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THE DEVELOPMENT OF A PROCEDURE AND RIG FOR TESTING THE RACKING RESISTANCE OF CURTAIN WALL GLAZING

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Preface

The Building Research Association of New Zealand commissioned this report from WORKS Technical Services to assist in the development of a procedure and test rig for conducting racking tests on curtain wall glazing systems. The views represented are not necessarily those of the Association.

Acknowledgements

The author wishes to acknowledge the help and support of Adrian Bennett, BRANZ scientist responsible for technical liaison on this contract.

This report is intended mainly for glazing manufacturers, structural engineering consultants and architects.

THE DEVELOPMENT OF A PROCEDURE AND RIG FOR TESTING THE RACKING RESISTANCE OF CURTAIN WALL GLAZING

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REFERENCE

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KEYWORDS

From Construction Industry Thesaurus - BRANZ edition: Deflection; Deformation; Curtain Walls; Dynamic Loads; Glass; In-Plane; New Zealand; Performance Concepts; Racking Forces; Racking Strength; Structural Design; Test Procedures; Testing Equipment.

ABSTRACT

For some years the construction industry has been installing curtain wall glazing systems on multi-storey buildings. The New Zealand loadings code has requirements for the separation of non-structural building elements from the building skeleton, to prevent damage during a seismic event. Until now, there has been no procedure for the testing of curtain wall glazing systems under racking loads, to assess their performance.

A literature survey conducted on the subject failed to find sufficient useful information on which to build a test procedure, so an investigation was undertaken on likely building deformations and code deformation requirements. This work has culminated in the design of a testing procedure and test rig capable of racking most available curtain wall glazing systems.

C O N T E N T S

FIGURES

INTRODUCTION

This report contains the results of Phase I of a research contract conducted by WORKS Technical Services, Central Laboratories, for the Building Research Association of New Zealand (BRANZ).

Prior to letting this contract, BRANZ had determined from literature surveys that the racking performance of curtain wall glazing systems did not appear to have been addressed in depth anywhere in the world. Recent visits to the earthquake prone West Coast of North America by BRANZ researchers confirmed that, so far, little has been done to investigate the problem there.

In this study it has been necessary to consider the types of in-plane deformation that can be imposed on a glazing system, and to determine the significance of these deformations on the overall performance of the glazing system. These aspects are addressed in this report under the headings Building Deformations, and Code Requirements.

By surveying consulting engineering firms and glazing system manufacturers, the commonly used procedures for calculating deformations and the information consequently supplied to manufacturers on inter-storey drift have been obtained.

The information contained in the first two parts of this report, in combination with computer studies of model buildings, has been used to formulate an appropriate test procedure and test rig design. Details of these are presented in the second half of this report.

In designing the procedure and rig, it has been necessary to restrict the scope to consideration of building deformations due to earthquake only. Deformations due to other causes (e.g., gravity load, settlement, thermal) may erode clearances provided for earthquake movement. Conversely, the earthquake performance may be enhanced by clearances provided for other reasons (e.g., erection tolerances, thermal).

LITERATURE SURVEY

At the outset of the project, listings of possibly relevant publications were supplied by BRANZ. These were obtained by searching the COMPENDEX, BRIX and PICA databases using keywords appropriate to this study. Further searches were made of the MWDINFO and COMPENDEX databases available through the MWD Central Library.

A large proportion of the references were eliminated as being irrelevant on inspection of the abstracts. Copies of the remainder were obtained for further inspection. However, nothing of use for this project was discovered. It is worthwhile noting that the majority of the references related to the out-of-plane performance of curtain wall glazing systems, especially with respect to wind loading and weathertightness.

BUILDING DEFORMATIONS

General

The range of deformations occurring in structures supporting curtain wall glazing when subjected to earthquake loading is considerable. The building response, structural form and geometry, and the manner in which the glazing is fixed to the structure will all contribute to the deformations imposed on the glazing and its ultimate aseismic performance.

The weight of the glazing system itself will contribute extra inertial loads on fixings and also deformation of the glazing bars, particularly if heavy architectural panels are incorporated.

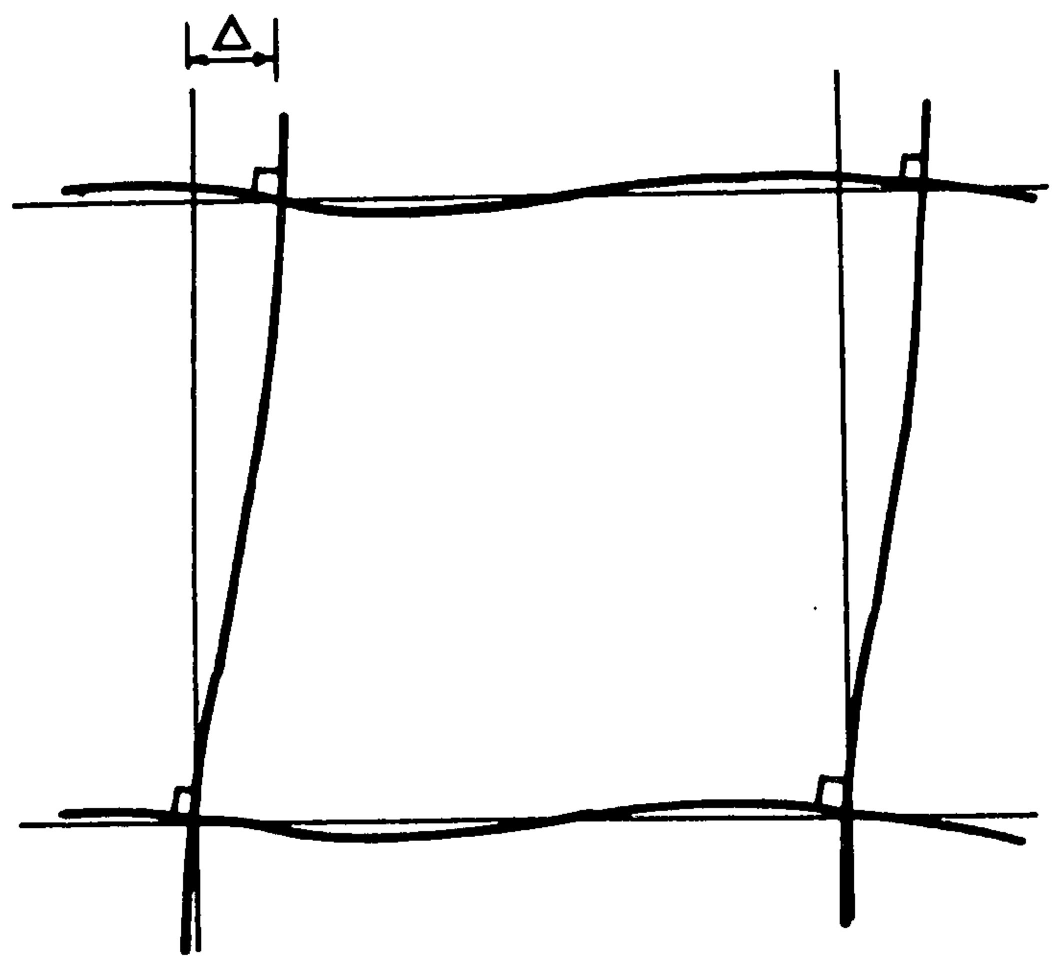
Curtain walling, while providing a sheer exterior, is constructed from individual components that are themselves not rigidly fixed together. The size of panel that can be employed is limited by transportation and erection requirements, and clearances are provided to accommodate thermal and seismic movement, building gravity load movement, and construction tolerance. Consequently, localised rather than gross building deformations will be of greatest significance, and only those occurring over a height of one or two storeys need be considered.

Primary Deformations

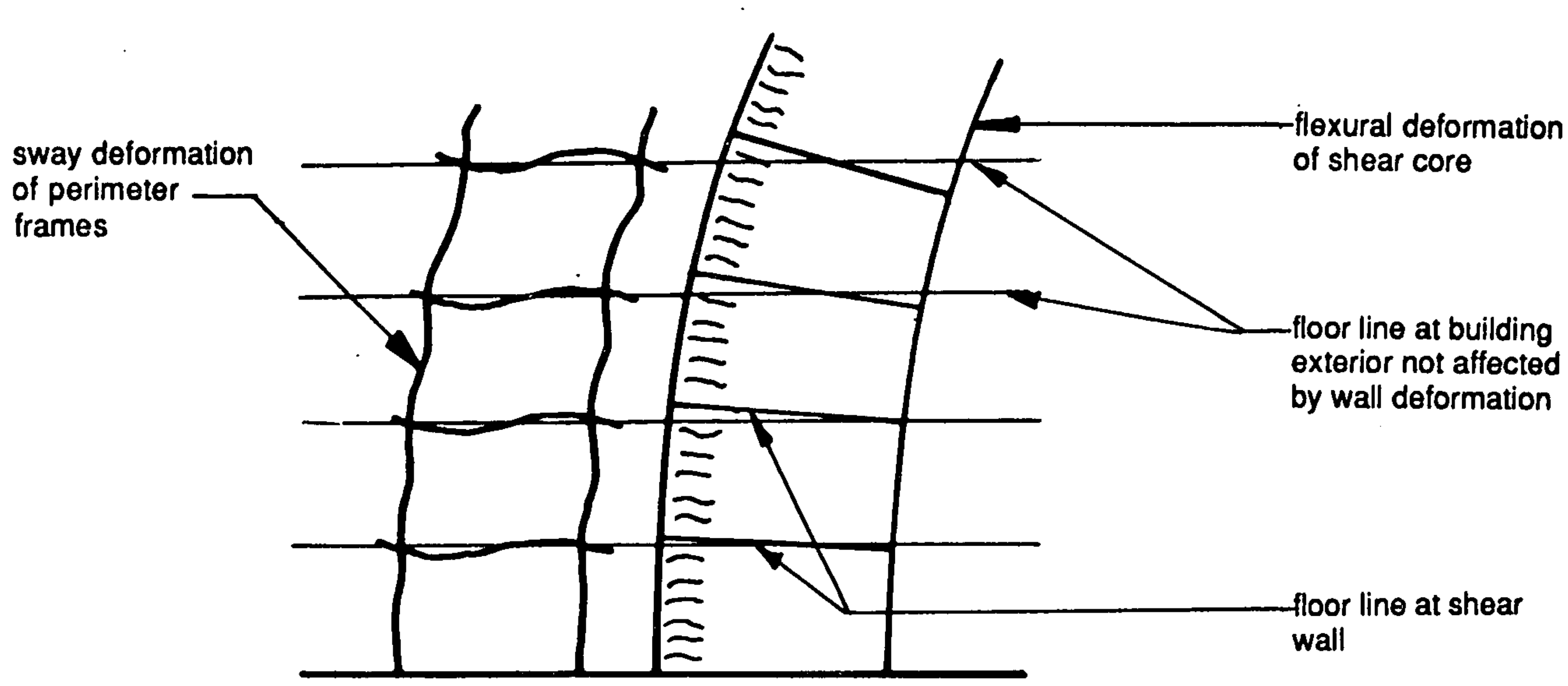
The principal structural deformation (and hence the most significant in terms of glazing performance) that occurs during earthquake shaking is storey sway. This is the lateral displacement of one level with respect to those adjacent, with the floors remaining essentially parallel to each other, resulting from predominantly flexural deformation of the building frame - see Figure 1(a).

This mechanism will apply to those structures which have frames at the building exterior supporting the curtain walling, regardless of whether lateral loads are resisted by frame action or shear walls - see Figure 1(b).

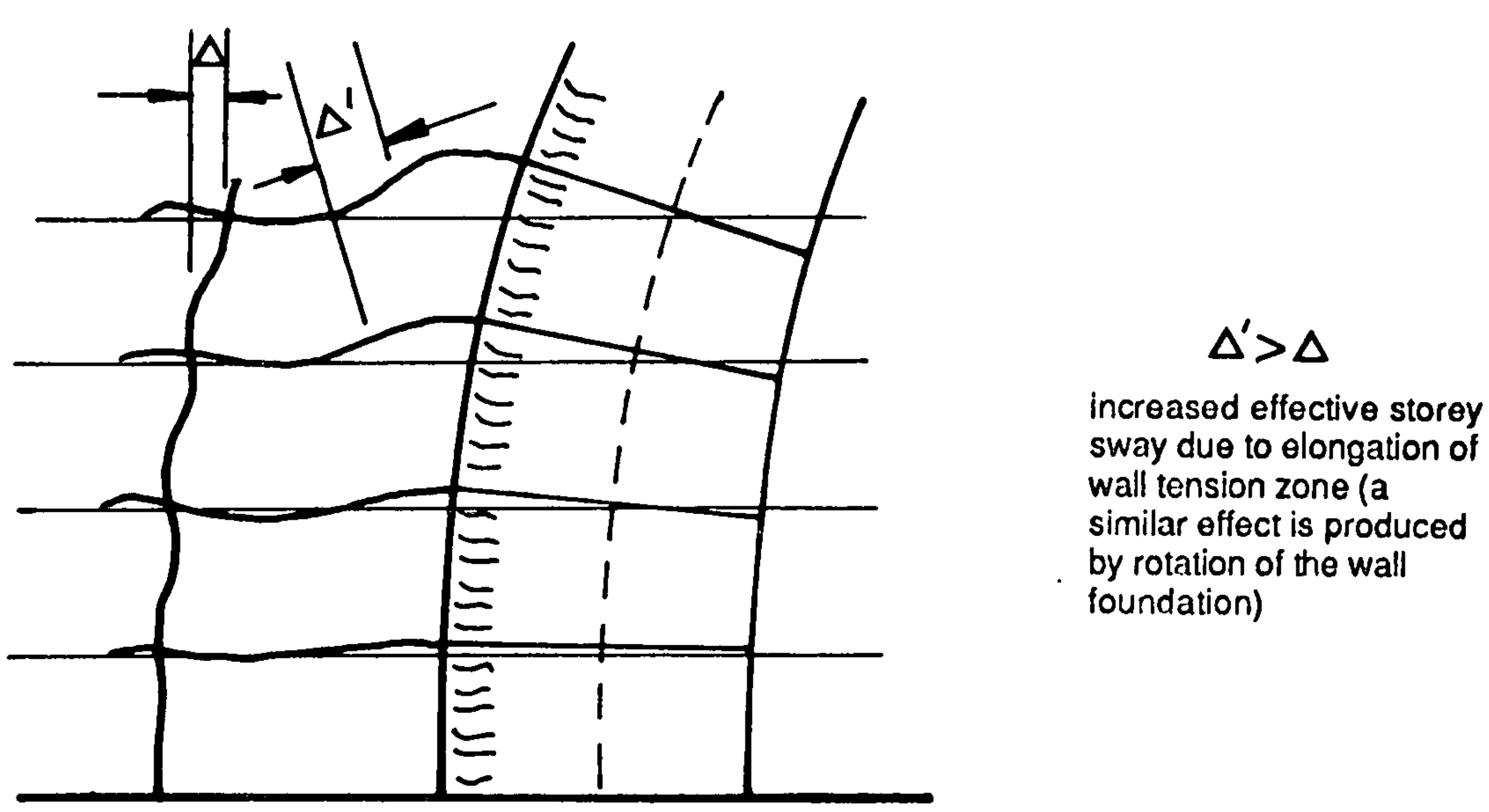
Significant vertical deformations can result from large flexural strains in the tension region of shear walls, producing an effective increase in the inter-storey deflection, Δ, of adjacent framed bays. This, however, will only influence the performance of curtain walling where the shear walls are at the outside of the building - see Figure 1(c). A similar effect will occur in the end columns of moment resisting frames where the earthquake induced axial force will produce elongation or shortening. Foundation rotation will also have this effect if the foundation of the intended lateral load resisting system is able to move with respect to the remainder of the structure. The effective variations in curtain wall racking that can be produced in this manner are similar to those resulting from secondary frame deformations, as shown in Figure 2. (But in this case the racking variations are generated by differential vertical movement of adjacent columns.)



(a) Frame sway deformation

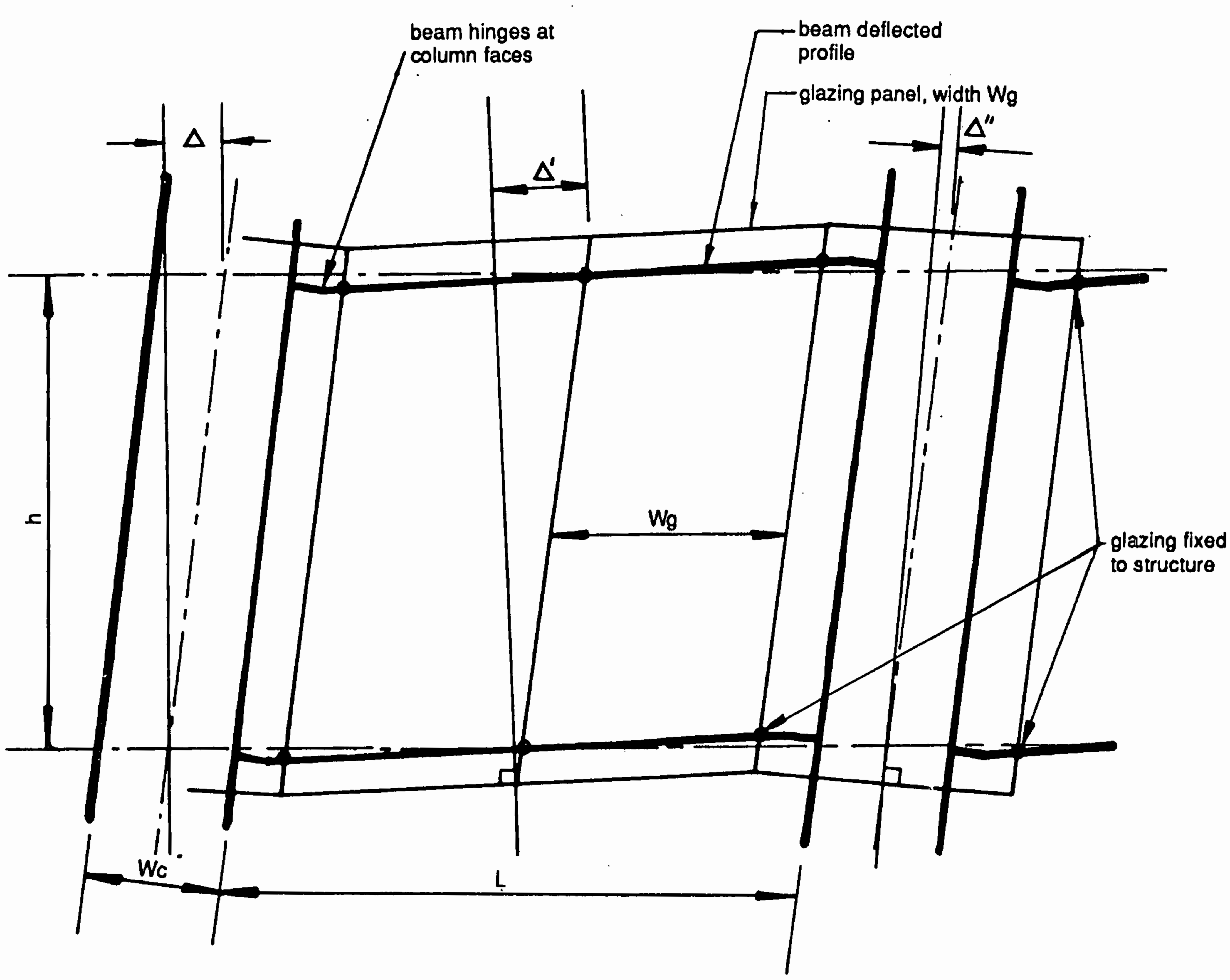


(b) shear core building - does not increase effective sway of perimeter frames



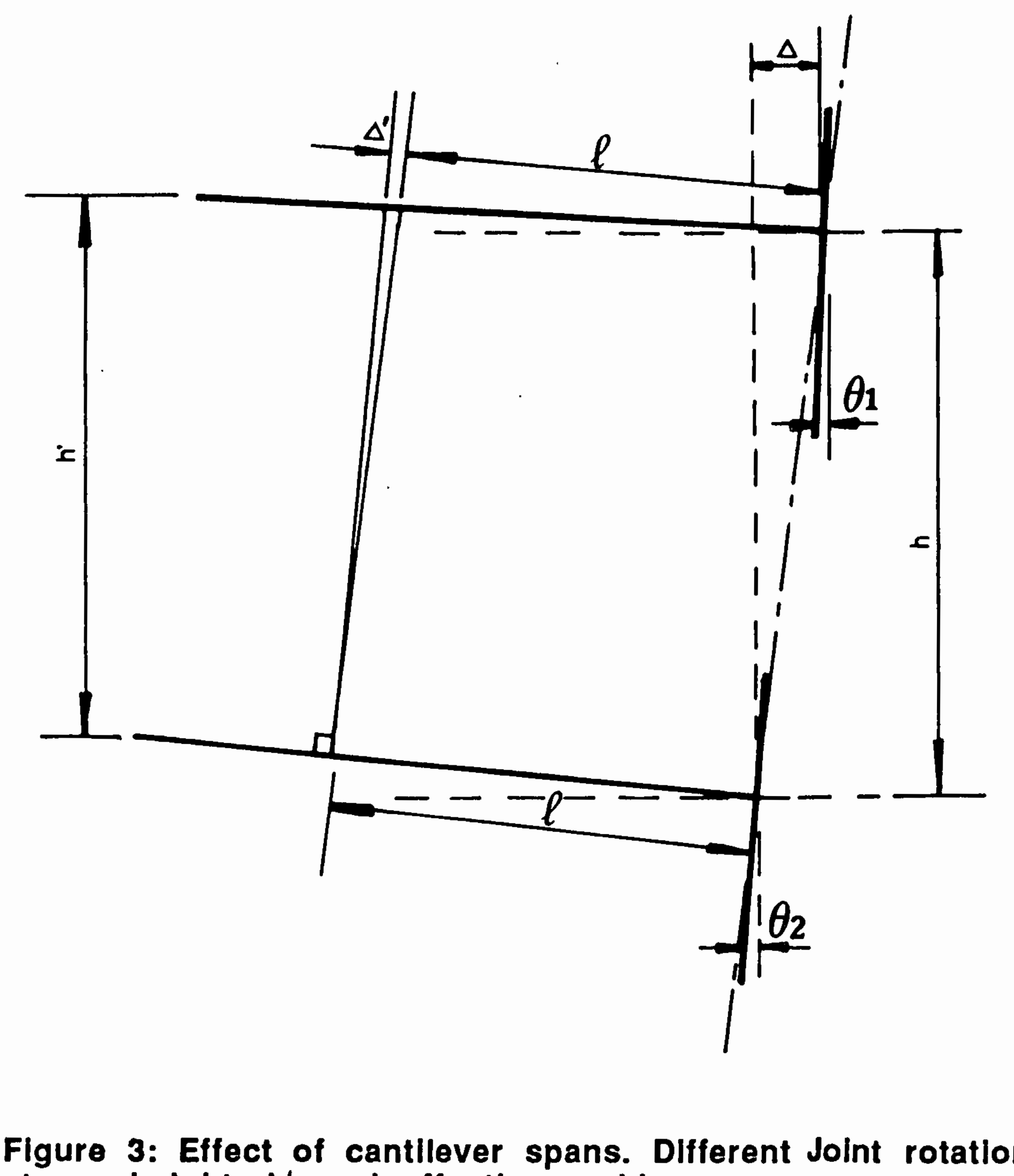
(c) Perimeter shear wall - produces increased effective sway of adjacent framed bays

Figure 1: Effect of building type on deformation



Δ= interstorey deflection of building
Δ'= effective racking of beam mounted panels (> Δ)
Δ"= effective racking of panels fixed to or near columns (< Δ)
Δ"⟶ O as column width, Wc⟶Wg
Δ'⟶ Δ as Wc/L⟶ O (ie long spans)

Figure 2: Effect of column width and beam span on glazing racking



Δ= Interstorey Deflection
θ= Joint Rotation
If $\theta_1 < \theta_2$ then h' < h
$\Delta'/h = \Delta/h - \theta_2$
as $\theta_2 \longrightarrow \Delta/h$ then $\Delta' \longrightarrow 0$ (stiff column)
or as $\theta_2 \longrightarrow 0$ then $\Delta' \longrightarrow \Delta$ (flexible column)

Figure 3: Effect of cantilever spans. Different Joint rotations at each level will vary the storey height, h', and effective racking.

Secondary Deformations

Secondary deformations are considered to be those occurring in the individual members of the building frame when it is subjected to the primary deformations outlined above. These will consist of the elastic curvature of both beams and columns, and the localised rotation at plastic hinges due to storey sway.

The overall building curvature and non-simultaneity of maximum inter-storey displacement in adjacent storeys will produce varying deformations at each level. The most significant effect of this is the vertical movement of cantilevers, as shown in Figure 3. Equal column rotations at both levels will cause the cantilevers to rise and fall equally, but a differential column rotation will cause the cantilevers to spread or close. Generally this type of deformation will produce a reduction in the effective inter-storey sway displacement along the cantilever, depending on the relative flexibility of the column, as shown.

The effect of member size and frame geometry is shown in Figure 2. Here it is assumed that beam hinges will form at the column face and the elastic curvature of the deformed members is ignored. It can be seen that glazing panels crossing the face of columns and fixed at or near the column, are subjected to a reduced racking deformation due to rotation of the panel as a whole. For panels fixed along the beams, this effect is reversed. If the frame has been detailed to force the beam plastic hinges away from the column faces, the column width can be considered to be the distance between beam hinges. The resulting deformations will be as for a wide column. (An eccentrically braced steel frame with the yield zone at mid span can be considered an extreme example of this.)

For example, consider a reinforced concrete frame where

Storey height, $h = 3.8$ m
Column width, $W_c = 1.0$ m
Clear span, $L = 6.8$ m
Inter-storey deflection, $\Delta = 0.01\ h$

If elastic curvatures of the beam and column are ignored and the centre of the beam plastic hinge is 0.4 m from the column face:

$$\text{Beam slope} = [(W_c/2 + 0.4) \times \Delta/h) \div (L/2 - 0.4)$$

$$= 0.003$$

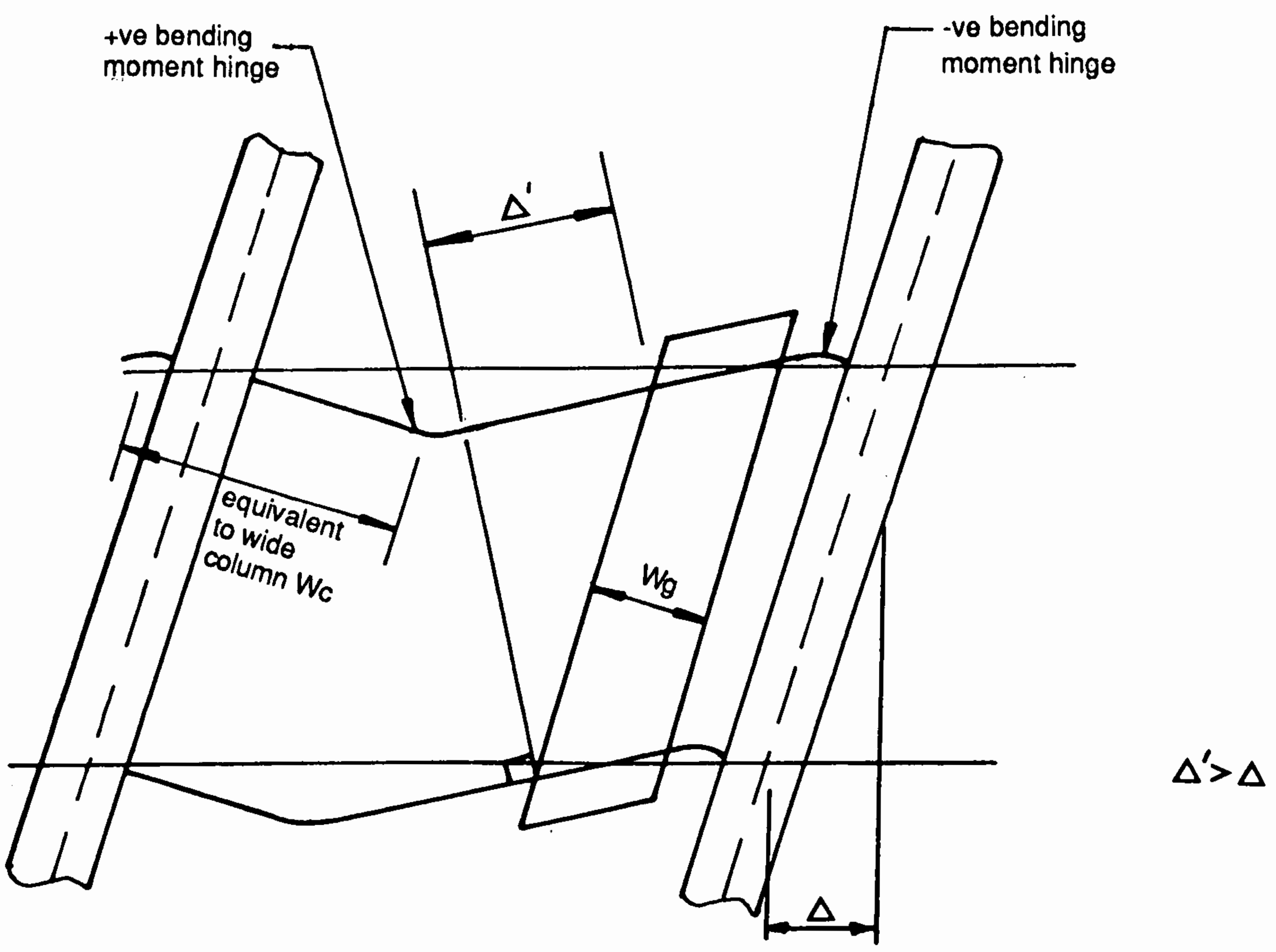
$$\Delta' = \Delta + 0.003\ h$$

$$\Delta' = \Delta\ (1 + 0.003\ h/\Delta)$$

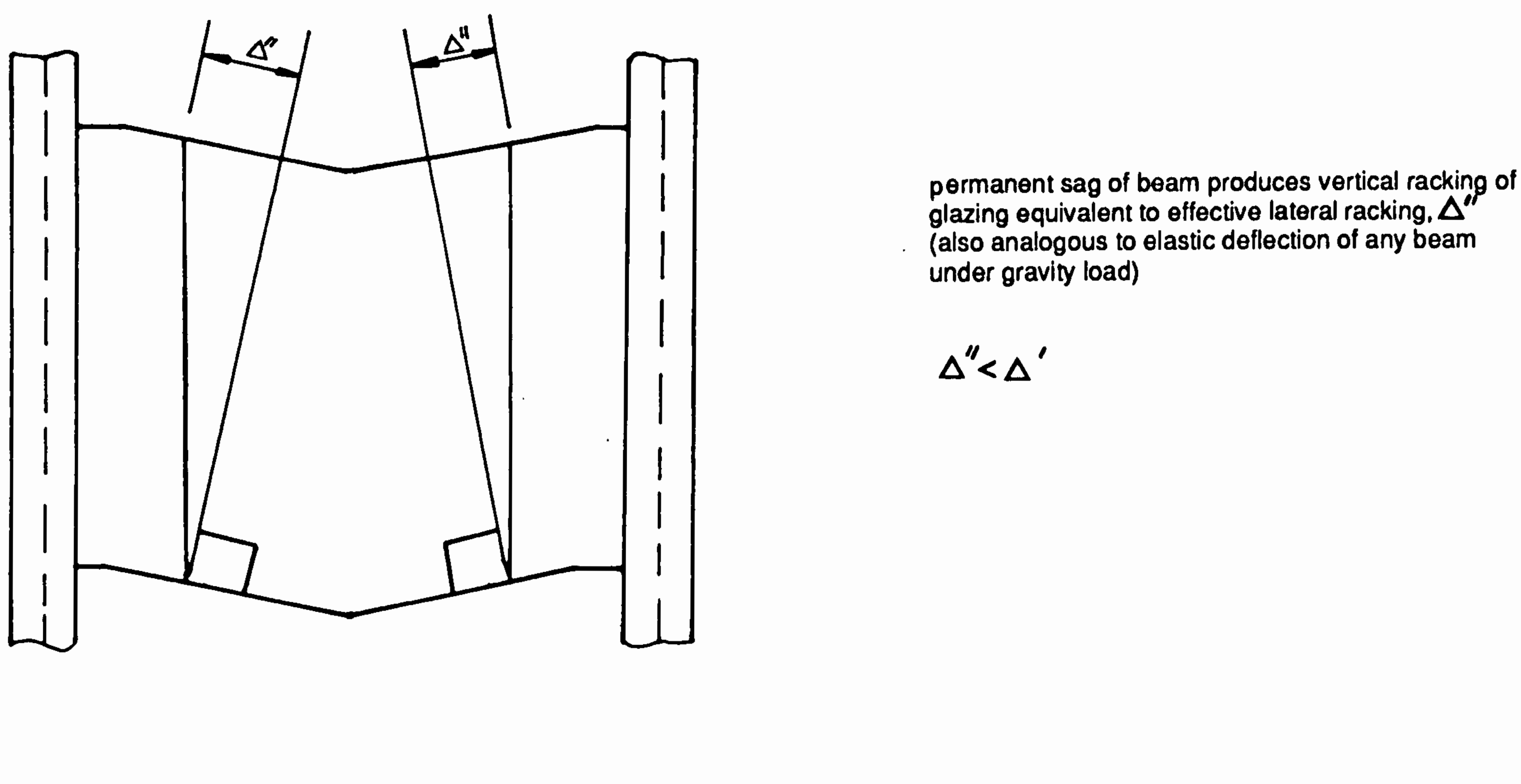
$$= \Delta\ (1 + 0.003/0.01)$$

$$= 1.3\ \Delta$$

If the building frame is "gravity dominated" the large gravity load bending moments may prevent positive moment beam hinges forming at the column faces under sidesway. Plastic rotations will accumulate at the hinges and the beam will ratchet downwards as shown in Figure 4.



deformation during earthquake



after earthquake - no permanent interstorey deflection of building

Figure 4: Effect of hinging remote from column face

If the floor slab cantilevers beyond the face of the beams with the glazing fixed to the outer edge, there can be some smoothing of the deflected profile due to slab flexing. However, should the slab incorporate a non-structural spandrel beam at the outer edge for fire rating reasons, which is not continued past the columns (perhaps being precast), abrupt changes of slope may result.

While this review does not claim to be exhaustive, it does show how curtain walling can be affected by localised earthquake induced building deformations. It also shows that in most cases the deformations imposed on the curtain wall can be resolved into an equivalent lateral racking which may be more, or less, than the inter- storey deflection of the supporting structure.

CODE REQUIREMENTS

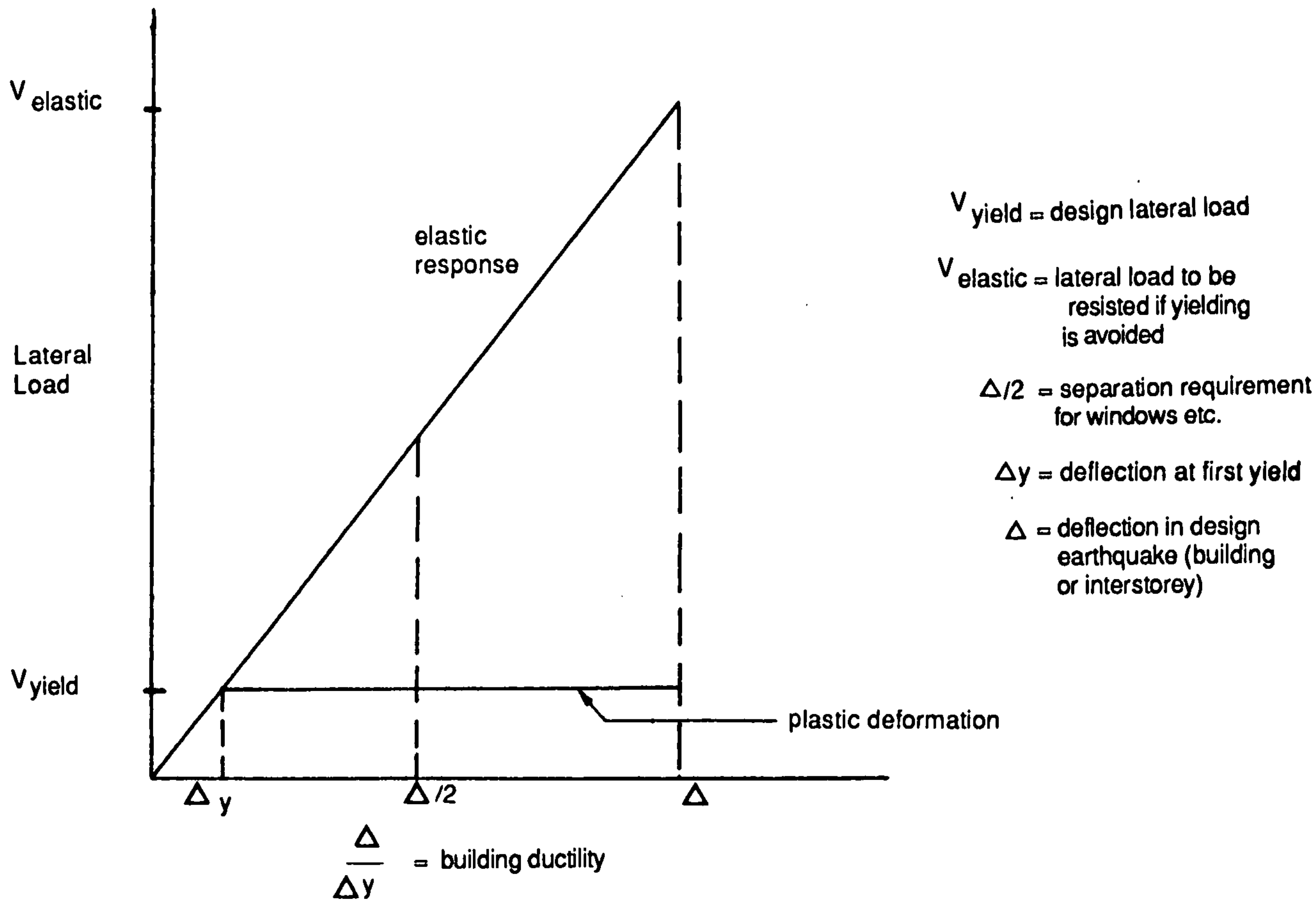
The New Zealand Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1984, seeks to control building deformation due to earthquake loading by stipulating limits for the inter-storey deflection. Separation requirements for non-structural elements are also specified.

Two upper limits for the inter-storey deflection are given. The smaller value is used where no separation is to be provided to non- structural elements. The very small deflection allowed (0.0006 of the storey height for seismic zone A) is intended to ensure that the structural behaviour of the building is not altered by the non-structural components and that damage is kept to a minimum. This small deflection can usually only be achieved by wall structures of low height. Where such structures use curtain wall glazing, it is obvious that the small deflections can be easily sustained.

For more flexible structures, the inter-storey deflections are limited to control P-Δ effects (the secondary forces induced by displacing sideways the vertical line of action of gravity loads) and non-structural elements are required to be separated from the structure. The separation distance to be provided is dependent on the consequences of the particular element impacting the structure. Non-structural elements "that are capable of altering the intended structural behaviour to a significant degree", such as stairways and rigid partitions, are required to be separated by an amount equal to the maximum deflection likely to occur during the design earthquake. Other non-structural components, including precast concrete cladding and brittle exterior elements, such as glazing, are required to be separated by half this amount.

The smaller separation requirement for elements such as glazing accepts that damage, including possible loss of glass, can occur during severe earthquakes (any event that will produce inter-storey deflections greater than half those anticipated for the design earthquake). This will be consistent with damage to the supporting structure itself, which may be required to dissipate large amounts of seismic energy by ductile yielding. The required separation will ensure that moderate, frequently-occurring, earthquakes can be survived with minimal damage, thereby reducing the risk to life and the cost of repairs.

The code assumes that an elastic analysis of the structure is being performed and, using the principle of equal displacements, multiplies the calculated deflection under the design lateral load, Δ_y, by a ductility factor, to obtain the combined elastic and plastic deflection as shown by Figure 5.



Principle of equal displacements:

Deflection of the yielding structure will be the same as an elastic structure of the same initial stiffness i.e., the deflection is a function of initial stiffness and not building strength

Figure 5 : Principle of equal displacements

For a given building of particular stiffness, therefore, the maximum deflection is proportional to the lateral load, V, which is itself proportional to the basic seismic coefficient, C, and seismic weight, W_t.

The basic seismic coefficient is obtained from the graph shown in Figure 6, taken from NZS 4203. The curves are grossly smoothed representations of typical earthquake response spectra and are consequently quite approximate. The period, T, is the first mode free vibration period of the structure which is, itself, a function of both building weight and stiffness.

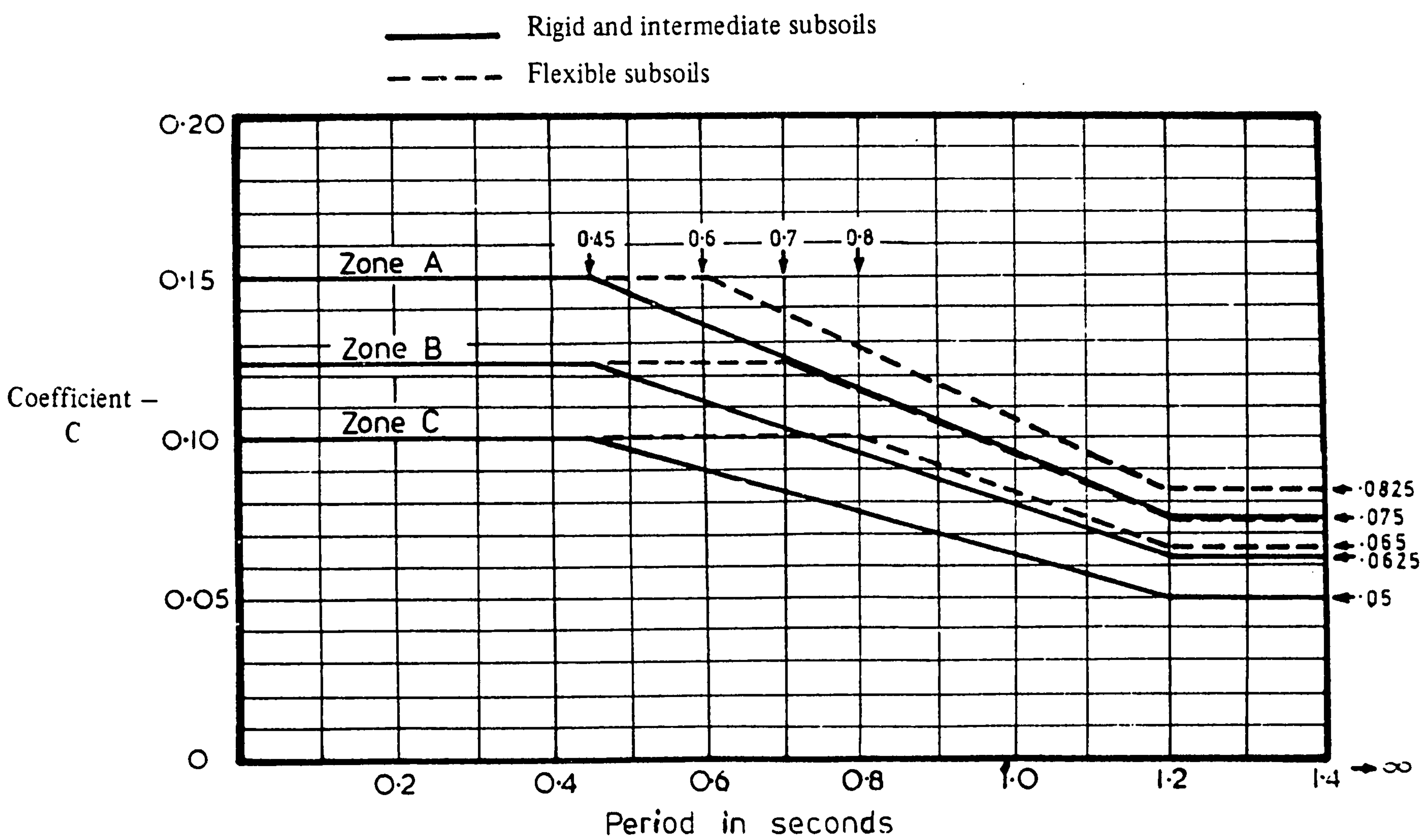


Figure 6 : Basic seismic cofficient C

It is therefore obvious that the building deflection (both overall and inter-storey) calculated in accordance with NZS4203 can only be approximate, and cannot be significantly improved by rigorous attention to the parameters within the structural designer's control, namely the determination of building weight and stiffness.

The NZS 4203 separation requirement for glazing is a horizontal inter-storey deflection of 0.010 of the storey height for seismic zone A, with smaller values for zones B and C. The reduced deflections for zones B and C will ensure that the lateral load stiffness is similar for all three zones, and that the previously mentioned P-Δ effects are adequately controlled (i.e., P-Δ generated forces will be the same proportion of the total force for all zones).

While the technique outlined above is satisfactory for overall lateral deformations, it cannot be used directly for determining the deformation of individual components. This is because the ductility required of the particular component may differ significantly from the overall building ductility, and the elastic deformation under the design lateral load cannot be factored in the same way. Specific limits are not given for such secondary deformations and their significance in determining separations is left to the discretion of the designer.

CONSULTANT AND MANUFACTURER SURVEY

Consultants Surveyed

The following consulting engineers were surveyed to determine the details of the deformation information currently provided by designers to the curtain wall suppliers and how this information was derived:

Firm	Contact
Structon Group	Roger Shelton
Morrison Cooper and Partners	Wyn Clarke
WORKS Technical Services Structural Design	Garry McKay
Beca Carter Hollings and Ferner	Brian Smith
Holmes Consulting Group	Peter Johnstone

General Findings

Without exception, the method of analysis used by the above consultants for multi-storey building design is the modal analysis technique. For smaller structures the equivalent static force technique is used.

For the greatest economy of structure, consultants generally design to the deflection limit allowed by NZS 4203. An iterative process is normally used to size the structural components, although this is not refined beyond one or two steps. Some consultants use percentages of gross moment of inertia values for beam elements, columns, etc which have been suggested in the commentary to NZS 3101:1981 The Design of Concrete Structures, and by Paulay and Williams (1980).

The structural component dimensions are then adjusted in order to obtain deflections just less than the code deflection limits. However, because of the inexactness of the analysis, it has been known for the percentages of the moments of inertia to be adjusted to make the design comply with the code.

Generally, either the actual calculated inter-storey deflection is given to the glazing designers (e.g., in the specification) or otherwise the code limit is supplied. Sometimes this is provided as a storey slope (e.g., one per cent of element height for seismic zone A). If, for other reasons, the building is required to be stiffer than the code requires then the actual calculated deflection is given. An example is the National Library building where greater stiffness was required to protect shelving.

Manufacturers Surveyed

The following manufacturers were surveyed to determine what building deformation information is supplied to them by designers.

Firm	Contact
Horizon Aluminium Products Ltd	Mr R Hanley
Adlite Aluminium Ltd	Mr R Mason

General Findings

In general, it was found that the manufacturers were in agreement with the designers in the information supplied.

However, it is important to note that often in the case of "developer" projects no inter-storey deflections (nor for that matter any other glazing requirements) are given at all. In this situation the manufacturers choose a system that they consider to be the most appropriate, and use "off the shelf" components to minimise cost. This approach will not be adequate in many instances, particularly for seismic zone A. For example, the "Horizon Series 22" system for medium height buildings has a nominal 20 mm inter-storey deflection capability. However, for zone A, 0.01h for a storey height of 3.5 m is 35 mm. Even in zone C, 0.01h x 2/3 is 23 mm.

The glazing fixings used are many and varied and can add a degree of flexibility which is not taken account of in design. Conversely, structure creep, settlement, live load deflection, thermal movement, etc may erode clearances provided for seismic movement.

GLAZING (CURTAIN WALL) SYSTEMS AND DESIGN PROCEDURE

In general, the choice of curtain wall type for a particular building is based on the requirements for wind loading and weathertightness and the visual effect desired by the architect, with the seismic performance being a secondary consideration.

Most manufacturers have a number of systems to cover a range of loadings which can be adapted to most situations, but frequently special designs are called for.

Three types of glazing system appear to be in use:

Stick Systems: These are built and glazed in situ from individual components.

Insert Systems: Mullions are fixed to the building and factory assembled infill panels (with or without glass fitted) are inserted. Generally the seismic separation is provided between the infill panel and mullion.

Panel Systems: These consist of interlocking or overlapping panels that attach to brackets fixed to the structure. Seismic separation is usually provided within the glass pockets of the panels themselves, with the clearance between panels accommodating thermal movements only.

Emphasis can be given to either horizontal or vertical lines by exposing the mullions or transoms to the exterior and using structural silicone sealants at the other edges. The use of structural silicone sealants is largely limited to glazing systems of the insert type where large deformations of the silicone are not required. If the structural silicone is intended to allow for seismic movement also, problems arise from the thickness required to ensure the shear strain is kept to an acceptable level.

The mullions are the primary members resisting wind loading. Their size is governed by the wind load, storey height, disposition of fixings, and mullion spacing (the latter is a function of glass thickness and architectural requirements).

Glazing systems of the insert type allow the use of mullions which are continuous over more than one storey, and are consequently more suitable for resisting high wind pressures.

A vertical gap is provided between each mullion section to allow relative movements resulting from thermal effects and movement of the building. Gravity load support of each section is provided at only one level, with other connections detailed to resist only face loading. This again allows thermal movements of the mullion and differential gravity load deflection between adjacent levels of the structure. The mullions are often spigotted together to maintain the correct alignment or to laterally support the free end.

Glazing panels of the insert or panel type are generally fabricated as one-storey-high units, as larger assemblies become difficult to transport and handle. Gravity support is usually provided along the bottom edge.

Having sized and located the mullions the necessary seismic movement is provided by suitable detailing of the glazing pockets or insert panel support within the mullions.

TEST DESIGN AND PROCEDURE

The Test Rig

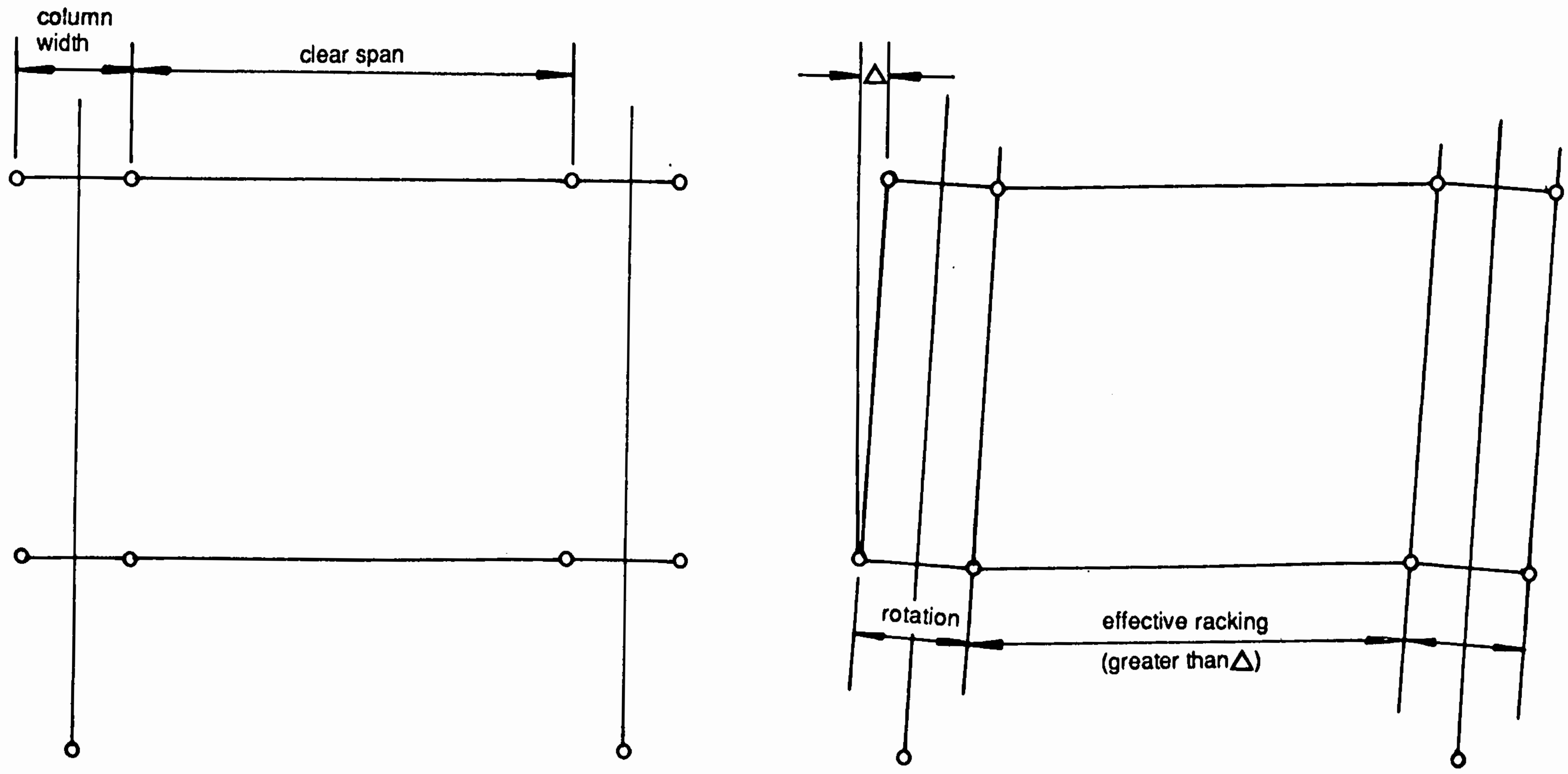
The range of in-plane deformation that can be imposed on a curtain wall glazing system during an earthquake is considerable, and is very dependent on the geometry of the supporting structure and that of the glazing itself. As it is not practical to simulate the full range of structural deformations for a wide variety of structural geometries in one test rig, it is necessary to restrict testing to the parameters of greatest importance.

The most significant parameter is obviously the inter-storey deflection; with the variation in effective racking deformation between adjacent panels, caused by localised building deformations, coming second. It is believed that only the first of these can be adequately tested in the laboratory.

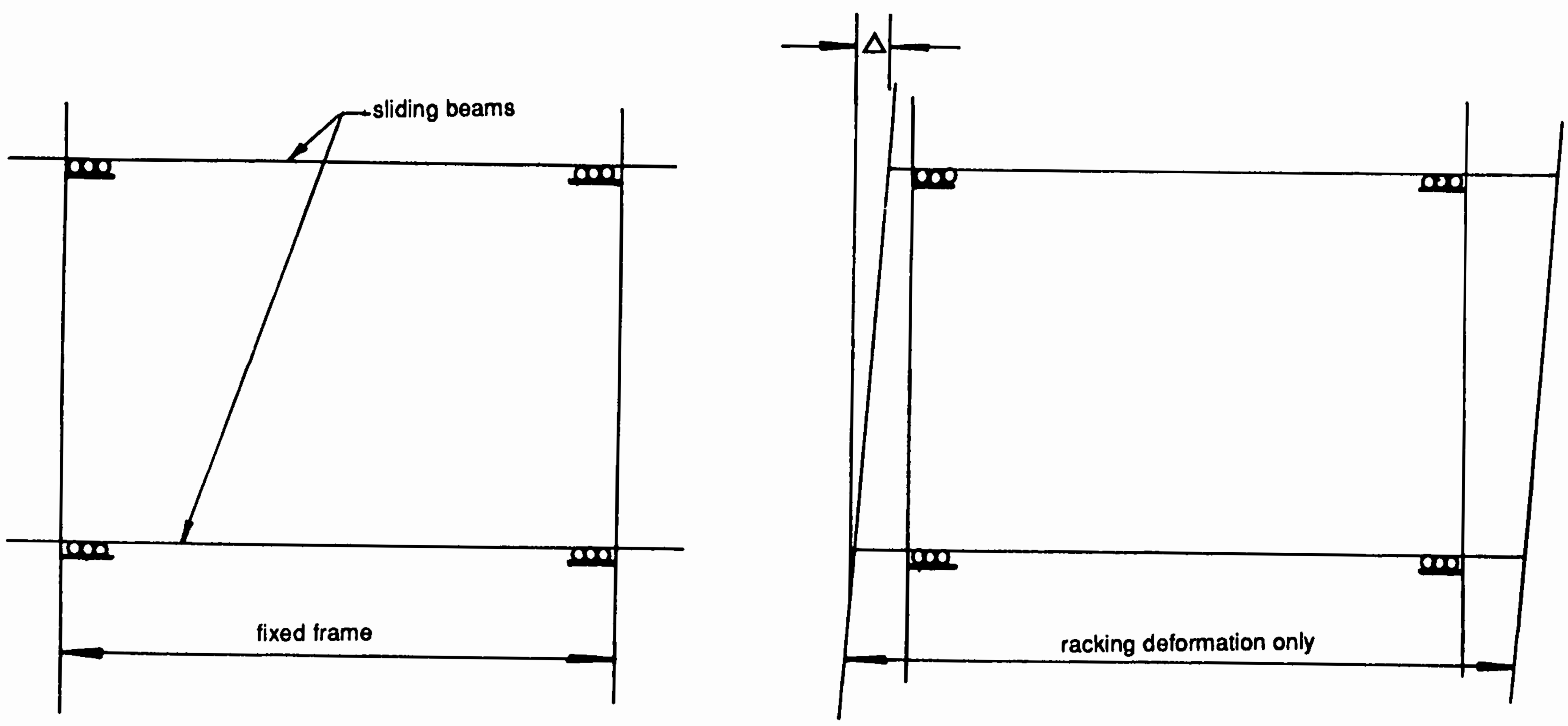
Practical size considerations will mean that it is not possible to test a truly typical "interior" glazing panel, but careful attention to the test panel boundary conditions should allow this to be fairly represented. The boundary conditions are the confinement or restraint that must be provided at the edges of the test panel to represent the influence of adjacent panels on the structure. It may ultimately be found, however, that the boundary conditions have a more significant effect on the performance of the test panel than considerations such as differential racking between panels on the prototype structure.

Situations will also occur where the requirements of the architect result in the use of unusually shaped or large glazing assemblies which cannot be adequately tested on a standardised rig. It may not be possible to attach the assemblies or reproduce the building deformations they were designed to accommodate, making it necessary to develop special "one-off" testing methods.

Two types of test rig were initially considered and are shown schematically in Figure 7.



parallelogram frame



sliding beam rig

Figure 7 : Test rig options considered

The parallelogram frame represents part of a ductile frame structure where beam plastic hinges are expected to form at the column faces. Glazing panels fixed to the frame will undergo a range of deformations depending on where the attachments occur: panels spanning the columns will be rotated while those fixed to the beams will be racked.

How accurately this system represents the prototype structure will depend on the effect of ignoring the beam elastic curvature, and the ratio of column width to clear span. A large range of adjustment will be required for both column width and span length, as well as storey height, making the rig cumbersome, and tedious to set up. Although this arrangement may quite accurately reproduce the deformations to which the glazing can be subjected, it is quite specific in its application and will not easily test a system intended for use on a variety of buildings.

The above considerations led to the development of the sliding beam test rig, which will subject the test panels to racking displacements only. Beam curvatures are again ignored as are column rotation effects, but this system has the widest applicability by being largely independent of the structure geometry. Standard glazing systems intended for use on different buildings can be tested as well as specific installations. The significance of omitting secondary deformation effects (column rotation, beam curvature, etc) should be determined by an engineering study and due allowance made in the interpretation of the test results. It should be noted, however, that in some situations this pure racking test will impose more severe deformations than will occur in practice, producing a conservative result.

A further advantage of this type of rig is that it is a comparatively simple matter to include an extra sliding beam to test double storey height panels. The effect of continuity above and below the test panel can then be modelled and building curvature introduced by applying a differential racking at the two levels.

The sliding beam test rig is therefore the recommended option and is discussed more fully in a subsequent section.

Performance Criteria

The setting of definitive performance criteria for curtain wall glazing when subjected to earthquake loading or a test regime is beyond the scope of this study. However, the basic requirements are comparatively clear.

The glazing must be able to withstand, without failure, deformations imposed by the supporting structure that are significant compared with the deformation expected in a major earthquake. This reduces both the hazard from falling glass and the need for expensive repair work for all but the most severe earthquakes.

What constitutes a glazing failure needs to be rigorously defined. Obviously the breaking or shedding of glass constitutes failure; however, damage may be sustained at lower deformation levels that compromise serviceability (weathertightness for example) and may not easily be repaired.

With these considerations in mind, it is recommended that a three tier system be adopted, with performance levels defined as follows:

Level 1: No damage shall occur for deformations up to this level that will reduce the specified requirements for air leakage and weather-tightness, i.e., no repair work will be required.

Level 2: Failure of the glass, or significant damage to the glass support and framing system, shall not occur for deformations less than this level. Significant damage can be considered to be that requiring replacement of components or their removal for repair. Partial failure of sealants or dislodging of gaskets can be accepted. The system's ability to survive future earthquakes and serve its other functions will not have been reduced once repairs are effected.

Level 3: The ultimate failure of the glass or framing system should not produce a greater risk to life during the design earthquake than other parts of the structure.

The most appropriate value for the Level 2 deformation is the separation requirement of NZS 4203. This standard requires separation to prevent glass windows impacting each other or the structure for inter-storey deformations up to one half those expected from the design earthquake.

In view of the compromise nature of the test rig and testing procedure, a factor should be introduced that will ensure the required performance is achieved in situ. Because of the range of structural deformations possible, and their interaction with a variety of curtain wall systems, the choice will be qualitative unless an in-depth study is undertaken for every test. As shown previously, the effective racking occurring on parts of a building frame can be greater than the nominal inter-storey deflection. Therefore an over-deflection of (at least) 20 per cent is suggested. That is, the Level 2 performance limit, when determined by racking test, should be 1.2 times the separation required by NZS 4203.

The Level 1 performance limit is harder to define in terms of actual displacement, and would require a study of repair costs for earthquakes of different intensities, or a history of test results, before a value can be assigned. However, as a starting point, it is proposed that a deflection of approximately two thirds the separation required by NZS 4203 be used. This level is also significant compared with the maximum inter-storey deflection in the design earthquake (about 30 per cent) and is somewhat greater than the deflection at first yielding of a typical ductile frame structure.

The Level 3 limit is a life risk consideration that might require the use of laminated glass or the provision for substantially larger separations. Alternatively, the separation could be chosen to give a level of protection consistent with the risk from failure of other non-structural components (e.g., precast cladding panels).

TEST SEQUENCE

In order to devise a test sequence that would realistically represent the deformation occurring during an earthquake, it was necessary to perform time history computer analyses for a range of building sizes. Analyses were performed on typical regular frame structures of 6, 12 and 20 storeys. The buildings were modelled as vertical elasto-plastic shear beams on rigid bases and were subjected to the 1940 El Centro earthquake record (north-south component). The vertical shear beam is a single cantilever member in which only shear deformations are permitted to occur, the shear stiffness being adjusted to match the lateral storey stiffness of the prototype structure. A static elastic analysis was performed to determine the required stiffness based on the earthquake loading provisions of NZS 4203, and using the maximum inter-storey deflection allowed by this code. The models were then subjected to the El Centro earthquake record, as base accelerations, using the two dimensional dynamic inelastic analysis program, DRAIN 2D.

Inter-Storey Deflection

The plots shown in Figure 8, and Figure 9, give the inter-storey deflection time histories for the critical storey of the six storey building when subjected to the first 20 seconds of the El Centro record, with amplitude scaling factors of 100 per cent and 60 per cent respectively.

The full amplitude El Centro record is of similar intensity to the design earthquake of NZS 4203, for seismic zone A, and it can be seen from the plot that the maximum inter-storey deflection is approximately twice that of the smaller earthquake. It can also be seen that the separation required for glazing by NZS 4203 (and the Level 2 performance limit described previously) is exceeded within one cycle of building deformation for the design earthquake. This indicates that the Level 3 performance limit (ultimate failure mode) can be determined by monotonic test.

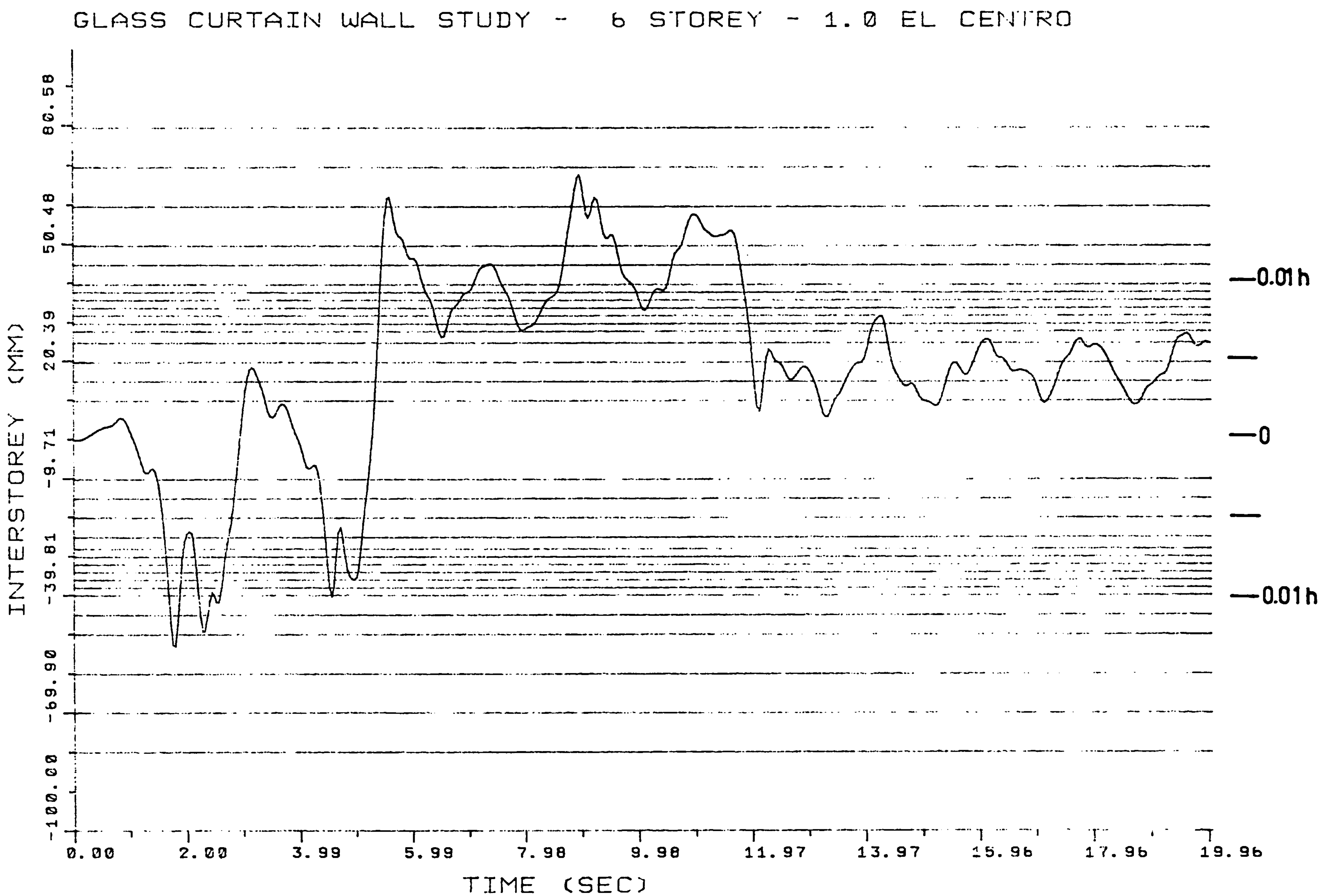


Figure 8 : Inter-storey displacement between first and second floors 6-storey building model, storey height 4 metres (1.0 El Centro)

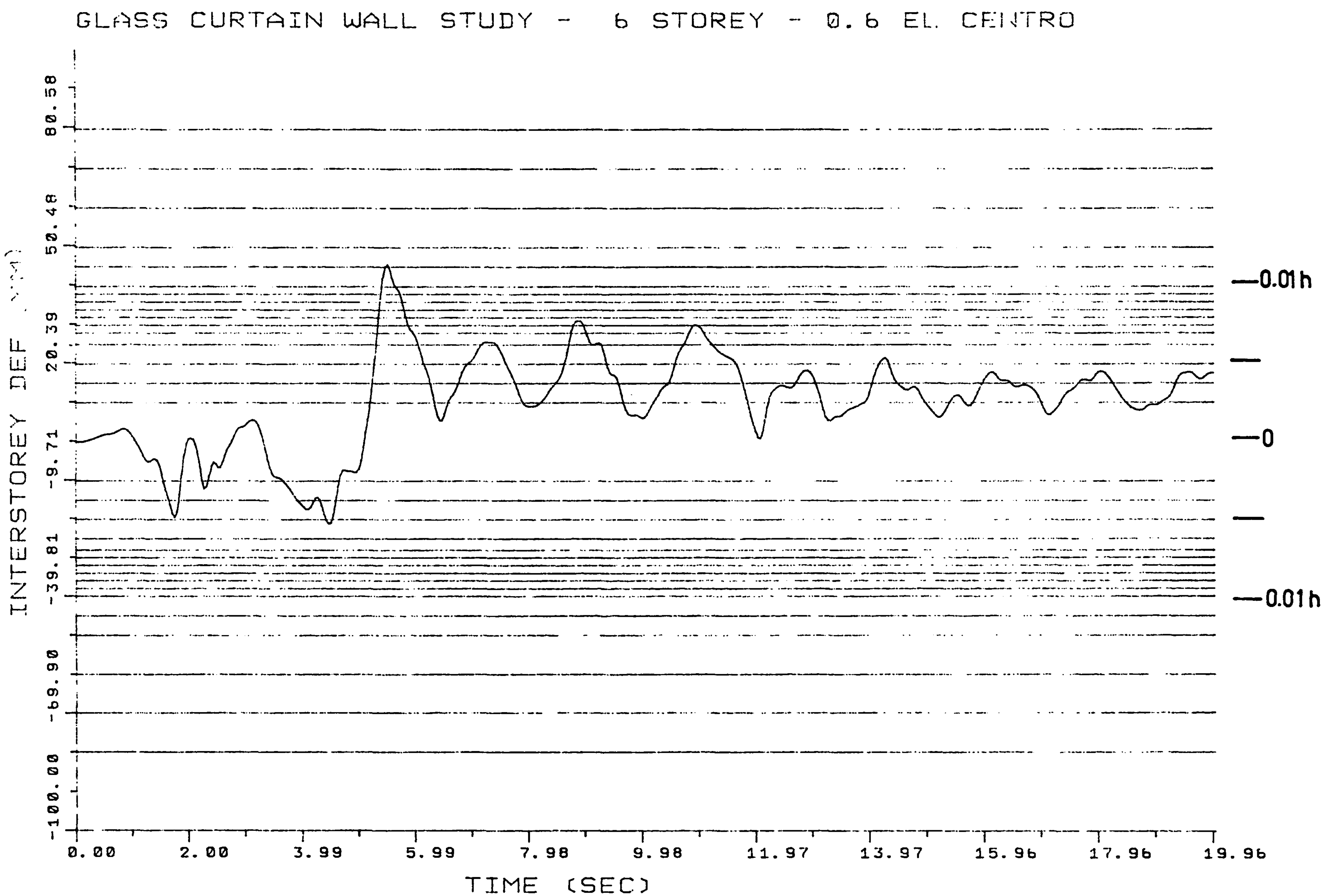


Figure 9 : Inter-storey displacement between first and second floors 6-storey building model, storey height 4 metres (0.6 El Centro)

The El Centro record was scaled to 60 per cent to give a maximum interstorey deflection, Δ, of 0.01 times the storey height (see Figure 9). With the next largest peak being $^{2}/_{3}$ Δ, it is apparent that the testing need not include a large number of cycles at high amplitudes. However, in order to determine the actual deflection at which the glazing is deemed to fail, either according to NZS 4203 or the previously proposed Level 2 performance limit, the maximum deflection must be approached in comparatively small increments. This may result in the glazing being subjected to more large amplitude cycles than will occur during an earthquake of this intensity, but is a necessary compromise if the failure deflection is to be identified, and will give a conservative result.

The following cyclic displacement sequence is proposed:

INTER-STOREY DEFLECTION		NUMBER OF CYCLES	ACTUAL DEFLECTION FOR 3.8 M STOREY HEIGHT SEISMIC ZONE A (mm)
Level 1 Performance Limit (approximate)	0.3 Δ	5	11
	0.6 Δ	5	23
	0.9 Δ	2	34
	1.0 Δ	1	38
	1.1 Δ	1	42
Level 2 Performance Limit	1.2 Δ	1	46

Level 3
Performance Limit : monotonic loading to failure

where Δ equals 0.01 times the storey height (mm)

It is likely that several earthquakes will be experienced during the life of the building. This is reflected in the larger number of cycles up to the Level 1 performance limit than would be indicated by the time history plot of a single earthquake.

Figures 10 and 11 show plots of critical inter-storey deflection for the 12-storey building subjected to 50 per cent and 100 per cent of El Centro respectively. Under full amplitude El Centro, the separation required by NZS 4203 for glazing is almost reached within one cycle of building deformation and significantly exceeded on the second cycle, confirming that again the Level 3 performance limit can be determined by monotonic test. A scaling factor of approximately 80 per cent would limit the maximum deflection to 0.01 h, and shows the reduced response of this building to earthquake shaking of the El Centro type when compared to the smaller six-storey structure.

Plots for the 20-storey building critical inter-storey deflection are shown in Figures 12 and 13, for 50 per cent and 100 per cent El Centro respectively. In this case the full amplitude El Centro produces critical inter-storey displacements somewhat less than the NZS 4203 allowable

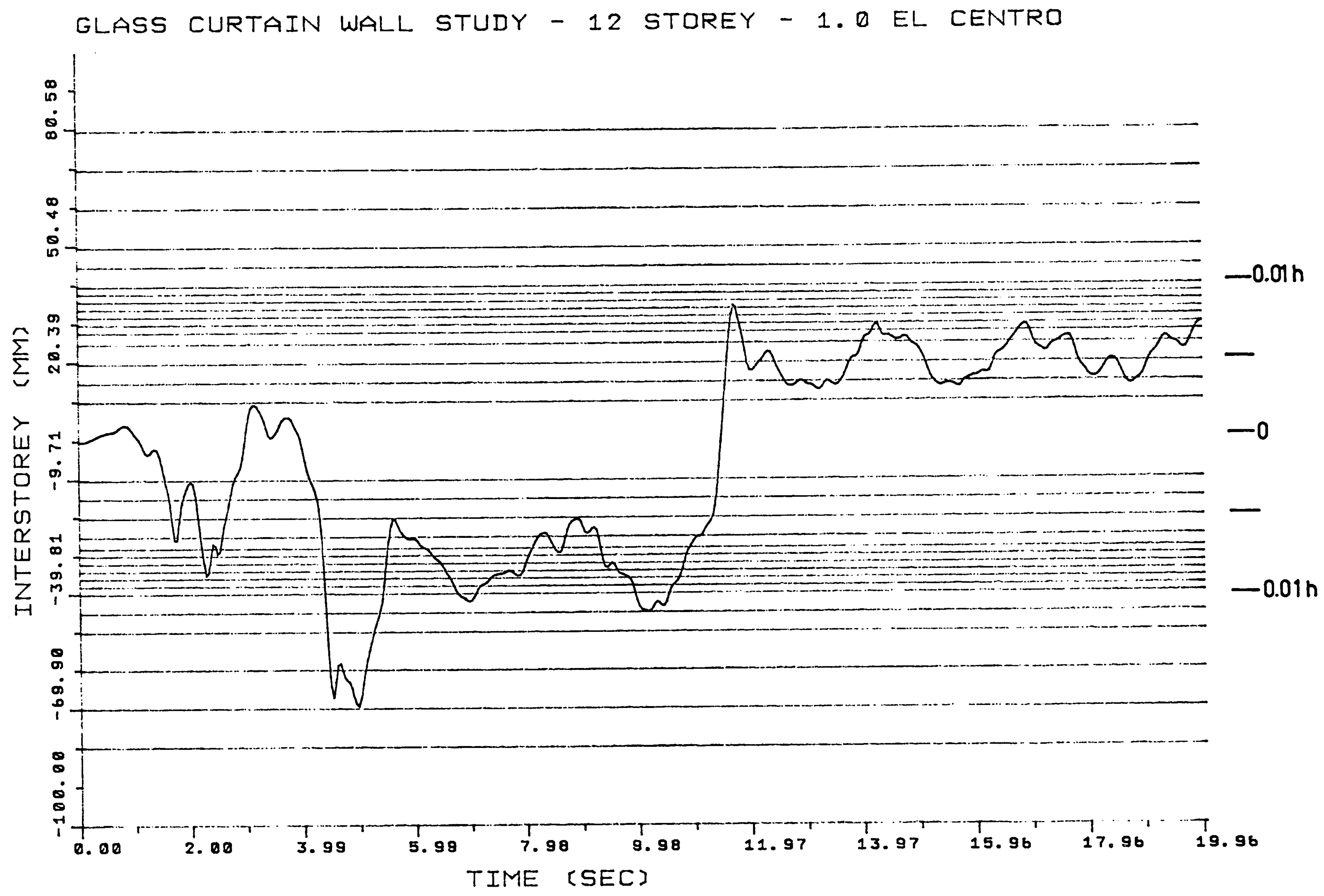


Figure 10 : Inter-storey displacement between first and second floors 12-storey building model, storey height 4 metres (1.0 El Centro)

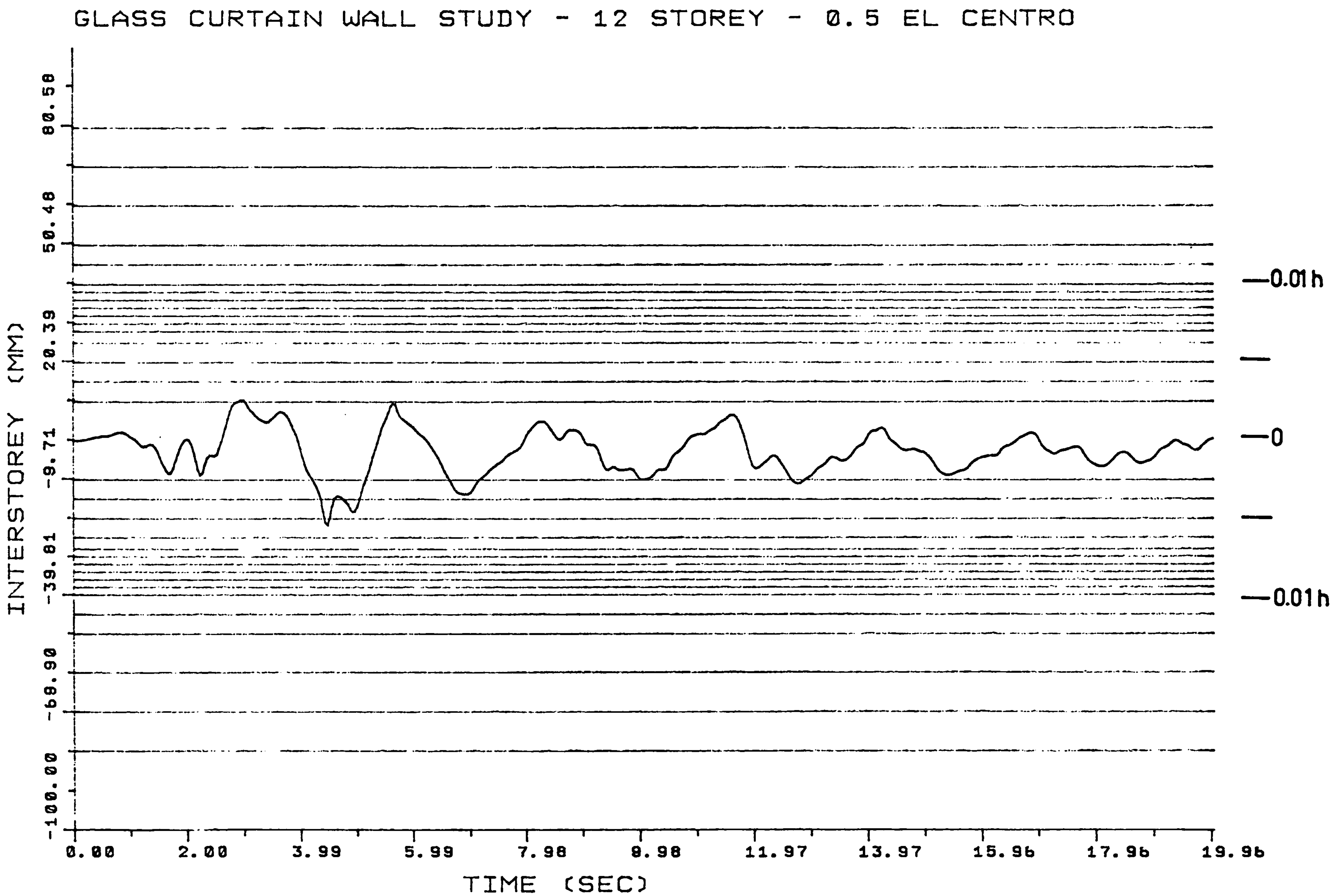


Figure 11 : Inter-storey displacement between first and second floors 12-storey building model, storey height 4 metres (0.5 El Centro)

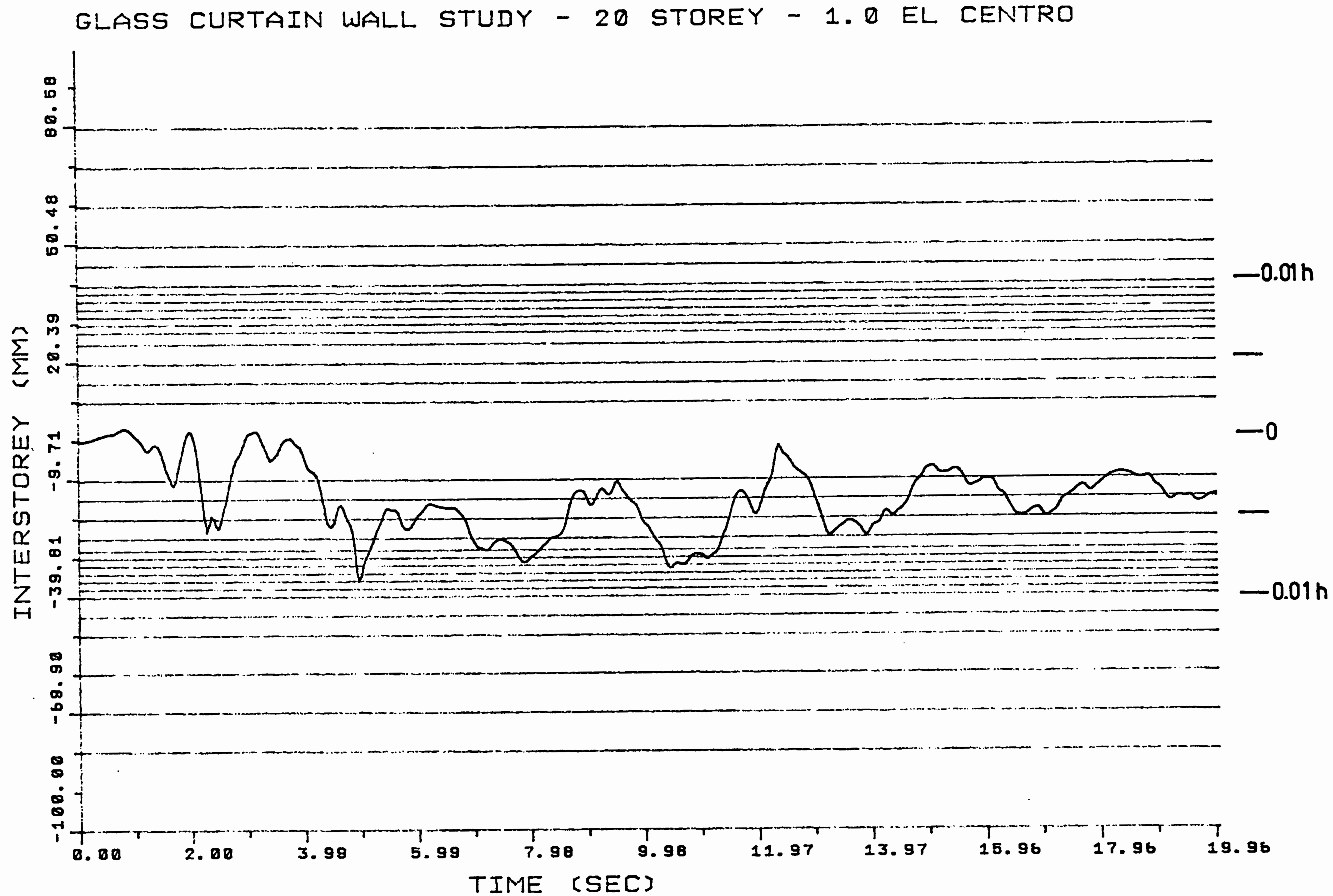


Figure 12 : Inter-storey displacement between first and second floors 20-storey building model, storey height 4 metres (1.0 El Centro)

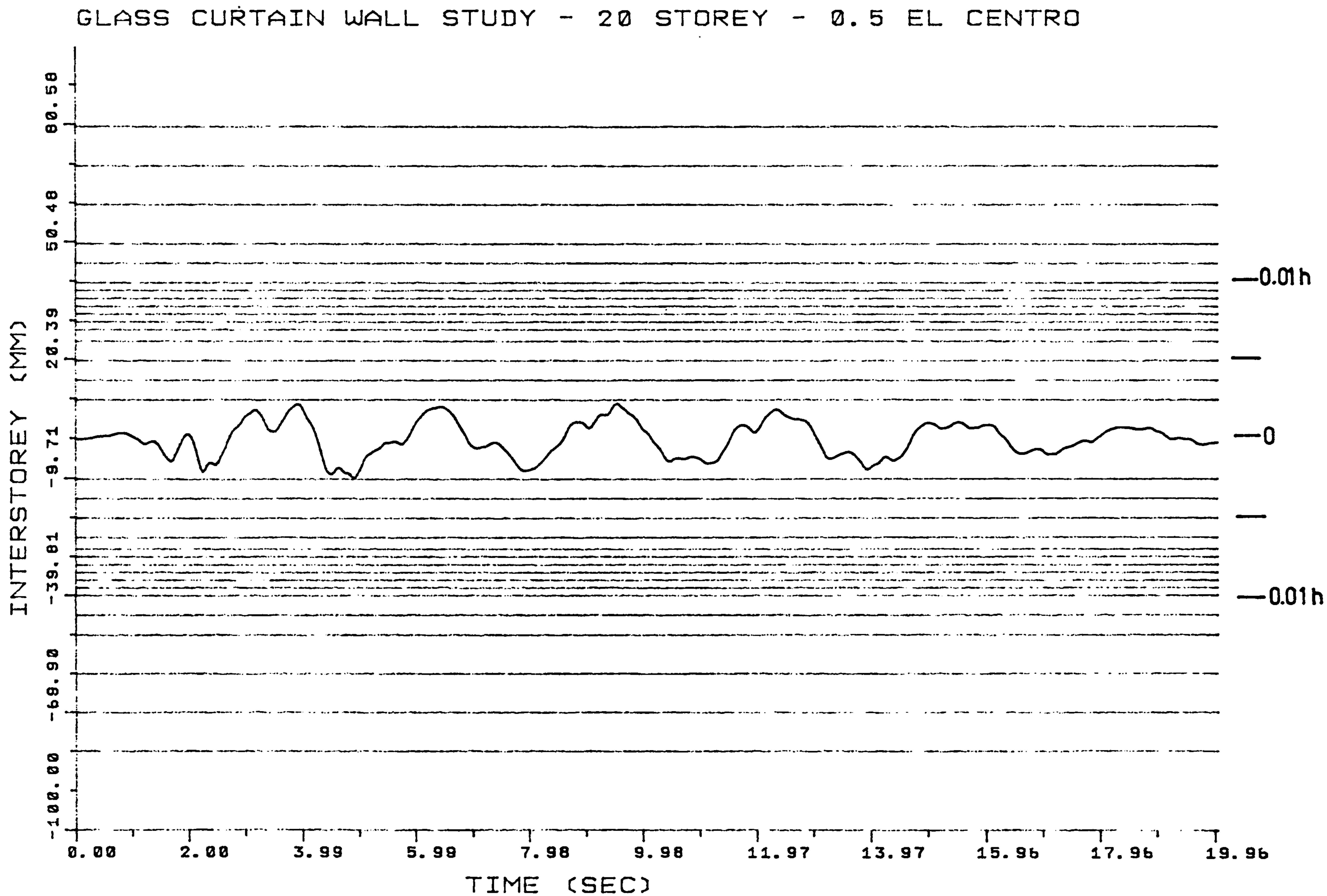


Figure 13 : Inter-storey displacement between first and second floors 20-storey building model, storey height 4 metres (0.5 El Centro)

limit. The argument for a monotonic test to the Level 3 performance limit is therefore less convincing in this case, and further study of the deflection characteristics of higher buildings may be necessary.

Building Curvature

Figure 14 shows the overall deflected shape of the six storey building for the 0.6 El Centro earthquake at two time intervals in each direction. These time intervals give the maximum building deflection at certain floor levels as can be seen from the envelopes of maximum deflection.

The maximum inter-storey deflection occurs in the second storey and at the same instant the adjacent storeys are racked between a quarter and a half of this amount. The maximum building curvature (for the four profiles given) appears to occur at first floor level and t = 6.30 seconds, but this is an illusion created by the horizontal exaggeration of the profiles as shown in Figure 15(a) and (b). With the maximum curvature occurring simultaneously with maximum inter-storey deflection, it is a simple matter to allow for both effects during the one test by racking a two storey test panel different amounts at each level.

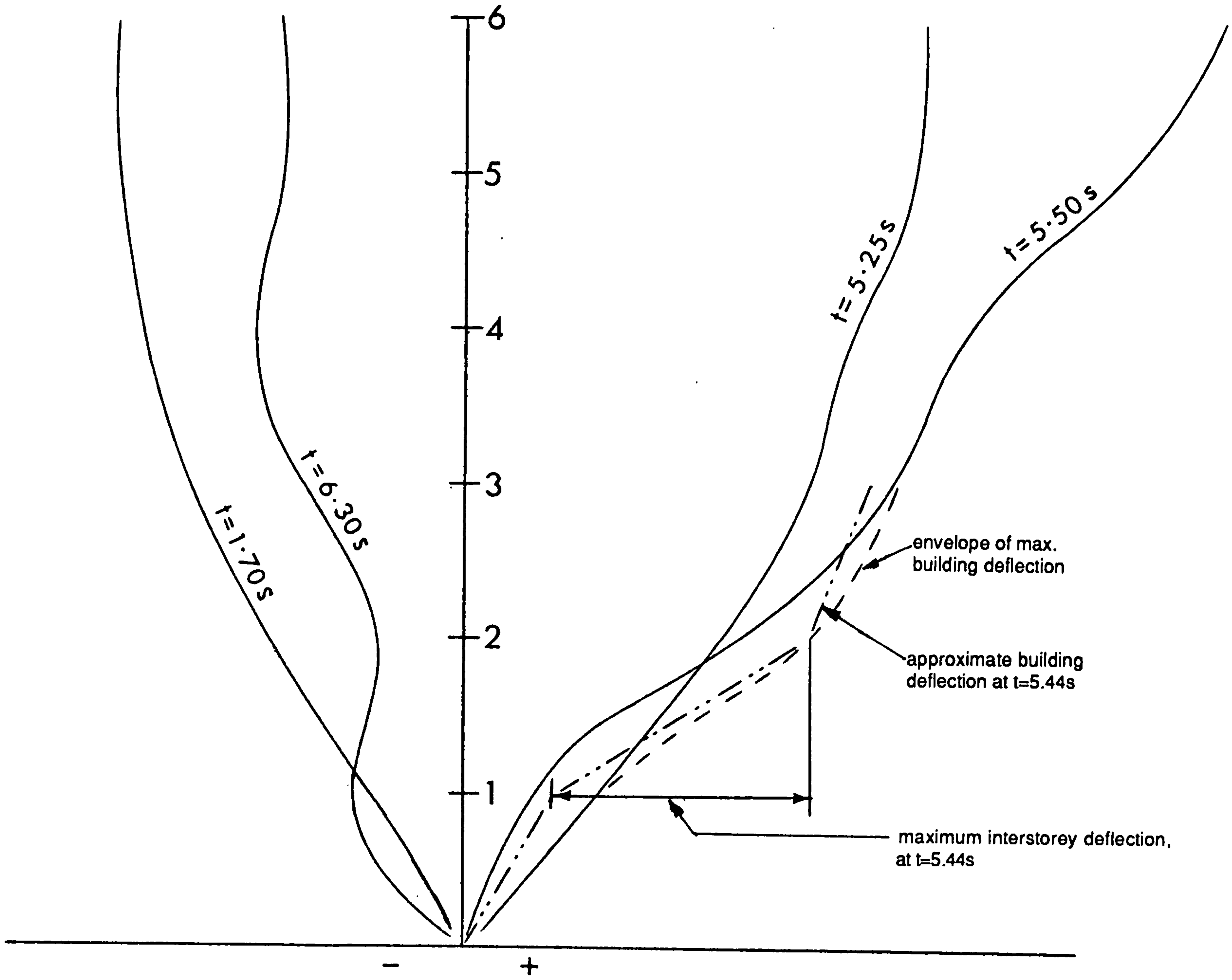


Figure 14 : Deflected shapes of six storey building at different intervals, 0.6 El Centro 1940 N-S

Study of computer output for the 12 storey building has indicated that maximum curvature and inter-storey deflection appear to occur together. In the case of the 20 storey building there has been insufficient time to check the concurrency of maximum curvature and inter-storey deflection. However, a conservative result will be obtained if this assumption is made.

From Figure 15(a), for the six storey building, a displacement of Δ/4 in the storey adjacent to that undergoing the maximum inter-storey deflection, Δ, appears appropriate. A significant simplification in the operation of the test rig would result, however, if the smaller displacement were reduced to zero as shown in Figure 15(c). The curvature is increased by approximately 35 per cent, but only one level need be displaced; the other two remaining fixed to the reaction frame. Further computer studies are required to verify this proposal.

It must be emphasised that in the computer studies only the El Centro north-south record has been used to excite the model structures. This record has been chosen because the NZS 4203 seismic coefficient has been derived from the El Centro response spectrum. Further computer studies are required using other earthquake records to fully investigate the influence of the response spectrum on the building curvatures.

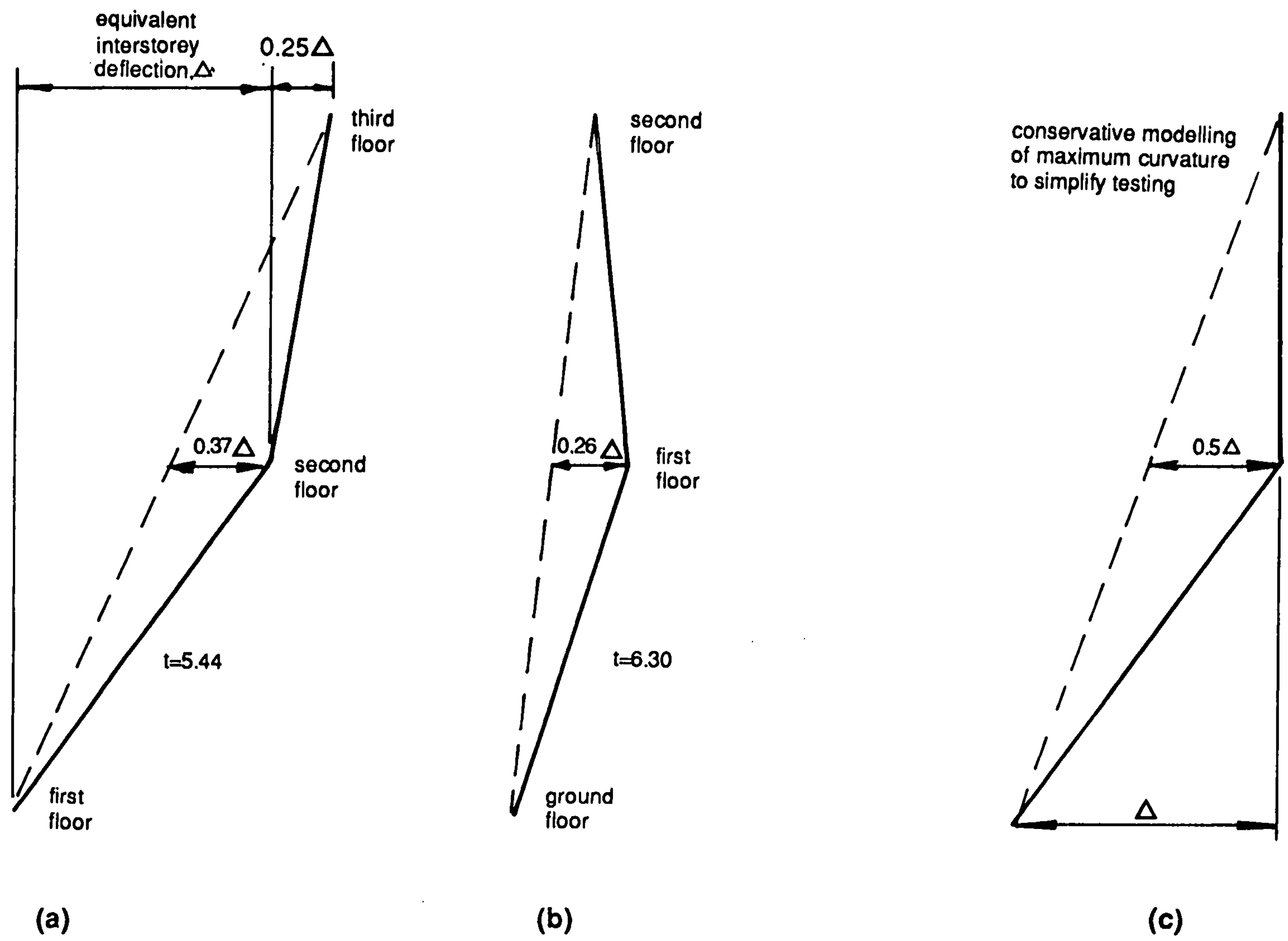


Figure 15: Straight line representation of building curvature at different time intervals for six storey building. Max. curvature occurs with max. inter-storey deflection.

TEST RIG DETAILS AND OPERATION

The principle of the sliding beam test rig has been considered in a previous section; this section develops the proposed scheme and indicates how it can be used.

Height Considerations

The overall height of the test rig is dictated by the need to allow for building curvature in the testing programme. If the configuration of the glazing requires two full storeys to be used, it is inevitable that the rig will be very tall and consequently difficult to house.

Figure 16 presents schematically the proposed test rig, configured for two different modes of operation, and showing two different overall heights.

The mode 1 configuration allows testing of a full storey height panel with part storeys above and below. The height of the part storey elements that can be accommodated will depend on the storey height of the test panel and the overall height of the test rig. The 7.5 m rig height proposed for this configuration will allow 4 m storeys to be tested with 1/3 storeys above and below. The mode 2 configuration allows the use of two full storeys but unless the maximum storey height is restricted the overall rig height becomes excessive.

The mode that is used for a particular test will depend on the expected glazing performance and the information required, and how the test panel boundary conditions are assessed. For example, the glazing panel edges may be unrestrained at the upper and lower sliding beams when mode 2 is used.

Design Details

The sliding "beams" represent the beams of the prototype structure and are constructed from two members that can be individually adjusted for beam depth, but raised or lowered together to give the required storey height. The sliding glazing fixings are similar to the "beams" but comprise single members to allow mid-storey height fixing of panels surrounding the test panel. All sliding members can be locked to the reaction frame as required or displaced by different amounts to introduce the appropriate building curvature.

The sliding beam members will be provided with mounting points along their length to accept brackets for attaching the glazing system. These will, as far as possible, replicate the actual fixings by allowing attachment to the beam top or underside, or off the face at different levels.

Mounting the sliding members off the face of the reaction frame will permit installation of the test panels as on the face of a building, but will also allow full size test units (comprising a number of panels) to be erected elsewhere before mounting on the rig, if required. This latter consideration will be of greatest use where silicone sealants are used, as these require several weeks to fully cure, and will maximise utilisation of the test rig.

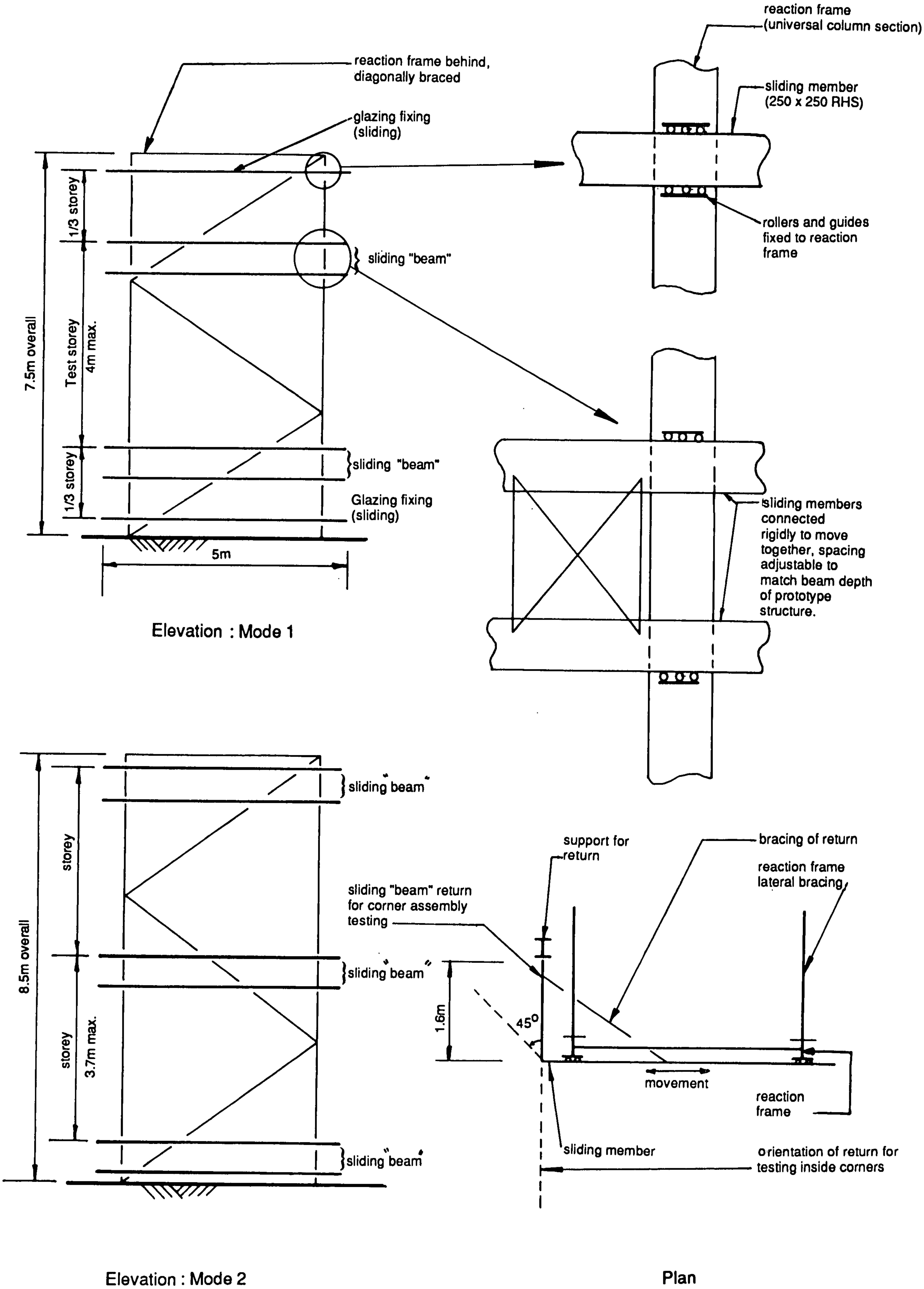


Figure 16:Schematic diagrams of proposed test rig and details

Face mounting of the sliding members and test panels will also allow testing of corner details. The sliding members can be provided with returns attached at any angle between 45 and 90°, supported off the reaction frame and rigidly braced off the main sliding member. This arrangement will allow racking deformation out of the plane of the glazing by simulating a direct translation of the supporting structure, various rotational effects being ignored. The returns attached to the sliding members can, in fact, be provided in either direction to allow testing of both outside and inside corner details. In the latter case the bracing of the return beam will be more difficult and may limit the width of panel that can be tested in this manner.

Load Application

The method of load (or more correctly, displacement) application will depend in large measure on the curvature of the prototype structure it is wished to represent during the test.

The simplest method uses a single hydraulic ram, as shown in Figure 17(a). This can be used for both mode 1 and mode 2 operation, although in the former case difficulties may arise in effectively joining the sliding beam and sliding glazing fixing if these are widely spaced.

A further range of motions can be generated by interposing a vertical pendulum between the ram and the various sliding members. By moving the pendulum pivot up or down one column of the reaction frame, various fractions of the ram extension (either greater or smaller) can be produced at the different levels as shown by Figure 17(b).

Multiple rams could be used to produce any pattern of deflection, but would require a much more sophisticated control system. This added complexity is only warranted if it is determined that the building curvature cannot be adequately approximated with the single ram.

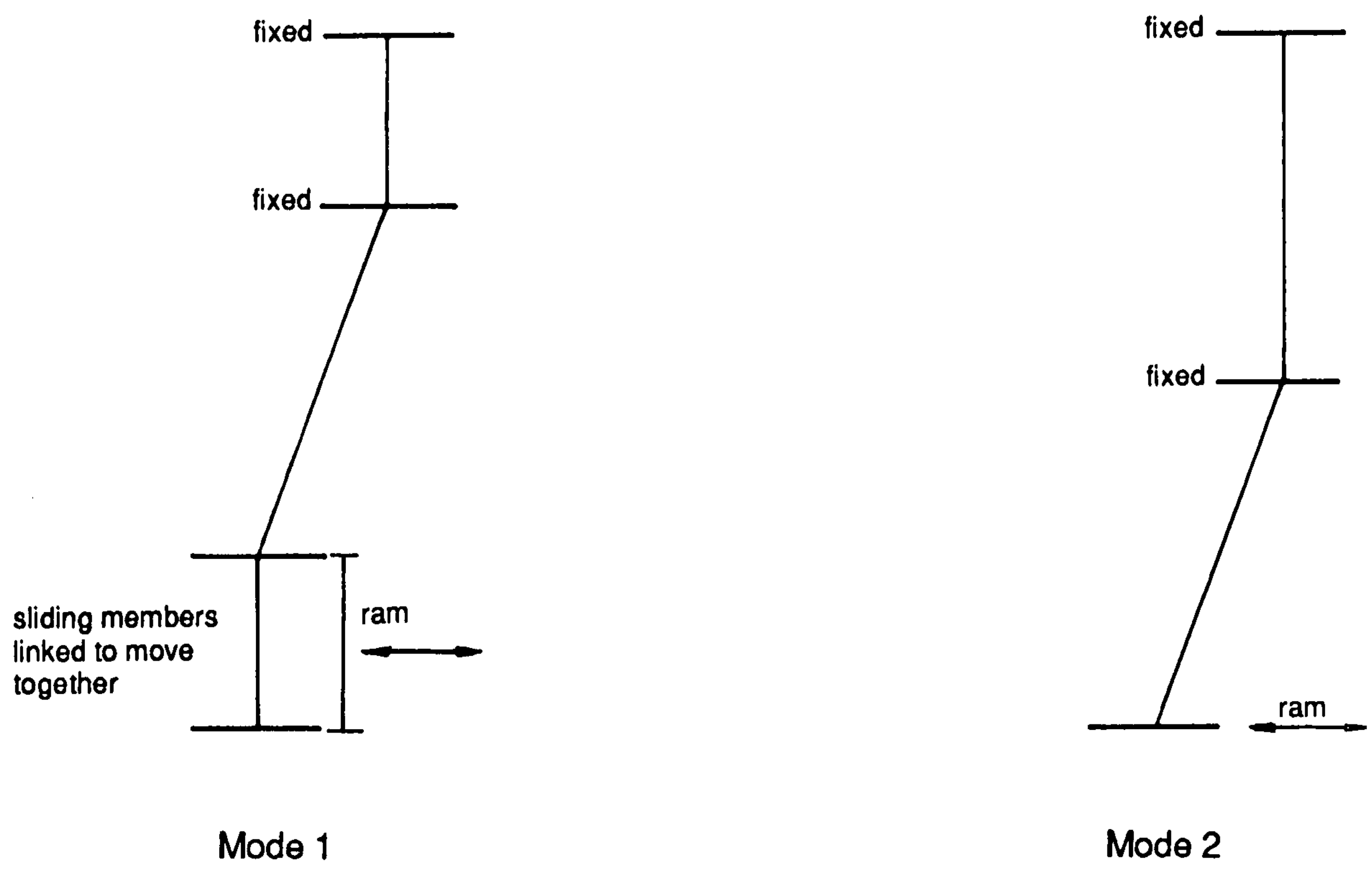
Consideration has been given to the rate of loading, but it has not been possible to determine whether the performance is sensitive to this. A slow loading rate is to be preferred as it will simplify the data acquisition and allow visual inspection during the test. A fast loading rate at typical building response frequencies of 0.5 to 1.0 cycles per second would generate significant inertia forces within the test rig and require computer control of the ram or rams. If such equipment were available it would be possible to input actual earthquake response time histories.

To determine the significance of loading rate small scale cyclic tests should be performed on gasketed and silicone sealed samples before the detailed design of the test rig.

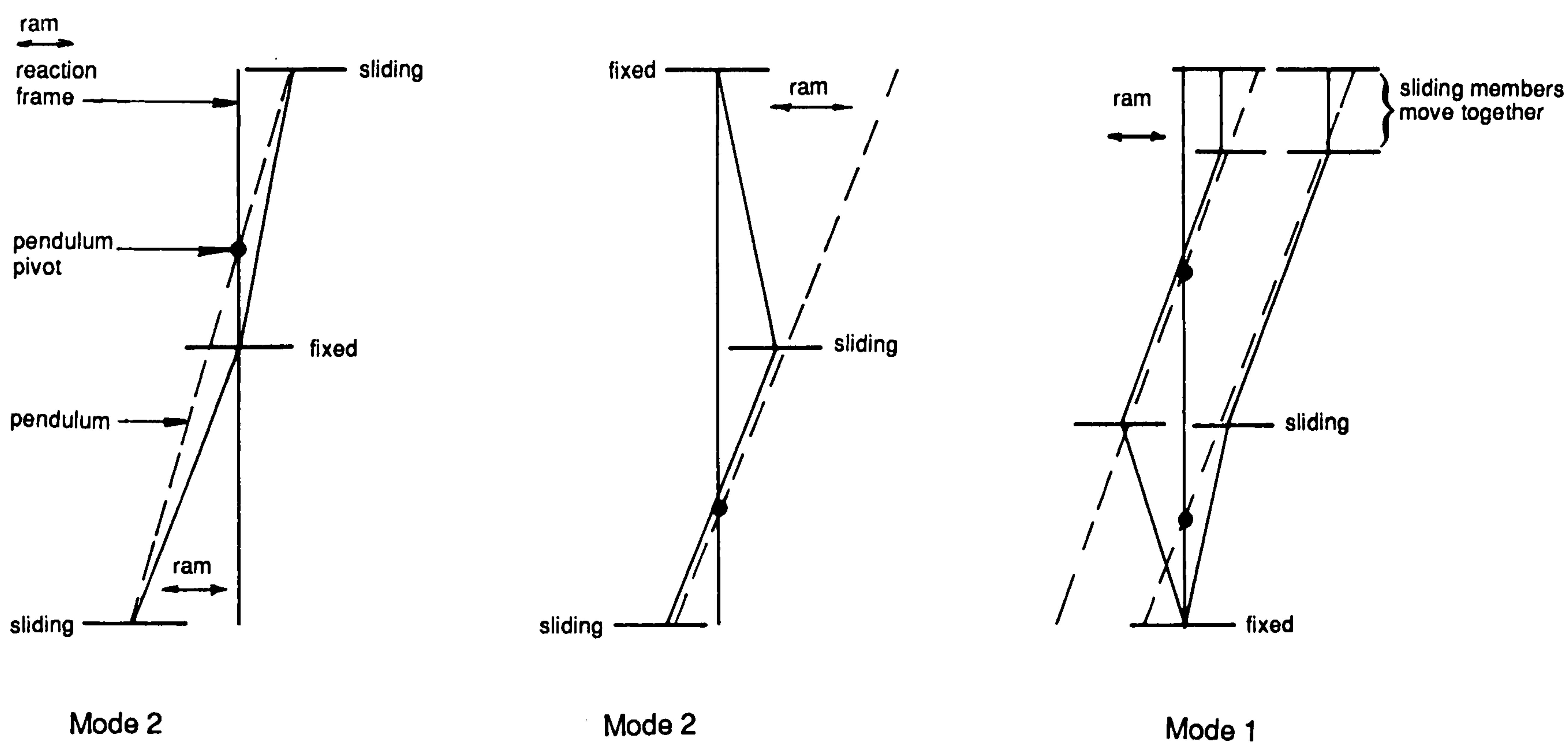
Instrumentation

The instrumentation that will be required is likely to vary from test to test and is therefore difficult to define.

The most significant measurement will be the lateral displacement at the various levels, as a means of controlling the test and identifying the required performance levels.



(a) simple single ram loading



(b) advanced single ram loading

Note: pin ended links join pendulum to sliding members

Figure 17: Possible deflection profiles using single hydraulic ram loading

Further instrumentation will be provided to determine whether the performance criteria are being satisfied and will probably be different in every case. Permanent deformation of framing members, joints and fixings can be monitored as a means of assessing damage levels, but acceptable limits that can be related to the proposed performance criteria will require further study.

The applied loads will be measured but are likely to be of secondary importance to the deformation. The overall lateral stiffness of the curtain wall can thereby be determined, but maximum forces within the system and its fixings can be underestimated by ignoring inertia effects.

Example Test Configuration

To illustrate how the test rig would be used, consideration was given to the testing of a current production curtain glazing system. Figure 18 shows the arrangement that could be used for testing the Horizon Aluminium Products Ltd 22 Series curtain wall. The dimensions have been taken from an actual installation for which drawings were available, but the arrangement will also suit any system of similar size where two glazing panes are used in each storey-high panel.

The Horizon 22 Series is intended for use on medium height buildings with less than the maximum allowable (by NZS 4203) inter-storey deflection. The system utilises split interlocking mullions and transoms around each panel with solid transoms between the vision and spandrel panes. Each panel is fixed to the building at one bottom corner only and is supported by its neighbours elsewhere, the clearances within the split mullions and transoms allowing thermal movement of the curtain wall and live load and creep movement of the structure. The nominal seismic clearance is provided within the glazing pockets.

Figure 18 shows the mode 1 configuration being used. This is most appropriate because each storey high panel consists of two panes, allowing the intermediate transom to be attached to the glazing fixing members. The test panel boundary conditions are thereby realistically represented because the panes above and below are still free to move in their pockets.

The lower "beam" and glazing fixing are prevented from moving by locking to the reaction frame, while the upper members are linked together and move equal amounts. The resulting reversed curve overall deflected profile in essence assumes that no racking occurs in the storeys above and below the test panel. This may be a somewhat harsh curvature requirement but is warranted to simplify the testing, and will produce a conservative result.

Figure 18 also shows how the effective racking can be increased by rigid fixings at the upper and lower surfaces of deep beams, emphasising the need to accurately represent the prototype fixings. Figure 19 shows how this effect will influence the glazing performance and also the effect of different vision pane-to-spandrel pane height ratios.

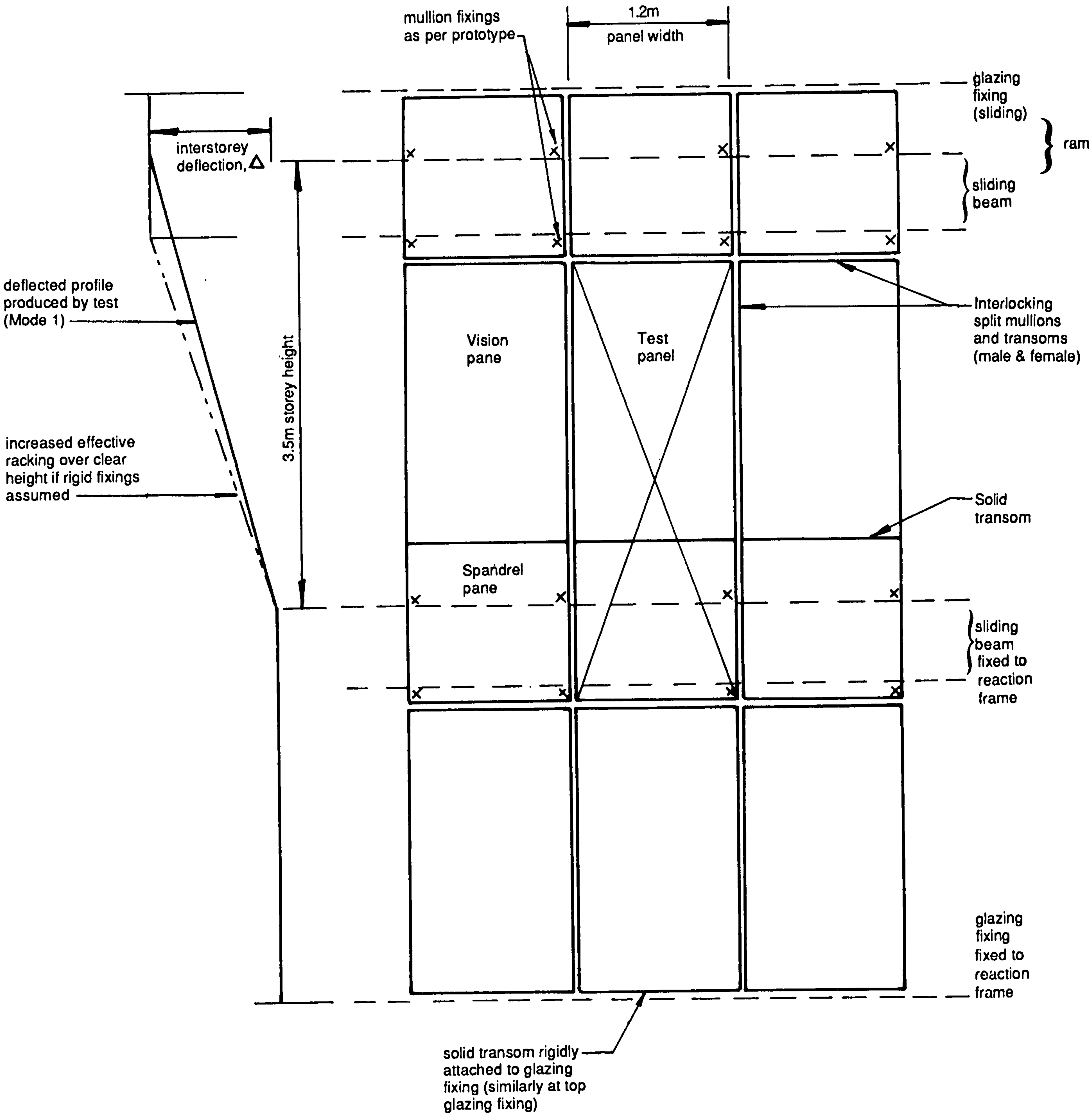
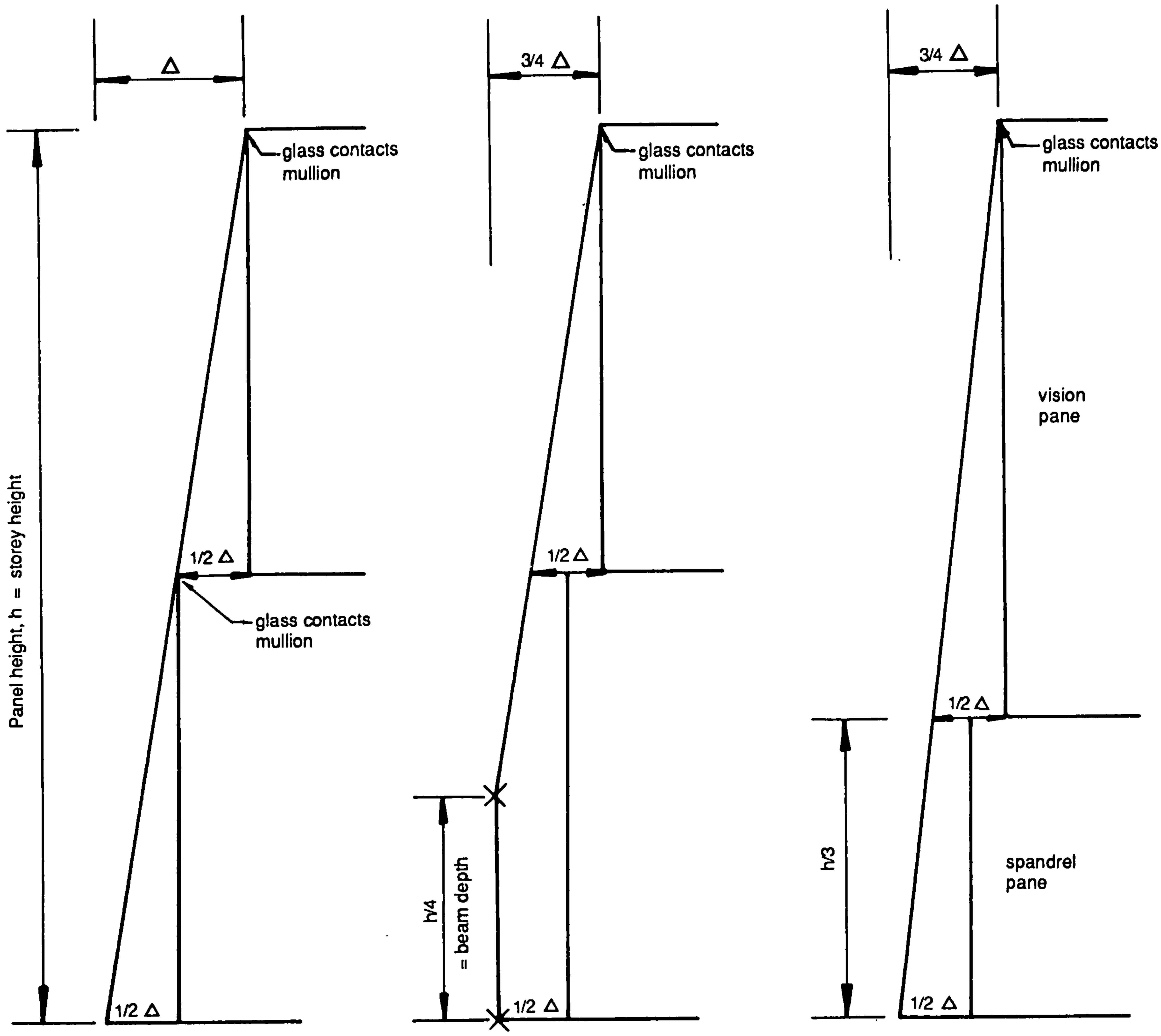


Figure 18: Arrangement for testing Horizon 22 Series' and other similar systems using Mode 1 rig configuration.



(a) transom at mid-height of panel

(b) rigid fixing of mullion at top and bottom of beam, effective racking increased

(c) vision pane and spandrel pane of different heights

Note: Initial lateral glass clearance in frame = 1/2 Δ in each case where Δ = design inter-storey deflection of building

Figure 19: Effect of rigid mullion fixings and relative pane height on lateral deflection capability (assuming no rotation of glass)

SUMMARY AND CONCLUSIONS

This study has been split into three parts. Firstly, an investigation was made into the types of in-plane deformation that can be imposed on a glazing system, the relative magnitudes of these deformations and the significance of their influence on the racking performance of the glazing. This was followed by the development of a procedure for the testing of glazing systems based on the findings of the first part. And finally, a test rig was designed that, while remaining relatively simple, should be able to reproduce the most significant deformation characteristics of buildings subjected to earthquake loading.

Significant conclusions from the study are:

(1) While buildings can be subjected to a large range of deformations during an earthquake, depending on such factors as structural geometry and type, the predominant deformation affecting curtain walling is in-plane racking. Virtually all structural deformations can be resolved into in-plane racking of the glazing.

(2) The effective racking deformation that can be imposed on certain panels within a curtain wall may be significantly greater than the inter-storey deflection of the structure, and any test programme should take account of this.

(3) Although precise limits for lateral building deflection are given by NZS 4203, the manner of their determination is far from precise.

Inter-storey deflections are generally provided to glazing designers by building specifiers, but whether these are always adequate is open to speculation, particularly in view of item 2 above.

(4) Glazing performance should be assessed on the basis of performance levels associated with significant inter-storey deflections, the level of performance required being based on both the uncertainty in determining the inter-storey deflection, and the consequence of failure (both monetary and life risk).

(5) A test programme and test rig have been developed that will allow cyclic testing of large specimens, with inter-storey deflection and building curvature being simulated. The proposed number of cycles is representative of a moderate earthquake but should be confirmed by further computer studies.

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