STUDY REPORT

NO. 30 (1990)
STRUCTURAL PERFORMANCE OF CONSERVATORIES

C.A. Wade

PREFACE

This report on a project carried out at the Building Research Association of New Zealand describes an investigation into the structural performance of glazed domestic conservatories.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the advice and assistance of Messrs A.B. King and R. Holbrook of BRANZ and Mr R Seddon for helping with the industry survey. The assistance of those conservatory manufacturers who willingly shared information about their product is also warmly appreciated.

This report is intended primarily for designers and manufacturers of domestic conservatories but parts will be of interest to code writers and approving authorities.

STRUCTURAL PERFORMANCE OF CONSERVATORIES

BRANZ Study Report SR 30 C A Wade

REFERENCE

Wade, C.A. 1990. Structural Performance of Conservatories. Building Research Association of New Zealand Study Report SR 30 Judgeford, New Zealand.

KEYWORDS

From Construction Industry Thesaurus - BRANZ Edition: Aluminium; Bracing; Building Codes; Conservatories; Deformation; Glass; Glazing; Limit State Analysis; Loads; Performance Standards; Roofs; Safety; Strength; Structural Design; Testing; Walls; Windows;

ABSTRACT

The numbers of conservatories in New Zealand have increased markedly over the past five years or so, particularly the aluminium frame and glass variety which currently dominate the market. Designers and approving authorities have mainly attempted to apply structural performance criteria intended for light timber frame construction to these types of predominantly glazed structures with varying degrees of compromise and success.

The behaviour of most conservatories differs from that of traditional light timber frame buildings and this report investigates their structural performance by examining the ability of a typical aluminium-framed conservatory to resist in-plane racking loads, uniform face loads, impact loads and concentrated maintenance loads. Information collated from an industry survey is also presented in a summary form. The report concludes that many serviceability criteria adopted for timber frame buildings could be relaxed or removed with respect to common aluminium-framed and predominantly glazed conservatory constructions.

CONTENTS

APPENDIX A

TABLES

FIGURES

1. INTRODUCTION

Conservatories are arguably one of the fastest growing sectors of the residential construction industry. They have evolved from traditional greenhouses or glasshouses to be increasingly designed and used as parts of houses. For the purposes of this study, conservatories are considered to be parts of houses or attachments to houses and which include a substantial proportion of glazing in the walls and/or roof.

The aim of the study was to investigate the structural performance of domestic conservatories and the performance criteria applicable to their structural design.

Due to the lack of suitable published guidelines, performance criteria intended for light timber frame construction have been, by default, adopted for these predominantly glazed structures, with varying degrees of compromise and success. It was suspected, however, that as such criteria are frequently deflection controlled, they may well be inappropriate and lead to overly conservative estimates of the structural performance of these structures. The expectation was that different performance criteria should be applied to conservatories than to traditional timber frame constructions.

The consequences of failure of conservatories which, in the main, are intended to be used to enjoy the sun in a relatively pleasant and sheltered environment, should also be considered. Failure is generally of lesser consequence, in terms of monetary loss and risk to life, than failure of components in multi-storey public or commercial buildings. This does not mean compromising the safety of conservatory structures, but only that lower levels of performance, (mainly in some issues of serviceability) may be tolerated.

2. BACKGROUND AND INDUSTRY SURVEY

During 1988 an industry survey was conducted of domestic conservatory manufacturers and approving authorities throughout New Zealand. Subsequently, a report was produced by Seddon (1988) looking at problems with domestic conservatories in the Wellington area, which made use of some of the survey information collected. Further information was gathered in 1989 to complement the information already collected.

2.1 Manufacturers - Conservatory Design

Information of a sufficiently detailed nature was collected from seven manufacturers selected from throughout the country. This information was generally presented in the form of engineer's design calculations for their conservatories. Although there were only seven manufacturers, through various franchising arrangements throughout the country, they represented a significant proportion of the total market. The author estimates in excess of 80%. Six of the seven represented aluminium-framed constructions with the other being timber framed. These are the commonly used structural framing materials for conservatories at present in New Zealand, although uPVC products have recently appeared on the market.

2.1.1 Design Loads

A summary of the design loads used by each manufacturer is provided in Figures 1 to 5 for dead, uniform live, concentrated live (or maintenance), wind and snow loads respectively. Individual manufacturers are not identified but labelled A to G. Where no value for the load is given for a manufacturer it is because it was not considered, or was ignored, in the design. In most cases either NZS 4203 : 1984 New Zealand Standard Code of Practice for General Structural Design and Design Loadings for Buildings or NZS 4211 : 1985 New Zealand Standard Specification for Performance of Windows were used as the basis for the design. Earthquake loads were generally not considered as they were expected to be relatively low, in comparison with wind loads. This aspect will be discussed later in the report.

2.1.2 Mullion and Rafter Stiffness Properties

A summary of the stiffness properties of commonly used wall mullions and rafters is presented in Tables 1 and 2 respectively. For a given load, the value EI/L^3 is inversely proportional to the maximum deflection of the component. The moment of inertia values used related to the bending axis of interest.

Table 3 summarises deflection criteria used in the design of mullions and rafters for each manufacturer under the loads described in Figures 1 to 5. The criteria selected by the various manufacturers are generally similar, except for one (D) where design is for strength only.

2.1.3 Bracing

Resistance to in-plane racking loads or bracing is not required for conservatories built flush with a wall (in this instance, the conservatory may be more correctly described as a series of windows), or into the corner of a house provided that the remaining walls of the house can provide for the total amount of bracing required. However, conservatories which are built on to the side of a house will require bracing only in the wall parallel to the house. It is usually reasonable to assume that the house is able to carry any additional loads transmitted back via the conservatory end walls, given that these loads will be small in comparison to the total racking loads to be resisted by the house.

All of the previously mentioned manufacturers (A to G) of conservatories utilise the in-plane strength of the glass, to a greater or lesser degree, in providing bracing of the structure. Sometimes additional bracing, usually in the roof, is provided when the strength of the glass alone is not considered adequate. The planar, monoslope roof is able to provide some diaphragm action in transmitting racking forces back to an attached building, and in fact this type of roof is the most commonly used in practice. The smaller number of conservatories with a pitched roof or other complex geometrical shapes are generally unable to rely upon the roof contributing to the resistance of racking forces.

Presently there are no readily available procedures for manufacturers to determine the racking resistance of conservatory walls other than by physical testing (but see 2.1.4). The test method most commonly used, and specified by some approving authorities, is BRANZ Technical Paper P21 - A Wall Bracing Test and Evaluation Procedure (Cooney and Collins, 1979). This test method was developed specifically for use with light timber frame constructions which are within the scope of NZS 3604 : 1984 Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design. It was not intended for, and is inappropriate to, glazed conservatory type construction. The main reasons for this relate to the various deflection limits, number of replicates and factors of safety used being only applicable to light timber frame construction. Its use continues, due to the lack of a published alternative method, and the effect may be to produce overly conservative values for racking resistance of conservatories using a test which is unnecessarily expensive. The development and publication of a new test method for conservatory construction would be useful. The ability of a typical aluminium-framed and glazed conservatory to provide racking resistance will be considered later in the report.

2.1.4 Calculation of Strength Under Racking Loads

Racking strength is generally determined by test rather than by calculation, however, there have been some instances of attempts of the latter by New Zealand conservatory manufacturers. The approach adopted required the shear stress at rupture in the sealant to be determined in a small scale test. The allowable shear load per metre length was then calculated and used to estimate the required number of glazed panels necessary to resist the expected racking load.

2.1.5 Calculation of Strength Under Face Loads

Strength of conservatory walls and roofs are usually assessed by calculation. A uniformly distributed wind load is assumed to act on the face of the glass, a proportion of which is then transferred to the framing members. These members are then designed accordingly taking into account their section and material properties, applied loads and span. Strength of the glazing is usually checked for separately, using design charts from NZS 4223 : 1985 Code of Practice for Glazing in Buildings.

2.2 Approving Authorities

Nine approving authorities were surveyed in the Auckland, Wellington and Christchurch regions to gauge their attitude to, and treatment of, domestic conservatories. The main points to arise from the survey were:

a) Building permits were almost always required.

b) It is quite common for safety glazing material to be required in conservatory roofs.

c) A concentrated live load of 1 kN (from NZS 4203) applied to each rafter is sometimes specified.

d) A wall bracing test may be required, with BRANZ P21 the only one quoted, to demonstrate sufficient bracing capacity.

e) Reported instances of structural failure of conservatories were very low.

f) The main difficulty approving authorities appear to have, is in the area of conservatory definition, and whether they need to comply with ventilation and insulation requirements of the Bylaw. These aspects were, however, beyond the scope of this study.

3. DESCRIPTION OF EXPERIMENTAL WORK

3.1 Aim

This section describes a series of structural tests aimed at determining the characteristic behaviour of a typical aluminium-framed and glazed domestic conservatory under simulated wind, human impact and maintenance loads.

Due to the apparent dominance of aluminium-framed conservatories in the market place, the experimental part of this study has been confined to these types of structure, and hence some conclusions made in this report may not be applicable to other generic types of domestic conservatory e.g., timber and glass.

A characteristic of these aluminium-framed types of conservatory is the method of glazing used, the glazing material being compressed between polyvinyl chloride (PVC), neoprene or similar gaskets and wedges with an aluminium glazing or pressure bead used to hold the glass and gaskets in position. This detail allows some sliding to occur between the glazing and the frame, which can be utilised in dissipating energy, through friction, under extreme loading conditions resulting in the construction demonstrating relatively flexible and ductile, rather than brittle behaviour. Figure 6 shows a section through a typical glazing detail representative of this generic type of glazed construction.

The series of tests planned included: in-plane racking of the structure; face loading of the wall and roof sections under simulated positive wind pressure and wind suction respectively; distributed soft body impact tests to assess behaviour under accidental human impact; and finally, concentrated loading of the roof as might be expected during maintenance procedures.

3.2 General Description

The construction selected for testing comprised a nominal 4 m long x 2 m high front wall, made up of two frames, and coupled to a nominal 4 m long x 2.5 m wide planar roof section.

The wall section consisted of five bays, made up of a two-bay frame and a three-bay frame, with each bay separated by mullions at nominal 800 mm centres. It also included a horizontal vision rail at mid-height of each bay. The lower half of the wall was glazed with 5 mm clear annealed glass, while the upper half was glazed with 4 mm clear annealed glass. Each pane was held in place with PVC backing gaskets, wedges and clip-on aluminium glazing beads. The bottom edge of each pane rested on two rubber setting blocks, nominally 25 mm long x 8 mm wide x 4 mm high, and glued to the glazing bead. There were two opening sashes available for use in the wall section.

The roof section consisted of seven bays separated by rafters at nominal 585 mm centres. Two types of roof glazing were used, 6 mm laminated glass and 6 mm hollow core twin wall polycarbonate, held in place on three edges in the same manner as for the wall glazing except PVC wedges were not used over the full length of the polycarbonate sheets.

The principal rafters and wall mullions were T-shaped in section, extruded from aluminium alloy 6063 temper T5 with a theoretical EI value of 10162 Nm^2. Elevations of the roof and wall sections are shown in Figure 7.

3.3 In-plane Racking Strength

3.3.1 Description of Test Specimen

The conservatory wall and roof sections were assembled in general accordance with the manufacturer's instructions. The floor connection, shown in Figure 8, was modelled by nailing two ex 150 x 50 mm boundary joists to ex 100 x 50 timber cross pieces which were bolted to the concrete floor of the laboratory, simulating a rigid connection to the foundation. A nominal 100 mm wide x 20 mm thick sheet of flooring grade particle board was nailed over the top of the joists representing a typical domestic timber framed floor edge detail.

The conservatory front wall was fixed to the particle board floor and timber joists with 100 mm long x 5 mm diameter hexagonal head screws, one approximately 75 mm on either side of the frame verticals and another at mid-pane. Side walls were omitted from the test specimen.

The top edge of the conservatory roof was fixed to a special profiled aluminium fascia connector, shown in Figure 9, which was fixed to a rigidly held ex 200 x 50 mm timber fascia board with 50 mm long x 4 mm diameter hexagonal head screws, on either side of a rafter and another at an intermediate position, to represent the connection to an existing building. A general view of the construction is shown in Figure 10.

3.3.2 Equipment

The in-plane racking tests were performed using a 30 kN closed loop electro hydraulic ram bolted to the top corner of the wall and capable of providing both positive and negative displacements. Load was measured by means of a 100 kN loadcell with BS 1610 Grade 2 accuracy, displayed on a digital indicator. The load was applied at eaves level to the front wall.

Linear Voltage Displacement Transducers (LVDT's), which read within an accuracy of 0.25% of reading, were used to measure the displacement of different parts of the construction during testing. One of these LVDT's was positioned at the top corner of the wall, on the opposite side to the ram and was used to control the displacement cycles.

Test data was collected and recorded using analogue data-logging equipment.

3.3.3 Test Procedure

The test specimen was configured with a fully glazed front wall, with two closed sashes located in the two outer bays of the three-bay frame. The racking load, under displacement control, was applied to the top of the wall from one side using the hydraulic ram. The cyclic displacement regime for the series of non-destructive tests is shown in Table 4.

The test specimen was then configured with the roof fully glazed with hollow core twin wall polycarbonate and the front wall fully glazed with fixed panes. The racking was under displacement control and consisted of pushing the top of the wall out to an initial displacement of 10 mm; and then alternately reducing the displacement by 5 mm and increasing it by 15 mm (i.e., 0, 10, 5, 20, 15, 30 ...) until the ultimate strength (failure) of the conservatory was reached. The racking regime selected was essentially unidirectional reflecting the importance of wind rather than earthquake forces.

3.4 Face-load Strength of Wall Members

3.4.1 Description of Test Specimen

The three-bay section of wall frame from the test conservatory described earlier was selected for face-load testing in order to examine the stiffness of the wall construction and its behaviour under wind loads. The section of wall was placed in a horizontal position over a timber base with flexible nylon air bags sandwiched between the specimen and the base. The exterior face was on the underside, against the air bags, to simulate positive pressure on the wall. (Several preliminary tests were also conducted with the wall section turned over, to simulate wind suction). Only two adjacent bays of the three were glazed and these two bays were subjected to uniform pressure over their surfaces. This resulted in the central mullion, intermediate between the two glazed bays, being the only mullion fully loaded and therefore of particular interest.

The wall mullions were T-shaped and contained a fin on the exterior side of the wall which was shielded from contact by the air bags using a U-shaped channel section, thereby ensuring that the fin was free to buckle should that mode of failure occur. A view of the wall section is shown in Figure 11.

3.4.2 Equipment

The conservatory wall section was fixed to a timber sub-frame around the perimeter of the glazed sections of the wall. The fixings were along the wall head and sill edges only, these being rigidly held down using screws and additional clamps. Flexible nylon air bags were sandwiched between this assembly and a plywood base to enable pressure to be applied to the exterior face of the glazed wall. Examination of the pressure coefficients in 2/DZ 4203 : 1989 Code of Practice for General Structural Design and Design Loadings for Buildings, showed positive pressure on the exterior face to be the more severe condition.

A steel beam was independently supported above the test specimen to allow LVDT's to be positioned at various points over the central mullion and glazed areas to measure their deflection during loading.

The pressure was measured using a pressure transducer with an accuracy of ±0.002 kPa. The test data was collected and recorded using analogue data-logging equipment.

3.4.3 Test Procedure

The pressure, applied by the inflated air bags, was steadily increased until a strength failure occurred.

3.5 Face-load Strength of Roof Members

3.5.1 Description of the Test Specimen

Two types of roof glazing were used for the face-load tests; firstly, 6 mm laminated glass and later, 6 mm hollow core twin wall polycarbonate. Three adjacent roof bays (1.76 m total length) in the test conservatory were glazed for each case, allowing the two intermediate rafters to be effectively fully loaded.

3.5.2 Equipment

One end of each rafter was fixed to an aluminium box section, normally used at the eaves between the roof and the front wall. The other end was fixed to a profiled aluminium fascia connector typically used to fix the structure to an existing building. The fixings were generally in the manner specified by the manufacturer; except that only one screw was used (by the manufacturer's agent who installed the test specimen) to fix each rafter to the box section. The box section and fascia connector were then rigidly fastened to a timber sub-frame. The method of fastening assumed that the connections to the foundations and existing building would be adequate and these fixings were not modelled in this test. The side rafters were not fixed down along their length, being free to move representing three interior roof bays of a conservatory.

Face loads were applied to the underside of each, using the flexible nylon air bags as before, to simulate uniform wind suction on the exterior surface. LVDT's measured the displacement at various positions over the test specimen.

The test data was collected and recorded using analogue data-logging equipment. A view of the test specimen is shown in Figure 12.

3.5.3 Test Procedure

The air bag pressure was steadily increased until strength failure of a component occurred. This was done for both types of glazing.

3.6 Human Impact Resistance

3.6.1 Description of Test Specimen

Distributed soft body impact tests were conducted, both on the wall glazing and on the horizontal vision rail located at mid-height of the wall. Two different bays of the wall were used, when testing each of these two locations of impact.

The two-bay section of wall frame from the test conservatory described earlier was selected for use for the impact tests on the wall glazing. The frame was mounted in a vertical position and fixed to the floor in the same manner as that detailed for the in-plane racking test. In the absence of the roof section, a timber member was used to brace the top of the wall, in the out-of-plane direction.

One of the two bays in the frame was glazed with 4 mm clear annealed glass above the vision rail and the test pane of glass was mounted below. Each test pane measured 963 mm high x 755 mm wide (0.73 m^2). Five test panes of 5 mm clear annealed glass and six test panes of 4 mm clear annealed glass were selected for testing.

For the impact tests on the horizontal rail the three-bay section of wall was used. The frame was screw-fixed to timber members at the top and bottom. Bracing of the wall was provided with horizontal timber struts fixed to the top of the wall and in turn tied back to a 140 mm x 45 mm timber member fixed to and spanning between two steel columns 2225 mm apart. These columns were at a distance of 1000 mm behind the test frame. Only the central bay of the three was glazed, again using 4 mm annealed glass above the horizontal rail and 5 mm annealed glass below.

3.6.2 Equipment

For both configurations, a timber frame was constructed around the wall frame to support a canvas bag filled with dry sand (total mass 30.2 kg), with a maximum diameter of approximately 250 mm, and which was suspended from a stranded steel cable. The centre of mass of the bag was located opposite the appropriate point of impact. The orientation of the wall section was such that the impact was from the conservatory interior side. The end of the stranded steel cable was fixed to the timber frame such that the distance between the support point and the centre of mass of the sand bag was 1500 mm. A view of the apparatus and test specimen used for the glazing impact tests is shown in Figure 13.

3.6.3 Test Procedure

The sand bag was raised through a pre-determined vertical height corresponding to a known impact energy level and released to impact on the wall, after swinging as a pendulum through the drop height. For each of the five 5 mm panes and six of the 4 mm panes the energy level (and therefore the drop height) was incrementally increased by 15 J each time until the glass broke. The same procedure was followed for impact to the horizontal rail with the test repeated three times. The third test was abandoned part way through due to the deteriorated state of the conservatory wall after repeated impacts.

3.7 Maintenance Load on Rafters

3.7.1 Description of Test Specimen

The test specimen was the same as that used in the face-load tests of the conservatory roof except that only two of the roof bays were glazed with 6 mm laminated glass and a modification was made to the front wall end. The modification consisted of spanning the aluminium box section, on which the front end of each rafter was fixed, between two timber supports spaced 1032 mm apart. This arrangement was intended to simulate the eaves member acting as a lintel and being supported by two wall mullions.

A timber plank, measuring 1240 mm long x 145 mm wide x 47 mm thick and weighing 3.2 kg, was used to support the intended load. It was placed across three rafters, at right angles to the rafters, and at their approximate mid-span.

3.7.2 Equipment

The load was applied using a hydraulic jack connected to a load cell placed on the timber plank directly over the central rafter. A steel frame was used to provide the reaction against the jack.

As with the previous face-load tests, a steel beam was independently supported above the test specimen to allow LVDT's to be positioned at various points over the rafters and glazing. Due to the presence of the plank, the LVDT's recording the central deflection of the rafters could not be placed exactly at mid-span. They were placed to one side of the plank, offset by a distance of 168 mm from the mid-span of the rafters. Again, the test data was collected and recorded using analogue data-logging equipment. A view of the test specimen is shown in Figure 15.

3.7.3 Test Procedure

A load of 1 kN, 2 kN and 3 kN was applied by the jack and recorded by the load cell, and each time removed and the residual deflection measured. The load was then increased until a component failure occurred.

3.8 Failure Criteria

For the purposes of this study, the stucture was deemed to have reached its ultimate limit state (strength) when any one of the following occurred: the glass broke; the framing material yielded or buckled; the fixings failed either in withdrawal or shear; or components of the structure became detached, leading to an inability to continue carrying load. These criteria are applied to all the tests reported here.

4. TEST RESULTS AND DISCUSSION

4.1 In-plane Racking Strength

4.1.1 Maximum Expected Design Racking Load

The maximum design racking force, expected to be resisted by the front wall of a 2.4 m and 4.0 m wide conservatory was calculated to be 2.15 kN and 3.59 kN, respectively. A maximum dynamic wind pressure of 1561 Pa, applied to the end wall, was assumed. The basis for the calculation is contained in Appendix A.2.

4.1.2 Observed Behaviour in Racking Test to Destruction

In preparation for the in-plane racking test to destruction, the conservatory wall was fully glazed with fixed panes of clear annealed glass, 4 mm in the upper half and 5 mm in the lower half. The roof was glazed with 6 mm hollow core twin wall polycarbonate.

The load-deflection relationship at the eaves level of the test conservatory wall is shown in Figure 14. The behaviour appeared to be initially elastic in the range up to about 4 mm displacement. This was followed by the glass sliding between the backing gaskets and wedges in the frame, up to about 30 mm, where some of the PVC glazing wedges were observed to be working loose around the edges of the wall glazing. As only a few glazing wedges were involved it was unlikely to have had much effect on the racking resistance of the structure overall. At larger displacements, the glass appeared to be increasingly carrying load as it became restrained from further sliding by bearing against the framing members.

At an applied load of 2.3 kN, the displacement of the front wall at eaves level was recorded at 10 mm.

A maximum load of 8 kN was achieved with a displacement of 88 mm, at the top corner. Failure occurred during the following cycle, at a displacement of 93 mm and a lower load of 6.6 kN. The failure was in a glass pane in the lower half of the wall, and consisted of an arc-shaped crack near the top left corner of the pane. A view of the failure is shown in Figure 16.

After this observed failure, racking continued to a maximum displacement of 112 mm with a slight drop off in the load observed. The load was then removed completely and a residual displacement of 93 mm remained.

4.1.3 Relative Contributions of the Wall and Roof Plane

A series of non-destructive in-plane racking tests were carried out to investigate the relative contributions of the roof and wall plane to the overall resistance to in-plane racking loads of the conservatory structure. Both 6 mm laminated glass and 6 mm twin wall polycarbonate as roof glazing material were considered.

A load-deflection plot for the eaves level of the wall is shown in Figure 17 for a fully glazed wall - with no roof glazing (Fig 17a), and 6 mm laminated glass roof glazing (Fig 17b). A comparison of the applied loads at a displacement of 8 mm shows that the laminated roof glazing accounted for 29% of the total load of 3.5 kN with the fully glazed wall and skeletal roof frame accounting for the remaining 71%.

A load-deflection plot for the eaves level of the wall is shown in Figure 18 for a twin wall polycarbonate roof with - a fully glazed wall (Fig 18a), and no wall glazing (Fig 18b), respectively. A comparison of the applied loads at a displacement of 10 mm shows that the skeletal wall frame with twin wall polycarbonate roof accounted for approximately 21% of the total load of 2.4 kN with the wall glazing accounting for the remaining 79%.

A preliminary test of the skeletal aluminium frame only, showed it resisted about 0.1 kN at a displacement of 10 mm.

4.1.4 Discussion

The single test result of 8 kN maximum applied load did not permit the characteristic resistance (i.e., lower 5 percentile) of the conservatory to be determined as this relies on the spread of results from several tests. However, this value of 8 kN would typically be three to four times the maximum expected design racking load (from wind pressures of 1561 Pa applied to the end wall -see Section 4.1.1 and Appendix A.2). The conservatory thus demonstrated a considerable reserve in strength. It was also clear that the glass was capable of performing a structural function in distributing some of the in-plane load.

Typical serviceability limit states considered for traditional constructions include:

1. Deflection, vibration and movement of upper storeys;
2. Noise emission;
3. Visually offensive permanent set of the structure;
4. Loss of weathertightness due to glazing gaskets becoming dislodged;
5. The non-functioning of opening sashes and doors; and
6. Movement of the structure causing concern to occupants.

Deflection and vibration of upper storeys (1) is clearly not applicable to single-storey domestic conservatories while the level of noise emission (2) from the conservatory under test was considered to be relatively low and less than the background noise expected to accompany a severe windstorm.

Permanent set (3) is relatively easily corrected (by pushing the conservatory back to plumb) and for the tested conservatory occurred at loads above those expected in practice. Loss of weathertightness (4) is more difficult to assess without further testing. Sashes and doors will cease to function correctly (5) when their frames start to become restrained against further distortion by bearing against the surrounding structure. The final limit state (6) could be neglected as a glazed domestic conservatory is unlikely to be occupied during a severe windstorm.

It can therefore be argued that serviceability limit states are not of particular importance in the design of domestic conservatories of the generic type tested for in-plane racking loads in this study and that design for their ultimate limit state is the sole concern.

The displacement limitation of height/300 included in the BRANZ P21 test method (Cooney and Collins, 1979) at which the bracing rating of many light timber frame constructions are determined will penalise a neoprene (or similar) gasketed and fixed glazed, aluminium-framed panel construction and this criterion could be reasonably relaxed. The height/300 criterion is a serviceability control intended to limit plaster cracking, noise emission and lateral vibration of upper storeys in traditional timber framed construction (Cooney, 1989). It is clearly inappropriate for conservatories.

It was also established, that for the test conservatory, the contribution of the roof plane to the overall amount of resistance to in-plane racking loads was not substantial, even though the area of roof glazing exceeded that of the wall glazing. The contribution was less than one-third (29%) for the 6 mm laminated glass roof glazing and less than one-fifth (17%) for the 6 mm twin wall polycarbonate roof glazing. The contribution of the skeletal frame alone was negligible. The extent that the roof contributes to the lateral resistance is likely to partially depend on the aspect ratio (height/width) of the glazing elements, increasing as this aspect ratio decreases. The tested roof glazing possessed an aspect ratio of 0.25 and in practice will not often exceed about 0.5 for a relatively narrow conservatory. In contrast the wall glazing consisted of relatively squat glass panels with a higher aspect ratio of 0.8, with this geometry proving more effective in providing racking resistance.

4.2 Face-load Strength of Wall Members

4.2.1 Ultimate Design Wind Forces

The ultimate design wind force expected to be resisted by a conservatory wall was calculated to be 1561 Pa (positive pressure on the exterior face). The basis for the calculation is contained in Appendix A.3.

4.2.2 Serviceability Design Wind Forces

The serviceability design force expected to be resisted by a conservatory wall was calculated to be 809 Pa (positive pressure on the exterior face). The basis for the calculation is contained in Appendix A.3.

4.2.3 Observed Behaviour

A pressure - deflection plot is shown in Figure 19 for the movement of the middle of the central mullion, relative to its end. The pressure was corrected by subtracting the estimated weight of the wall (0.13 kPa) from the reading. As the pressure was increased, there were no signs of distress observed, until lateral buckling of the fin on the T-shaped central mullion (flexural compression flange) occurred. At this time the pressure was 3.3 kPa and the maximum mullion displacement relative to its end was 52 mm. This was accompanied by the two 5 mm glass panes shattering, but the two 4 mm panes remaining intact. The breaking of the more rigid thicker panes was attributed to the higher stresses generated in the thicker panes of glass at a given mullion displacement. A view of the failure is shown in Figure 20.

Preliminary tests of the wall under simulated wind suction resulted in 1.0 kPa pressure applied without failure or signs of distress in the wall. As positive pressure was expected to be a more severe case, further testing of the wall under wind suction was not pursued.

4.2.4 Discussion

Adequate reserve in strength was demonstrated by the conservatory wall under positive face loads, failing at 3.3 kPa. This failure pressure is approximately twice the maximum expected design ultimate wind pressure of 1.6 kPa derived in Appendix 2.

The measured deformation of a typical glazed pane and surrounding frame components in the lower half of the wall is illustrated in Figure 21 at applied pressures of 0 kPa (initial), 1.6 kPa (ultimate design wind pressure), and 3.3 kPa (failure).

In NZS 4211 : 1985 Specification for Performance of Windows there is a deflection limit of span/180 placed on the movement of a window mullion relative to its end. This limit is commonly used for the design of conservatory walls also. In the face load test of the wall, the mullion had moved 11 mm (span/180) at a pressure of 0.92 kPa. Thus walls of this rigidity may be expected to satisfy the serviceability limit state implicit in this criterion.

Overseas standards also control the maximum movement of windows. AS 2047 : 1977 Australian Standard Specification for Aluminium Windows for Buildings specifies a deflection test where no structural member in a completely assembled and glazed window may deflect by an amount greater than span/180 for windows and span/150 for sliding doors for residential applications.

For the purpose of determining glass thickness to resist wind loads, the deflection of each edge of four-edge fully supported glass is recommended by BS 6262 : 1982 Code of Practice for Glazing for Buildings to be limited to span/125 for single glazing.

The reasons for limiting the movement of window mullions out-of-plane, under face loads, are mainly for serviceability; however, there are good arguments for having stricter controls for windows than for domestic conservatory walls. Firstly, occupants in typical buildings, including multi-storey office buildings, will continue to occupy those buildings in the event of severe windstorms. Also, in those buildings during windstorms, the movement of window glass, apparently amplified by reflections, may cause occupants to fear for their safety.

In contrast, domestic conservatories are much more likely to be vacated in windstorm conditions, in which case the movement of glass in the walls is not so important, as long as it remains safe and does not break. The consequences of failure should also be considered. Much more serious than the failure of a domestic conservatory located close to the ground is failure of a window at a great height above ground level in a public place; there is the danger of jagged glass pieces falling from height to street level and endangering pedestrians.

Potential problems relating to loss in weathertightness were not able to be assessed during the face-load test. However, the level of noise emission was considered to be relatively low.

Although limits on maximum deflection (on the basis of this study) appear to be unwarranted, with design for strength considerations apparently adequate, there may well be practical reasons why manufacturers may elect to use stiffer components, one reason being to maintain consistency in the design of window joinery and conservatory walls allowing identical components for each to be used.

4.3 Face-load Strength of Roof Members

4.3.1 Ultimate Design Wind Forces

The ultimate design wind force expected to be resisted by a conservatory roof was calculated to be 1405 Pa (wind suction on the exterior face). The basis for the calculation is contained in Appendix A.3.

4.3.2 Serviceability Design Wind Forces

The serviceability design wind force expected to be resisted by a conservatory roof was calculated to be 728 Pa (wind suction on the exterior face). The basis for the calculation is contained in Appendix A.3.

4.3.3 Test of 6 mm Laminated Glass

A pressure - deflection plot is shown in Figure 22 for the movement of the middle of the rafter relative to its end. The pressure was corrected by subtracting the estimated weight of the roof (0.15 kPa) from the reading. The mode of failure observed was the withdrawal of a 25 mm x 10g stainless steel screw connecting one of the rafters to a box section at the eaves. This occurred at a pressure of 4.4 kPa. The maximum deflection of the rafter relative to its ends was 78 mm (span/29). As a result of the failure the rafter was observed to spring free and cause the laminated glass to crack. It was noted that only one screw was used to fix the end of each rafter to the eaves box section, as described in section 3.5.2, contrary to the two screws specified in the manufacturer's design information. The asymmetrical location of the fixing, to one side of the fin, caused the rafter to twist during the test. A view of the failure is shown in Figure 23.

4.3.4 Test of 6 mm Hollow Core Twin Wall Polycarbonate

A pressure - deflection plot is shown in Figure 24 for the movement of the middle of the rafter relative to its end. The mode of failure observed was a long edge of the polycarbonate glazing popping out and separating from the aluminium rafter in the mid-span area and spreading over a distance of some 1.5 m. There was no damage to the conservatory roof other than slight creasing of the polycarbonate glazing. The failure occurred at 1.0 kPa with a rafter deflection relative to its ends of 17 mm. The centre of the polycarbonate glazing was observed to lift higher than the adjacent rafter (and therefore to its own edge) by approximately 35 mm and thus contributed to the withdrawal of its edges from the rafter. A view of the failure is shown in Figure 25.

4.3.5 Discussion

Again, adequate reserve in strength was demonstrated in the face-load test on the conservatory roof, as previously for the wall under in-plane racking and face loads. The strength failure pressure of 4.4 kPa for the laminated glass roof compares favourably with the ultimate design wind pressure of 1.4 kPa indicating that considerable reserve strength was available.

The previous discussion on the relevance of various serviceability limits is also applicable to the movement of rafters and roof glazing under wind loads. There is probably even less of an argument for limiting the movement of conservatory roofs than there is for conservatory walls, as the visual cues of roof movements compared with walls are generally less obvious. Occupants are more aware of wall movement and reflections from walls, than they are of roof movement. There is very little published information on the effects of glass movement on the senses of building occupants and the quantification of how much movement is acceptable. This is an area where further social science research would be useful.

The hollow core twin wall polycarbonate proved to be a quite flexible glazing material under wind suction. In contrast to the laminated glass, which moved in much the same way as the rafters, the polycarbonate tended to move to a greater extent than the rafters resulting in the ballooning of each glazed bay. At the point of failure of this type of glazing, which was due to this ballooning effect, the movement of the rafter was only 17 mm relative to its end or about span/140. However, it is apparent that the amount of edge cover is crucial to the satisfactory performance of this type of comparatively flexible glazing under face loads and that failure appears to be related more to the properties of the glazing than to movement of the rafter. The ultimate design wind pressure (1.4 kPa) was not resisted by this particular system which failed at 1 kPa.

4.4 Human Impact Resistance

4.4.1 Observed Behaviour

The test results for impact on the glass, showing the energy level at which glass failure occurred for the 4 mm thick and 5 mm thick clear annealed glass panes mounted in a section of conservatory wall are presented in Table 5.

The results for impact on the horizontal rail are presented in Table 6. During all the tests the wall was observed to noticeably flex during the impacts.

4.4.2 Discussion

The aim of the distributed soft body impact tests was mainly to assess the potential for injury to a conservatory occupant in the event of a collision between the occupant and a predominantly glazed wall from the interior side.

The test results indicated that for impact at the centre of the glass panes, all the 5 mm glass broke at an energy levél greater than 90 J, while all the 4 mm glass broke at a level less or equal to 90 J. NZS 4223 discusses the use of safety glazing materials in locations where glass is likely to be subjected to human impact. Safety glazing materials are considered to be those which comply with AS 2208 : 1978 Safety Glazing Materials for Use in Buildings. 90 J is an arbitrary level set in AS 2208 as a breakage performance requirement for a Grade B safety glazing material. It was established as being practically related to those situations where the glazing is around a confined space and a limited acceleration path is available. It equates approximately to a 45 kg child moving at a speed of 2 m/s (a fast walk) - from AS 2208.

For the test conservatory examined in this investigation, impact at the centre of the glass panes proved more critical than impact on the horizontal rail, for both the 4 mm glass and 5 mm glass thicknesses. This may not necessarily be the case for other designs, as the stiffness of the rail and frame is very important in determining the performance of the wall under human impact loads. At least both these locations, rail and glass, would need to be considered in the impact testing of other conservatory designs.

In general, soft body impact test results will depend upon the actual stress level in the glass. According to Toakley (1977), the breaking stress level of glass is dependent upon annealing procedures, surface quality, panel size and support conditions. It also depends upon the type and duration of loading. During these impact tests, the aluminium frame was observed to flex during the impact, although measurements were not taken. This behaviour is a desirable feature as it increases the absorption capacity of the frame and supports, resulting in an apparent increase in the energy level required to break the glass. The amount of flex in the frame is in turn dependent upon the stiffness of the wall mullions, generally decreasing as the stiffness of the frame components increase. It is therefore very important to closely model the support conditions for the frame and glazing, so their stiffness is representative of that to be found in practice.

The safety provisions in NZS 4223 are based on the premise that the risk of injury due to glass impact can be reduced by controlling the following factors: the strength of the glass, the fracture characteristics of the glass, its maximum area and location. The standard identifies locations where glass is more likely to be subjected to human impact and special provisions are required at these locations. With regard to typical conservatories, the locations of interest are: framed glass doors, side panels adjacent to framed glass doors (on the assumption they may be mistaken for the door) and low level glazing, identified by the standard as that within 200 mm of the finished floor level. The previous discussion has related to wall glazing, with respect to roof glazing, NZS 4223 says "Where human safety is of concern, Grade A safety glazing shall be used on all sloped and overhead glazing". Grade A safety glazing material is required to resist breakage by a soft body at an impact energy of 135 J by AS 2208. Human safety will only be of concern for overhead glazing for conservatories when a person is on the roof, presumably standing on a plank spread across several rafters, for reasons of maintenance. Locations where special provisions are required are highlighted in Figure 26 representing a typical example of a lean-to conservatory.

Special provision for safety glazing will usually amount to the use of a "safety glazing material" complying with the requirements of AS 2208. In effect this means using toughened or laminated glass or plastics (meeting requirements of AS 2208) which in some instances can be substituted by annealed glass of a specified minimum thickness and maximum area. In areas where special provisions for human impact are not required then wind load and support conditions will determine the glass thickness and size.

When applied to the test conservatory, the above effects of human impact considerations has led to the use of 0.73 m^2 panes of 5 mm annealed glass in the lower half of the wall only, part of the panes being within 200 mm of the finished floor level. The panes of glass in the upper half were 4 mm thick to resist wind loads. There was no door included in the test specimen, but had there been, 4 mm panes would not have been sufficient in adjacent upper panes and additional protection would be required (e.g., 5 mm panes).

4.5 Maintenance Load on Rafters

4.5.1 Observed Behaviour

A load - deflection plot is shown in Figure 27 for the deflection of the central rafter at a point 168 mm to one side of mid-span. The deflection data between 3 and 4 kN was unreliable and is not included in the plot. Failure occurred at a load of 4.1 kN, when all three rafters buckled beneath the plank and caused a sheet of the laminated glass to crack. A view of one of the rafters at failure is shown in Figure 28. The residual deflection in the central rafter, recorded at the same position, after loads of 1, 2 and 3 kN were applied and released is shown in Table 7.

4.5.2 Discussion

NZS 4211 limits the amount of residual deformation to 5% of span/180 or 0.2 mm whichever is the greater. Therefore in this case the maximum permitted residual deflection was 0.64 mm. A 2 kN load was applied to the timber plank on the roof without exceeding this limit when the load was removed.

A 1 kN concentrated live load is specified in NZS 4203 to represent the effect of a person carrying a load. Common sense suggests that it be applied to the structure and not the glass cladding; however, it is still almost impossible for this to occur in practice on a glazed roof as it requires considerable skill to remain balanced on one foot on a single rafter. Instead, loads are likely to be distributed by using a timber plank spread across at least two or three rafters to carry the weight of a person and load, and this was the situation the test attempted to simulate.

The level of load effectively transferred by the plank to rafters will depend upon the relative stiffness of the plank and rafters. The rafters used in the test were noted to be at their maximum recommended span. From the test data on the relative deflection of the three rafters it was calculated that during the test, approximately 40% of the applied load was distributed by the plank to the rafter directly beneath it and approximately 30% was distributed to each of the side rafters. The stiffness of the timber plank as represented by the EI value, was 13200 Nm^2 compared with 10200 Nm^2 for each of the aluminium rafters.

During the test, no reason became apparent for deflection controls to be required on the movement of rafters under maintenance loads; and again, design for strength alone seems appropriate.

4.6 Discussion of Other Loading Cases

4.6.1 Earthquake Forces

Earthquake forces are very rarely important in the structural design of domestic conservatories. Because of the lightweight nature of the structure, earthquake loads are generally low in comparison to wind forces. It can be shown (see Appendix A.4) that in a low wind zone (375 Pa) wind forces will always govern for conservatories less than about 7 m in length. For locations where the wind pressure is higher, this limiting length will increase to about 11 m at 550 Pa and to 25 m at 1100 Pa.

It is reasonable then, for the vast majority of designs, to neglect earthquake loads. Therefore, when determining the racking resistance of these types of structure, a test method using unidirectional racking rather than cyclic would be sufficient, provided the test specimen is symmetrical, as cyclic racking is designed specifically to assess performance when the direction of load is alternately reversed.

4.6.2 Other Loads

A uniform live load of 0.25 kPa is sometimes used as seen in the survey data already presented. It is however intended mainly as a construction load which is largely irrelevant for this type of domestic conservatory construction and hence could be neglected. Furthermore, even if considered in the design it would rarely govern.

Snow loads on conservatory roofs should be considered as appropriate to the location and in consultation with the loadings code, NZS 4203.

5. CONCLUSIONS

The following conclusions apply specifically to the glazed, aluminium-framed structures of the type considered in this report. They may not necessarily apply to other types of domestic conservatory, particularly those where the glazing detail is not of the glass and gasket type.

5.1 In-plane Racking Strength

The contribution of the roof to the overall resistance to racking of an aluminium-framed glazed conservatory is not substantial for common roof geometries. For the particular geometry and design tested, the roof contribution was estimated at about 30% of the total for a laminated glass roof, and about 20% for a hollow core twin wall polycarbonate roof.

Design should be controlled by ultimate strength considerations only. The serviceability criteria may be relaxed for this type of structure and this limit state ignored.

A test method developed specifically for determining the resistance to racking for conservatories could differ from currently accepted methods for light timber frame construction in the following areas: bracing rating could be determined at larger displacements than height/300; racking may be unidirectional, for symmetrical constructions, rather than cyclic, as wind and not earthquake forces govern; and there is no need to downgrade the rating on the basis of large residual deflections.

5.2 Face-Load Strength of Wall Members

There is no sound basis for the commonly applied deflection limits (height/180 NZS 4211 or height/167 from NZS 4203) used in the ultimate strength design of conservatory walls.

Where it can be shown that a conservatory is likely to be occupied in an extreme windstorm, a control on the movement of the wall mullions, in the order of height/165, could be used to avoid undue concern to occupants due to movement of wall glazing.

5.3 Face-load Strength of Roof Members

There is no sound basis for the commonly applied deflection limits (height/180 from NZS 4211 or height/167 from NZS 4203) for use in the design of conservatory roofs.

The performance of flexible glazing materials such as hollow core twin wall polycarbonate under face loads is partially dependent upon the rafter spacing and the amount of edge cover provided.

5.4 Human Impact Resistance

It is desirable for domestic conservatories to comply with the requirements of the glazing code, NZS 4223 : 1985.

A test method for distributed soft body impact should pay close attention to accurately modelling the glass mounting and frame support conditions. Higher levels of impact before breakage can be attained for common conservatory construction tested in this manner rather than using impact test results from glazing materials mounted in a rigid frame.

5.5 Maintenance Load on Rafters

It is reasonable to assume that a 1 kN concentrated maintenance load can be distributed across three rafters by using a plank to spread the load.

It is concluded that there is no sound basis for serviceability deflection limits to be placed upon the movement of rafters during loads incurred during maintenance procedures.

5.6 Other Load Cases

It is concluded that earthquake loads and uniform live loads on domestic conservatories can be neglected. Snow, wind and dead loads should be considered as specified in appropriate codes.

6. SUGGESTIONS FOR FURTHER WORK

A test method for determining the racking resistance of glazed conservatories, and other similar types of construction should be developed.

The potential for calculation methods based upon the amount of friction developed between the glazing and gasket should be investigated.

An investigation should be made into the characteristic behaviour of glazed timber framed and uPVC conservatories to establish if the conclusions made in this report for glazed, aluminium-framed constructions are applicable.

Research be conducted into the effects of glass movement under wind forces which cause concern or alarm to building occupants and the quantification of the amount of movement that is regarded as unacceptable.

7. REFERENCES

Standards and Codes

British Standards Institution. 1982. Code of Practice for Glazing for Buildings. BS 6262. London, UK

Standards Association of Australia. 1977. Specification for Aluminium Windows for Buildings. AS 2047. North Sydney, Australia

Standards Association of Australia. 1978. Safety Glazing Materials for Use in Buildings (human impact considerations). AS 2208. North Sydney, Australia

Standards Association of Australia. 1989. Loadings Code, Part 2, Wind Forces. AS 1170. North Sydney, Australia

Standards Association of New Zealand. 1984. Code of Practice for General Structural Design and Design Loadings for Buildings. NZS 4203. Wellington, New Zealand

Standards Association of New Zealand. 1984. Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design. NZS 3604. Wellington, New Zealand

Standards Association of New Zealand. 1989. Code of Practice for General Structural Design and Design Loadings for Buildings. 2/DZ 4203. Wellington, New Zealand

Standards Association of New Zealand. 1985. Specification for Performance of Windows. NZS 4211. Wellington, New Zealand

Standards Association of New Zealand. 1985. Code of Practice for Glazing in Buildings. NZS 4223. Wellington, New Zealand

General

Cooney, R.C. and Collins, M.J. 1979 (revised 1982, 1987). A Wall Bracing Test and Evaluation Procedure. Building Research Association of New Zealand Technical Paper P21. Judgeford, New Zealand

Cooney, R.C. 1989. Personal Communication. Building Research Association of New Zealand, Judgeford, New Zealand

Seddon, R. 1988. Domestic Conservatories in Wellington - A Study of Problems. Victoria University of Wellington, School of Architecture, ARCH 389 Research Report. Wellington, New Zealand

Toakley, A.R. 1977. Stresses and Safety Levels for Glass Liable to Human Impact. Building and Environment, Vol 12, pp 87-95.

APPENDIX A

A.1 General Introduction

For the purpose of determining expected design loads on a typical conservatory the following assumptions regarding the design and location of the conservatory have been made in Sections A.2 and A.3.

The conservatory is located in a region with a basic ultimate wind speed of 50 m/s. The terrain is suburban in nature with numerous closely spaced obstructions and the conservatory is positioned on the crest of an exposed hill. The maximum eaves height of the conservatory above ground is taken as 5 m.

Where expected design wind pressures are included in the body of the report, they refer to calculations in accordance with 2/DZ 4203 : 1989. This draft standard is considered to contain a more rigorous and realistic treatment of wind loads than the existing NZS 4203 : 1984. Equivalent wind pressures calculated in accordance with NZS 4203 are also included here for comparison.

A.2 Maximum Expected Design Racking Load

From 2/DZ 4203 :

Ms = Mo = Mc = Me = 1.0
Mx = 0.75 terrain multiplier
Mt = 1.36 topographical multiplier (moderate hillslope)
Vu = 50 m/s basic ultimate wind speed, 5% probability of exceedance in 50 years

Design gust wind speed for site, Vzu = Mx Mt Vu = 0.75 x 1.36 x 50 = 51.0 m/s

Design wind pressure for the site, Qzu = 0.6 Vzu^2 = 0.6 x 51.0^2 = 1561 Pa

Two widths of conservatory will be considered, one 2.4 m wide and one 4.0 m wide.

Area of the windward side wall,

A = average height x width = 2.3 m x 2.4 m = 5.5 m^2 (for 2.4 m wide)
= 2.3 m x 4.0 m = 9.2 m^2 (for 4.0 m wide)

Pressure coefficients are taken from the New Zealand draft loadings code 2/DZ 4203 : 1989, adapted from AS 1170 : Part 2 : 1989, being the results of more up to date research.

Cpe (windward wall) = +0.7 Table 4.3.8 (a)
Cpe (leeward wall) = -0.3 Table 4.3.8 (b) (d/b = 2, say)
Cf (building) = 1.0

Total wind force on the building, F = Cf Qzu A

However, it is reasonable to assume that only a quarter of this total will be required to be resisted by the front wall in racking, due to half the load on the end wall being transmitted via the wall mullions to the foundation and half to the side wall eaves. Similarly, the side wall eaves and/or the roof plane diaphragm will then transmit half back to the building and half to the top of the front wall (this amount being the racking load).

Force resisted by the wall, Fw = F x 0.25

Fw = 1.0 x 1.561 x 5.5 x 0.25 = 2.15 kN (for 2.4 m wide)
= 1.0 x 1.561 x 9.2 x 0.25 = 3.59 kN (for 4.0 m wide)

Equivalent calculations in accordance with NZS 4203 : 1984 lead to a design gust wind speed for the site of 42 m/s (V=50m/s; S1=1.2; S2=0.70; - GR=3, Class A, H = 5 m) and a design wind pressure for the site of 1081 Pa. To enable comparison with 2/DZ 4203, a load factor of 1.3 should be applied to the design wind pressure for the site to give 1406 Pa (compare with 1561 Pa using 2/DZ : 1989).

A.3 Ultimate and Serviceability Design Wind Forces

From 2/DZ 4203 : 1989

Ms = Mo = Mc = Me = 1.0
Mx = 0.75 terrain multiplier
Mt = 1.36 topographical multiplier (moderate hillslope)
Vs = 36 m/s basic serviceability wind speed
Vu = 50 m/s basic ultimate wind speed, 5% probability of exceedance in 50 years

Design gust wind speed for site,
Vzs = Mx Mt Vs = 0.75 x 1.36 x 36 = 36.7 m/s
Vzu = Mx Mt Vu = 0.75 x 1.36 x 50 = 51.0 m/s

Design wind pressure for the site, $Qzs = 0.6\ Vzs^2 = 0.6 \times 36.7^2 = 809$ Pa
$Qzu = 0.6\ Vzu^2 = 0.6 \times 51.0^2 = 1561$ Pa

External pressure coefficients

walls - Cpe = +0.7	Table 4.3.8 (a)	windward wall
roofs - Cpe = -0.9	Table 4.3.9 (b)	h/d=0.5,α<10

External Pressure

Pe = Cpe Ka Kl Kp Q

Ka = area reduction factor
Kl = local pressure factor
Kp = reduction factor for porous cladding

Internal pressure coefficients

Cpi = -0.3 Table 4.3.13 Condition 4(c) side wall opening
or 0 Table 4.3.13 Condition 5 fully sealed

Pi = Cpi Q

Design wind forces

F = S Pz Az

where Pz = Pe - Pi

Az = area on which design wind pressure operates

Design wind forces on a conservatory wall

i) ultimate

F/Az = Pe - Pi = (Cpe Ka Kl Kp Q) - (Cpi Q)
= (+0.7 x 1 x 1 x 1 x 1561) - (-0.3 x 1561)
= 1561 Pa

ii) serviceability

F/Az = Pe - Pi = (Cpe Ka Kl Kp Q) - (Cpi Q)
= (+0.7 x 1 x 1 x 1 x 809) - (-0.3 x 809)
= 809 Pa

Design wind forces on a conservatory roof

i) ultimate

F/Az = Pe - Pi = (Cpe Ka Kl Kp Q) - (Cpi Q)
= (-0.9 x 1 x 1 x 1 x 1561) - (0 x 1561)
= -1405 Pa

ii) serviceability

F/Az = Pe - Pi = (Cpe Ka Kl Kp Q) - (Cpi Q)
= (-0.9 x 1 x 1 x 1 x 809) - (0 x 809)
= -728 Pa

A.4 Earthquake Forces

Earthquake and wind forces are compared for a W (m) wide x L (m) long x H (m) high conservatory, with a dead weight of 150 N/m^2 assumed (typical for 6 mm thick glass). The conservatory is assumed to be located in a suburban region with a basic ultimate wind speed of 32 m/s on a site unaffected by local topographical features and with a maximum eaves height of 3 m. Seismic zone is A (NZS 4203).

The areas of the conservatory assumed to contribute to the seismic force, in the direction parallel to the front wall of the conservatory, and to be resisted by the front wall are: half the dead weight of the roof; and one quarter of the dead weight of each end wall.

Area, $A = (0.50 \times L \times W) + 2 \times (0.25 \times W \times H) = 0.50\ W\ (L + H)$ $[m^2]$

Dead load, $Wp = 150\ N/m^2 \times 0.50\ W\ (L + H)\ m^2 = 75\ W\ (L + H)$ [N]

$Cp = A\ Kx\ Z\ R\ Cp = 0.3$ (from NZS 4203 : 1984) using:

Sp = 1 (reasonably ductile - Table 8)
Cpmax = 0.3 (adjacent to exitway - Table 9)
R = 1 (from Table 4)
Kx = 1 (single-storey)
a = 1 (single-storey)
Z = 1 (seismic zone A)

Seismic Force on the part, $Fp = Cp \times Wp = 0.3 \times 75\ W\ (L + H)$ [N]

$= 22.5\ W\ (L + H)$ [N]

Now, in a low wind area -

From 2/DZ 4203 : 1989

Ms = Mo = Mc = Me = 1.0
Mx = 0.75 terrain multiplier
Mt = 1.00 topographical multiplier
Vu = 32 m/s basic ultimate wind speed, 5% probability of exceedance in 50 years

Design gust wind speed for site, $Vzu = Mx\ Mt\ Vu = 0.75 \times 1.04 \times 32 = 25.0$ m/s

Design wind pressure for the site, $Qzu = 0.6\ Vzu^2 = 0.6 \times 25.0^2 = 375$ Pa

Assuming one quarter of the end wall area contributes to the racking load to be resisted by the front wall.

Area, $A = 0.25\ W\ H$

The wind load on the same structure would be:

$F = Cf\ Qzu\ A = 1.0 \times 375 \times 0.25\ W\ H = 94\ W\ H$ [N]

Now, for the earthquake load to exceed the wind load -

$22.5\ W\ (L + H) > 94\ W\ H$ or $L > 3.2\ H$

For this condition to be satisfied the length of the conservatory must exceed 3.2 times the conservatory height. Since the average height is not usually less than 2.2 metres, wind loads will always govern for conservatories less than 7.0 metres in length. For locations where the design wind pressure is higher than the 375 Pa assumed here, this limiting length will increase to 11.2 m at 550 Pa (5.1 x H) and to 24.7 m at 1100 Pa (11.2 x H).

Equivalent calculations in accordance with NZS 4203 : 1984 lead to a design gust wind speed for the site of 20 m/s (V=32 m/s; S1=1.0; S2=0.64; - GR=3, Class A, H = 3 m) and a design wind pressure for the site of 257 Pa. To enable comparision with 2/DZ 4203, a load factor of 1.3 should be applied to the design wind pressure for the site to give 334 Pa (compare with 375 Pa using 2/DZ 4203 : 1989).

TABLE 1

STIFFNESS PROPERTIES OF SOME COMMONLY USED WALL MULLIONS			
MATERIAL(1)	MAX. SPAN(2), L (m)	EI (Nm^2)	EI/L^3 (N/m)
A	2.00	10162	1270
T	2.00	10815	1352
A	2.43	20264	1412
A	1.96	10655	1415
T	1.89	10815	1602
A	2.40	41282	2986
A	2.00	26660	3333

(1) A = Aluminium, T = Timber
(2) In a high wind zone.

TABLE 2

STIFFNESS PROPERTIES OF SOME COMMONLY USED RAFTERS			
MATERIAL(1)	MAX. SPAN(2), L (m)	EI (Nm^2)	EI/L^3 (N/m)
A	2.30	10162	835
A	2.20	10162	954
A	2.75	21298	1024
A	2.30	13200	1085
A	2.83	26660	1176
A	2.04	10654	1255
A	3.76	66712	1255
T	1.80	8197	1406
T	2.50	28705	1837
A	2.00	15375	1922
A	2.00	19418	2427
A	2.40	35071	2537

(1) A = Aluminium, T = Timber
(2) In a high wind zone.

TABLE 3

DEFLECTION CRITERIA USED IN CONSERVATORY DESIGN		
MANUFACTURER	RAFTERS	MULLIONS
A	Span/180	Height/180
B	Span/167	Height/167
C	Span/180	Height/180
D	None	None
E	Span/167	Height/167
F	Span/167 Span/100[1]	*
G	Span/180[2]	Height/180

[1] Only for twin wall polycarbonate roof glazing
[2] Does not apply to concentrated live load
* unclear from design data but suspect H/180

TABLE 4

CONFIGURATION OF TEST SPECIMEN AND LOADING REGIME FOR RACKING TESTS	
TEST CONFIGURATION	CYCLIC DISPLACEMENT REGIME
Fully glazed front wall with two closed sashes	
and; laminated glass roof	4 cycles at ±5, ±10 mm
and; polycarbonate roof	2 cycles at ±2, ±4, ±6, ±8 mm
Fully glazed roof of hollow core twin wall polycarbonate	
and; glazed front wall of fixed panes	2 cycles at ±2, ±4, ±6, ±8, ±10, ±15 mm
and; unglazed front wall	2 cycles at ±10, ±20, ±30, ±40 mm

TABLE 5

DISTRIBUTED SOFT BODY IMPACT TESTS IMPACT ON THE GLASS		
SPECIMEN[1]	DROP HEIGHT, h (mm)	BREAKAGE LEVEL[2], E (J)
5 mm glass		
No. 1	506	150
No. 2	405	120
No. 3	506	150
No. 4	506	150
No. 5	405	120
4 mm glass		
No. 1	304	90
No. 2	253	75
No. 3	203	60
No. 4	203	60
No. 5	203	60
No. 6	304	90

[1] All glass sheets measured 963 mm high x 755 mm wide
[2] $E = mgh$ where $m = 30.2$ kg, $g = 9.81$ m/s^2
Actual breakage level could be up to 15 J less than the stated value.

TABLE 6

DISTRIBUTED SOFT BODY IMPACT TESTS IMPACT ON THE HORIZONTAL RAIL			
TEST NO.	DROP HEIGHT (mm)	BREAKAGE LEVEL (J)	RESULT
1	557	165	lower glass shattered
2	608	180	lower glass shattered
3	Test abandoned due to deterioration of the wall frame after repeated impacts		

TABLE 7

RESIDUAL DEFLECTION OF CENTRAL RAFTER	
LOAD (kN)	RESIDUAL DEFLECTION[1] (mm)
1.13	0.14
2.19	0.47
3.18	2.49

[1] Deflection measured at distance 168 mm from mid-span of rafter.

Span of rafter = 2290 mm

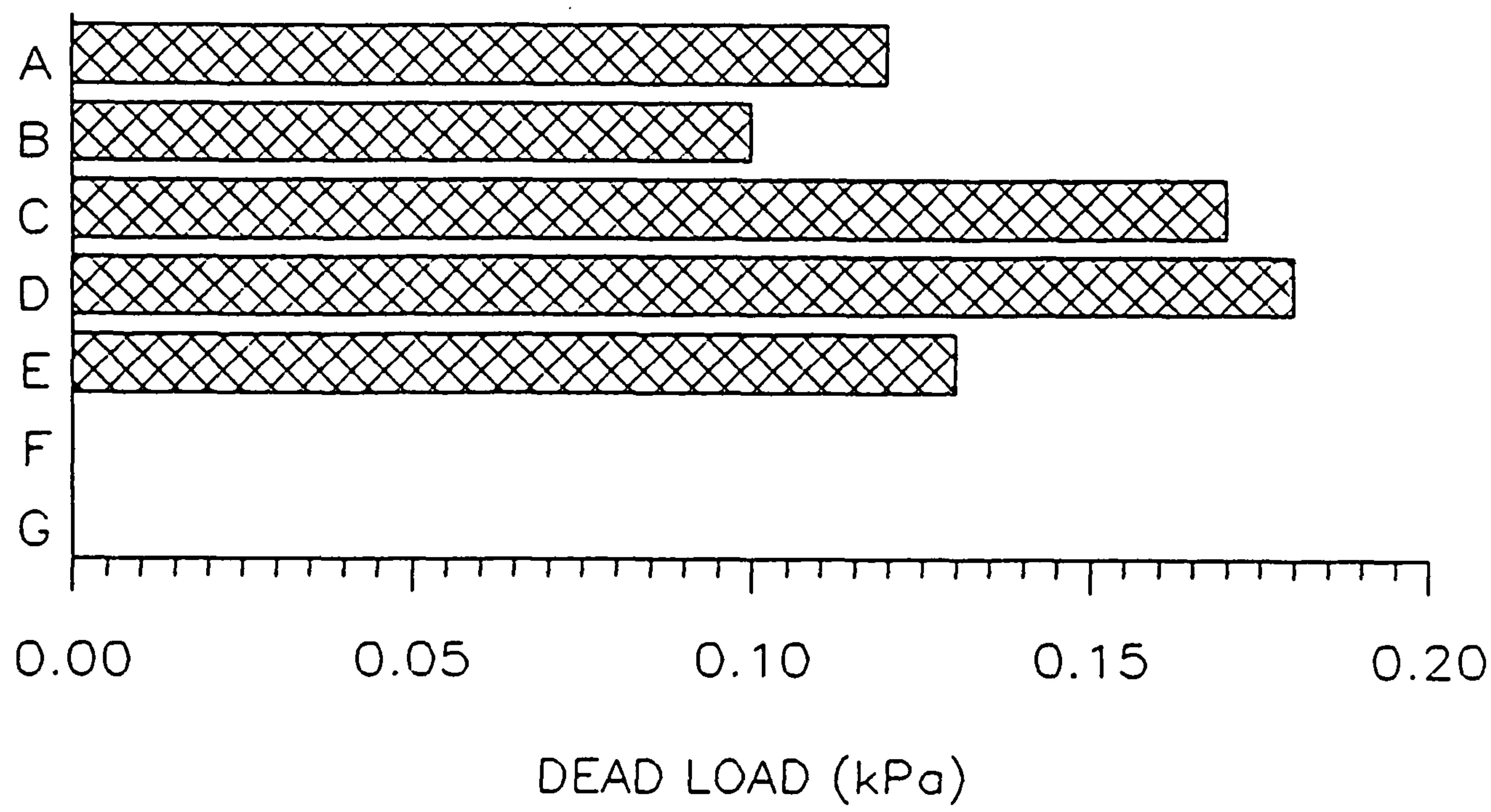


Figure 1 : Summary of Manufacturers' Design Dead Loads

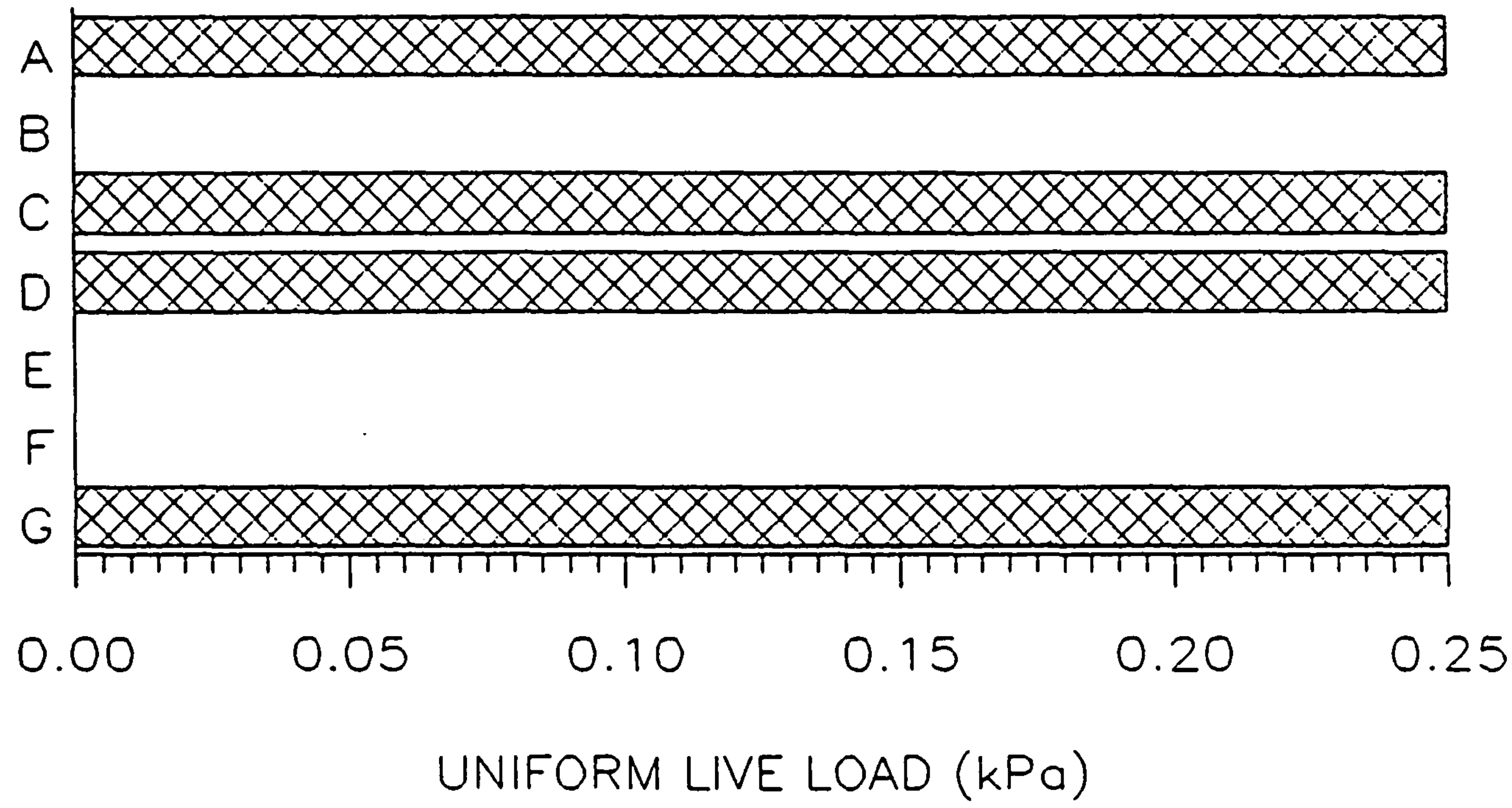


Figure 2 : Summary of Manufacturers' Design Uniform Live Loads

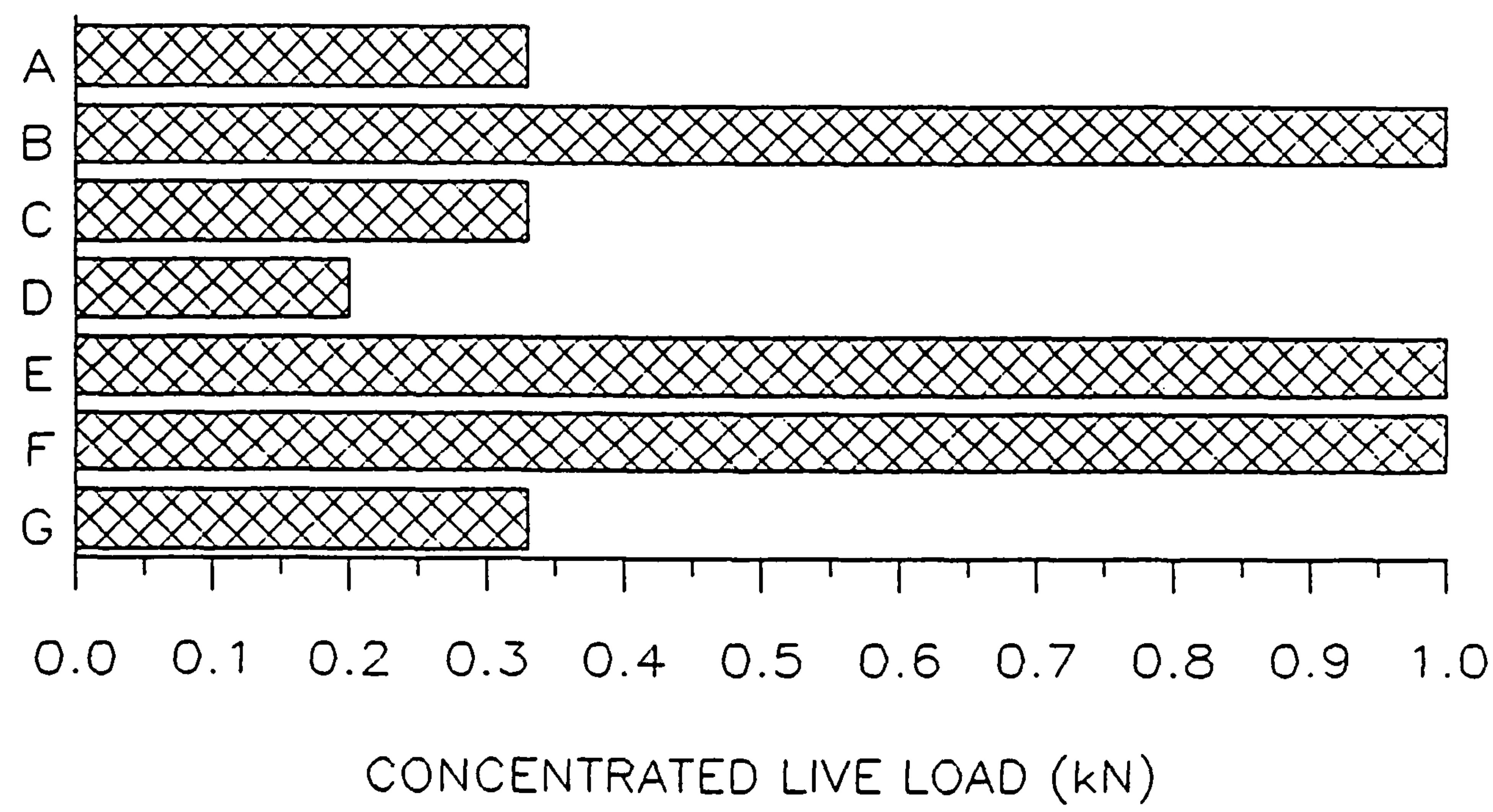


Figure 3 : Summary of Manufacturers' Design Concentrated Live Loads

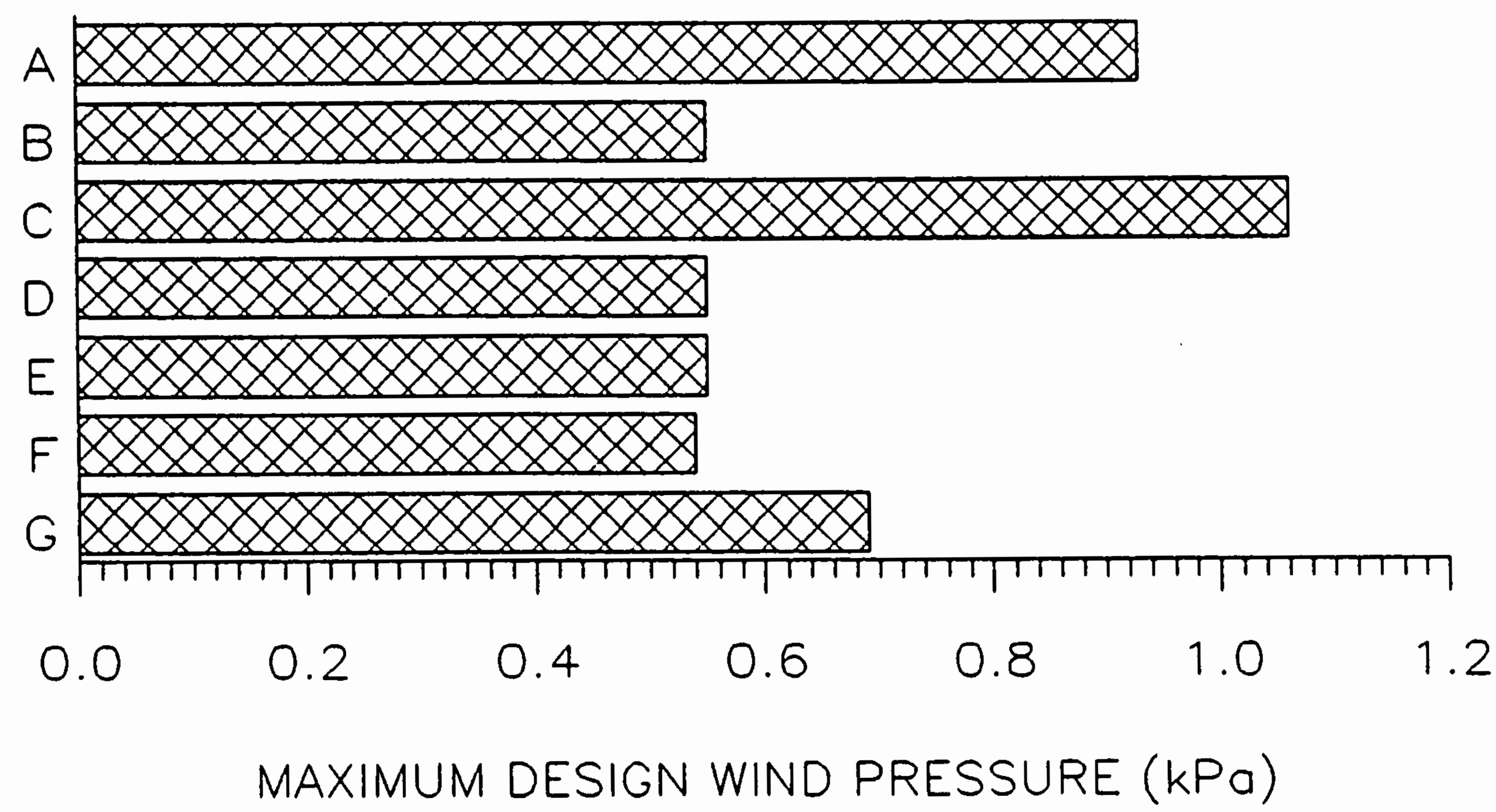


Figure 4 : Summary of Manufacturers' Design Wind Loads

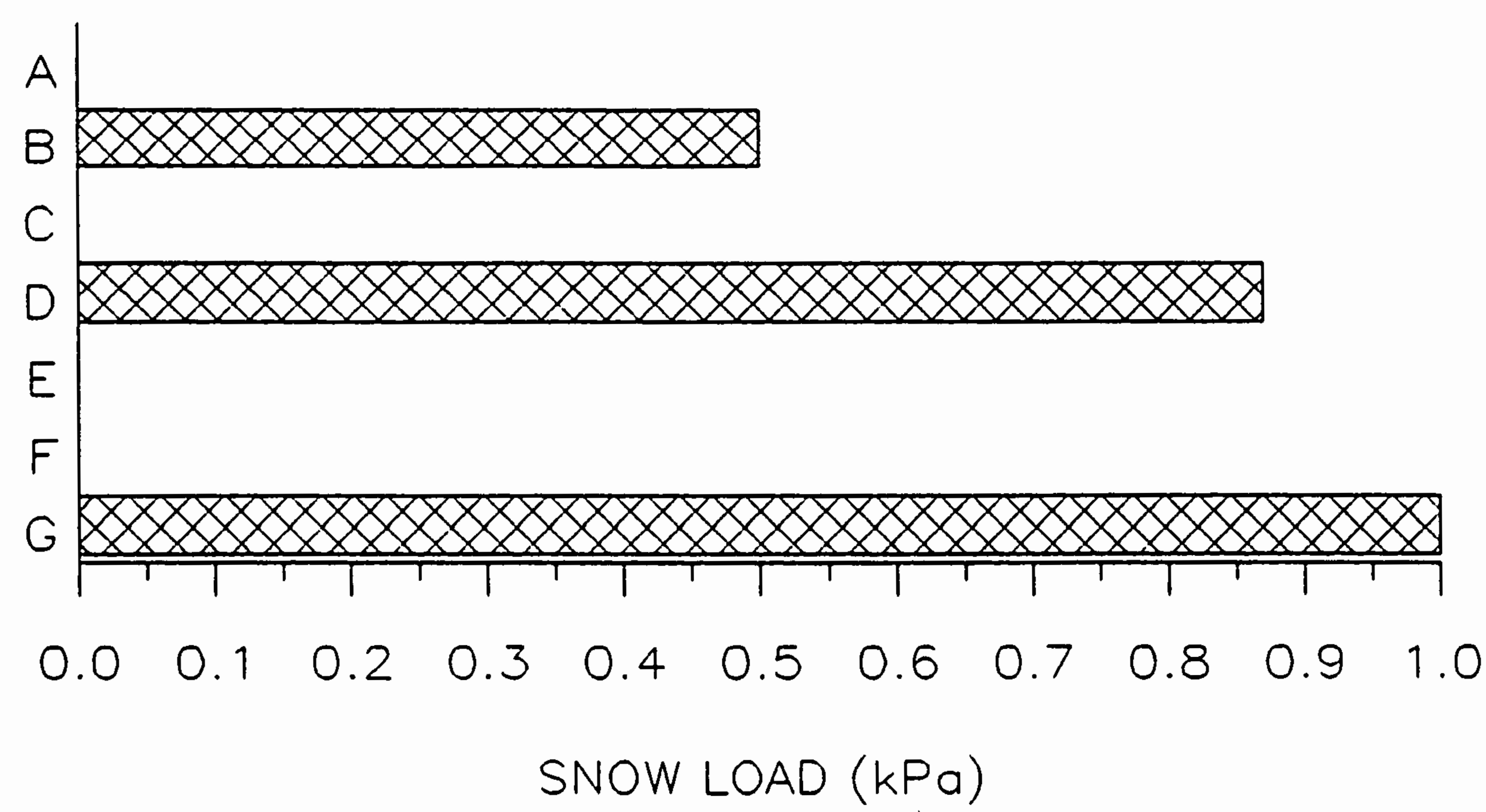


Figure 5 : Summary of Manufacturers' Design Snow Loads

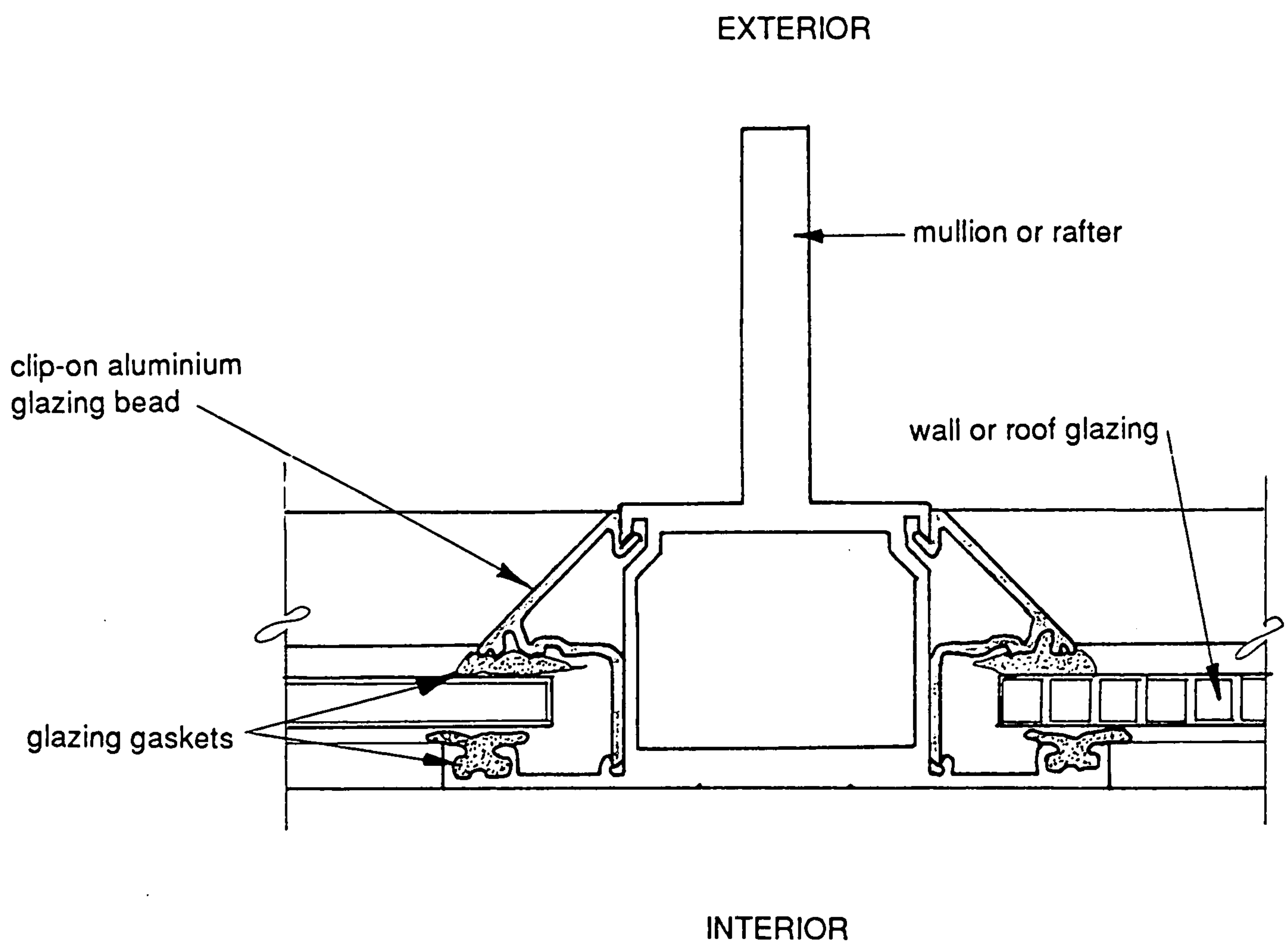
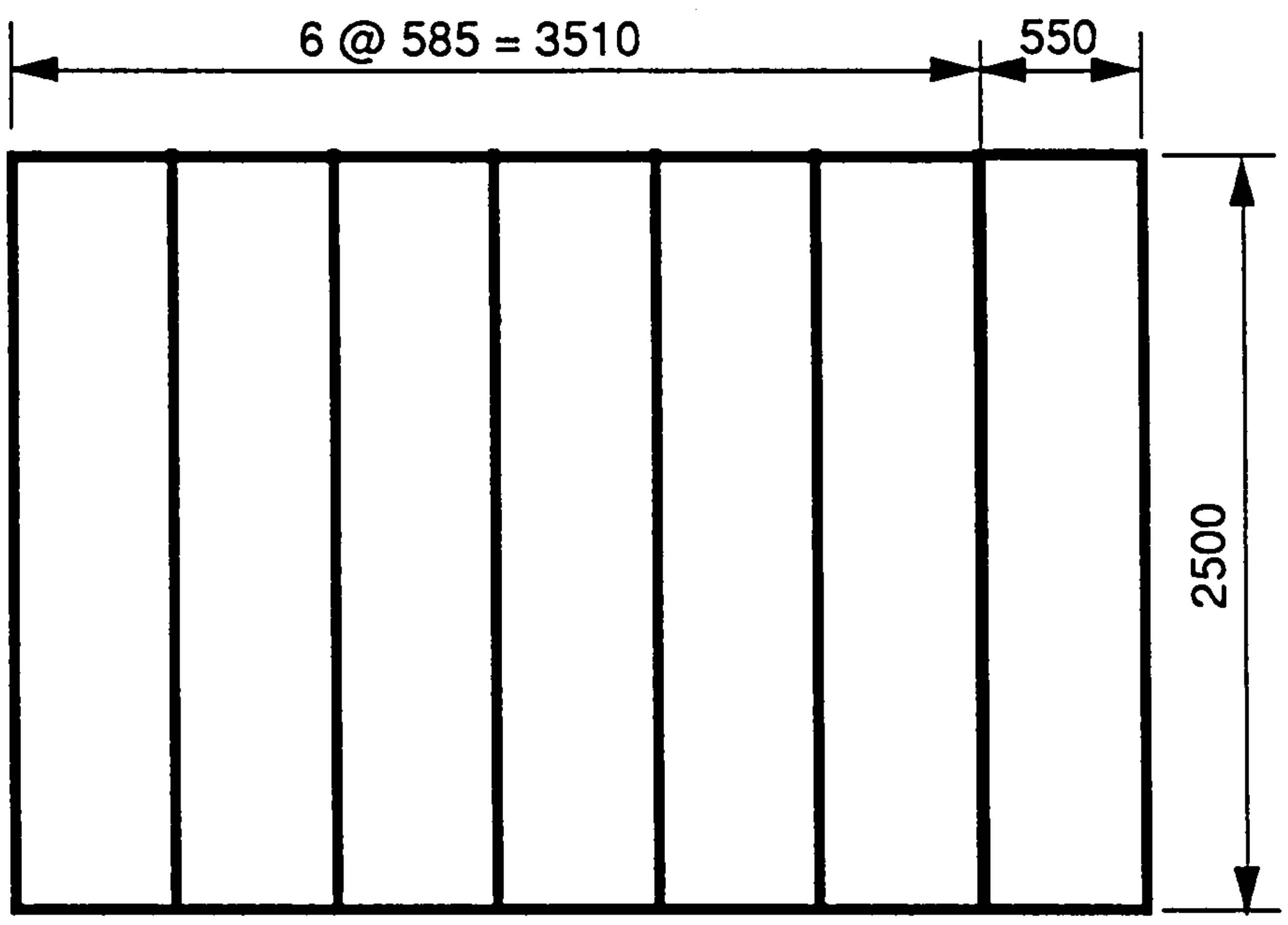
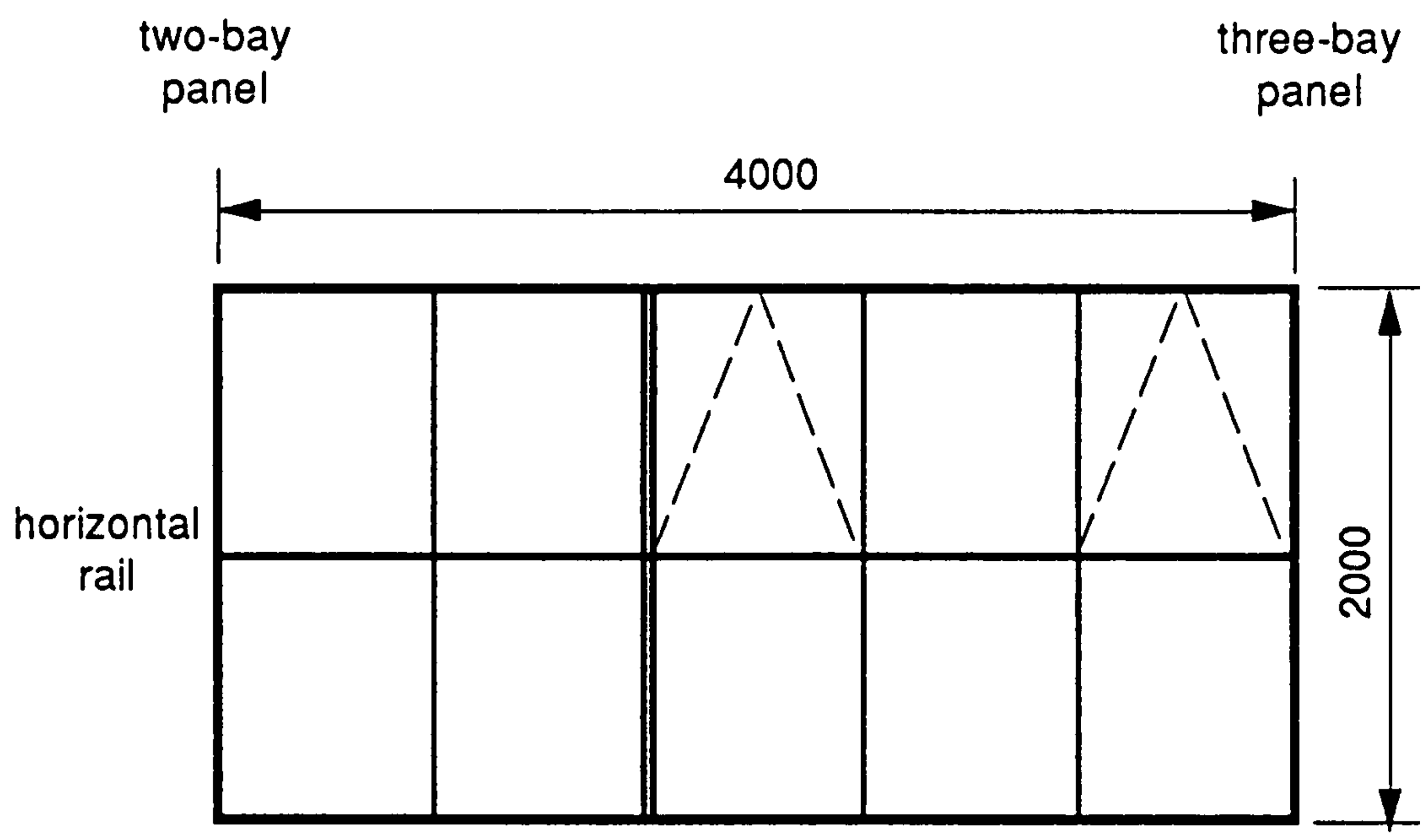


Figure 6 : A Typical Glazing Detail for an Aluminium Framed Conservatory (schematic only)



Roof Plan



Wall Elevation

Figure 7 : Elevations of Test Conservatory Wall and Roof Sections

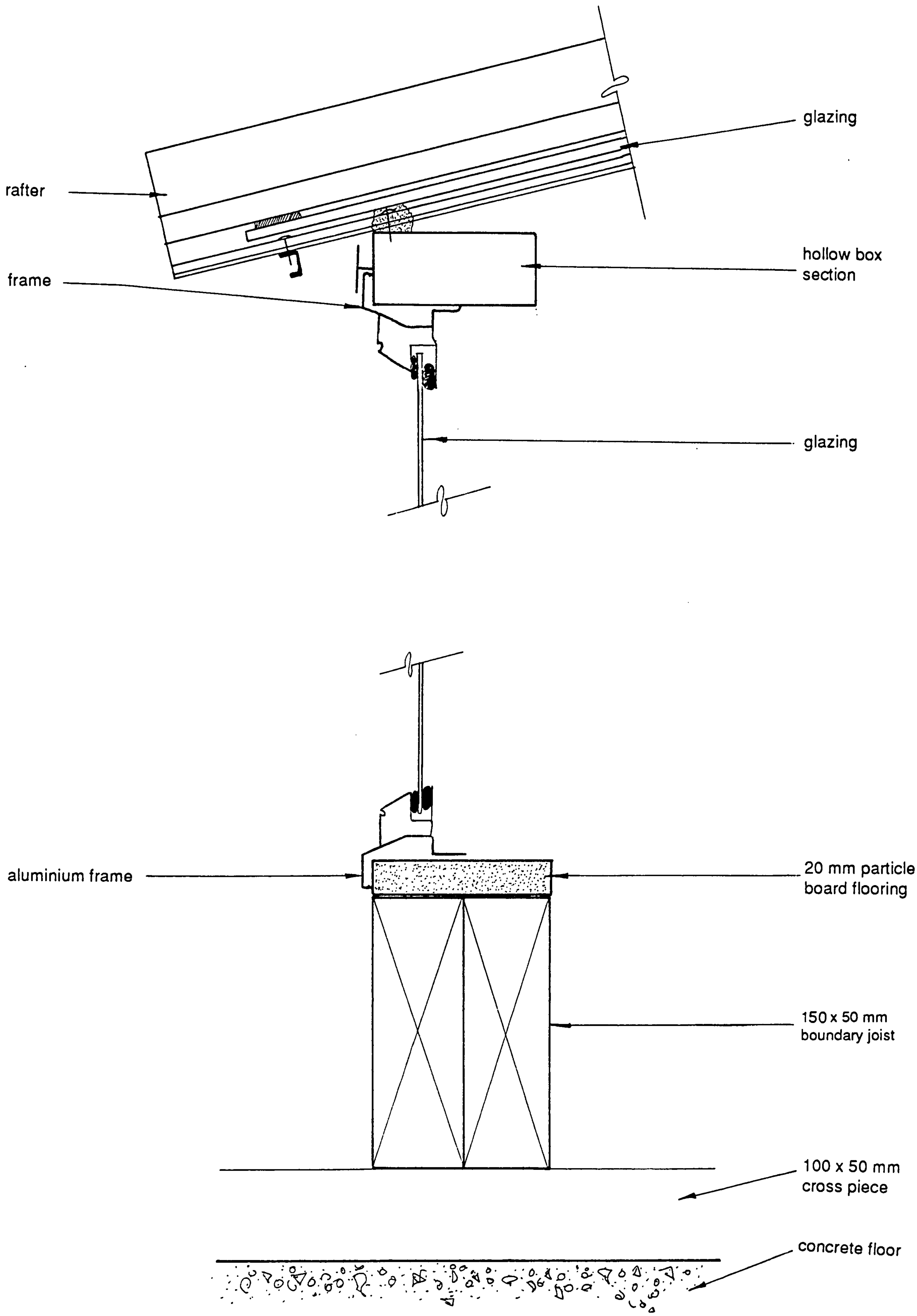


Figure 8 : Cross Section through Conservatory Wall

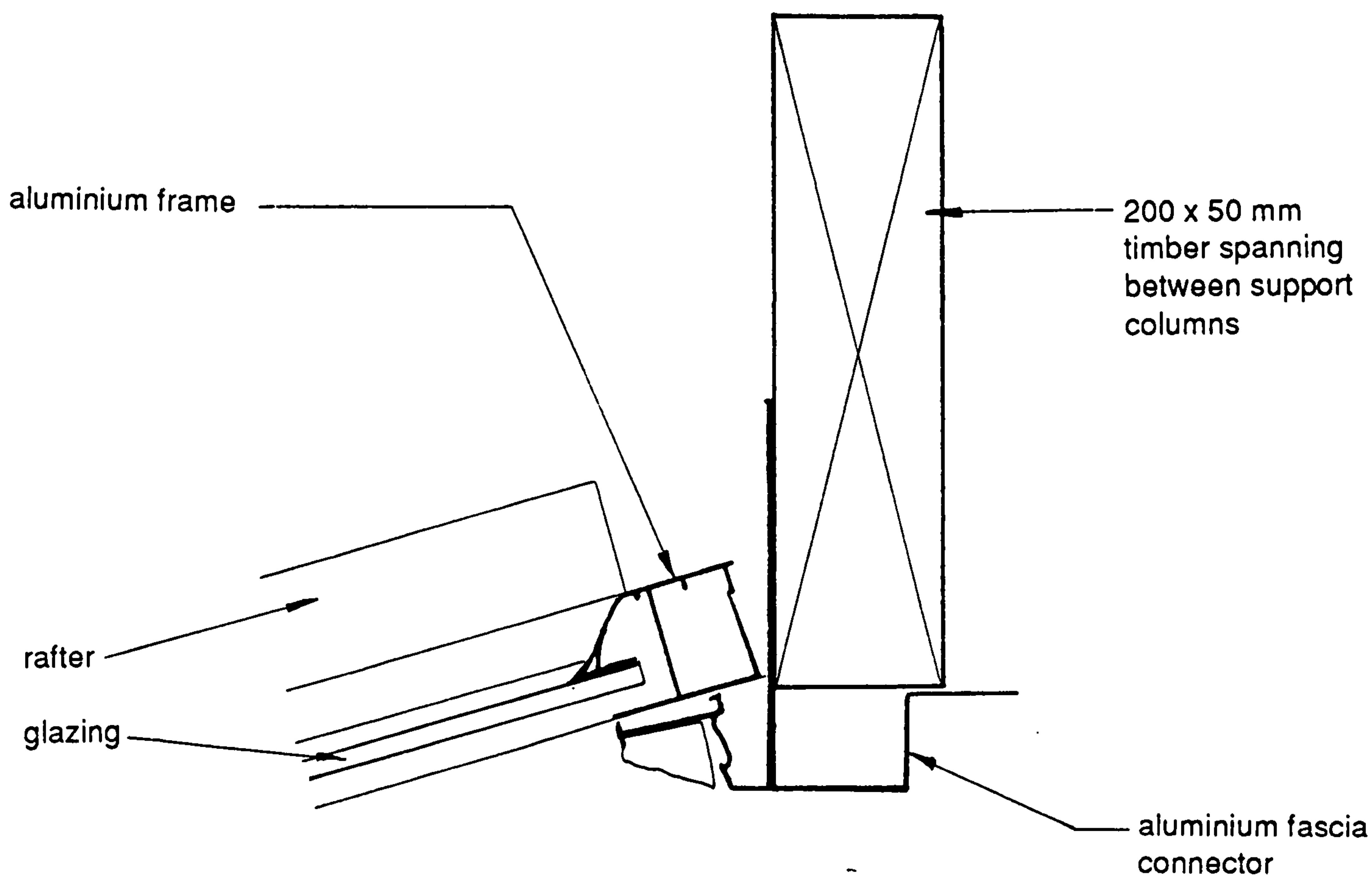


Figure 9 : Cross Section through Conservatory Roof

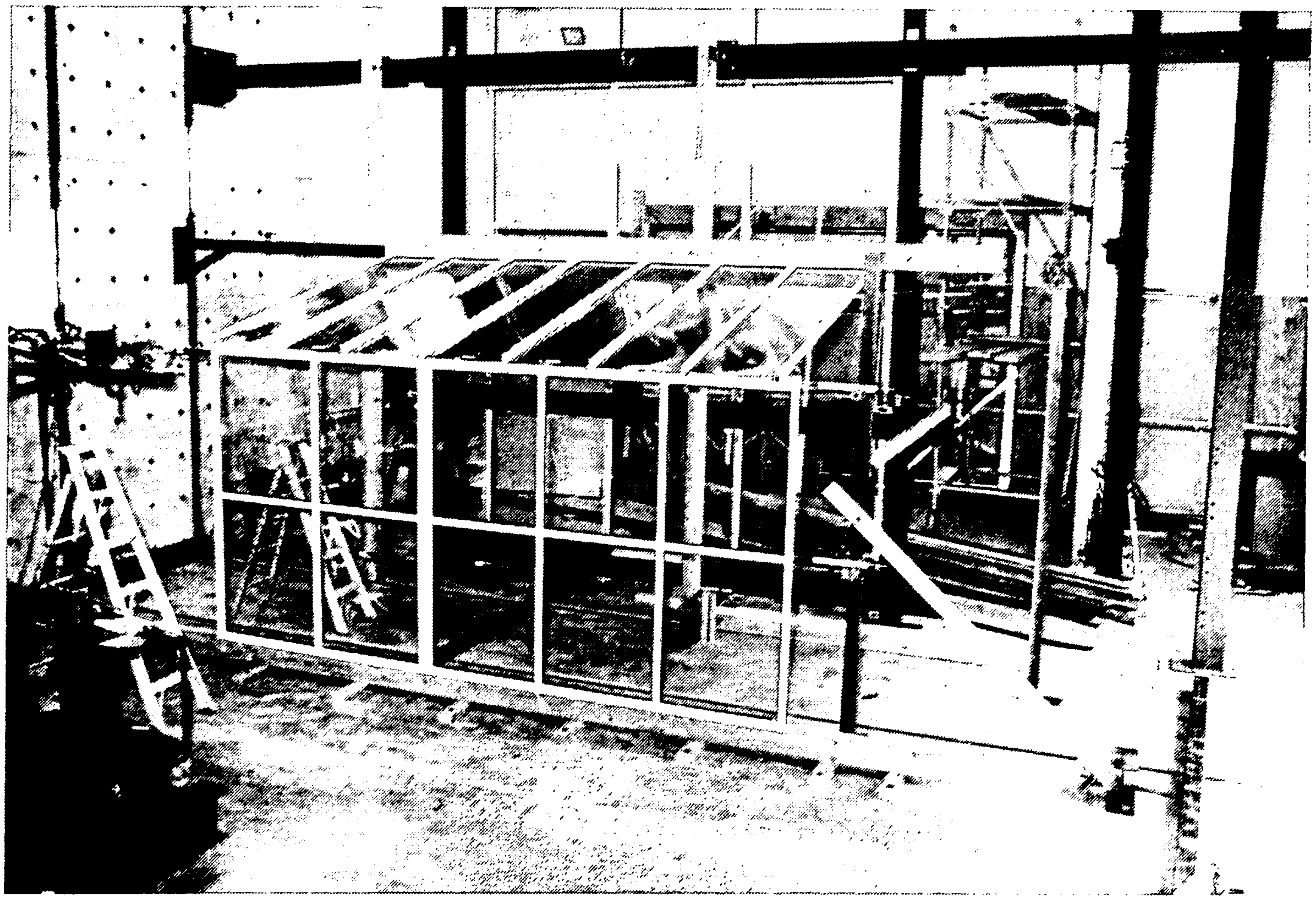


Figure 10 : A View of the Conservatory Prior to Racking

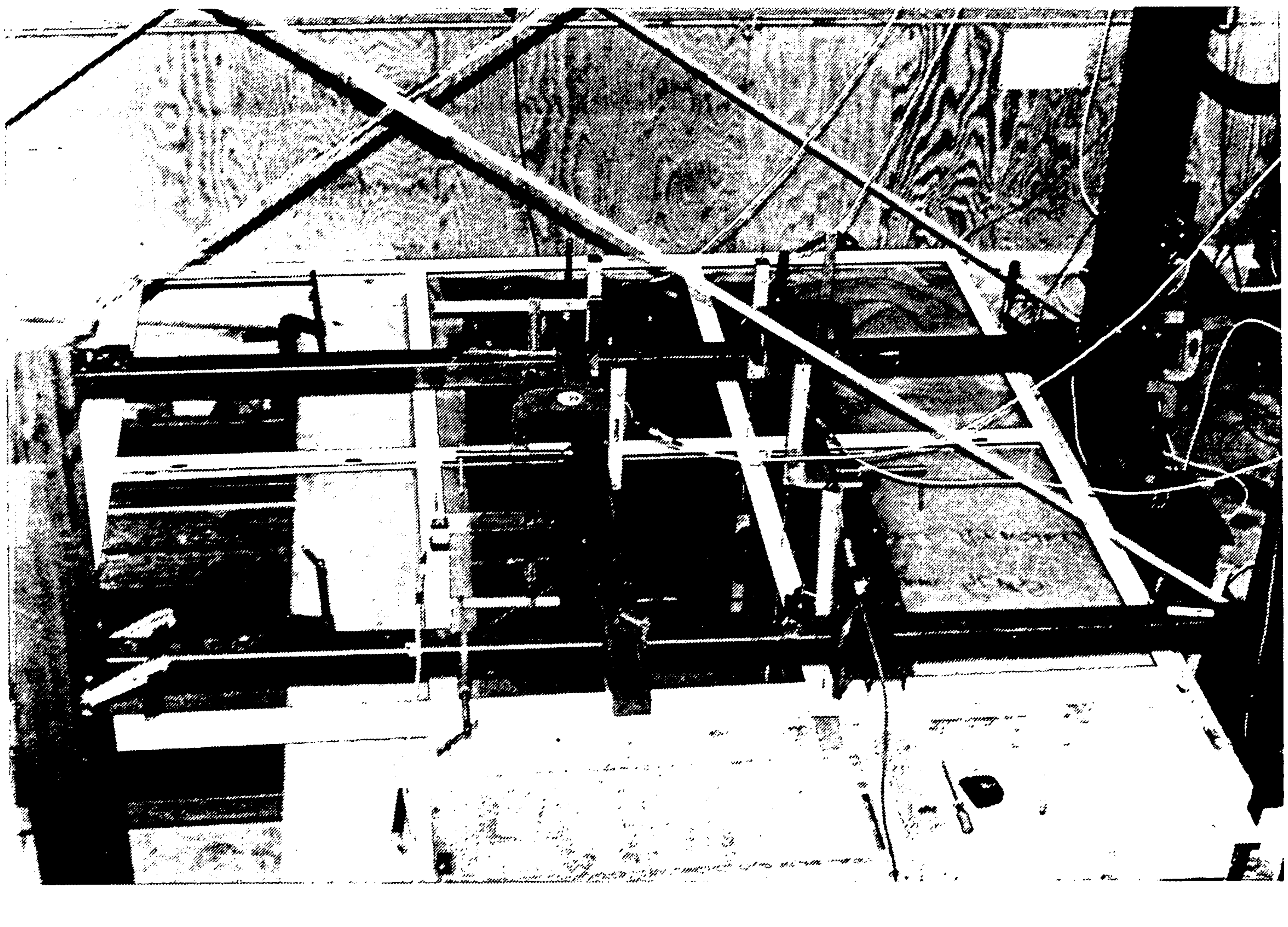


Figure 11 : A View of the Wall Section Used in the Face Load Test

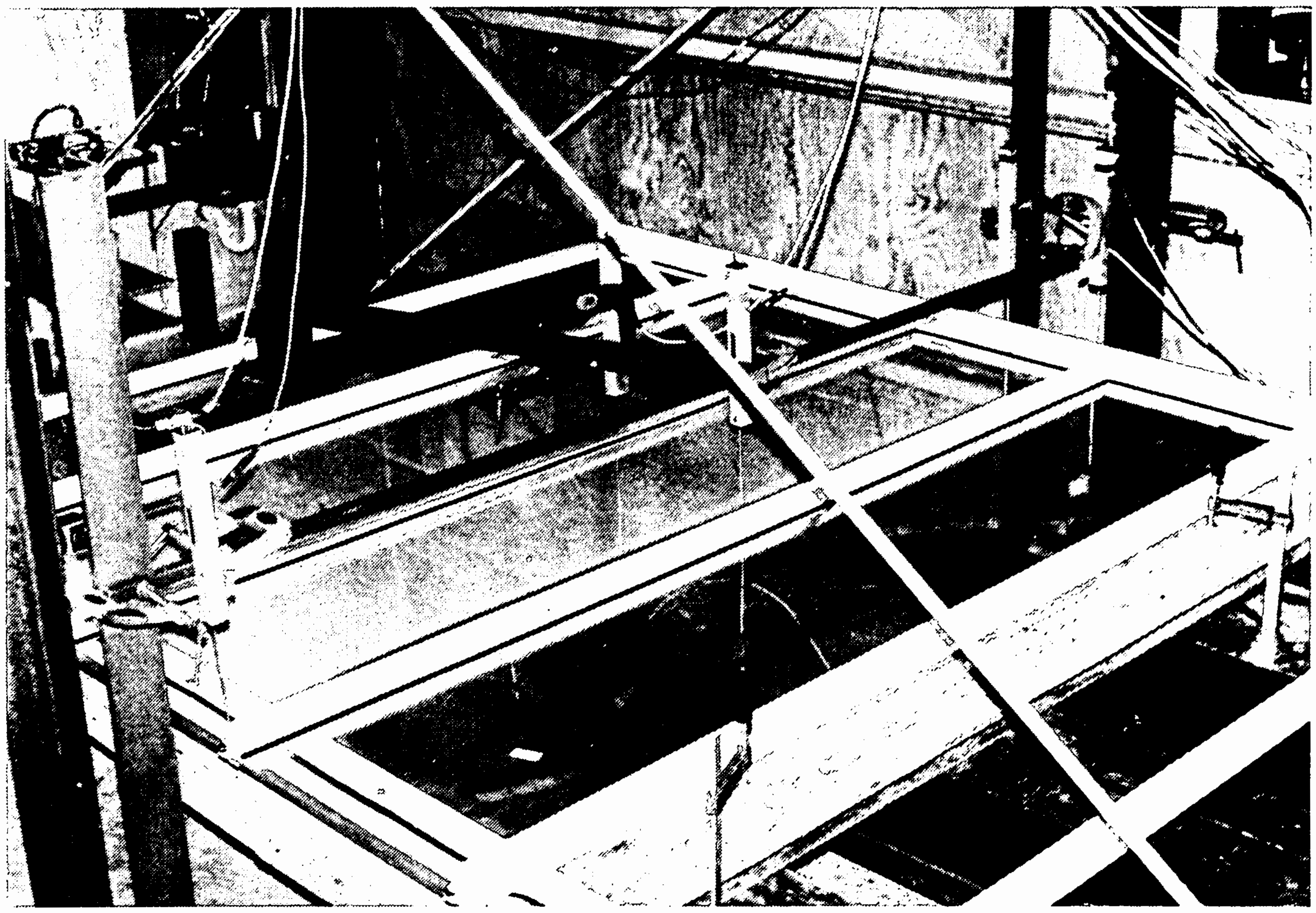


Figure 12 : A View of the Roof Section Used in the Face Load Test

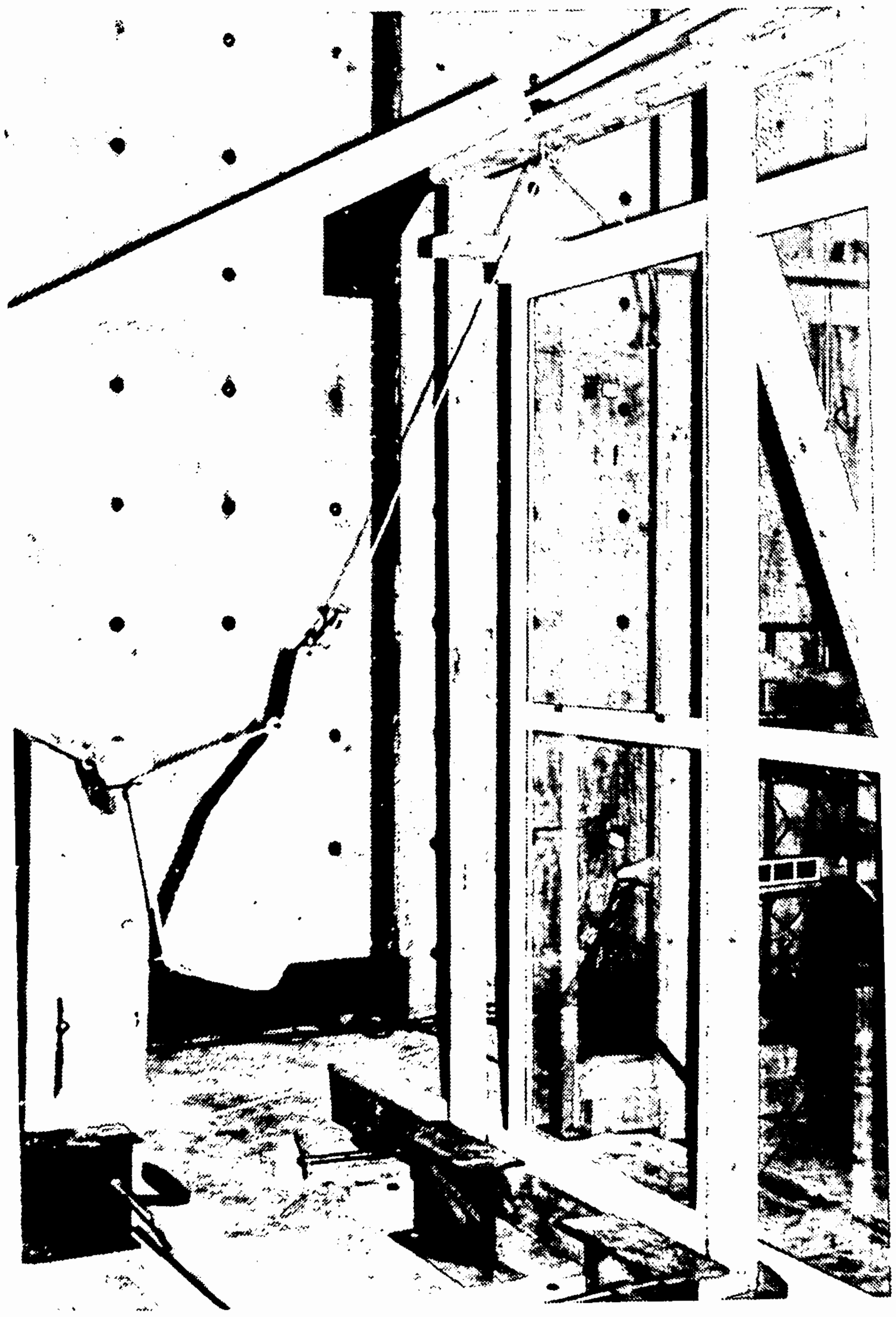


Figure 13 : A View of the Specimen Used for Soft Body Impacts on the Glazing

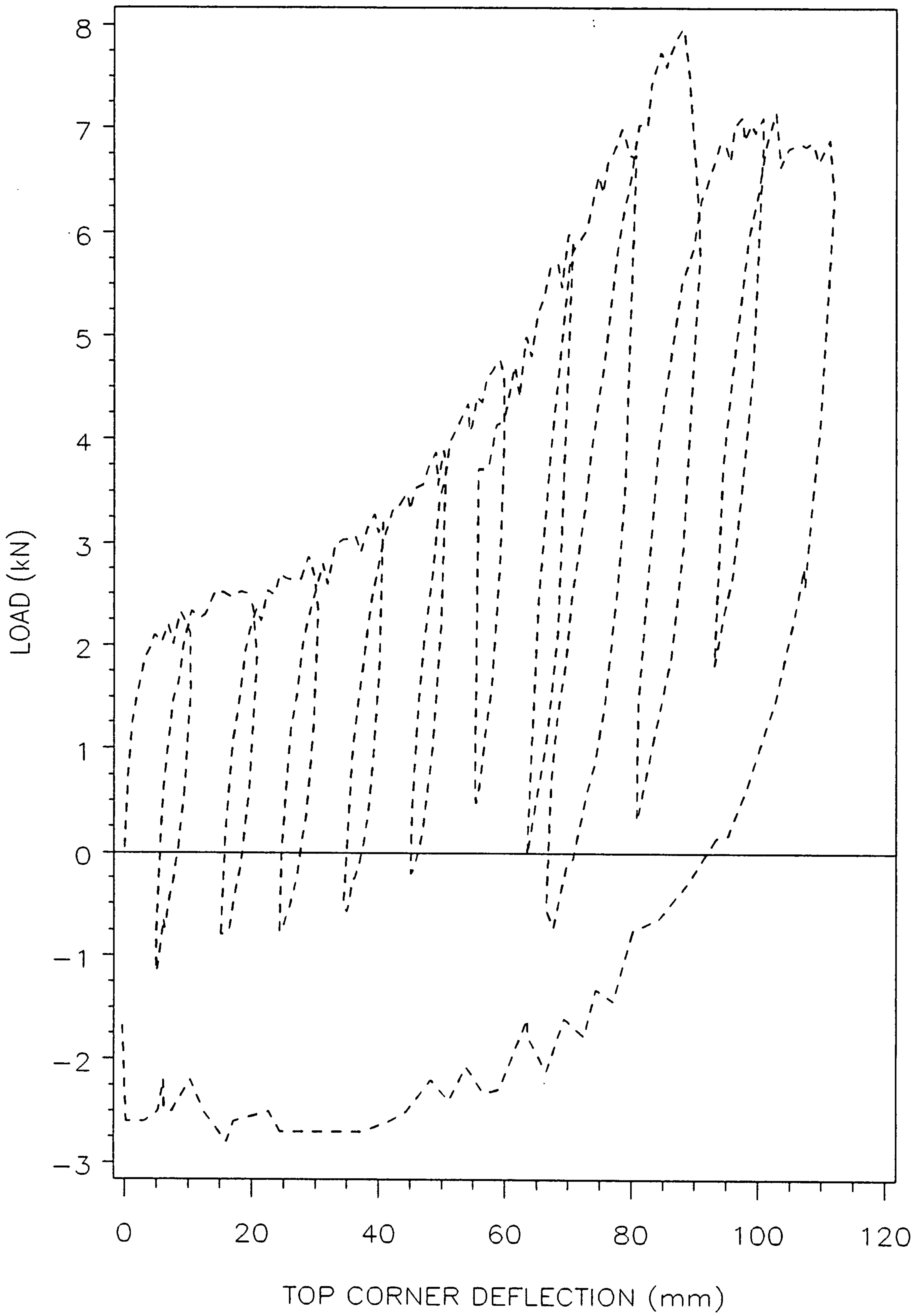


Figure 14 : Load - Deflection Plot for Test Conservatory, With a Fully Glazed Wall and Twin Wall Polycarbonate Roof

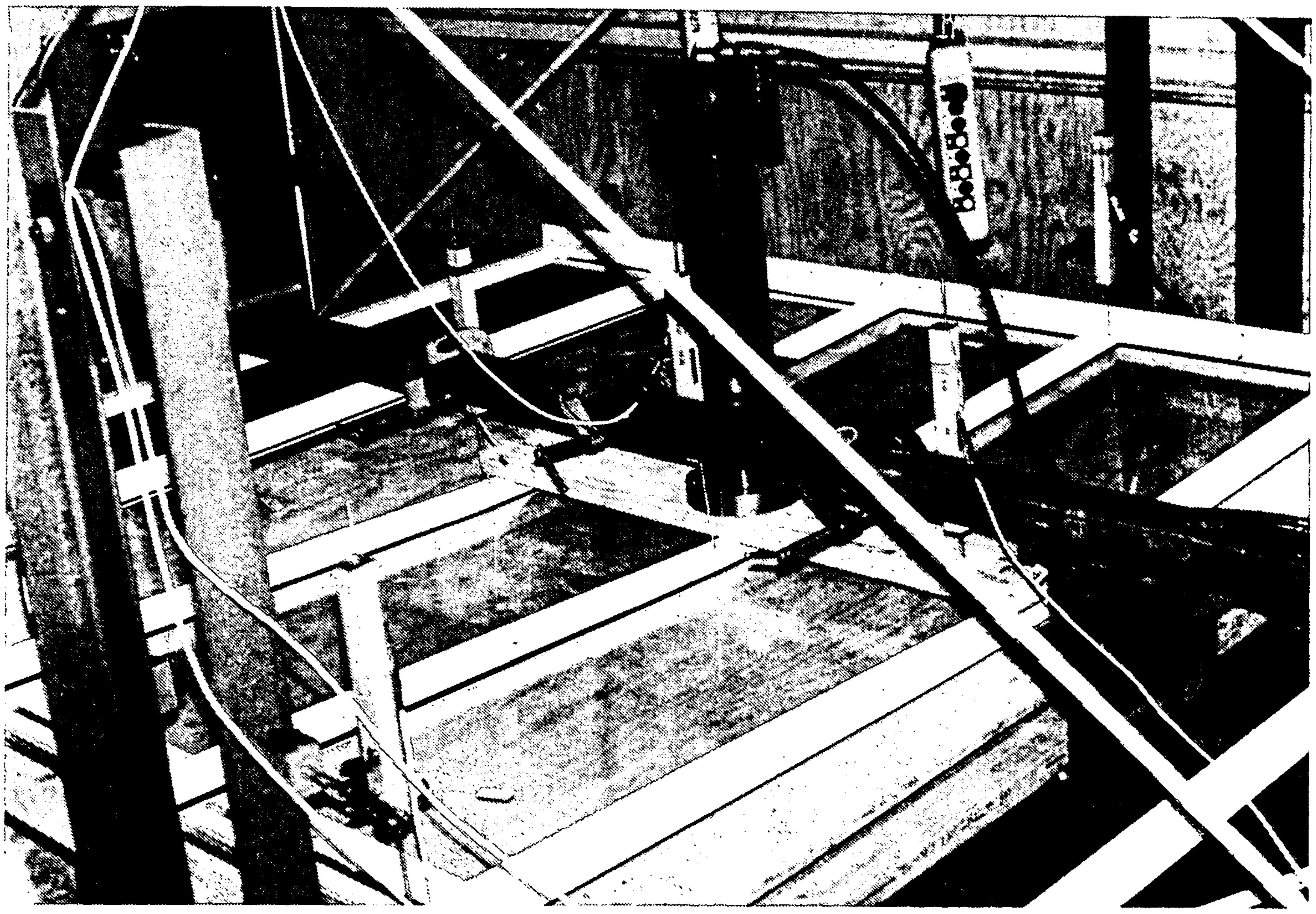


Figure 15 : A View of the Roof Section Prior to the Application of a Distributed Maintenance Load

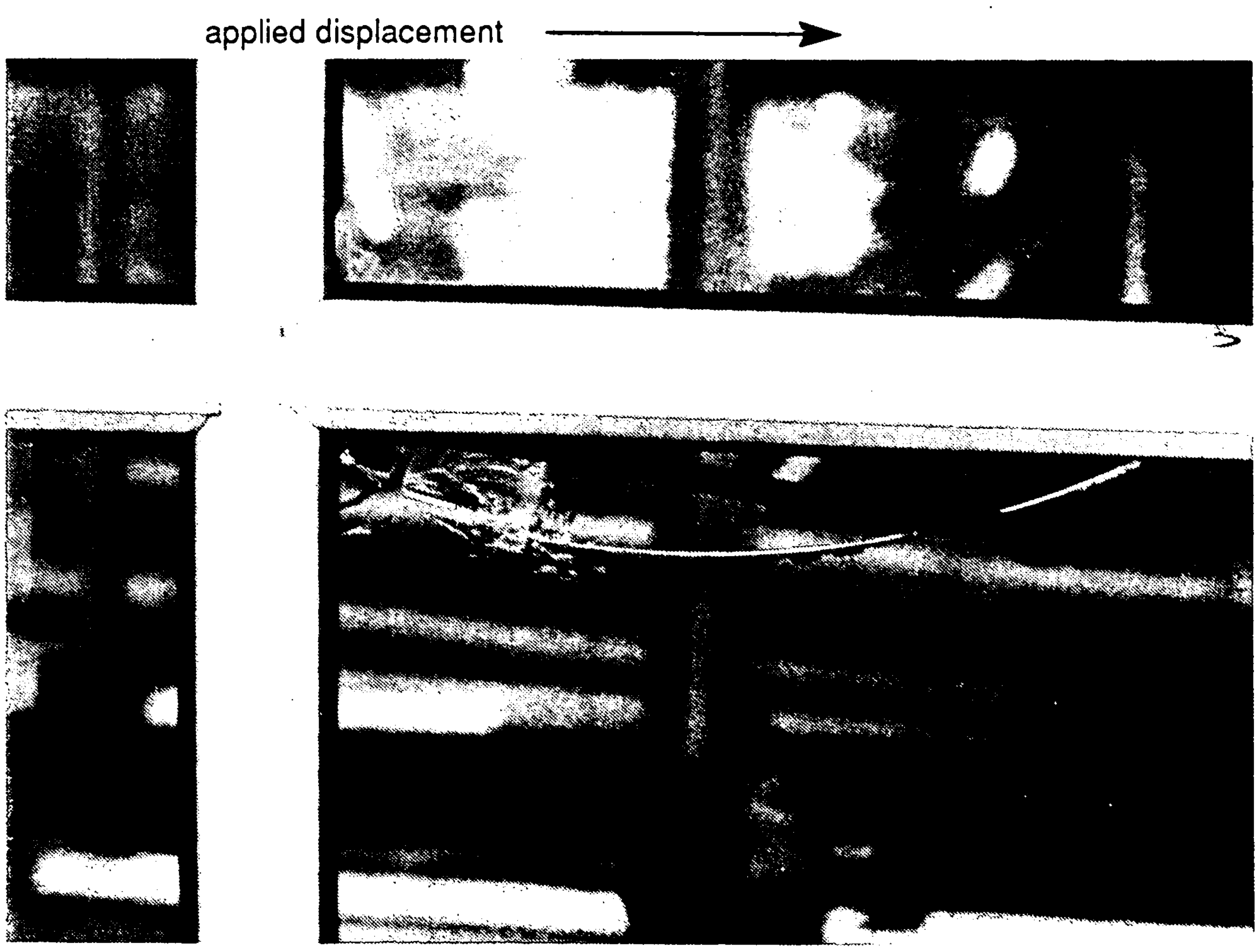


Figure 16 : A View of Glass Failure by Racking

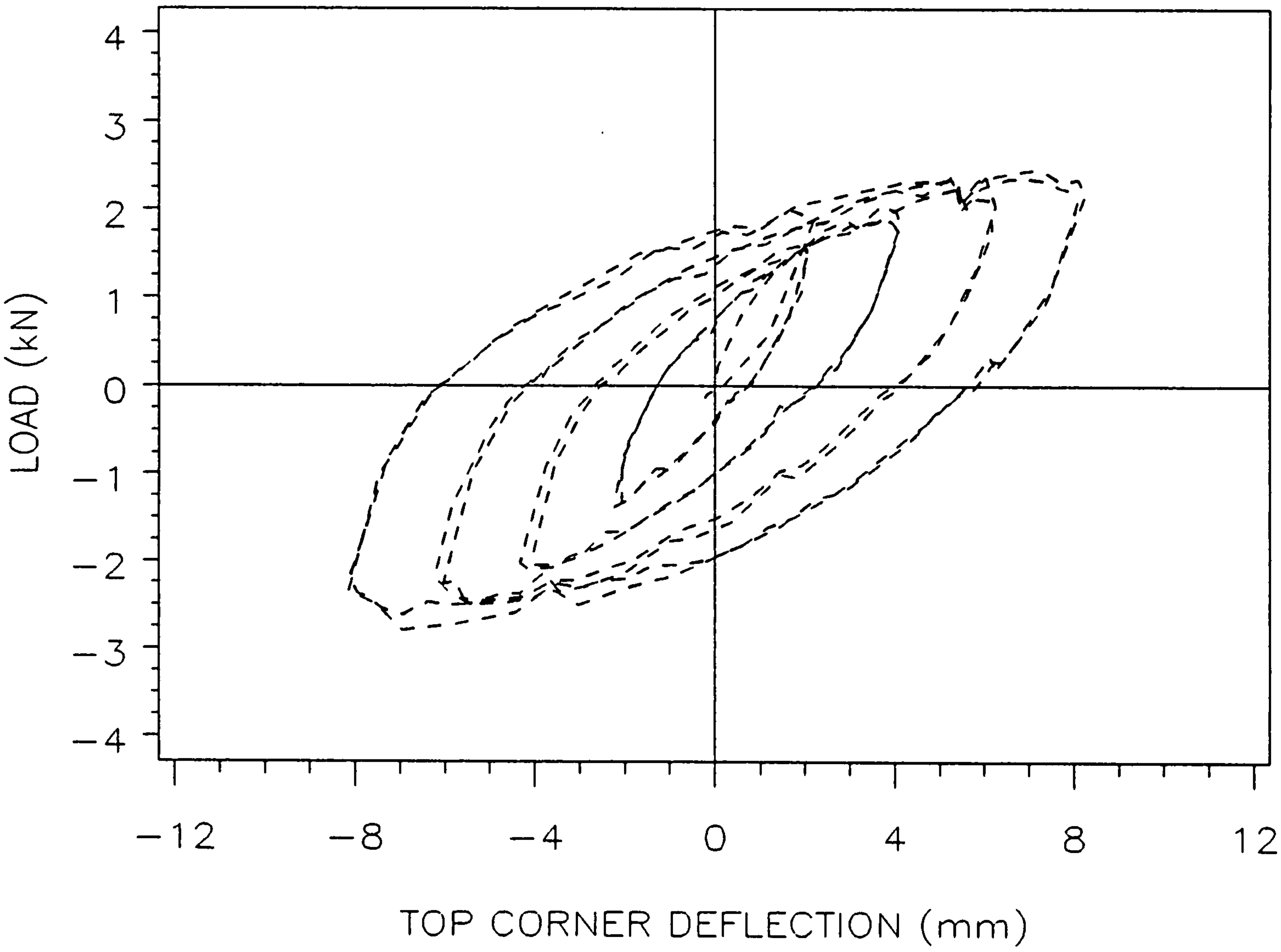


Figure 17a : Load - Deflection Plot for Test Conservatory, With a Fully Glazed Wall and No Roof Glazing

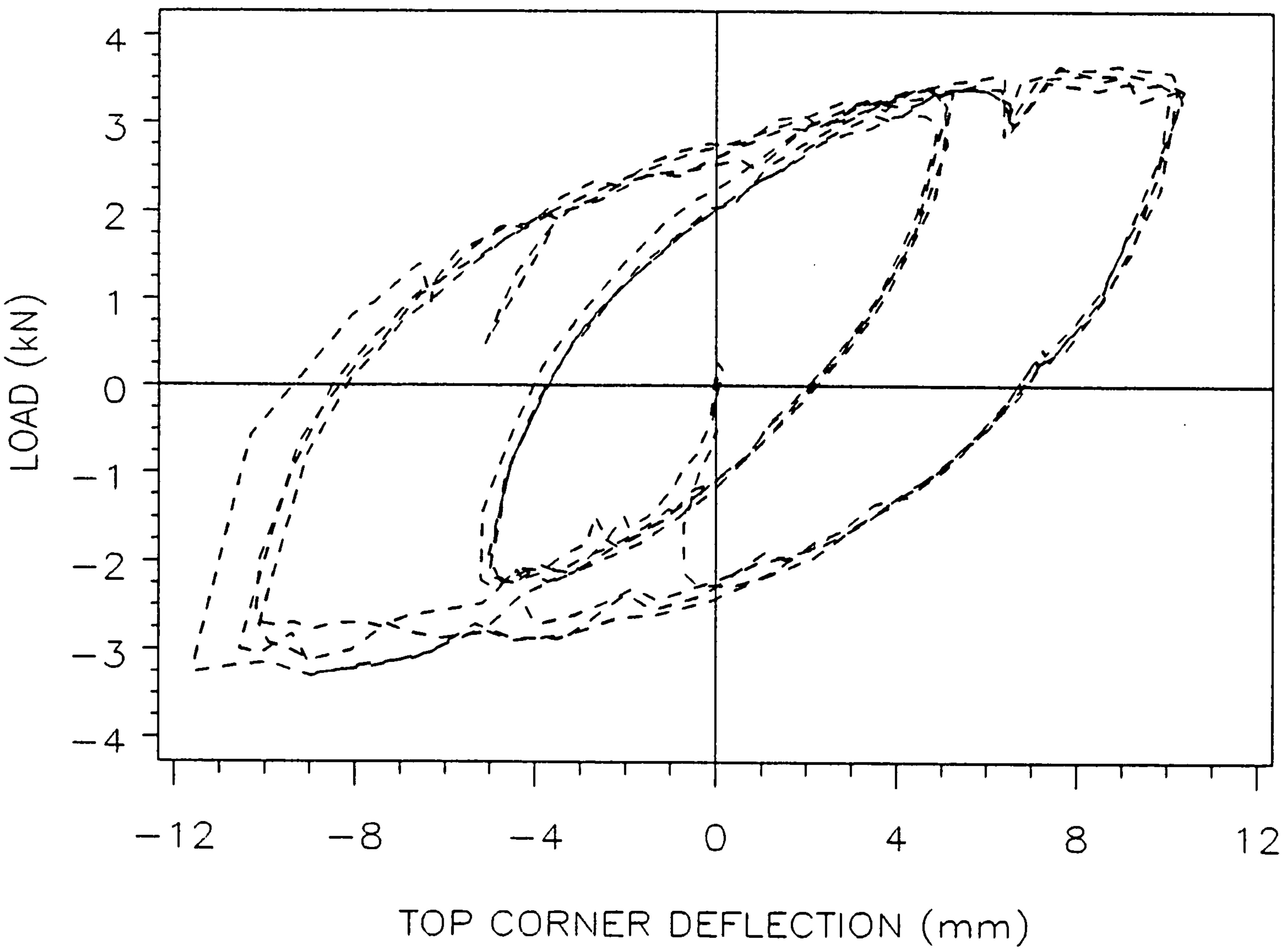


Figure 17b : Load - Deflection Plot for Test Conservatory, With a Fully Glazed Wall and Laminated Glass Roof

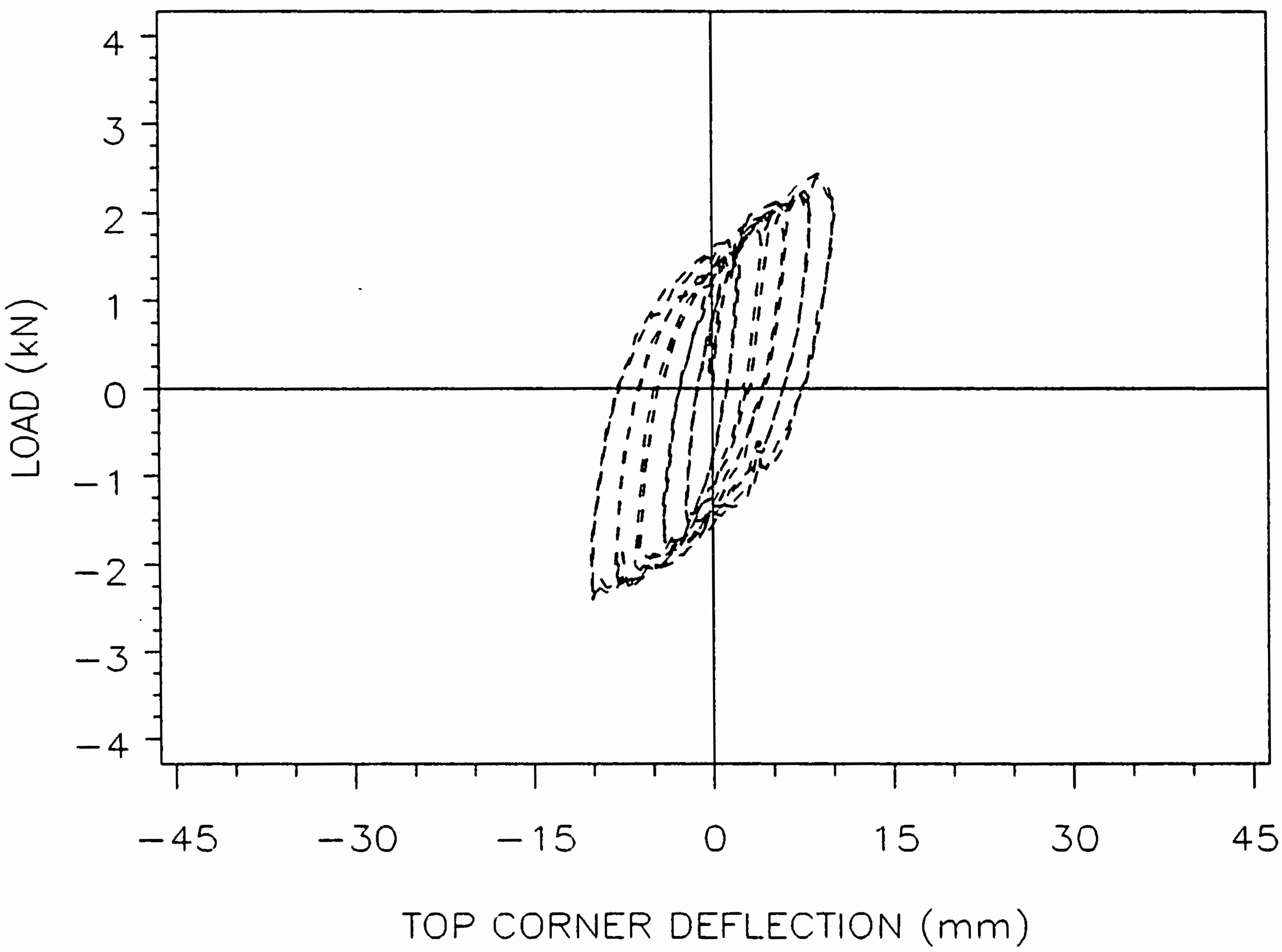


Figure 18a : Load - Deflection Plot for Test Conservatory, With a Twin Wall Polycarbonate Roof and Fully Glazed Wall

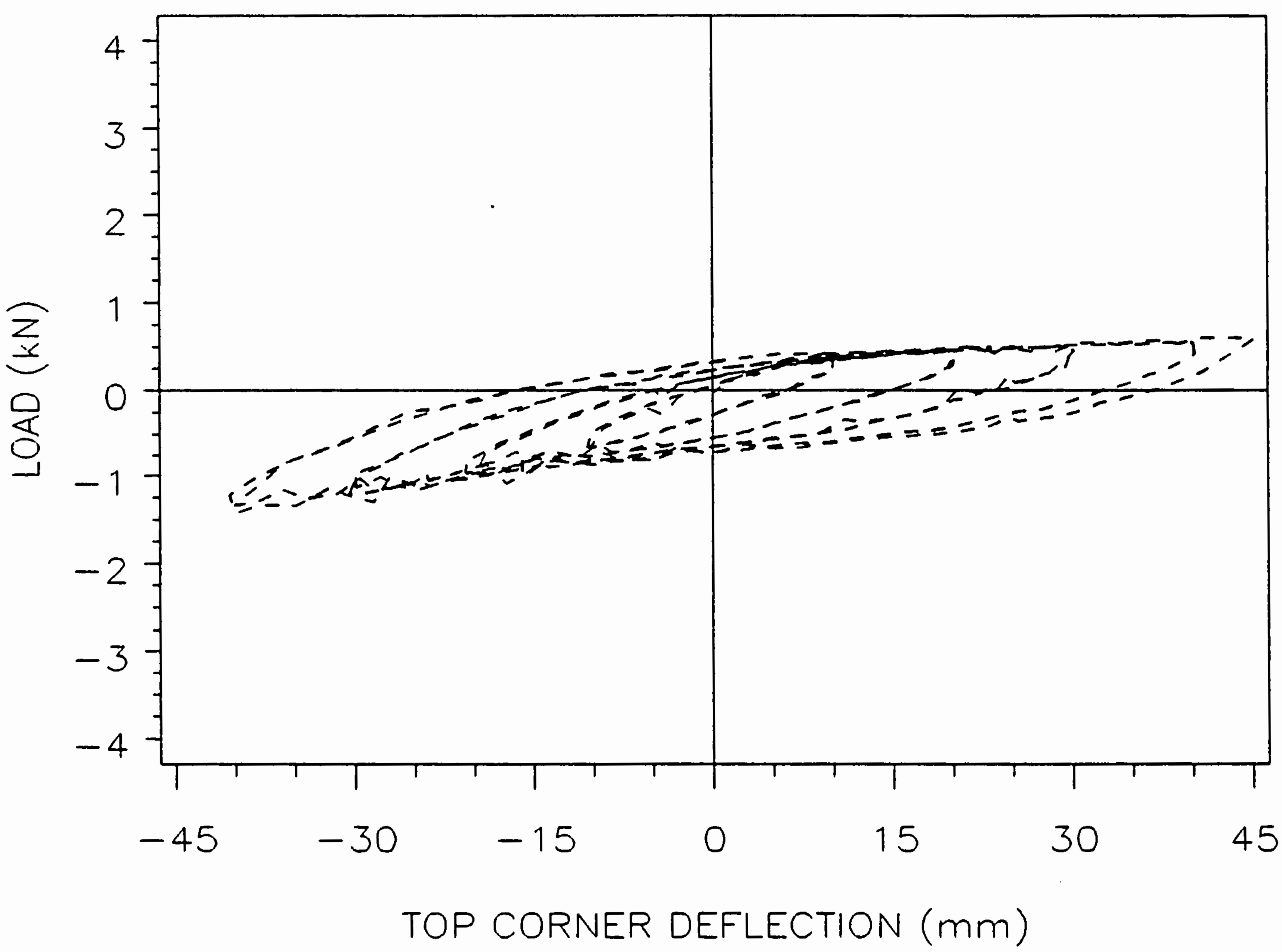


Figure 18b : Load - Deflection Plot for Test Conservatory, With a Twin Wall Polycarbonate Roof and No Wall Glazing

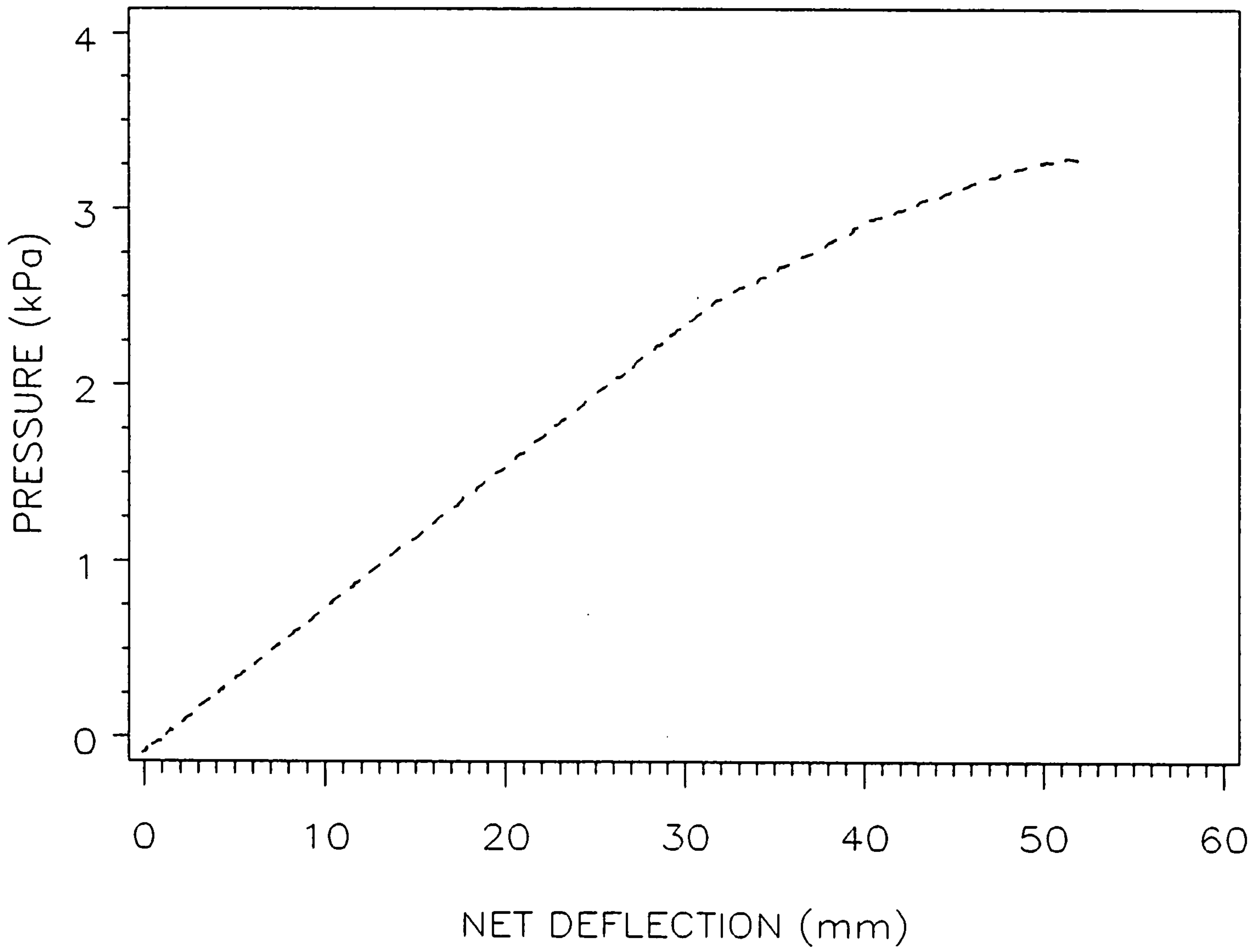


Figure 19: **Pressure Versus Mid-Span Deflection of Wall Mullion, During Face Load Test Under Positive Pressure From Exterior Side**

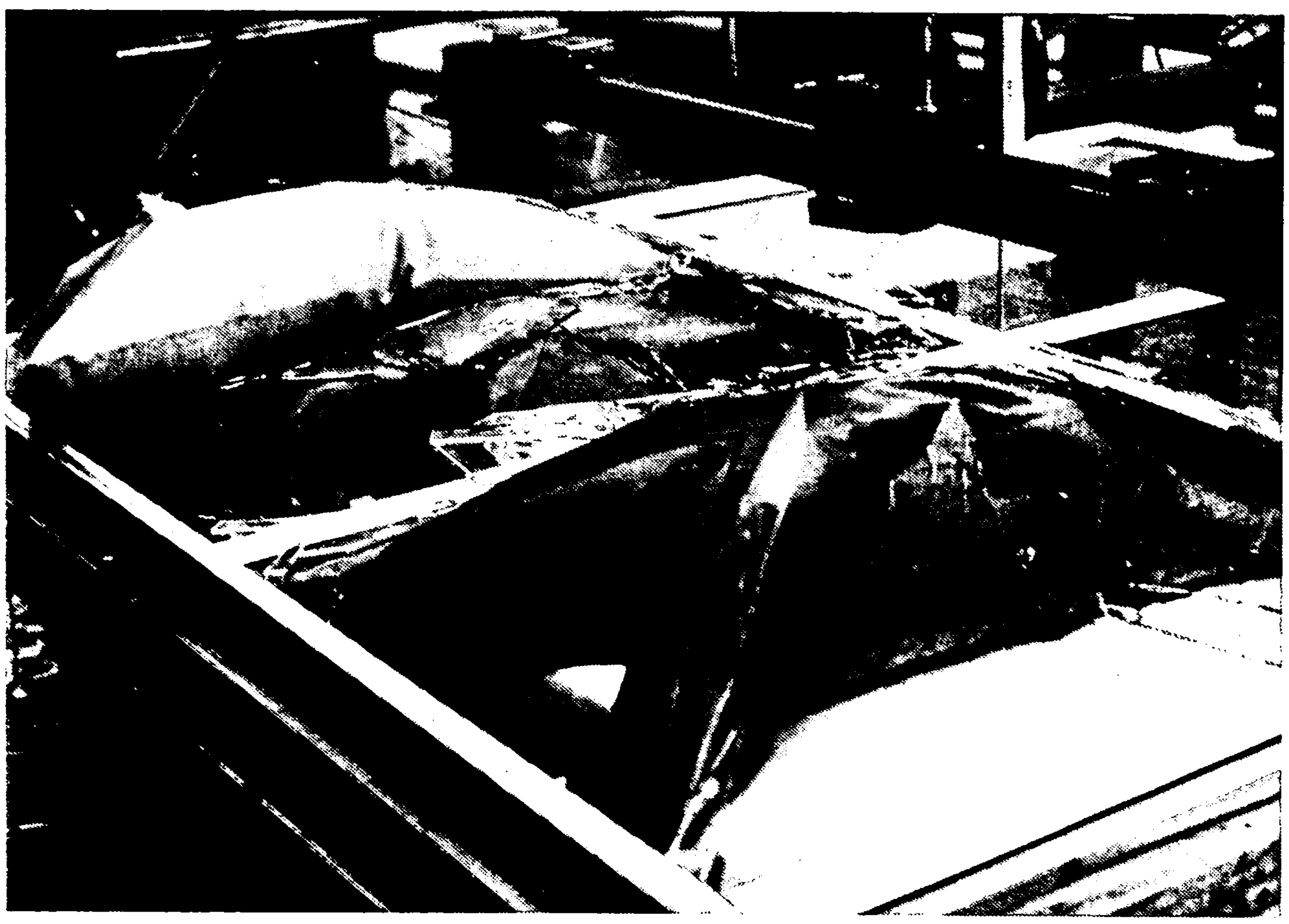


Figure 20: **A View of the Glass Failure During Face Load Test on the Wall**

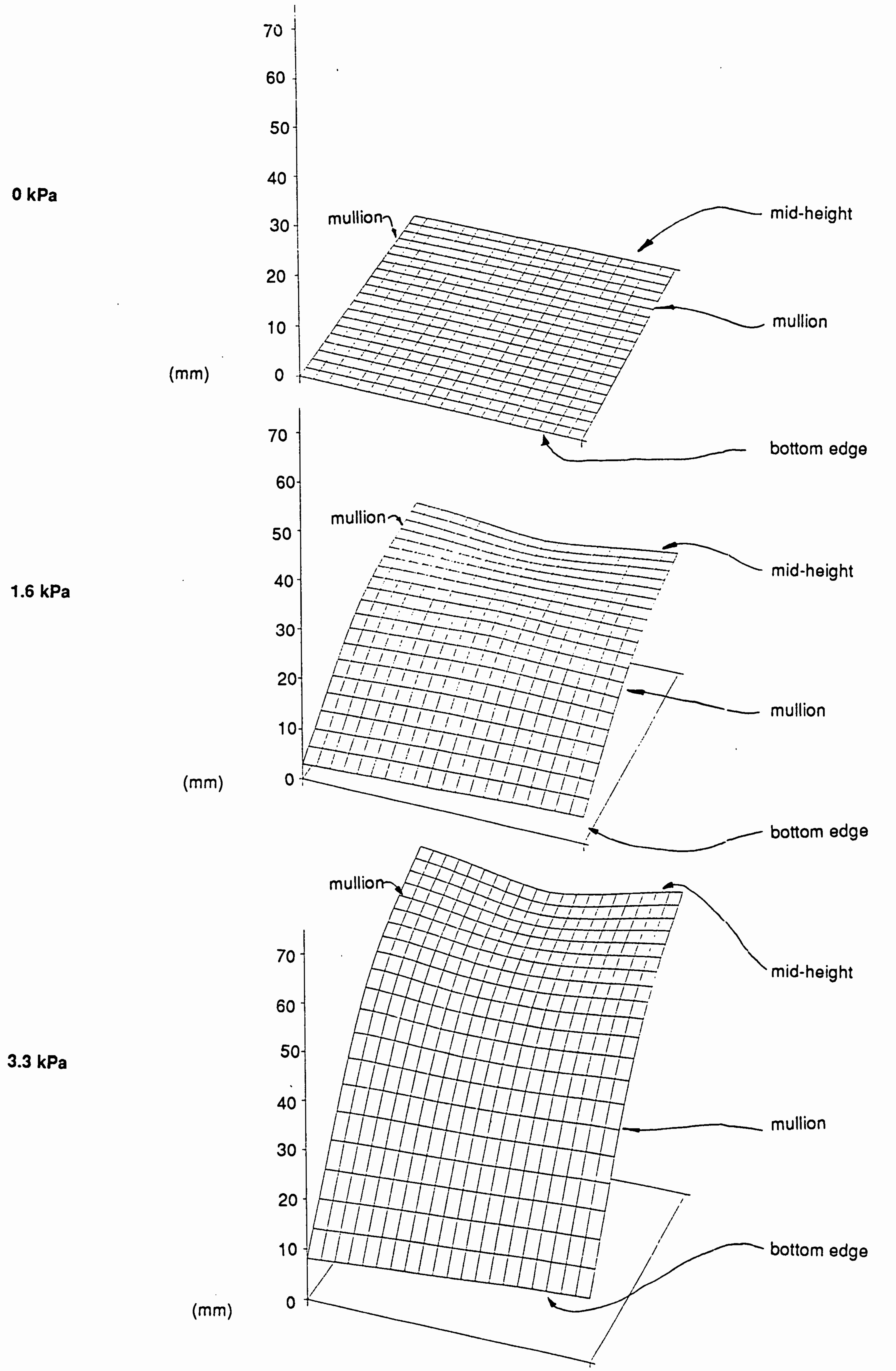


Figure 21 :Deformation of a Typical Glass Pane and Frame in the Lower Half of the Wall during a Face Load Test

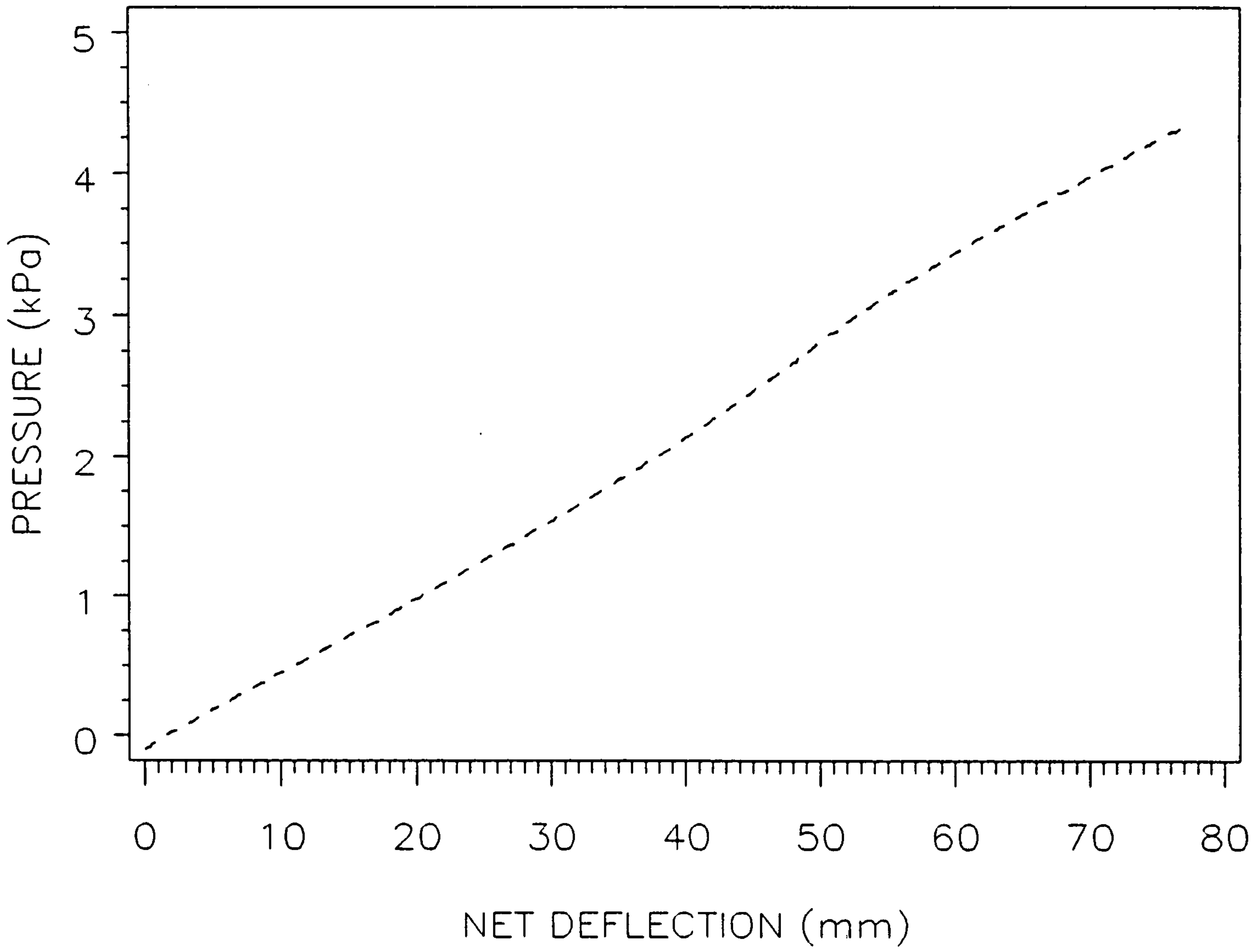


Figure 22: Pressure Versus Mid-Span Deflection of Rafter, During Face Load Test Under Negative Pressure From Exterior Side 6 mm Laminated Glass

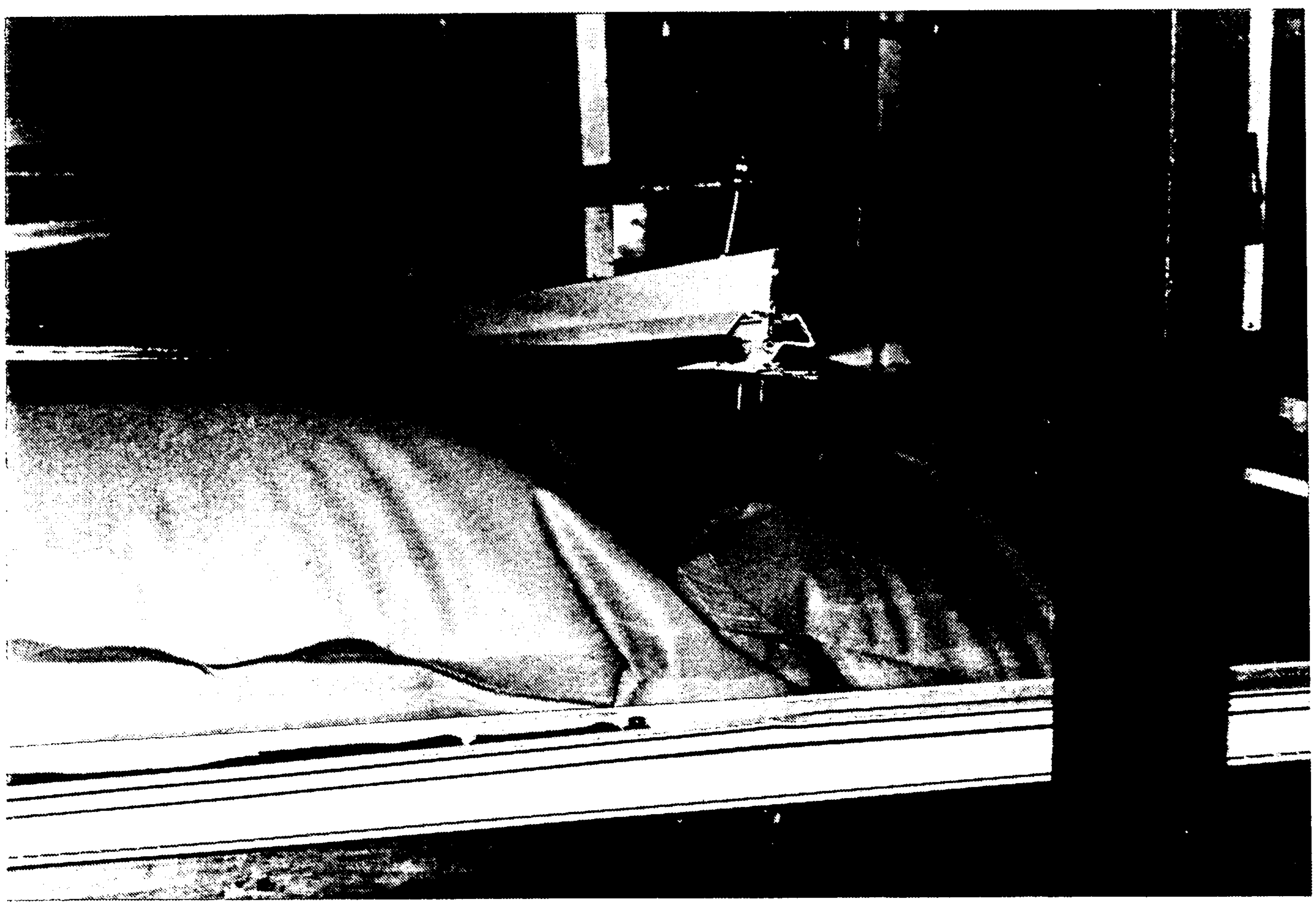


Figure 23: A View of the Failure During a Face Load Test on Roof with 6 mm Laminated Glass

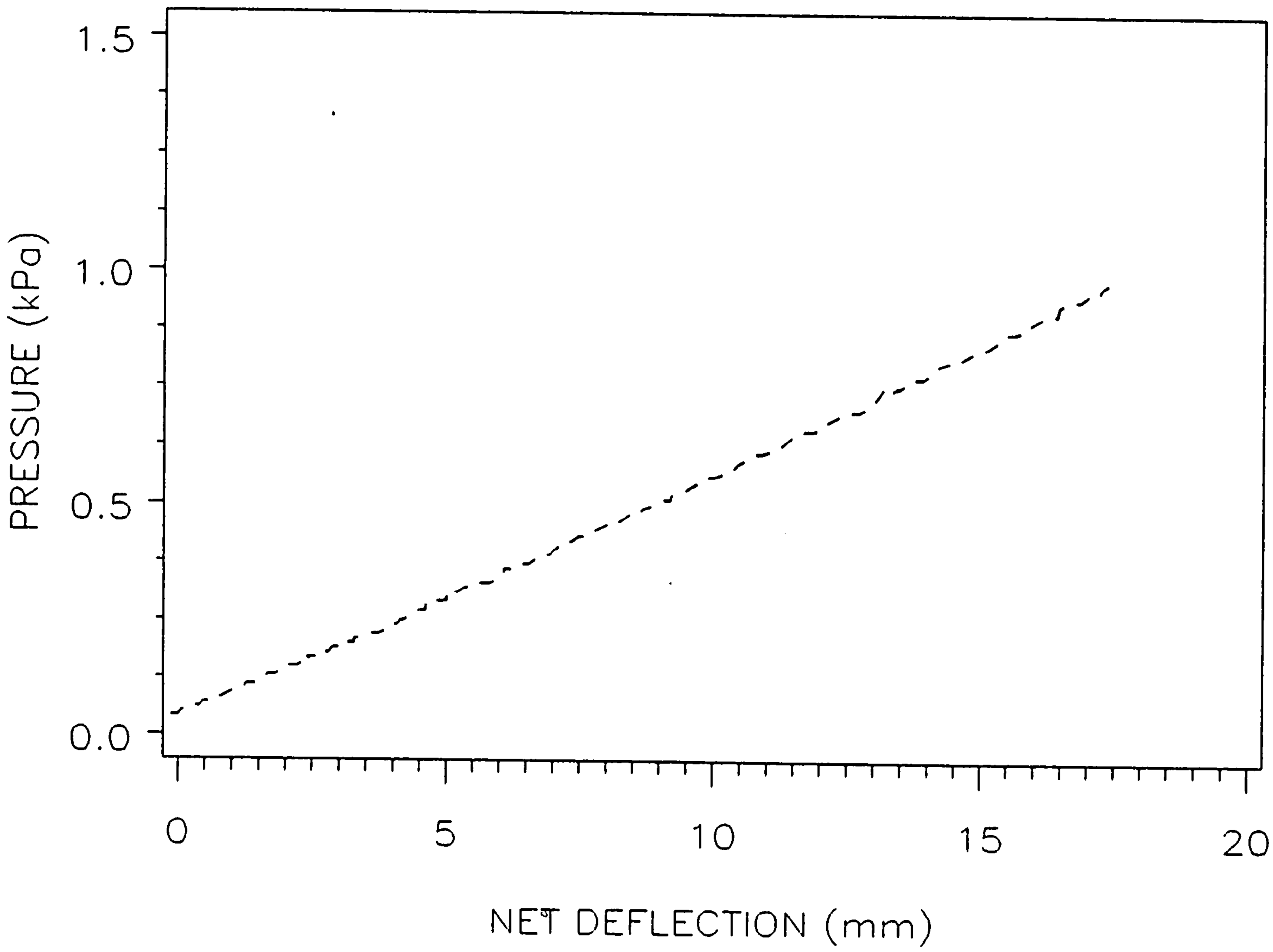


Figure 24: Pressure Versus Mid-Span Deflection of Rafter, During Face Load Test Under Negative Pressure From Exterior Side 6 mm Hollow Core Twin Wall Polycarbonate

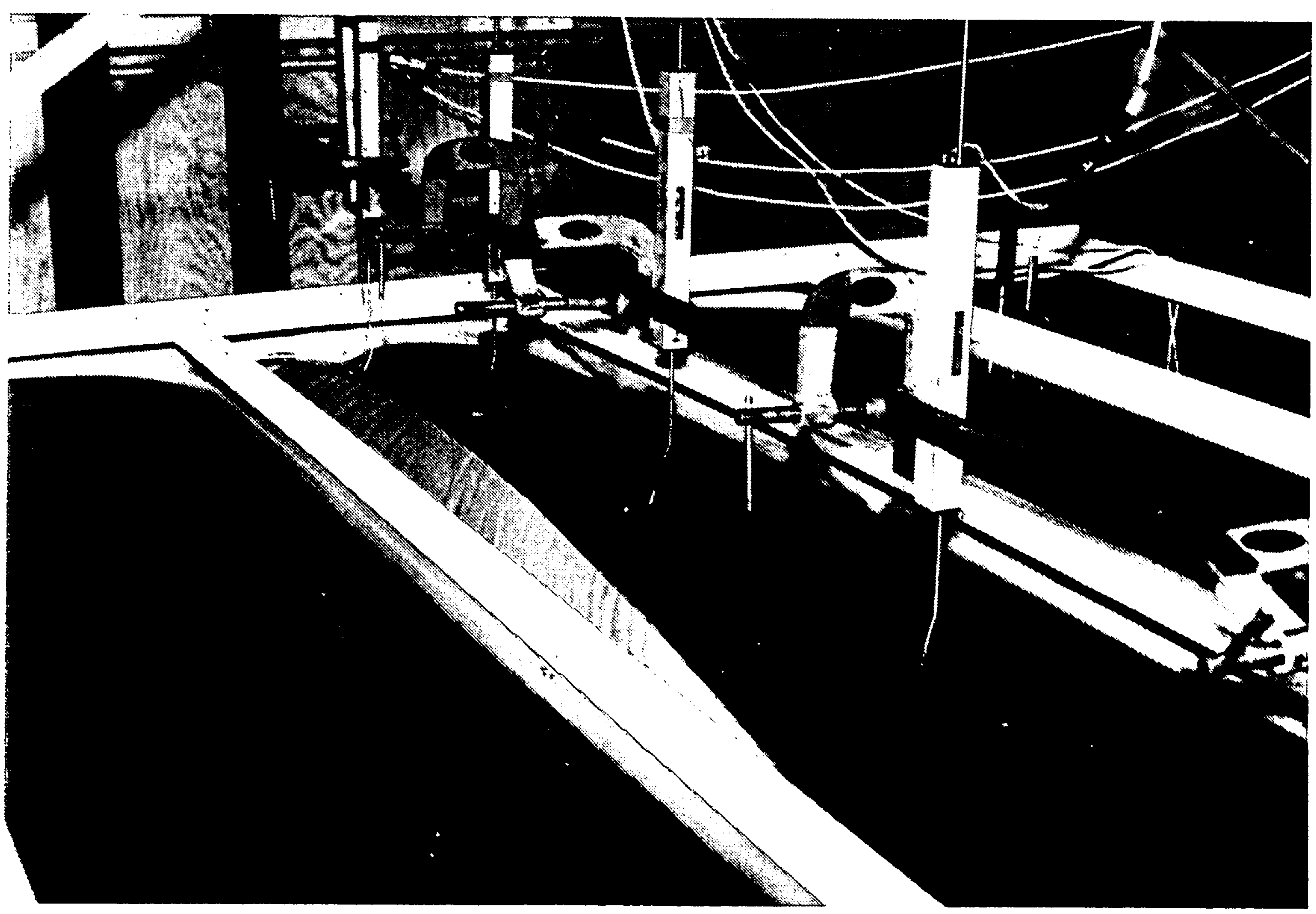


Figure 25: A View of the Polycarbonate Glazing popping out along the edge of the Rafter

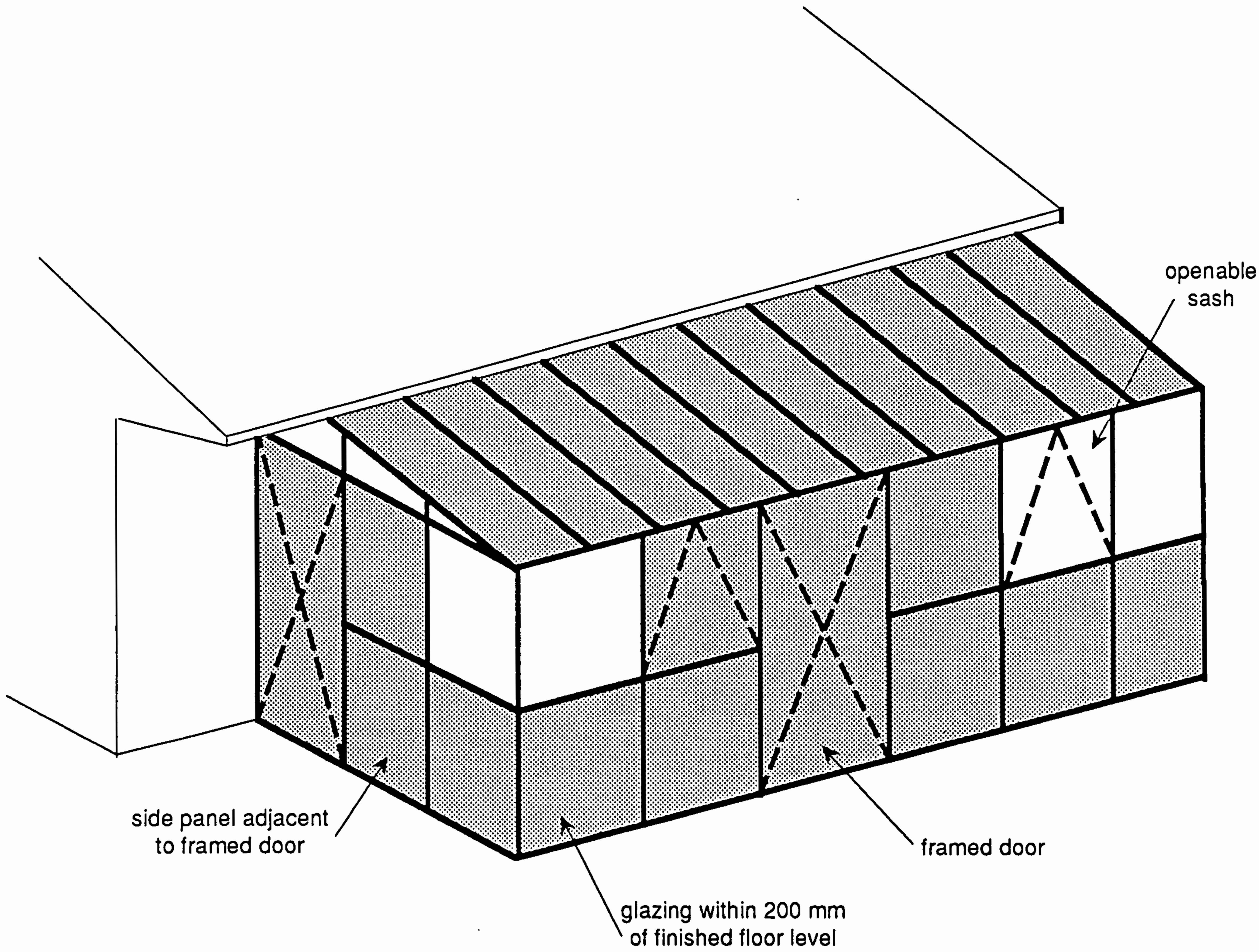


Figure 26 : Areas (shaded) in a Typical Lean-to Conservatory which require special provisions against human impact to comply with NZS 4223

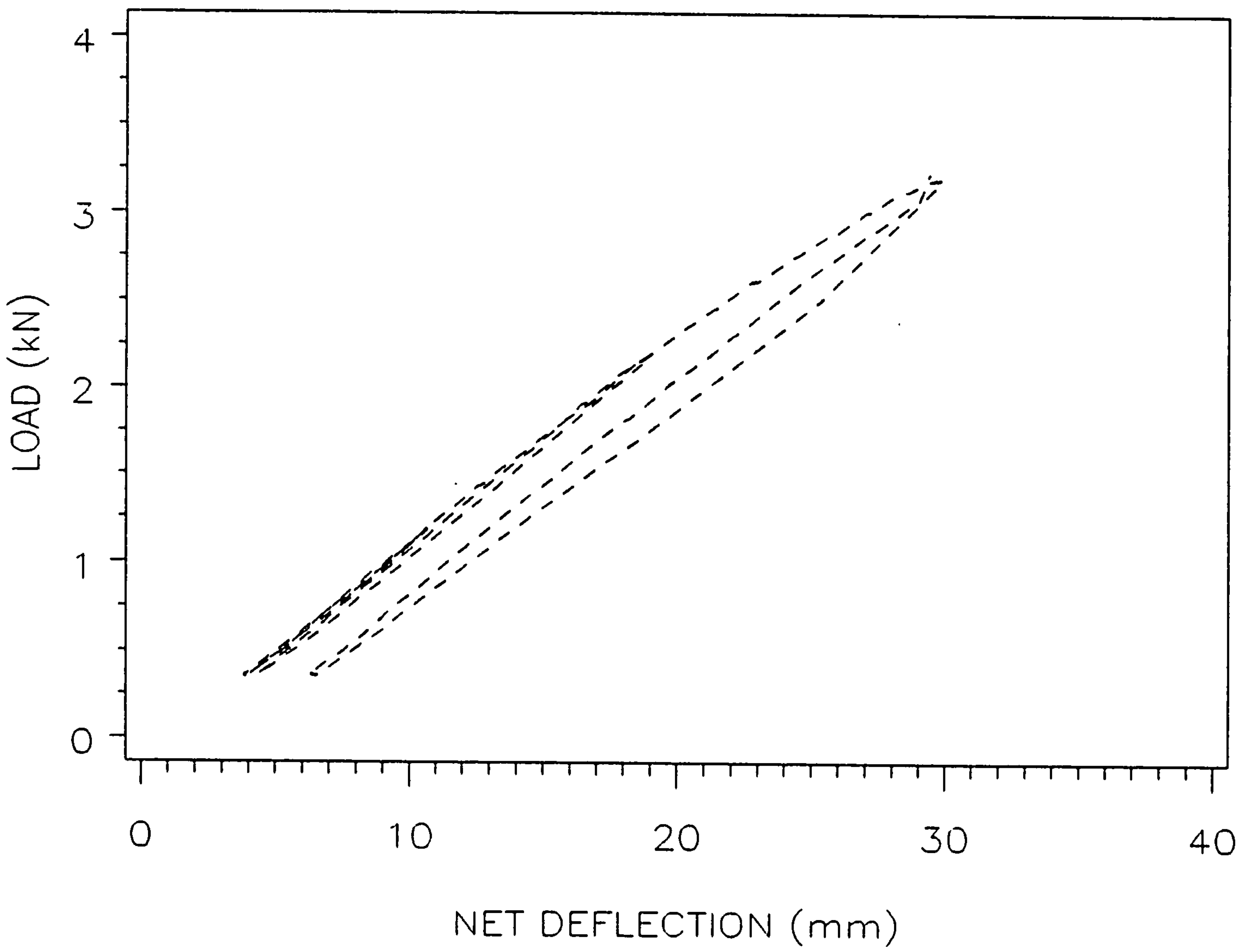


Figure 27: **Load Versus Deflection of Central Rafter, Under Concentrated Load Distributed Across Three Rafters**

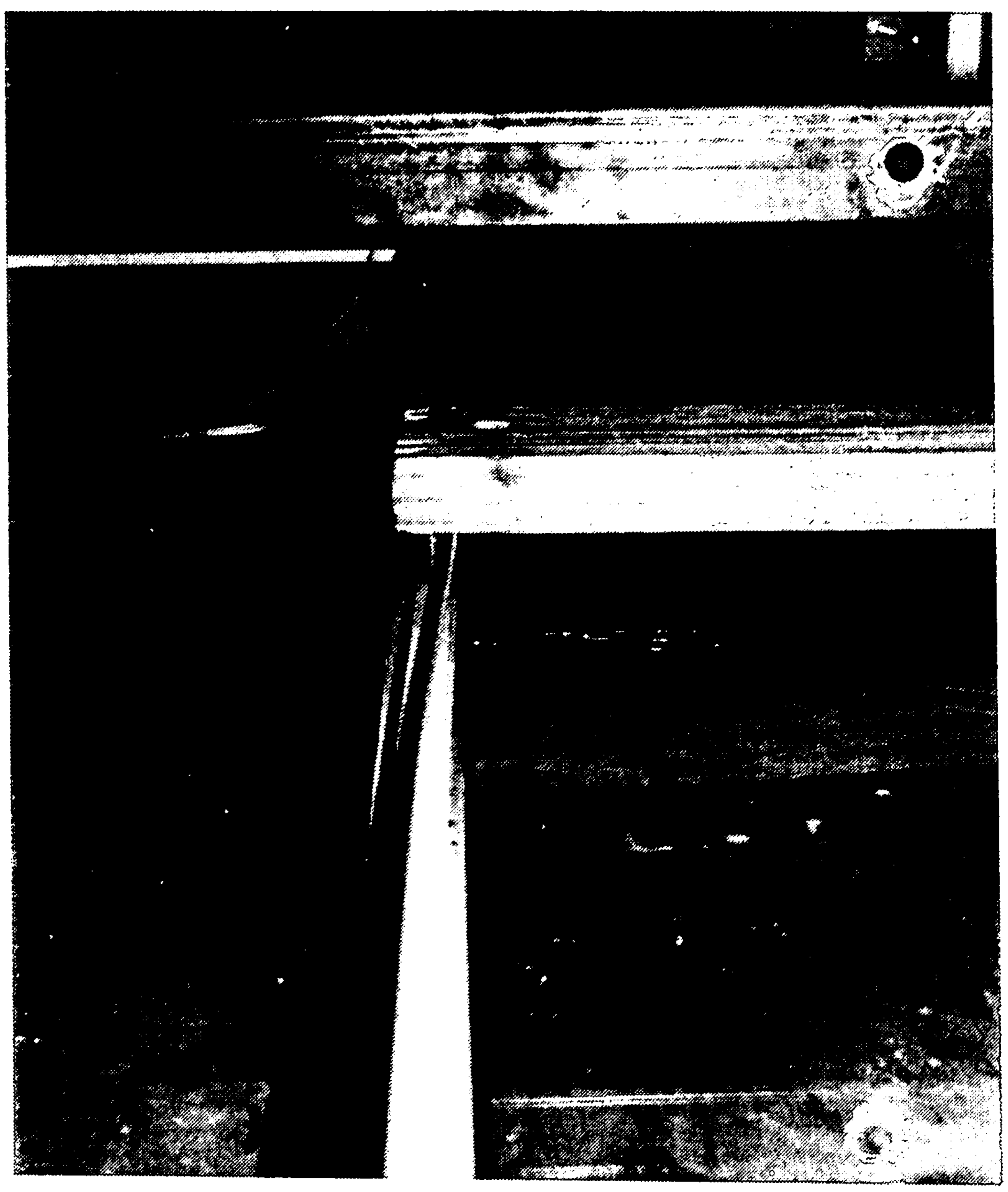


Figure 28: **A View of Buckling of a Side Rafter Beneath a Distributed Maintenance Load**

BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

To promote better building through the application of
acquired knowledge, technology and expertise.

HEAD OFFICE AND RESEARCH STATION

Moonshine Road, Judgeford
Postal Address - Private Bag, Porirua
Telephone - (04) 357-600, FAX 356-070

REGIONAL ADVISORY OFFICES

AUCKLAND
Telephone - (09) 5247-018
FAX - (09) 5247-069
290 Great South Road
PO Box 17 -214
Greenlane

WELLINGTON
Telephone - (04) 357-600
FAX 356-070
Moonshine Road, Judgeford

CHRISTCHURCH
Telephone - (03) 663-435
FAX - (03) 668-552
GRE Building
79-83 Hereford Street
PO Box 496

DUNEDIN
Telephone - (024) 740-454
FAX - (024) 778-436
6 Hanover Street
PO Box 1635

ISSN 0113 - 3675

B R A N Z

THE RESOURCE CENTRE FOR BUILDING EXCELLENCE