the autoclave, steadily increasing steam pressure after the autoclave door(s) have been secured, maintaining the pressure for the prescribed number of hours, then steadily reducing the pressure to ambient before opening the autoclave door. An indication of the size of an autoclave may be obtained by comparing the size of the van and the autoclave shown in Figure 4.

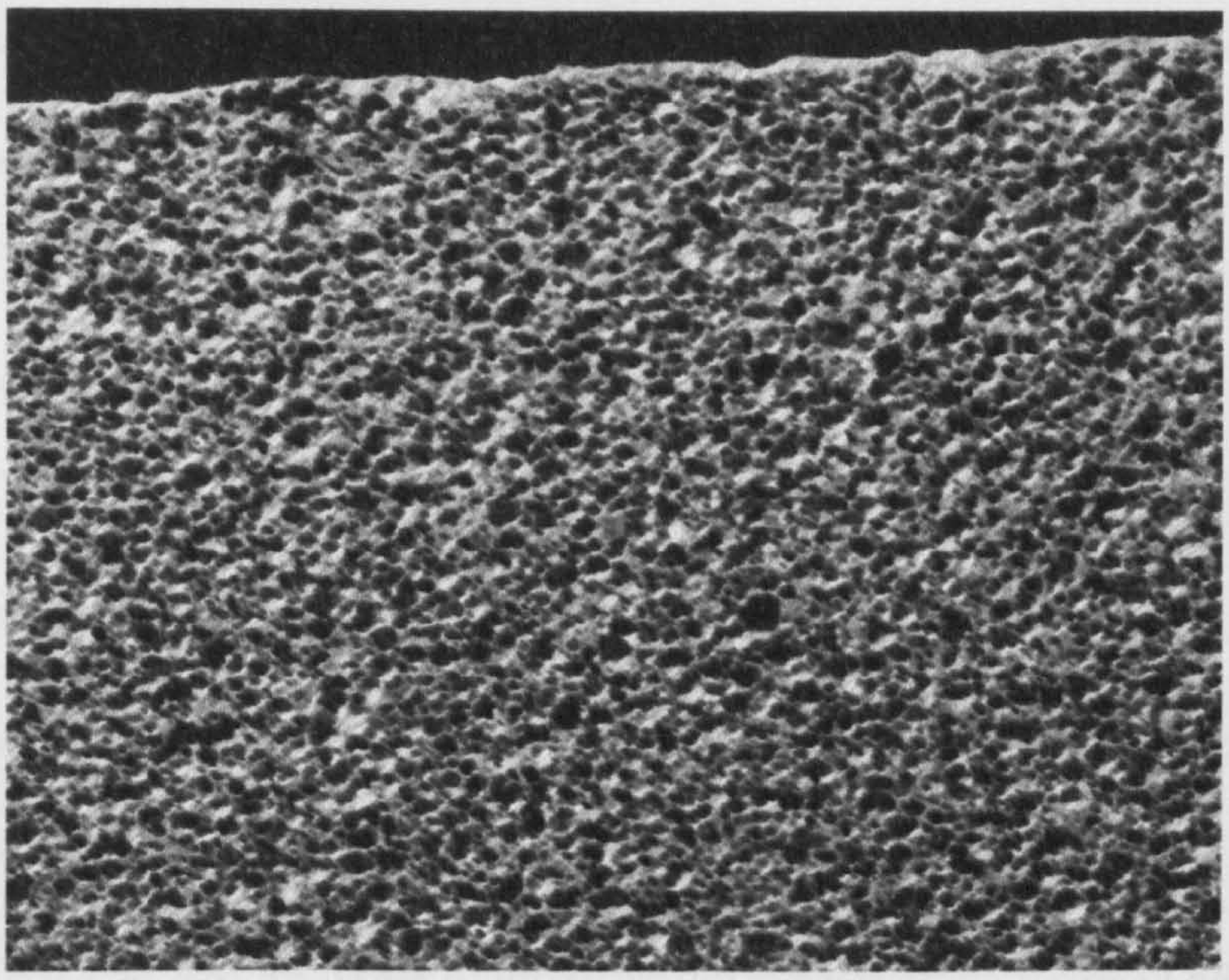


Figure 3: Aircrete Internal Structure

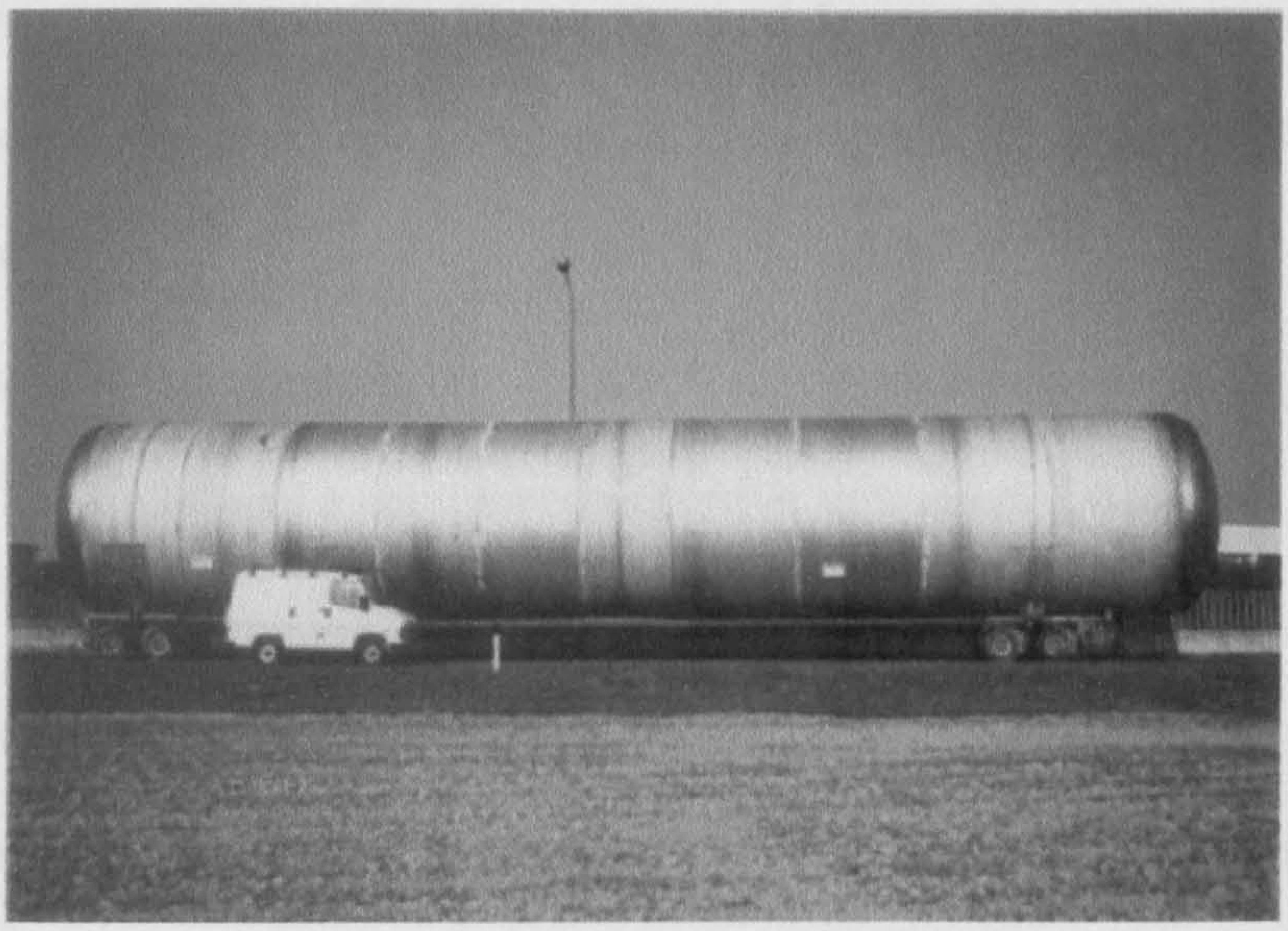


Figure 4: New Autoclave for Aircrete manufacture being transported

The manufacturing process is completed within 24 hours. When the products are removed from the autoclaves they are ready for use as soon as they have cooled. The energy input of autoclaving is off set by the use in the manufacturing process of recycled material such as pfa, recycling "soft" waste (the green process off cuts) and by the energy savings in use due to the thermal insulation provided(37).

Energy used in transport to site is less than that for heavier masonry materials as a greater volume of material is delivered for a given transport weight.

2.4 Properties

The range of physical properties of Aircrete (aac) are well documented (38)(39)(40)(41)(42)(43)(44).

Compressive strengths are in the range 2.0 – 10.0 N/mm^2 and densities range from 300 to 800 kg/m^3.

There are RILEM test methods to measure the compressive strength and just about all other Aircrete properties(45)(46) as well as a range of CEN Standard(47) test methods. The British Standard Test methods for concrete masonry which also applied to Aircrete but there were no specific tests for Aircrete(10).

In the context of this research, low density Aircrete means a nominal (dry) density of 350 kg/m^3 and a nominal compressive strength of 2.0 N/mm^2.

2.5 Concentrated loads on masonry

The maximum stresses that walls in buildings are likely to have to withstand are from various forms of concentrated loading e.g. from steel and concrete beams spanning perpendicular to the walls, from lintels spanning over openings in the plane of the walls, from timber joists supported on joist hangers and from columns and stanchions bearing on the wall. Consequently, these stresses frequently dictate the necessary masonry strengths and wall thicknesses in a building. The structural engineer usually calculates these stresses using the relevant design code of practice for the area where the building is to be erected or uses locally accepted rules of thumb for the dimensions of the bearing.

Regardless of the material used to manufacture the masonry units, cracks tend to develop in low rise masonry buildings during their service life. Cracking is generally due the effects of minor movements or settlements on the masonry which is a brittle material. Sometimes cracks occur at or near the end of lintel bearings. Part of this research will be concerned with the post first crack behaviour of bearings when tested to failure.

The earliest UK "bible" for modern building construction, Mitchell's "Building Construction" had reached it's eleventh edition by January 1930(5). The chapter on brickwork did not deal with concentrated loading as one of the three general causes of instability which it identified. However, under corbelling, it states "In the case of concentrated loads transmitted by the ends of main girders, the centre of pressure should always be arranged to come within the middle third of the wall or pier, to avoid tensional stresses being set up on the side of the wall remote from the load" At that time, walls were generally more massive than they are now. Furthermore, as is shown in the diagrams, piers were invariably introduced at the location of girders spanning onto the wall. In the chapter on "Pillars and Girders" bearing surface is defined as that part of the lower face of the girder which rests upon the support and that the minimum area of the bearing surface may be obtained by dividing the pressure on the support by the safe strength of the template. "The length of the bearing should be sufficient to allow its centre to fall within the prescribed limits for centre of pressure of the support."

Masonry is a composite inhomogeneous structural material unlike other structural materials such as timber, reinforced concrete and structural steel work. Its mechanical properties depend on the properties of and the interaction between the composite materials i.e. the masonry unit (brick or block) and the mortar.

In the early 1930's Structural Engineers realised they needed to know how to get a brick wall to carry concentrated loads from steel beam bearings. The Institution of Structural Engineers produced a series of reports into providing one of more courses of blue engineering bricks at the top of the wall immediately under the steel bearing and the stiffness of a steel bearing plate necessary to spread the load applied to the brickwork(48)(49). The results of tests on brickwork piers surprised the Institution of Structural engineers. Piers with a top course of blue engineering bricks failed at a lower load than those without. "It would seem that the provision of a hard course between the load and the softer supporting course below leads to comparatively early failure, by cracking, of the hard course immediately under the load."(50) The results would even surprise some engineers even today.

The first Code of Practice for loadbearing walls (of brickwork) was published in 1948 and revised in 1964 and finally in 1970 (metric version)(51). Permissible stresses for brickwork were given based on data produced in the 1920's by the Building Research Station(52). In the 1970 version of CP 111, under "Design Considerations" "Load Dispersion" it states "The angle of dispersion of loading of walls should be taken as not more than 45° from the direction of such loading." and indicates the stress under a concentrated load should not exceed the permissible stress for the wall by more than 50%. It did not distinguish between concentrated loads within the length of the wall and those at the end of a wall. The 1978 version of BS 5628 contained guidance for Aircrete blockwork based on wall tests. The concept of shape factors dependent on the shape and size of the blocks was introduced. This enabled blockwork to be designed to carry higher loads than brickwork of the same unit strengths. Research in the UK on masonry generally excluding AAC proved very useful in the process of amending BS 5628(53).

Most low rise masonry, particularly in dwellings, is designed by rules of thumb. These rules whose origins have been lost have been in use for decades and were included in "Schedule 7" of the first Building Regulations for England and Wales published in 1968 which have been superseded by successive versions of the Regulations(23). The rules of thumb of the original Schedule 7 have remained essentially unaltered for small buildings (particularly dwellings) to this day. In the 1980's it was realised that house construction had evolved and rules of thumb which had been developed for solid 225mm thick brickwork were being applied to cavity walls with a loadbearing inner leaf of Aircrete. A government funded research programme was undertaken to test the validity of the rules of thumb. Generally the tests showed the level of safety to be adequate. However, when the tests were taken to failure, crushing tended to come from concentrated loads at lintel bearings(54).

Conclusions from 298 tests on brickwork confirmed the apparent enhancement of compressive strength under concentrated loading and the enhancement factor is dependent on load position and brick strength(55). A preliminary Finite Element study of concentrated loading on brickwork was undertaken by Ali and Page(56)

An investigation of 50 half storey height walls of AAC, dense aggregate concrete blockwork and clay brickwork proposed an enhancement factor "R". The AAC used in the investigation was constructed with weak general purpose mortar (1:2:9). R was dependent on the material density, wall thickness, eccentricity and position of the load(57).

Proposed design rules for concentrated loading were put forward by Page and Hendry in 1988 which were "derived from all previously reported experimental and analytical studies of this problem and apply to all types of masonry built with solid units"(58). They indicate that, because of the wide scatter of results, sophisticated design provisions with separate allowance for the influence of each parameter do not appear warranted. Their proposals seem to align to a degree with the CIB masonry code published a year earlier in 1987(60) but no reference is made to the BS 5628-1 1978 version(61). They assert that while there are compressive isobars of stress immediately below the concentrated load, at lower levels the stresses change to tensile directly beneath the applied concentrated load. They further assert failure is initiated in this zone. This is based on what happens in the vicinity of the anchorages in pre-stressed concrete. When compared with the available numerical and experimental results, conservative estimates of ultimate strength are obtained in all cases using the Page and Hendry proposed formula.

These recommendations form the background to the current codes. However, only the Canadian code(82) currently uses the Page and Hendry formula. In the absence of research Page and Hendry indicated that the spread (at mid height) could be taken as 45° or 60°.

They proposed a strength enhancement factor (R) for the area under the bearing of:

$0.55[1+0.5a_1/L]/[A_b/A_e]^{0.33}$

A_b is the bearing area

A_e is the effective wall area (load spread at H/2 x thickness)

a_1 is the distance from the end of the wall to the nearest end of the bearing area

L is wall length

H is wall height

My previous research on behaviour under concentrated loads was undertaken on Aircrete of heavier density and a range of higher compressive strengths than that used in this current research(31). In that research the test samples had been built in conventional thick joint general purpose mortar compared with thin layer mortar used in this research.

Generally in masonry built using conventional thick joint general purpose mortar, the weakest part of the wall is the adhesion between the mortar and the masonry unit. The interface between mortar and the units in the perpend joints tends to be weaker than that between the mortar and the units in the bed joints. The weakness of the mortar/unit interface has been demonstrated in the tests of flexural strength where it has also been shown that the flexural strengths are sensitive to bad workmanship(62).

For Aircrete built in general purpose mortar under concentrated loading, the mode of cracking failure normally appears to initiate at a perpend joint in an interface between the end of a block and a mortar joint as this is the weakest position under tension. The properties of thin layer mortar are such that these weaknesses should not normally occur.

Design rules derived from numerical and experimental studies have been proposed for Canada(63) for assessing the bearing strengths of hollow concrete masonry walls built with conventional mortar subjected to in-plane concentrated loads. Only one of the two possible zones of failure considered may be relevant to Aircrete i.e. when failure is initiated in the area of solid–grouted masonry directly beneath the concentrated loads. The important factors influencing the bearing strength are taken into account: loading eccentricity across the wall width, effective loading area, loading plate length, and loading location along the wall. An angle of 22° or slope (vertical to horizontal) of 2.5:1 is chosen for a safe estimate of the dispersion of concentrated load through the solid–grouted masonry. Further work carried out by Arora at BRE, UK on hollow concrete blockwork does not appear to be very relevant to Aircrete. Aircrete is always solid(64). Arora concludes that not all hollow blocks will behave the in the same manner as solid blockwork walls. He observes ".....solid walls tend to fail by formation of long vertical and diagonal cracks normal to the face of the wall....." and "Hollow walls tend to fail quite differently, for example, by in-plane splitting of the webs......".

Basically, the assumption behind the longstanding guidance in UK codes and regulations, principally BS5628 Code of practice for use of masonry, is that the spread from concentrated loads is 45° with some restrictions. The spread assumed in Eurocode 6: Design of Masonry Structures – Part 1-1(65) is 60^0. In the design of all masonry subjected to concentrated loads one has to consider the maximum pressure on the wall immediately under the applied load, the degree of spread of the load throughout the height of the wall, or at least to the level in the wall (usually mid height) where it has to be checked, and the mode of failure. If the load is assumed to be evenly distributed, in simple terms, the maximum pressure on the wall immediately under the applied load is easily calculated. However, the masonry is in a state of biaxial or triaxial stress. Existing design rules world wide(66) typically allow enhanced stress immediately below the concentrated load for this reason. In the case of Eurocode 6, an enhancement factor β which is never greater than 1.5 is applied to the loaded area A_b and the design strength of the wall $f_d\beta$ is reduced from this maximum value using a formula which takes account of the distance from the end of the wall to the nearest edge of the applied load (Equation 1 below). It is difficult to follow the reasoning for this as it appears to be taking into account the amount the load can spread at a lower level. In the case of masonry in conventional mortar it is generally agreed that failure is caused by horizontal tensile strain exceeding the tensile strain capacity of the masonry. As the weakest link nearest to the applied load is the mortar/unit interface at the end of the unit immediately under the load, a more relevant criterion might be the distance from the end of this unit. In the case of Aircrete jointed with thin layer mortar, the adhesion is much higher and it is reasonable to support the view that the wall may behave more monolithically.

Where $\beta = \left[1 + 0{,}3\frac{\alpha_1}{h_c}\right]\left[1{,}5 - 1{,}1\frac{A_b}{A_{ef}}\right]$ ....Equation 1

.

and

α_1 is the distance from the end of the wall to the nearer edge of the loaded area,

h_c is the height of the wall to the level of the load,

A_b is the loaded area

A_{ef} is the effective area of bearing i.e. $l_{efm}.t$

There was a view from Germany(67) that an enhancement for end loading was not justified but they suggested a compromise of 1.15 and a maximum enhancement for end loading was finally agreed as 1.25. A spread of 60^0 was supported by the majority of countries drafting EC6 despite the UK proposal that the spread is 45^0.

Eurocode 6 will replace the existing design codes of all European countries which are members of CEN after a period of co-existence. It will also be used in various other countries around the world. There are detailed differences between the treatment of concentrated loads in both Eurocode 6 Part 1 - 1 and BS5628 Part 1 compared with the German DIN 1053 Masonry code(68). In addition, the enhancement factors vary and there are different calculation procedures. Considerable research has been undertaken on the relationship between the strength of a masonry unit and the strength of a wall(69). It is generally agreed that the strength of the mortar has a lesser influence on the strength of the masonry than the strength of the masonry unit. The most recent work relevant to this thesis compared calculated masonry strengths according to Eurocode 6 with was a review of the experimental work(33). The work confirmed the formulae for the strength of masonry in EC6 reflect the relative importance of the strength of the masonry unit compared with the strength of the mortar on the masonry strength.

One thing the various sets of rules have in common is that they ignore the bonding patterns of the masonry, including the face size of the masonry units, except to the extent that they influence the overall strength of the wall under uniform loading. In addition, although they distinguish between hollow and solid units, they do not make any distinction between different masonry materials or different strengths. The EC6 rules do have some limitations regarding the relative position of the masonry unit immediately under the concentrated load.

In England and Wales, Approved Document A(70) to the Building Regulations(27) gives some simple guidance on the treatment of concentrated loads, as does BS 8103 Part 2.(71).

The status quo is that Aircrete blockwork appears to satisfactorily support commonly encountered concentrated loads such as those from floor joists and lintels. The design of the bearing is either by rule of thumb or by calculation using the methods in the masonry structural design codes. These methods have been developed generically to be used for all types of masonry from the very hard and dense to the light and relatively weak, although the actual behaviour may be different. There is little consideration of the different way the loads are applied depending on the material and profile of the beam, joist hanger or lintel. There is no distinction between the simply supported and encastré conditions or the span dependent stiffness or deflection. There is no consideration of the effect of the different jointing media used. Consequently the true levels of safety in use are not known and optimum economy cannot be achieved.

2.6 Masonry Standards and Codes

The earliest standard for a brick, *The R. I. B. A. Standard Size of Bricks* agreed between the R.I.B.A. and the Brick Makers' Association in consultation the Institution of Civil Engineers, came into force on the 1st May, 1904(5). The earliest British Standard for Aerated Concrete Blocks was published in 1947(25). Autoclaved Aerated Concrete had been developed commercially abroad but not in this country at that time. It was seen, though, that there was some probability of extensive production. So it was thought desirable to issue a standard for dimensions of aerated concrete blocks to avoid the production of differing sizes by the various manufacturers. This would ensure that lightweight building blocks of this material should be made in sizes compatible with other standard units. It was not considered desirable to introduce quality clauses until the qualities achieved in general production were known. Following that, some time later in 1953 BS 2028(8) was published for all concrete masonry units. This was in turn replaced by BS 2028:1364(9) and subsequently by BS 6073 Parts 1 and 2 in 1981(10).

In 2005 the CEN Standard EN 771-4(72) replaced BS 6073 Part 1 for Autoclaved Aerated Concrete Masonry Units after a period of co-existence. Code of Practice CP 111 was written in permissible stress terms and was first published in 1948 and revised in March 1964 and in 1970(35). It stated "The angle of dispersion of loading of walls should be taken at not more than 45° from the direction of such loading." It went on to say, for "*Walls subjected to concentrated loads*" Additional stresses of a purely local nature, as at girder bearings, column bases, lintels or other concentrated loads, were to be calculated, and the maximum stress resulting from a combination of these with those provided in other sub-clauses and should not exceed the normal permissible stress by more than 50%. Where indeterminate but very high stresses occur, such as at the outer edge of a wall supporting a cantilever, a spreader should be provided.

The first version of BS 5628: Part 1: 1978 contained more comprehensive guidance on the treatment of concentrated load than CP 111 which it replaced. This was to the effect that:

Increased local stresses may be permitted beneath the bearing of a concentrated load such as beams, columns, lintels, etc. provided either that the element applying the load is sensibly rigid, or that a suitable spreader is introduced.

The concentrated load may be assumed to be uniformly distributed over the area of the bearing, except in the special case of a spreader located at the end of a wall and spanning in its plane and dispersed in two planes within a zone contained by lines extending downwards at 45 ° from the edges of the loaded area.

The effect of the local load combined with stresses due to other loads should be checked at the bearing, assuming an enhanced local design bearing strength. The degree of enhancement depended on the type of bearing which were differentiated by the variables considered to have an effect i.e. distance from the end of the wall and proportion of the thickness of the wall on which the concentrated load bears.

The resulting stresses at a distance of *O.4h* below the bearing where the design strength should be calculated in the normal way for distributed loads given in the code. BS 5628 -1:1978 was superseded by BS 5628-1:1992 which has recently been revised to refer to CEN Standards for masonry units and test methods. The requirements for concentrated loading in the current version of BS 5628 are dealt with more fully in Chapter 7.

2.7 Test Methods

Internationally recognised test methods for Aircrete including compressive strength and density have been in existence since 1975(45). Compressive strength and density (which are interrelated to an extent in Aircrete) are the physical properties which characterise Aircrete blocks. In the UK, however, British Standard test methods were the same for all concrete blocks including Autoclaved Aerated Concrete in BS 6073(10). It was not until the new European (CEN) test methods were introduced that separate tests for Aircrete were extant in UK. All of the relevant test methods for Aircrete blocks are in the EN 772(47) Methods of Test for Masonry Units. The test methods for mortar are in the EN 1015(73) Methods of Test for mortar for Masonry. Both EN772 and EN1015 are each in over 20 separate parts, only some of which are relevant to Aircrete.

The European test methods include test methods to measure some properties of masonry assemblages(74).

Chapter 3. Earlier Research

My earlier work on "The Structural Behaviour of Autoclaved Aerated Concrete Blockwork Subjected to Concentrated Loads"(31) was entirely on "normal" density Aircrete. Therefore the compressive strength and density was higher than that of the Aircrete used in the present research. Lower density Aircrete was not produced in the UK at that time. Furthermore the normal density Aircrete blockwork researched at that time was built in general purpose mortar rather than the thin layer mortar used for this research(75).

The present research was on lower density Aircrete material, possibly approaching the lower limits of compressive strength and density suitable for loadbearing blockwork applications. Some lessons learned from the previous work remain relevant although some of the previous conclusions have been amended due to advances in knowledge and testing techniques.

At the time the previous work was proposed, the Code of Practice for loadbearing brickwork and blockwork(61) was in the course of being rewritten. The well established concept of allowing some overstress immediately beneath concentrated loads was not being questioned. Generally everyone accepted that a situation similar to the familiar situation of confined compression in soil mechanics pertained. But there was some debate whether the maximum of 50% over-stress, depending on the circumstances, was justified. Although the allowable over-stresses did not appear to have caused any problems at that time, concern was expressed as to whether it was too great. The substantial work of Page and Hendry did not appear until 10 years later which tended to support the traditional assumptions from an assessment of all the research published at that time (published 1988)(58).

At the time of my earlier research (published 1978)(31) all of the British Masonry Code recommendations were based on the knowledge of the behaviour of clay brickwork extrapolated to include concrete blockwork. The experience of those questioning the extent of the allowable overstress was largely in the area of loadbearing clay brickwork with its inherent variability. Aircrete was shown to have greater consistency due to the method of manufacture. Even at that time, the British Standard for concrete blocks (including Aircrete)(9) required the compressive strength of an individual block to be not less than 80% of the mean strength of the sample of 10 blocks taken from a batch of 1000 blocks.

The research studied the structural behaviour of autoclaved aerated concrete (the term Aircrete had not been coined at that time) under concentrated loads. It set out to determine the loadbearing capacity of autoclaved aerated concrete blockwork under concentrated loading, establish the amount of spread of the load and study the modes of failure.

Aircrete blockwork "piers", which today would be termed wallettes, were load tested. The testing was undertaken in a steel rectangular loading frame with a height of 3.0m and a width of 2.4m constructed from rolled steel joists. The frame had an intermediate cross head beam which could be set at different heights depending on the size of the wallette to be tested.

The loads were applied through a hydraulic ram fixed to the centre of the intermediate crosshead beam. The loading ram was connected to a Dennison compression testing machine which controlled the rate of load application on to the piers.

The research commenced with a series of photoelastic tests on small araldite models to obtain an understanding of the principles of photoelasticity(76) and obtain familiarity with the testing techniques. This included understanding that the fringes, as viewed through a polariscope, are closely spaced in the areas of high stress and virtually non existent in areas of very low stress. At any point the planes of refraction are those of the principal stresses.

A plane polariscope was used as the simplest optical system used in photoelasticity suitable for studying biaxial stress. It consisted of two linear polarizers (which transmited light only along their axis of polarization) and a white light source. The polarizer nearest the light source is called the *polarizer*, and the second is the *analyzer*. They are illustrated diagrammatically in Figure 5. In the plane polariscope, the two axes of polarization are crossed; hence no light is transmitted through the analyzer.

The stress field at any point in a photoelastic specimen can be related to its index of refraction. The light emerging from the analyzer is subject to prior conditioning from the polarizer and specimen. There are two possible fringe patterns of points where the light is extinguished: "Isochromatics", which indicate areas of constant stress and "Isoclinics", which indicate principal stress directions.

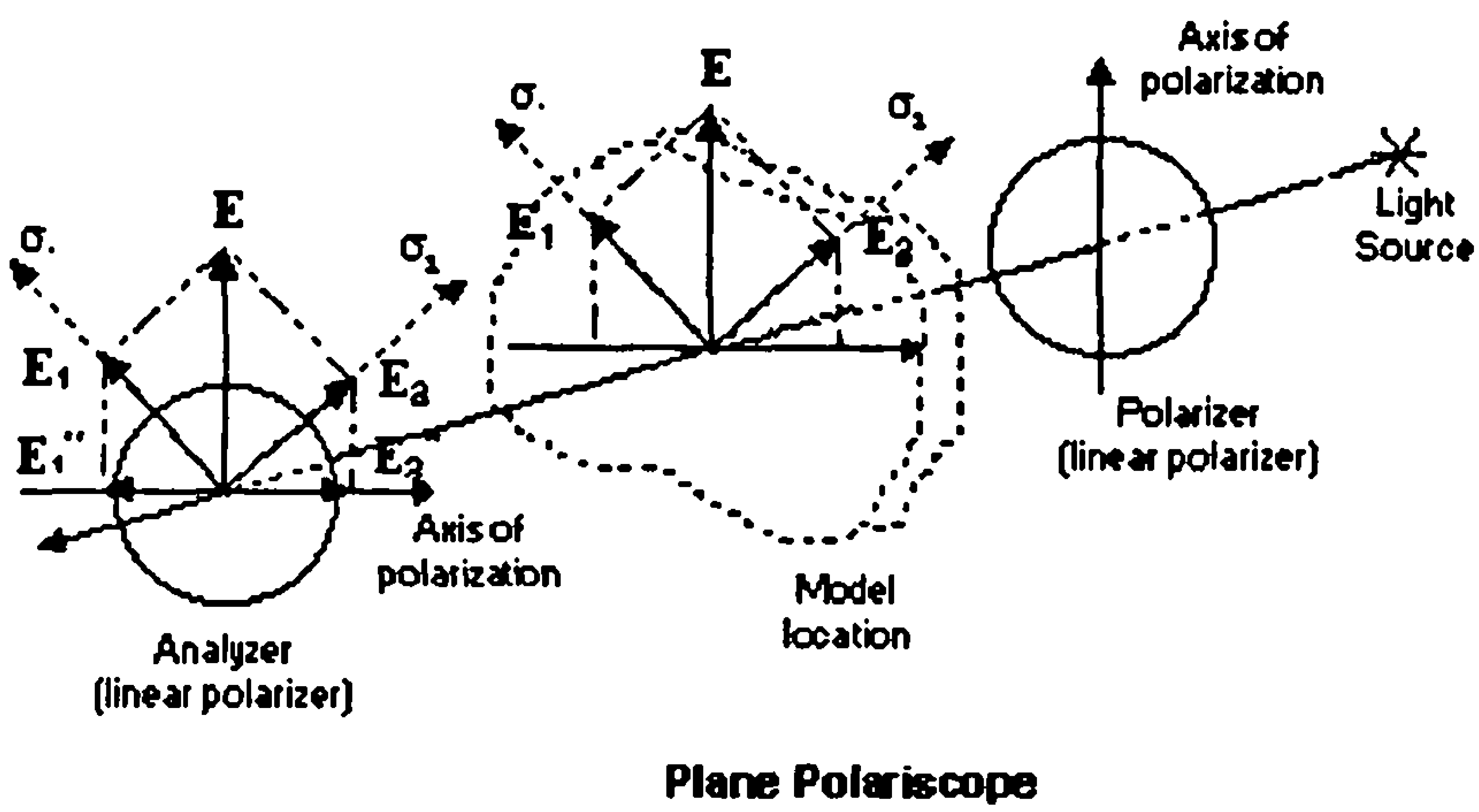


Figure 5: Diagrammatic Representation of Polariscope

Scale models of individual blocks, lintels and walls with openings were made of Araldite. A third scale Araldite model of a single block was used to give qualitative information on strain patterns in a single block and brought into focus the problems of fit. The block was subjected to a load distributed over a third of the block length, both in the centre and at the end. The observed stresses using a bench polariscope were seen to be largely due to random "high spots" of contact between the Araldite model and the base. Disregarding the effect of irregularities, very little spread of the load was observed which seemed to indicate that if the loaded material is too soft and flexible in relation to the applied load, compression merely takes place predominantly immediately under the load. Generally a "column" of high strain directly under the load was observed. The other Araldite models were used to get a "feel" for the spread of stress from various concentrated load conditions.

Further photoelastic work was undertaken using photoelastic coatings applied to Aircrete blockwork piers, using a reflective adhesive to provide a silver reflective surface behind the coating. To facilitate applying the coatings, smooth faced

Aircrete blocks were used in place of the more common "scratch" faced blocks. Using a powerful white light source, photographs were taken through a portable polariscope prior to and during the loading. Stress concentrations were noticed at the interfaces between mortar joints and blocks prior to loading. It is thought this was probably due to drying of the mortar joints during curing after the coating had been applied. During loading, the strain increased at the mortar joint/block interfaces while there was very little increase in the strain in the Aircrete blocks themselves.

Relatively low strength general purpose mortar is recommended for use with Aircrete blockwork (1:1:6 or 1:2:9 cement: lime: sand) to reduce the risk of cracking. Such mortars were used generally in the research to simulate normal practice. But after observing the development of high strains at the weak mortar/block interfaces, some piers were constructed using strong mortar with good adhesion. There was then generally no observable concentration of strain at the mortar/block interface leading to the hypothesis that the blockwork was behaving monolithically.

A large number of compression tests to failure were undertaken on single Aircrete blocks and small piers several blocks high. The ultimate loads were measured and the failure patterns observed and recorded. The blocks used were all 100 mm thick Aircrete as that was by far the most common size used in practice at that time. The concentrated loads were generally applied over the full block thickness. If this had been perfectly concentric, the stress in the structure would have been biaxial. As the loading was not perfectly concentric, triaxial stresses were set up. During the progress of the research, the shortcomings of the test apparatus became increasingly apparent. There was a small distortion in the loading frame which caused the hydraulic ram to descend at an angle of $1/2°$ to $1°$ to the vertical when viewed from the side of the frame. Although the effect on the component vertical force was in itself negligible there was an unquantifiable adverse effect on the ability to apply the loads uniformly from the point of first contact as the loading ram was not able to rotate. The failure modes indicated that the loads were applied eccentrically to the plane of the piers. This combined with the effect of standing the piers directly on the bottom steel joist of the testing frame without mortar bedding led to premature failure loads. Many of the tests produced failure at overall stresses lower than the measured compressive strength of the blocks probably due to these factors.

The British Standard for Concrete Blocks(9) current at the time gave very precise details of the loading conditions and methods of ironing out the high spots on the bed faces of the blocks which were not covered meticulously enough in the preparation of the specimens. The measured block compressive strength was sensitive to these details(76).

Some single block samples were tested under concentrated loading in a concrete block testing machine where the applied load could articulate, instead of in the test frame. Generally the quantified information gained from this earlier research is of little practical value in relation to enhanced bearing stresses, due to the shortcomings of the test equipment. It was learned that the measured strengths were very sensitive to the loading conditions in the same way that they are with the testing of individual blocks to obtain their compressive strength. The real value of the work is in the qualitative information on load spreading and the observations on modes of failure obtained.

The conclusions drawn stated that the presence of normal mortar joints significantly influences the strain distribution in the blockwork due to high strain concentrations at the weak block/mortar interfaces particularly in the perpend joints and that the failure is influenced by the resistance to tensile splitting of the blockwork(78) The comments on future work made at the end of the thesis suggest including different strengths, densities and thicknesses in the research and that that the further research should be based on physical testing in parallel with Finite Element analysis.

Some later work by CERAM(79) on concentred loads on normal density aac blockwork in general purpose looked at the effect of concentrated end loads of various lengths compared with axial uniformly distributed load and a central concentrated load. The mode of failure for the axial uniformly distributed load was vertical splitting perpendicular to the plane of the wall accompanied by some spalling from the face of the top course. The central concentrated load failed by local crushing under the load. The three types of end concentrated loads which were 100 mm, 200 mm and 300 mm long all failed by vertical splitting perpendicular to the plane of the wall. Whereas the central concentrated loads had an average enhancement of almost 2, the end loads showed small and inconsistent enhancements. There was nothing much to chose between the effect of the different lengths other than a slight tendency for the enhancement to reduce with length of bearing. In all cases there was measurable lift at the end of the wall remote from the end concentrated load. There was no lift at either end in the case of the uniformly distributed load or the central concentrated load.

The present research on low density Aircrete includes Finite Element analysis aims at quantifying load spread from a concentrated load. In the physical testing programme, strain measurements are taken across the perpend joints formed with the thin layer mortar. These are compared to the strains within the blocks to compare the quantified levels of strain and to establish whether the same strain concentrations occur in thin joints as in conventional mortar joints. The modes of failure are examined to establish whether thin joints play the same role in the failure patterns as general purpose mortar joints do. Attempts are made to confirm the stress enhancement immediately under the concentrated loads more convincingly than in the previous work.

Chapter 4. Test Series 1 to 5

4.1 Testing Equipment and Methods

The testing programme embraced half height Aircrete block test walls (1.8 m wide x 1.0 m high) and smaller test wallettes (675 mm x 675 mm) built in the laboratory and placed in a substantial steel testing frame for testing. Concentrated loads from a calibrated hydraulic jack were applied through steel bearing plates at the centre and at the ends of the test walls. Some of the loading was applied eccentrically to the longitudinal centre line of the wall. The faces of most of the half height test walls were fitted with Demec strain gauges.

Demountable mechanical strain gauges (Demec)

Demountable mechanical strain gauges (Demec) were developed at the Cement and Concrete Association to enable strain measurements to be made at different parts of a structure using a single instrument.

The Demec gauge (Figure 6) consists of a standard or a digital dial gauge attached to an Invar bar.

A fixed conical point is mounted at one end of the bar, and a moving conical point is mounted on a knife edge pivot at the opposite end. The pivoting movement of this second conical point is measured by the dial gauge.

A setting out bar is used to position pre-drilled stainless steel discs which are attached to the structure using a suitable adhesive.

Each time a reading has to be taken, the conical points of the gauge are inserted into the holes in the discs and the reading on the dial gauge noted. In this way, strain changes in the structure are converted into a change in the reading on the dial gauge.

The gauge has been designed so that only minor temperature corrections are required for changes in ambient temperature, and an Invar reference bar is provided for this purpose.

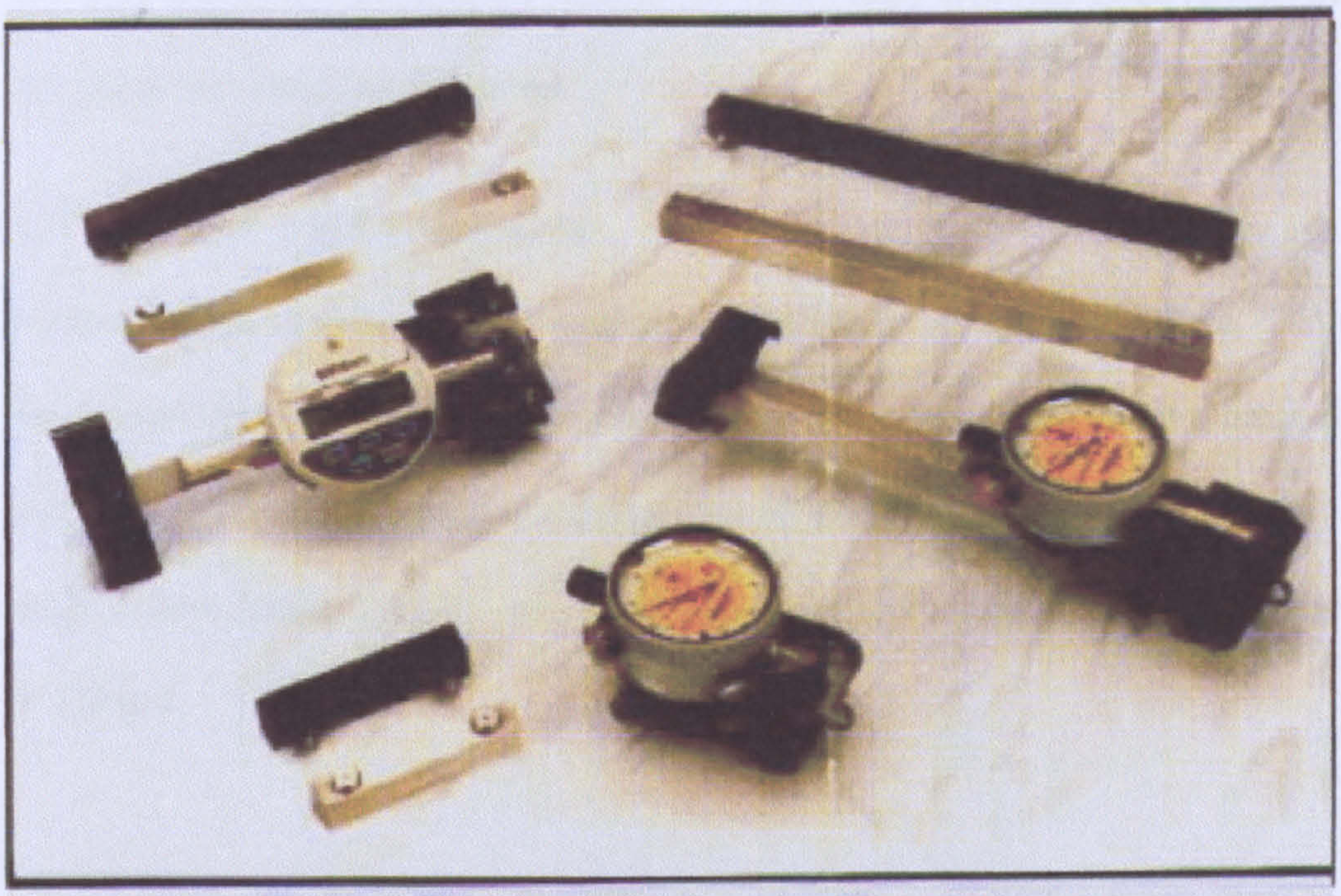


Figure 6: Demec Strain Gauges

Although originally designed for use on concrete structures, the Demec gauge is just as useful on masonry provided that they are positioned to include strains across mortar joints in addition to strains within the blocks.

To simplify analysis of the results, a software programme was used to plot the strains and deduce the formula of the curve through the measured strains.

Between tests each instrument, together with reference bar, setting out bar and user instructions was kept in a purpose made wooden box to protect it.

4.1.2 Test Frame

The test frame used was the existing Wall Test Frame in the Structures Laboratory at Kingston University which consisted of welded, 25.4 mm (1 inch) steel plate. The internal dimensions were Depth 310mm (12.25 inch), Width 2440mm (96 inch) and Height 1525mm (60 inch). Safe Working Load (SWL) of the frame was stated as 10 Tonne.

4.2 Test Series 1 to 5 Programme

Measurements of ultimate load capacities, observation of modes of failure, measurements of pre-failure strain distributions in the laboratory under a range of loading conditions and mathematical models of movements were all made on a range of low density Aircrete walls subjected to concentrated loading as detailed in Table 1.

Table 1: Test Programme Series 1 to 5

Test Series 1 - Concentrated loading tests on ½ storey height wall Type 1
1(a) Central load through 100mm x100mm bearing plate - Strain distribution in the elastic range
1(b) Central load through 100mm x150mm bearing plate - Strain distribution in the elastic range
1(c) End load through 100mm x150mm bearing plate - Strain distribution in the elastic range
Test Series 2 - Concentrated loading tests on ½ storey height wall Type 2
2(a) Central load through 100mm x100mm bearing plate was not undertaken
2(b) Central load through 100mm x150mm bearing plate, replicate of Test 1(b) with the addition of a cluster of Demec gauges under the central load to examine behaviour immediately under the load and loading taken to ultimate
2(c) End load through 100mm x150mm bearing plates, replicate of Test 1(c) with the addition of a cluster of Demec gauges under each load position to examine behaviour immediately under the loads and loaded both ends simultaneously to prevent tilting and loading taken to ultimate
Test Series 3 - Concentrated loading tests on wallettes built in general purpose mortar with different unit sizes Type 3
3(a), 3(b) and 3(c) 100mm x 100mm load applied at 1/3rd point on a whole unit in the top course and taken to ultimate. Three further wallettes 3(d), 3(e) and 3(f) where the top course was replaced with three courses of one third height blocks were then subjected to the same loading regime for comparison.
Test Series 4 - Compressive strength tests on thin layer mortar
Compressive strength tests on 42 No. mortar cubes
Test Series 5 - Concentrated loading from lintel bearings on wallette Type 5
Proof tests on lintel bearings for a range of mortars and two different block strengths

4.2.1 Test Series 1 - Wall Type 1 - Strain distribution in the elastic range

Three half storey height test walls Type 1 were built by a skilled craftsman, four courses high and three blocks long. The internal dimensions of the test frame and the space needed for the loading ram limited the maximum size of wall to be tested to approximately 1.0 m high (taken to be ½ storey height for the purposes of the tests and calculations) x 1.86m long in thin layer mortar.

In the design of masonry walls subjected to concentrated loads there are two limiting criteria. The overstress immediately below the bearing is limited and the stress at the mid height of the wall must not exceed the maximum distributed stress at that level (together with the stress from any other loads). Consequently the height of the test walls was taken as half storey height as this enabled the maximum spread to be measured in the test wall to be treated as the spread at mid height.

The Aircrete blocks used were 620mm long x 250mm high x 150mm thick low density Aircrete blocks with a compressive strength of 2.0 N/mm^2 and a nominal density of 350 kg/m^3. The thin layer mortar compressive strength was nominally 10 N/mm^2 at 28 days and was supplied in 22 kg bags.

The walls were carefully lifted into position in the steel testing frame and bedded on a 10mm layer of general purpose mortar (1:6 cement:sand with a plasticizer) laid on the steel joist forming the base of the test frame. Under normal site conditions the base course of the wall is similarly laid on a general purpose mortar bed. The purpose is to enable the base course to be laid as accurately as possible to provide a level horizontal top surface by ironing out any vertical discrepancies course using the thickness of the general purpose mortar bed. This is essential to enable the second and subsequent courses to be laid accurately. It also reduces the risk of premature failure being caused by high local stresses from any high spots that could otherwise exist between the base course and the steel joist base of the test frame.

The strains were measured using 150mm Demec gauges strategically arranged on the faces of the walls. The distribution of the Demec strain gauges for Walls Type 1 is shown in Figure 7 and Figure 8.

There were three loading conditions for Walls Type 1:

Test 1(a) Central load through 100mm x 100mm bearing plate

Test 1(b) Central load through 100mm x 150mm bearing plate

Test 1(c) End load through 100mm x 150mm bearing plate

The loading was applied to the walls through bearing plates which were 15 mm thick mild steel at a bearing stress ranging from zero up to approximately 1.2 N/mm^2 in steps of 0.1 N/mm^2. In all three cases, when looking at the face of the wall, the dimension of the steel bearing plate was 100 mm.

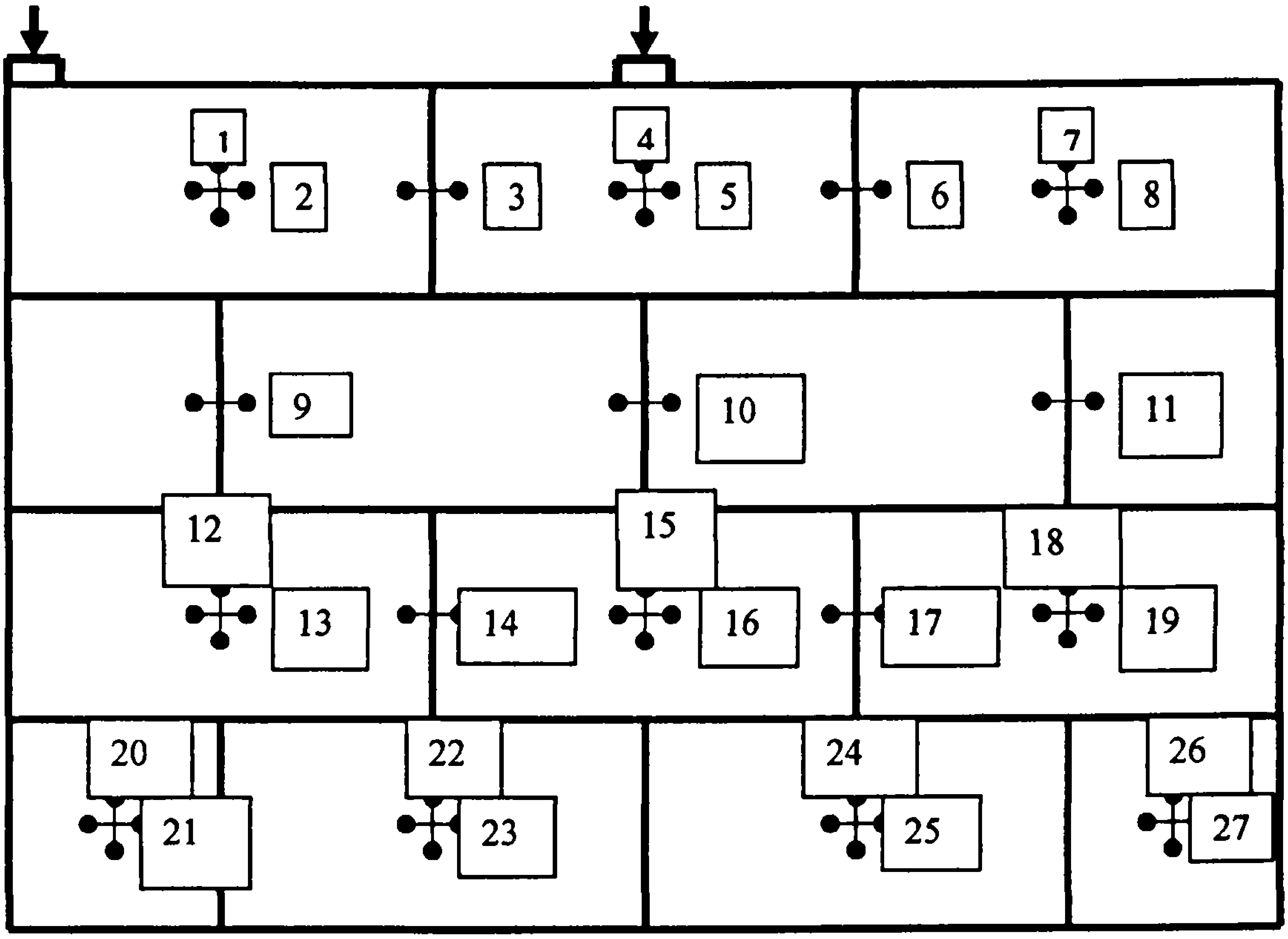


Figure 7: Wall Type 1 – 150 mm Demec Strain Gauge Numbering

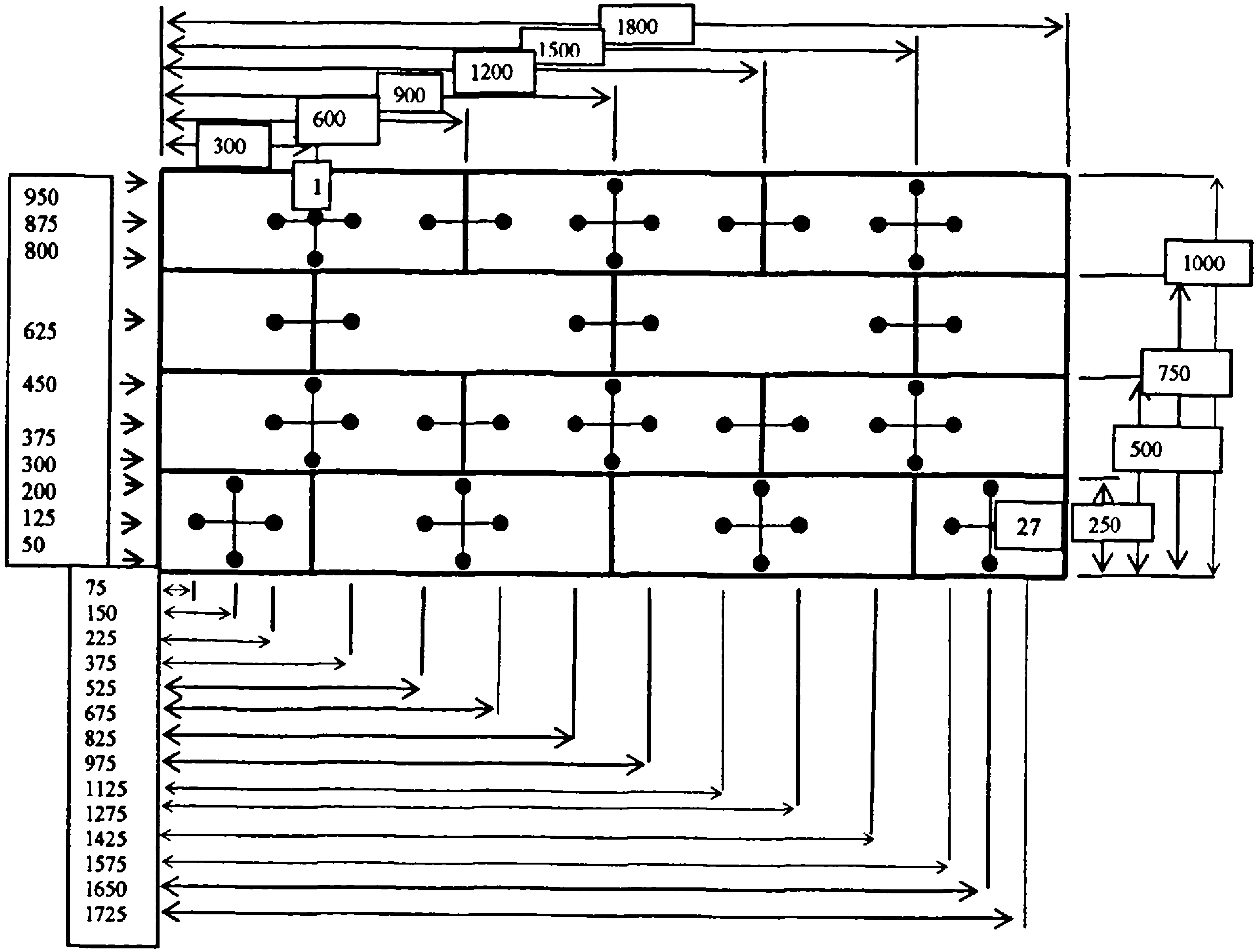


Figure 8: Wall Type 1 - 150 mm Demec strain gauge location dimensions along x and y axes (mm)

4.2.1.1a Wall Type 1 Test 1(a) - Central load through 100mm x 100mm bearing plate

The application of the central load is shown in Figure 9. The vertical demec gauge immediately below the load is Gauge 4.

Figure 9: Wall Type 1 Test 1(a) – Central load through 100mm x 100mm bearing plate

The calculated vertical compressive strains from the Demec gauge No 4 measurements on the central block in the top course directly under the load in Figure 10 show that the strain/stress relationship is almost perfectly linear.

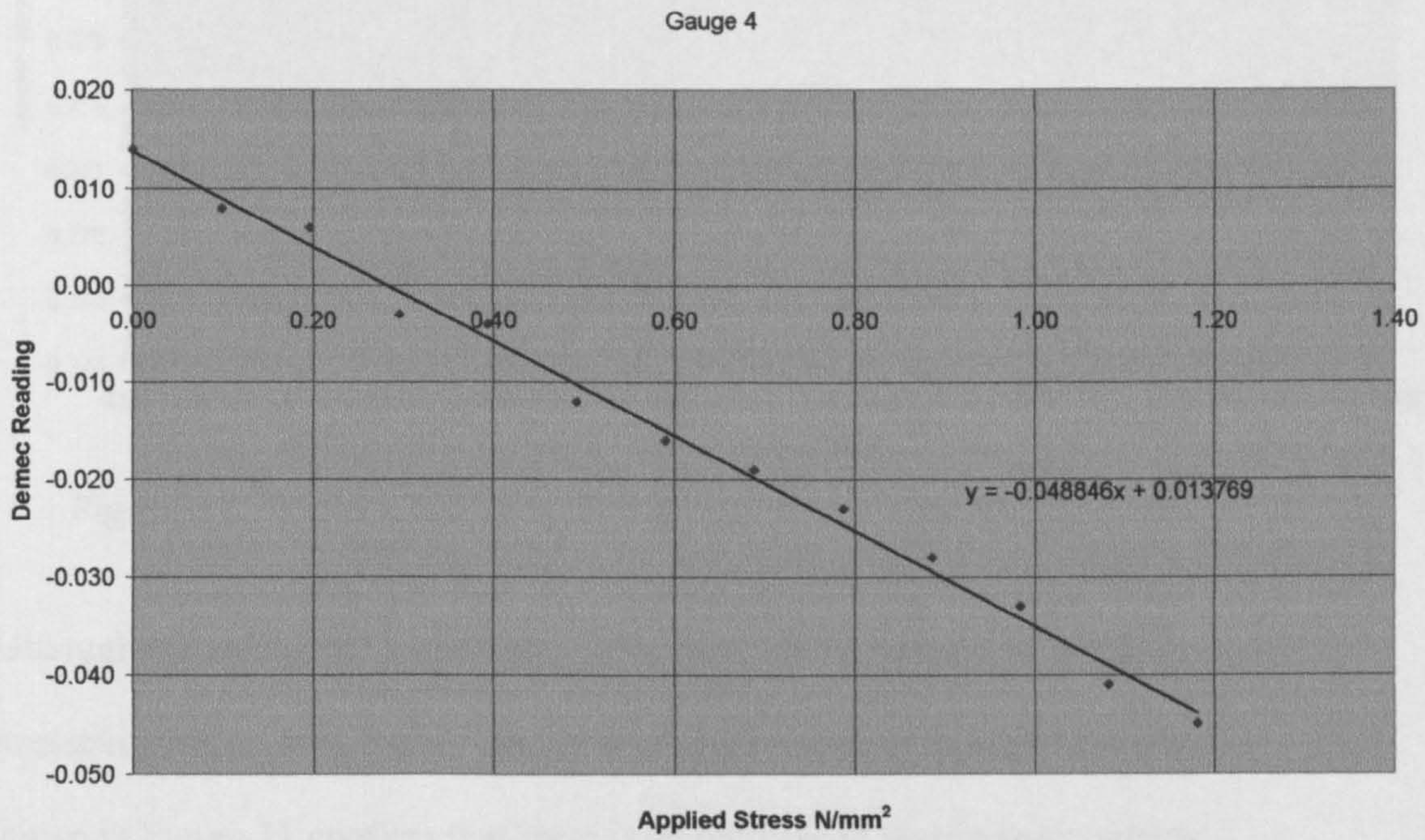


Figure 10: Wall 1 Test 1(a) – Vertical measurements gauge No 4

All of the other strain gauges maintained their original sign i.e. compression or tension throughout the loading range and maintained the same numerical relationship with the strains in the top central block. The geometry of the load spread remained consistent across the test wall throughout the loading range within the elastic range i.e. from zero up to 60% of the compressive strength of the blocks.

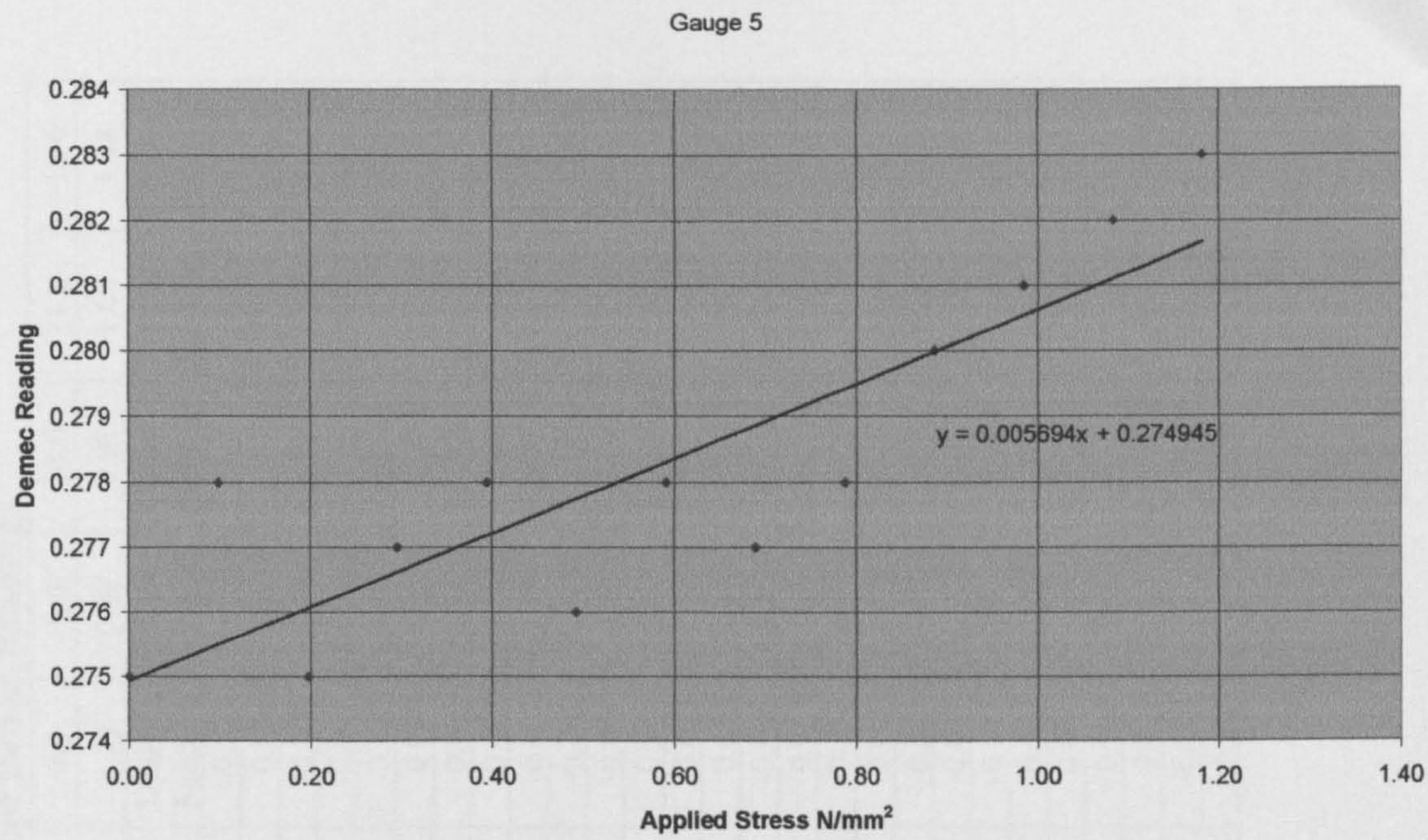


Figure 11: Wall 1 Test 1(a) – Horizontal measurements gauge No 5

Although the calculated horizontal strains from the Demec gauge No 5 measurements on the central block in the top course directly under the load shown in Figure 11 confirm that there is an increase in tensile strain against increased applied stress, the correlation is not as good as that for the vertical compressive strain in Gauge No. 4.

The (150mm long) Demec strain gauge readings in Table 2 were multiplied by 5.33×10^{-6} to obtain the strain ($\mu\varepsilon$) readings in Table 3. The strain distribution in the elastic range across the face of the wall is shown in Table 4.

Table 2: Wall Type 1 Test 1(a) – 100 x 100 mm central load Demec Readings

Load (kN)	0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0
Stress (N/mm^2)	0.00	0.10	0.20	0.30	0.39	0.49	0.59	0.69	0.79	0.89	0.98	1.08	1.18
Demec ID	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading	Demec Reading
1	-0.201	-0.205	-0.204	-0.205	-0.203	-0.204	-0.206	-0.205	-0.203	-0.203	-0.203	-0.202	-0.202
2	0.014	0.014	0.015	0.017	0.015	0.014	0.010	0.014	0.013	0.013	0.014	0.013	0.015
3	0.002	-0.001	-0.002	0.000	-0.003	-0.003	-0.006	-0.004	-0.003	-0.007	-0.006	-0.008	-0.006
4	0.014	0.008	0.006	-0.003	-0.004	-0.012	-0.016	-0.019	-0.023	-0.028	-0.033	-0.041	-0.045
5	0.275	0.278	0.275	0.277	0.278	0.276	0.278	0.277	0.278	0.280	0.281	0.282	0.283
6	0.015	0.012	0.011	0.009	0.012	0.008	0.008	0.007	0.006	0.006	0.008	0.005	0.005
7	0.021	0.020	0.021	0.023	0.022	0.016	0.021	0.023	0.023	0.022	0.016	0.021	0.023
8	0.003	0.005	0.001	0.001	0.002	0.001	0.004	-0.001	-0.001	0.002	0.002	0.000	0.000
9	-0.004	-0.005	-0.005	-0.004	-0.003	-0.002	-0.004	-0.004	-0.002	0.000	-0.001	-0.002	-0.002
10	-0.060	-0.060	-0.060	-0.059	-0.059	-0.058	-0.060	-0.057	-0.055	-0.054	-0.055	-0.056	-0.054
11	-0.005	-0.004	-0.005	-0.003	-0.004	-0.004	-0.005	-0.003	-0.004	-0.002	-0.004	-0.004	-0.003
12	0.247	0.245	0.243	0.244	0.245	0.242	0.239	0.237	0.239	0.236	0.236	0.238	0.238
13	0.008	0.008	0.008	0.010	0.008	0.007	0.009	0.009	0.013	0.010	0.010	0.012	0.013
14	0.017	0.016	0.014	0.014	0.015	0.016	0.014	0.015	0.019	0.016	0.017	0.017	0.019
15	0.001	0.002	-0.002	-0.003	-0.003	-0.007	-0.010	-0.009	-0.008	-0.010	-0.011	-0.016	-0.014
16	0.010	0.011	0.012	0.015	0.013	0.007	0.011	0.012	0.014	0.012	0.013	0.013	0.014
17	0.328	0.333	0.332	0.335	0.337	0.331	0.333	0.330	0.335	0.330	0.334	0.335	0.333
18	0.012	0.012	0.013	0.008	0.005	0.005	0.003	-0.012	0.000	-0.003	-0.015	-0.001	-0.013
19	-0.363	-0.360	-0.359	-0.362	-0.364	-0.364	-0.359	-0.363	-0.361	-0.361	-0.359	-0.359	-0.360
20	-0.019	-0.018	-0.021	-0.020	-0.024	-0.023	-0.021	-0.024	-0.024	-0.025	-0.021	-0.025	-0.022
21	-0.140	-0.138	-0.137	-0.137	-0.136	-0.138	-0.135	-0.139	-0.137	-0.137	-0.138	-0.138	-0.138
22	-0.258	-0.258	-0.259	-0.258	-0.261	-0.262	-0.263	-0.263	-0.262	-0.261	-0.261	-0.261	-0.261
23	0.005	0.008	0.006	0.008	0.007	0.008	0.007	0.007	0.006	0.008	0.012	0.010	0.011
24	0.008	0.006	0.003	0.008	0.007	0.006	0.006	0.006	0.005	0.004	0.002	0.004	0.003
25	0.009	0.011	0.012	0.008	0.013	0.009	0.009	0.012	0.013	0.014	0.016	0.015	0.017
26	0.017	0.018	0.019	0.020	0.019	0.022	0.020	0.000	-0.002	0.018	0.003	0.000	-0.009
27	0.004	0.004	0.007	0.002	0.004	-0.001	-0.002	0.006	0.005	0.001	0.009	0.002	0.004

Table 3: Wall Type 1 Test 1(a) - 100x100 mm central load elastic - strain readings corrected using a best fit straight line

Demec ID	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)	Strain (με)
1	0.0	0.5	1.0	1.4	1.9	2.4	2.9	3.3	3.8	4.3	4.8	5.2	5.7
2	0.0	-0.5	-1.1	-1.6	-2.2	-2.7	-3.3	-3.8	-4.4	-4.9	-5.5	-6.0	-6.6
3	0.0	-3.5	-7.0	-10.5	-14.0	-17.4	-20.9	-24.4	-27.9	-31.4	-34.9	-38.4	-41.9
4	0.0	-25.2	-50.4	-75.7	-100.9	-126.1	-151.3	-176.6	-201.8	-227.0	-252.2	-277.4	-302.7
5	0.0	2.9	5.9	8.8	11.8	14.7	17.6	20.6	23.5	26.5	29.4	32.3	35.3
6	0.0	-3.7	-7.4	-11.2	-14.9	-18.6	-22.3	-26.0	-29.7	-33.5	-37.2	-40.9	-44.6
7	0.0	0.1	0.2	0.3	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.0
8	0.0	-1.3	-2.5	-3.8	-5.1	-6.3	-7.6	-8.9	-10.1	-11.4	-12.7	-14.0	-15.2
9	0.0	1.6	3.2	4.8	6.3	7.9	9.5	11.1	12.7	14.3	15.9	17.4	19.0
10	0.0	2.9	5.8	8.6	11.5	14.4	17.3	20.2	23.1	25.9	28.8	31.7	34.6
11	0.0	0.6	1.2	1.7	2.3	2.9	3.5	4.0	4.6	5.2	5.8	6.3	6.9
12	0.0	-4.6	-9.1	-13.7	-18.2	-22.8	-27.3	-31.9	-36.4	-41.0	-45.5	-50.1	-54.6
13	0.0	2.0	4.0	6.1	8.1	10.1	12.1	14.1	16.1	18.2	20.2	22.2	24.2
14	0.0	1.2	2.4	3.6	4.8	6.1	7.3	8.5	9.7	10.9	12.1	13.3	14.5
15	0.0	-7.2	-14.4	-21.5	-28.7	-35.9	-43.1	-50.2	-57.4	-64.6	-71.8	-78.9	-86.1
16	0.0	1.0	2.1	3.1	4.2	5.2	6.2	7.3	8.3	9.3	10.4	11.4	12.5
17	0.0	0.8	1.6	2.4	3.2	4.0	4.8	5.6	6.5	7.3	8.1	8.9	9.7
18	0.0	-11.2	-22.3	-33.5	-44.6	-55.8	-66.9	-78.1	-89.2	-100.4	-111.6	-122.7	-133.9
19	0.0	1.0	1.9	2.9	3.8	4.8	5.7	6.7	7.6	8.6	9.5	10.5	11.4
20	0.0	-2.0	-4.0	-6.0	-8.0	-9.9	-11.9	-13.9	-15.9	-17.9	-19.9	-21.9	-23.9
21	0.0	0.1	0.3	0.4	0.6	0.7	0.9	1.0	1.2	1.3	1.4	1.6	1.7
22	0.0	-1.5	-3.1	-4.6	-6.1	-7.6	-9.2	-10.7	-12.2	-13.7	-15.3	-16.8	-18.3
23	0.0	1.9	3.9	5.8	7.7	9.7	11.6	13.5	15.4	17.4	19.3	21.2	23.2
24	0.0	-1.7	-3.5	-5.2	-6.9	-8.6	-10.4	-12.1	-13.8	-15.6	-17.3	-19.0	-20.8
25	0.0	3.0	6.1	9.1	12.1	15.1	18.2	21.2	24.2	27.2	30.3	33.3	36.3
26	0.0	-11.0	-21.9	-32.9	-43.8	-54.8	-65.7	-76.7	-87.6	-98.6	-109.5	-120.5	-131.4
27	0.0	0.1	0.2	0.3	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4

Table 4: Wall Type 1 Test 1(a) - 100mm x 100mm central concentrated load measured vertical and horizontal strains

Vertical Location (fraction of height down)	Horizontal Location (Fraction of wall length from left)						
	0.083	0.167	0.333	0.500	0.667	0.833	0.917
0.125		5.7		**-302.7**		1.0	
0.375							
0.625		**-54.6**		**-86.1**		**-133.9**	
0.875	**-23.9**		**-18.3**		**-20.8**		**-131.4**
Vertical Strains at 1.18 N/mm^2 compressive stress under 12 kN central concentrated load							
Compressive vertical strains shown bold							
0.125		-6.6	-41.9	35.3	-44.6	-15.2	
0.375		19.0		34.6		6.9	
0.625		24.2	14.5	12.5	9.7	11.4	
0.875	1.7		23.2		36.3		1.4
Horizontal Strains (με) at 1.18 N/mm^2 compressive stress under 12 kN central concentrated load							

4.2.1.1b Wall Type 1 Test 1(b) – Central load through 100mm x 150mm bearing plate - Strain distribution in the elastic range

From the demec gauge readings (Table 5), the largest horizontal strains calculated were the compressive strains at each end of the central block in the top course similar to Test 1(a). This seems to indicate a "buttressing" effect at the ends of the central block from the outer two blocks in the top course.

As in Test 1(a), in the bottom course, the vertical demec readings were all compressive and the horizontal readings in the three bottom courses were all tensile (Table 6). Thus indicating that the load was distributed over the full width of the wall before reaching the level of the base.

Comparison between the behaviour of Test 1(a) and Test 1(b)

The vertical compressive stress (applied by the central concentrated load) / strain (as measured by the demec strain gauges) relationship remained remarkably linear over the range in each of the tests. Because the bearing plate through which the concentrated load was applied was 50% larger in area in Test 1(b) than in Test 1(a), the applied load was also larger to obtain a similar bearing stress. However, the vertical compressive strains in Test 1(b) were seen to be up to double those in Test 1(a).

The load in Test 1(b) was applied over the full 150mm thickness of the wall. Therefore it could only spread in one direction i.e. along the length of the wall. The load in Test 1(a), however, was only applied across the centre 100 mm of the thickness. It could therefore spread in both directions. Based on a 45^0 spread, at 25mm below the top surface the load in Test 1(a) is spread over 22,500 mm^2 and the load in Test 1(b) is also spread over 22,500 mm^2. The stress (and therefore the elastic strain) in Test 1(b) at 25mm below would be 18000/(150 x 150) N/mm^2 compared with Test 1(a) which is 12000/(150 x 150) N/mm^2 i.e. the stress in the wall from the concentrated load in Test 1(b) is increases from approximately equal directly under the bearing to 50% higher 25 mm below. The Demec strain measurements confirm the proposition that the load in Test 1(a) spread in both directions.

Page and Hendry state "In the zone immediately below the loading plate a triaxial compressive stress state is developed whereas, a little further down the wall, the stress state changes to one of vertical compression and biaxial tension." They further qualify that statement for concentrated loads over the full thickness of the wall which induce a state of biaxial stress. Unfortunately they do not quantify "a little further down the wall".

It can be seen from the results of Tests 1(a) and 1(b) that directly beneath the load, within the triaxial (Test 1(a)) or biaxial (Test 1(b)) stress state, vertical compressive stresses and horizontal tensile stresses have developed at the position of the demec gauge 100mm below the load. At that level the vertical strains change to tensile further along the walls both sides of the load while the horizontal strains remain tensile. At the lower levels the vertical compressive strains and the horizontal tensile strains are spread out over the full length of the wall.

Table 5: Wall Type 1 Test 1(b) Central 100mm x 150mm load Demec Readings

Load (kN)	0.0	1.5	3.0	4.5	6.0	7.5	9.0	10.5	12.0	13.5	15.0	16.5	18.0	0
Stress (N/mm^2)	0.00	0.10	0.20	0.30	0.41	0.51	0.61	0.71	0.81	0.91	1.02	1.12	1.22	0.00
Demec ID	Demec Readings													
1	-0.195	-0.198	-0.198	-0.198	-0.196	-0.196	-0.198	-0.196	-0.198	-0.198	-0.199	-0.198	-0.198	-0.200
2	0.017	0.015	0.015	0.016	0.017	0.014	0.014	0.013	0.014	0.012	0.015	0.014	0.012	0.014
3	0.005	0.004	-0.001	-0.002	-0.003	-0.002	-0.005	-0.005	-0.006	-0.007	-0.009	-0.009	-0.011	-0.001
4	0.017	0.012	0.005	-0.005	-0.009	-0.014	-0.036	-0.048	-0.061	-0.068	-0.077	-0.084	-0.096	0.012
5	0.277	0.279	0.279	0.276	0.281	0.280	0.280	0.278	0.281	0.285	0.286	0.288	0.284	0.271
6	0.018	0.019	0.018	0.016	0.017	0.014	0.012	0.009	0.010	0.009	0.010	0.006	0.006	0.015
7	0.022	0.019	0.018	0.020	0.019	0.020	0.020	0.019	0.020	0.019	0.020	0.018	0.018	0.021
8	0.005	0.007	0.005	0.003	0.004	0.006	0.004	0.004	0.004	0.005	0.005	0.004	0.004	0.003
9	-0.001	0.001	-0.001	-0.001	0.002	0.000	0.002	0.002	0.002	0.002	0.002	0.002	0.000	-0.004
10	-0.055	-0.055	-0.058	-0.060	-0.055	-0.054	-0.056	-0.052	-0.053	-0.052	-0.054	-0.053	-0.051	-0.060
11	-0.001	-0.002	-0.003	-0.002	0.000	0.000	0.000	0.000	-0.002	-0.001	0.000	-0.001	-0.002	-0.003
12	0.254	0.250	0.248	0.245	0.247	0.241	0.240	0.238	0.238	0.236	0.235	0.231	0.230	0.250
13	0.011	0.010	0.011	0.011	0.012	0.013	0.011	0.012	0.014	0.016	0.017	0.017	0.015	0.008
14	0.017	0.018	0.018	0.018	0.016	0.018	0.019	0.019	0.018	0.017	0.018	0.019	0.019	0.016
15	0.004	0.001	0.000	-0.002	-0.005	-0.016	-0.019	-0.021	-0.021	-0.025	-0.026	-0.029	-0.031	0.000
16	0.009	0.012	0.011	0.012	0.012	0.012	0.014	0.015	0.013	0.016	0.015	0.017	0.018	0.006
17	0.332	0.333	0.332	0.331	0.333	0.334	0.334	0.334	0.334	0.335	0.334	0.334	0.335	0.328
18	-0.006	-0.007	-0.008	-0.012	-0.009	-0.009	-0.011	-0.013	-0.013	-0.016	-0.018	-0.018	-0.017	0.004
19	-0.365	-0.361	-0.362	-0.365	-0.359	-0.360	-0.360	-0.358	-0.361	-0.360	-0.359	-0.358	-0.357	-0.363
20	-0.014	-0.014	-0.017	-0.016	-0.015	-0.018	-0.017	-0.016	-0.021	-0.021	-0.022	-0.024	-0.021	-0.019
21	-0.136	-0.135	-0.136	-0.135	-0.133	-0.134	-0.133	-0.135	-0.133	-0.134	-0.132	-0.133	-0.133	-0.140
22	-0.259	-0.259	-0.260	-0.261	-0.258	-0.258	-0.256	-0.256	-0.264	-0.262	-0.265	-0.261	-0.266	-0.258
23	0.006	0.009	0.007	0.006	0.010	0.011	0.012	0.009	0.010	0.008	0.013	0.011	0.012	0.004
24	-0.009	-0.008	-0.006	-0.009	-0.005	-0.009	-0.008	-0.009	-0.010	-0.014	-0.013	-0.014	-0.017	0.009
25	0.008	0.010	0.010	0.012	0.012	0.011	0.015	0.016	0.014	0.016	0.019	0.016	0.021	0.009
26	0.005	0.001	0.003	0.003	0.004	0.005	0.001	0.000	0.005	-0.001	0.000	-0.001	0.003	0.018
27	0.010	0.010	0.008	0.004	0.010	0.010	0.009	0.010	0.008	0.011	0.008	0.010	0.009	0.008

Table 6: Wall Type 1 Test 1(b) - 100mm x 150mm central concentrated load vertical and horizontal strains

Vertical Location (fraction of height down)	Horizontal Location (Proportion of wall length)						
	0.083	0.167	0.333	0.500	0.667	0.833	0.917
0.125		**-9.1**		**-637.1**		**-8.1**	
0.375							
0.625		**-119.1**		**-200.3**		**-65.0**	
0.875	**-48.1**		**-29.9**		**-46.0**		**-17.2**
Vertical Strains (με) at 1.22 N/mm^2 compressive stress under central concentrated load							
Compressive vertical strains shown bold							
0.125		-19.0	-76.3	49.2	-73.4	-6.0	
0.375		12.0		28.1		3.5	
0.625		35.5	6.7	39.4	15.8	30.9	
0.875	16.2		26.0		58.0		3.9
Horizontal Strains (με) at 1.22 N/mm^2 compressive stress under 18 kN central concentrated load							

4.2.1.1c Wall Type 1 Test 1(c) – End load through 100mm x150mm bearing plate - Strain distribution in the elastic range

The test was undertaken with a concentrated load at one end only (Figure 12).

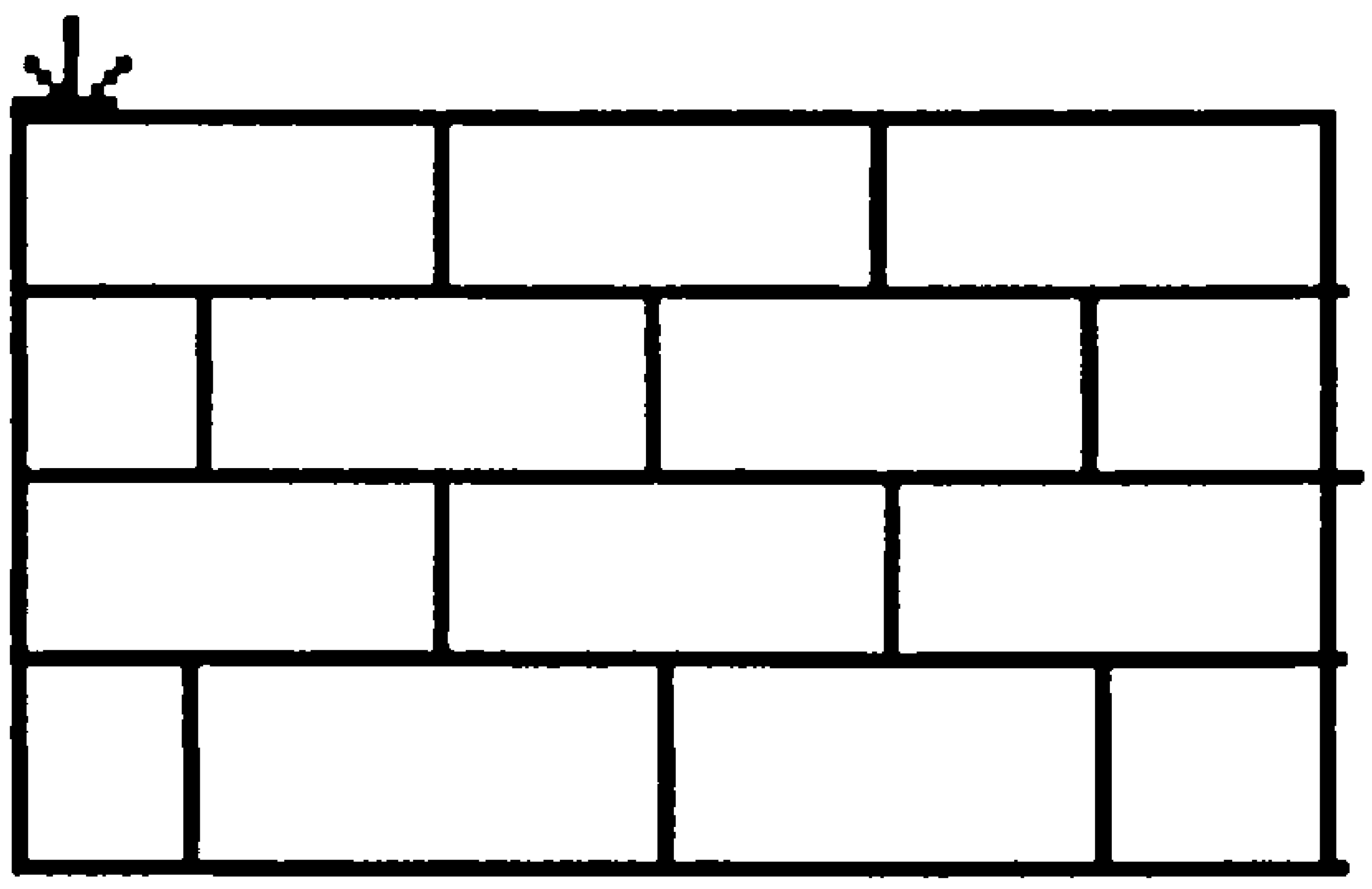


Figure 12: Test 1(c) End load through 100mm x150mm bearing plate

Test 1(c) was not very satisfactory. As the wall was loaded at one end only (the left hand end), there was a tendency for the wall to rotate downwards at the loaded end. This prevented the concentrated load from fully spreading across the wall length and the strains in the bottom course make no sense because of the rotation along the length of the wall. The position of the nearest vertical strain Demec gauge was 250mm away horizontally from the vertical centre line of the concentrated load at mid height of the top course. The vertical compressive strain gauge readings were so low, as indicated in Figure 13 and Table 7, that they could not be taken as a very useful indication of the spread of strain from under the load.

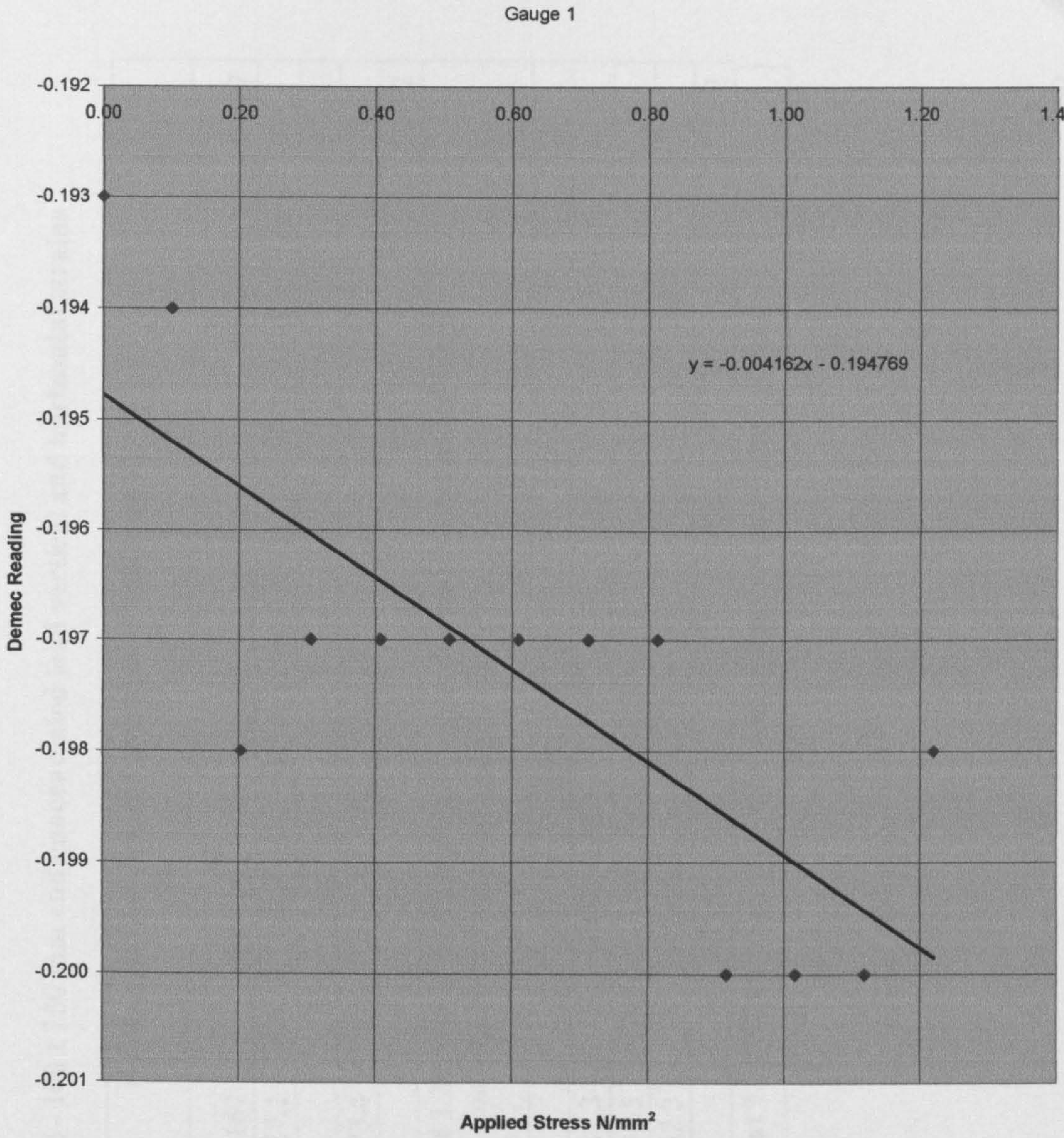


Figure 13: Wall Type 1 Test 1(c) – 100mm x 150mm end load vertical measurements Gauge 1

Table 7: Wall Type 1 Test 1(c) - 100 x 150 mm end concentrated load vertical and horizontal strains

Vertical Location (fraction of height down)	Horizontal Location (Proportion of wall length)						
	0.083	0.167	0.333	0.500	0.667	0.833	0.917
0.125		**-27.1**		6.7		6.3	
0.375							
0.625		**-73.4**		43.2		6.7	
0.875	38.7		1.8		**-5.6**		37.2
Vertical Strains (με) at 1.22 N/mm² compressive stress under end concentrated load *Compressive vertical strains shown bold*							
0.125		187.3	40.8	17.6	2.8	10.9	
0.375		-35.5		-10.2		15.1	
0.625		-32.3	-41.1	-5.3	6.0	-1.1	
0.875	-29.9		23.5		0.0		11.2
Horizontal Strains (με) at 1.22 N/mm² compressive stress under end concentrated load							

4.2.2 Test Series 2 Wall Type 2 - Strain distribution in the elastic range

The test walls were replicates of Wall Type 1, i.e. 4 courses high, three blocks long (1000mm x 1800mm nominal) - 2.0 N/mm^2 block compressive strength with a similar arrangement of 150 mm long demec gauges on the wall front faces as in Figure 14. Clusters of 50 mm long demec gauges were added on the top course of blocks under the load positions. The cluster layout and 50 mm long gauge numbering and direction is shown in Figure 15.

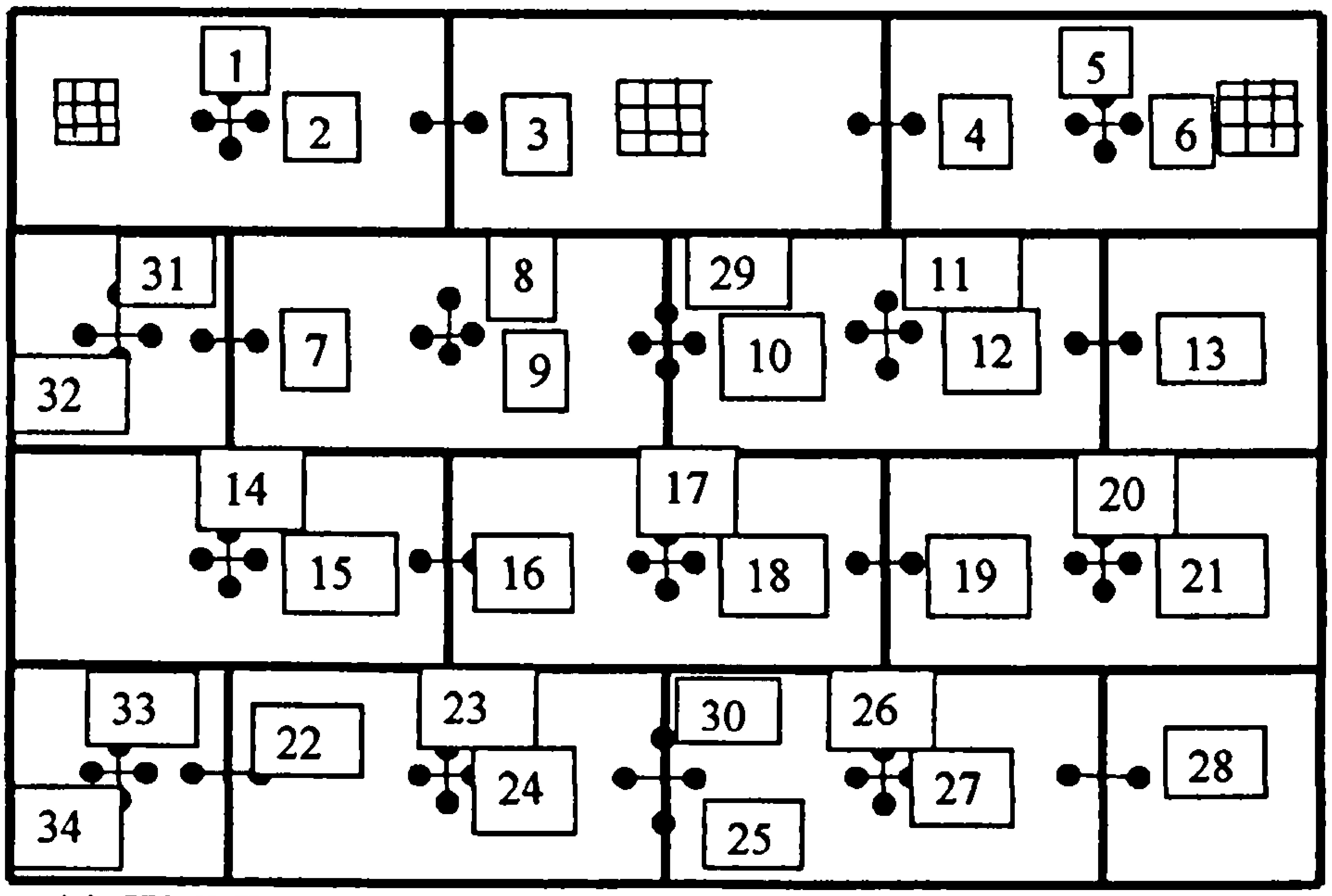


Figure 14: Wall Type 2 Numbering of 150 mm long Demec gauges across the wall

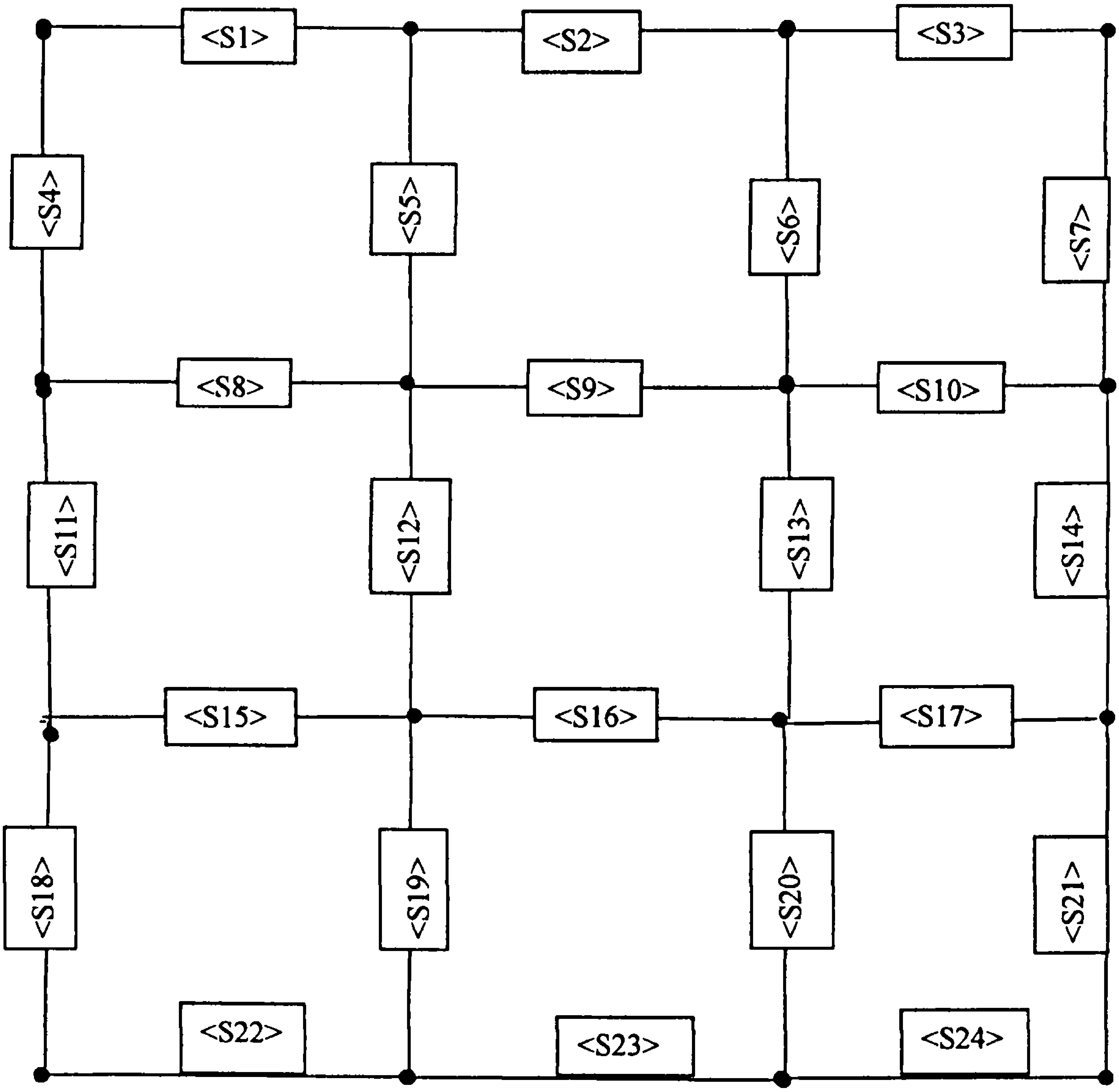


Figure 15: Wall Type 2 - Numbering and direction of 50mm long Demec strain gauges

Details of 50 mm Demec gauges layout

The clusters of 50 mm Demec gauges, numbered S1, S2, S3 etc. to S24 as shown in Figure 15, were positioned at three locations i.e. Centrally on the centre block in the top course, at the right hand end (under the end load) of the right hand block in the top course and at the left hand end (under the end load) of the left hand block in the top course.

The 50mm Demec strain gauge readings were multiplied by 1.973×10^{-5} to obtain the strain ($\mu\varepsilon$).

4.2.2.1 Wall Type 2 Test 2(a)

Wall Type 2 Test 2(a) had been intended to replicate Test 1(a) but was omitted from the tests.

4.2.2.2 Wall Type 2 Test 2(b) Central load through 100 mm x 150 mm bearing plate

Test 2(b) was a replicate of Test 1(b) except for the cluster of 50 mm long demec gauges under the central load used to examine the strain behaviour immediately under the load then taking the loading to ultimate (Figure 16).

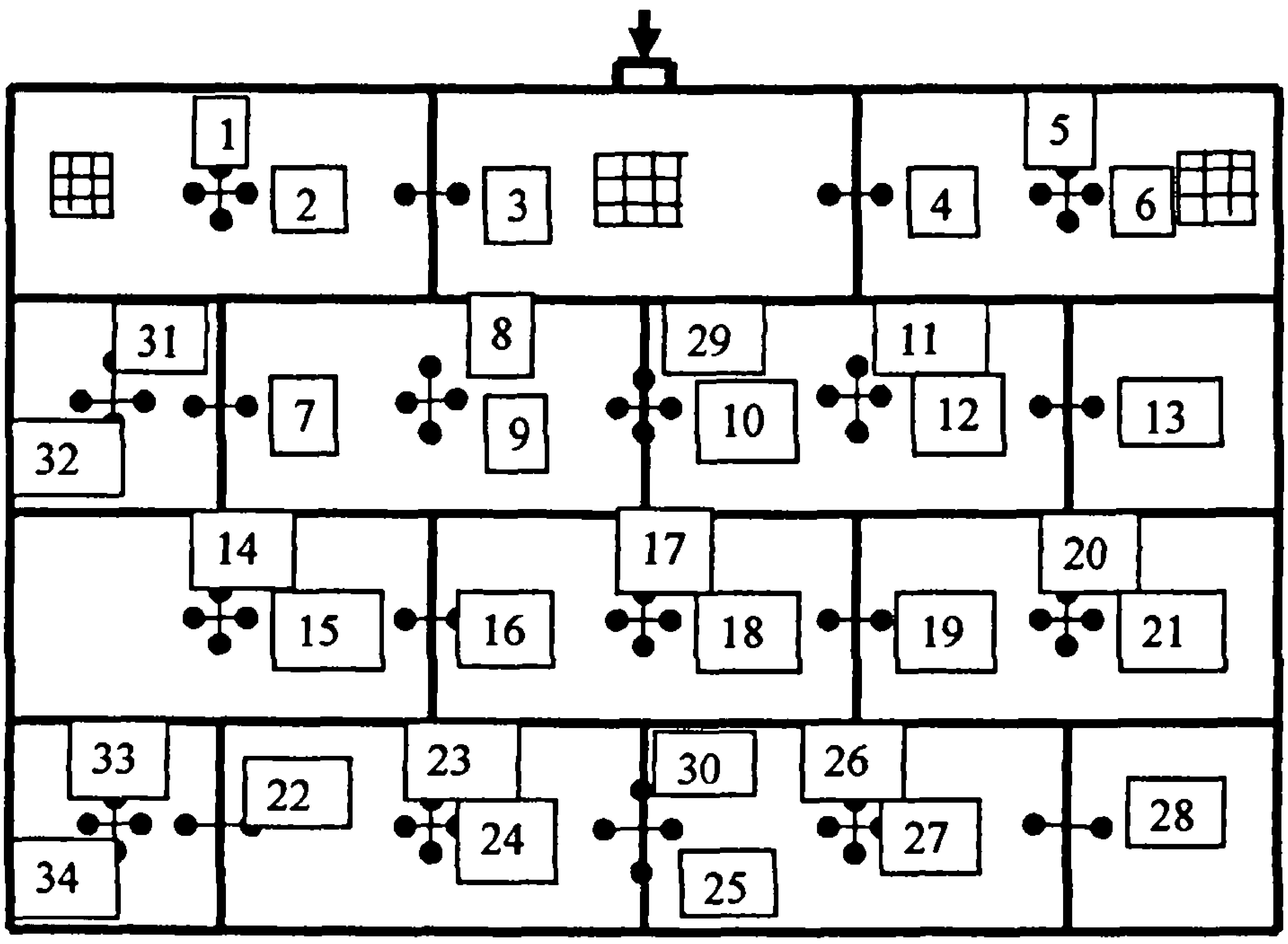
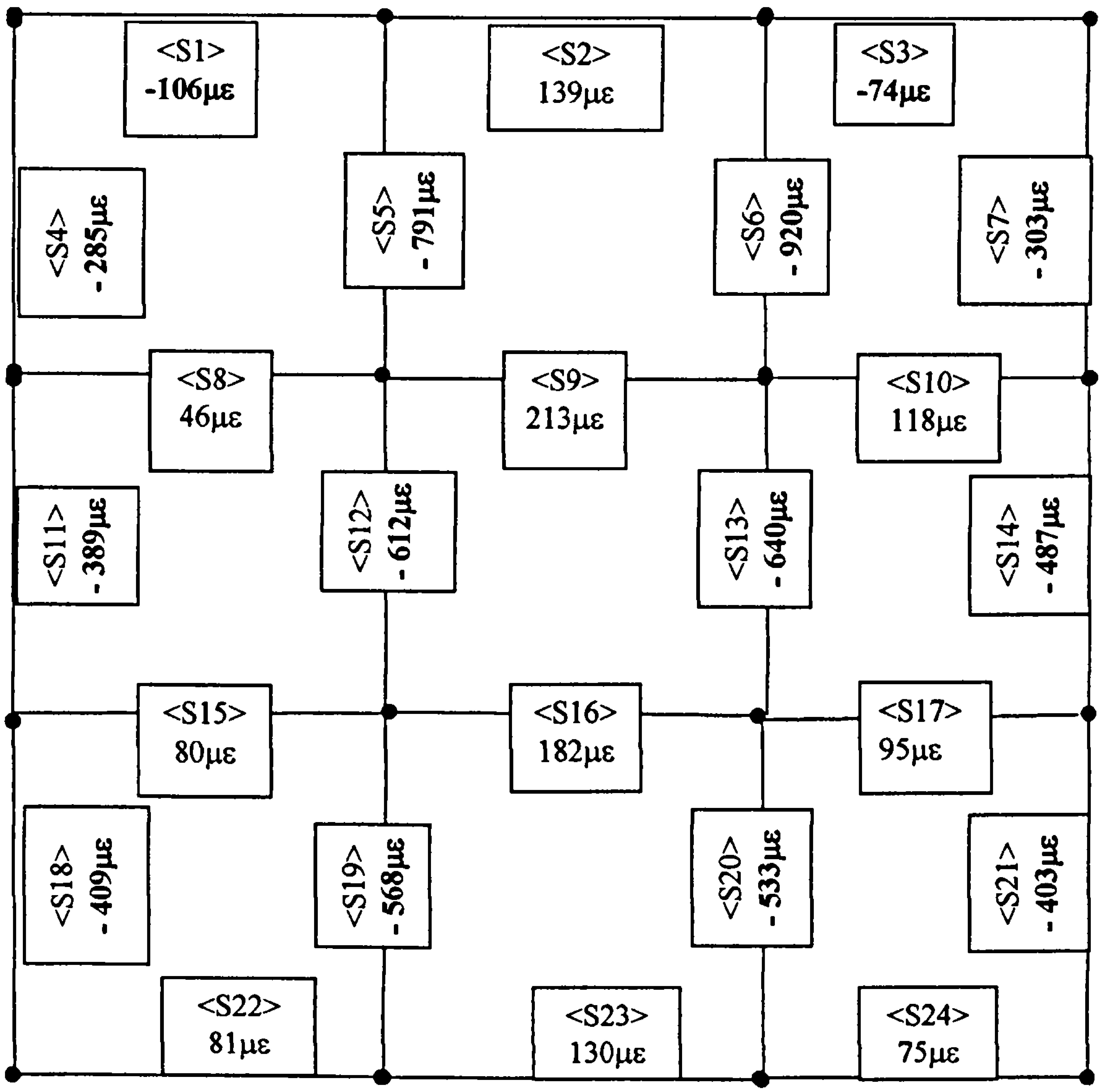


Figure 16: Wall Type 2 Test 2(b) Central load through 100mm x 150mm bearing plate

The failure load represented by cracking under the central load was 41.5 kN. The load was applied through a 150 x100 mm bearing plate giving a stress under the bearing plate = 41500/(150x100) = 2.77 N/mm^2 i.e. 38.5% enhancement. The ultimate failure load was 44.0 kN giving a stress = 44000/(150x100) = 2.93 N/mm^2 i.e. 47.0% enhancement over the mean block compressive strength.



Compressive strains shown bold

Figure 17: Wall Type 2 Test 2(b) Vertical and Horizontal Strains (με) in the central block in top course when the wall was subjected to 24 kN central loading

Table 8: Wall Type 2 Test 2(b) Vertical and Horizontal Strains (με) in the central block in top course when the wall was subjected to 24 kN central loading

Demec	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11	S12	S13	S14	S15	S16	S17	S18	S19	S20	S21	S22	S23	S24
Vstr				**-285**	**-791**	**-920**	**-303**				**-389**	**-612**	**-640**	**-487**				**-409**	**-568**	**-533**	**-403**			
Hstr	-106	139	-74					46	213	118					80	182	95					81	130	75

Compressive vertical strains shown bold

The values in Figure 17 and Table 8 show that there is vertical compressive strain across the whole cluster of 50 mm long Demec gauges on the centre of the face of the central block in the top course. The strain was greatest in the centre of the top row of vertical strain gauges, tending towards equalling out across the full 150mm width of the cluster in the third row of 50 mm long demecs down. The mean vertical strain on the centre line over 150mm

$= -(791+920+612+640+568+533)/6 =-4064/6 = -676\mu\varepsilon$

The horizontal strains are generally tensile except for two outer horizontal strain gauges in the first row immediately under the load with the tensile readings tending to spread as indicated by the lower strain gauge readings.

Table 9 shows the distribution of vertical and horizontal strains at positions across the whole face of Wall Type 2 under Test 2(b). It can be seen that the vertical compression was applied over the full width of the wall in the base course.

Tan of the angle of spread = 1000/850 =1.1765. Therefore the spread was greater than 40^0 to the vertical.

Table 9: Wall Type 2 – Test 2(b) - Distribution of vertical and horizontal strains (με) across the wall under central 100mm x 150mm load

Vertical Location (fraction of height down)	**Horizontal Location (Proportion of Wall Length)** *Horizontal dimensions taken from bottom left hand corner of wall*						
	0.083	0.167	0.333	0.500	0.667	0.833	0.917
0.125		8		**-677**		11	
0.375	23		**-50**	**-262**	**-62**		
0.625		**-8**		**-164**		**-26**	
0.875	**-15**		**-60**	**-119**	**-67**		
Vertical Strains (με) under central concentrated load							
Vertical compressive strains shown bold							
0.125	3		-60	198	-59		15
0.375				118			
0.625				96			
0.875	-9	26	36	68	46	-6	

Horizontal Strains (με) under central concentrated load

4.2.2.3 Wall Type 2 - Test 2(c) – End load through 100mm x150mm bearing plates

Test 2(c) was a replicate of Test 1(c) with a cluster of 50 mm long demec gauges added under each load position, to examine the strain behaviour immediately under the loads, in addition to the 150 mm demec gauges used to measure the strain distribution over the wall as a whole. See Figure 18.

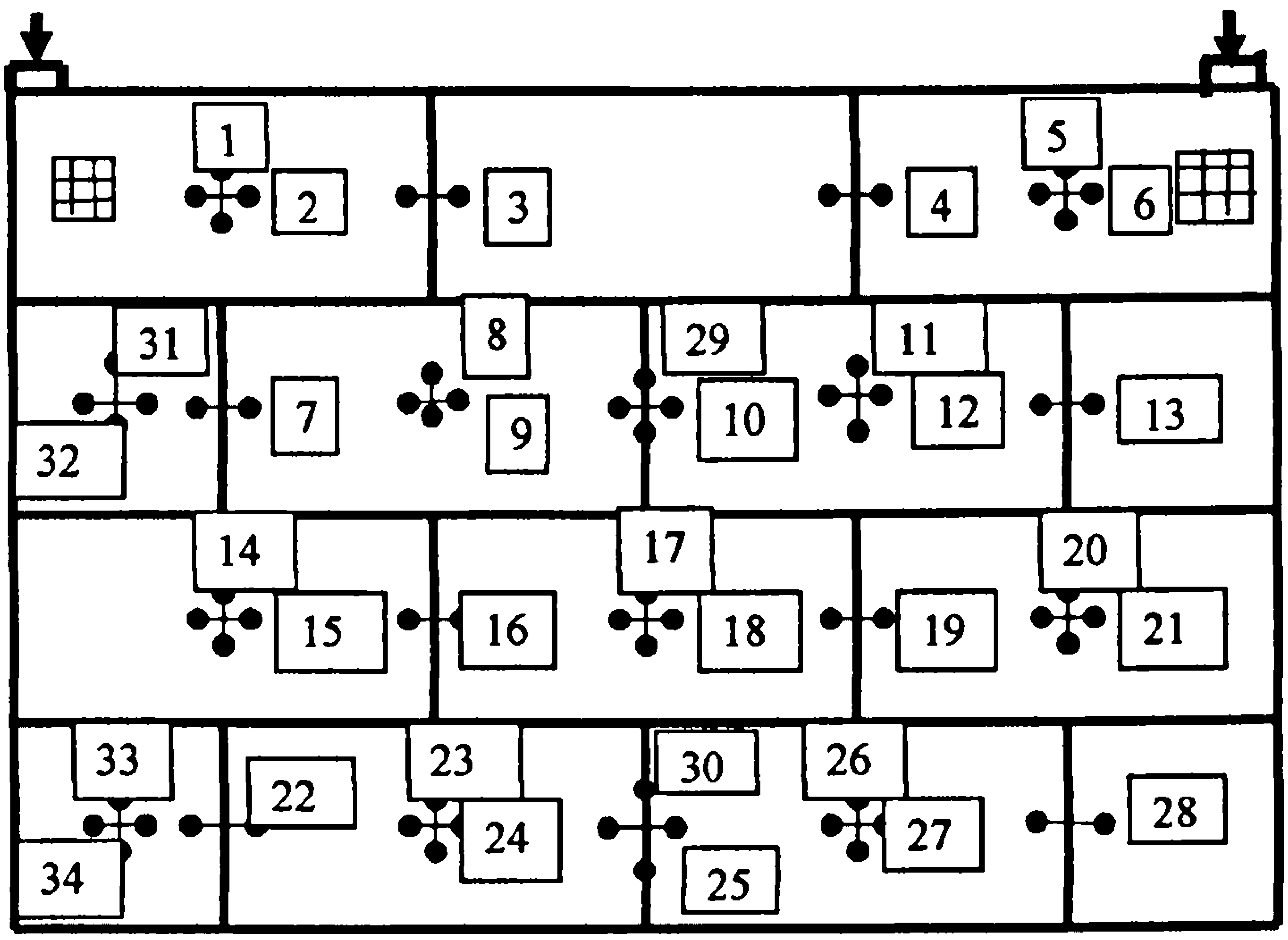


Figure 18: Wall Type 2 Test 2(c) – End load through 100 mm x150 mm bearing plates

Both end loads were applied simultaneously to prevent tilting and the loading was taken to ultimate. Care was taken to see that the loads were balanced at each end.

Strains were measured at 3.0, 6.0 and 9.0 kN each end then while keeping the loads balanced, the loading was taken up to failure. The ultimate loads under end loading were 10.64 kN (left edge), giving a bearing stress = 10640/(150x100) = 0.71N/mm^2 and 12.13 kN (right edge) giving a bearing stress =12130/(15000) = 0.81N/mm^2 i.e. 35% and 40% respectively of the nominal block strength.

The strains, based on measurements under the maximum applied loads (9kN) in the elastic range, in the cluster of 50mm gauges under the left hand load are given in Table 10 and illustrated diagrammatically in Figure 19.

As a check that the strains were balanced, some strain measurements were taken in the cluster under the right hand load and illustrated in Figure 20.

The distribution of vertical and horizontal strains across the whole wall is shown in Table 11.

It can be seen from the Figures that there were good spreads of the vertical compressive strains from the concentrated loads. Initially in the 50mm demec clusters positioned under the loads (there was a 100mm off set from the centre of the load to the centre of the cluster) the compressive vertical strains spread effectively across the width of the cluster, reducing as the distance from the edge of the load increases.

In the wall as a whole can be seen to spread at the mid height of the bottom course to 535mm horizontally from the edge of the load in a height of 875mm. The tangent of the angle of the spread from the vertical = 535/875 = 0.6114 giving an angle of 31.5^0.

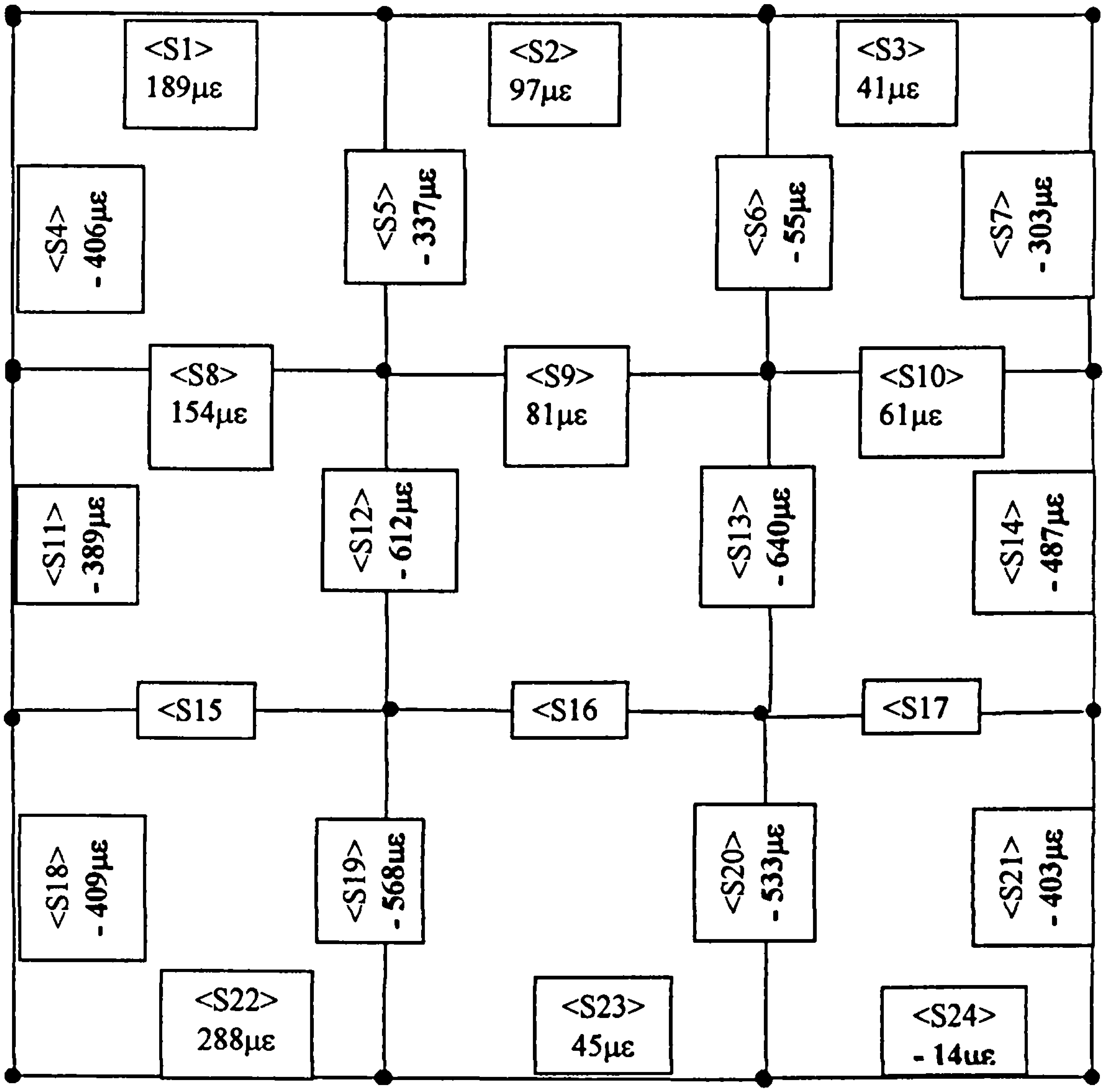


Figure 19: Wall Type 2 Test 2(c) Strains (με) in 50 mm Demec gauge cluster on the left hand end block in the top course under end loading

Table 10: Wall Type 2 Test 2(c) Strains (με) measured by 50 mm Demec gauge cluster on the left hand end block in the top course under end loading

Demec	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11	S12
Vstr				-406	-337	-55	8				-371	-432
Hstr	189	97	41					154	81	61		
Demec	S13	S14	S15	S16	S17	S18	S19	S20	S21	S22	S23	S24
Vstr	-101	-53				-355	-187	-130	-24			
Hstr										288	45	-14

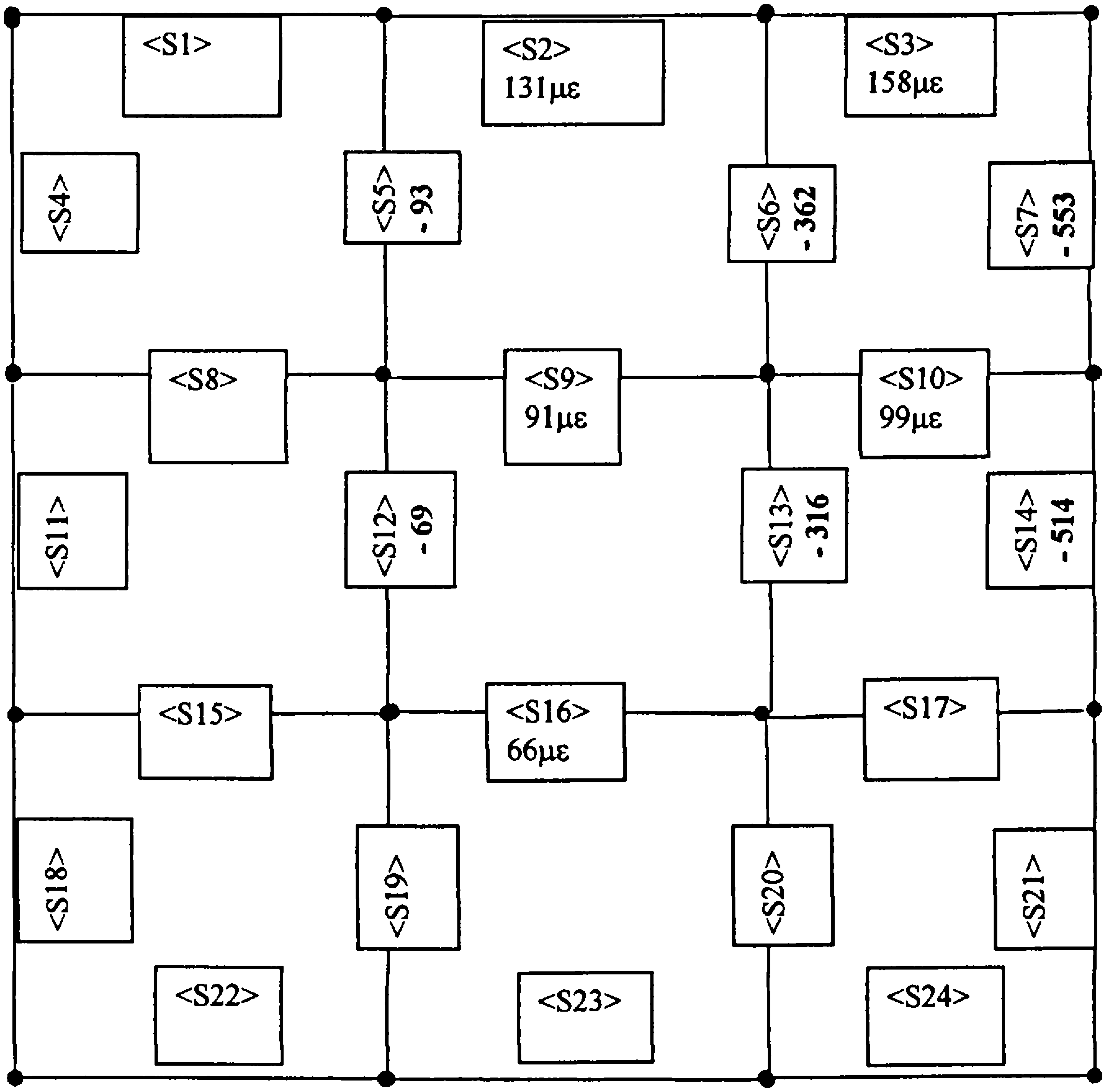


Figure 20: Wall Type 2 Test 2(c) Strain (με) distribution in Right Hand End Block

Wall Type 2 Test 2(c) 100 x150mm each end - Consider the whole wall

Table 11: Wall Type 2 Test 2(c) Distribution of vertical and horizontal strains under 100 mm x 150 mm - 9kN load each end

Vertical Location (fraction of height down)	Horizontal Location (Proportion of Wall Length from the left)						
	0.083	0.167	0.333	0.500	0.667	0.833	0.917
0.125	**-110**	42				**-23**	**-193**
0.375	**-141**		**-29**	10	**-36**		
0.625		**-60**		16		**-96**	
0.875	**-117**		0	23	0		
Vertical Strains ($\mu\varepsilon$) under central concentrated load							
Vertical compressive strains shown bold							
0.125	80	85	99		67	31	78
0.375	29	-98		-91	13	-21	
0.625		-5		-28	-28	-5	
0.875	15	41	-2	-54	-80		
Horizontal Strains ($\mu\varepsilon$) under central concentrated load							

4.2.3 Test Series 3 - Concentrated Loading Tests on Aircrete Wallettes

The wallettes were built using general purpose mortar and the effect of two different course heights was examined.

Three wallettes 3(a), 3(b) and 3(c), each 3 courses high and one and a half blocks long were built using 100mm thick Aircrete blocks with a measured compressive strength of 4.1N/mm^2 and a density of 440 kg/m^3. The mortar joints were general purpose 1:1:6 cement:lime:sand 10mm thick (Figure 21).

Figure 21: Tests 3 - 3 courses high 1.5 blocks long wallette

Table 12: Concentrated Loading Tests on Aircrete Wallettes built in general purpose mortar and two different course heights

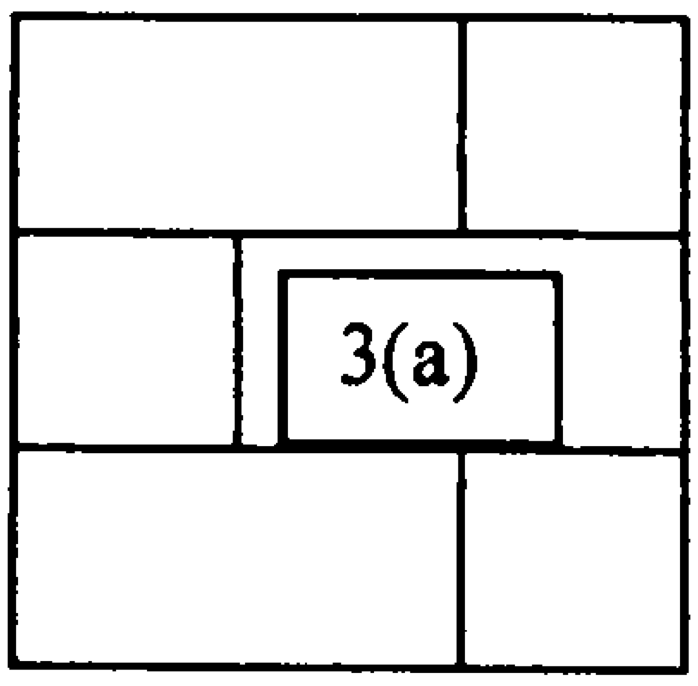
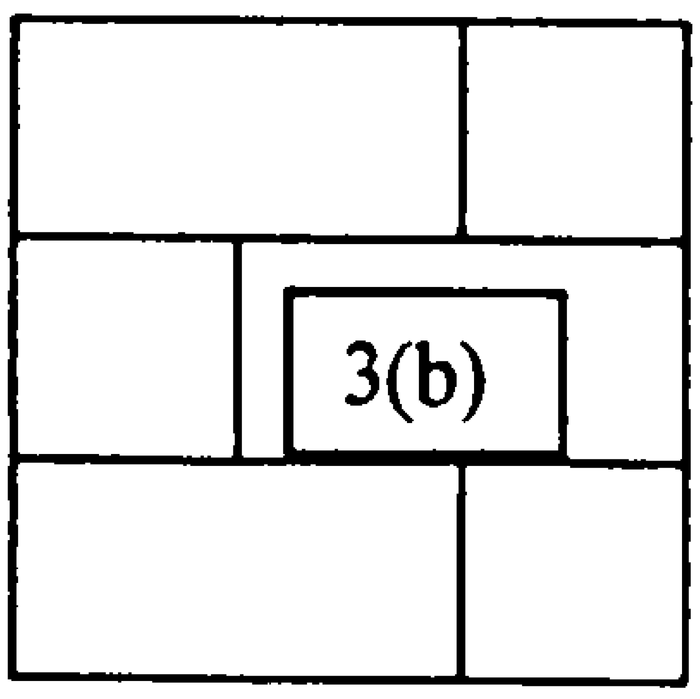
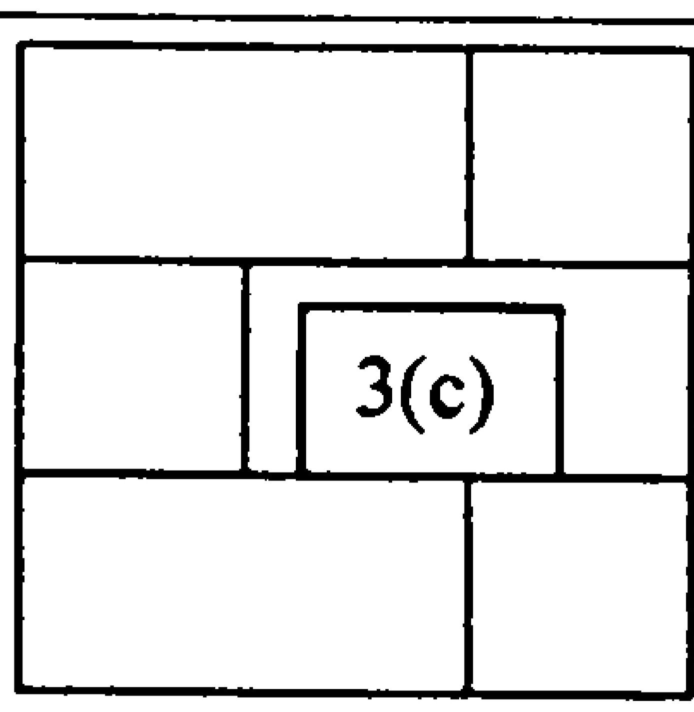
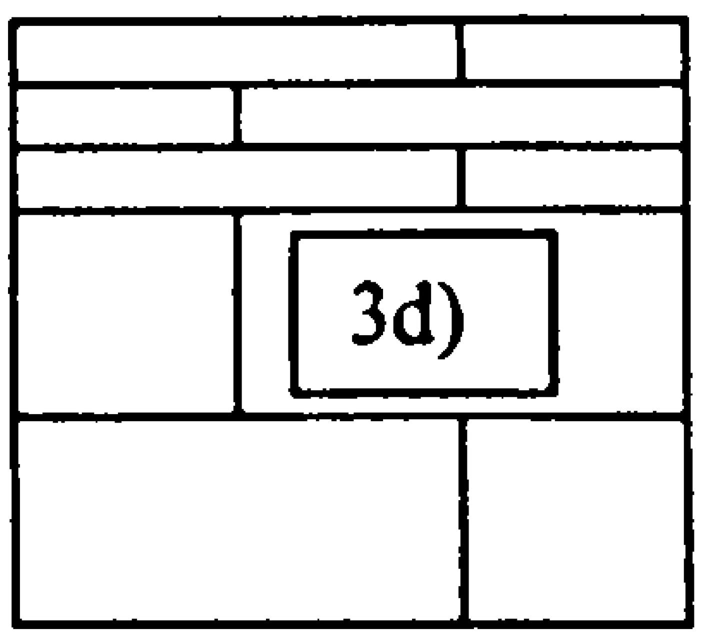
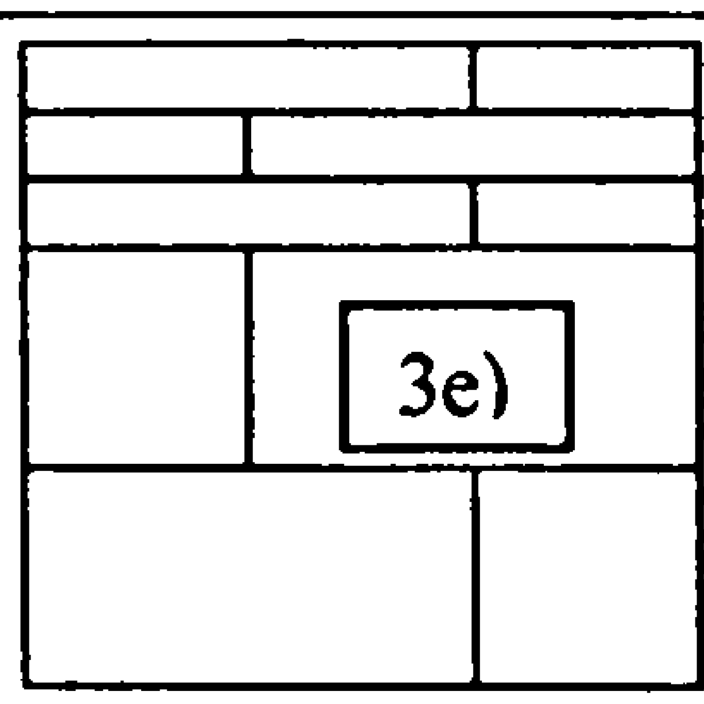
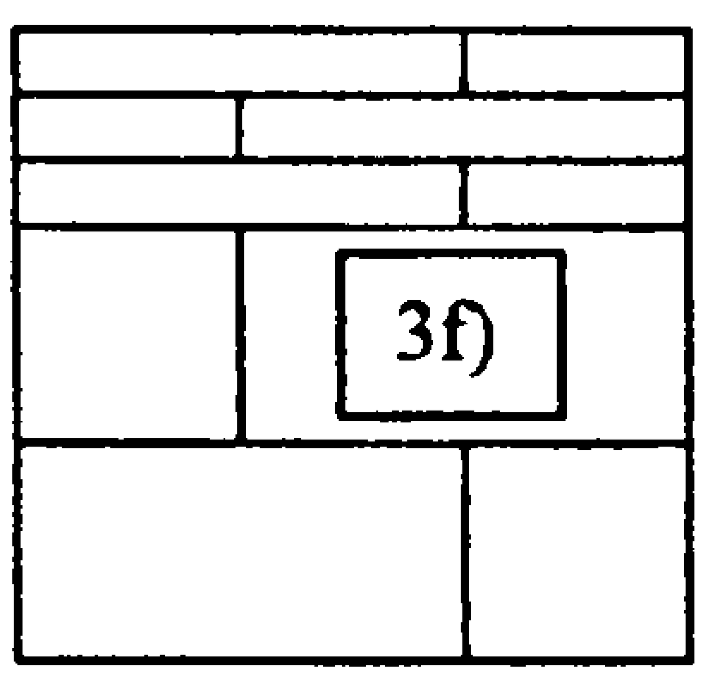
Specimen	Load at First Crack, kN	Ultimate Load, kN
	36.5	57.0
	10.0 / 17.0	58.7
	10.9	55.5

Table 13: Concentrated Loading Tests on Aircrete Wallettes built in general purpose mortar and two different course heights

Specimen	Load at First Crack, kN	Ultimate Load, kN
	26.2	47.9
	26.6	52.8
	23.0	51.0

A 100mm x 100mm load was applied at 1/3rd point concentrically on the whole block in the top course. The ultimate stress from the concentrated load ranged from 1.43 down to 1.35 times the block compressive strength (Table 12).

Figure 22: Test Series 3 Wallette failure mode Test 3(a)

The mode of failure was a sensibly vertical split which opened out in a Vee under the concentrated load. Attempts to establish conclusively whether failure is propagated first by splitting along the line if one of the interfaces between the vertical mortar joint in the middle course or first by a wedge forming under the load have not been successful. To the human eye they are virtually instantaneous. An investigation using high speed photography or other techniques would be a suitable subject for future research.

The mode of failure illustrated in Figure 22 is typical but not consistently reproducible. The wedge as seen in the photograph does not always form. But observations are that failure cracks frequently align with the one of the mortar/block interfaces in the perpend joint one course down from the top when general purpose mortar is used. The failure loads were reasonably consistent but the loads at which a first crack appears were scattered.

Figure 23: Tests 3 Wallette with1/3rd height Blocks in top three courses

Three further wallettes 3(d), 3(e) and 3(f) were built (Figure 23) and tested where the top course was replaced with three courses of one third height blocks. The wallettes were loaded similarly to the previous ones and the average ultimate stress from the concentrated load was 1.26 times the compressive strength of the blocks (Table 13). The mode of failure (Figure 24) was also similar to that obtained with full bed height blocks.

This seems to indicate that the ultimate load is not very sensitive to even a major rearrangement of the mortar joints provided they are of a similar type.

Figure 24: Test Series 3 Wallette mode of failure Test (3f)

4.2.3.1 Summary of Results for Series 1, 2 and 3

A summary of the results from Test Series 1, 2 and 3 is given in Table 14.

Table 14: Summary of Tests 1(a) to 1(c), 2(a) to 2(c) and 3(a) to 3(f) loading enhancements and spreads

Test	Ultimate stress under bearing N/mm^2	Enhancement over block mean compressive strength	Spread angle to vertical	Centre (C) or End (E) and Concentric (con) or Eccentric (ecc)	Bearing plate dimensions (mm)
1(a)	Not tested	Not tested	47.5^0++	Ccon	100 x100
1(b)	Not tested	Not tested	38.5^0	Ccon	100 x150
1(c)	Not tested	Not tested	Not valid (wall tipped)	Econ	100 x150
2(a)	Not tested	-----------------	-----------	-------------	-------------
2b)	2.77, 2.93	1.39, 1.47($2.0N/mm^2$)	42^0+	Ccon	100 x150
2(c)	0.71,0.81	0.35, 0.4($2.0N/mm^2$)	$>31^0 < 42^0$	Econ	100 x150
3(a)	5.7	1.39($4.1N/mm^2$)	not measured	1/3rd point	100 x100
3(b)	5.9	1.43($4.1N/mm^2$)	not measured	1/3rd point	100 x100
3(c)	5.55	1.35($4.1N/mm^2$)	not measured	1/3rd point	100 x100
3(d)	4.8	1.17($4.1N/mm^2$)	not measured	1/3rd point	100 x100
3(e)	5.3	1.29($4.1N/mm^2$)	not measured	1/3rd point	100 x100
3(f)	5.1	1.24($4.1N/mm^2$)	not measured	1/3rd point	100 x100

++ *indicates the spread from central loads was greater than the full width of the wall*

4.2.4 Test Series 4 Properties of Aircrete units and thin layer mortar

4.2.4.1 Compressive and flexural strength testing of thin layer mortar

The compressive strength results from a series of 42 No. thin layer mortar prisms (up to 28-days curing) tested in accordance with CEN Test Method BS EN 1015-11:1999 - Methods of Test for Mortar for Masonry. Determination of Flexural and Compressive Strength of Hardened Mortar(73) are given in Table 15. At each test age, the reported values were the average of three prisms. The test specimens were prisms 160 mm x 3 40 mm x 3 40 mm. For the compressive strength test, the prisms were broken into halves to provide six half prisms. Figure 25 shows the compressive strength development of the thin layer mortars.

Table 15: Test Series 4 - Thin Layer Mortar Compressive Strengths

Curing Age (Days)	Prism Compressive Strength (N/mm^2)	
	A	B
1	7.5	2.9
3	11.9	5.8
7	14.9	8.6
10	16.0	10.0
14	17.0	11.5
21	17.4	11.8
28	17.6	12.0

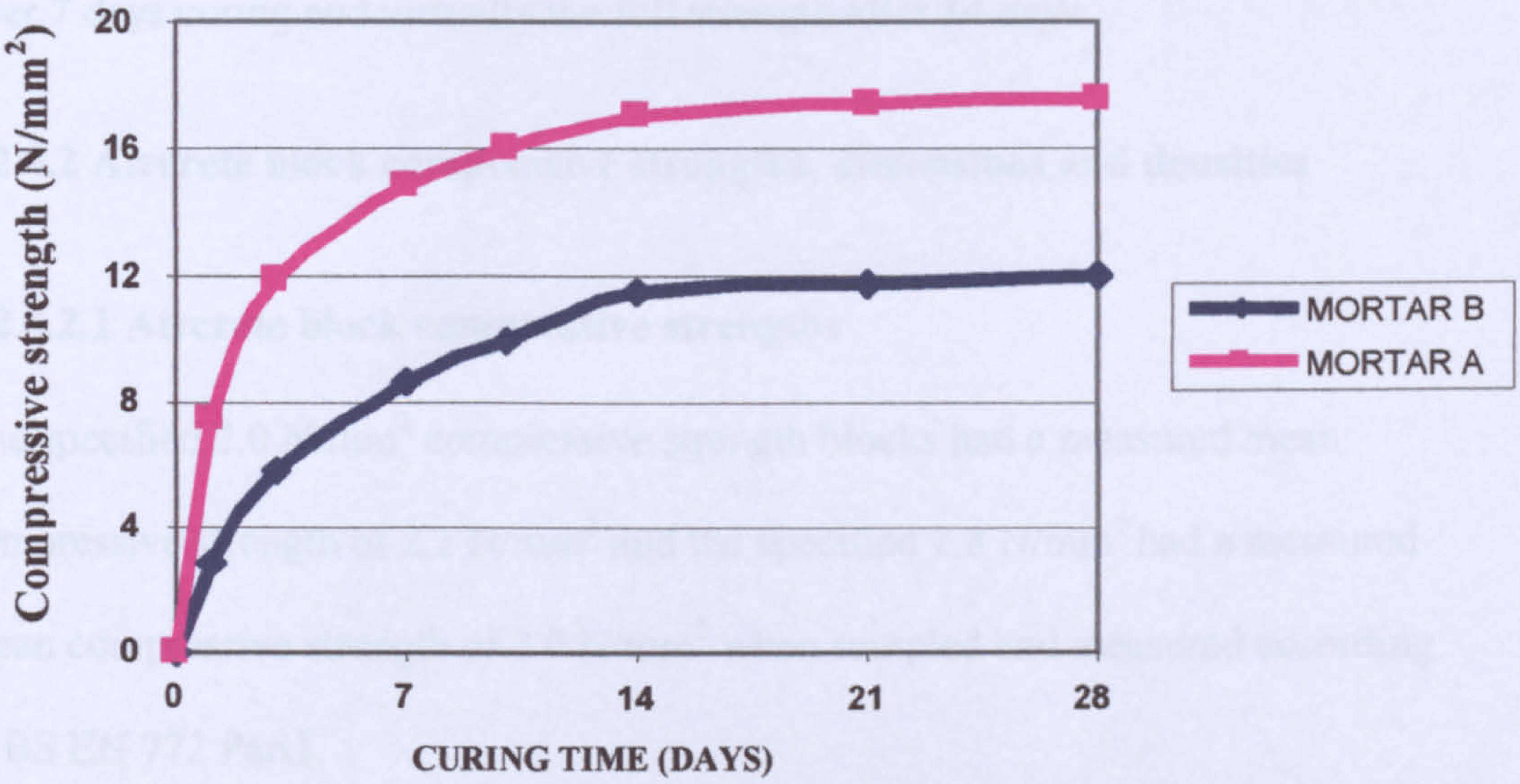


Figure 25: Tests 4 Compressive Strength (N/mm^2) Development for the two Thin Layer Mortars

The 28-day compressive and flexural strengths of the two thin layer mortars are given in Table 16.

Table 16: Tests 4 - 28-day Compressive and flexural Strengths of Thin Layer Mortars

Thin Layer Mortar	Compressive Strength (N/mm^2)	Flexural Strength (N/mm^2)
A	17.6	4.6
B	1 2.0	3.6

Both thin layer mortars are very strong in relation to Aircrete block strengths. Thin Layer Mortar A had a compressive strength nearly 50% greater than Mortar B (Table 16). The thin layer mortars gave more consistent flexural strength results despite the variation in compressive strengths. General purpose mortars show strength development of approximately 60% after 7 days, in comparison, for the thin layer mortars tested, nearly 75% of the final strength was reached after 7 days curing and virtually the full strength after 14 days.

4.2.4.2 Aircrete block compressive strengths, dimensions and densities

4.2.4.2.1 Aircrete block compressive strengths

The specified 2.0 N/mm^2 compressive strength blocks had a measured mean compressive strength of 2.1 N/mm^2 and the specified 2.8 N/mm^2 had a measured mean compressive strength of 3.0 N/mm^2 when sampled and measured according to BS EN 772 Part1.

4.2.4.2.2 Aircrete block dimensions

The dimensions and densities were measured in accordance with BS EN 772 Part 16, 2000 – Determination of dimensions (to the tolerances shown in Table 17) (Table18) and BS EN 772-13 BS EN 772, Part 13, 2000 – Determination of net and gross dry density of masonry units (Table 19), respectively and summarised in Table 20.

Table 17: Measurement precision when measuring dimensions to EN 772

Tolerance on the dimension being measured as specified in EN 772 (mm)	Measuring error (maximum) (mm)
≤ 1	0.2
> 1	0.5

Six specimens were tested for each of the two specified block compressive strengths. During testing, any superfluous material adhering to the unit as a result of the manufacturing process was removed before measuring.

Table 18: Measured dimensions of 2.0 and 2.8 N /mm^2 Aircrete Blocks

Aircrete Block Compressive Strength (N/mm^2)	Length (mm)	Thickness (mm)	Height (mm)
2.0	620.0	149.8	249.9
2.0	619.5	149.6	249.6
2.0	620.0	150.0	250.0
2.0	620.0	149.8	249.7
2.0	619.8	150.0	250.0
2.0	619.6	149.4	249.4
2.8	440.0	149.6	214.5
2.8	439.6	150.0	215.0
2.8	439.6	149.7	214.5
2.8	440.0	149.8	214.8
2.8	439.8	150.0	215.0
2.8	439.5	149.5	214.8

4.2.4.2.3 Aircrete block densities

After measuring the dimensions, the density of the same specimens used for measuring the dimensions was determined. A minimum number of six specimens were tested.

The test specimens were dried to constant mass m_{dry}, in a ventilated oven at a temperature of 105 C ± 5 °C. Constant mass is reached when during the drying process in two subsequent weighings with a 24 hour interval, the loss in mass between the two determinations is not more than 0.2 % of the total mass. The mass m_{dry} was recorded.

The volume (V) of the material was then calculated using the formula:

Volume = length x width x height, is expressed to the nearest 10^4 mm^3.

The dry density(σ) was calculated as follows:

$$\sigma V = m_{dry} \times 10^6 \ kg/m^3, \text{ where}$$

m_{dry} = dry mass of material (g)

V = volume of material (mm^3)

σ = dry density (kg/m^3)

Table 19: Measured dry density of 2.0 and 2.8 N /mm^2 Aircrete Blocks.

Aircrete Block Compressive Strength (N/mm^2)	Density (kg/m^3)
2.0	350
2.0	352
2.0	354
2.0	351
2.0	352
2.0	353
2.8	474
2.8	475
2.8	477
2.8	479
2.8	480
2.8	477

Table 20: Dimensions and Densities of 2.0 and 2.8 N /mm^2 Aircrete blocks complying with BS EN 771 - 4

Aircrete Block Compressive Strength (N/mm^2)	Average Block Length (mm)	Average Block Thickness (mm)	Average Block Height (mm)	Average Block Density (kg / m^3)
2.0	619.8	149.8	249.8	352
2.8	439.7	149.8	214.8	477

The manufacturer's specified dimensions in accordance with BS EN 771 - 4 are:

2.0 N/mm^2 Aircrete blocks: 620 x 150 x 250mm and

2.8 N/mm^2 Aircrete blocks: 440 x 150 x 215mm

The average measured dimensions for both materials were within 0.3mm of the specified dimensions. The manufacturer's stated densities were 350 and 475 kg/m^3 for 2.0 and 2.8 N /mm^2 Aircrete blocks, respectively; The average measured densities were 352 and 477 kg/m^3, which is within 0.57% and 0.42% of the stated value for the 2.0 and 2.8 N /mm^2 Aircrete blocks, respectively.

4.2.5 Test Series 5 - Concentrated Loading on Wallettes from Lintel Bearings

Proprietary steel box lintels with a width of approximately 150 mm, a nominal height of 219 mm and a standard length of 2700mm were used to apply loads to the lintel bearings supported on wallettes 1.5 blocks wide by 3 courses high constructed from Low Density Aircrete blockwork (2.0 and 2.8 N/mm^2 blocks). The lintels were described by the manufacturer as having a safe working load of 54 kN. The weight of the lintels was 18.0 kg/m

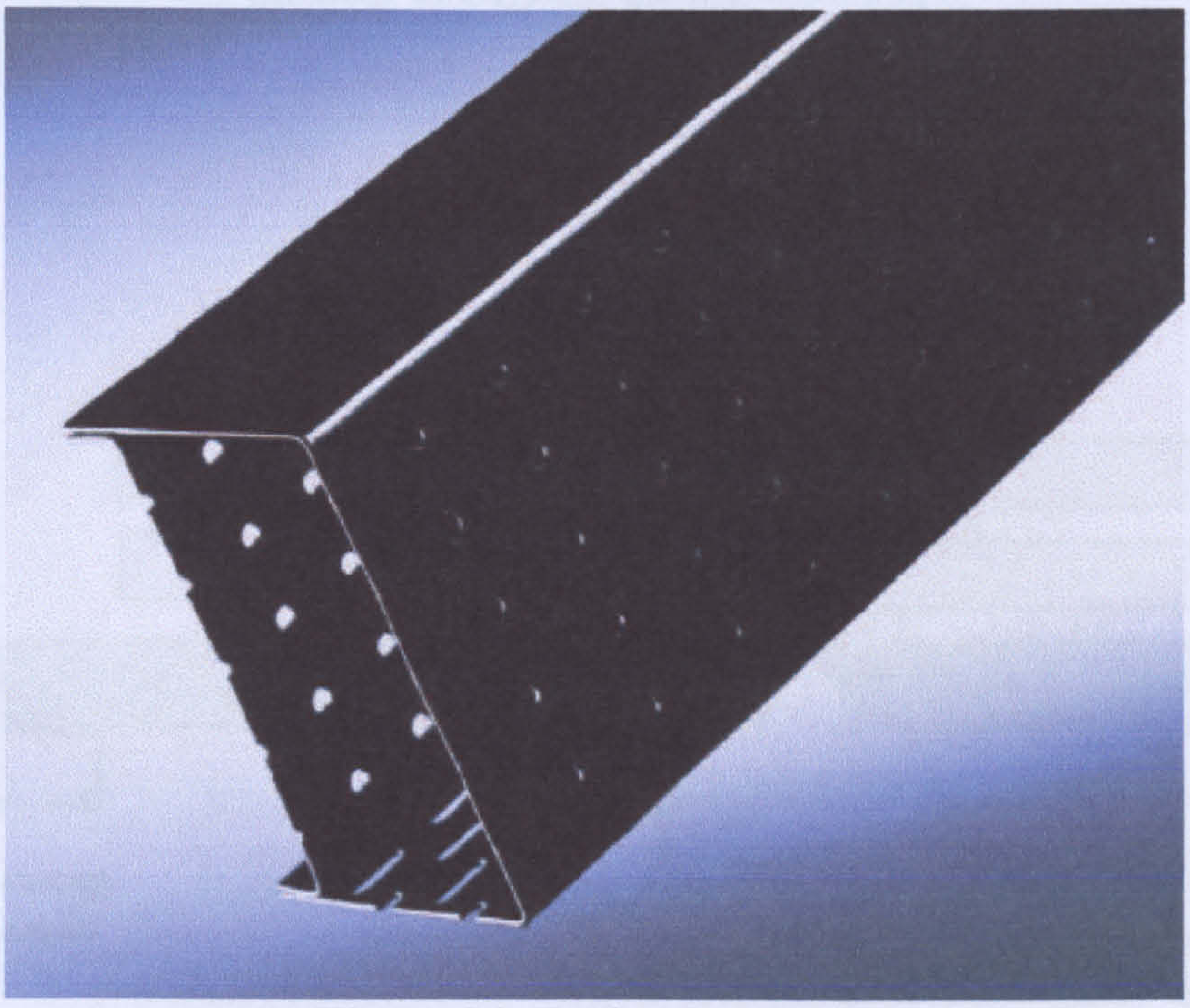


Figure 26: Typical box lintel

There is no European or British Standard test specifically for concentrated loads or lintel bearings. The tests undertaken were generally in accordance with the procedure described in European Standard BS EN 846; Part 9 BS EN 846, part 9, 2000 – Determination of flexural resistance and shear resistance of lintels, British Standards Institution, 2000 (84) although they are intended for testing the strength of lintels.

Hence the lintels were subjected to 4 point loading as shown in Figure 27 and were tested with bearing lengths of 150 and 300mm. The minimum bearing length recommended by the lintel manufacturer is 150 mm. For each test, the central load was applied in increments of 5 kN, but due to stability problems in the lintels, the maximum load had to be limited to 75 kN. The mid – span vertical deflection of the lintel was recorded. All visible signs of distress in specimen, fixings or supporting member throughout the test were noted.

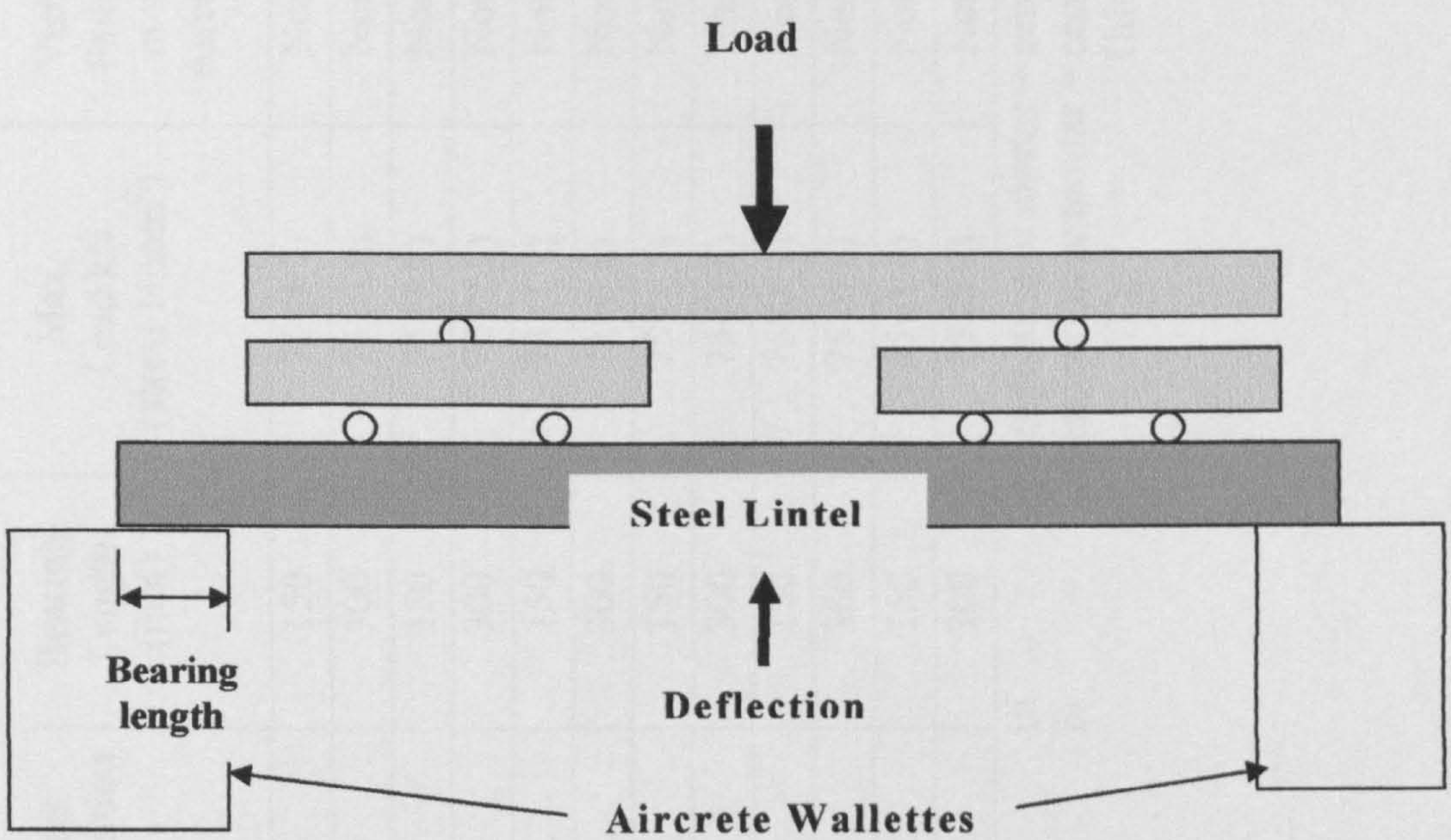


Figure 27: Lintel Bearings Experimental Test Arrangement

Table 21: Lintel Bearings Results for Different Aircrete Block and Mortar Combination

Aircrete Block Strength (N/mm^2)	Mortar Designation	Bearing Length (mm)	Max. Load kN (stress N/mm^2)	Visible Failure in the Aircrete	Mid span deflection at 75kN (mm)	Deflection after load release (mm)
2.0	iii	150	75 (1.7)	None	8.7	0
2.0	iii	300	75 (0.8)	None	5.0	0
2.0	iv	150	75 (1.7)	None	9.0	0
2.0	iv	300	75 (0.8)	None	5.2	0
2.0	TL	150	75 (1.7)	None	7.8	0
2.0	TL	300	75 (0.8)	None	5.6	0
2.8	iii	150	75 (1.7)	None	8.5	0
2.8	iii	300	75 (0.8)	None	6.9	0
2.8	iv	150	75 (1.7)	None	8.3	0
2.8	iv	300	75 (0.8)	None	5.4	0
2.8	TL	150	75 (1.7)	None	6.5	0
2.8	TL	300	75 (0.8)	None	6.0	0

iii = Designation iii mortar = cement:lime:sand ratio of 1:1:6 (volume)
iv = Designation iv mortar = cement:lime:sand ratio of 1:2:8 (volume)
TL = Thin Layer mortar

It can be seen from Table 21 that all wallettes withstood a load of at least 75 kN or a local compressive stress of 1.7 N/mm^2. At this load, which was 85% of the compressive strength of the Aircrete blocks, there was no visible failure and on removal of the load, the lintel deflection reading went back to zero, hence, only elastic deflection took place in the lintel. The deflections of the lintels were measured. When the bearing length was 150mm the lintel elastic deflections were greater than those when the bearing length was 300mm. This is interpreted as indicating that the measured lintel deflection was actually the sum of the lintel deflection and the elastic shortening in the Aircrete support. As the bearing stress under the lintel bearing would be inversely proportional to the bearing area, the compressive strain under the 300mm long bearing would be less. The phenomenon is further interpreted as indicating that probably the full length of the 300mm bearing was effective, The deflection with 150mm bearing length was consistently less for the bearings using thin layer mortar. The deflection with 300mm bearing length was similar for the bearings using thin layer mortar and general purpose mortar.

As the lintel deflection was greater for the 150mm long bearings, there was probably a slightly greater local crushing of the Aircrete at the inside edge of the bearing. This was too small to be measured and was judged to be of no consequence as the deflections fully recovered on removal of the load.

Chapter 5. Test Series 6 and Series 7

5.1 Test Series 6 and Series 7 Programme

The programme of tests for Test Series 6 and Test Series 7 is given in Table 22.

Table 22: Programme Tests Series 6 and 7

Test Series 6 - Concentrated load tests to failure on nine ½ storey height walls Type 6
6(a) Central load through 100mm x 100mm bearing plate 25mm eccentric to wall longitudinal centre line taken to ultimate
6(b) Central load through 100mm x 100mm bearing plate 25mm eccentric to wall longitudinal centre line taken to ultimate
6(c) Central load through 100mm x 100mm bearing plate concentric to wall longitudinal centre line taken to ultimate
6(d) Central load through 100mm x 150mm bearing plate concentric to wall longitudinal centre line taken to ultimate
6(e) Central load through 100mm x 150mm bearing plate concentric to wall longitudinal centre line taken to ultimate
6(f) End loads through 100mm x 100mm bearing plates both ends simultaneously concentric to wall longitudinal centre line taken to ultimate
6(g) End loads through 100mm x 100mm bearing plates both ends simultaneously concentric to wall longitudinal centre line taken to ultimate
6(h) End loads through 100mm x 100mm bearing plates both ends simultaneously 25mm eccentric to wall longitudinal centre line taken to ultimate
6(i) End loads through 100mm x 100mm bearing plates both ends simultaneously 25mm eccentric to wall longitudinal centre line taken to ultimate
Tests 7 Concentrated loading from joist hangers on Aircrete wallette Type 7
Loads applied to Aircrete wallettes through joist hangers

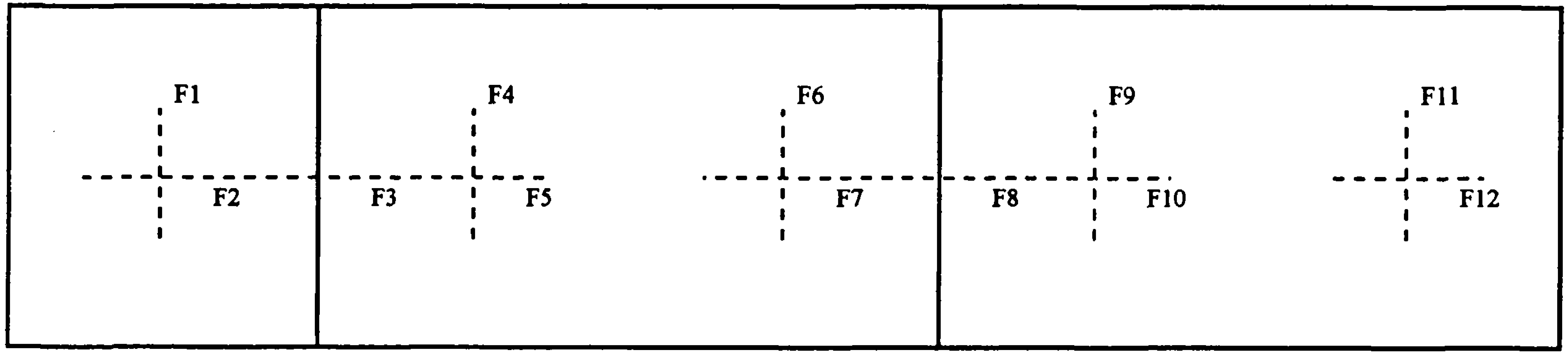
5.1.1 Test Series 6 – Concentrated loading on nine half storey height walls

The nine tests (Table 23) were designed to measure strains in the blocks in the bottom courses and ultimate loads.

Table 23: Test Series 6 - Nine Half Storey Height Walls tested to Failure

Wall number and Loading	**Wall details**
Tests 6(a) and 6(b) Central load through 100mm x 100mm bearing plate 25mm eccentric to wall longitudinal centre line	4 courses high, two and a half blocks long (1025 x 1565 mm) Block compressive strength 2.0 N/mm^2
Test 6(c) Central load through 100mm x 100mm bearing plate concentric to wall longitudinal centre line	5 courses high, two and a half blocks long (1270 x 1570 mm) Block compressive strength 2.0 N/mm^2
Tests 6(d) and 6(e) Central load through 100mm x 150mm bearing plate concentric to wall longitudinal centre line	5 courses high, three and a half blocks long (1010 x 1545 mm) Block compressive strength 2.8N/mm^2
Tests 6(f) and 6(g) End loads through 100mm x 100mm bearing plates both ends simultaneously concentric to wall longitudinal centre line	5 courses high, three and a half blocks long (1095 x 1550 mm) Block compressive strength 2.8 N/mm^2
Tests 6(h) and 6(i) End loads through 100mm x 100mm bearing plates both ends simultaneously 25mm eccentric to wall longitudinal centre line	5 courses high, two and a half blocks long, (1275 x 1565 mm) Block compressive strength 2.0N/mm^2

Demec locations (A) at front of wall (Wall front is defined as the side with the half block to the left on the bottom course)



Demec locations (B) at back of wall (Wall back is defined as the side with the half block to the right on the bottom course)

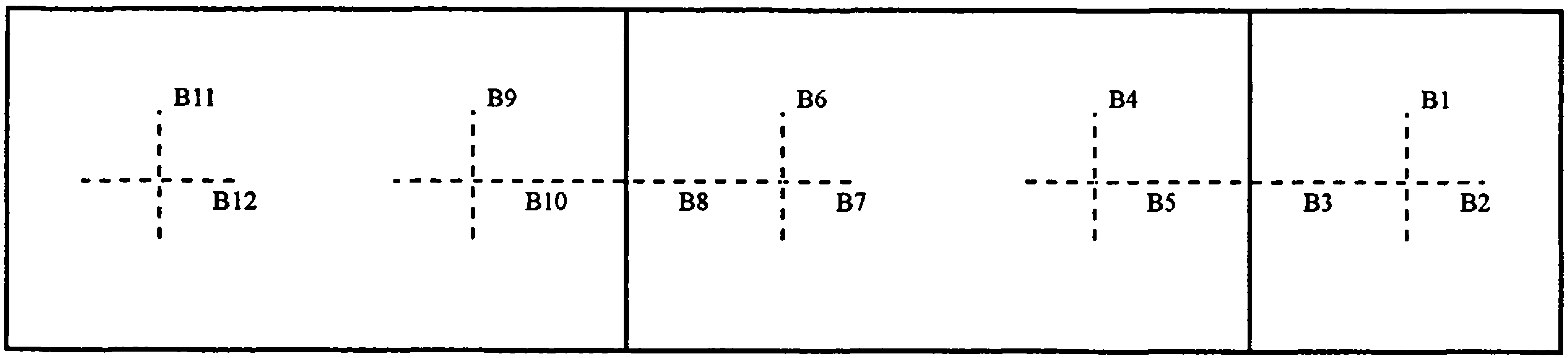


Figure 28: Demec gauge identification numbers and locations in the bottom course of 2.0 N/mm^2 walls Horizontal gauges across mortar joints indicated in red

5.1.1.1 Test Series 6 Procedure

Each wall had twenty four Demec gauges fixed to the bottom course of blocks. On each face there were twelve with seven horizontal and five vertical. The Demec gauge identification in Figure 28 is preceded by a "B" (wall back) or "F" (wall front) and a broken line indicates which gauges are vertical and which are horizontal. Corresponding Demecs on the front and the back of the wall are given the same number preceded by B or F as appropriate for the opposite faces. The vertical location of all Demec gauges is at mid block height. The wall front face is defined as that where the half block in the bottom course was on the left. The zero reference point for all walls was taken from the front face left hand edge.

Where the loading was eccentric to the centre line of the thickness of the wall, the location of the load was always closer to the front face of the wall.

Graphs were drawn for the experimental strains over the whole range of applied stresses on a location by location basis. These strains have been fitted over a stress range which can be regarded as elastic from zero load to less than but close to failure. Failure in this case is defined as either the onset of visible cracking or when the crushing strength of the bearing has been reached.

Graphs were also produced of the fitted strains as a function of stress on a location by location basis. The data was calculated using the functional form and coefficients obtained from the fitting functions.

A table was produced of fitted strains as a function of position at a set arbitrary stress just below the observed failure stress whether that was first crack or crushing under the bearing.

Vertical strain was plotted as a function of position based on identifying both wall front and wall back horizontal strains. Those horizontal strains measured across perpend joints were examined to see if they showed strains which were out of step from those within the blocks.

The walls which were subjected to end loading were loaded at both ends simultaneously to eliminate the rotation which has been observed previously when loading at one end only. The stress in the test walls was taken as the sum of both loads divided by the total loaded area (i.e. the plan area of both loading plates). Where the bearing plate did not bear evenly on the block a thin layer of building sand was used to even out any irregularities.

5.1.1.1a Test 6(a) Central 100mm x 100mm load 25mm eccentric to the wall longitudinal centreline

Test wall 6(a) was 4 courses high, two and a half blocks long (1025mm x 1565mm actual) built with 2.0 N/mm^2 compressive strength blocks (Figure 29).

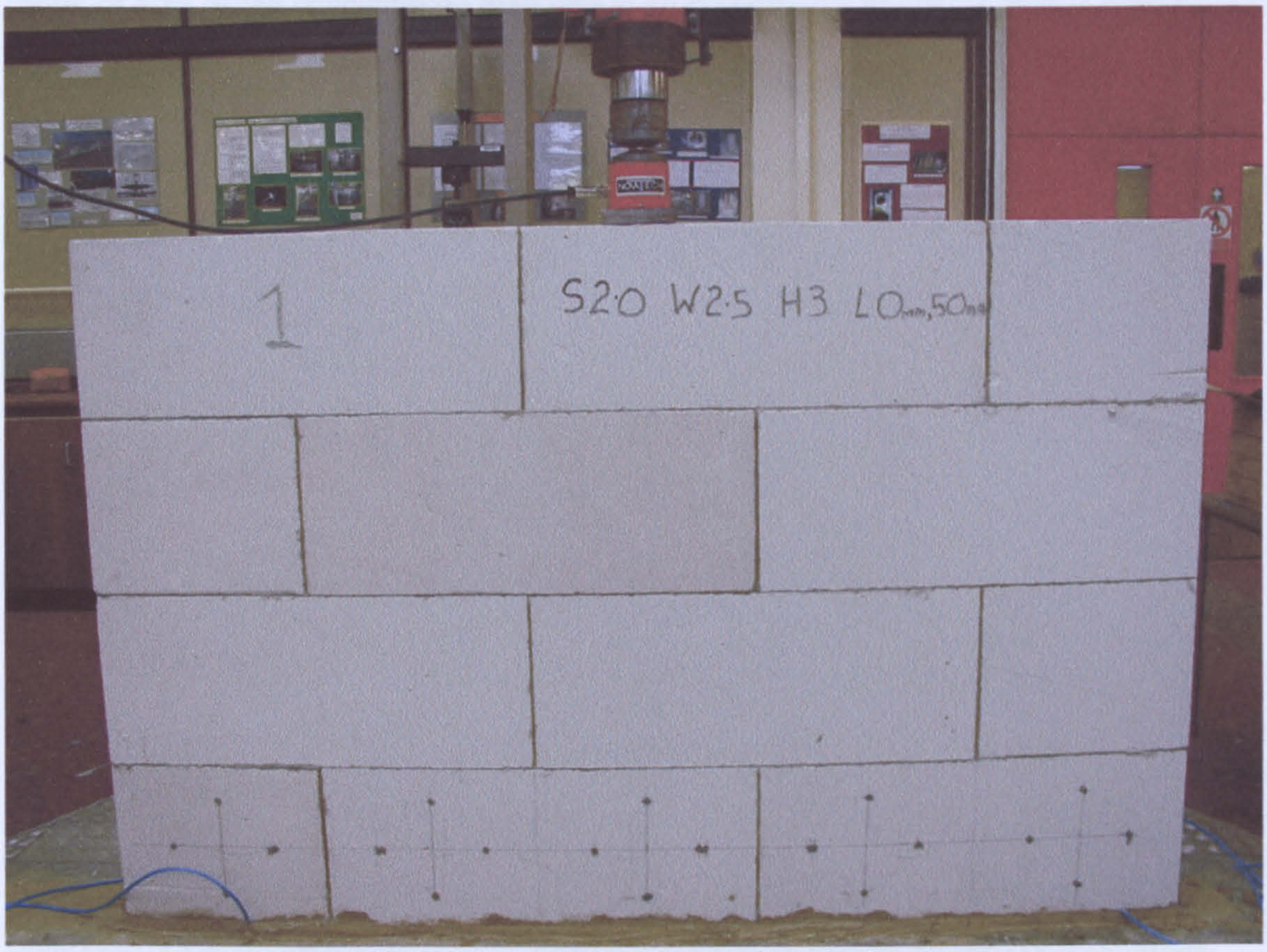


Figure 29: Wall Test 6(a) Front view during loading

The maximum concentrated load of 33.0 kN gave a direct compressive stress of 3.30 N/mm^2 immediately beneath the load, which is 1.65 times greater than the 2.0 N/mm^2 nominal block mean compressive strength.

Plan area of the wall = 1565 x 150 = 234,750mm^2

If the load was concentric and spread over the full area of the base, the average stress when the load is 18.3kN would be 0.077 N/mm^2. From Table 24, the mean vertical strain was

-122.1 με at the back and -7.7 με at the front, an average of approximately -65 με and therefore Modulus of Elasticity can be calculated to be 0.077/65 x 1000000 ≈1185 N/mm^2. As the load was applied eccentrically, if the wall had a fixed base and was elastic, the load would apply a bending moment to the wall, putting the front face into compression. As the centre of area of the concentrated load was at an eccentricity of 1/6th, a simple elastic calculation would indicate that the compressive bending stress at the front face would be equal to double the direct compressive stress and at the back face would be equal to zero.

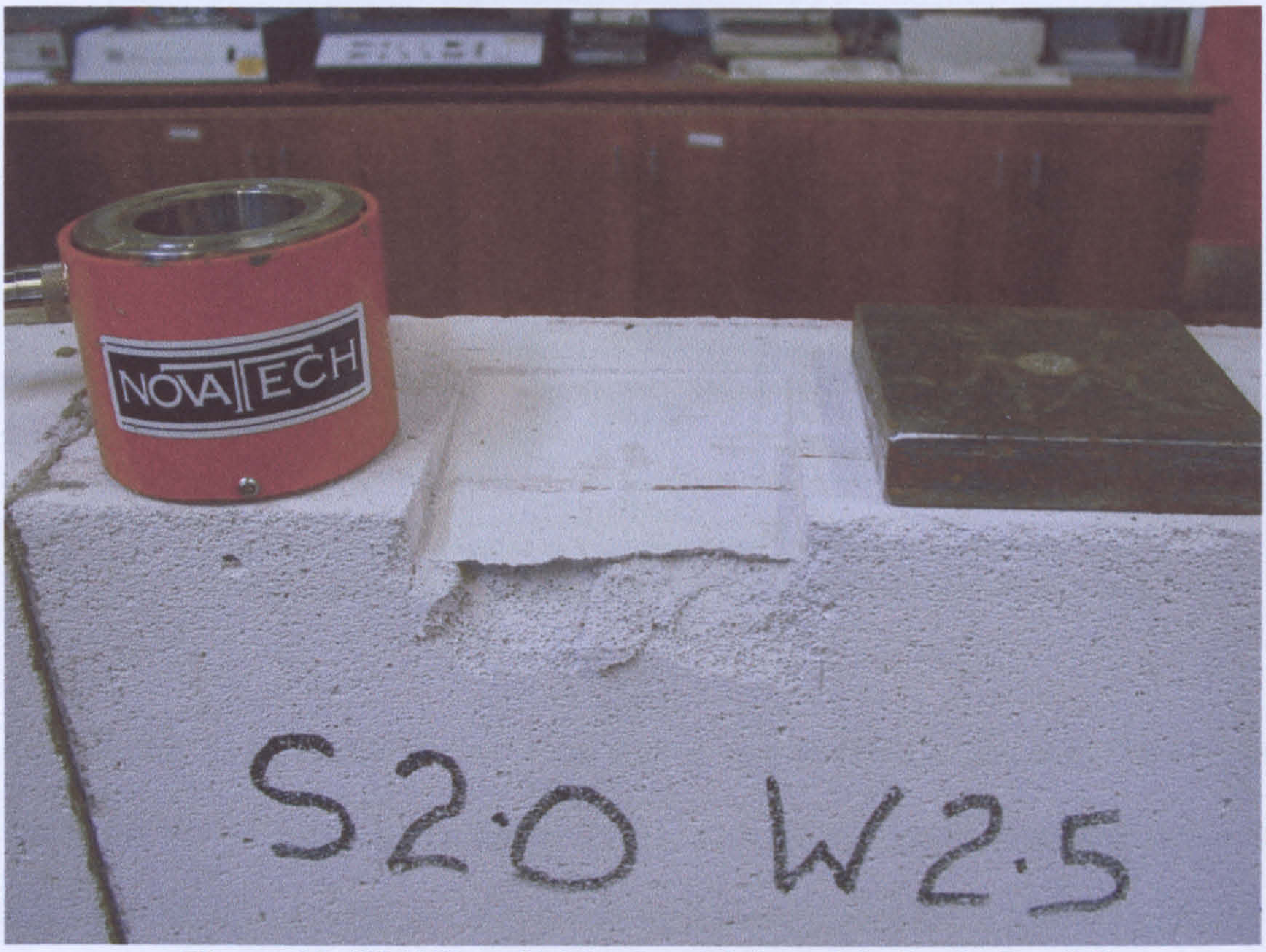


Figure 30: Wall Test 6(a) in the vicinity of the load after failure

This reasoning, however, only applies if the bearing plate is completely free to rotate in the plane perpendicular to the face of the wall. In the practical case, where this is not perfectly so, it might be expected that the front or back edge of the bearing plate will bite into the Aircrete when the wall moves out of its vertical plane as its ultimate load is approached.

The vertical strains in the bottom course were not what would be expected from a simple elastic analysis; instead of the front face (i.e. in the direction of eccentricity) being the face with maximum compressive strain, it was the back face which had the greatest compressive vertical strain as indicted in Table 24.

Table 24: Vertical and Horizontal Strains in bottom course Test 6(a)

Position (mm)		Front Vertical $\mu\varepsilon$ @ 1.83 N/mm^2	Front Horizontal $\mu\varepsilon$ @ 1.83 N/mm^2	Back Vertical $\mu\varepsilon$ @ 1.83 N/mm^2	Back Horizontal $\mu\varepsilon$ @ 1.83 N/mm^2
G1	155	6.0		-138.3	
G2	155		**-44.3**		11.4
G3J	310		**-41.2**		29.4
G4	470	-8.0		-139.9	
G5	470		**-22.4**		25.5
G6	780	-33.4		-133.9	
G7	780		8.1		20.0
G8J	930		**-9.3**		26.9
G9	1095	-12.4		-138.0	
G10	1095		**-13.9**		20.5
G11	1405	9.2		-60.4	
G12	1405		**-23.2**		18.6

The horizontal gauges across perpend joints are shown red and when the horizontal strains were compressive they are shown bold

Considering the horizontal strains, it can also be seen in Table 24 that at the back of the wall all of the horizontal strains are tensile and they are reasonably consistent across the full width of the wall with only a slight increase in magnitude at the perpend joints. In walls constructed in general purpose mortar, the horizontal tensile strains tend to be concentrated at the perpend joints at the interface between the mortar and the block due weak adhesion between the mortar and the block. The picture of horizontal strains is not so clear on the front face although the horizontal strains at the perpend joints are not greater than the general run all the measurements except one are in compression.

Figure 30 shows the damage under the load position after failure shows that the wall did not rotate in the direction of the eccentricity. It was in the opposite direction causing the vertical compressive strains on the back face of the base course to be the larger.

The Tan of the angle of load spread from the vertical =732.5/1025 = 0 .7146 giving 36^0.

But it can be seen from the strains on the back face that the spread would be much greater if the wall had been wider.

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5.1.1.1b Test 6(b) - Central 100mm x 100mm load 25mm eccentric to the wall longitudinal centreline

Test 6(b) wallette replicated Test 6(a) and was 4 courses high, two and a half blocks long (1020mm x 1565mm actual) built using 2.0 N/mm^2 compressive strength blocks.
The maximum load of 36.5 kN gave a compressive stress of 3.65 N/mm^2 which is 1.83 times greater than the 2.0 N/mm^2 nominal block mean compressive strength although the first sign of flaking of the surface of the block under the bearing was at 2.7 N/mm^2. As in this case no sand was used under the bearing it is possible that an irregularity caused the premature spalling.
If the load had been concentric and spread over the full area of the base, the average stress when the load was 18.3kN would again have been 0.077 N/mm^2. From Table 25, the mean vertical strain was roughly -56.4 $\mu\varepsilon$, and therefore $(0.077/56.4) \times 10^6 \approx 1365$ N/mm^2 approximate Modulus of Elasticity.
Again the vertical strains in the bottom course were not what would be expected from a simple elastic analysis i.e. instead of the front face being the face with maximum compressive strain because it is in the direction of eccentricity, it was the back face which had the greater compressive vertical strain as indicted in Table 25, although this time the difference was not so great. The vertical compressive strains spread over the full width of the wall were again comfortably greater than the spread assumptions in EC6.

In this test it was observed that the horizontal strains across the mortar joints were compressive on both front and back faces of the test wall while the horizontal strains in the blocks were generally tensile front and back.

Table 25: Wall Test 6(b) Vertical and Horizontal Strains

Position in from left hand end (mm)		Front Vertical με @ 1.83 N/mm^2	Front Horizontal με @ 1.83 N/mm^2	Back Vertical με @ 1.83 N/mm^2	Back Horizontal με @ 1.83 N/mm^2
G1	**155**	-36.0		-64.0	
G2	**155**		5.3		7.7
G3J	**310**		**-10.6**		**-13.1**
G4	**470**	-40.3		-86.8	
G5	**470**		6.7		13.5
G6	**780**	-46.4		-102.9	
G7	**780**		11.7		9.4
G8J	**930**		**-8.4**		**-5.5**
G9	**1095**	-49.7		-74.0	
G10	**1095**		-9.3		5.8
G11	**1405**	-22.9		-41.0	
G12	**1405**		-19.3		-12.8

The horizontal gauges across perpend joints are shown red and when the horizontal strains were compressive they are shown bold

5.1.1.1c Test 6(c) Central load through 100mm x 100mm bearing plate concentric to the wall longitudinal centre line

Test 6 (c) wallette was 5 courses high, two and a half blocks long (1270mm x 1570mm actual) built with 2.0 N/mm^2 compressive strength blocks.

The maximum load of 39.0 kN gave a compressive stress of 3.90 N/mm^2 which is 1.95 times greater than the 2.0 N/mm^2 nominal block mean compressive strength.

Both the way that the bearing plate dug into the wall more towards the back (Figure 31) and the different vertical strains front to back in the bottom course, indicate how difficult it is to achieve concentric concentrated loads in relation to the thickness of the wall. This difficulty is acknowledged in EC 6, as an accidental eccentricity is built into the wall strength calculation.

On the other hand as with the previous tests, the vertical strains confirm that the load spread is fairly even over the full width of the wall. But those on the back face were linear in the elastic range while those on the front face showed little change with increasing load. No clear picture emerges from the horizontal strains except to note that they are of opposite sign comparing the front face to the back.

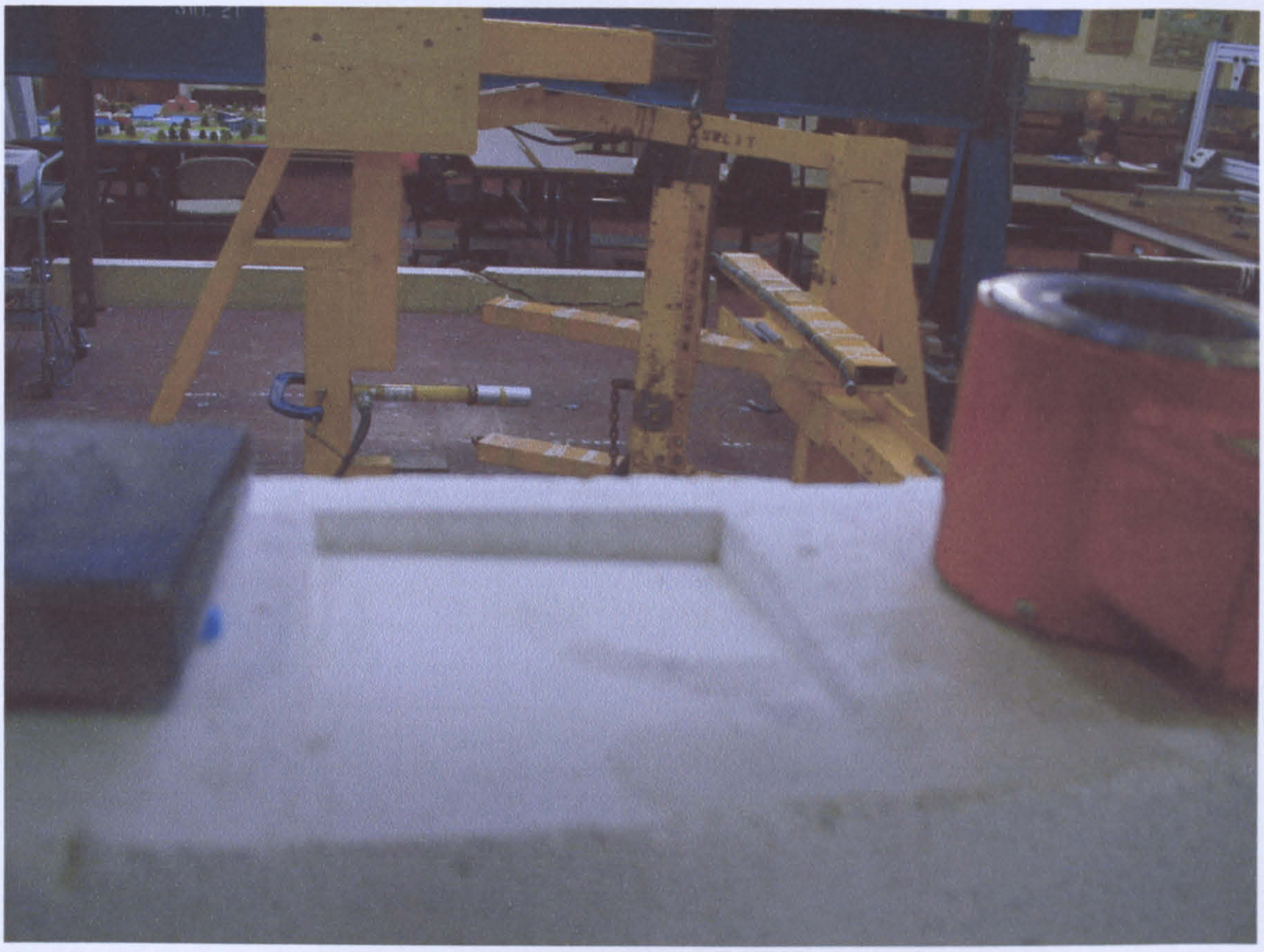


Figure 31: Test 6(c) in the vicinity of the load after failure

Table 26: Wall Test 6(c) Vertical and Horizontal Strains

	Position (mm)	Front Vertical με @ 1.83 N/mm²	Front Horizontal με @ 1.83 N/mm²	Back Vertical με @ 1.83 N/mm²	Back Horizontal με @ 1.83 N/mm²
G1	155	11.9		-128.9	
G2	155		-2.5		22.0
G3J	310		**-28.8**		**13.2**
G4	470	25.2		-140.5	
G5	470		-11.7		5.9
G6	780	-16.2		-160.3	
G7	780		-11.9		4.7
G8J	930		**-12.6**		**15.5**
G9	1095	21.0		-132.6	
G10	109.5		-13.9		9.1
G11	140.5	15.5		-155.5	
G12	140.5		-11.0		-26.8

The horizontal gauges across perpend joints are shown red and when the horizontal strains were compressive they are shown bold

The horizontal strains across the mortar joints in the bottom course given in Table 26, (unlike those in Test 6(b)), can be seen to be the same sign as the general run of horizontal strains on the respective faces of the wall although strangely the front horizontal strains were compressive while those at the back were tensile.

5.1.1.1d Test 6(d) Central load through 100mm x 150mm bearing plate concentric to the wall longitudinal centre line

The Test wall 6(d) was 5 courses high, three and a half blocks long (1100mm x 1545mm actual) built from 2.8 N/mm^2 compressive strength blocks. There were 22 No. demec gauges on the bottom course of blocks (Figure 32 and Figure 33)

The maximum load of 74 kN Maximum gave a stress of 4.93 N/mm^2 which is 1.76 times greater than the 2.8 N/mm^2 nominal block mean compressive strength.

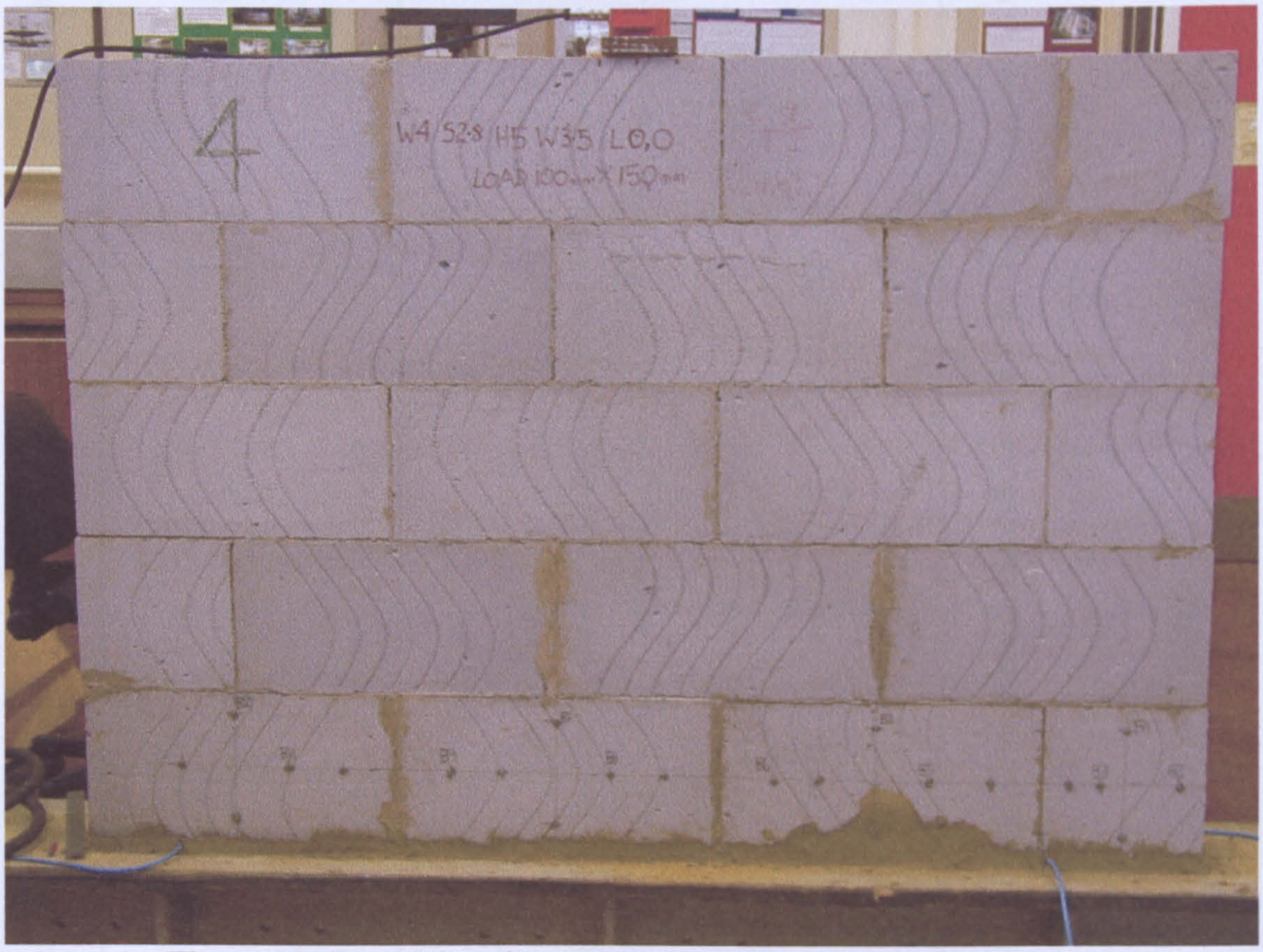


Figure 32: Test 6(d) – Central load and location of strain gauges

Demec locations (F) at front of wall (Wall front is defined as the side with the half block to the left on the bottom course)

Demec locations (B) at back of wall (Wall back is defined as the side with the half block to the right on the bottom course)

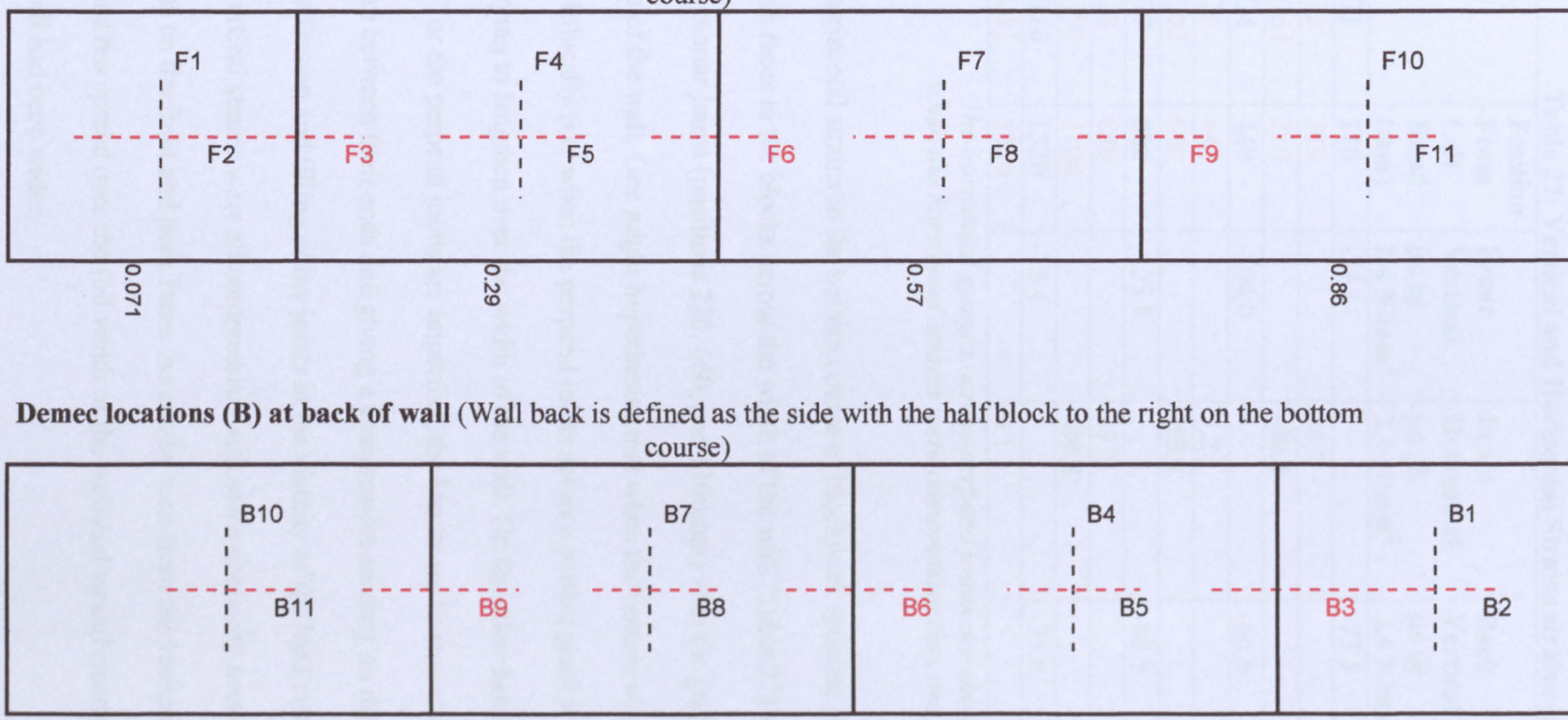


Figure 33: Demec gauge identification numbers and locations in the bottom course of 2.8 N/mm^2 walls

Table 27: Vertical and Horizontal Strains strains in Test 6(d)

	Position From Left Hand (mm)	Front Vertical με @ 2.4 N/mm^2	Front Horizontal με @ 2.4 N/mm^2	Back Vertical με @ 2.4 N/mm^2	Back Horizontal με @ 2.4 N/mm^2
G1	**110**	-74.1		-73.5	
G2	**110**		15.7		20.6
G3J	**220**		**-40.4**		**-30.3**
G4	**440**	-104.0		-66.6	
G5	**440**		5.2		-0.8
G6J	**660**		**-55.7**		**-8.8**
G7	**880**	-125.8		-88.2	
G8	**880**		17.1		18.8
G9J	**1100**		**-20.8**		**-32.4**
G10	**1320**	-78.4		-56.6	
G11	**1320**		0.1		8.6

The horizontal gauges across perpend joints are shown red and when the horizontal strains were compressive they are shown bold

The horizontal strains in the bottom course of blockwork indicate similar and low tension on both faces in the blocks across the width of the wall (Table 27) and compressive strain in the mortar joints (positions 220, 660, and 1100mm) with the greatest strain near the centre of the wall. One might hypothesise that when the bottom course behaves monolithically (i.e. when the perpend joints act as a perfect glue) it is in horizontal tension and trying to lengthen over the width of the wall. On the other hand, when there is no "glue" or the perpend joints are imperfect, the blocks try to extend individually reducing the gap between their ends and giving a compressive reading on the demec gauge. The poor adhesion and filling of the joints in the vicinity of the load can be seen in Figure 34. The vertical strains were all compressive with some similarity between corresponding gauges on the front and back faces. As can be seen from the vertical compressive strains, the load has spread over the full width of the wall and would clearly have spread further if the wall had been wider.

Figure 34: Test 6(d) in the vicinity of the load after failure

5.1.1.1e Test 6(e) Central load through 100mm x 150mm bearing plate concentric to the wall longitudinal centre line

The Test wall 6(e) was 5 courses high, three and a half blocks long (1095mm x 1545mm actual) built from 2.8 N/mm^2 compressive strength blocks.

The maximum load of 73.0 kN gives a stress of 4.92 N/mm^2 which is 1.75 times greater than the 2.8 N/mm^2 nominal block mean compressive strength.

As in Test 6(d) the horizontal strains in the bottom course of blockwork indicate similar tension on both faces in the blocks across the width of the wall and compression in the mortar joints with the greatest strain in those near the centre of the wall.

The vertical strains were all compressive.

5.1.1.1f Test 6(f) End loads through 100mm x 100mm bearing plates both ends simultaneously, concentric to the wall longitudinal centre line

The Test 6(f) was 5 courses high, three and a half blocks long (1100mm x 1545mm actual) built in 2.8 N/mm^2 compressive strength blocks. End loads were applied through 100mm x 100mm bearing plates both ends simultaneously, axially to the wall longitudinal centre line (Figure 35).

The sum of loads both ends of 36 kN at first cracks gives a stress of 1.80 N/mm^2 which is only 0.6 times the 2.8 N/mm^2 nominal block mean compressive strength. This appears to have been initiated by poor adhesion in the perpend joint of the end block supporting the load (Figure 36). Crushing of the bearing did not occur until a sum of loads both ends of 80 kN was applied which gives a stress of 4.0 N/mm^2 which is 1.43 times greater than the 2.8 N/mm^2 nominal block strength.

The vertical strains in the bottom course of blockwork are compressive reducing towards the centre of the wall indicating a spread over half the width of the wall 36%.

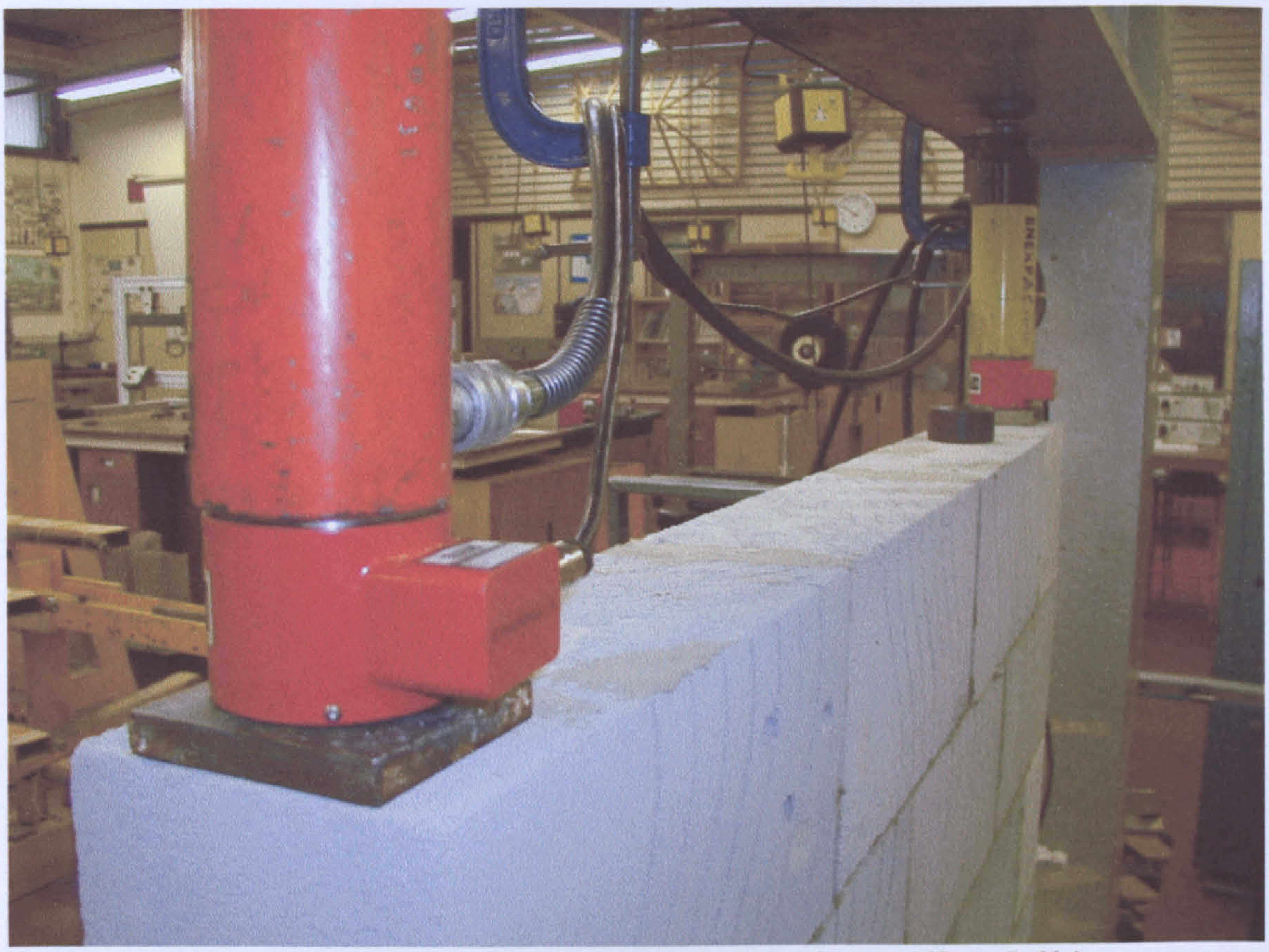


Figure 35: End loading arrangement for Tests 6(f) and 6(g)

Figure 36: Test 6(f) First crack

5.1.1.1g Test 6(g) End loads through 100mm x 100mm bearing plates both ends simultaneously concentric to the wall longitudinal centre line

The Test wall 6(g) was 5 courses high, three and a half blocks long (1095mm x 1550mm actual) built from 2.8 N/mm^2 compressive strength blocks.

The sum of loads both ends of 29.6 kN at first crack at one end gives a stress of 1.48 N/mm^2 which is 0.53 times the 2.8 N/mm^2 nominal block strength. Like test 6(f), Test 6(g) suffered from poor adhesion in some perpend joints (see Figure 37).

Crushing of the bearing did not occur until a sum of loads both ends of 86 kN was applied which gives a stress of 4.3 N/mm^2 which is 1.54 times greater than the 2.8 N/mm^2 nominal block strength.

The vertical strains in the bottom course of blockwork were compressive reducing towards the centre of the wall indicating a spread over half the width of the wall of 36^0 or more if there was any overlap of the strains.

The horizontal strains were also compressive, possibly because of a clamping action caused by loading at both ends simultaneously.

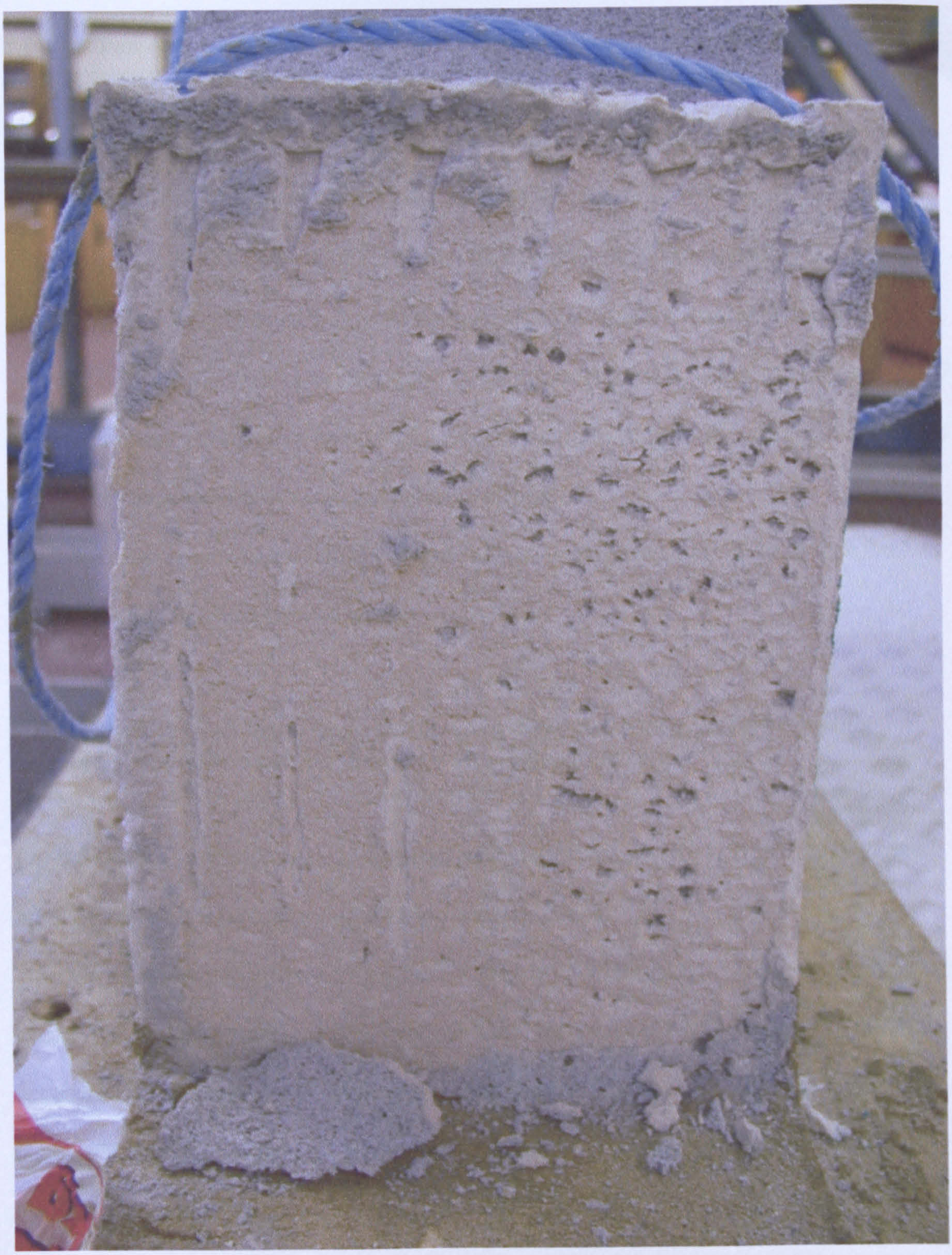


Figure 37: Test 6(g) Perpend joint in bottom course after failure

5.1.1.1h Test 6(h) End loads through 100mm x 100mm bearing plates both ends simultaneously 25mm eccentric to the wall longitudinal centre line

The Test wall 6(h) was 5 courses high, two and a half blocks long (1275mm x 1565mm actual) built from 2.0 N/mm^2 compressive strength blocks.

The sum of loads both ends of 24 kN at first visible cracking gives a stress of 1.2 N/mm^2 which is only 0.6 times the 2.0 N/mm^2 nominal block strength. Crushing of the bearing at one end and spalling at the other did not occur until the sum of the loads at both ends of 46 kN was applied. This gives a stress of 2.3 N/mm^2 which is only 1.15 times greater than the 2.0 N/mm^2 nominal block mean compressive strength.

The horizontal strains in the bottom course showed consistency front to back with the strains being tensile beneath the loads.

5.1.1.1i Test 6(i) End loads through 100mm x 100mm bearing plates both ends simultaneously 25mm eccentric to the wall longitudinal centre line

The Test wall 6(i) was 5 courses high, two and a half blocks long (1275 mm x 1560 mm actual) built from 2.0 N/mm^2 compressive strength blocks.

The sum of loads both ends of 27 kN at first visible cracking gives a stress of 1.35 N/mm^2 which is only 0.68 times the 2.0 N/mm^2 nominal block strength. End failure and spalling occurred after the sum of the loads applied at both ends of 51 kN was applied for about a minute. This gives a stress of 2.5 N/mm^2 which is 1.25 times greater than the 2.0 N/mm^2 nominal block mean compressive strength.

The vertical strain gauges at each end of the bottom course of blocks F1, F10, and B1 showed very close matching strains to each other with linear strain increase with load up to first crack followed by further linear strain increase (but on a different slope) from first crack to ultimate load. Strain gauge B10 also showed similar behaviour up to first crack but showed very little increase in strain after that. Furthermore, all of both the vertical and horizontal strain gauges showed good correlation (front face to back face) at maximum reading, indicating that although the loads were applied eccentrically, no eccentricity was apparent at the base.

5.1.1.2 Effects of Eccentricities

Walls 6(a), 6(b), 6(h) and 6(i) had their loads applied 25mm eccentrically to their longitudinal centre lines towards the front of the wall. According to a simple calculation using statics for the cases considered i.e. 150mm thick wall, 100x100mm loading plate, the load is at the edge of the "middle third" giving compressive stress at the front face of the wall as twice the stress under the bearing and the stress at the back of the wall as zero i.e. no tension.

5.1.1.3 Comparison between Test 6(a) and Test 6(b)

Comparing the results of Test 6(a) and 6(b), the crude calculations of the Modulus of Elasticity compare well with the value adopted for the FE calculations (Chapter 6).
In both tests, the eccentric loading caused the back face of the base of the wall to attract the greater magnitude of compressive strain across the whole width of the wall. Commonly, when designing loadbearing masonry it is assumed that the eccentricity of the load at the top of the wall become concentric by the time it reaches the base of the wall. This would fit with the wall rotating away from the eccentric load to make it axial. BS 5628 cl 27 (29) reads".........The resultant eccentricity of the load at any level may be calculated on the assumption that the total vertical load on a wall is axial immediately above a lateral support". It can be seen that the consequence of not doing this would result in all the eccentricities in a multi storey loadbearing masonry building becoming accumulative.

As the load spread is over the whole width of the wall in the base course, the Tan of the angle of spread is greater than 1025/732.5 = 1.3993 which gives an angle of spread greater than 35^0 to the vertical i.e. greater than the 30^0 spread allowed in EC6.

5.1.1.4 Comparison between Tests 6(d) and 6(e)

The Test walls 6(d) and 6(e) were 5 courses high, three and a half blocks long built from 2.8 N/mm^2 compressive strength blocks. A central load was applied through a 100mm x 150mm bearing plate concentric to wall longitudinal centre line.

The comparison back to front for the vertical strains in the bottom courses of Wall 6(d) and Wall 6(e) appear to be fairly axial in Table 28 and Table 29 and indicate that the concentrated wall has spread over the full width of the wall (Table 30). Although the walls were not wide enough at 1545 mm (36^0) to confirm a 45^0 spread they showed that the spread was considerably more than the 30^0 of EC6 which would only give a spread of 1250 mm. The consistency of the vertical compressive strains spread across the full width of the base of both walls points to a spread of 45^0 (2120 mm) or more.

Horizontal strains in the blocks were consistently tensile while at the perpend joints they are compressive. This phenomenon has been commented on previously.

Table 28: Test 6(d) Vertical and horizontal strains in the bottom course

	Position (mm)	Front Vertical με @ 2.4 N/mm²	Front Horizontal με @ 2.4 N/mm²	Back Vertical με @ 2.4 N/mm²	Back Horizontal με @ 2.4 N/mm²
G1	110	-74.1		-73.5	
G2	110		15.7		20.6
G3J	220		**-40.4**		**-30.3**
G4	440	-104.0		-66.6	
G5	440		5.2		-0.8
G6J	660		**-55.7**		**-8.8**
G7	880	-125.8		-88.2	
G8	880		17.1		18.8
G9J	1100		**-20.8**		**-32.4**
G10	1320	-78.4		-56.6	
G11	1320		0.1		8.6

vertical gauges shown black, horizontal gauges red

Table 29: Test 6(e) Vertical and horizontal strains in the bottom course

	Position (mm)	Front Vertical με @ 2.4 N/mm²	Front Horizontal με @ 2.4 N/mm²	Back Vertical με @ 2.4 N/mm²	Back Horizontal με @ 2.4 N/mm²
G1	110	-130.9		-37.4	
G2	110		3.8		-3.5
G3J	220		**-21.2**		**6.0**
G4	440	-61.6		-85.1	
G5	440		9.8		7.0
G6J	660		**-47.5**		**-21.5**
G7	880	-113.3		-94.4	
G8	880		11.9		12.2
G9J	1100		**-25.0**		**-28.8**
G10	1320	-91.8		-55.3	
G11	1320		5.6		18.5

vertical gauges shown black, horizontal gauges red

Table 30: Tests 6(d) and 6(e) Mean Vertical compressive strains in bottom course

Position from left hand end (mm)	6(d) Front Vertical με @ 2.4 N/mm^2	6(e) Front Vertical με @ 2.4 N/mm^2	6(d) Back Vertical με @ 2.4 N/mm^2	6(e) Back Vertical με @ 2.4 N/mm^2	Mean Vertical με @ 2.4 N/mm^2
110	-74.1	-130.9	-73.5	-37.4	-79.0
440	-104.0	-61.6	-66.6	-85.1	-79.3
880	-125.8	-113.3	-88.2	-94.4	-102.9
1320	-78.4	-91.8	-56.6	-55.3	-70.5

5.1.1.5 Comparison between Test 6(f) and Test 6(g)

The Test walls 6(f) and 6(g) were 5 courses high, three and a half blocks long built from 2.8 N/mm^2 compressive strength blocks. End loads were applied through 100mm x 100mm bearing plates at both ends simultaneously axial to wall longitudinal centre line.

It can be seen from Table 31 in both walls the vertical strains in the bottom course of blockwork were compressive reducing towards the centre of the wall indicating a 36%+ spread from the vertical over half the width of the wall.

Table 31: Test 6(f) and Test 6(g) Mean vertical compressive strains in bottom course

Position from left hand end (mm)	6(f) Front Vertical με @ 1.42 N/mm^2	6(f)Back Vertical με @ 1.42 N/mm^2	6(g) Front Vertical με @ 1.42 N/mm^2	6(g) Back Vertical με @ 1.42 N/mm^2	Mean vertical με @ 1.42 N/mm^2
110	-91.0	-135.5	-132.8	-133.6	-123.2
440	-40.2	-56.7	33.9	-34.9	-24.0
880	8.8	-25.0	-24.9	-25.5	-16.6
1320	-82.0	-130.9	-40.4	-107.9	-90.3

5.1.1.6 Comparison between Test 6(h) and Test 6(i)

The Test wall 6(h) and 6(i) were 5 courses high, two and a half blocks long built from 2.0 N/mm^2 compressive strength blocks. End loads were applied through 100mm x 100mm bearing plates at both ends simultaneously 25 mm eccentric to wall longitudinal centre line.

Table 32 shows that in both walls the vertical strains in the bottom course of blockwork were compressive reducing towards the centre of the wall indicating a spread in excess of 36^0. The difference between the compressive strains on the front and the back faces of the wall was less than would be expected possibly due to the eccentric loading at the top of the wall at both ends becoming axial or nearly axial by the time it reaches the base. The trend lines in Figure 38 show the vertical compressive strains reducing from a maximum under the end loads towards zero near the centre of the test wall. Most of the horizontal strains are also compressive, possibly because of a clamping action caused by loading at both ends simultaneously. The two tests gave comparable results.

Table 32: Front and back Vertical compressive strains με in bottom course for Tests 6(h) and 6(i)

Position from left hand end (mm)	**Test 6(i) front**	**Test 6(i) back**	**Test 6(h) front**	**Test 6(h) back**
155	-98.8	-103.8	-49.7	-142.8
470	-61.1	-50.5	-8.5	-64.7
780	-36.2	-34.1	29.8	-24.2
1095	-58.3	-74.6	-20.6	-68.9
1405	-121.8	-147.8	-112.9	-149.2

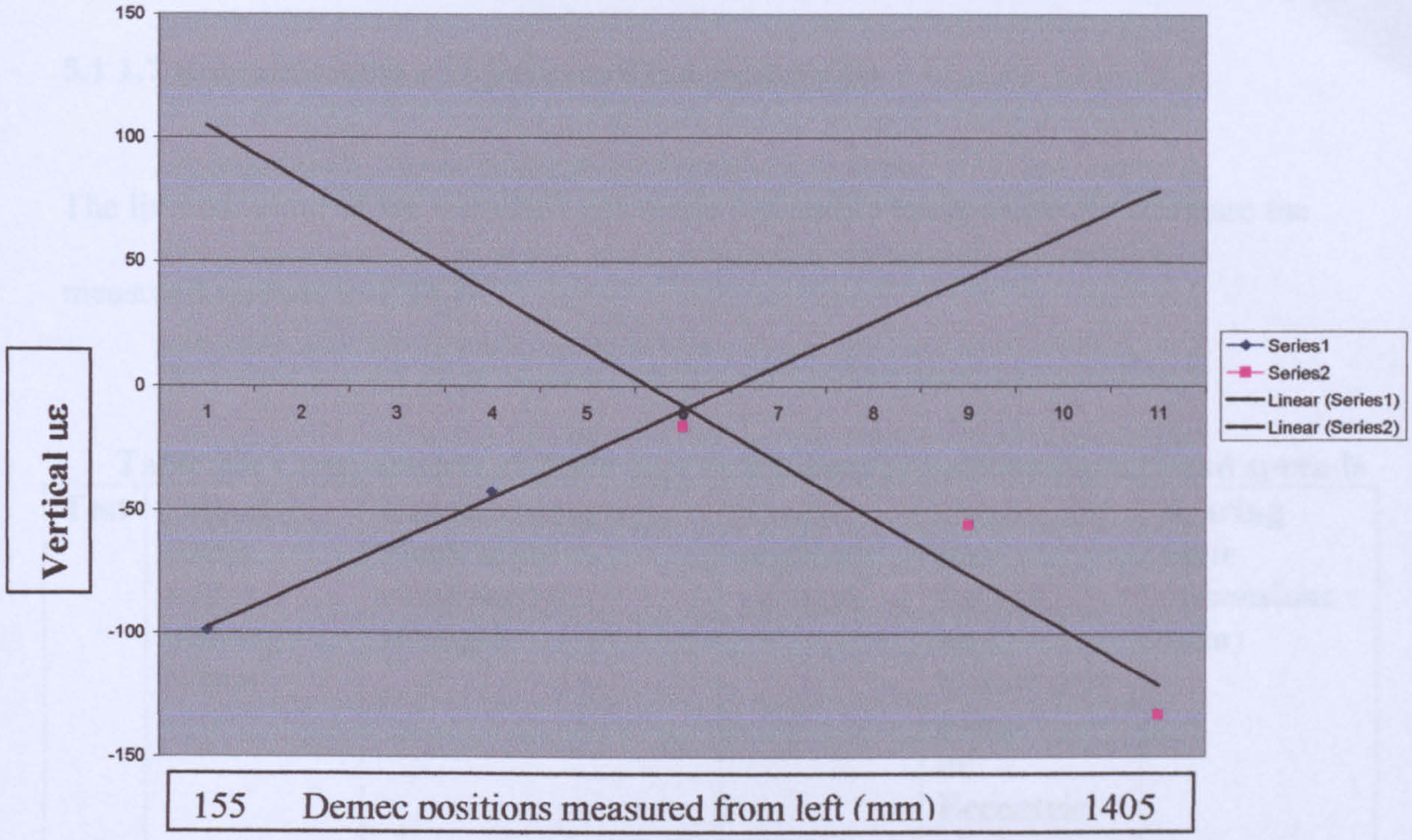


Figure 38: Trend lines for mean Vertical Compressive strains for Tests 6(h) and 6(i)

5.1.1.7 Enhancements and Spreads Tests 6(a) to 6(i)

The limited width of the test walls has made it possible to only directly compare the measured spreads with EC6.

Table 33: Comparisons of Tests 6(a) to 6(i) Loading enhancements and spreads

Test	Ultimate stress under bearing N/mm^2	Enhancement over block mean compressive strength	Spread angle to vertical*	Centre (C) or End (E) and Concentric (con) or Eccentric (ecc)	Bearing plate dimensions (mm)
6(a)	3.30	1.65(2.0N/mm^2)	36^0 ++	Cecc	100 x100
6(b)	3.65	1.83(2.0N/mm^2)	36^0 ++	Cecc	100 x100
6(c)	3.90	1.95(2.0N/mm^2)	36^0 ++	Ccon	100 x100
6(d)	4.93	1.76(2.8N/mm^2)	36^0 ++	Ccon	100 x150
6(e)	4.92	1.75(2.8N/mm^2)	36^0 ++	Ccon	100 x150
6(f)	4.0	1.43(2.8N/mm^2)	36^0 +	Econ	100 x100
6(g)	4.3	1.54(2.8N/mm^2)	36^0 +	Econ	100 x100
6(h)	2.3	1.15(2.0N/mm^2)	36^0+	Eecc	100 x100
6(i)	2.5	1.25(2.0N/mm^2)	36^0+	Eecc	100 x100

++ *indicates the spread from central loads was over more than the full width of the wall*
\+ *indicates the spread from end loads was over more than half the width of the wall*

Loads within the length of the wall achieved enhancements over the nominal block mean compressive strengths (measured in accordance with BS EN 772 Part 1) which ranged from 1.65 to 1.95. The enhancement for end loads ranged from 0.68, when the load was eccentric, to 1.54 for the best concentric case (Table 33).

In all cases of loads within the length of the wall, the spreads are greater than that allowed by EC6. All central load spreads exceeded the width of the test panels. Simple extrapolation down to the base level of the test panel indicates the 45^0 spread allowed by BS 5628 is safe. From an examination of the vertical strains from central loads in Table 34 it can be gleaned by taking the average strains in all positions that the strains at the base reduce out from the vertical centre line of the wall. At the ends of the walls the strains average half of the value the central strain. Assuming a triangular distribution, the total spread at the level of the base would approximately equal 45^0.

In the case of end loading, the spread angles from the vertical were in excess of 36^0 which again is greater than that allowed by EC6.

Summary of the vertical strains in the bottom courses of Wall Types 1, 2 and 6

The vertical strains in the bottom courses of Wall Types 1, 2 and 6 are summarised in Table 34.

Table 34: Vertical strains in the bottom course of Wall Types 1, 2 and Tests 6

Fraction h from top	**Horizontal Location (Fraction of wall length from left)**						
	0.08	0.17	0.33	0.5	0.67	0.83	0.92
Vertical Strains at front							
0.875	-23.9		-18.3		-20.8		-131.4
Wall Type 1 Test 1(a) - 100 mm x 100 mm central concentrated load vertical strains							
0.875	48.1		29.9		46		17.2
Wall Type 1 Test 1(b) – 100 mm x 150 mm central concentrated load vertical strains							
0.875	38.7		1.8		-5.6		37.2
Wall Type 1 Test 1(c) – 100 mm x 150 mm end concentrated load vertical strains							
	-15		**-60**	**-119**	**-67**		
Wall Type 2 Test 2(b) – 100 mm x 150 mm central concentrated load vertical strains							
	-117		0	23	0		
Wall Type 2 Test 2(c) – 100 mm x 150 mm end concentrated loads vertical strains							
	Horizontal Location (Fraction of wall length from left)						
	0.1		0.3	0.5	0.7		0.9
	Vertical strains at front and back						
Front	6.0		-0.8	-33.4	-12.4		9.2
Back	-138.3		-139.9	-133.9	-138.0		-60.4
Test 6(a) Central Load 100 mm x 100 mm 25 mm eccentric							
Front	-36.0		-40.3	-46.4	-49.7		-22.9
Back	-64.0		-86.8	-64.0	-74.0		-41.0
Test 6(b) Central Load 100 mm x 100 mm 25 mm eccentric							
Front	11.9		25.2	-16.2	21.0		15.5
Back	-128.9		-140.5	-160.3	-132.6		-155.5
Test 6(c) Central Load 100 mm x 100 mm load concentric							
Horizontal Location (Fraction of wall length from left)							
	0.07		0.28	0.57		0.85	
	Vertical strains at front and back						
Front	-74.1		-104.0	-125.8		-78.4	
Back	-73.5		-66.6	88.2		-56.6	
Test 6(d) Central Load 100 mm x 150 mm concentric							
Front	-130.9		-61.6	-113.3		-91.8	
Back	-37.4		-85.1	-94.4		-55.3	
Test 6(e) Central Load 100 mm x 150 mm concentric							
Front	-91.0		-40.2	8.8		-82.0	
Back	-135.5		-56.7	-25.0		-130.9	
Tests 6(f) End loads 100 mm x 100 mm 25 mm eccentric							
Front	-132.8		-34.9	-24.9		-40.4	
Back	-133.6		-34.9	-25.5		-107.9	
Tests 6(g) End loads 100 mm x 100 mm 25 mm eccentric							
Horizontal Location (Fraction of wall length from left)							
	0.1		0.3	0.5	0.7		0.9
	Vertical strains at front and back						
Front	-49.7		-8.5	29.8	-20.6		-112.9
Back	-142.8		-64.7	-24.2	-68.9		-149.2
Test 6(h) End loads 100 mm x 100 mm concentric							
Front	-98.8		-61.1	-36.2	-58.3		-121.8
Back	-103.8		-50.5	-34.1	-74.6		-147.8
Test 6(i) End loads 100 mm x 100 mm concentric							

5.1.2 Test Series 7 - Concentrated Loading on wallettes from joist hangers

The ability of low density Aircrete blockwork to support timber joists on joist hangers was assessed by proof testing. The bearing area of a joist hanger ancillary component is designed to work with a particular block nominal compressive strength. The tests were undertaken using blocks with compressive strengths of 2.0 and 2.8 N/mm^2. Wallettes which were "C" shape on plan were constructed of low density Aircrete blockwork. Steel joist hangers and timber joists were arranged as specified in the test procedure in *BS EN 846-8* part 8, 2000 – Determination of load capacity and load-deflection characteristics of joist hangers.

Thin layer mortar and two general purpose mortars were used to construct the Aircrete test wallettes which were prepared by a professional blocklayer.

Proprietary steel Joist Hangers were used (see Figure 39). They were single-piece, non-welded construction manufactured from 2 mm thick pre-galvanised mild steel to BS EN 10142 : 1991, DX51D + Z600. with 75mm load bearing surface in a standard width to support timber joists 100mm deep by 38mm wide. The joists were coniferous timber with a strength class of C16 measured in accordance with BS EN 338, 2003 – Structural timber. Strength classes.

Typical 2.0 and 2.8 N/mm^2 wallette specimens are illustrated in Figures 40 and 41 respectively. The dimensions of the 2.0 N/mm^2 blocks were 620 x 250 x 150mm, and the 2.8 N/mm^2 blocks were 440 x 215 x 150mm. After construction, the wallettes were covered with polyethylene sheets and allowed to cure for 28 ± 1 days before testing.

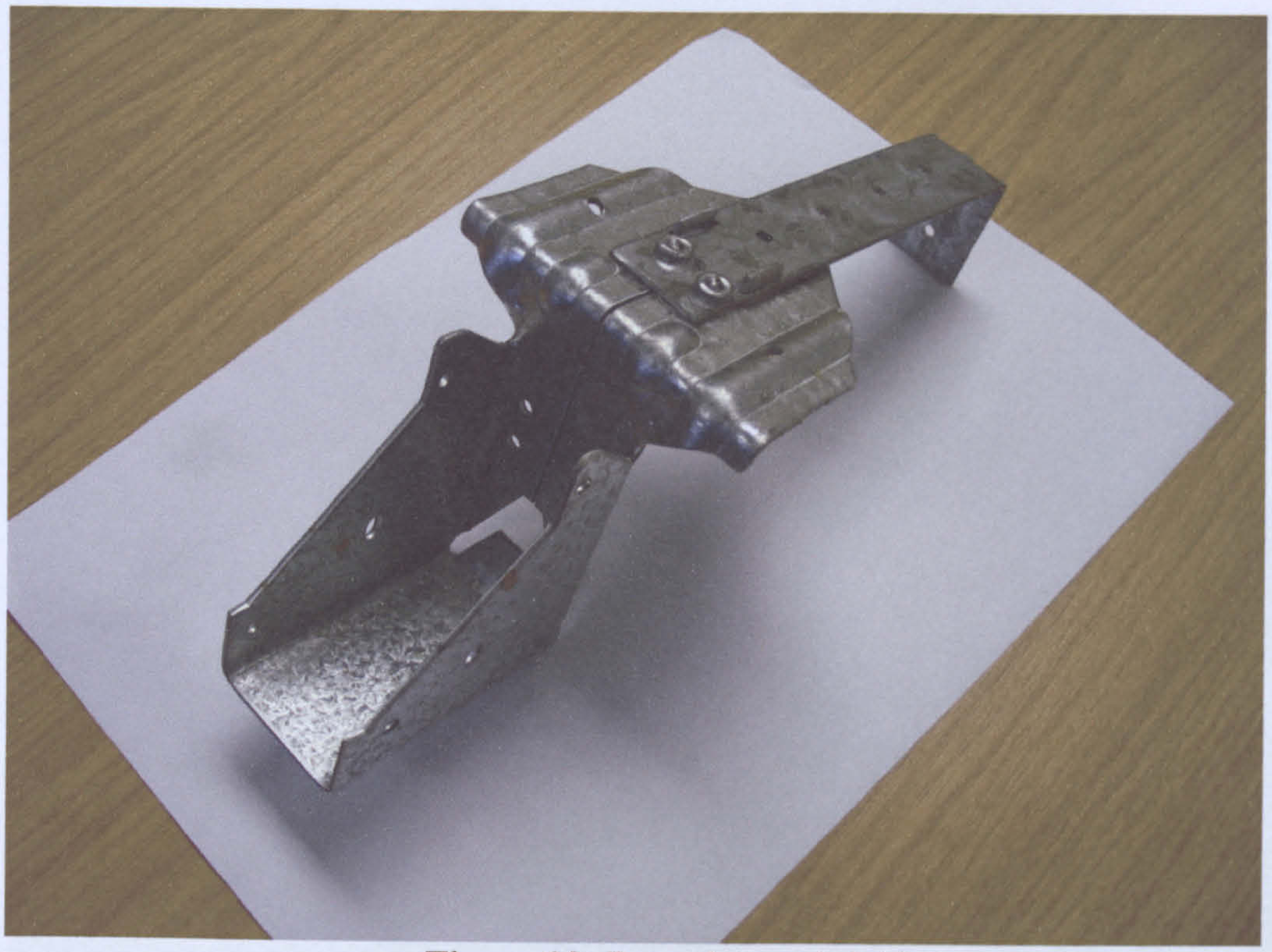


Figure 39: Proprietary joist hanger

Figure 40: Aircrete wallette built with 2.0 N/mm^2 blocks

Figure 41: Aircrete wallette built with 2.8 N/mm^2 blocks

Figure 42: Joist Hanger test arrangement

The method of applying a vertical load is shown in Figure 42, using a 100mm by 38mm timber joist 1000mm long. Three joist hangers were fixed to each wallette. The load was applied 2L / 3 from the joist hanger, where L is 1000 mm.

A pre-load of 1.0 kN was applied to the test specimen and held for a period of 1 minute. After the pre-load was removed, a load was applied at a rate of 1.0 kN per minute until failure occurred. Failure was defined as the load at which further deflection occurred without increase in test load. The failure load and the mode of failure were recorded.

Table 35 shows the Aircrete Block Compressive Strength and Mortar Designation of the wallettes tested.

Table 35: Aircrete Block Compressive Strength and Mortar Designation

Aircrete Block Compressive Strength (N/mm^2)	Mortar designation
2.0	iii
2.0	iv
2.0	Thin Layer
2.8	iii
2.8	iv
2.8	Thin Layer

Mortar designation iii = cement: lime: sand ratio of 1:1:6 (volume)
Mortar designation iv = cement: lime: sand ratio of 1:2:8 (volume)

Table 36 shows the maximum loads and the mode of failure for each of the lintels. As the load was applied at a distance 2L/3 from the joist hanger, the force on each individual hanger was one third of the applied maximum load. The joist hangers are numbered 1, 2 and 3 from left to right (see Figures 41 and 42). Joist hanger 2 is in the middle position, located one course higher than joist hangers 1 and 3. The maximum load value for the timber joist is the value sustained by the timber joist at a distance of 2L/3 from the joist hanger – point A (Figure 43).

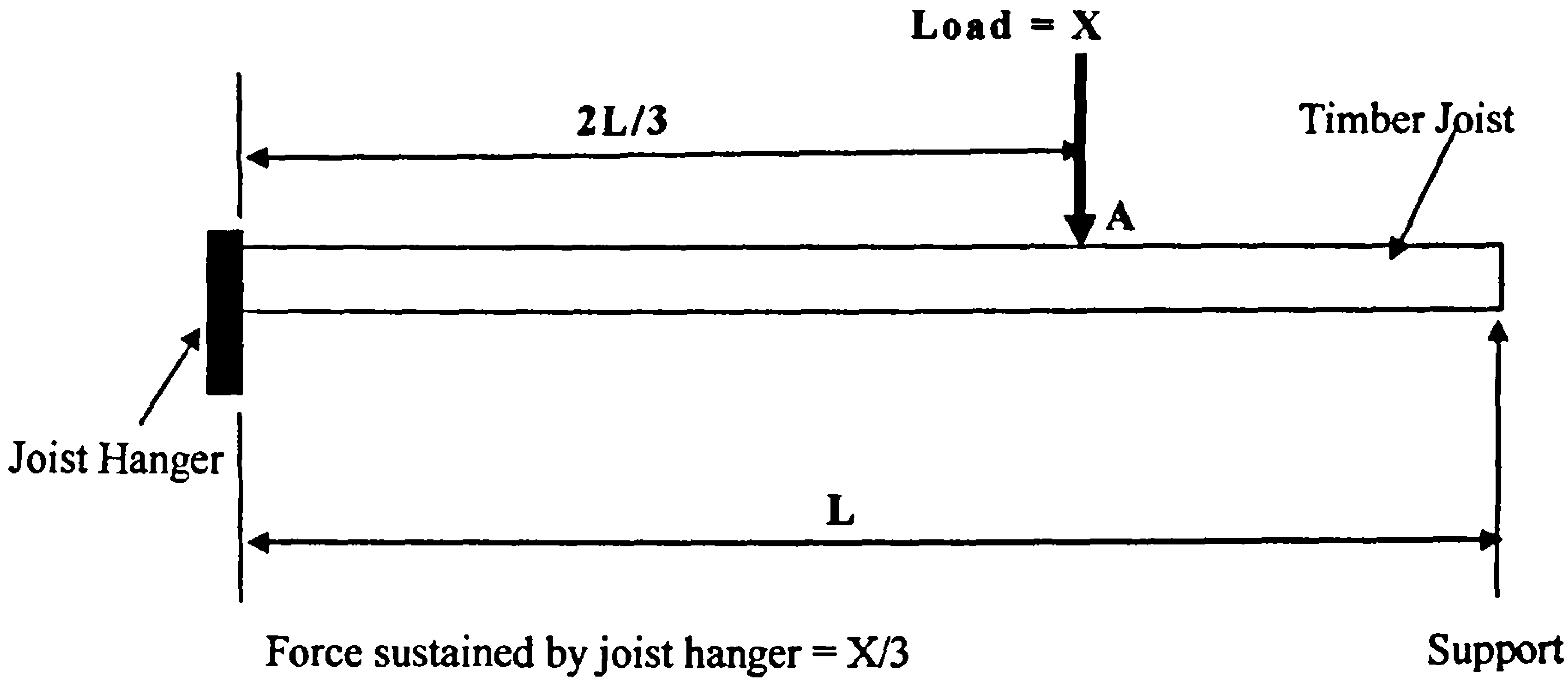


Figure 43: Schematic representation of joist hanger test

Table 36: Timber Joist failure loads

		Maximum Load (kN)				
Aircrete Block Strength (N/mm^2)	Mortar Designation	Timber Joist (Range)	Joist Hanger 1	Joist Hanger 2	Joist Hanger 3	Mode Of Failure
2.0	iii	16.1 – 16.5	5.4	5.4	5.5	Joist
2.0	iv	17.0 – 17.5	5.7	5.8	5.7	Joist
2.0	Thin Layer	16.6 – 16.8	5.5	5.6	5.5	Joist
2.8	iii	17.3 – 17.5	5.8	5.8	5.8	Joist
2.8	iv	16.9 – 17.2	5.7	5.6	5.7	Joist
2.8	Thin Layer	17.4 – 17.5	5.8	5.8	5.8	Joist

Table 36 gives the modes of failure for each test. In all cases the timber joist failed with all six Aircrete wallette specimens remaining unscathed. The results were very consistent with the timber joists failing at loads between 16.1 – 17.5 kN. The joist hanger bearings on the 2.0 N/mm^2 compressive strength Aircrete masonry safely sustaining a load between 5.4 – 5.8 kN. The joist hanger manufacturer gave the safe working load of this type of joist hanger as 3.08 kN for a masonry strength of 2.8 N/mm^2.

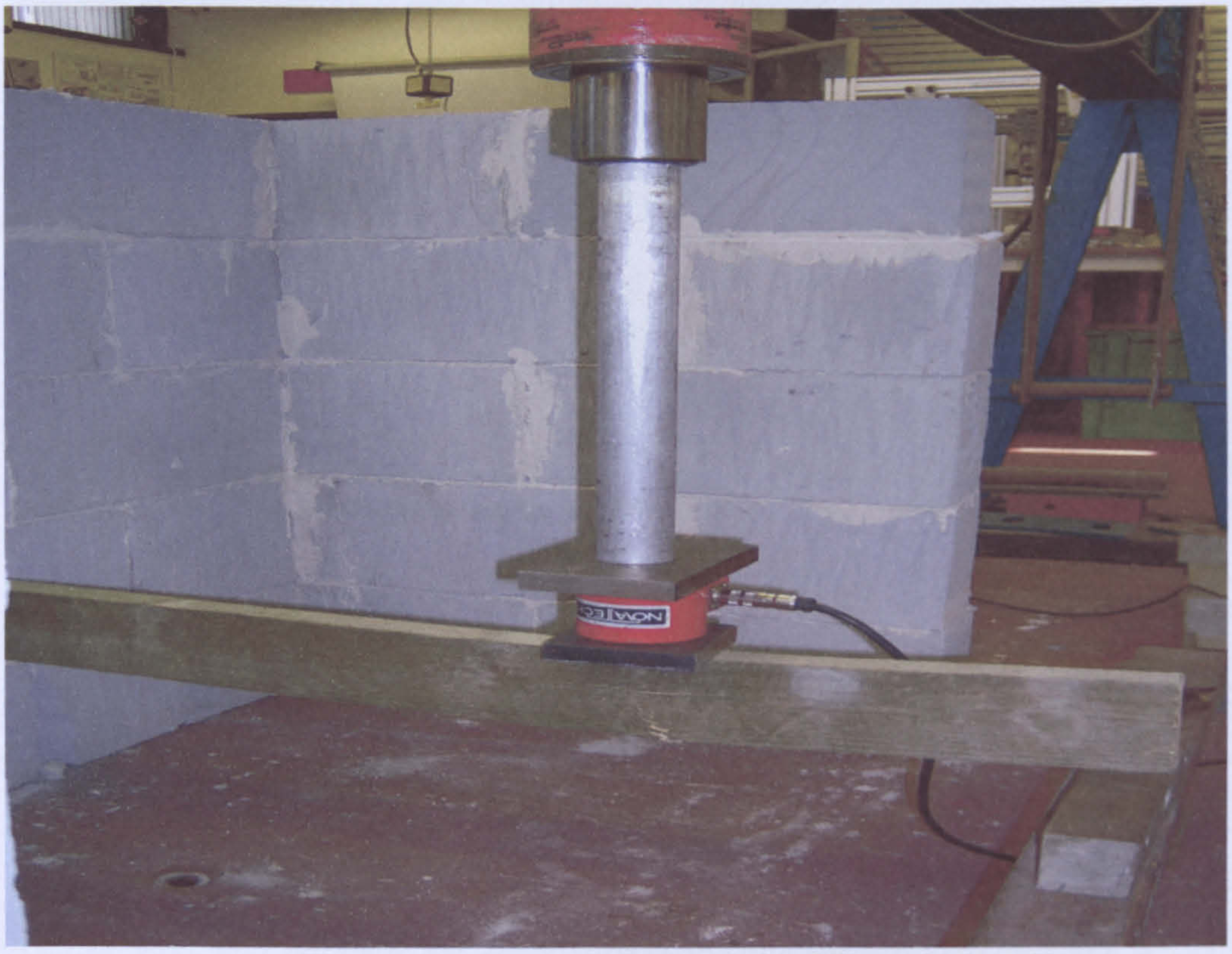


Figure 44: Typical timber joist failure in bending

As the timber joist failed in all tests (Figure 44), it was decided to replace the timber with a rectangular steel hollow section in order to evaluate the maximum load capacity of the joist hanger. The wallette specimen was 2.0 N/mm^2 Aircrete with Thin Layer mortar. The test arrangement is shown in Figure 45 with Figure 46 showing the joist hanger failure.

Figure 45: Test arrangement to measure the failure load of the joist hanger

Figure 46: Failure of joist hanger using metal joist

The mode of failure was localised crushing at the front edge of the wall with no visible crack of the wall (Figure 46). The joist hanger did not fully pull-out. It is assumed that this is due to the restrain imposed on the back of the wall by the joist hanger "tongue" at the back of the wall. Thus, the ultimate load of 8.0 kN represents more of a serviceability limit of the hanger than its ultimate load. The shape of the joist hanger where it bears on the blockwork and the eccentric nature of the load complicated by the action of the strap to the back face of the wall do not lend themselves to a calculation of stress in the wall immediately below the load. However the load achieved 0f 8.0 kN gives a factor of safety of 3.5 over the manufacturers stated safe working load. BS 5626 Part1 cl 27 reads".........Where joist hangers are used, the load should be assumed to be applied at the face of the wall."(29)

In all tests using timber joists (100 x 38 x 1000mm), the timber joist failed before the Aircrete wallette or the joist hanger. In each case the Aircrete masonry wallette was left unscathed. Therefore the maximum test loads were proof loads as far as the Aircrete was concerned which demonstrated there is no problem with the principle of using joist hangers to support joists on low density Aircrete.

Chapter 6. Finite Element Analysis

Finite element analysis (FE) was used as a tool to measure the distribution of displacements across the masonry, treating the thin joint masonry wall as a plate with nodes were placed to coincide with the corners of the Aircrete blocks. The wall used for the FE analysis was considered to be an Aircrete masonry wall, 4 block courses high and three blocks long in thin layer mortar (i.e. 1.0 m high x 1.80 m long). The thickness was taken to be 150 mm. Thus the dimensions were similar to those used for the walls in the laboratory testing programme.

For this form of macro modelling, considering the masonry as a one phase material, no distinction between the masonry units and mortar was made. This assumption is likely to be appropriate for thin joint masonry while not being valid for masonry in general purpose mortar where there are planes of weakness at the interfaces between mortar joints and blocks, particularly in the perpends. The diagrammatic mesh of elements representing the wall was set out and resulted in 633 Node Numbers and their dimensional positions were spaced at the distances shown in Table 37. Their positions are also shown diagrammatically in Figure 47.

An elastic strain analysis was undertaken assuming isotropic elastic behaviour. In the setting up of the stiffness matrix, Young's Modulus was taken as 1500 N/mm^2 and Poisson's Ratio as 0.2.

The vertical and horizontal strains were then calculated from the displacements of the nodes. Compressive strains were treated as negative and tensile strains as positive. Any strengthening or stiffening provided by the thin jointing material was ignored.

6.1 Central Concentrated Load

The first stage in the FE process for a central concentrated load was to recognise that, as the wall was symmetrical about its centre line, only half the width of the wall needed to be used in the analysis.

The load was considered to be applied through a flexible bearing plate 150 mm wide by 150 mm long (i.e. over the full wall thickness).

An 18000 N load was applied to the model via 5 nodes, 1/12, 1/3, 1/6. 1/3 and 1/12, i.e. nodes number 631, 632 and 633 in the FE calculation.

Therefore 1/6, 2/3, 1/6 of 9000 N was applied through 3 nodes per side.

To prevent the structure "flying", zero horizontal displacements were applied to the nodes on the centre line although they were allowed to move vertically.

Two base conditions were considered:

A "fixed" base where the nodes had zero horizontal and vertical displacements

and

A "roller" base where the nodes had zero vertical displacements but were free to move horizontally except for the central node had zero vertical and horizontal displacements.

Table 37: Node Numbers and dimensional positions for the Finite Element Analyses

Node	x mm	y mm	mm
1	0	0	150
2	37.5	0	150
3	75	0	150
4	112.5	0	150
5	150	0	150
6	187.5	0	150
7	225	0	150
8	262.5	0	150
9	300	0	150
10	337.5	0	150
11	375	0	150
12	412.5	0	150
13	450	0	150
14	487.5	0	150
15	525	0	150
16	562.5	0	150
17	600	0	150
18	637.5	0	150
19	675	0	150
20	712.5	0	150
21	750	0	150
22	787.5	0	150
23	825	0	150
24	862.5	0	150
25	900	0	150
26	0	12.5	150
27	75	12.5	150
28	150	12.5	150
29	225	12.5	150
30	300	12.5	150
31	375	12.5	150
32	450	12.5	150
33	525	12.5	150
34	600	12.5	150
35	675	12.5	150
36	750	12.5	150
37	825	12.5	150
38	900	12.5	150
and then repeat sequence in second (x) column from			
39	0	25	150
to			
63	900	25	150

then from			
64	0	62.5	150
to			
76	900	62.5	150
then from			
77	0	100	150
to			
101	900	100	150
then from			
102	0	137.5	150
to			
114	900	137.5	150
then from			
115	0	175	150
to			
139	900	175	150
then from			
140	0	187.5	150
to			
152	900	187.5	150
then from			
153	0	200	150
to			
177	900	200	150
then from			
178	0	212.5	150
to			
190	900	212.5	150
then from			
191	0	225	150
to			
215	900	225	150
then from			
216	0	262.5	150
to			
228	900	262.5	150
then from			
229	0	300	150
to			
266	900	337.5	150
then from			
267	0	375	150
to			
291	900	375	150
then from			
292	0	387.5	150

to			
304	900	387.5	150
then from			
305	0	400	150
to			
329	900	400	150
then from			
330	0	412.5	150
to			
342	900	412.5	150
then from			
343	0	425	150
to			
367	900	425	150
then from			
368	0	462.5	150
to			
380	900	462.5	150
then from			
381	0	500	150
to			
405	900	500	150
then from			
406	0	537.5	150
to			
418	900	537.5	150
then from			
419	0	575	150
to			
443	900	575	150
then from			
444	0	587.5	150
to			
456	900	587.5	150
then from			
457	0	600	150
to			
481	900	600	150
then from			
482	0	612.5	150
to			
494	900	612.5	150
then from			
495	0	625	150
to			
519	900	625	150

then from			
520	0	662.5	150
to			
532	900	662.5	150
then from			
533	0	700	150
to			
557	900	700	150
then			
558	0	737.5	150
559	75	737.5	150
560	150	737.5	150
561	225	737.5	150
562	300	737.5	150
563	375	737.5	150
564	450	737.5	150
565	525	737.5	150
566	600	737.5	150
567	675	737.5	150
568	750	737.5	150
569	825	737.5	150
570	900	737.5	150
571	0	775	150
572	37.5	775	150
573	75	775	150
574	112.5	775	150
575	150	775	150
576	187.5	775	150
577	225	775	150
578	262.5	775	150
579	300	775	150
580	337.5	775	150
581	375	775	150
582	412.5	775	150
583	450	775	150
584	487.5	775	150
585	525	775	150
586	562.5	775	150
587	600	775	150
588	637.5	775	150
589	675	775	150
590	712.5	775	150
591	750	775	150
592	787.5	775	150
593	825	775	150
594	862.5	775	150
595	900	775	150
596	0	787.5	150
597	75	787.5	150
598	150	787.5	150
599	225	787.5	150
600	300	787.5	150
601	375	787.5	150
602	450	787.5	150
603	525	787.5	150
604	600	787.5	150
605	675	787.5	150
606	750	787.5	150
607	825	787.5	150
608	900	787.5	150
609	0	800	150
610	37.5	800	150
611	75	800	150
612	112.5	800	150
613	150	800	150
614	187.5	800	150
615	225	800	150
616	262.5	800	150
617	300	800	150
618	337.5	800	150
619	375	800	150
620	412.5	800	150
621	450	800	150
622	487.5	800	150
623	525	800	150
624	562.5	800	150
625	600	800	150
626	637.5	800	150
627	675	800	150
628	712.5	800	150
629	750	800	150
630	787.5	800	150
631	825	800	150
632	862.5	800	150
633	900	800	150

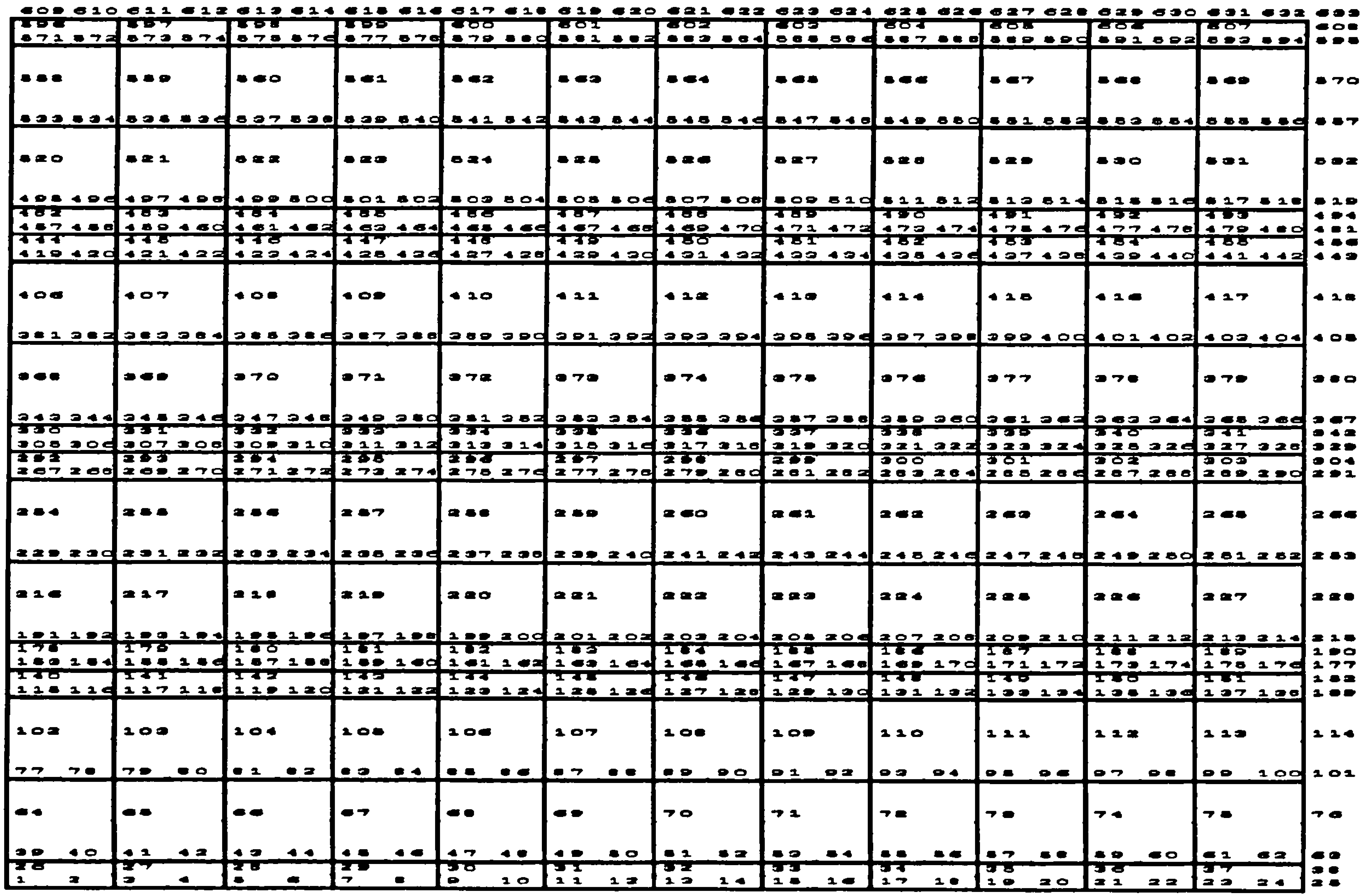


Figure 47: Blockwork half storey height wall –mesh-node number arrangement for FEA

The movements of the nodes from a central point load i.e. placed at the right hand end of the half wall used in the FE analysis were plotted on spread sheets. The extremely localised load makes the mesh show some of the defects of the 8-noded element, namely a ripple in the displacements close to the load application point (Figure 48).

The analysis was undertaken using a simple program based around plane stress,

8-node, numerically-integrated, isoparametric elements of Zienkiewicz's "Serendipity family" (80)(programmed for the computer by Professor E N Bromhead of Kingston University).

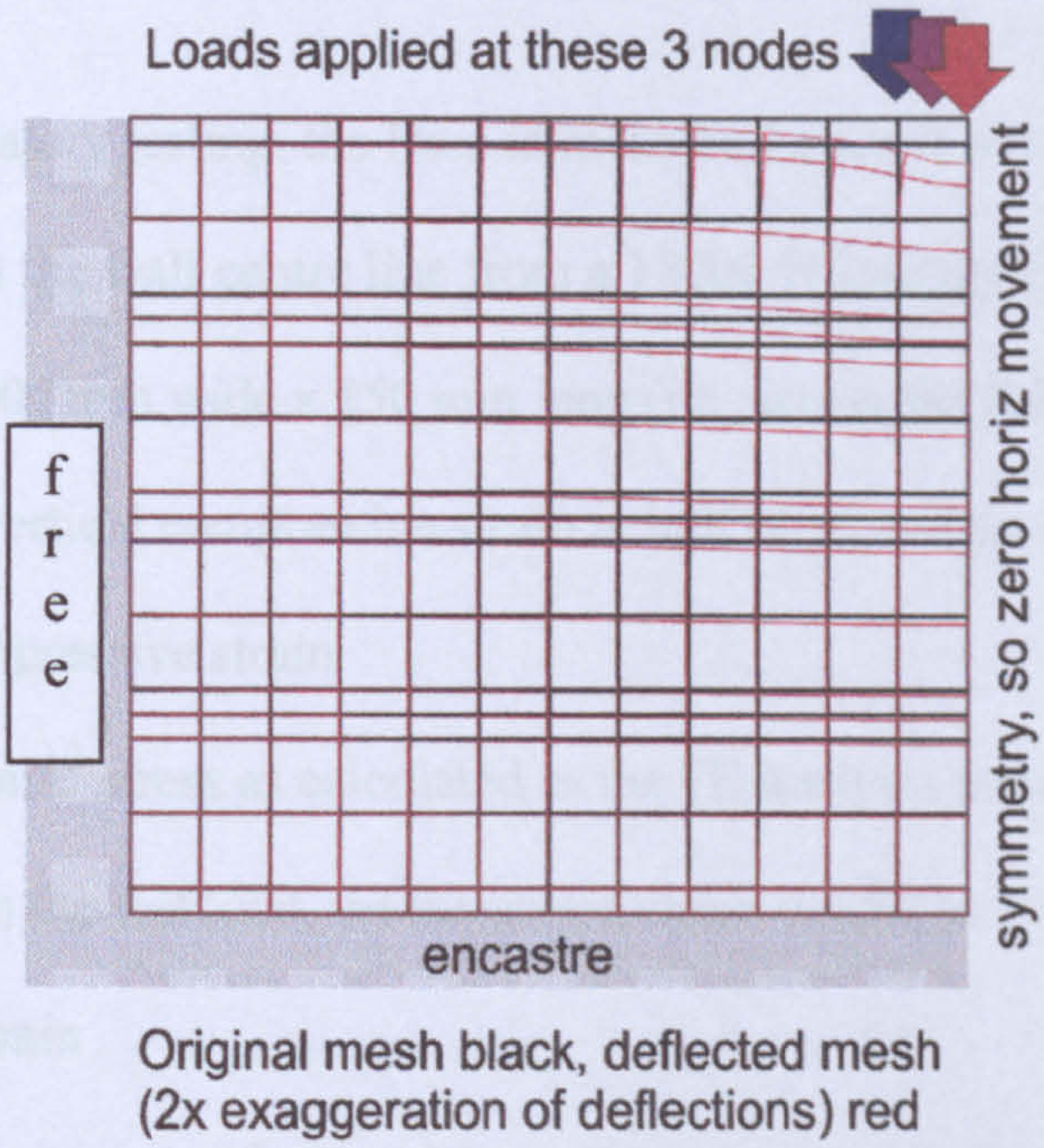


Figure 48: Mesh under central load FEA

Results for Central concentrated load
Strain immediately below the concentrated load

For comparison, the approximate stress immediately under the bearing plate from a manual elastic calculation assuming the 18000N load is applied over a bearing area of 150mm x 150mm = 9000/(75x150) N/mm^2 = 0.8 N/mm^2 and the corresponding vertical strain = 0.8/1500 = 0.00053 compression

From the FE analysis the vertical compressive strain at half block height in top course is calculated to be = vertical deflection of node 570, minus vertical deflection of node 532, divided by the distance between the nodes (75mm) = (0.127 - 0.098)/75 = 0.000387 compressive strain

From the laboratory testing, the Demec measured compressive strain in vertical gauge No. 4 on the wall centre line from a 18000 N load applied through a rigid bearing plate 100 mm wide x 150 mm long (i.e. across the full wall thickness) the measured vertical compressive strain at half block height in the top course = 0.0006371 compressive strain.

For the 0.8 N/mm^2 stress as calculated in the FE analysis instead of 1.2 N/mm^2 calculated from the test load, the measured strain would be 0.000421 compressive strain

There is close agreement for the compressive strain immediately under the load between the three methods i.e. simple elastic calculation, FE calculation and physical test (Table 38).

Table 38: Compressive strain immediately under the central concentrated load

	Compressive strain immediately under the central concentrated load
Simple manual elastic calculation assuming uniform stress distribution	0.00053
laboratory test measurement	0.000637
FEA calculation	0.000421

Strain at the base of the wall for central concentrated load

Assuming uniform distribution over the full width of the base, the stress across the base of the wall by simple manual elastic calculation = 18000/150/1800 = 0.066 N/mm^2 compression.

The corresponding compressive strain = stress/Young's Modulus = 0.066/1500 = 0.000044.

Assuming triangular distribution over the full width of the base, the stress the base of the wall by simple manual elastic calculation at the centre of the wall = 0.132 N/mm^2 compression and the stress at the end of the wall is zero.

The corresponding compressive strain at the centre of the wall = stress/Young's Modulus = 0.132/1500 = 0.000088.

Vertical and horizontal strain from a central load comparing fixed and roller base conditions are shown in Table 39. They indicate that the vertical strains in the roller base and the fixed base conditions are reasonably similar but there was no measurement in the base course at the centre There were larger differences in the horizontal strains except on the centre line, as would be expected.

In addition, the compressive vertical strain on the centreline in both cases reduces down the height of the wall, while the tensile horizontal strain, although much smaller than the vertical compressive strain, remains fairly constant. This may account for the vertical splitting mode of failure which is common under concentrated loads.

The distribution of vertical strains across the roller base case is given in Table 40 and in Table 41 for the fixed base case and show that the vertical compressive strain at the centre of the wall in the bottom course in both cases is approximately double the average strain.

Table 39: FE Vertical and horizontal strains from a central concentrated load comparing fixed and roller base conditions

ROLLER BASE				
Proportions of length of wall from left hand end				
	0.00833	**0.167**	**0.33**	**0.5**
FE vertical strains				
Top course		**-0.000001**		**-0.00029**
3rd course				
2nd course		**-0.000008**		**-0.000062**
Base course	**-0.000002**		**-0.000053**	
Vertical compressive strains shown bold				
FE horizontal strains				
Top course				0.000027
3rd course				0.000030
2nd course		0.000001	0.000021	0.000026
Base course	0.000001		0.000030	
FIXED BASE				
Proportions of length of wall from left hand end				
	0.00833	**0.167**	**0.33**	**0.5**
FE vertical strains				
Top course		0.000117		**-0.00019**
3rd course				
2nd course		**-0.000008**		**-0.000057**
Base course	**-0.000005**		**-0.00003**	
Vertical compressive strains shown bold				
FE horizontal strains				
Top course				0.000026
3rd course				0.000026
2nd course		-0.000013	0.000004	0.000017
Bottom course	-0.000008		0.000004	

Table 40: FE vertical strains from central load at bottom course - roller base

FE vertical strains at nodes across the roller base from Central load												Centre line	
Node 1	3	5	7	9	11	13	15	17	19	21	23	25	
-Node 26	-27	-28	-29	-30	-31	-32	-33	-34	-35	-36	-37	-38	**Average strain at base**
0.000156	0.000059	-0.000046	-0.00016	-0.00028	-0.00042	-0.00057	-0.00073	-0.00089	-0.00103	-0.00116	-0.00124	-0.00127	-0.0006

Table 41: FE vertical strains from central load at bottom course - fixed base

FE vertical strains at nodes across the fixed base from Central load												Centre line	
Node 1	3	5	7	9	11	13	15	17	19	21	23	25	
-Node26	-27	-28	-29	-30	-31	-32	-33	-34	-35	-36	-37	-38	**Average strain at base**
-0.000532	-0.000181	-0.000165	-0.000214	-0.000290	-0.000383	-0.000489	-0.000603	-0.000718	-0.000825	-0.000914	-0.00097	-0.000993	-00056

Although the laboratory tests for both types of central concentrated loads indicated that the load spreads across the full width of the base of the wall used in the tests, the results of the FE analysis above indicate that the spread is 1.8-.225=1.575 i.e. 45^0 for the roller base condition as the vertical strains at nodes No.1 and No.3 are tensile.

The FE analysis for the fixed base condition indicates that the load spread is over the full width of the wall and beyond and is rather more evenly spread than the roller base case. Thus the fixed base analysis is felt to reflect the practical situation of a wall bedded on a mortar bed better that a roller base analysis although the average strain over the full width of the base is similar. In both cases the vertical strain across the base spreads at an angle of at least 45^0 but reduces from a maximum under the concentrated load to zero at the end of the spread. This reduction can be taken as linear over most of the distance.

6.2 End concentrated loads

The same FE mesh with the same node numbers and positions as was used for the central concentrated examples was adopted. A vertical concentrated load was applied at both ends of the wall used for the FE analyses to stop the wall "flying" as was done in the physical testing for end loading. As the wall was symmetrical about its centre line, only half of the wall needed to be used in the FE analysis.

Zero horizontal displacements were applied to the nodes on the centre line to prevent the structure "flying", although the nodes were allowed to move vertically.

As was the case of the central loading condition, two base conditions were considered:

A "fixed" base where the nodes had zero horizontal and vertical displacements and

A "roller" base where the nodes had zero vertical displacements but were free to move horizontally except for the central node had zero vertical and horizontal displacements.

FE analyses were undertaken for each of the cases above under three different end loading conditions as indicated in Table 42.

Table 42: Loading conditions for FE analysis of End Loading

	Fixed Base	Roller Base
Single point load at the LH end	9000N Case A	9000N Case Jan-1
Three point loads closely distributed over LH end element	1500, 6000 and 1500N Case B	1500, 6000 and 1500N Case Jan-2
Forced three point vertical load on LH end element, nodes free to move in the x direction	1500, 6000 and 1500N Case D	1500, 6000 and 1500N Case Jan-4

Calculated Elastic Strain immediately below an end concentrated load

For comparison, the approximate stress immediately under the bearing plate from a manual elastic calculation assuming the 9000N load is applied over a bearing area of 75mm x 150mm

= 9000/(75x150) N/mm^2

= 0.8 N/mm^2

and the corresponding free vertical strain = 0.8/1500 = 0.00053 compression where Modulus of Elasticity is 1500 N/mm^2 although the actual strain is bound to be less due to the constraint of the adjacent material.

Average stress and average strain at the base of the wall from a vertical end load both ends

Stress at the base of the wall by simple manual elastic calculation assuming uniform distribution over the full width of the base

= 2x9000/150/1800 = 0.066 N/mm^2 compression

The corresponding average compressive strain = stress/Modulus of Elasticity = 0.066/1500 = 0.000044

Case A and Case Jan-1

These single point load analyses were carried out but they were not considered to adequately model the actual loading condition and are not reported here.

Case B (fixed base)

The load was considered to be applied through a flexible bearing plate 75 mm wide by 150 mm long (i.e. over the full wall thickness).

A 9000 N load was applied to the model at the end via 3 nodes, 1/6, 2/3, 1/6, i.e. 1500N, 6000N and 1500N at nodes number 609, 610 and 611 in the FE calculation in a similar way to the central concentrated load.

From the FE analysis the vertical compressive strain at mid block height at:

a) Left hand end of the block at the left hand end of the top course

= vertical deflection of node 558, minus vertical deflection of node 520, divided by the distance between the nodes (75mm)

= [(-0.170)-(-0.132)]/75compression = -0.038/75 = -0.00051

Equivalent vertical stress = 0.000285 x 1500 = 0.76 N/mm^2

b) 37.5mm in, strain = [(-0.165) – (-0.106)]/150 = -0.000787

Equivalent vertical stress = 0.000787 x 1500 = 1.18 N/mm^2

c) 75mm in, strain = [(-0.121)-(-0.102)]/75compression = -0.00025

Equivalent vertical stress = 0.38 N/mm^2

According to the FE analysis therefore, the average stress beneath the bearing plate is 0.77 N/mm^2 which is very close to the figure of 0.8 N/mm^2 from a simple elastic calculation. But neither of the values relates to the measured values from the laboratory tests.

Table 43: FE Base Reactions end loads Case B

-9.63E-02	7.56E-01	1	0.00E+00	6.43E+01
-1.79E+01	8.35E+00	2	0.00E+00	2.48E+02
-1.79E+01	6.82E+00	3	0.00E+00	1.20E+02
-5.40E+01	2.98E+01	4	0.00E+00	2.24E+02
-3.61E+01	2.30E+01	5	0.00E+00	1.04E+02
-9.10E+01	7.34E+01	6	0.00E+00	1.90E+02
-5.49E+01	5.05E+01	7	0.00E+00	8.54E+01
-1.29E+02	1.41E+02	8	0.00E+00	1.51E+02
-7.37E+01	9.02E+01	9	0.00E+00	6.53E+01
-1.66E+02	2.33E+02	10	0.00E+00	1.11E+02
-9.19E+01	1.43E+02	11	0.00E+00	4.62E+01
-1.99E+02	3.50E+02	12	0.00E+00	7.54E+01
-1.07E+02	2.08E+02	13	0.00E+00	2.92E+01
-2.25E+02	4.91E+02	14	0.00E+00	4.44E+01
-1.18E+02	2.84E+02	15	0.00E+00	1.52E+01
-2.38E+02	6.51E+02	16	0.00E+00	1.93E+01
-1.20E+02	3.67E+02	17	0.00E+00	4.12E+00
-2.29E+02	8.17E+02	18	0.00E+00	6.07E-02
-1.09E+02	4.50E+02	19	0.00E+00	-4.06E+00
-1.91E+02	9.72E+02	20	0.00E+00	-1.37E+01
-8.18E+01	5.21E+02	21	0.00E+00	-9.63E+00
-1.13E+02	1.09E+03	22	0.00E+00	-2.25E+01
-2.85E+01	5.59E+02	23	0.00E+00	-1.29E+01
2.26E+01	1.12E+03	24	0.00E+00	-2.68E+01
6.46E+01	3.20E+02	25	-6.35E+00	-6.95E+00
Horizontal	**Vertical**	**Node**	**Horizontal**	**Vertical**
Reactions at fixed base			**Reactions at roller base**	

In the case of the fixed base, the vertical reactions at Nodes 1 to 25 across the base (Table 43) are upwards and in total equal the applied load (9 kN) indicating that the load spread is right across the base area i.e. at an angle of spread to the direction of the load in excess of 45^0.

In the case of the roller base, the spread is less giving an angle of spread to the direction of load of.675/800 = Tan 41^0.

In Figure 49 the FE calculated horizontal displacements are shown diagrammatically for the right hand end of the wall for a fixed base in Case B. The figure is not to scale vertically and vertical displacements are not shown. Allowing for the lack of scale vertically, the double curvature in the horizontal displacements in the lower part of the wall can be seen. The maximum horizontal displacement occurs at the top of the wall (level of bold horizontal line). The vertical displacements are much larger than the horizontal displacements but similarly are at a maximum at the top of the wall

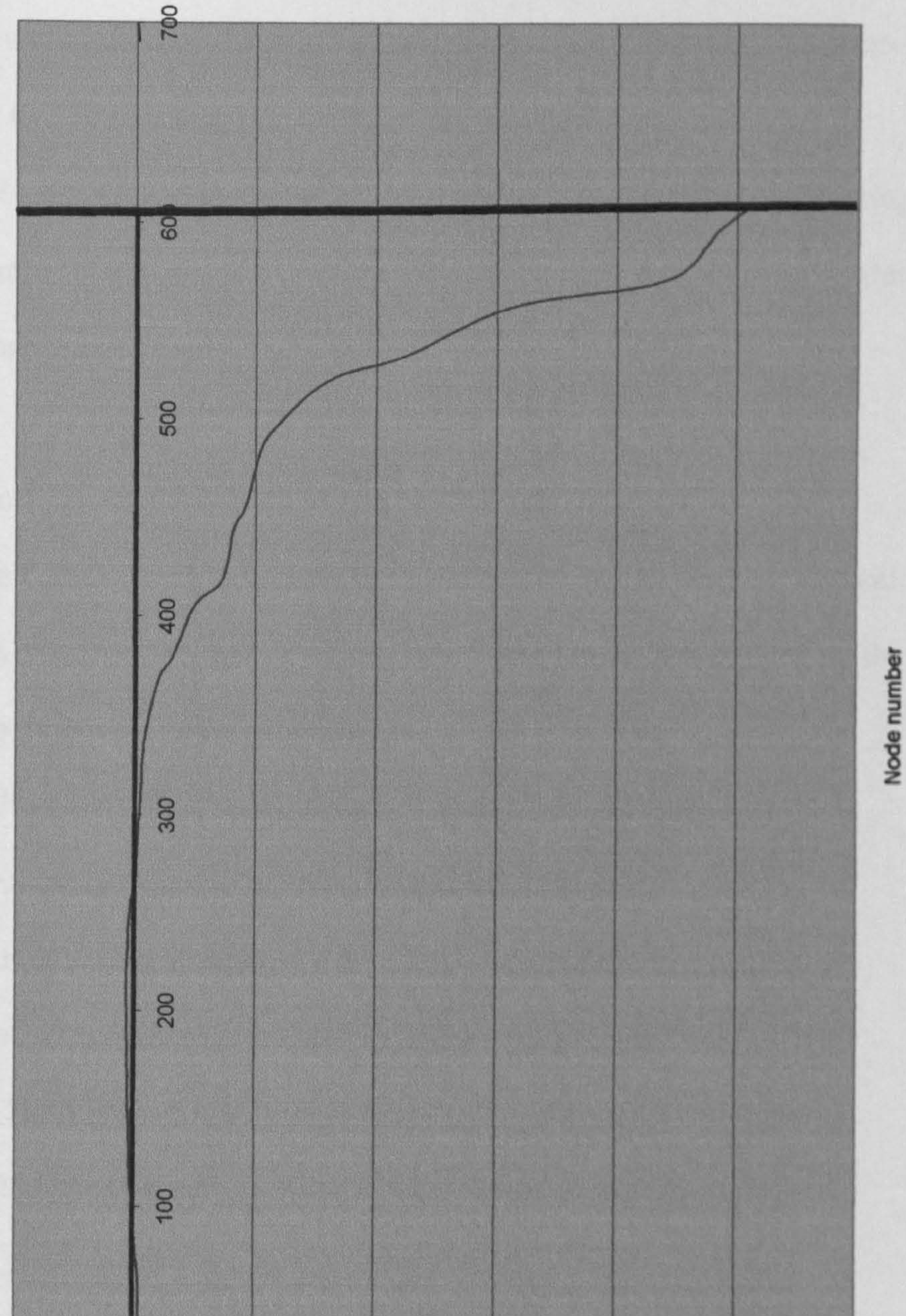


Figure 49: FE calculated horizontal movement (mm) at Right Hand end of wall

(fixed base case B)

Case Jan-2 (roller base)

The load was again considered to be applied through a flexible bearing plate 75 mm wide by 150 mm long (i.e. over the full wall thickness).

A 9000 N load was applied to the model at the end via 3 nodes, 1/6, 2/3, 1/6, i.e. nodes number 609, 610 and 611 in the FE calculation in a similar way to the central concentrated load.

The vertical compressive strain at mid block height at:

a) Left hand end of the block at the left hand end of the top course = vertical deflection of node 558, minus vertical deflection of node 520, divided by the distance between the nodes (75mm)

= [(-0.0378)-(-0.0258)]/75compression = -0.012/75 = -0.00016

Equivalent vertical stress = 0.00016 x 1500 = 0.24 N/mm2

b) 37.5mm(572,496) in strain = [(-0.0322) – (-0.0204)]/150 = -0.000157

Equivalent vertical stress = 0.000157 x 1500 = 0.24 N/mm2

c) 75mm(559,521) in = [(-0.0204)-(-0.0192)]/75compression = -0.00017

Equivalent vertical stress = 0.00017 x 1500 = 0.26 N/mm^2

Case D (fixed base)

The load was considered to be applied through a stiff bearing plate 75 mm wide by 150 mm long (i.e. over the full wall thickness).

A forced vertical deflection of 0.1mm was applied to the model at the left hand end via 3 nodes, 1/6, 2/3, 1/6, i.e. nodes number 6609, 610 and 611 so that they moved equally in the y direction. The nodes were free to move in the x direction.

The vertical compressive strain at mid block height at:

a) Left hand end of the block at the left hand end of the top course = vertical deflection of node 558, minus vertical deflection of node 520, divided by the distance between the nodes (75mm)

strain = -0.000172 compression

Equivalent vertical compressive stress = 0.000172 x 1500 = 0.258 N/mm^2

b) 37.5mm in from LH end (572,496)

strain = (-0.0318)/150 = -0.000212

Equivalent vertical stress = 0.00021 x 1500 = 0.31 N/mm^2

c) 75mm in from LH end strain (559,521)

strain = 0.0133/75compression = -0.000177

Figure 50 gives a diagrammatic indication of the magnitude and direction of principal stresses from the end loads and Figure 51 shows movement contours of the wall under end loads.

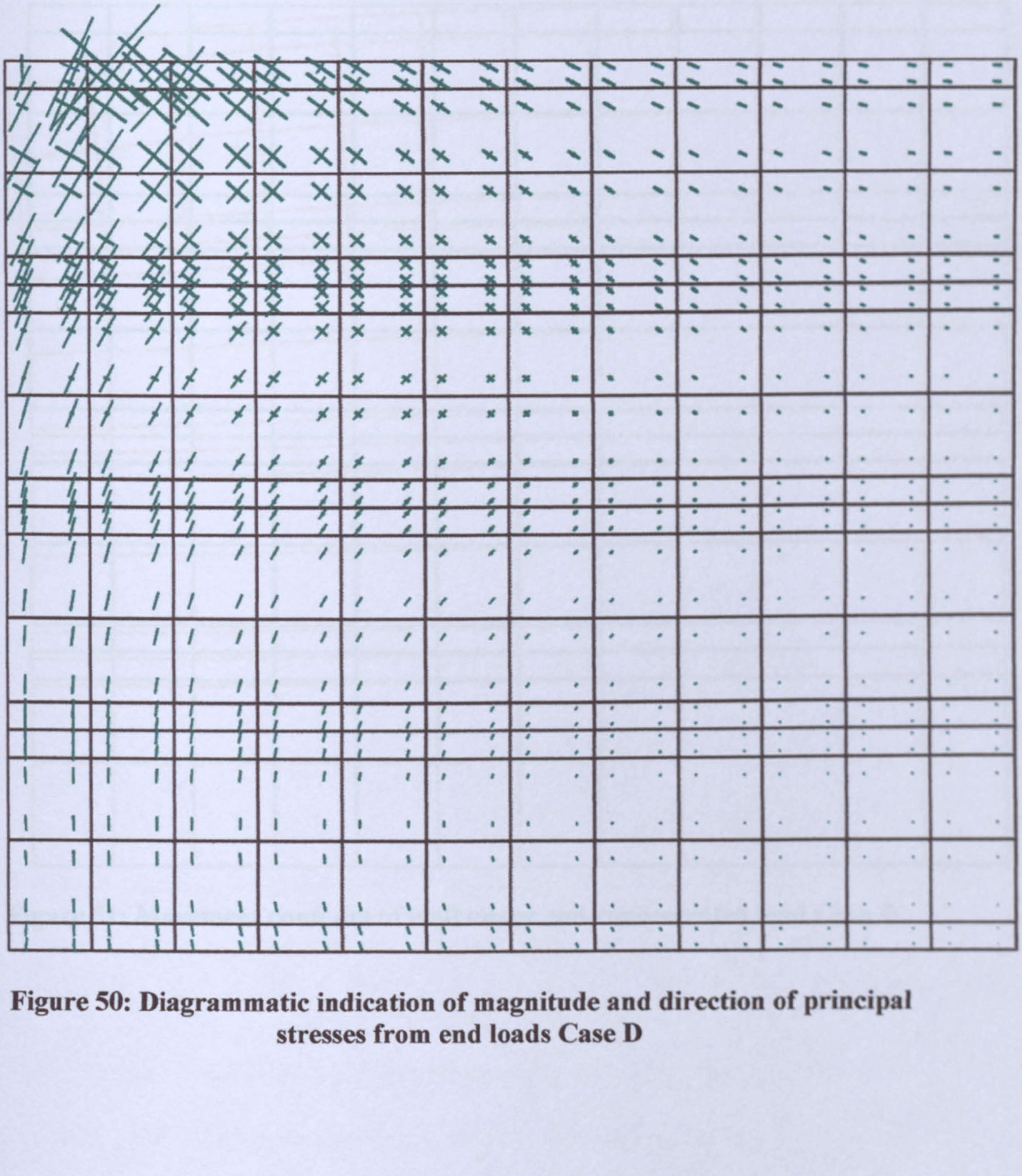


Figure 50: Diagrammatic indication of magnitude and direction of principal stresses from end loads Case D

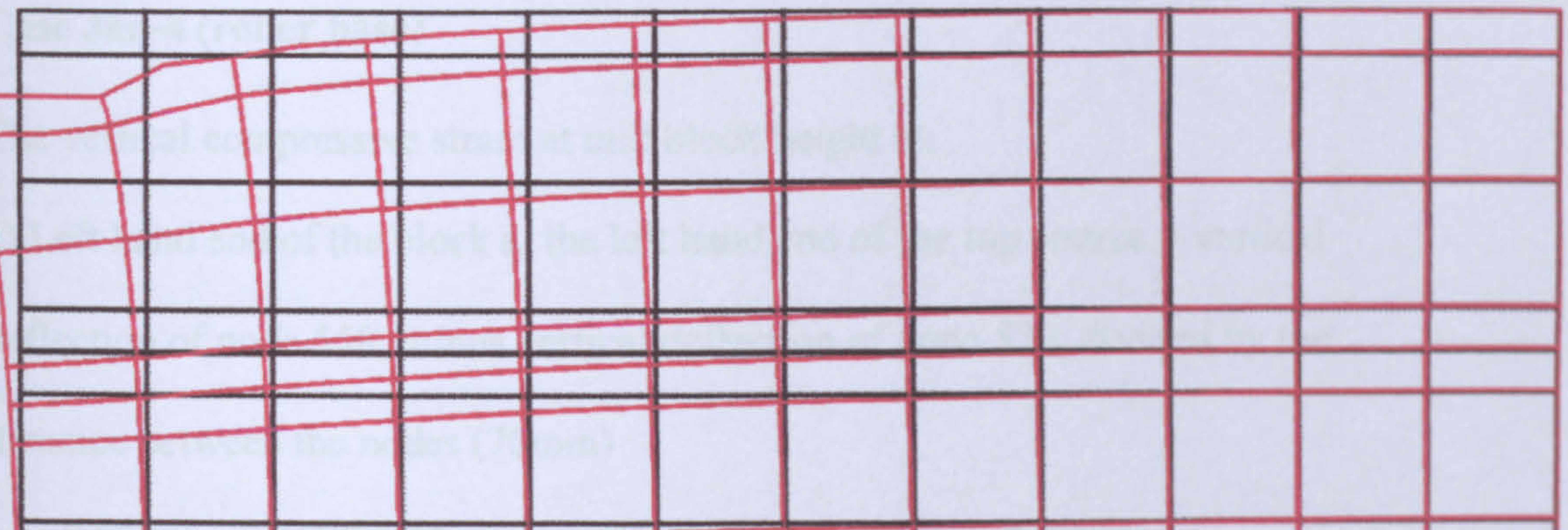


Figure 51: Movement contours of wall under end concentrated load Case D

Case Jan-4 (roller base)

The vertical compressive strain at mid block height at:

a) Left hand end of the block at the left hand end of the top course = vertical deflection of node 558, minus vertical deflection of node 520, divided by the distance between the nodes (75mm)

= [(-0.0929)-(-0.0778)]/75compression = -0.0151/75 = -0.000201

Equivalent vertical stress = 0.000201 x 1500 = 0.30 N/mm^2

b) 37.5mm in strain (572,496) = [(-0.0951) – (-0.0641)]/150 = -0.00021

Equivalent vertical stress = 0.000201 x 1500 = 0.30 N/mm^2

c) 75mm in strain (559,521) = [(-0.000756)-(-0.000625)]/75compression =
-0.00017

Equivalent vertical stress = 0.00017 x 1500 = 0.26 N/mm^2

From the laboratory testing Test 2(c), the Demec measured compressive strain on the end blocks at half block height in the top course from a 9000 N load each end applied through a rigid bearing plate 100 mm wide x 150 mm long (i.e. across the full wall thickness) the mean measured vertical compressive strain = 0.00015. This is slightly less than the calculated strains in Case D and Case Jan-4. The FE model with a forced vertical strain under a rigid plate appears to be the best model for the practical condition. The chosen forced strain of 0.1 mm in this case produces a very close match with the measured strains from the laboratory tests when the magnitude of the stress under the bearing plate used in the FE is reduced by 1/3rd to compensate for the different size of bearing plate.

Chapter 7. National Masonry Design Code Comparisons and Calculations

All of the national masonry design codes compared in this study are written for the structural design of masonry generally and those that mention autoclaved aerated concrete merely include some material specific parameters. Where the codes allow an enhancement of stress immediately beneath a concentrated load, it is always relative to the compressive strength of the masonry.

7.1 India

Indian Standard IS1905-1980 Code of Practice for Structural Safety of Buildings: Masonry Walls

The principle of allowing up to 50% overstress immediately under concentrated loads on masonry walls has been adopted widely around the world. For example, 25 years ago, the Indian Standard IS1905-1980 Code of Practice for Structural Safety of Buildings: Masonry Walls (66) stated "Masonry is capable of taking 50 percent greater stress if load is of concentrated nature." It also stated "If the bearing area under a load does not exceed one-third of the total cross-sectional area of the member supporting the load, the load may be termed as "concentrated load""

On the question of spread or dispersion, the code stated "the angle of dispersion of the loading shall be taken as not more than 30° from the direction of that loading". It continued:

"d) Assuming that concentrated loads bear on full thickness of the masonry element and are concentric, the length of the element to be considered as effective in resisting a concentrated load shall not exceed the centre to centre distance between the loads nor shall it exceed the width of the bearing plus four times the thickness of the masonry element.

e) Whenever, there is a concentrated load on masonry, it should be checked whether bearing stress is within permissible limit or not. If not, concrete bed block should be introduced below the load to bring down stress in masonry to safe limits. An increase in stress of 50 percent is not permitted for cross section of masonry below a bed block. It is assumed that angle of dispersion within the bed block is 45°.

NOTE – When bearing area under a load is greater than one-third but less than the full cross-sectional area of the member supporting the load, the permissible bearing stress may be interpolated between 1.0 and 1.5 times the allowable compressive stress."

7.2 Canada

The first edition of the National Standard of Canada CAN3-S304-M78 published in 1978 stated that "Where (plain) masonry supports concentrated loads, the contact pressure directly under the bearing shall not exceed the allowable compressive stress by more than 25 per cent."(81) By the time the 2005 version had been published the position had become much more complicated as is illustrated in the example below.

In the 2005 version (82), although an increase in bearing strength is included, the designer must assume triangular distribution of stress under the bearing for beams due to beam rotation at the support. Concentrated loads are assumed to disperse wholly within the wall section being considered, downward and outward from the outer edges of the bearing plate at an angle of 45° for solid unit brick masonry and fully grouted masonry. The dispersion shall not overlap the dispersion zone of another concentrated load for the purposes of calculation or extend beyond the end face of the wall or a movement joint or continuous vertical mortar joint in the wall unless the tying or bonding across the joint has been designed to transfer of compressive loads to the adjacent masonry.

Concentrated load calculation to Canadian Standards Association, S304.1-04 Design of Masonry Structures

"Bearing plate" is used to indicate either a bearing from a beam or column transmitted to the masonry below through a bearing plate or by direct surface contact.

The stress distribution on a bearing plate from any member that spans like a beam shall be triangular unless precautions are taken, such as a rocker plate, to obtain uniform bearing stress which would hardly be appropriate for a light material like Aircrete. It appears that the writers of the Canadian Code did not envision the use of masonry materials with low strength such as Aircrete. The maximum length of bearing for beams spanning in the plane of a wall if they are to be treated as concentrated loads should not exceed 300mm because rocker plates or similar are required "to locate the load physically at the mid-length of the bearing plate,"

The local factored bearing resistance (Br) of solid unit brick masonry and fully grouted masonry is calculated as in the following equations.

$B_r = K_1 A_{bp}\, \varphi m\, f'_m$ for rectangular stress distribution

or

$B_r = 1/2\ K_1 A_{bp}\ \varphi m\ f'_m$ for triangular stress distribution

where

enhancement factor $K_1 = 0.55[1+0.5a_1/l_2]/[A_{bp}/A_h]^{0.33}$

or $K_1 = 1.5 + a_1/l_2$

whichever is less,

but K_1 shall not be less than 1 .0

where

a_1 = the distance from the end of the wall or pier to the nearest edge of the bearing plate, mm

A_h = the effective area of dispersion of the concentrated load at mid-height of the wall, having the area of the bearing plate A_{bp} as the source of dispersion, mm^2

A_{bp} = area of the bearing plate, mm^2

l_2 = the length of the wall between ends and/or movement joints, mm

Eccentricity of the bearing load across the width of the wall shall be limited to not more than $1/5^{th}$ the wall thickness.

Consider a wall of the geometry of Wall 1, 2.0m high (h) x 1.86 m long (l_2) x 150 mm thick (t) in thin layer mortar (the test panel height 1.0 m represents half the storey height). The concentrated load is applied across the full thickness of the wall through a bearing plate 150 mm x 100 mm. Assume a 45^0 distribution and a rectangular stress distribution under the bearing.

Table 44: Wall 1 Concentrated load calculations to S304.1

	a_1, mm	h, mm	A_{bp}, mm^2	A_h, mm^2	t, mm	width, mm	$K_1 = 0.55[1+0.5a_1/l_2]/[A_{bp}/A_h]^{0.33}$	or $K_1 = 1.5 + a_1/l_2$
Concentrated load at he centre of the wall i.e. Wall 1 Test 1b)	880	2000	15,000	279,000	150	100	1.79*	1.97
Concentrated load at the end of the wall i.e.Wall1 Test 1 c)	0	2000	15,000	279,000	150	100	1.45**	1.5
K_1 when concentrated load is only 150mm from the end of the wall	150	2000	15,000	279,000	150	100	1.50	1.58

$K_1 = 0.55(1+0.5.880/1860)/(15000/279,000)^{0.33} = 0.68/0.38 = 1.79$* for a central load or $= 0.55/(15000/279,000)^{0.33} = 1.45$** for an end load

When the concentrated load is at the centre of the wall the maximum calculated enhancement which is permitted is 1.79 (i.e. the smaller of the two options) and when the concentrated load is at the end of the wall it is 1.45, assuming a rectangular stress distribution under the load. However, when the central concentrated load is applied such that the bearing stress is triangular, the maximum calculated enhancement is 0.9 and therefore the limit "shall not be less than 1.0" would apply to both the central and end loads.

The maximum spread along the length of the wall at mid height is limited to 1.86 m (i.e. the full width of the test wall) when the concentrated load is at the centre of the wall.

When the load is at the end of the wall and where a rectangular stress distribution applies, the enhancement is 1.45 and the spread is 1.1 m, which is unrestricted by the width of the wall.

The maximum enhancement is influenced by the ratio of the bearing area to the maximum spread area, which is dependent on the ratio of the height to the length of the wall when the width of the wall is less than the full potential spread. In addition, when there is more than one concentrated load, the spread is limited by the distance between them rather than letting them overlap using the principle of superposition. It can be seen that the formula containing the parameters which limit the enhancement is rather complicated even for the simplest case of a concentric concentrated load bearing on the full length of the wall. On the other hand an arbitrary cut off may dominate if it is less than the result of calculation to the formula e.g. when the load is at the end there is an arbitrary cut off limiting the enhancement to 1.5. On the other hand when the width of the bearing exceeds approximately 5% of the wall length (for a wall of constant thickness), the enhancement is less than 1.5 according to the formula.

When $a_1 = 0$, then $0.55/[A_{bp}/A_h]^{0.33} = 1.5$

$0.55/1.5 = [Abp/Ah]^{0.33}$

$Ah(0.367)^3 = Abp$

$0.05Ah < \text{or} = Abp$

Wall compressive strength calculation to S304.1

The masonry strength is taken as the strength obtained from five or more small masonry specimens (prism) tests using the particular masonry units. No prism tests according to the Canadian code test method have been done for this research so the masonry strengths for UK Aircrete units according to Edgell et al(33) are used. Therefore the measured Aircrete masonry strengths are equal to those calculated according to EC6.

Thus for Wall 1 Test 1a), The calculated vertical resistance per unit length is:

$\Phi\, t\, f_k = (1 - (2(0.75.2000/450/150))).150.1.44 = 211.2$ N/mm

7.3 USA

The National Bureaux of Standards 211 American Standard Building Code Requirements for Masonry in 1970 (A41.1) was a revision of the earlier 1944 edition where the stresses generally had been copied from the 1931 Modifications in Recommended Minimum Requirements for Masonry Wall construction. But in the case of concentrated loads, the enhancement of 50 per cent allowed in 1933 was omitted in the 1944 version and therefore again in the 1970 version because it was found that "smooth level horizontal joints" were practically unobtainable under field conditions". Currently masonry in the USA code appears to be covered by a plethora of other codes e.g. Building Officials and Code Administrators (BOCA), Universal Building Code (UBC), American Society of Civil Engineers (ASCE), American Society for Testing and Materials (ASTM), American National Standards Institute (AMSI) etc and the Masonry Standards Joint Committee (MSJC). As significant use of Aircrete in the USA is only just emerging, searches for information were not productive and the documents have not been referenced.

The allowable bearing stress in the 1997 Universal Building Code (UBC)(83) is given by:

"When a member bears on the full area of a masonry element, the allowable bearing stress F_{br} is:

$F_{br} = 0.26\, f'_m$

Although the term concentrated load is not used, when a member bears on one third or less of a masonry element, the allowable bearing stress F_{br} is:

$F_{br} = 0.38\, f'_m$

i.e. an enhancement of 0.38/0.26 = 1.46. The increase is permitted only when the least dimension between the edges of the loaded and unloaded areas is one fourth of the parallel side dimension of the loaded area. This accounts for confinement of the bearing area by surrounding masonry, which increases the bearing capacity of the wall. Interpolation is allowed between the two values.

In the Masonry Standards Joint Committee (MSJC) Code, the allowable bearing stress, F_{br}, is defined as a maximum of 0.25 f'_m However, an increase in capacity similar to that allowed in the 1997 UBC is allowed by the application of the concentrated vertical axial load over an increased area but where A_2 is not more than $2A_1$, where A_2 is the supporting surface wider than A_1 (bearing area) on all sides, or A_2 is the area of the lower base of the largest frustum of a right pyramid or cone having A_1 as upper base sloping at 45 degrees from the horizontal and wholly contained within the support." The maximum area allowed for a concentrated load = third wall plan area.

In the USA, because there are three codes which may be applicable to the design of masonry, there is not one way but the principal is to test prisms of the required masonry unit and mortar. Therefore the strength of the thin joint Aircrete masonry for this research is taken as that given by EC 6 based on the work of Edgell et al (33).

7.4 Europe

Eurocode 6 Design of masonry structures - Part 1

Eurocode 6- Part 1-1(65) states the design value of a concentrated vertical load, NSdc, applied to a masonry wall, shall be less than or equal to the design value of the vertical concentrated load resistance of the wall, NRdc, such that:

$N_{Sdc} \leq N_{Rdc}$.

When a wall, built with Group 1 masonry units (all Aircrete blocks are classified as Group 1 units) detailed in accordance with the Code and built with fully filled mortar joints, is subjected to a concentrated load, the design value of the vertical load resistance of the wall is given by:

$N_{Rdc} = \beta A_b f_d$

where:

$$\beta = \left(1 + 0{,}3\frac{\alpha_1}{h_c}\right)\left(1{,}5 - 1{,}1\frac{A_b}{A_{ef}}\right)$$

which should not be less than 1,0 nor taken to be greater than:

$1.25 + \alpha_1/2h_c$ or 1,5 whichever is the lesser

where:

β is an enhancement factor for concentrated loads;

α1 is the distance from the end of the wall to the nearer edge of the loaded area;

hc is the height of the wall to the level of the load;

Ab is the loaded area;

Aef is the effective area of bearing, i.e. lefm · t;

lefm is the effective length of the bearing as determined at the mid height of the wall or pier, obtained from a 30^0 spread from the direction of loading, (or the width of the wall if that is less)

t is the thickness of the wall, taking into account the depth of recesses in joints greater than 5 mm.

Ab/Aef is not to be taken greater as 0,45.

The concentrated load should bear on solid material of length equal to the required bearing length plus a length on each side of the bearing based on a 60° spread of load to the base of the solid material; for an end bearing the additional length is required on one side only.

For the load to be able to spread at least 60^0 to mid height, the minimum width of wall = 1.155h + bearing width to obtain maximum enhancement for a concentrated load which can spread in both directions and 0.58h + bearing width for an end load (which can only spread in one direction).

Calculations according to EC 6 are given in Table 45 when considering a wall of the geometry of Wall 1 from the test programme, i.e. 2.0m high (h) x 1.86 m long (l_2) x 150 mm thick (t) in thin layer mortar where the test wall height of 1.0 m is taken as representing half the storey height. The concentrated load is applied across the full thickness of the wall through a bearing plate 150 mm x 100 mm. The spread is taken to be 30^0 from the line of action of the load along the length of the wall.

Table 45: Wall 1 Concentrated load calculation to EC 6-Part 1-1

	α_1, mm	h_c, mm	A_b, mm^2	A_{ef}, mm^2	t, mm	Conc load width, mm	$\beta = \left(1 + 0{,}3\frac{\alpha_1}{h_c}\right)\left(1{,}5 - 1{,}1\frac{A_b}{A_{ef}}\right)$	$\beta = 1{,}25 + \frac{\alpha_1}{2h_c}$ or 1.5 whichever is the lesser
Concentrated load at the centre of the wall i.e. Wall 1 Test 1(b)	880	2000	15,000	279,000	150	100	1.63*	1.47**
Concentrated load at the end of the wall i.e.Wall1 Test 1 (c)	0	2000	15,000	279,000	150	100	1.44	1.25

e.g. for Test 1(b), $\beta = (1+0.3\text{x}880/2000)(1.5 - 1.1\text{x}15000/279000) = (1.132)(1.5\text{-}0.059) = 1.63$* or $\beta = 1.25 + \alpha 1/2h_c = 1.47$** which is the lesser.

7.4.1 Wall strength calculation to EC6

The characteristic compressive strength of Aircrete masonry (f_k) in general purpose mortar may be calculated as:

$$f_k = K\, f_b^{\alpha}\, f_m^{\beta}$$

where:

f_k is the characteristic compressive strength of the masonry, in N/mm^2

K is a constant

α, β are constants, normally 0.7 and 0.3

f_b is the normalised mean compressive strength of the units, in the direction of the applied action effect, in N/mm^2

f_m is the compressive strength of the mortar, in N/mm^2

K is 0.8

But the characteristic compressive strength of Aircrete masonry in thin bed mortar 0.5 mm to 3 mm thick may be calculated as:

$$f_k = 0.8\, f_b^{0.85}$$

Thus for the Aircrete masonry units used in this research f_k = 1.44 N/mm^2 for the 2.0 N/mm^2 compressive strength blocks and 1.92 N/mm^2 for the 2.8 N/mm^2 compressive strength blocks.

In calculating the vertical resistance of masonry walls, it may be assumed that plane sections remain plane and the tensile strength of masonry perpendicular to bed joints is zero.

The calculated vertical resistance of a single leaf wall per unit length:

$= \Phi\, t\, f_k$

where:

Φ is the capacity reduction factor, Φi, at the top or bottom of the wall, or Φm, in the middle of the wall, as appropriate, allowing for the effects of slenderness and eccentricity of loading,

t is the thickness of the wall

As we are considering the bearing stress at the top of the wall, Φi is appropriate.

At the top or bottom of the wall

$\Phi i = 1 - 2e_e/t$

where:

ee is equal to the initial eccentricity at the top of the wall

where ei is the initial eccentricity

ei, is assumed to be hef/450, where hef is the effective height of the wall which in the case of the walls in these tests is taken as 0.75h/450

t is the thickness of the wall.

Thus for Wall 1 Test 1a) The calculated vertical resistance per unit length is:

$\Phi\, t\, f_k = (1 - (2(0.75.2000/450/150))).150.1.44 = 211.2$ N/mm

Comparisons between the European EC 6 and the Canadian S304.1 Approaches to the Enhancement Factor

Comparisons between the Canadian Code and the EC 6 approaches to the enhancement factor calculation are particularly interesting because they each have adopted a complicated but different formula for the calculation.

The Canadian Code adopts the Page and Hendry formula (albeit with changed symbols). Each of the other codes examined have a much simpler approach.

The questions are raised when comparing EC 6 with S304.1-04: Is the difference in the two formulae justified?

Is the complication justified and do the formulae indicate an accuracy? which is not justified.

The two codes also adopt a different spread and is that related to the two different formulae?

S304.1-04 = $K_1 = 0.55[1+0.5a_1/l_2]/[A_{bp}/A_h]^{0.33}$

EC 6 = $\beta(1 + 0.3*\alpha_1/h_c)(1.5 - 1.1*A_b/A_{ef})$

Most of the parameters used are the same but different symbols are adopted.

Expressing the Canadian Code formula for the enhancement factor for a rectangular stress distribution using the same symbols as EC 6 gives:

$\beta = 0.55(1 + 0.5\alpha_1/l_2)/(Ab/Aef)^{0.33}$

Using the EC 6 notation the (arbitrary?) limits of EC 6 and S301 are compared in Table 46.

Table 46: Comparison of the enhancement width of bearing limits of EC 6 and S304.1

	EC 6-1	S304.1
Minimum β	1.0	1.0
Maximum β	the lesser of $1.25 + \alpha_1/2h_c$ or 1.5	the lesser of $0.55[1+0.5a_1/l_2]/[A_{bp}/A_h]^{0.33}$ or $1.5 + a_1/l_2$
Maximum A_b	$\leq 0.45\ A_{ef}$	Approximately 5% A_{ef} for an end load and 10% for a central load*

**Note: Linear interpolation is not possible due to the cubic term in the equation*

The maximum enhancement in EC 6-1 lies between 1.0 and 1.5. In S304.1 it lies between 1.0 and a figure potentially greater than 1.5 and usually is for concentrated loads remote from the ends of the wall. The enhancement is effectively halved when the concentrated load is the reaction from a beam.

7.5 Germany

7.5.1 DIN 1053

In Germany the following very simple expression applies:

$$\beta = 1+0.1*a_1/l_1 \leq 1.5$$

where "a_1" denotes the distance from the end of the wall to the nearer edge of the loaded area (similar to the notation in EC6) and "l_1" denotes the length of the loaded area i.e. width of bearing.

The approach is a model of simplicity where the maximum enhancement factor β is 1.5 and for a concentrated load at the end of the wall there is no enhancement.

Additionally, the loaded area "A_1" ($t*l_1$) must be less than or equal to twice the squared thickness of the wall (i.e. $A_1 <= 2*t^2$) and eccentricity of the loaded area from the centre line of the wall has to be smaller than one sixth of the thickness of the wall (i.e. $e \leq t/6$). Letting the load spread with an angle of 30^0 to the direction of the load, the normal compressive strength of the masonry must not be exceeded at the mid height of the wall.

The German rules are in every way the most conservative of all those considered.

Compare EC 6 and DIN 1053 approaches to the enhancement factor

The most obvious difference between DIN1053- Part 1 Masonry – Calculation and Execution, February 1990 and EC6 is the different structure of the formulae describing the stress enhancement factor β applicable to the compressive strength of the masonry immediately beneath the concentrated load.

A comparison of the two approaches reveals that in the EC6 the enhancement factor increases considerably more slowly with the distance of the loaded area from the end of the wall. Furthermore, the value of the enhancement factor of EC6 depends on the height of the wall as well, whereas the DIN1053 approach is independent of this parameter.

7.6 UK

7.6.1 BS5628 Part 1

Increased local stresses may be permitted beneath the bearing of a concentrated load of a purely local nature, such as beams, columns, lintels, etc. provided either that the element applying the load is sensibly rigid, or that a suitable spreader is introduced. The concentrated load may be assumed to be uniformly distributed over the area of the bearing and dispersed in two planes within a zone contained by lines extending downwards at 45° from the edges of the loaded area.

The effect of the local load combined with stresses due to other loads should be checked at the bearing, assuming a local design bearing strength of, $1.25 f_k/\gamma_m$ in the case of bearing type 1, (Figure 53) or $1.5 f_k / \gamma m$ in the case of bearing type 2 (Figure 54)

at a distance of 0.4 h below the bearing where

$\frac{\beta b t f_k}{\gamma_m}$ the design vertical load resistance should be calculated.

where

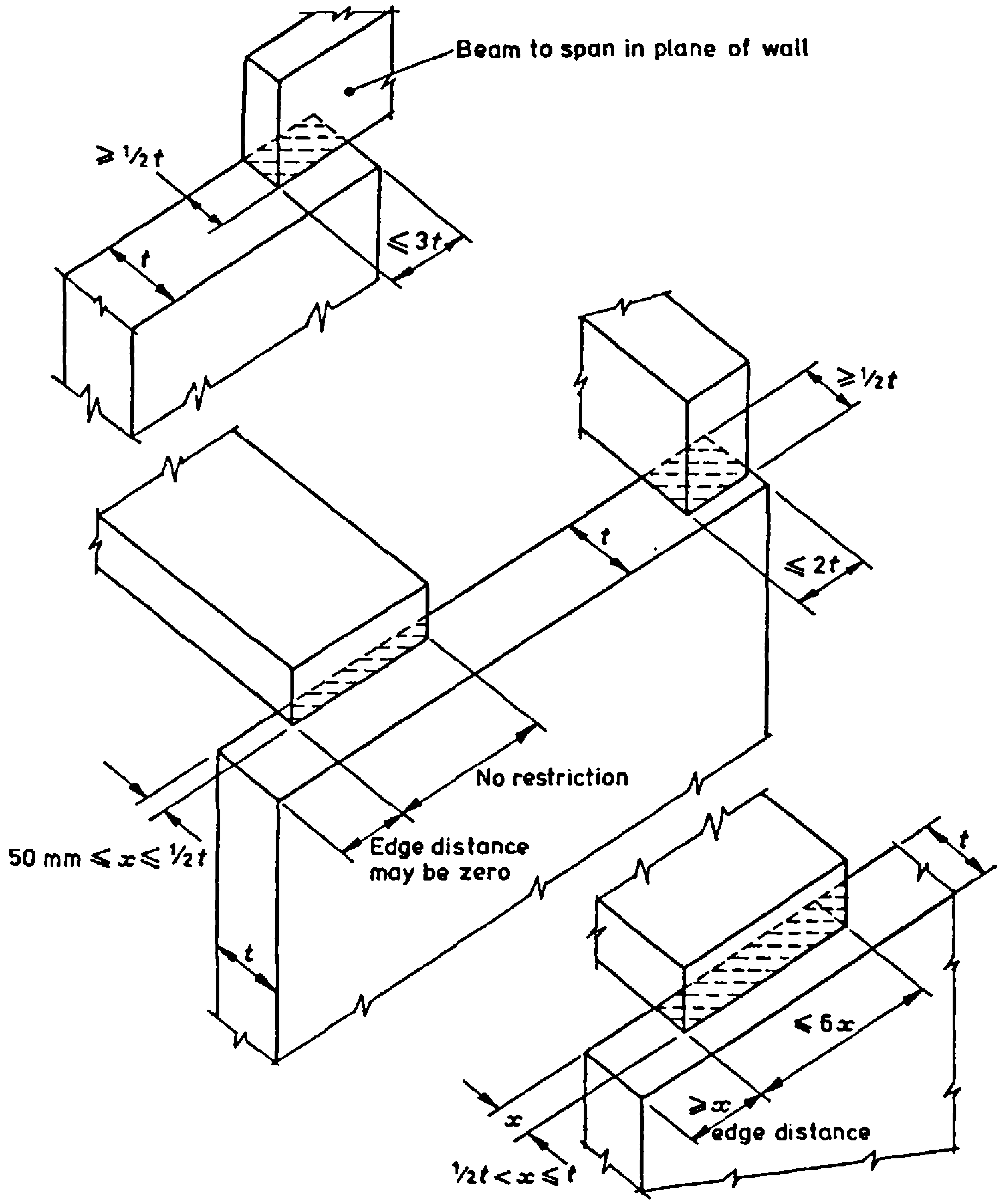
f_k is the characteristic strength of the masonry;

γ_m is the appropriate partial safety factor for the material which depends on the level of manufacturing control;

h is the clear height of the wall;

t is the thickness of the wall

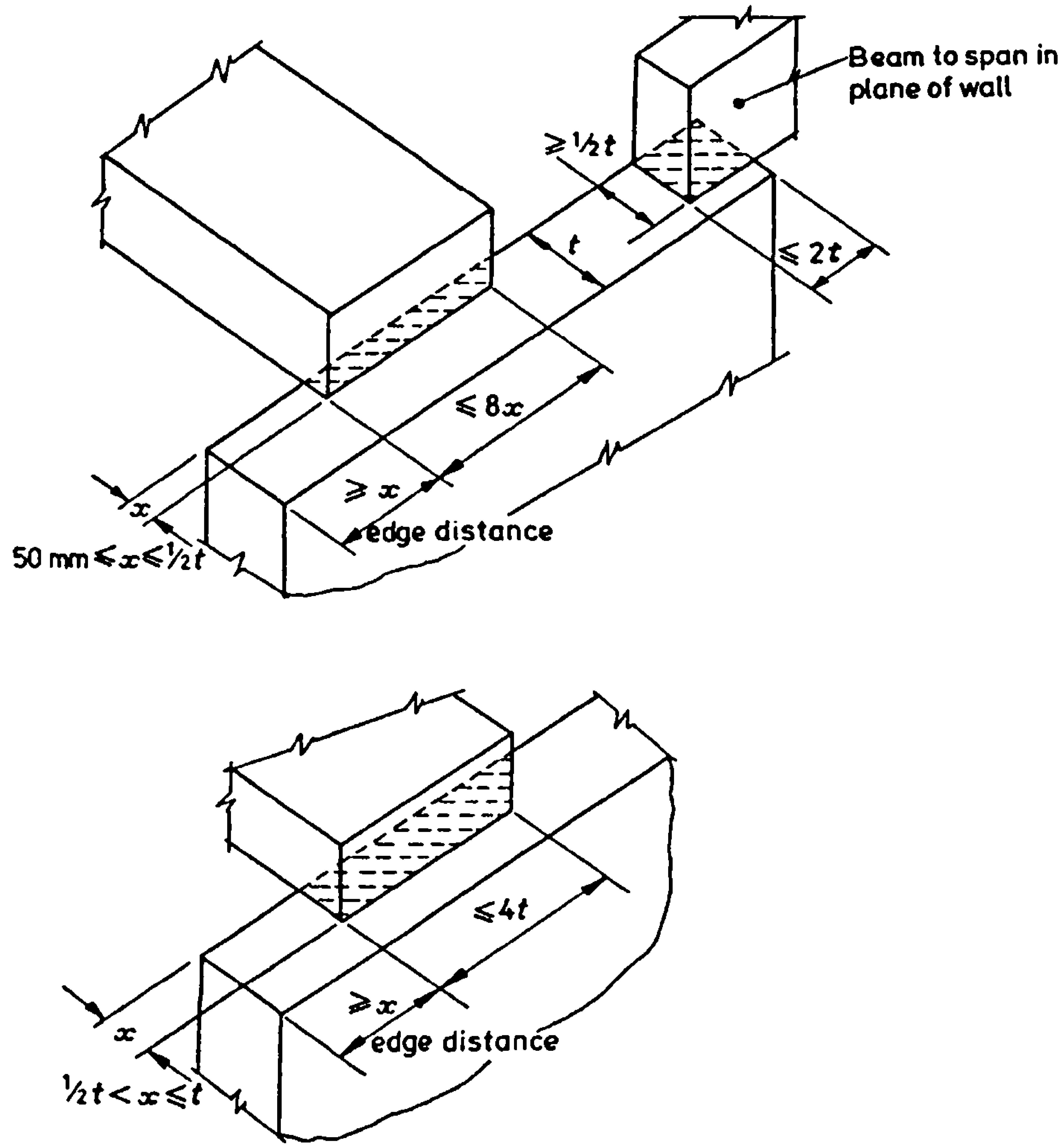
ß is the capacity reduction factor which depends on the slenderness ratio and the eccentricity at the top of the wall. (Note: this definition is different from the EC 6 definition of β as the Concentrated Load Enhancement Factor). When a concentrated load is applied eccentrically to the centreline of the thickness of the wall, BS 5628 assumes half the eccentricity is applied at the level where the stresses are calculated for the wall (0.4 of the height below the load) but the stresses become axial at the base of the wall (storey). EC 6 assumes there is a crossover moment at the base of the wall which might apply to walls with a small slenderness ratio i.e. a thick wall where there is no buckling. In the case of slender walls which can buckle and where the top is held in position, no moment is developed at the base of the wall and therefore the load is axial at that level.



Local design strength $\frac{1.25f_k}{\gamma_m}$

(a) Bearing type 1

Figure 50: Types of Bearing type 1 as in BS 5628 Part1



Local design strength $\frac{1.5 f_k}{\gamma_m}$

(b) Bearing type 2

Figure 51: Types of Bearing type 2 as in BS 5628 Part1

Wall strength Calculation to BS 5628 Part 1

t=150, h=2000, w= 1860 mm

The characteristic compressive strength of masonry constructed with Aircrete masonry unit is interpolated from Tables in the code. Strictly speaking the Tables do not cover compressive strengths below 2.8 N/mm^2. It is reasonable to suppose that if they did then the characteristic compressive strength of Aircrete masonry wall 150mm thick and units 200mm high with a compressive strength of 2.0 N/mm^2 in thin layer mortar would be 1.5 N/mm^2.

The wall vertical load resistance per unit length would be 225 N/mm as β the capacity reduction factor for the effects of slenderness and eccentricity is taken as 1.0 at that level.

7.6.2 BS 5628 Part 3

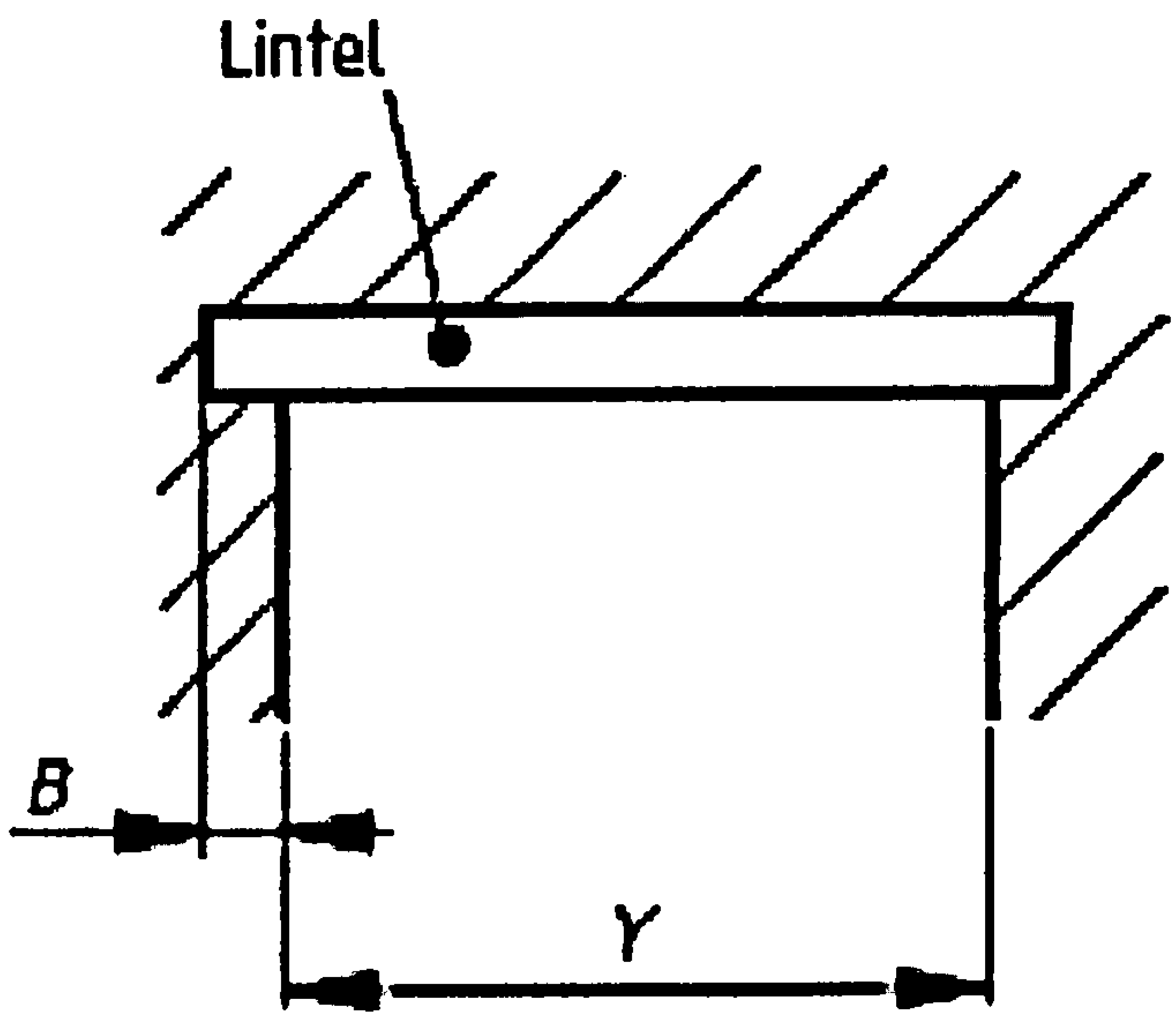
Where lintels are used, they should have bearings commensurate with the solidarity of the support and for the load for which they are designed and in any case not less than 100 mm in length. Where possible the masonry should be set out to provide a full length whole depth unit under a bearing.

7.6.3 BS 8103-2:2004 Structural design of low rise buildings —Part 2: Code of practice for masonry walls for housing

Guidance for Lintel bearings

Supported leaves should not overhang lintels by more than 10 mm.

The length of bearing for lintels (B) as illustrated in Figure 53, should be not less than the limiting dimensions given in Table 7 in the code (Table 47).



Key
1 Length of bearing, B
2 Clear span, Y

Figure 52: Lintel spanning in the plane of the wall

Table 7 — Minimum bearing length for lintels

Situation	Minimum length of bearing *B* mm		
	Lintel spanning in plane of (parallel to) the supporting wall (see Figure 13)	Lintel spanning at right angles to the support wall (see Figure 14)	
1. Lintel not supporting a concrete floor	150 (but see also 3 below)	100	
		where *d* is to be less than 200	where *d* is to be greater than or equal to 200, or no openings
2. Lintel supporting a concrete floor	$L/10$ or 150 whichever is the greater (but see also 3 below)	$L/10$ or 150 whichever is the greater	$L/15$ or 100 whichever is the greater
3. Thickness of supporting wall or width of lintel less than 100 mm	Value from 1 or 2 above multiplied by 100/wall thickness or 100/lintel width as appropriate (but see also 4 below)	—	
4. Where the mean strength of the unit used is 25 % or more than that indicated in Figure 12 and Table 5 or Table 6..	Value from 1, 2 or 3 above × 0.8, but not less than 150	—	

Table 47: Minimum bearing length for lintels

7.6.4 Approved Document A

1C25 Other loading conditions:

"a. Vertical loading on walls should be distributed. This may be assumed for concrete floor slabs, precast concrete floors, and timber floors designed in accordance with Section 1B, and where the bearing length for lintels is 150mm or greater. Where a lintel has a clear span of 1200mm or less the bearing length may be reduced to 100 mm."

7.7 Summary of enhancements and spreads in different design codes

Basically the assumption behind the longstanding guidance in UK codes and regulations is that there is a 45^0 spread with some restrictions versus the 60^0 embodied in EC6. It is not clear where the limitations come from; neither is their logic immediately obvious. They give the impression that they may be a bit arbitrary with the basic assumption of the angle of spread.

One thing both sets of rules have in common is that they ignore the bonding patterns of the masonry including that face size of the masonry units. In addition, although they distinguish between hollow and solid units, they do not make any distinction between different masonry materials or different strengths. The EC6 rules do though have some limitations regarding the relative position of the masonry unit immediately under the concentrated load.

Table 48: Comparison between the various codes treatment of concentrated loads on Aircrete Wall 1, (2.0 m high x 1.86 m long)

	Concentrated load within the length of the wall		Concentrated load at the end of the wall	
Code	**Maximum Enhancement**	**Degrees Spread from vertical**	**Maximum Enhancement**	**Degrees Spread (one way) from vertical**
Indian Standard IS1905	1.50	30[a]	1.50	30[a]
Canadian Standard CSA S304.1-04	1.97	45	1.5	45
USA	1.46?	45	?	45
German Standard DIN 1053	1.5	30	1.0	30
European Standard EC6	1.47	30	1.25	30
British Standard BS 5628	1.50	45	1.50[b]	45
BS 8103	unquantified	unquantified	unquantified	unquantified
Building Regulations AD A	unquantified	unquantified	unquantified	unquantified

a - not exceed the width of the bearing plus four times the thickness of the masonry element.
b - For concentrated load from beam spanning in the plane of the wall (bearing type 2) but
1.25 (for concentrated load at end of the wall from beam spanning perpendicular to the wall (bearing type 1)

Chapter 8. Discussion, Conclusions and Recommendations

8.1 Discussion

In masonry design, to allow an overstress immediately below a concentrated load on a solid masonry wall and to assume that the concentrated load spreads down the height of the wall at an angle to a level where it is then treated as a distributed load, is widely accepted. My research has shown that there are differences from the current codes which can be applied to low density Aircrete blockwork within those general assumptions.

It can be deduced that some of the differences from the current codes exposed by this research may equally apply to other forms of masonry and should be the subject of further research.

From the literature survey and from general enquiry, although there are relevant references about masonry, concentrated loads and Aircrete, it appears that nothing has been published previously which directly addresses the specific subject of concentrated loads on low density Aircrete masonry built in thin layer mortar.

The essence of the concept of permitting a higher intensity of stress immediately beneath a concentrated load bearing on masonry implies that the concentrated load in applied through a stiff bearing plate of limited size. Examples of what can constitute a concentrated load in practice include the load from a column bearing on the top of a wall and the end load from a beam bearing on a wall.

Although the ability of all heavier forms of solid masonry to safely withstand some degree of higher stress immediately beneath the bearing plate compared with the general maximum level of stress in the wall as a whole is generally accepted, the amount of enhancement permitted varies from country to country. The load in Test 1(a) was applied at the centre of the wall through a 100mm x 100mm bearing plate across the central 100 mm of the 150mm wall thickness. It could therefore spread in four directions i.e. across the thickness of the wall and along the length of the wall. At 25 mm below the top surface directly beneath the load in Test 1(a), the load could spread over 22,500mm^2 assuming a 45^0 spread. In Test 1(b) where the load was applied through a bearing plate 50% larger the spread could only be in two directions along the wall. It follows that the calculated stress (and therefore one would expect the elastic strain) below the load in Test 1(a) was similar to Test 1(b) at 25 mm beneath the load. Despite this, the measured strains in Test 1(b) were up to more than double those in Test 1(a) for an equivalent applied stress. Although Tests 1(a) and 1(b) were not taken to ultimate load, the tests taken to ultimate load in the remainder of the research indicate a 40% greater enhancement for concentrated loads which bear over the full thickness of the wall and can only spread in two directions (e.g. (6(d) and 6(e)) compared with those which can spread in four directions (e.g. Tests 6(a), 6(b) and 6(c)). This may be because there is greater containment immediately beneath the load as a result of the platen effect. These findings lead to the proposition that the percentage enhancement under a concentrated load applied over the full thickness of a low density Aircrete blockwork wall is greater than that for a concentrated load which does not bear over the full thickness of the wall.

The failure of low density Aircrete under a concentrated load is more likely to be by local crushing than is the case for denser masonry materials. This is in line with what would be expected as there is a greater proportion of air in the material.

The mode of failure of Aircrete blockwork in general purpose mortar appears to be crack initiation in a perpend joint at a mortar/block interface and subsequent propagation of the crack upward towards the concentrated load. This frequently leaves a wedge of material under the bearing due to the crack dividing and diverging to the two extremities of the applied load.

Low density Aircrete blockwork in thin layer mortar also fails by tensile splitting although it is much more clearly related to the position of the concentrated load and ignores well formed thin joints. In test 6(b) it was observed that the horizontal strains across the vertical mortar joints were compressive on both front and back faces of the test wall while the horizontal strains in the blocks were generally tensile front and back. This is not understood and is opposite to the situation with general purpose mortar joints. More research is required.

More research is also required on whether failure is due to the limiting tensile strain capacity of the blockwork assemblage and only indirectly related to the compressive strength of the Aircrete.

An overall observation, relevant to the suitability of this type of Aircrete block for general building construction, is that very few of the blocks used for the research reported in this thesis became damaged in handling despite their low density (350 kg/m^3) and low compressive strength (2.0 N/mm^2).

Although the average enhancement over the specified block compressive strengths was 1.59 for Tests 6(a) to 6(i), the maximum enhancement for an end load (1.54) was lower than the least enhancement (1.65) for a central load. The results overall indicate that a 50% enhancement for a "central" load is conservative, for a concentric end load it is probably reasonable, except for Test 2(c), and for an eccentric end load (Test 6(h) possibly excessive. This research has not examined the case when the eccentricity of a concentrated load is greater than $1/6^{th}$ of the wall thickness and it is recommended that no enhancement be allowed in such cases.

Eurocode 6 and the Canadian Code S304.1, both of which have been published recently, adopt complicated and different formulae to calculate the enhancement factor. The two formulae produce different results which cannot easily be compared. Due to the scatter of results on which they are based and their mutual incompatibility, it is suggested that nothing is gained by either of them over a simple approach which defines the limits of application of the enhancement factors.

This research has demonstrated that concentrated loads on low density Aircrete masonry can be afforded the same levels of enhancement given in the codes for heavier, stronger masonry.

This thesis contains recommendations later for a simple approach to the enhancement factor for the design of concentrated loads on low density Aircrete which are both economical and safe. This will be an advantage to the structural designer. By relating the enhancement to the block mean compressive strength rather than the masonry strength, the required size of the bearing can be determined in advance of the design of the masonry wall, where many factors have an influence, and the stress from the concentrated load, appropriately spread, added to the other stresses at mid height of the wall for that part of the calculation.

All of the National masonry design codes, except the Canadian Code S304.1, make the assumption that a concentrated load is distributed uniformly over the area of the bearing as it is applied to the top of the wall. This can only be even approximately valid when the concentrated load is applied through a stiff bearing bedded in a way that minimises the effect of any irregularities in the bearing surface of the masonry. It is seen that when this is not done premature failure can be initiated. In practice it is achieved by supporting the bearing plate on a bed of general purpose mortar. The type of mortar should be one normally recommended for use with the type of masonry. Other suitable means of overcoming any irregularities between the bearing face and the masonry can be used. Fine sand or fibre board packing was used for the laboratory wall tests but they would not be suitable for use in practice as they lack the necessary durability.

In the Canadian Code S304.1, the treatment of stress distribution under a concentrated load from any member which spans like a beam, is to take it as triangular unless precautions are taken to spread the stress uniformly. This has not been addressed in my research as most of the loads in the tests were applied by hydraulic loading jacks and have been treated as applying uniform stress. From the lintel tests, it appears that the stress distribution under a bearing plate must depend on the relative stiffness of the beam and the bearing. Aircrete has a low Modulus of Elasticity compared with steel, concrete and timber. Triangular stress distribution is unlikely to apply except with very long beams which deflect substantially. The stress distribution from a concentrated load from a beam bearing may also depend on whether the beam spans perpendicular to the wall or spans in the plane of the wall and this should be addressed in future research.

It is sensible to limit the maximum length of bearing which can be treated as a concentrated load and this is done in various ways in the various National codes. This research did not completely cover the question of the maximum area of bearing which can be treated as a concentrated load. Further research on this issue is required for all masonry types including low density Aircrete.

It is recognised good practice that the bonding arrangement of the masonry forming the wall generally should be detailed to allow the concentrated load to bear on a whole block (BS 5628 Part 3). Although this research indicates that this is probably not strictly necessary with well formed thin joint Aircrete blockwork, the compressive strength of the blockwork can be sensitive to any badly formed thin joints in the vicinity of the concentrated load.

From Test Series 3, it is seen that when using general purpose mortars, the ultimate load is not very sensitive to even a major rearrangement of the mortar joints provided they are of a similar type. Because of the superior adhesion and "thinness" of joints using thin layer mortars, it can be assumed that the same conclusion may be safely drawn.

It is generally accepted by the codes that concentrated loads spread at an angle down through the wall to either side of the concentrated load from the edge of the bearing. It is usually assumed that a uniform stress exists along the length of the wall within the angle of spread at any level. This would result in a step change at the end of the spread. My research, using both physical load testing and Finite Element Analysis, shows that this is not the case. Generally the strains are at maximum beneath the concentrated load and reduce as the load spreads outwards down the height of the wall, eventually becoming zero. The vertical strain distribution is triangular or a close approximation to triangular and the spread approximates to 45^0. This is likely to be true for all forms of masonry. Across half of the spread at mid height of the wall, the loading is in excess of the uniform load assumed in the codes. The increase ranges from nothing half way across the spread up to 100% under the concentrated load point. Therefore the strain at mid height directly below the concentrated load is double that which is assumed in the codes. This is most serious when the spread is taken as 45^0 and the only load on a wall is that from the concentrated load. When there is other loading on the wall to be added to, the total under-estimation is reduced. The consequences are also greater for end concentrated loads compared with those within the length of the wall. Care needs to be taken in wall design to consider the triangular nature of the loading particularly when there are openings in the lower part of the wall near the concentrated load.

The resulting vertical stress together with the stresses from all other loading are required to not to exceed the maximum stress allowed for the wall as a whole. This is normally checked at mid height for the purposes of design calculations, which seems sensible, although BS 5628 specifies that it be done at a level of 0.4 of the height of the storey below the concentrated load.

Although the angles of spread given in various national codes vary, they are generally either 30^0 or 45^0 to the direction of load. From this research, 45^0 or sometimes greater spread can be seen to be the more appropriate.

The FE analysis for the fixed base condition indicates that the load spread is over the full width of the chosen wall model and beyond and is rather more evenly spread than in the roller base case. The fixed base analysis is felt to simulate the practical situation of a wall bedded on a mortar bed more closely than a roller base analysis, although the spread of compressive strain over the full width of the base is similar. The FE model of end loading with a *forced vertical strain* under a rigid plate appears to be the best model for the practical condition and produced a very close match with the measured strains from the laboratory tests.

8.2 Conclusions

1. The percentage stress enhancement under a concentrated load applied over the full thickness of a low density Aircrete blockwork wall is greater than that for a concentrated load which does not bear over the full thickness of the wall.

2. Immediately under a concentrated load, a stress enhancement may be taken as over the block mean compressive strength, although in each of the National codes the enhancement is expressed in terms of the masonry strength.

3. Concentrated loads on low density Aircrete masonry wall spread at an angle of 45^0 or greater.

4. Although failure of low density Aircrete under concentrated loads is mostly by vertical splitting through the wall, unlike denser masonry materials, the failure is sometimes by local crushing under the load.

5. Immediately under a concentrated load, the ultimate compressive stress on a low density Aircrete block supporting the load can conservatively be taken as 50% higher than the mean compressive strength of the blocks:

a) when the load is axial or at an eccentricity not greater than 1/6th to the centre line of the wall thickness

and

b) the distance from the edge of the bearing the nearest end of the wall (or break or large opening) is sufficient to enable the load to spread at 45^0 to the level at which the strength of the wall is calculated (usually mid height).

6. Immediately under a concentrated load at the end of a wall, the ultimate compressive stress on a low density Aircrete block supporting the load can safely be taken as 50% higher than the mean compressive strength of the blocks:

a) when the load is axial to the centre line of the wall thickness

and

b) the distance from the edge of the bearing the other end of the wall (or break or large opening) is sufficient to enable the load to spread at 45^0 to the level at which the strength of the wall is calculated.

7. Immediately under a concentrated load at the end of a wall, the ultimate compressive stress on a low density Aircrete block supporting the load can safely be taken as 25% higher than the mean compressive strength of the blocks:

a) when the load is at an eccentricity of $1/6^{th}$ to the centre line of the wall thickness

and

b) the distance from the edge of the bearing the other end of the wall (or break or large opening) is sufficient to enable the load to spread at 45^0 to the level at which the strength of the wall is calculated.

For eccentricities between 0 and 1/6 linear interpolation from 1.50 to 1.25 should be allowed.

8. It follows from Conclusions 5 and 6 that, immediately under a concentrated load, an ultimate bearing stress up to 50% higher than the mean compressive strength of the low density Aircrete blocks supporting the load may be assumed when the load is axial to the centre line of the wall thickness.

9. For eccentricities between 0 and 1/6th of concentrated loads applied between the positions on the wall specified in Conclusion 5 and Conclusion 6, linear interpolation of the enhancement between 50% and 25% should be allowed.

8.3 Recommendations for adoption in design codes

1. The angle of spread under a concentrated load should be taken as 45^0 along the length of the Aircrete masonry and across the thickness where that is possible.

2. The applied stress should be calculated at mid height assuming the spread of stress from the concentrated load is triangular from the edges of the bearing plate i.e. the stress used under the concentrated load in the calculation should be double the stress from a uniform spread across the extent of the spread at mid height.

3. The recommendations from this research may only be taken to apply to a concentrated load on low density Aircrete thin joint blockwork when the length of bearing does not exceed the length of the block on which it bears up to a maximum of 600 mm or 25% of the wall length whichever is the lesser.

4. The enhancement allowed directly under a concentrated load on an Aircrete wall should be related to the mean compressive strength of the Aircrete unit for walls built with thin layer mortar rather than the (lower) compressive strength of the masonry. This will be of benefit for the structural designer who will want to design a wall of a given strength, density and thickness to meet other requirements of the building regulations such as acoustics, thermal, fire etc because he can immediately calculate the required areas of bearing for the concentrated loads the wall is required to support.

5. The maximum enhancement for axial concentrated loads both within the length of the wall and at the end of the wall should be 50% over the block mean compressive strength.

6. The maximum enhancement for concentrated loads with an eccentricity of one sixth the wall thickness at the end of the wall should be 25% over the block mean compressive strength. There should be no enhancement for greater eccentricities. For eccentric concentrated loads at the end of a wall where the eccentricity is less than one sixth, the enhancement can be extrapolated linearly between 50% at the centre line and 25%. No enhancement should be given for concentrated loads with an eccentricity greater than one sixth the wall thickness.

8.4 Recommendations for further work

Further work is required on low density Aircrete in thin layer mortar subjected to concentrated loads. It is suggested that the following be investigated using a combination of laboratory physical wall testing and Finite Element analysis.

1. Establish the effect of varying the width of bearings in relation to the wall thickness on the enhancement under the load.

2. Establish whether the enhancements and spreads from concentrated loads on Aircrete masonry apply equally across a range of wall thicknesses.

3. Establish the parameters that define the limits of plan area which constitute a concentrated load.

4. Investigate the stress distribution under the bearing when it is the reaction to a beam in the context of the relative stiffness of the beam and the bearing and whether the load is from a beam spanning perpendicular to or in the plane of the wall.

5. Investigate the effect of the lintel bearing being built in, especially for longer bearings where the encastré effect is likely to be significant.

6. Examine the effect of the bearing plate stiffness on the stresses in an Aircrete block beneath the bearing.

7. Investigate the possibility whether failure is due to limited tensile strain capacity and only indirectly related to the compressive strength of the Aircrete.

8. Investigate the effect of different degrees of eccentricity on the enhancement under the load and strain distribution at mid height of the wall.

9. Investigate why the horizontal strains across some mortar joints appear to be compressive (as measured on both front and back faces of the test wall) while the horizontal strains in the blocks were generally tensile both front and back.

10. Investigate the effect of openings in the wall on limiting enhancement according to the proximity of the concentrated load to the opening.

11. Investigate the effect of edge restraints on enhancement at the end(s) of the wall.

12. Investigate the propagation of cracking under concentrated loads at ultimate load for Aircrete built in general purpose mortar and thin layer mortar using high speed photography or other techniques which can capture the sequence of crack development.

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