Performance and testing of fixings for thin stone cladding

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Centre for Window & Cladding Technology 1999

This research report was written by the Centre for Window and Cladding Technology (CWCT) as guidance on the performance and testing of fixings for stone cladding, with particular emphasis on the fixing of thin stone panels. Cladding systems using thin stone units have not been commonly used in the UK and this document aims to remove some of the misunderstanding about their behaviour.

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Introduction

This report is concerned with the fixing of natural stone cladding units as Part of a curtain wall or rainscreen. Developments in the cutting and handling of stone mean that natural stone can now be cut, depending on the stone type and quality, to thickness as low as 20mm. This has a number of implications for the fixing of natural stone units, which make them different from the more traditional use of stone at thickness of 40-75mm. The use and fixing of thicker units is covered by BS 8298: 1994. This report is not intended to duplicate the British Standard but to extend advice on fixing to match the latest developments in the production of stone.

The use of thin stone units means that units of the same area are lighter in weight and that in general fixings are subjected to lower dead loads. The whole system of support for the stone may be different for lighter weight units and in general this means that smaller fixings and anchors may be used to connect the stone.

The reduced thickness of the units means that traditional details, for instance the use of kerfs for fixing, may not perform so well, if indeed they can be cut.

Stone cladding units are subject to temperature and moisture movement. Thinner stone units have less thermal mass and undergo greater thermal movement. Furthermore, improvements in the insulation of walls means that stone cladding may be subject to lower extreme temperatures. Thin stone units are more likely to be saturated through their thickness and undergo greater moisture movement.

This report details the loads and movements on stone cladding, describes the different methods of fixing stone cladding units and methods of test to ensure their satisfactory performance.

Recent developments in the production and cutting of stone have led to the manufacture of structurally assisted stone panels. These comprise a very thin stone veneer reinforced with aluminium or resin/fibre layers. In these products the stone veneer may be as thin as 6mm, and in all cases is too thin to carry wind and dead load without reinforcement. The fixing of these cladding panels is not the main subject of this report although many of the principles discussed will be applicable.

BS 8298: 1994 gives rules for the thickness of stones and fixing details that are to be used unless structural calculation and testing has been performed. This guide aims to advise engineers on how to proceed with appraisal and testing of fixings and fixing details.

Not withstanding these limitations of BS 8298: 1994 it contains much advice on selection of materials, bimetallic corrosion, staining and so forth. This document should be read in conjunction with the British Standard.

Further advice on the selection of stone including guidance on strength and durability testing of stone is available in 'Selection of natural stone cladding panels' - CWCT : 1997.

1.0 Support systems for stone cladding panels

1.1 Introduction

Stone cladding panels may be supported on a building facade in a number of ways. Stone panels are limited in size by the nature of the stone, the loads that are to be carried on the panels, the ability to cut, transport and lift large panels, and the need to accommodate building and cladding panel movement. Invariably the panels are too small to span the grid of the primary building structure and some form of secondary support structure is required.

There are three principal ways in which stone panels can be supported:

- Attachment to a precast concrete panel
- Support from an inner wall of masonry or concrete
- Support from a metal structural framework

1.1.1 Precast panels

The attachment of natural stone cladding panels as a facing to precast concrete panels is normally achieved by casting the precast panel onto the back of the stone panels. This method produces finished cladding units that may even have windows and glazing installed before they leave the factory. This method of construction places the stone panels in close proximity to the precast support panel to which they are fastened at close intervals by inclined interlocking dowels, Figure 1.1. Adhesion between the stone and concrete is prevented to allow relative movement to take place.

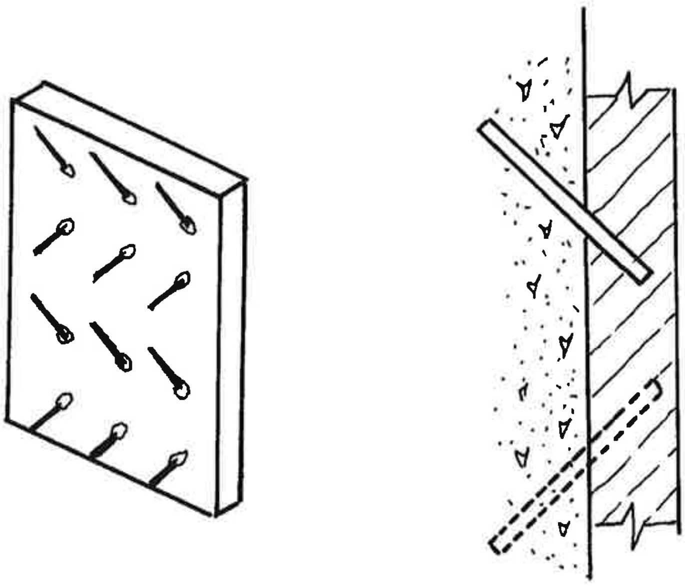

Figure 1.1 Attachment of stone units to a precast panel

This use of natural stone panels is different from the other methods of attachment where the stone is fastened at only a few points at or near to its perimeter. Stone panels fastened in this way behave differently under thermal and moisture movement and carry loads more as a veneer than as a structural element. Impact loading may be a consideration but wind loading and self-weight for each panel generally are not. This method of construction is well understood in the UK where a number of principal suppliers exist and guidance on design and specification has been published, BS 8298, Architectural Cladding Association (1990) and Taylor (1992).

An alternative method of attaching stone cladding to precast concrete panels is to fix the stone panels on site treating the precast panel in the same way as an in situ concrete or masonry support wall. This has the disadvantage of transferring operations from the factory to site. However it may be necessary for the construction of rainscreen walls with deep insulated cavities. Throughout this report the term inner support wall or back wall includes the possibility of using a precast concrete inner wall as well as block or brick masonry walls.

1.1.2 Support walls

Stone cladding panels supported directly from inner walls are site fixed using fixings either into the edge of the stone panels or near to the edge of the panel, Figure 1.2. In the latter case fixings may be into the rear face of the panel or through the panel. This form of support is comparatively rigid, the inner wall undergoes only small deflections and the stone cladding has to accommodate only limited movement of the support system. The stone panels themselves, however, may exhibit considerable moisture and temperature movement. This method of construction can allow for fixings to be attached to the support wall at any point. This obviates the need to co-ordinate the joints between the stone panels with the support system except that movement joints must occur at the same places. The support wall may be load bearing brick or block masonry or an infill panel to a primary structural frame. Note that not all infill panels are structural and brick or block masonry may have to be strengthened with a 'wind pole' or similar reinforcement.

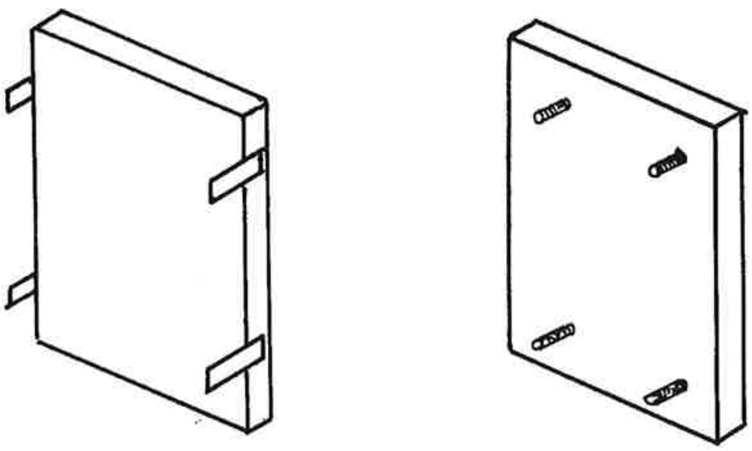

Figure 1.2 Attachment of stone units

1.1.3 Metal support frames

Stone panels may also be supported from a metal framework in which case they are again attached by fixings at or near their perimeter. The metal support frame takes one of two basic forms although both comprise support rails and some method of hooking the panels on or bolting them to the rails.

Support rails may be attached to an inner structural wall at intervals along their length in which case the wall will contribute to their stiffness, Figure 1.3. Deflections of the support system are a function of both the wall construction and the rail construction. In this use the rails may be little more than an alternative way of fixing the panels directly to the inner structural wall. In curtain walling construction a metal framework is used to span between the edges of the floor slabs. This metal framework transmits all loads from the cladding to the primary structure and is assumed to receive no structural assistance from the cladding, Figure 1.4. In this form of construction the deflections of the support rails are greater and the panels and their fixings often have to accommodate greater movement of the fixing points.

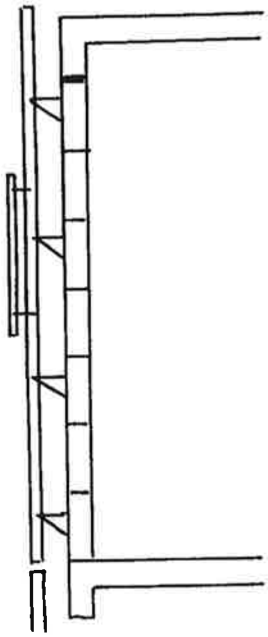

Figure 1.3 Stone cladding and support rails on a structural wall

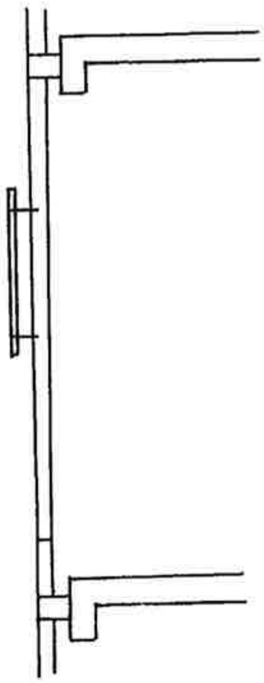

Figure 1.4 Stone cladding fixed to a metal support frame

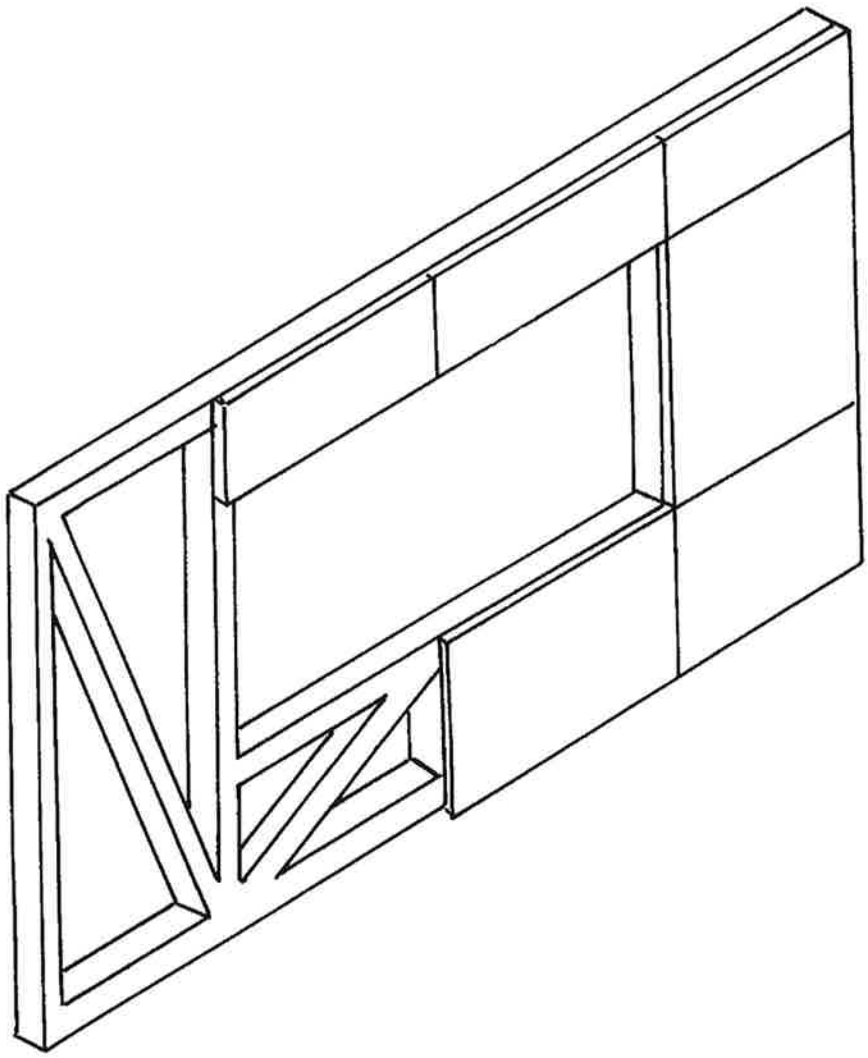

Figure 1.5 Stone cladding on a panellised curtain wall

A further form of curtain walling uses lightweight support rails for the stone but these are supported from a substantial structural steel sub-frame that forms a cladding unit spanning one or two storeys in height and up to a full structural bay in width. This panellised construction enables the manufacture of finished units in the factory, Figure 1.5, and offers the benefit of swift erection on site that is also achieved by using factory finished precast concrete units.

The use of a curtain walling grid often imposes a fixed rectilinear pattern to the panel fixings but it is normally possible to co-ordinate the frame grid with the pattern of joints in the stone cladding. It is however possible to construct support frames of more complex geometry in two or three dimensions. The principal constraint to such constructions is the ability to achieve the required tolerances for the frame and the stone panels, Section 4.

1.2 Load transfer

Loads on the stone cladding panels can be separated into those that act vertically such as self-weight and those that act normal to the face of the panel such as wind loading.

For vertical facades the self-weight loads act in the plane of the wall. It is therefore possible to support each and every stone panel independently by transferring its self-weight directly through the fixings of that panel to the support system Figure 1.6. Alternatively the stone panels may be stacked one on top of the other so that the Self-weight of several panels is eventually transferred back to the support system by a more substantial fixing typically at each floor level, Figure 1.7.

Common usage describes fixings that transfer wind loading only as restraint fixings, while those that transfer Self-weight and wind loading are known as support fixings. In practice all fixings offer restraint in one, two or all three translational degrees of freedom. This document refers to support fixings, - those that offer restraint and carry load in the plane of the panel - and non-support fixings, Section 5. For panels in a horizontal or inclined plane the distinction between support fixings and non-support fixings is more complex, Section 5.

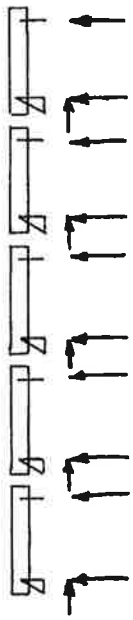

Figure 1.6 Stone cladding supported as individual units

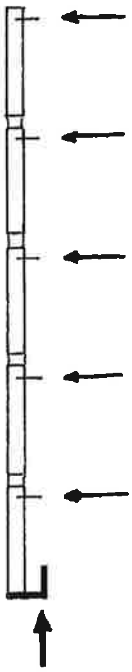

Figure 1.7 Stone cladding supported as a storey height panel

Stone cladding supported from block or brick masonry infill panels is commonly stacked from a support fixing attached at each floor level. Wind loading is then transferred from the stone panels to the back wall by non-support fixings connecting to each stone panel, Figure 1.7. Allowance for movement is made at the top of each bay of cladding immediately below the support fixing for the bay above.

Stone cladding panels supported from a metal framework or support rails in front of a block or brick masonry wall are often attached as separate panels with the Self-weight of each panel transferred through its own fixings. This is done to allow the use of identical fixings across the whole facade and to eliminate the need for large support fixings. Supporting each panel separately means that both horizontal and vertical joints can be left between the panels to accommodate movement more uniformly across the facade.

Properly identified load paths and simple bracketry and fixings facilitate design and testing and eliminate redundancy that may cause unanticipated loads as a result of movement, Section 3.0, and lack of fit, Section 4.

1.3 Wall construction

The external envelope of the building is required to fulfil a number of functions including provision of the external appearance, weathertightness and thermal insulation. Stone cladding is usually chosen for its appearance. It will act in conjunction with the other layers of the wall construction to provide weathertightness but will contribute little to the thermal insulation. The design of the wall as a whole will influence the conditions to which the stone is subjected and hence the required performance of the fixings.

1.3.1 Weathertightness

Stone cladding may be used to form a sealed façade or a rainscreen. In a face sealed wall, the joints between the stone panels are sealed forming the primary barrier to water leakage and the cavity behind the stone panels is only required to drain any unintended water leakage. A secondary seal is provided behind the drained cavity to ensure that air leakage is controlled and that no water passes through the wall.

The use of natural stone in a face sealed curtain wall may place limitations on the allowable movement of the stone panels in order to permit effective sealing of the wall. Relative movement of adjacent panels will influence the design of the sealed joints, CIRIA, 1998.

Rainscreen facades are a layered form of construction in which the outer layer of the wall is not necessarily sealed but acts as a rainscreen. This layer contains lapped and drained joints that prevent entry to most of the rainwater and allow any water that passes the rainscreen to drain outward from the wall. The building is then sealed by an inner wall and air barrier, which control air and moisture movement through the wall. Between the inner wall and the outer rainscreen there is a cavity which allows any water passing the rainscreen to drain and not cross the cavity onto the inner wall. The performance of rainscreen systems depends upon the way in which joints, air gap and backing wall are created.

In the 'drained and ventilated' method, the air gap is left continuous, perhaps running through several storeys, and across several panel widths. With this approach, water penetration is allowed to drain downwards and outwards, and air movement is encouraged to dry out any water that enters the cavity. To enable this ventilation the rainscreen may have all joints left unsealed or open, or may only have unsealed joints at the top and bottom of the air gap.

In the pressure equalised system, the air gap is compartmentalised and the openings are sufficiently large for air to move behind the stone panels and create a pressure on the back of the panels. The reduced pressure difference across the joints limits water penetration. The inner and outer face pressures do not balance exactly and the stone panel will still experience a load from the wind. However a correctly designed pressure equalised rainscreen can reduce the wind loads on the panels by up to thirty percent, CWCT, 1998.

The presence of water in the cavity of rainscreen cladding may affect the extent of saturation of the stone units and hence their moisture movement, Section 3.

Natural stone facades constructed as rainscreens are growing in popularity in the UK and their use is well established in Central European countries. The performance of rainscreen walls is described in 'Standard for walls with ventilated rainscreens', CWCT, 1998.

1.3.2 Thermal insulation

The requirement to use less energy in buildings has led to improved thermal insulation of facades. This means that the outer layer of the wall may experience lower temperatures than used to be the case. Mechanically fixed stone may also be subject to large and more rapid changes in temperature than stone used in more traditional ways. This arises because the stone is thinner and has less thermal mass itself and because it is thermally isolated from any inner wall by an insulated cavity and sparse connections. The resulting thermal movements of the stone have to be accommodated by the fixings and appropriate movement joints, Section 3.

1.4 Design

The primary object in the design of a fixing system is to ensure that the loads on the cladding are safely transmitted to the supporting structure. However it is also necessary to allow for tolerances and movements. Absolute accuracy is not achievable and tolerances define how much variability is allowable. The fixings can be designed to accommodate tolerances by providing adjustment. Changes in environmental conditions and loads will cause movements of the cladding and supporting structure. The fixings must be designed to ensure that these movements do not lead to over stressing of the cladding or fixings.

The design process is illustrated in Figure 1.8. The first stage is to establish the basic parameters of the stone cladding such as the type of stone, size and thickness of panels, details of overall wall construction and the supporting structure. Having established these basic parameters it is possible to calculate the loads, movements and tolerances which the fixing system must accommodate as described in Sections 2, 3 and 4 respectively. Having established the tolerances and movements to be accommodated, one of the fixing types described in Section 5 may be selected taking into account the various aspects of performance described in Section 6. The final stage of the process is to check that the proposed fixings are capable of carrying the proposed loads as described in Section 7. If the proposed fixings are not able to carry the proposed loads it may be necessary to modify the fixing system or change the basic parameters of the cladding, for example by using smaller panels.

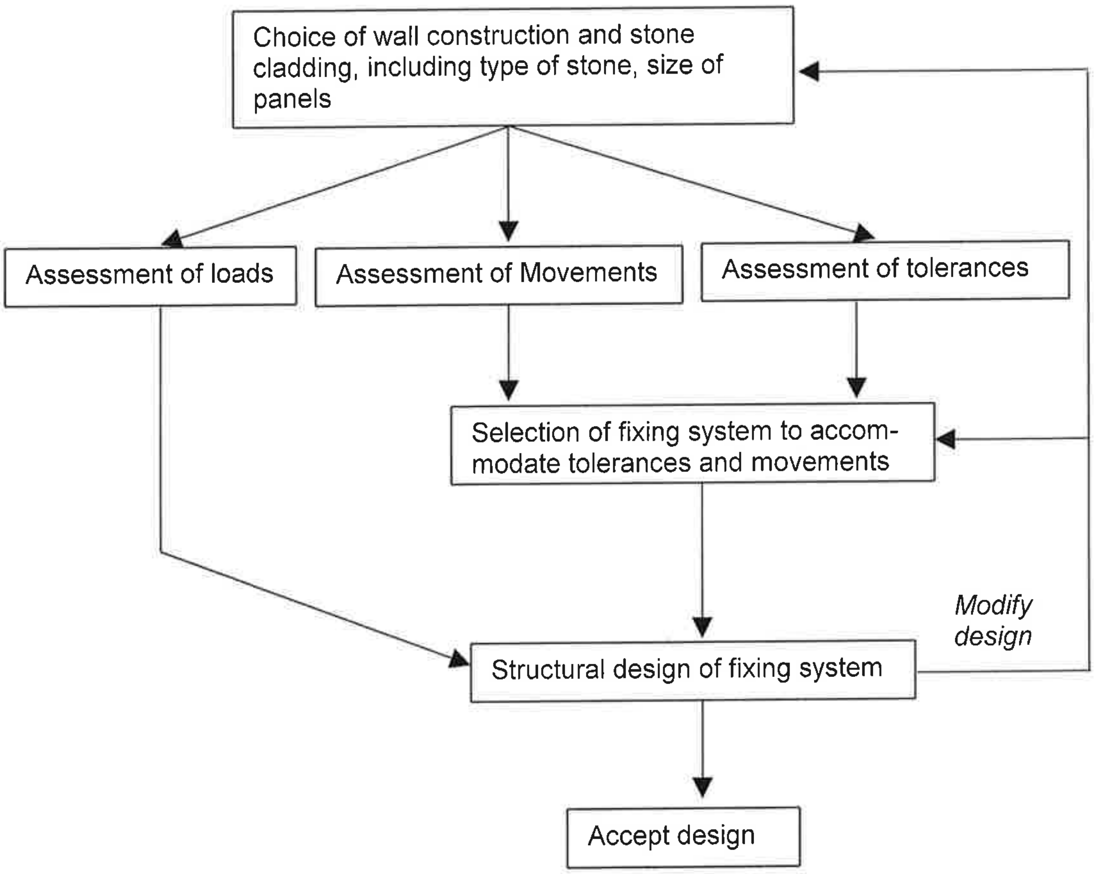

Figure 1.8 Stages in the design of fixings

2.0 Loads

2.1 Introduction

Natural stone cladding panels may be subject to the following forms of loading:

- Self-weight
- Wind load
- Snow and ice load
- Applied loads
- Accidental impact loads
- Access loads

The sections below describe these loads and how nominal values of the loads are obtained. For design purposes these values will normally be increased by the application of a safety factor as described in Section 7.

2.2 Self-weight

The self-weight per unit wall area of the stone is determined by the density of the stone and its thickness. The stones suitable for use as stone cladding panels all have dry densities lying within a narrow range, Table 2.1. However the serviceability condition must assume that the facade is wet and that the stone panels are saturated. The self-weight of the stone becomes the dry weight plus an allowance for the weight of water absorbed by the stone.

Stone type		Dry density kg m^{-3}	Wet density kg m^{-3}
Limestone	Low density	1760	1880
	Medium density	2160	2235
	High density	2560	2590
Sandstone		2160	2360
	Quartzitic	2400	2430
Granite		2560	2565
Slate		2700	2700
Marble		2600 - 2800	2600 - 2800
Quartzite		2560	2570

Table 2.1 Typical density of stones

When calculating the self-weight of the stone panels it is necessary to account for tolerances on the cutting of the panels. The dimension of greatest influence will be the thickness, which may easily be increased by ten percent or more. It is necessary to specify tolerances and the range of finished dimensions not only as lower limits to give an adequate flexural resistance but also as upper limits to prevent excess self-weight loads. A panel required to have a minimum thickness of 30mm could be specified as 33 +/- 3 mm and supplied at a thickness of 36mm, a 20 percent increase in Self-weight, Section 4.

It is easy to fall into the trap of reducing the load on the individual fixings by simply providing more fixings for each panel. However this may increase the degree of redundancy in the attachment of the panel and induce additional loads in both the panel and the fixings as a result of movement, Section 3.

The use of only two support fixings per panel is recommended so that the load is carried equally on each. The use of three or more support fixings will comprise a redundant system of support and the load path will depend on the exact positioning of the fixings. In general the load will still be carried on just two fixings one of which may carry most of the load, Figure 2.1. If the load on a fixing is too great then either a different design of fixing or a smaller stone panel should be used.

Self-weight loads will act through the mid-plane of the stone panels and will be offset from the support frame or wall. The design of fixings, brackets and support frames should take account of the eccentricity of the load and be capable of carrying any resulting moments or torques, Figure 2.2. The eccentricity of the self-weight load will be determined by the finished position of the stone relative to the support frame or wall. The eccentricity assumed in calculation and testing should take account of the construction tolerances, Section 4.

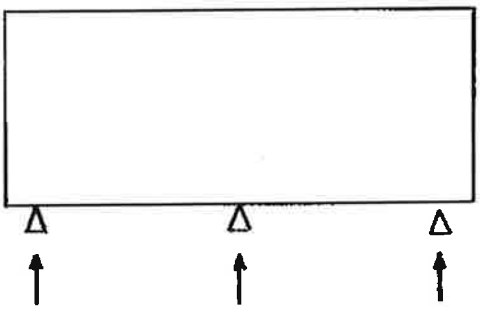

Figure 2.1 Redundant supports – the central support will carry most of the load

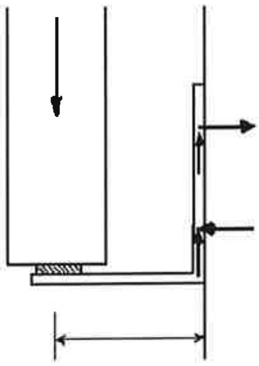

Figure 2.2 Load paths

2.3 Wind loading

Wind load acting normal to the stone panel may be a positive or negative pressure. In addition to the load on the outer face there may be a wind pressure acting in the cavity of a rainscreen wall creating a load on the inner face of the panel. Under these circumstances the panel and its fixings have to resist the net pressure acting on it. Any wind pressure acting inside the building will act on the inner layers of the wall and not the outer layer of stone. Wind loadings are calculated in accordance with BS 6399 Part 2. The wind loading on a panel is calculated as:

$$p = q\ C_{pe}$$

or, for a pressure equalised wall:

$$p = q\ (C_{pe} - C_{pc})$$

Where p = design wind pressure

q = factored wind pressure

C_{pe} = Co-efficient of external pressure

C_{pc} = Co-efficient of cavity pressure

Values for q and C_{pe} are obtained in accordance with BS 6339 Part 2. C_{pc} depends on the degree of pressure equalisation obtained as described in CWCT, 1998. In the absence of a clear understanding of the pressure in the cavity it should be assumed that the panel receives no benefit from pressure equalisation. The panel then has to carry the full wind loading on the outer face of the building.

2.3.2 Cyclic wind loading

The serviceability loading of BS 6399 Part 2 is assumed to occur only once in the design life of the facade however smaller, but significant, loads occur many times during the life of the facade. The cladding panels are subject to a complex wind loading that changes from positive to negative loading and varies continuously with time. This form of loading, particularly the reversal of loading can lead to failure of panels and fixings in a form of low cycle fatigue. It is not possible to exactly replicate this loading sequence in the laboratory but an equivalent loading sequence has been developed by the BRE, Table 2.2.

Wind load	No of cycles	
90%	1	)
40%	960	)
60%	60	) Repeat this sequence
50%	240	) five times
80%	5	)
70%	14	)
100%	1	Final load

Table 2.2 Fatigue wind loading from BRE Digest 346, Part 7, 1989

2.4 Snow and ice loads

Snow and ice are not common on vertical facades in the UK. However loads may arise where snow can rest on ledges and sills. Designers should satisfy themselves that these loads will not occur or that they can be carried safely. Snow loadings for the UK are given in BS 6399 Part 3.

2.5 Applied loads

Applied loads may act on stone cladding and its fixings when external services or fixtures such as lights and signs are attached to a facade. It might be possible to carry these loads through the stone panels and their fixings but it is preferable to carry the loads direct to the support wall or frame used to support the stone.

2.6 Accidental impact loads

Impact loads may occur adjacent to trafficked areas around the building such as paths, roads and balconies. Vehicle impact loads will normally be so great that it is not possible to design panels and fixings to resist them. In this case it may be appropriate to provide some form of protective traffic barrier or use a more robust form of cladding in the areas at risk. Impact loads from pedestrians are of smaller magnitude and may be represented by tests such as the soft body impact test described in BS 8200. This specifies two magnitudes of impact suitable for the design of cladding panels.

When designing fixings for stone panels, sensible design criteria would be either that the panel and fixings should survive a specified soft body impact test or that the stone panel should fail in flexure before failure of the fixings or of the stone adjacent to the fixings.

2.7 Access loads

Loads may arise during maintenance and repair of the facade depending on the method of access. A load of 500N acting over a square with 100mm sides may be taken to represent the force due to a person standing on a ladder leaning against the facade. This load may damage lightweight cladding but is unlikely to be of concern for most stone cladding panels. When designing and testing fixings it should be taken as a single point load at the fixing.

Greater loadings can arise from impact by maintenance cradles. An assessment should be made of likely impact loads arising from maintenance access and panels and fixings should be designed accordingly.

2.8 Induced loads

Loads may be induced in the stone panels and fixings as a result of redundancy of the fixing system. This can arise from thermal and moisture movements that are restrained by the system of fixing, Section 3. Flexure of the panel under wind loading may be restrained by fixings that do not allow rotation, Section 3.

A further cause of induced load is lack of fit as a result of induced deviations, Section 4. Panels are generally fastened such that there are four paths to carry wind loading back to the structure. If at the time of fixing the stone three fixings are adjusted to set the panel in its correct position it is possible, by adjusting the fourth fixing, to pull or push the panel and this will induce unintended stresses in the panel and its other fixings, Figure 2.3. Correct site procedures have to be followed to ensure that no induced loads arise as a result of the fixing process.

Induced loads are not easily calculated and fixing systems should be designed to eliminate the possibility of induced loads wherever possible.

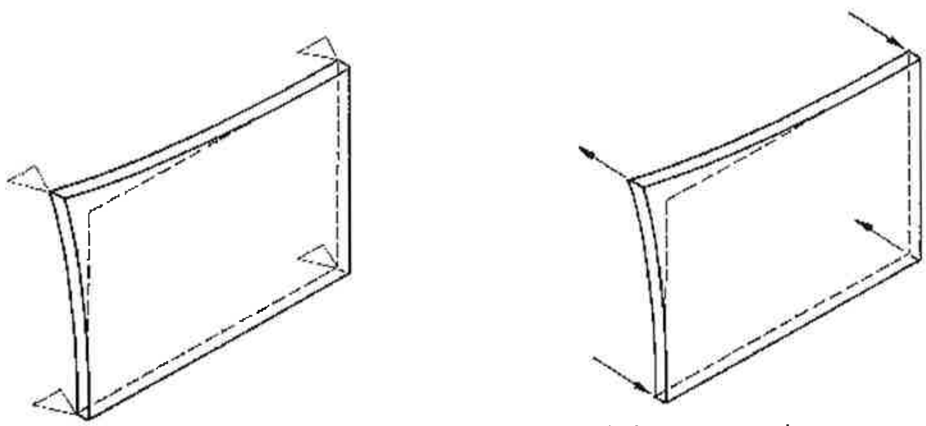

Figure 2.3 Restraints and induced loads in a stone unit

3.0 Movement

3.1 Introduction

Natural stones undergo expansion and contraction with changes in temperature and degree of saturation. Moisture and temperature changes may also cause movement of the support wall or frame. Failure to accommodate these movements at the fixings can lead to over stressing and failure of stone cladding panels.

Thin stone panels undergo greater flexural deflections under wind loading than thicker panels used in more traditional construction. Failure to accommodate edge rotation or curvature at the fixings may also lead to over stressing and fracture of the stone cladding panels.

Building movements arise from:

- Wind sway
- Floor deflections
- Settlement
- Creep
- Temperature change
- Moisture change
- Earthquake

Each of these will give rise to relative movement of the primary building structure and the cladding attachment points. It is important that the fixings for stone cladding panels can accommodate these movements and prevent the transfer of loads to the cladding. This can be done by using a support frame or wall separated from the movements or by fixing the individual cladding panels in such a way that allows movement.

Lightweight metal frames and support rails used to carry thin stone cladding will flex under wind loading. This causes both rotation and displacement of the panel fixings. Fixings have to accommodate rotation and should be positioned so that movement can take place without over stressing of the stone panels.

3.2 Thermal and moisture movement of panels

These movements are of two fundamentally different forms depending on whether or not the temperature and moisture levels are constant through the thickness. For uniform conditions throughout the stone linear expansion or contraction occurs. More normally the temperature and degree of saturation of the stone vary from the outer face exposed to the elements to the sheltered inner face. In this case dishing or bowing of components and panels occurs. The surface temperatures will depend on the colour and thickness of stone, its orientation and any shading that it experiences. Typical ranges for the outer surface temperature are given in Table 3.1.

Material	Colour	Min °C	Max °C	Range °C
Heavy weight	Light	-20	50	70
Heavy weight	Dark	-20	65	85
Light weight insulated	Dark	-25	80	105
Light weight insulated	Light	-25	60	85

Table 3.1 Surface temperature ranges for the UK from BRE Digest 228 : 1979

3.2.1 Thermal movement

Changes in temperature of stone cause thermal strains proportional to the change in temperature. These depend on the stone used and typical stone properties are given in Table 3.2.

Stone type	Co-efficient of thermal expansion $10^{-6}\,K^{-1}$	Modulus of elasticity $kN\,mm^{-2}$
Limestone	3 to 10	40 to 90
Sandstone	7 to 12	8 to 18
Granite	8 to 10	38 to 76
Slate	6 to 12	
Marble	3 to 15	
Quartzite	9 to 12	74 to 77

Table 3.2 Typical stone properties

The thermal strain can be calculated from:

$$\varepsilon_t = \alpha\ T$$

where ε_t = thermal strain

α = Co-efficient of thermal expansion (Table 3.2)

T = temperature change

The thermal strain, if unrestrained, will give rise to a change in length:

$$\Delta L = L\,\alpha\ T$$

where ΔL = change of dimension

L = length of dimension

However if the stone is fully restrained so that no thermal movement can take place then thermal stresses are induced in it. These can be calculated from:

$$\sigma_t = \varepsilon_t\ E$$

where E = modulus of elasticity of the stone

σ_t = thermal stress

Example 1

A sandstone panel with dimensions and properties:

Height	*500 mm*
Width	*800 mm*
Thickness	*40 mm*
Co-efficient of thermal expansion (α)	$10^{-5}\ K^{-1}$
Elastic modulus	$12\ kN\ mm^{-2}$

experiences temperatures from -20^{o} *C when the building is unoccupied in the winter to* 50^{o} *C in the summer sun. It will undergo uniform thermal strain of:*

$$\varepsilon_t = 0.00001\ x\ (50 - (-20)) = 0.0007$$

and change in dimension of:

$$\text{Change in width} = 0.0007\ x\ 800 = 0.56\ mm$$

$$\text{Change in height} = 0.0007\ x\ 500 = 0.35mm$$

If this movement is totally restrained by a closed joint or rigid fixing then a compressive stress may occur of:

$$\text{compressive stress}\ \sigma_t = \varepsilon_t\ x\ E = 8.4\ N\ mm^{-2}$$

If the panel is installed on a hot day and the thermal contraction is restrained by rigid fixings then a tensile stress of:

$$\text{tensile stress}\ \sigma_t = \varepsilon_t\ x\ E = 8.4\ N\ mm^{-2}$$

will be generated and cracking may occur.

3.2.2 Thermal dishing of panels

Where stone cladding panels are subject to rapid change of surface temperature, normally from solar warming or nighttime radiation cooling, the temperatures through the thickness of the panel are not uniform. The outer face of the stone experiences a greater thermal strain than the inner face resulting in curvature of the panel, Figure 3.1. If the stone is unrestrained this will result in dishing of the panel as it deflects to take the same curvature in every direction and becomes part of the surface of a sphere, Figure 3.2. If the panel is free to adopt this shape then it will and thermal stresses in the stone will be minimised (there will still be some thermal stress generated if the temperature gradient through the stone is non linear but these stresses will normally be very small). If, however, the stone is prevented from adopting this shape then thermal stresses will be generated. For this reason the fixing positions should always be chosen so that they can be located on the surface of a sphere. This condition can be met by using just three fixings. However it is normally more convenient to use four fixings. In this case all of the fixings should be located so that they sit on a circle, and hence a sphere, Figure 3.3. Provided all the fixings are located on a circle any number of fixings may be used.

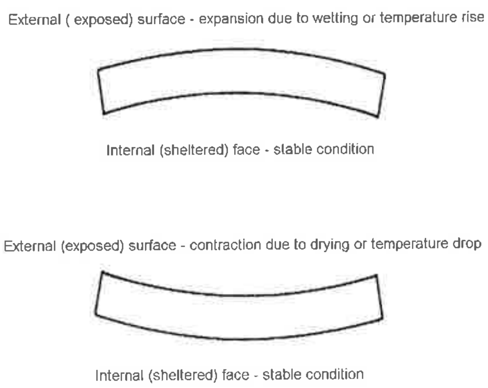

Figure 3.1 Curvature due to thermal or moisture gradient

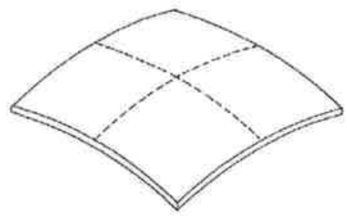

Figure 3.2 Dishing of stone panel due to biaxial curvature

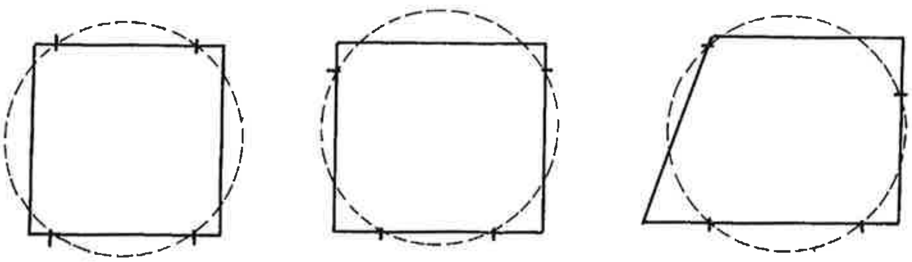

Figure 3.3 Location of fixings to allow dishing of units

Thinner stone panels undergo greater out-of-plane deflection for a given temperature difference across the panel. However restraint of the thermal movement gives rise to the same stresses in the stone irrespective of the thickness. Thinner panels are more susceptible to cracking as a result of thermal or moisture gradients because they displace further and are therefore more likely than thicker panels to meet a restraint. Thinner panels are also more likely to be subject to a temperature gradient due to the reduced thermal mass. Proper fixing design will eliminate this problem.

Example 2

Taking the panel of example 1, if the inner face of the panel remains at 20° C while the outer face is heated to 50° C then the thermal strain at the inner face will be zero while that at the outer face will be:

$$\varepsilon_t = 0.00001 \times (50 - 20) = 0.0003$$

The change in width of the faces will be:

$$\text{outer face} = 0.0003 \times 800 = 0.24 \text{ mm}$$

$$\text{inner face} = 0 \text{ mm}$$

The resulting dishing in the panel can be calculated as follows;

$$\theta\,(r - 20) = 800$$

$$\theta\,(r + 20) = 800.24$$

where r is the radius of curvature and θ is the angle subtended at the centre of curvature by the length of the panel (see Figure 3.4).

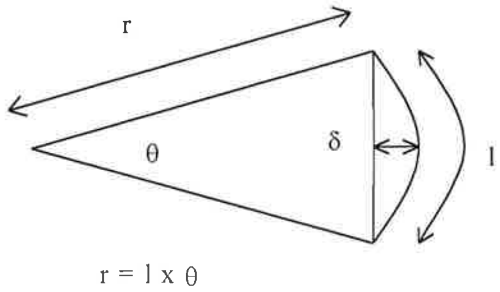

Figure 3.4 Relationship between deflection and curvature

eliminating θ gives

$$r = 20 \times 1600.24 \ / \ 0.24 = 133400 \ mm$$

and mid-span deflection can be calculated from:

$$(2 \times r - \delta) \times \delta = (l/2)^2$$

where δ is the deflection and l is the length of the panel. Substituting the known values gives:

$$(2 \times 133400 - \delta) \times \delta = 400^2$$

from which:

$$\delta = 0.6 \ mm$$

If the panel is restrained so that these deflections cannot occur then thermal stresses are generated:

$\sigma_t = 0.5 \times \varepsilon_t \times E = 1.8 \ N \ mm^{-2}$ *compression on the outer face*

$\sigma_t = -0.5 \times \varepsilon_t \times E = -1.8 \ N \ mm^{-2}$ *tension on the inner face*

for a similar panel of thickness just 20 mm the corresponding deflection is 1.2 mm and the deflections are:

Middle of top/bottom edge	*1.2 mm*
Middle of side	*0.9 mm*
Centre of panel	*2.1 mm*

If the panel is again restrained then the same thermal stresses are generated:

$\sigma_t = 0.5 \times \varepsilon_t \times E = 1.8 \ N \ mm^{-2}$ *compression on the outer face*

$\sigma_t = -0.5 \times \varepsilon_t \times E = -1.8 \ N \ mm^{-2}$ *tension on the inner face*

3.2.3 Thermal movement of supports

Support rails within the cavity of a wall are in a sheltered position and unlikely to experience the same extremes of temperature as the exposed stone panels. Furthermore the support rails are likely to be at a similar temperature to the back face of the stone panels. However rails of aluminium or steel will expand and contract to a greater extent than the stone, Table 3.3, unless restrained.

Material	Co-efficient of thermal expansion $10^{-6}\,K^{-1}$	Modulus of elasticity $kN\,mm^{-2}$
Steel	12	200
Aluminium	24	70
Glass	10	70
Brick and Block masonry		
Concrete	12	15 to 30

Table 3.3 Properties of other materials

Example 3

Aluminium support rails are used to span between two floors and support limestone panels. The floor to floor height is 3000 mm and the limestone has a thermal Co-efficient of expansion $\alpha = 6 \times 10^{-6}\,K^{-1}$. Assuming the cladding and support rails both experience the same temperature range of -10^{o} C to 50^{o} C, the rail will experience a change of length of:

$$\Delta L = 3000 \times 24 \times 10^{-6} \times (50 - (-10)) = 4.32\ mm$$

and the limestone will experience a change of length of:

$$\Delta L = 3000 \times 6 \times 10^{-6} \times (50 - (-10)) = 1.08\ mm$$

A differential movement of 3.24 mm per storey has to be accommodated at the fixings and the joints, either between individual panels or at each floor level.

3.2.4 Moisture movement of panels

Change of dimension due to moisture will occur for some natural stones. These are generally reversible movements for the stones used in cladding panels and can be calculated from:

$$\varepsilon_m = f_m / 100$$

Where ε_m = moisture strain

f_m = factor of moisture movement given in Table 3.4.

The strains calculated in this way may be converted to changes of dimension from:

$$\Delta L = L\ \varepsilon_m$$

Where L = dimension

ΔL = change of dimension

Material	Repeated movement f_m (%)	Initial movement (%)
Limestone	0.01	Nil
Sandstone	0.07	Nil
Granite/Slate	nil	Nil
Marble	see below	
Concrete	0.02 to 0.1	0.03 to 0.1
Concrete Bricks	0.02 to 0.06	0.02 to 0.09
Clay or shale bricks	0.02	0.02 to 0.07

Table 3.4 Moisture movement from BRE Digest, 228, 1979

Example 4

A sandstone panel of width 800 mm, height 500 mm and thickness 30 mm becomes saturated throughout its thickness. This causes it to expand by 0.07 % and its width increases by:

$$\Delta L = 0.0007 \times 800 = 0.56 \text{ mm}$$

If the same panel is saturated on its outer face but dry at its inner face the change in width of the outer face will be 0.56 mm while the inner face experiences no change.

The resulting dishing of the panel can be calculated as follows;

$$\theta\,(r - 15) = 800$$

$$\theta\,(r + 15) = 800.56$$

$$r = 15 \times 1600.56 \,/\, 0.56$$

$$r = 42870 \text{ mm}$$

and deflection at the middle of the top or bottom edge can be calculated from:

$$(2 \times 42870 - \delta) \times \delta = 400^2$$

from which:

$$\delta = 1.9 \text{ mm}$$

If the panel is restrained so that these deflections cannot occur then stresses are generated. If the elastic modulus is 12 kN mm^{-2} these are:

$$\sigma_m = 0.5 \times 0.0007 \times E = 4.2 \text{ N mm}^{-2} \text{ compression on the outer face}$$

$$\sigma_i = -0.5 \times 0.0007 \times E = -4.2 \text{ N mm}^{-2} \text{ tension on the inner face}$$

As with thermal movement, moisture movement frequently leads to dishing of the panels. Either the outer face experiences greater wetting and expands more than the inner face or a fully saturated panel dries from the outer surface. Again, if the panel is not free to adopt a spherical shape stresses will be induced in the panel and fracture may occur.

3.3 Flexural movement

Support rails, frames, and the cladding panels themselves will all undergo flexure as a result of the lateral loads on the cladding.

3.3.1 Support rails and frames

Support rails and frames will deflect as a result of wind loading on the cladding panels. Rails should be designed to limit deflections to acceptable levels. Rails spanning from floor to floor are usually limited to deflections of 1/200 storey height. Localised deflections are limited to 1/360 of the panel dimension, Figure 3.5, CWCT 1996. These deflections lead to rotations of the fixings which have to be accommodated. Where adjacent support rails undergo different deflections, for instance at the end of a bay of cladding, the fixing points may not remain in one plane.

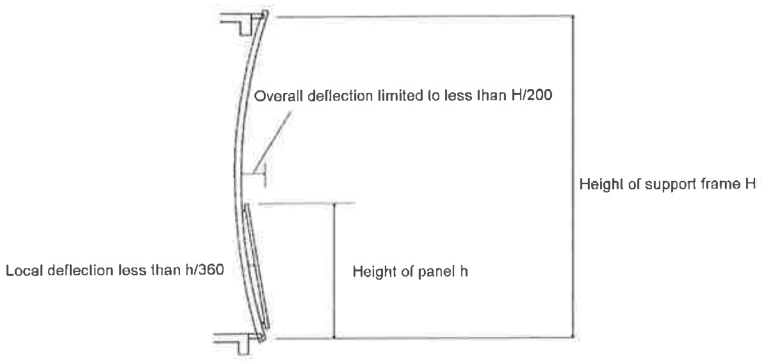

Figure 3.5 Deflection of curtain walls

Deflection of the support structure may also cause changes in the distance between fixing points for the cladding in the plane of the cladding. The significance of this effect will depend on the method of fixing the stone and will be greater for independently fixed stones than stones stacked in storey height panels. The change in distance between fixing points will depend on the offset of the cladding from the neutral axis of the support rails. For a 3m high cladding rail, with a mid-span deflection of 15mm and the plane of the cladding offset from the rail by 100mm, there will be a change in length of approximately 4mm in the height of the cladding zone. This movement will be distributed between the joints over the storey height with a corresponding and opposite movement concentrated in the joint between successive storeys.

3.3.2 Cladding panels

Cladding panels subjected to wind loading undergo flexural deformation and this gives rise to rotation of the panel at the fixing. These rotations are greater for thinner stone panels and become important when designing fixings for very thin stone panels.

Example 5

The deflections of square panels are governed by equations of the form, Timoshenko and Woinowsky-Krieger, 1970;

$$\delta = \alpha \; w \; a^4 \; / \; D$$

Where w = *wind loading*

a = *length of side*

α = *Co-efficient dependent on fixings*

$$D = E \; t^3 \; / \; 12\,(1 - v^2)$$

Where E = *elastic modulus*

t = *panel thickness*

v = *Poisson's ratio*

It follows that:

displacement, δ, *is proportional to* a^4 / t^3

Similarly:

edge rotation, θ, is proportional to a^3 / t^3

Reducing the thickness of a panel from 40mm to 30mm increases the deflection by:

$$(40/30)^3 = 2.4$$

Reducing the thickness of a panel from 40mm to 20mm increases the deflections and edge rotation by:

$$(40/20)^3 = 8.0.$$

3.4 Building movement

3.4.1 Structural movement

This may occur as a result of loadings on the building or settlement of foundations. Typical allowable deflections for structural frames are given in Table 3.5. Note that greater deflections may be experienced at the edge of a floor slab that cantilevers over a beam. Sway deflections may be reduced for structures that are braced or constructed with shear walls, for instance masonry infill panels to a concrete frame. Sway due to an earthquake may be greater. In addition to these movements elastic shortening of the columns will occur. The Structural Engineer should be consulted on the movements of a particular building.

	Concrete	Steel
Beam with brittle finishes	L/360	L/360
Beam	L/300 - L/360	L/200
Cantilever		L/180
Sway	H/300	H/300
Settlement	L/500	L/500

Table 3.5 Typical building structural movements
(limiting deflection ratios; L = length, H = Height)

3.4.2 Temperature movement

Movement of the primary building structure due to temperature change can be ignored when designing stone cladding provided the primary structure is protected by the cladding and is not subject to extremes of temperature. An external building frame will of course be subject to extremes of temperature and the Structural Engineer should be consulted to establish the temperature movements.

3.4.3 Moisture movement

This takes two forms. Reversible movement occurs as the material is wetted and dried in use. Initial movement takes place following construction as a one-off movement resulting from drying or moisture uptake of the construction materials. Significant reversible movements of the building frame are unlikely to occur if it is protected from wetting by the cladding. However, concrete frames are subject to initial movement as are infill panels of brick or block masonry and load bearing masonry walls. Movements can be calculated using the factors from Table 3.4. Typically a concrete frame will shorten by 2 to 3mm in a storey height. A blockwork wall will shorten by a similar amount. Clay and shale brickwork will expand rather than contract.

Example 6

A concrete building frame with storey height of 3500mm will experience a one off shrinkage movement in the range:

$$\Delta H = H \ \varepsilon_m = H \ f_m / 100$$

$$\Delta H = 3500 \ x \ 0.03 / 100 \ \text{to} \ 3500 \ x \ 0.1 / 100$$

$$\Delta H = 1.05 \ \text{to} \ 3.5mm \ \text{per storey height}$$

This shortening of the columns will reduce the width of the joints in the stone cladding which must be designed to accommodate this movement in addition to other cladding and structural movements.

3.5 Accommodation of movement

If all parts of the cladding and the supporting structure were to move by the same amount, there would be no need to accommodate movements at the fixings. However differences in the movements of different elements of the structure must be taken into account in the design of fixings. Differential movements may arise due to differences between the overall vertical movements of the cladding and structure, differences in horizontal movements of cladding and structure or variations in the movements of different parts of the structure due to settlement or deflection for example.

To accommodate movements it is desirable to have only one fixing point per panel which is fixed in both in plane horizontal and vertical directions. To carry the dead loads it is necessary for one other fixing to be fixed in the vertical direction but horizontal movement should be allowed. Where the horizontal movement to be accommodated is small there may be sufficient inherent flexibility in the fixing selected to allow movement to take place without further action by the designer. However where larger movements occur or rigid bolted fixings are used additional provisions may be required to permit movement to take place.

Vertical movements are often larger than horizontal movements as there are more potential sources of vertical movement and movements are often concentrated at greater spacings coinciding with the floor levels. Movement joints must be provided to accommodate the vertical movement of the facade. These are usually at each floor level unless the stone cladding panels have open joints between them that can accommodate the movement. If joints are positioned at each floor level relative movement between the stone cladding panels and a back wall or support frame still have to be accommodated at the intermediate fixings.

The in-plane shear that occurs as a result of sway and settlement can be accommodated in a number of ways depending on how the cladding is attached to the primary structural frame. Vertical and horizontal movement joints are incorporated in the cladding as appropriate. The movement joints provided to accommodate vertical movement will also accommodate the horizontal sliding of panels one past the other. Vertical joints have to be provided to allow for settlement.

Movement joints should be designed to accommodate the total movement that may occur as a result of all the sources described above. However account may be taken of the fact that not all movements will occur simultaneously. For example high temperatures will cause drying out hence the maximum thermal expansion is unlikely to coincide with the maximum moisture expansion.

3.6 Sub-frame movement

Where stone cladding is fixed to a complex, possibly three dimensional, lightweight metal carrier frame, the fixing points may undergo comparatively large and complex movements. This behaviour is complicated because the stone cladding panels may contribute to the stiffness of a light structural frame and will share some of the load unless the fixings are very flexible. In some cases it may be desirable to use the stone panels as shear panels that transmit loads and stiffen the structure.

3.6.1 In-plane movement

In-plane movement of the fixings and support frame will lead to direct stresses in the stone panels and shear loads on the fixings unless they are sufficiently flexible to isolate the panels from the frame movement. Fixings that are not able to accommodate movement should be designed to carry the full loads transmitted to the panels and a structural engineer should carry out full stress calculations for the frame and cladding as a whole or oversee appropriate testing. Stone panels are good at transmitting in-plane loads but are very stiff in shear and attract large forces at the fixings. The strength of fixings is most critical in this use of stone panels.

3.6.2 Out-of-plane movement

Out-of-plane movement of the fixings and support frame will lead to flexure or rotation of the stone panels. Plain rotation of the panels will not stress them provided that the fixings can accommodate the rotations. Because panels are normally connected by more than three fixings the attachment is a structurally redundant system and out-of-plane movement of the support points such that they do not remain co-planar will lead to flexure of the stone panels. Induced tensile stresses may then cause fracture of the panel, Figure 3.6. Fixings should be designed to accommodate out-of-plane movements and eliminate flexure of the panels.

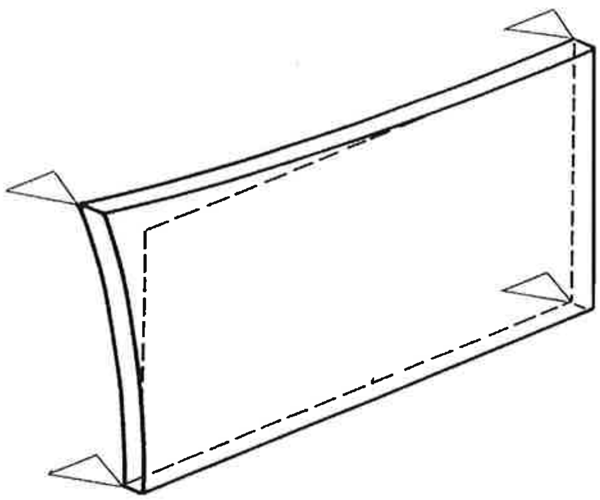

Figure 3.6 Movement inducing flexural stress

4.0 BUILDING AND CLADDING TOLERANCES

4.1 Introduction

Good appearance of a building facade depends on the satisfactory fit, position and alignment of cladding panels. Buildings that are clad with natural stone are normally prestigious buildings and the finished appearance of the installed cladding is paramount. The support system and fixings used to attach natural stone panels have to be capable of adjustment to provide the required degree of fit.

Fit is not merely being able to, for instance, insert one component within another, but also requires that the assembly functions as intended. Satisfactory fit is rarely achieved by accident - it is the result of good planning by all participants. This requires an understanding of some fundamental concepts, which are defined and described below.

- Deviation: The difference between a specified nominal value and an actual measured value. There are two types of deviation.

- Induced deviations: The inevitable result of variations in the manufacturing process and the inability of human operatives and instrumentation to measure with absolute precision. Induced deviations are permanent and whilst the design needs to take account of their presence at the time of construction, it is not necessary to allow for further induced deviations in service.

- Inherent deviations: Inevitable inaccuracies due to the physical properties of materials. Inherent deviations are post-installation dimensional changes and movements in both the structure and the cladding. Sources of inherent deviations include moisture and thermal movement, elastic deformations, concrete creep and shrinkage and foundation settlement. Inherent deviations are discussed in Section 3.

- Tolerance: Tolerances define the limits of induced deviation for which allowance should be made in design, and within which actual sizes are acceptable. Tolerances therefore describe how much variability is allowable, not how much is present, which is termed deviation (see above).

Building structures are assembled on site with comparatively large deviations whilst factory cut stone panels are produced with smaller but significant deviations. Support systems and fixings have to be adjustable over a range that allows for large induced deviations in the structural frame. Fixings also have to be capable of fine adjustment to allow the accurate final positioning of the panels.

The finished appearance of the cladding is often specified as permissible deviations on joints, position, and more importantly relative position of cladding panels. It should be remembered that whilst some parts of a building are viewed close up other parts of facades are viewed at a distance. It may be appropriate to consider different forms of fixing on, for instance, upper storeys to simplify the fixing of the panels.

4.2 Building tolerances

The primary structural frame of the building will be constructed to tolerances that are agreed between the contractor and specifier. The induced deviations of structural frames are a continual source of problems when installing cladding. It is common practice to specify tolerances as two sided tolerances (+/- values) but it is more meaningful to specify one-sided tolerances that aim to prevent the construction of the structural frame in the designated cladding zone. Fixings then have to be designed to bridge the gap between the cladding and the finished position of the structural frame.

Characteristic building tolerances, as measured on completed buildings, are given in BS 5606. Values for structural frames are given in Table 4.1. It should be noted that these values are based on statistical principles and even when the construction is carried out to the intended standard, there will be a chance of approximately 1 in 20 that the actual deviation will lie outside these tolerances. Contractors may work to tolerances different from these and it is necessary to agree tolerances for each individual project.

Material	Between columns	Between floors
Brickwork	+/- 20mm	
In-situ concrete	+/- 18mm	+/- 22mm
Precast concrete	+/- 13mm	+/- 23mm
Steel	+/- 12mm	+/- 20mm

Table 4.1 Characteristic building tolerances

4.3 Stone panel tolerances

Deviation in the size and shape of natural stone cladding panels will depend on the method of measuring and machining the stone and on the overall size of the panel. The stone supplier should be consulted to ascertain the achievable tolerances on a particular contract.

The overall finished dimensions of the panel, squareness of the panel and location of fixing holes will affect its positioning and will dictate the need for fixings to be adjustable in the plane of the panel. The thickness of the panel and the location of kerfs and mortices will dictate the need to adjust the panel normal to the plane of the cladding.

The actual deviations on the stone panels are frequently greater than the tolerances specified for fit and position on a facade and some form of fine adjustment is normally required in all directions.

Achievable panel deviations are given in Table 4.2. Many suppliers would regard these as large deviations and are able to supply stone to finer tolerances.

Dimension	Range	Deviation
Height	*for units up to 50 mm thick*	+/- 2mm
Width	*for units up to 50 mm thick*	+/- 2mm
Diagonal		lesser of +/- 5mm *or 0.5%*
Thickness	up to 50mm	+/- 3mm
Kerf width		+/- 1.5mm
Kerf position		+/- 1.5mm
Hole position in face		+/- 6mm

Table 4.2 Achievable deviations of stone panels

4.4 Finished tolerances

Tolerances for curtain walling are given in 'Standard for curtain walling' CWCT, 1996, Table 4.3. There are also limits on joints. Widths of fixed joints should not vary by more than 10% of the joint width and offsets at joints should not exceed 10% of the joint width either in plane or out of plane.

	In any one storey	Overall
Line	+/- 2mm	+/- 5mm
Level	+/- 2mm	+/- 5mm
Plumb	+/- 2mm	+/- 5mm
Plane	+/- 2mm	+/- 5mm

Table 4.3 Permissible deviations for curtain walling

These tolerances are intended to give satisfactory appearance, which is affected by relative positions of the visible cladding elements rather than absolute position in space. They should therefore be measured with respect to the plane of the cladding or in more practical terms a series of adjacent reference lines. There will be an additional deviation in the setting out of the reference lines with respect to the basic reference points for the site.

These tolerances are intended to be applied to curtain walling which is made up of smooth straight and flat elements and therefore the permitted deviations are relatively small. The tolerances for stone cladding should be chosen by the specifier taking account of the importance of the structure, the additional cost associated with tight tolerances and the nature of the stone. For example the tolerances given above may be deemed applicable to a polished granite cladding but would not be appropriate for a stone with a textured surface finish.

Further constraints may be placed on the deviations of the installed stone panels to satisfy other requirements. For example, if the joints between stone panels are to be sealed using say a silicone sealant, the joint width must be maintained within the practical and physical limitations of the joint seal. A joint that is narrower than intended may be difficult to construct and will cause excessive stress in the sealant that may lead to rupture of the joint. Positioning of the panels must take account of the acceptable deviations in joint widths. There may also be a requirement to position panels relative to other components such as windows.

4.5 Accommodation of tolerances

To give satisfactory fit and appearance, the designer must calculate the required adjustment at the fixings and ensure that the fixings are designed to provide this adjustment. If the proposed fixings are unable to provide this level of adjustment, the design must be modified, for example by requiring tighter tolerances or using a different fixing method.

4.5.1 Calculation of required adjustment

The required adjustment is calculated by combining all the deviations which affect the distance between the fixing point on the cladding and the fixing point on the structure. These deviations are generally considered to follow a normal distribution and simple arithmetical addition of the deviations will be unnecessarily conservative as it does not take account of the reduced probability of more than one deviation being at the maximum value at the same time. To maintain the same probability of failure, a number of deviations should be combined by taking the square root of the sum of the squares of the individual deviations. It should be recognised that the statistical distribution of deviations means that there will always be a small chance of the actual deviation being greater than the stated tolerance. Values of tolerances are often quoted with a chance of approximately 1 in 22 of being exceeded. This may give an unacceptable risk of lack of fit and it may therefore be necessary to allow for greater tolerances, which have a lower risk of being exceeded. Increasing the tolerance by a quarter reduces the probability of being exceeded to 1 in 80 and increasing by a half reduces the probability to 1 in 1430.

It should be recognised that the way that dimensions are set out will affect the overall deviation. For example consider a dowel joint between two stone panels. Assuming the dowel is a close fit in the holes in both panels, there will be no scope for adjustment of the panels relative to each other in the direction perpendicular to the plane of the cladding. If the panels are 20+/-3mm thick and the positions of the dowel holes are accurate to within +/-1.5mm, the offset of front faces of adjacent panels will be

$$\text{+/-}\sqrt{\left(1.5^2 + 1.5^2\right)} = 2.1$$

assuming the holes are set out from the front faces of the panels, and

$$\text{+/-}\sqrt{\left(1.5^2 + 1.5^2 + 3^2 + 3^2\right)} = 4.7$$

if the holes are set out from the back faces of the panels.

5.0 Fixing types

5.1 Introduction

This Section deals with the different forms of fixings used for the attachment of natural stone cladding panels. It describes the mechanical performance of the fixings but does not discuss the different materials that may be used to manufacture the fixings.

Guidance on the selection of materials for fixings is available in BS 8298 and Wilson and Harrison 1993. Materials are primarily selected to resist corrosion and materials such as stainless steel are often specified. However, material selection can affect structural performance. Most fixings have to have a degree of flexibility if they are to accommodate movement. Many fixings will displace under load and the deformed shape may alter the load path and load transfer into the stone, Section 6. Substitution of one metal for another may lead to a more flexible fixing that will deflect too far or a more rigid fixing that will not accommodate movement.

Fixings for stone cladding fall into two broad categories. Those that carry in plane loads and provide some restraint to in-plane movement and those that are designed to carry only forces normal to the plane of the cladding. These offer restraint to movement normal to the cladding panel but allow in-plane movement. This later category of fixings have traditionally been called restraint fixings whilst fixings that carry in-plane loads have been termed support fixings. This report continues to use the term support fixing. However, the term non-support fixing is used to describe those fixings that cannot carry in-plane load.

For panels mounted in the vertical plane this is a convenient distinction as support fixings carry gravity loads and possibly wind loading whilst non-support fixings carry only wind load. However for soffit panels and inclined panels all fixings will carry both gravity loads and wind loads. A full structural consideration of the panel, fixing system and degrees of restraint is necessary for any panels that are not in the vertical or horizontal plane. Guidance on structural calculation is given in Stein, 1993.

5.2 Support fixings

These fixings provide restraint and carry loads in the plane of the panel. Some, but not all, also provide restraint and carry loads normal to the wall. The basic forms of fixing are described but each can be detailed in many different ways.

5.2.1 Angles and shelves

These consist of hot rolled or cold-formed angles that are fixed to the floor edge or a beam of the primary structural frame. These fixings are used to support large areas of cladding and are necessarily large to carry the self-weight of a whole storey height of individual stone panels stacked one above the other.

These supports contact cladding panels along the whole of their lower edge and have to be detailed to allow dishing of the panels. It follows that lateral restraint cannot be provided along the whole lower edge of the panel and the lateral restraint is usually provided by separate non-support fixings, Figure 5.1.

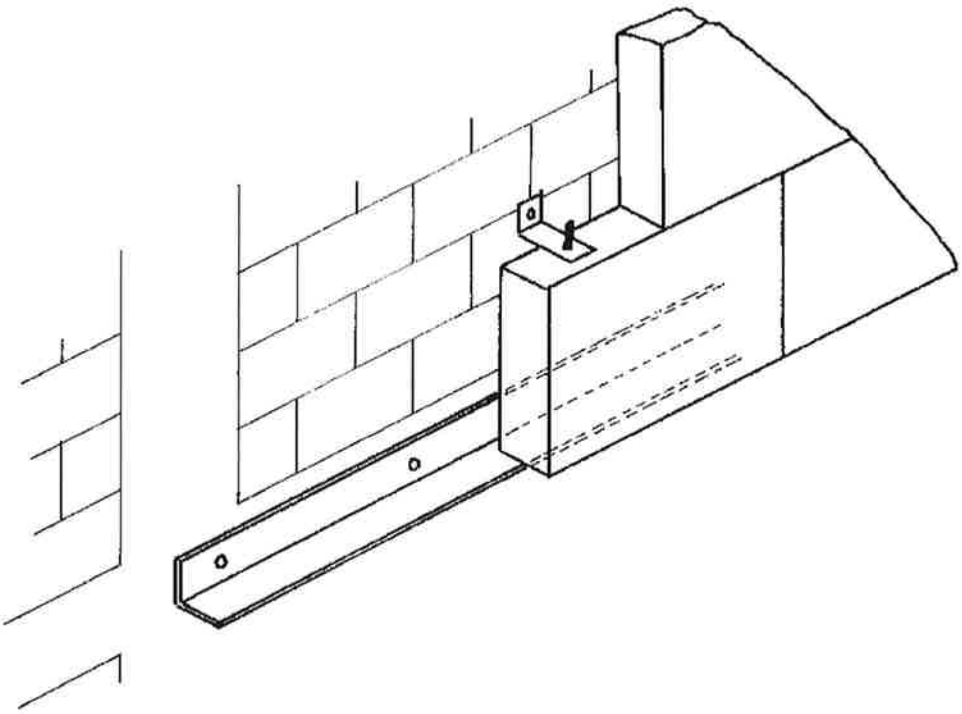

Figure 5.1 Angle support for stone units

5.2.2 Edge of panel support brackets

Discrete brackets may be used in place of a continuous angle. These provide support at discrete points and may be used to provide restraint normal to the plane at the same points without preventing the panels from dishing, Figure 5.2. To provide restraint the brackets have to be engaged in the edge of the panel by means of dowels, kerfs or mortices, see Section 5.4. Support brackets may permit in plane movement if they are able to flex in that direction or sliding may occur between the bracket and the panel. Brackets of this form may be used at each floor level to support the bottom course of a large panel of stone cladding, on which the upper courses are stacked. However support brackets can be provided for each panel in every course of panels. They are then attached either to a structural back wall or a metal carrier frame.

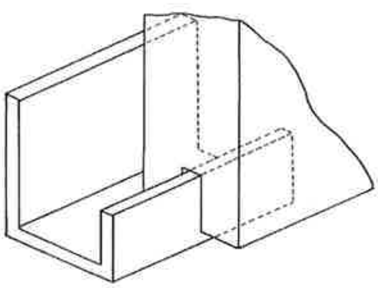

Figure 5.2 Support bracket

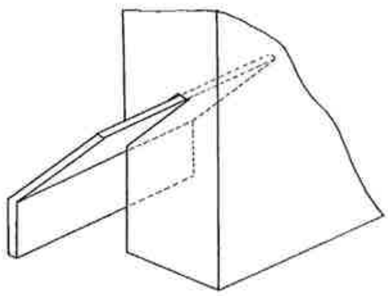

Figure 5.3 Corbel plate

5.2.3 Corbel plates

These fixings comprise angled metal plates that locate into inclined slotted holes cut in the inner face of the cladding panels, Figure 5.3. They provide support and restraint in the same way as support brackets but have the advantage that the point of support is remote from the edge of the panel hence the bending stresses in the stone panel are reduced. Corbel plates provide support and restraint at discrete points and do not prevent dishing of the panels. Corbel fixings may allow in-plane movement by flexing or by sliding of the plate in the slot.

5.2.4 Back face support brackets

Corbel plates have traditionally been used for fixing to the back face of stone panels. Other methods of fastening brackets and plates to the back face of panels are now in use, Figure 5.4. These include the use of undercut anchors, expansion bolts, resin anchors and through bolting, Section 5.4.

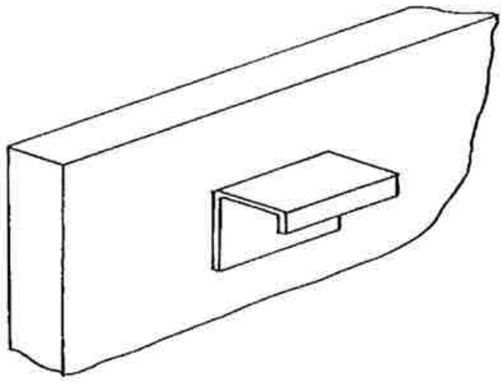

Figure 5.4 Back face support bracket

5.2.5 Channels

There have been examples of stone panels fixed into frame rebates in the same way that glass is installed into a glazing frame. This provides a continuous support to the edges of the panel, which will restrict dishing of the panel. These systems of support are complex in their behaviour and invariably induce stresses in the stone panels. They should only be used after full appraisal of movement, restraint and induced stresses in the panels. This will involve testing.

5.3 Non-support fixings

Fixings provided to carry only the out-of-plane loading will generally be of lighter construction than support fixings. They are articulated or flexible so that they allow in-plane movement to occur and do not attract in-plane loads.

The following forms of non-support fixing are used:

5.3.1 Ties or cramps

Ties or cramps fabricated from metal sheet are frequently used to provide restraint normal to the plane of the cladding and transfer wind loading. They attach to the edge of the panel by means of dowels, kerfs or mortices. They may be deformed as shown in Figure 5.5 to allow complete freedom of movement in the plane of the cladding or they may be as shown in Figure 5.6 in order to resist movement and carry load in one of the principal in-plane directions.

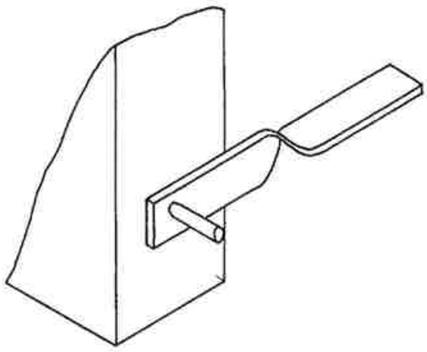

Figure 5.5 Deformed tie

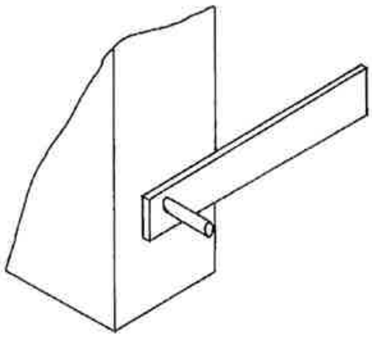

Figure 5.6 Plain tie

5.3.2 Wire ties

Wire ties may be used to provide lateral restraint. They locate into the edge of the stone panels in the same way as dowels and are able to accommodate movement in the plane of the cladding because of their inherent flexibility in this direction. The load carrying capacity of these fixings is limited and they are not normally used to fix large and heavy stone panels.

Wire ties may also be used as a connection between adjacent panels and provide restraint at corners and returns in the wall, Figure 5.7.

5.3.3 Brackets

Corner brackets may be used to connect adjacent panels and provide out-of-plane restraint at corners, Figure 5.8. They are attached to the inner face of the stone panels using either expansion, undercut or resin anchors.

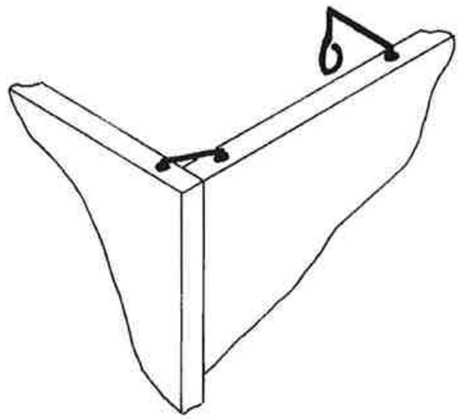

Figure 5.7 Wire ties

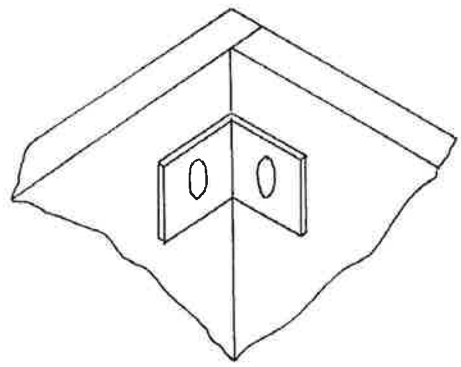

Figure 5.8 Corner bracket

5.4 Connection to stone panels

5.4.1 Kerfs

Kerfs and mortices cut into the edge of stone panels are used to locate kerf plates or cramps that carry normal to plane loads and may also carry in plane loads, Figure 5.9. Kerfs are slots extending the full length of the edge of the stone. Mortices are similar in performance but are cut over a limited edge distance. Kerf or mortice fixings may be used on any edge of the stone but should only be used to provide fixing at discrete points. Continuous fixing or fixing at more than two points along one edge will restrain dishing of the panel. In plane movement of the panel may be accommodated by slippage of the kerf plate or cramp in the kerf or flexure of the tie or cramp.

The strength of a kerf or mortice fixing will vary with the dimensions of the kerf slot or mortice and its distance from the face of the panel. This is subject to the deviations given in Section 4.3. The failure mode and strength of a kerf fixing is highly dependent on the geometry of the kerf plate or cramp. The use of kerfs or mortices with thin stone panels may create insurmountable difficulties of tolerance, machining and achievable strength.

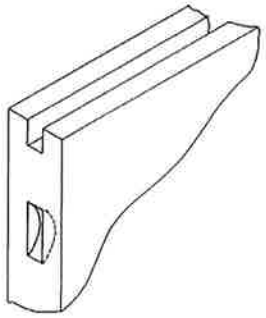

Figure 5.9 Kerf details

5.4.2 Dowels

Dowels are used to locate ties and brackets on the edge of stone panels. The dowels provide a positive location for the tie or bracket and transfer normal to plane and in-plane loads, Figure 5.5. Dowels are inserted into pre drilled holes or, for softer stones, site drilled holes. The strength of the connection depends on the distance from the dowel hole to the face of the stone panel and, as with kerfs and mortices, some difficulties may be experienced with thin stone panels.

5.4.3 Expansion bolts

Brackets and other fixings may be attached to the stone panels by expansion bolts placed in the back face of the panels. Expansion anchors expand in a pre-drilled hole with parallel bore. They can only resist pull out by developing friction or mechanical interlock as a result of radial forces developed at the interface. This leads to a complex stress field in the stone and tensile zones that may lead to fracture. Proper consideration has to be given to the possibility of edge spalling and complex failure patterns in the stone panels.

5.4.4 Undercut anchors

Undercut anchors may be used instead of expansion bolts. They locate in under reamed holes, hence their name. Holes are drilled with parallel bore and then under reamed using a proprietary drilling system. The undercut anchor is then expanded into the undercut chamber. The anchor is expanded only until it contacts the undercut conical face and does not at this stage induce stresses in the stone panel. The undercut anchor resists pull out by the obvious mechanical interlock and fails by pulling a symmetrical cone from the stone.

Undercut anchors in diamond drilled holes are available for stone panel thicknesses down to 20mm. They may also be used with ceramic and glass panels down to 10mm thick.

5.4.5 Resin anchors

Resin anchors are used in a similar way to expansion and undercut anchors to secure brackets and other fixings to the inner face of the stone panels. Resin anchors consist of metal pins secured into pre-drilled holes with a resin adhesive. Concerns have been raised about the long-term strength of resin anchors and their performance in the case of fire. However cranked pins may be used so that there is always a residual mechanical interlock, Figure 5.10.

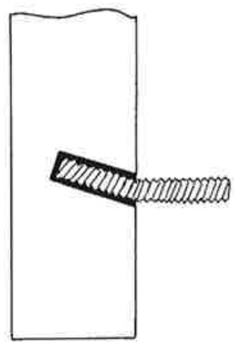

Figure 5.10 Resin anchor detail

5.4.6 Through bolts

Stone panels may be connected by bolting through the stone from the outer face. The bolt head is subsequently concealed by a plug of stone fitted flush in the outer face of the panel. These fixings may be used to fix brackets onto the inner face of the stone panel. Traditionally they have been used for fixing soffit units and making connections at corners and returns.

6.0 Fixing performance

6.1 Introduction

When assessing the integrity of fixings for thin stone cladding it is important to fully understand the behaviour of the stone panels and fixings as constructed on the face of a building. This may differ greatly from the idealised performance seen in some tests and from the behaviour assumed in some calculations.

The behaviour of the fixings has to be considered as Part of a system; neither integrity of the stone panels nor that of the fixings should be considered separately. The support frame or back wall, fixings, method of attachment to the stone panels and the stone panels themselves should be considered as a whole. Only then is it possible to separate out the behaviour of individual components.

Loss of integrity of the system may occur as a result of:

- Over stressing and fracture of the stone panels;
- Over stressing or excessive deformation of the fixings;
- Failure of the support wall or frame, or the connections to it.

6.2 Effect of induced stresses

Stone panels, fixings and fixing systems have to provide the necessary movement accommodation but also provide the required restraint and carry the applied loads. The integrity of the panels and the fixings under load will depend on both the externally applied loads and the loads induced as a result of restrained movement, Section 2.8. The stress fields set up in the stone panels result from both forms of load and fracture will be determined by the combined effects.

It follows that the integrity of the fixing system cannot be predicted unless the induced stresses are known. It is only possible to test the integrity of the fixings if the induced stresses that occur in the finished construction are also present in any specimens used for strength testing.

When evaluating integrity it is necessary to consider the worst combinations of load. This process is complicated if induced loads are to be considered during the test. In order to test all combinations it would be necessary to test for load carrying capacity at both extremes of temperature and with the stone panels both dry and saturated. The numerous combinations of load and environmental conditions make it impractical to take account of induced stresses in this way.

It is good practice to detail stone fixing systems so that movement can take place freely and no stresses are induced in the stone panels. This maxim of 'keeping things simple' aids the understanding of load transfer and of the stresses in the stone. Only if this is done and no stresses are induced in the stone panels is it possible to assess the panels and fixings for integrity with confidence.

6.3 Loads and load paths

The stresses in a stone panel will depend on the magnitude of the applied loads and their positions. The positions of the loads are clearly affected by the deviations induced during manufacture. The magnitude of the loads is also affected by the induced dimensional deviations. The positions and magnitude of the loads may also change as a result of deflections of the stone panel or fixing. It is important that the load and load paths are known for the purpose of stress calculation. Any integrity tests performed should correctly model the loads and load paths that occur in the completed wall.

6.2.1 Effect of induced deviations

When calculating forces and stresses or testing for integrity under self-weight loading, it is desirable to use stone panels of the greatest expected dimension in order to maximise the self-weight. It may however be desirable at the same time to use the thinnest stone panel expected in order to maximise the flexural stresses arising from wind loading.

The strength of many attachments to natural stone depends on the geometry of the stone. Embedment depths of anchors affect the pull out strength and the strength of kerf and mortice fixings will always depend on the distance of the kerf from the face of the panel. This is best understood in each case by considering the performance and failure modes of different fixings, Section 6.4.

The self-weight load of the stone is offset from the support frame or back wall and this leads to bending of the fixings. The exact off set from the wall will depend on the position of the structural frame and the finished plane of the stone cladding. The maximum off set that can be expected in practice will lead to the greatest bending moments in the fixing and greatest loads in the connection to the support frame or back wall. The smallest offset that can be expected will give the least movement accommodation at the fixing.

6.3.2 Effect of movement

Load is transferred from the stone panels to the fixings through the point of contact of the fixing on the stone. Under the influence of self-weight or wind load it is possible that a fixing will deflect so that a different point of contact is made and a different load path is set up. This will change the stresses in the stone panel and the fixing and invalidate any analysis that does not take account of the new load path. This is best understood by considering the alternative load paths and failure modes that can be set up for each type of fixing. Fixing systems for stone panels should be designed to avoid the setting up of alternative load paths. This simplifies stress calculations and the maxim 'keep it simple' again applies.

If load paths are clearly and uniquely determined by the detailed design of the panels and support systems then it is possible to calculate loads and conduct tests on components of the fixing system, for instance pull out tests on resin anchors or tests of individual kerf fixings. If load paths are not clearly defined then whole panels of cladding may have to be tested to create the movement that sets up the correct load paths, loads and stresses.

6.4 Failure modes - failure of the stone

6.4.1 Kerfs and mortices

Failure of the stone at a kerf or mortice detail will occur as a fracture surface and pulling away of Part of the stone as shown in Figure 6.1. The strength of the fixing depends on the strength of the stone and the area of the failure surface. A deeper kerf, thicker shoulder or greater width of contact on to the kerf plate or cramp will all increase the load carrying capacity of these details. Use of a kerf plate or cramp too close to a corner may mean that the failure surface is reduced in area and that the strength of the fixing is reduced.

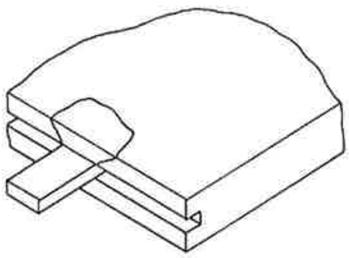

Figure 6.1 Stone failure at a kerf

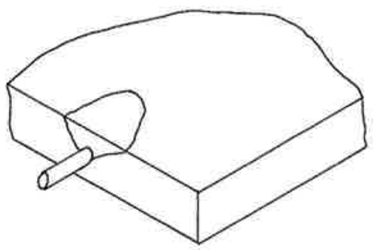

Figure 6.2 Stone failure at a dowel

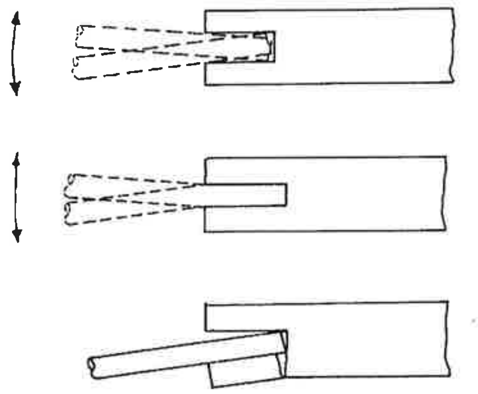

Figure 6.3 Effect of rotation at the edge of a unit

Rotation of the edge of the stone panel or rotation of the fixing both have the potential to change the load path through the kerf or mortice. Figure 6.3 shows a kerf plate or cramp that has rotated and come into contact with the shoulder of the kerf. Under these circumstances the shoulder of the kerf breaks away at a much-reduced load. It is important when detailing fixings into kerfs and mortices that the correct failure mode is identified to avoid over estimating the strength of the fixing. Similarly it is important to achieve the correct failure mode under test.

The load carrying capacity of a kerf or mortice may vary as a result of thermal or moisture caused dishing of the panel. Figure 6.4 shows two details for a kerf plate or cramp. In the first case the load paths are clearly identified and are independent of small rotations of the panel or fixing as indicated. The second detail has no clearly defined contact points to transfer either in-plane or out-of-plane loads. The load paths are totally dependent on the relative rotations of the panel and the cramp. If fixing strength depends on deflection and rotation of components then the worst assumptions have to be made in any calculations. When testing it is essential to achieve the correct failure mode.

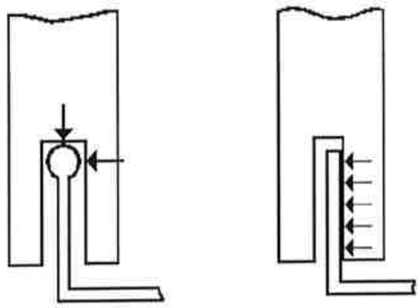

Figure 6.4 Load paths at a kerf

6.4.2 Dowels and wire ties

Failure of dowels and wire ties is very similar to the failure of kerfs and mortices. Figure 6.2 shows a failure of a dowel in the edge of a stone panel. The load capacity of this detail is dependent on the strength of the stone, embedment depth of the dowel and the distance from the dowel to the face of the stone panel. As with cramps and mortices, rotation of the panel edge or the fixing may lead to a different form of failure that is more localised, Figure 6.3, and to a considerably reduced load carrying capacity.

With softer stones, holes are often drilled on site to accommodate dowels. These holes are positioned to give the correct face alignment of the stone and may be positioned closer to one face than specified. The strength of the fixing is then reduced below the design capacity. Factory drilled holes are usually positioned at a constant distance from the outer face of the stone panels so that the outer faces are correctly aligned when installed. The distance from the dowel hole to the inner face of the panel is then dependent on the overall thickness of the panel and the achievable deviation will be greater than that given for the position of a hole in Table 4.1. The design load capacity of the fixing is governed by the minimum anticipated distance from the dowel hole to either face of the stone panel.

Site drilled dowel holes may not be exactly aligned with the plane of the stone or of the correct diameter. Attempts to force fit dowels into holes will induce stresses in the stone around the dowel. These will lead to a reduction in the load carrying capacity of the fixing.

6.4.3 Bolts and anchors into stone

When bolts and anchors into stone fail as a result of fracture of the stone, failure takes the form of a cone of stone pulled (or pushed) from the face of the panel. Figure 6.5 shows a Section through a failure cone pulled by a resin stud anchor. The load capacity of the fixing is determined by the strength of the stone and the size of the cone that is pulled from the stone. Greater embedment depth of the anchor will give a greater load capacity but use of the anchors too close to the edge of the panel will lead to a reduction in load capacity, Figure 6.6.

The load capacity of through bolts and undercut anchors will depend on the diameter of the washer or the diameter of the bottom of the undercut anchor. Omission or substitution of washers for through bolts may reduce the load capacity of these anchors.

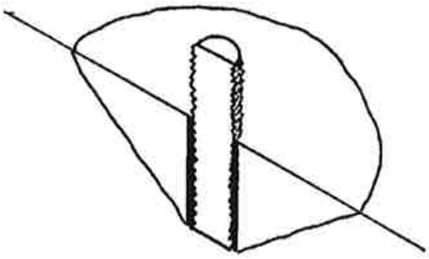

Figure 6.5 Failure cone at a stud or anchor

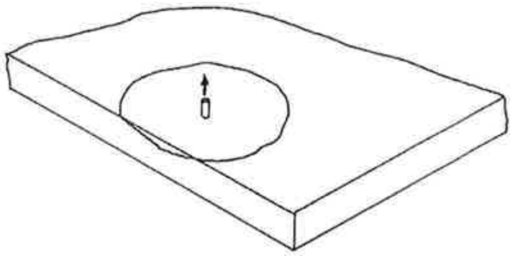

Figure 6.6.. Edge effect on anchor failure

Fixing bolts and anchors are normally designed to resist pull-out loads however wind loading on panels is reversible and loads on bolts and anchors may be in either axial direction. Either panels and anchors should be designed to resist compressive loads or an alternative load path should be set up as shown in Figure 6.8. In this case the bracket transfers compressive loads direct to the inner face of the panel and the anchor carries only tensile loads. Through bolts will have no strength in compression and should only be used with brackets as shown in Figure 6.8.

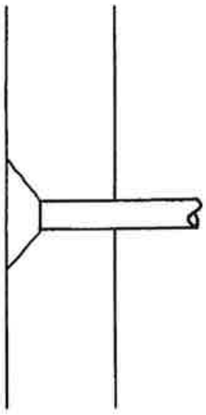

Figure 6.7 Punching failure due to compression load on anchor

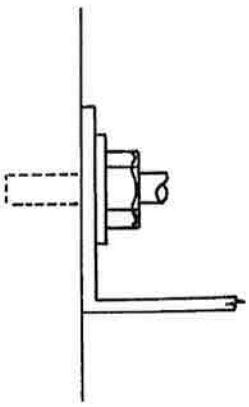

Figure 6.8 Compression transfer through bracket

Rotation of the bolts as a result of bending of the fixings or flexure of the panel can lead to a reduced load capacity of the fixing. For thin stone panels and structurally assisted stone panels consideration should be given to testing complete panels and not just single fixings.

6.5 Failure modes - failure of the fixing

6.5.1 Angles and shelves

Failure of an angle or shelf support may be manifest as bending of the support as shown in Figure 6.9. This may lead to gross deflections and unacceptable movement of the stone panel on the facade. However failure may occur at lower deflection of the bracket. For instance the bracket may deflect so that the inner edge of the stone panel bears on the bracket and local spalling occurs. Another failure mode occurs when the support deflects and contacts the top of the panel below. This transfers load to supports further down the wall, which may in turn fail.

Angles may also deflect as shown in Figure 6.10 where the spacing between fixings to the primary structure is too great. The combined effects of flange bending, torsion and overall bending between supports should not lead to unacceptably large deflections that change the load paths in the system and stress distribution in the stone.

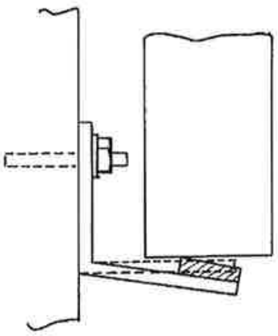

Figure 6.9 Deflection of bracket

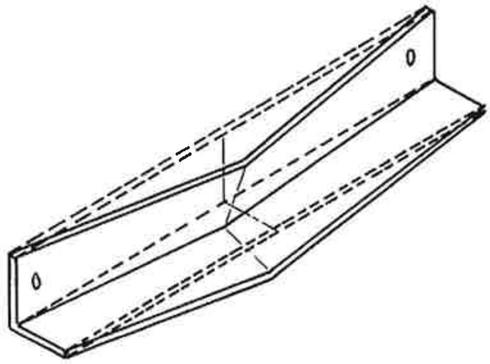

Figure 6.10 Deflection of angle support

6.5.2 Brackets and ties

Brackets, whether used as edge supports or attached to the inner face of the stone panels, may deflect under load and cause failure of the system. This may occur because the bracket induces stresses in the stone or increases the load in the anchors.

Brackets attached to the inner face of a stone panel will attempt to move with the panel as it rotates, Figure 6.11. If the bracket is free to rotate relative to the wall neither the bracket nor its attachment to the panel will experience any additional loads. Otherwise flexure of the bracket will occur or, if there is flexible packing between the bracket and the stone, the bracket will rotate relative to the stone panel. In the former case failure may occur as a result of fatigue of the bracket. In the latter case flexure of the anchor may take place and premature failure occur as a result of fatigue, Section 2.3.2. The correct failure modes have to be identified before testing can be undertaken.

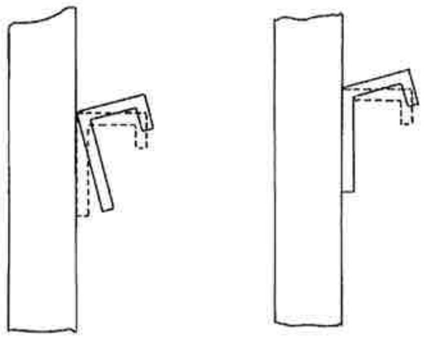

Figure 6.11 Deflection of back support brackets

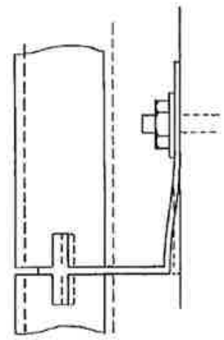

Figure 6.12 Movement of a tie

Brackets and ties may deflect under out-of-plane loading as shown in Figure 6.12. This is a common failure mode with ties, particularly thin metal cramps. These are often supplied with the intention that they are bent on site to set the stone panels the correct distance from the support structure. They are made flexible to allow site adjustment but depending on the position of the anchor to the back wall will allow different amounts of movement and excessive movement may occur. Deflections of this form may be classed as failure or they may be the cause of some other failure mode, for instance they may allow a greater offset of the dead load from the support wall that leads to overload of the supports.

6.5.3 Anchors

Anchors and bolts frequently fail by pulling out of the stone panel or other substrate into which they are fixed. However, movement of the stone panels, both rotation and in-plane movement can lead to flexure of bolts and studs. This will occur where a nut is used to stand a panel away from a bracket but also occurs inadvertently where shims are used to position a panel in the correct position. The failure mode of an anchor is dependent on the extent of any shimming and the method of attachment of the anchor to a bracket or other fixing point. In general anchors should be used to hold components in close contact so that no flexure occurs or they should be Part of a system that is articulated or flexible so that no flexure of the anchor takes place.

7.0 STRUCTURAL DESIGN

7.1 Introduction

The structural design of fixings for a thin stone cladding system has to take account of the required performance of the system and of the fixings. It should assess all possible failure modes and show that in every case the required margin exists between the expected load capacity and the expected loads.

The design process should evaluate the four aspects of the fixing:

- Attachment to the support wall or frame
- Attachment to the stone panel
- Integrity of the fixing itself
- Integrity of the panel

The ability of the support wall or frame to resist the loads imposed by the cladding must also be checked but is outside the scope of this document.

As with most other structures the purpose of structural design is to ensure that the cladding and fixings are strong enough to withstand the loads to which they are subjected safely and without impairing their performance.

To satisfy safety requirements, the structure should be able to carry the design loads without failing in an unsafe manner. This is commonly referred to as the ultimate limit state. In practice this is achieved by limiting the stresses on the components under a design load which is unlikely to be exceeded in service.

Although the safety requirement ensures that the structure can withstand the design loads, the overall performance may be impaired. For example excessive deflection may impair weathertightness. To ensure serviceability, limits are placed on the deflection under the design load. The check on serviceability or deflection is normally carried out at a lower design load than used for the safety check.

When designing fixings the same principles apply however it may be necessary to consider additional situations. For example a support bracket could have sufficient strength to satisfy the safety requirements and the deflection could be acceptable at the serviceability design loads but the greater deflection at the ultimate design loads could result in the bracket failing to support the cladding securely.

In structural design it is normal to apply a factor of safety to ensure a margin between the expected load capacity and the expected loads. This margin allows for the possibility of weaknesses in the material or loads being greater than expected. In some cases a single global factor may be used but it is more common to apply partial factors to both the loads and the material strengths. Different factors may be used for dead and live loads and when considering safety and serviceability. Different factors may also be used for different materials to take account of differences in performance. For example lower factors may be used for predictable materials which fail in a ductile manner than unpredictable materials which fail in a brittle manner.

For many construction materials including steel and aluminium there are established design procedures given in British Standards which give safety factors. However for stone there are no clearly defined standards as discussed in CWCT, 1997. BS 8298 gives factors of safety for stainless steel components of fixings but does not give safety factors for stone and makes reference to guidance from the Copper Development Association for copper and copper alloy fixings. The Building Regulations Approved Document A gives guidance on the required strength of cladding fixings, Section 7.3.1.

The choice of factors of safety for a given situation is a matter of engineering judgement taking account of the relevant guidance documents and the details of the fixing under consideration. Typically the global factor of safety for steel components at the ultimate limit state will be 1.4 or 1.5 whereas for stone panels and fixings a factor of 3 or 4 is more likely. When comparing factors of safety it is important to ensure that the comparison is being made on the same basis. For example an apparently higher factor of safety may give a lower margin of safety if it is applied to the average material strength rather than a lower bound to the test results. Partial factors of safety for loads from one source should not be used in conjunction with factors for materials from another as different design methods may make different assumptions about what each factor is expected to allow for.

7.2 Design methods

Some aspects of performance may be assessed by calculation, for instance the deflection of an angle support, the tensile stress in a hanger, or the buckling load in a tie. However many aspects of fixing design are difficult to model in calculations and must be assessed by testing. It is unlikely that all aspects of the fixing can be proven in a single test and a programme of tests or combination of calculations and tests has to be developed.

Recourse may also be made to previous use and testing of a component. The use of anchors in concrete and blockwork is well documented and design charts exist for many products. In this case it may not be necessary to test to prove the design. However, site installed anchors should still be tested after installation to check workmanship and in situ strength.

7.2.1 Testing programmes

It is possible to test large full scale cladding specimens and this is common for bespoke curtain walling. The cost of constructing and testing large specimens is justified because it is the only way to test for water penetration resistance and integrity under wind loading. However many of the structural aspects of a rainscreen may be proven by the testing of individual panels and components, CWCT, 1998.

When testing a thin stone cladding system it will be necessary to test at the design load. As the factor of safety on loading is normally different for stone, metal and concrete components there is no single proof load at which tests on complete cladding systems, or even individual panels, can be carried out.

If a large specimen comprising one or more panels of cladding is tested at a load in excess of 1.5 times the nominal load, the load is likely to produce an expected failure in the support frame, or in the fixings remote from the stone, before the anchors and stone details experience the required proof load. Similarly, if a single stone panel is fixed as intended on site and then loaded, failure will occur in the fixings remote from the stone before the panel and stone details experience the required proof load. For these reasons it is normally necessary to conduct separate tests for:

- Attachment of the fixing to the support frame or wall
- Attachment to the stone (anchors, kerfs, dowels, etc.)
- Fixings remote from the stone
- Flexure of the stone panels

Natural stone is a variable material and when undertaking tests it is necessary to perform enough tests to give a statistically valid answer with the required degree of confidence, CWCT, 1997. Typically every test is repeated ten times, more for stone with variable properties, but this is not feasible when testing a large-scale cladding panel. A single result from a large scale test may be misleading and of less value than a series of repeated tests on a single component.

Large test panels comprising a number of stone cladding panels and fixings will be erected to the tolerances required of the test rig and the induced deviations of the stone will be 'run of the mill'. The tests will not then be representative of the worst possible design condition on the wall. When testing individual components it is possible to set fixings at their maximum extension, position dowels and kerf plates close to the face of the stone and use the thinnest stone panels. Above all it is possible to observe the failure clearly and record the dimensions of the test specimen. With this information it may be possible at a later date to calculate stresses and engineer any required design changes.

If a large specimen has been constructed for the purpose of air and water penetration testing then it may be used to prove some aspects of the structural performance of a thin stone cladding, for instance thermal movement or movement of a structural support frame. The testing regime for a curtain wall may call for an ultimate load test, CWCT 1996. In this test the wind loading on the specimen is increased until a failure occurs. The test is intended to show the ultimate failure mode and may show whether failure occurs in stone panels, fixings or anchors. Of course all individual components show a range of strengths and it may be difficult to rely on the results or even interpret them.

Tests of individual components may not be sufficient when testing some thin stone cladding systems. If the stone panels undergo large flexural movement during wind loading or thermal and moisture movement they may impose a bending load on the fixings and the anchors, kerfs or dowels in the stone panels. This is not generally regarded as a problem for stone panels thicker than 20mm; for thinner panels such as glass, ceramic, and structurally assisted stone veneers it should be considered. This aspect of behaviour can only be tested by loading a complete panel or panels and the corresponding fixing system.

7.2.2 Calculation

Where the stress distribution in a component can be established by calculation, the design of the component may be based on calculation of the load bearing capacity of the component. BS 8298 gives permissible tensile and shear stresses for stainless steel and SCI, 1993 gives further guidance on the design of stainless steel fixings. For more complex components in steel and aluminium guidance is given in BS 5950 and BS 8118 respectively.

7.3 Attachment to the support wall or frame

7.3.1 Concrete and masonry walls

The design of fixings in concrete and masonry must be based on tests. Site specific tests may be carried out in some cases but it will usually be possible to use manufacturers design charts based on previous standard tests.

Approved Document A of the Building Regulations 1991 gives guidance on the design of fixings for cladding. Current requirements (given in the 1992 edition of Approved Document A Section 2) are as follows:

- For expanding bolt type fixings the safe working shear and tensile loads should not exceed the lower of the following values
  a) The mean shear or tensile failure test load less 3 times the standard deviation derived from the tests x 1/3.
  b) The mean of the loads which cause a displacement of 0.1mm under direct tension and 1.0mm under direct shear.

- For resin bonded fixings, the safe working shear and tensile loads should not exceed the mean shear or tensile failure test load less 3 times the standard deviation derived from tests x 1/3.

Testing of anchors in concrete and masonry is described in BS 5080: Part 1 and BS 5080: Part 2, which deal with loading in tension and loading in shear respectively. The standards make provision for testing of anchors into both concrete and brick or block masonry. The method of test may be applied to:

- Expanding and undercut anchors
- Bonded fixings
- Cast in fixings
- Channel inserts

BS 5080 tests single anchors as a means of proving anchor performance. Groups of anchors may perform differently according to their spacing. Manufacturers guidance should be followed on the use and spacing of anchors. If there is any doubt about the strength of a group of anchors in a fixing then it may be appropriate to conduct tests on the fixing and anchor group as a whole.

BS 5080 is primarily intended for the proof testing of anchors. It makes provision for testing of anchors in a standard specimen of base material so that results can be compared. Where it is required to test the performance of anchors in the actual material of the wall BS 5080 gives no guidance on the number of tests to be performed. CWCT, 1997, gives guidance on the statistical basis for testing of natural stone materials with variable properties.

The BS 5080 tests are for static loading which is applicable to self-weight loading and static equivalent wind loading. In order to test for reversals of loading or cyclic loading it would be possible to modify the test procedure appropriately. A testing regime for cyclic wind loading has been developed by the BRE, Section 2.3.2. In practice it may be acceptable to test to 10000 cycles of full static wind loading and this may be a simpler test to conduct. Whichever test regime is adopted the test cycles should be at a low frequency, typically 1Hz.

The tests described in BS 5080 do not subject the anchor to combined tension and shear. In practice the anchor load capacity is measured separately in tension and shear to establish the proof loads,

F_t = failure load in tension

F_s = failure load in shear

The combined shear and tension loads on the anchor then have to meet the conditions:

$$F \times f.o.s. / F_t < 1$$

$$V \times f.o.s. / F_s < 1$$

$$F \times f.o.s. / F_t + V \times f.o.s. / F_s < 1.2$$

Where:

F = axial tensile load

V = shear load

f.o.s. = required factor of safety

The guidance in the Building Regulations is likely to be revised shortly to take account of the introduction of European Technical Approvals (ETAs) for construction fixings. The system of ETAs has been developed by the European Commission to assess products which are not covered by European Standards. Products are awarded an ETA after a rigorous assessment programme in accordance with procedures detailed in the relevant Guideline for European Technical Approval (ETAG). Products which have an ETA may be CE marked. CE marking is not currently mandatory in the UK but the use of products with an ETA may be accepted as a means of complying with the Building Regulations. Currently ETAGs only cover the use of undercut anchors, torque controlled expansion anchors and deformation controlled anchors for use in concrete.

The most significant effect of the ETAG is the treatment of fixings in cracked concrete. The current Building Regulations state that the effect of cracking of concrete on the strength of fixings should be taken into account but gives no guidance on how this should be achieved. The ETAG on the other hand gives a stringent test regime to evaluate the effect of cracking and requires that the fixing design assumes that the concrete is cracked unless proved otherwise. It is anticipated that revisions to the Building Regulations will give guidance on when concrete can be assumed to be non-cracked.

7.3.2 Support frames

Attachment to metal support frames falls into two broad categories:

Simple attachment points may be fabricated as Part of the steel or aluminium fabricated frame. These are normally straightforward brackets or plates and are amenable to stress calculations for strength provided there are no unknown induced loads or movements.

More complex attachments may be encountered as parts of proprietary systems. In these, components are punched, pressed and folded to complex shapes and stress calculation is not possible. However, these components are usually tested at the time the proprietary system is developed.

Whilst the strength of attachment points can be considered in this isolated way it may still be necessary to test them when integrated into the whole construction. This is essential when the movement of the stone and any forces induced in the fixings are unknown. It is also required if deflection of the attachment points is likely to affect the performance of the fixings or stone panels.

Testing of attachments to metal frames as Part of the complete system is relatively straightforward. The proof load for this component will be based on a lower factor of safety than most other parts of the system. The attachment points will normally be one of the first components to reach their proof load in any test.

7.4 Attachment to the stone panel

The design of the attachment to the stone panels will follow the same principles as attachments to a concrete or masonry wall. Failure of the attachment to the stone panel is manifest by fracture of the stone, Section 6. Accordingly tests are conducted at proof loads that incorporate a factor of safety of 3 or 4, and some times higher. Some forms of fixing are able to carry this high load, for instance expansion or undercut anchors. Other forms of fixing such as brackets or ties located with dowels may fail before the proof load on the stone is attained or may have so deformed as to have changed the load path and the loading on the stone panel. For this reason it may be appropriate to fracture the stone using a stiffer and stronger connection.

Failure of the attachment results from fracture of the stone and the results from a series of tests may show considerable variation. Some dense uniform stones will give consistent failure strengths whilst other stones will be highly variable in strength and fracture at vastly different attachment loads. The number of tests conducted should be decided in the same way as for other stone strength tests, CWCT, 1997. The greatest variability will occur for non-uniform stones, particularly those containing inclusions.

The strength of the attachment points will often depend on the dimensions of the stone panel tested as described below. Test specimens will be subject to induced deviations but an attempt should be made to select specimens that are dimensioned so as to exhibit the least strength. Dimensions of all test specimens should be checked and recorded.

Strength of the attachment may be affected by displacement and rotation of the fixings and by rotation of the panel as a result of flexure or dishing. If this is the case then it is not normally possible to test the attachment to the stone panel in isolation. Tests of at least a whole panel complete with all attachment details have to be conducted in this case. For building stones greater than 20mm thick it is normal practice to ignore deflection of the stone panel and conducted tests on individual fixing points.

7.4.1 Kerfs

When testing the strength of a kerf detail it is important to use specimens with 'shoulders' of minimum thickness expected in the finished cladding. This will give the lowest strengths under test. The depth of the kerf and more importantly the position of the kerf plate or cramp in the kerf will also affect the strength under test. The minimum anticipated penetration into the kerf should be used when setting up the test specimen.

If the kerf plate or cramp is sufficiently strong then the kerf may be tested using the fixing components to be used on the finished construction. If the kerf plate or cramp is going to fail or deflect excessively before the proof load on the stone is reached then it is necessary to substitute a stronger fixing solely for the purpose of testing. This should be of similar geometry to the original at its interface with the stone to create the same loads and load paths.

Where a single kerf plate secures two adjacent panels it is usually necessary to test in this configuration to avoid rotation of the kerf plate under asymmetric loading.

Wind loading on stone cladding may be both positive and negative. If there is a significant difference in magnitude between the two loadings, or the kerf detail is strongly asymmetrical then the kerf should be tested in the weakest direction and for the greatest loading. It may be necessary to test for both positive and negative load.

7.4.2 Dowels and wires

Dowels and wire ties lead to similar failures as those in kerfs, Section 6.4.2. The testing regime should be considered in the same way as for testing of kerfs, with one exception. Dowels and wire ties in particular are more flexible and their deflection under load may change the load paths and the loading regime in the stone. It is difficult to substitute a stronger or stiffer component and still ensure the correct load transfer as for kerfs and tests have to be conducted using the true components. Any unexpected or premature failure should be investigated as deflection of the dowel or tie at exceptionally high load may have invalidated the test.

7.4.3 Face anchors

Where expansion anchors, resin anchors or undercut anchors are used to connect to the stone panels they may be tested in accordance with BS 5080 Parts 1 and 2 for pull out strength. The considerations set out in Section 7.3.1 apply. However the tests will be into natural stone and the number of tests must be decided accordingly.

It should be remembered that with thin stone panels failure may also occur as a result of punching through of the anchor. If the fixing detail requires the anchor to transmit compressive as well as tensile forces, Section 6.4.3, then the anchor should be tested under both positive and negative load.

Some undercut anchors are used with very thin stone panels and other thin materials. In these uses they may be subject to bending as well as tension or compression. In this case it is necessary to test the panel and all its fixings as a whole.

7.4.4 Through bolts

The testing of through bolts requires the same considerations as the testing of face anchors. They behave in the same way and differ only in their method of construction.

7.4.5 Frame rebates

When a frame rebate is used to attach a thin stone panel it invariably induces stresses in the panel, Section 3. These details have to be considered with great care and should always be the subject of tests. Testing may only be conducted on specimens comprising the whole stone panel and its fixing system. Tests should include thermal and wetting tests to reproduce any dishing that may occur in the panel.

7.5 Fixings

7.5.1 Support angles

It is seldom necessary to test the load capacity and deflection of a support angle as these can normally be calculated. If it is necessary to test then it will normally be easier, and more informative, to apply the loads with loading jacks. It is of course important to apply the loads at the greatest anticipated offset from the support wall or frame. If the loads are applied by loading jacks then a proof load can be applied to determine the factor of safety.

7.5.2 Support brackets

Brackets used to support the stone panels at one edge may be tested to determine their strength and deflection under load. It may be necessary to test to ensure that excessive deflections do not occur and that the bracket does not contact the stone panel in some unintended way. This may be done using an assembly of a stone panel with all brackets, anchors, dowels, cramps and so forth, Sections 7.6 and 7.7.

If the only concern about the brackets is one of strength then it will be possible to conduct the tests by applying loads to the brackets through loading jacks.

The deflection of any brackets will depend on their fixity to the supporting frame or structural wall. If brackets are attached to a supporting metal frame or support rails then it will normally be necessary to test a complete frame with all brackets and loads present to determine the deflections of the brackets.

7.5.3 Ties and cramps

The tensile strength of a straight tie can be calculated but it may be necessary to test to check for prying of the tie from the back wall, Figure 6.12, or buckling of the tie. Ties and cramps can normally be tested as Part of a complete assembly (panel, tie, anchor, kerf, dowel etc.) as they should reach their proof load before the stone panel or anchors fail.

When ties and cramps are tested alone any premature failure or excessive deflection should be investigated. It may be that the observed failure mode is not the one that occurs when the stone panels and support frame or wall are present.

7.5.4 Face brackets

Face brackets attached to the inner face of the stone panel will want to rotate with the panel as it dishes as a result of temperature and moisture change and flexes as a result of wind loading. For panels thicker than 20mm this is often judged not to have a significant effect on the panels and fixings. For thinner panels it is necessary to test a stone panel complete with all brackets and fixings, sections 7.6 and 7.7 and to investigate the effects of thermal and moisture change along with wind load, dead load and other applied loads.

For thin panels it is advisable to further test the assembly by subjecting it to cyclic wind loading, Section 2.3.2, or an equivalent load such as 10000 cycles of full wind load, Section 7.3.1.

7.5.5 Support rails

For support rails attached to brick or block masonry back walls the principal concern is local deformation of the support rails that may cause excessive deformation of the fixing brackets or ties. Support rails may be tested separately from the stone panels by applying dead and wind loads through loading jacks attached to the support points, brackets or ties.

7.6 Stone panels

Stone panels may be tested for flexure but it is more normal to carry out bending tests on specimens of stone and calculate the bending strength of the panel, CWCT 1997. This avoids the cost of preparing a statistically representative number of panels for test. Test specimens of standard dimensions may be used but it is sometimes preferable to use test specimens of the same thickness as the proposed cladding. Samples should also have the same finish, particularly where the finishing process may affect the strength of the material near the surface. A flexure test on a complete panel and fixings is difficult to perform, as the fixings will frequently fail before the proof load on the panel is reached.

Individual stone panels complete with fixings may be tested to show that the fixings are sufficiently flexible to not induce stresses in the stone and that the fixings do not deflect so far that they contact the stone panel in some unanticipated way. When testing panels and fixings in this way it is important that fixings are not loaded asymmetrically if that is inappropriate and it may be necessary to test a larger specimen that contains several stone panels, Section 7.7. For instance panels fixed by rear face brackets may always be tested individually whilst panels attached by dowels cannot normally be tested separately.

Individual stone panels may be tested to determine what movement occurs when the temperature or moisture changes. Panels complete with fixings may be subjected to a thermal cycling test, CWCT 1996, or a test that alternately wets and dries the stone. Advice on the testing of cladding materials subject to thermal and moisture change is given in BBA :MOAT 22. It may be necessary to test a larger specimen containing more than one stone panel if asymmetric loading on shared fixings is likely to affect the result. Panels that are attached by back face brackets or a rebated frame and do not have fixings common to adjacent panels can always be tested as individual panels.

Panels may have to be tested using positive or negative wind loads or both. Negative wind loads are generally applied using an air bag inserted behind the stone panel. A method of test using an air bag is given in ASTM C1201 : 1991. The loading regimes given in CWCT 1996 for curtain walling may be applied to such an air bag test. At least one laboratory is able to apply negative wind loads using suction cups to pull on the stone panel. These have to be articulated so that they rotate with the panel and do not induce stresses in it. Positive loads may be applied by surrounding the panel and back wall with polythene or similar sheet and lowering the pressure in the cavity, Figure 7.3. Alternatively it may be possible to apply the loads by laying the panel and support system down, outer face up, and using sand-bags to apply the required loadings. The loading regimes to be used are the same as for negative loading.

7.7 Testing of large specimens

Large specimens should ideally represent the most critical conditions of the wall as actually constructed. This is unlikely to be the case, Section 7.2 and load tests are generally restricted to testing of particular aspects of individual components.

Large facade specimens may be constructed to test for air and water tightness in accordance with 'Standard and guide to good practice for curtain walling', CWCT 1996. These specimens, which will be two or more storeys in height and typically one structural bay wide, may be used for the testing of natural stone fixings. However it is unlikely that the proof loads on the stone panels will be achieved before some other aspect of the fixing or wall fails. It is also unlikely that the test chamber used to load the specimen will be capable of withstanding the design wind load multiplied by a factor of four.

Large specimens offer the opportunity to load a single stone panel, or several stone panels within the specimen, using an air bag as described in Section 7.6. The adjacent panels will provide the correct edge restraint to the panel and fixings under test. The disadvantage of this form of testing is that a single result is obtained to show the performance of a sometimes highly variable material.

Large specimens may be subjected to the cyclic temperature test described in CWCT, 1996. This is the best way to check the effects of thermal movement as all panels in the specimen are heated and cooled. The test will only show the amount of movement and dishing of panels that occurs. It is not a satisfactory test of integrity as the induced stresses will be the actual stresses and no allowance will have been made for the load factor to be applied to stresses in the stone. With a safety factor of four on the stone stresses, failure is not expected to occur at the operating temperature. Moisture movement may also be assessed using a large specimen but again the induced stresses will be actual stresses and not factored stresses.

The one case for which testing of large specimens is useful if not essential is when the stone is used as a load bearing element that stiffens a support frame through plate action of the stone panels. For constructions of this type the performance of the stone is critical and a comprehensive stone testing programme should be completed. Full scale tests of complete frame assemblies and stone panels can then be undertaken to ascertain the failure modes and overall factor of safety. An example of the structural use of stone is the use of pre-stressed Ketton stone for the Queen's Building at Emmanuel College Cambridge, Structural Engineer, 1999.

GLOSSARY

Anchor	Device used to attach the metal fixing to a natural stone panel or a back wall of concrete, brick or block masonry. Typically an expansion bolt, resin anchor or undercut anchor.
Anchorage	The point of attachment of a support bracket or support rail to the primary structure or back wall by means of one or more anchors.
Factor of safety	Factor applied to the required load capacity to give a safety margin between anticipated load capacity and required load capacity.
Fixing	Device for positioning and restraining a stone panel as Part of a facade.
Non-support fixing	A fixing that provides no restraint to movement in the plane of the stone panel but provides restraint in the out-of-plane direction.
Proof load	Load obtained by multiplying the required load capacity by the required factor of safety for any component or assembly. The load applied to components and assemblies under test.
Restraint	The restriction of movement of a stone panel or fixing in one or more directions.
Restraint fixing	The traditional name for a non-support fixing, see above.
Stone panel	A single monolithic piece of stone cut to form Part of a cladding panel.
Structurally assisted stone	A panel fabricated as a thin veneer of stone that has no structural integrity of its own but is strengthened by a backing material or structure.
Support fixing	A fixing that provides restraint to movement in the plane of a stone panel and transfers in-plane loads from the stone panel to the support frame or back wall.
Thin stone	Stone panels less than 40mm thickness and greater than 20mm thickness.

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