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STRUCTURAL ENGINEER'S REPORT ON

THE ENGINEERING PROGRESS AT MENOKIN

REPORT No. **26 March 2010**

Menokin Project
Engineering Report

This report was commissioned by the Menokin foundation as part of a wider package of information intended to inform the advisory board in their peer review of the work carried out at Menokin. The report investigates the engineering work carried out by Dewhurst Macfarlane and partners, particularly the work carried out by Ed Lowe, the author.

There has not been enough time to perform a complete and systematic review presenting all the material available and so priority has been given to topics for an audience of technically competent professionals selected from disciplines other than structural Engineering.

The report is structured in two parts. Part 1 is an overview of the engineering considerations that have lead to the details and is structured around the material properties. Part 2 looks at the scope and products of the design work carried out by Dewhurst Macfarlane and partners. All the graphics, drawings and information discussed in the report have been available in soft copy.

PART 1

Materials Properties

The proposed Menokin glasshouse exhibit will be composed of a rare palette of materials. There are the existing rubble walls and timber beams; the new glass rafters and acrylic connections; and even carbon fibre reinforcement and resin repairs. Each material has been chosen to make use of key properties, but all materials have important limitations. These considerations are made explicit in this section, in order to bring clarity to the factors driving the design of each element.

The Performance of Structural Glass

Glass is a notoriously fragile material that can fail without warning. Everybody has experienced a broken ornament or a smashed windowpane and as a result there is a perception that glass is far less durable than the alternatives. There is some truth in this assertion, but it is probable that the perception of risk outweighs the actual risk. Glass is a very stable material and will survive extreme environments if it is used carefully. In a well-designed structure the risk of failure is controlled to an acceptable level and this is the case for any designs constructed of any material. The following paragraphs explain how engineers apply processed glass so that it can be used as a reliable structural material.

The practical strength of glass is a tiny fraction of its theoretical capacity, because unlike almost any other structural material, glass can only deform elastically. Therefore any concentration of high stress, like that found on the tip of a crack, cannot be relieved by local yielding. In contrast, timber is very forgiving and will deform plastically by reorganising the material structure in the immediate area of concern and this allows you to drive nails into wood. Glass cannot dissipate energy in this way and once an unbearable stress is reached, a crack will propagate rapidly across the element and separate it into two or more pieces, without warning.

Premature failure of any individual glass element is caused by an imperfection being loaded in tension, for example; a chip on the surface of the glass that is subjected to bending.

Unfortunately, it is inevitable that all sheets of float glass will accumulate thousands of microscopic defects as soon as they are produced. These are known as Griffith's flaws and they provide opportunities for unsustainable crack growth. However, it has been shown that cracks will not propagate within an element that is loaded with an average stress of less than $7N/mm^2$ (1 kPSI) and this sets out a safe long-term design strength for standard annealed glass.

There are various ways to improve the reliability of glass. Fibreglass involves many tiny strands of glass locked in a resin matrix. The strength of the whole is far greater than the parts because unstable cracks do not propagate through the matrix and therefore the fate of the element is not dependant on any single unfortunate flaw. In the same way, laminated glass beams are more reliable than solid glass elements, because the failure any single ply does not result in the failure of the whole element. To avoid catastrophic failure, all primary glass elements are designed to have sufficient redundant strength so that they remain safe after the failure of one ply. This means the element can be replaced before the occupants are put at risk.

The failure load of a single ply of glass is unpredictable and there is a large variation between the weakest and the strongest specimens in any given batch of glass. Statistical models show that where fins or beams are edge loaded, two plies of glass are about twenty times more reliable than a single ply of glass under the same load and three plies are two hundred times more reliable. The reliability of multi-ply laminated glazing could be used to economise on glass, but in general we keep it as a margin of safety, because glass can be heat treated to create a material that is very reliable as long as it is loaded carefully.

Heat Treated Glass

Three main classes of structural glass are produced by the heat-treatment of float glass. Float glass is annealed after production and annealed glass is widely used because it is cheaper and it can be machined on site. Heat strengthened and toughened glass are manufactured by heat-treating annealed glass to produce useful internal stresses within the glass. Heat strengthened glass has a balanced internal stress profile with a surface compression of around $29N/mm^2$ (4.2 kPSI) and heat toughened glass has a surface compression of at least $69N/mm^2$(10 kPSI). In the same way that load bearing masonry secures mortar joints in tension, heat treated glass can be relied upon under tensile loads that do not overcome the surface compression. Higher tensile loads will start to load the surface flaws in tension will less predictable consequences.

Many significant flaws are created during the machining of glass and freshly cut edges and holes will need to be ground before any load is applied. Toughened glass cannot be machined because any practical process releases the internally stressed material leaving unbalanced forces that will activate the flaws as they are created. Therefore toughened glass is only used in primary load bearing structure, such as the joists, beams, fins and rafters.

Toughened glass is more capable of resisting impact than annealed glass, but when it does break, all the internal stress is rapidly released shattering the element into tiny pieces. Heat strengthened glass has not been stressed to the point where it shatters on failure and so laminated or well supported annealed glass and heat strengthen glass maintain some load bearing capacity after failure. Secondary glass elements that will not require machining on site, like the glass shelves that support the belt course, could be heat strengthened to give them good failure properties and some toughness. All glass that may need to be machined after it has been supplied must be annealed and so the building envelope will probably be constructed entirely from annealed glass, which may or may not be laminated.

The Performance of Historic Timber

The properties of historic timbers can compare favourably to the predictions of modern timber strength grading systems because the techniques for selecting, sawing and assembling the timbers have changed significantly over the past 240 years. A designer of modern timber buildings must assume that a low quality timber has been positioned with its weakest point in the most unfavourable location and that often leads to a considerable amount of redundancy in the design of timber elements. It is possible to inspect a historic timber and make judgements about its history in order to gain more certainty of its probable strengths and weakness'.

It is probable that the building economics of C18th Virginia enabled the use of well-selected high-grade timber. The building stood for many years before it was damaged and so we can assume that the characteristic strength of the timber is at least as much as would be required to carry the actual loads experienced up until the time of failure. Later in the project it will be necessary to make a more detailed assessment of the strength of individual timbers, especially where they have failed in service or are showing signs of distress or deterioration. In the meantime DMP are comfortable assuming that the characteristic strength of Menokin timber is 10N/mm^2 (1.45 kPSI).

Testing of Timber

In most cases it would be counterproductive to test historic timbers to destruction, but if there are some dispensable timbers, then we would test in accordance with standard testing procedures. The results of this test would supersede DMP's assumptions on characteristic strength and a factored version would apply to all timber deemed free from signs of distress or deterioration.

The traditional method of non-destructive testing is inspection, which still is the most important analysis tool in the process. Any judgements will consider the proposed arrangement of the timber when it is in service and experienced inspection is required to determine whether further testing is necessary and which tests are required.

"Signs of distress, excessive deflection, water leakage and cracking all indicate problems. An assessment of knots and other defects can allow an approximate strength grade to be assigned to the timber and enable design stresses to be determined." (TRADA Wood Information - Non-destructive testing of timber)

The applicability of inspection is governed by the accessibility of the supposed weakness. Some flaws are beneath the surface and require probing techniques. Some available techniques are listed. These tests have not yet been applied to the timbers in any systematic way. At some point there should be an investigation into the most appropriate techniques for Menokin.

- **Non Destructive Bending** will determine stiffness and help make comparative judgements about load carrying capacity.
- **Resonant Vibration** frequency depends upon the stiffness, density, shape and fixing conditions and can be used in conjunction with more certain information to fill in the missing details.
- **Ultrasonics and Stress waves** interact with the internal structure of the timber and can be used to determine the presence and locations of flaws
- **Hardness** testing can be carried out to determine the surface condition of timber for bearing or to make wider judgements

- **Energy absorption** can be measured by a number of techniques and is a strong indication of decay and therefore toughness and strength.
- **Microscopy** is method of identifying species and fungal types in samples.
- **Endoscopy** is a form of remote visual inspection carried out using rigid tube borescopes or optical fibres and miniature video cameras.
- **Electrical Resistance** will indicate the moisture content of a timber.
- **Bioassay** is the process of incubating samples to determine the presence biological elements that indicate fungal decay.
- **Infra-red spectroscopy** studies the characteristics of a material sample and identifies the chemical elements and compounds.

	Imperial			Metric			Normalised to Steel		Normalized to Steel by Specific Weight	
	Stiffness	Tensile Stength	Density	Stiffness	Tensile Stength	Density	Stiffness	Tensile Stength	Stiffness	Tensile Stength
	MPSI	KPSI	lb/in3	N/mm2	N/mm2	te/m3	-	-	-	-
Carbon Fiber	33	580	0.07	227535	3999	1.8	1.1	14.5	4.8	63
Carbon Epoxy Composite	17	275	0.06	117215	1896	1.6	0.6	6.9	2.9	35
Aluminium 6063-T6 Aluminimum	10	36	0.10	68950	248	2.6	0.3	0.9	1.0	3
Titanium	15	170	0.16	103425	1172	4.5	0.5	4.3	0.9	7
High Carbon Steel 1090	30	122	0.27	206850	841	7.6	1.0	3.1	1.0	3
Mild Steel S275	30	40	0.28	205000	275	7.8	1.0	1.0	1.0	1
Aramid Fiber	18	400	0.05	124110	2758	1.4	0.6	10.0	3.3	54
Aramid Epoxy Composite	9.8	190	0.05	67571	1310	1.3	0.3	4.8	1.9	28
E-Glass Fibre	10.5	500	0.09	72398	3448	2.5	0.4	12.5	1.1	38
E-Glass Fibre Epoxy	6.2	257	0.07	42749	1772	2.0	0.2	6.4	0.8	25
S-Glass Fibre	12.5	665	0.09	86188	4585	2.5	0.4	16.7	1.3	52
S-Glass Fibre Epoxy	7.7	342	0.07	53092	2358	2.0	0.3	8.6	1.0	34
Menokin Timber (DMP)	1.2	1.45	0.07	8500	10	2.0	0.04	0.04	0.16	0.14

http://www.carbonfibertubeshop.com/tube%20properties.html

*“**Aramid** fibers are a class of heat-resistant and strong synthetic fibers. They are used in aerospace and military applications, for ballistic rated body armor fabric, in bicycle tires, and as an asbestos substitute.*”(Wikipedia!)

The Performance of Carbon Fibre

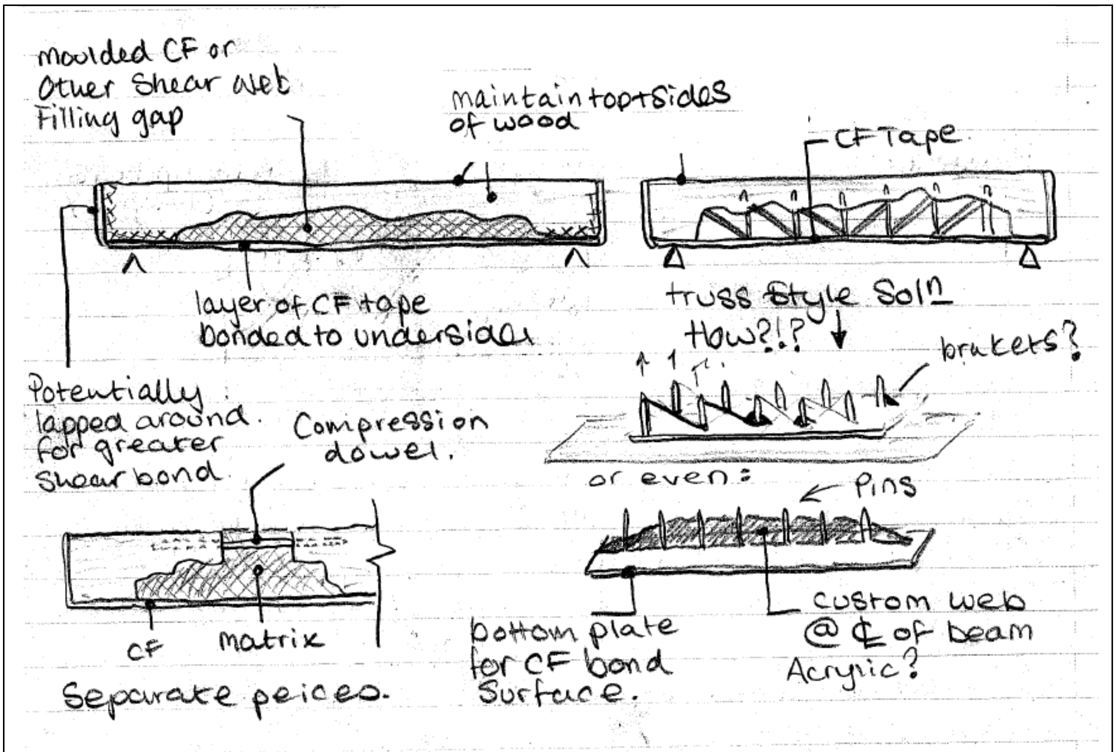
Carbon fibre (CF) is more than fourteen times stronger than mild steel and if account is taken of the varying densities, then CF over 60 times more efficient than steel. That doesn’t permit a factor 60 reduction of section dimensions; carbon fibre is only four times as stiff and considerably more expensive. However, CF is a remarkable improvement on timber, as CF is about 350 times stronger than Menokin Timber. This means that thin sheets of CF cloth can easily bear the tensile stress range expected in timber bending. Being cloth it cannot take significant compression.

These properties make CF the ideal tension element in a composite CF, epoxy and timber beam. Timber artefacts are more capable of resisting compression than tension. The failure sequence of a timber in bending becomes unstable when the tension flange fails. Premature compression damage causes more tension in the fibres, when the tension fibres fail, the element loses its ability to carry load altogether. This means that the strength of a timber is significantly improved by bonding a CF strip along the tension flange – the bottom surface of a simply supported beam. The strength of this arrangement is typically limited by the epoxy bond strength. Test of this arrangement have shown some impressive results.

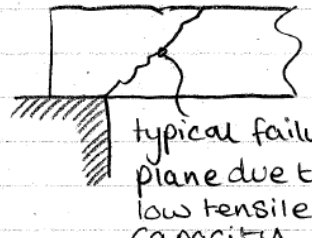
Timber Repairs

All timber repair techniques are in the experimental stage and have not been applied to significant Timbers. John Greenwalt Lee has been perfecting various techniques and some of his work is shown. In this section I have included sketches showing how we could optimise repairs by applying proven structural principles to reinforce the repair. These are early ideas and they need more development.

	
Timber Splice Detail	*Carbon Fibre Repair*

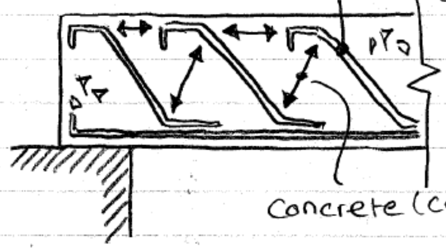


Reinforced Concrete Analogy:



typical failure plane due to low tensile capacity.

Bent-up bars. (tension)

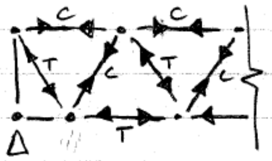


Concrete (compression)

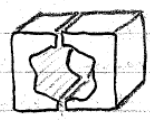
Bent-up bar reinforced Concrete

Assume
- Epoxy good in Comp.
- bad in tension.

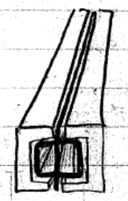
Solⁿ... Action simular to a truss:



Case of Rotten Center...



Current Solⁿ:-

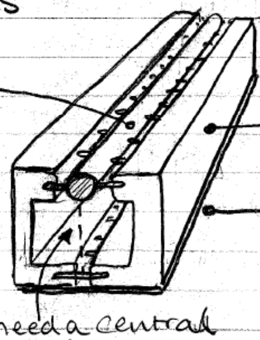


Insert CF Beam into rotten core.

- In this example the remaining fragments look suitable as Shear blocks/webs

model:

M.Steel or CF Rod between peices linking them together provides Compression Flange.

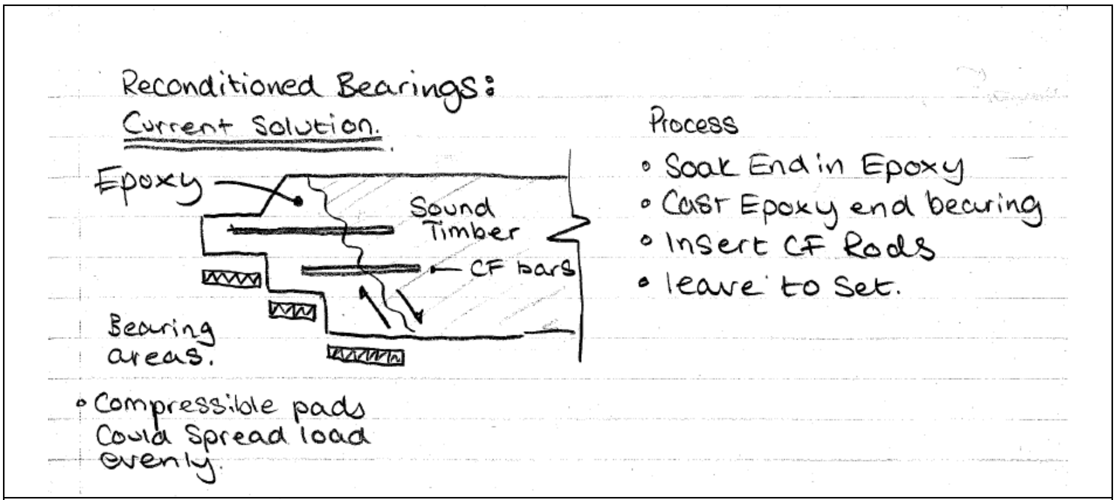


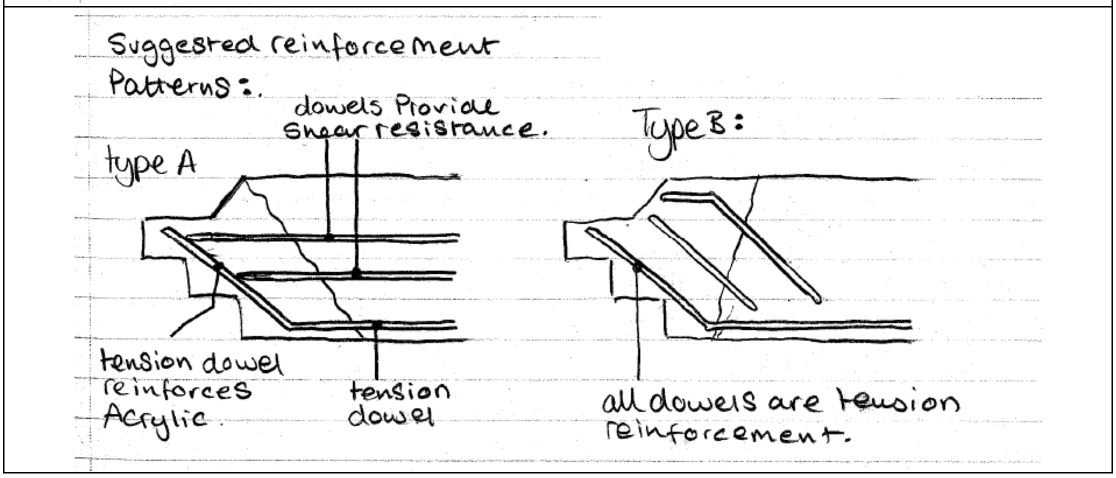
Sides act as Shear web.

CF Tape bonded to underside provides tension Flange.

N.B. this arrangement increases efficiency by separating flanges.

may need a central web depending on geometry.

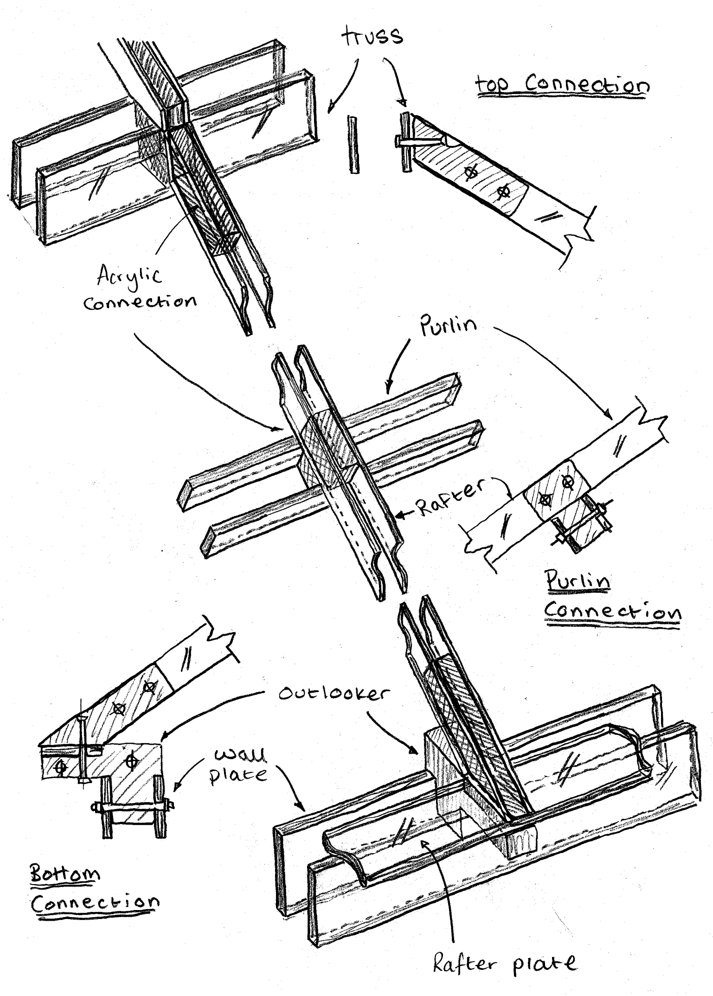




Glass Beam Design Rational

This section sets out a general philosophy for designing prosthetic replacements for missing or severely damaged timbers that have to be replaced. New elements will be constructed from transparent materials, such as glass and acrylic. The dimensions of the replacement elements will conform to the original cross section dimensions throughout the length of the element and in way of the connections.

DMP have assumed that 10N/mm^2 (1.45 kPSI) is safe design strength for undamaged Menokin Timbers. Timbers that will be replaced by glass beams will made out of multi-ply laminations of toughened glass. The characteristic strength of toughened glass is 70N/mm^2 (10.15 kPSI), but the material design rational for primary glass elements includes a 25% margin to insure there is sufficient redundant material to allow the failure of one ply. This implies a design bending strength for toughened glass elements of 50N/mm^2 (7.25 kPSI).



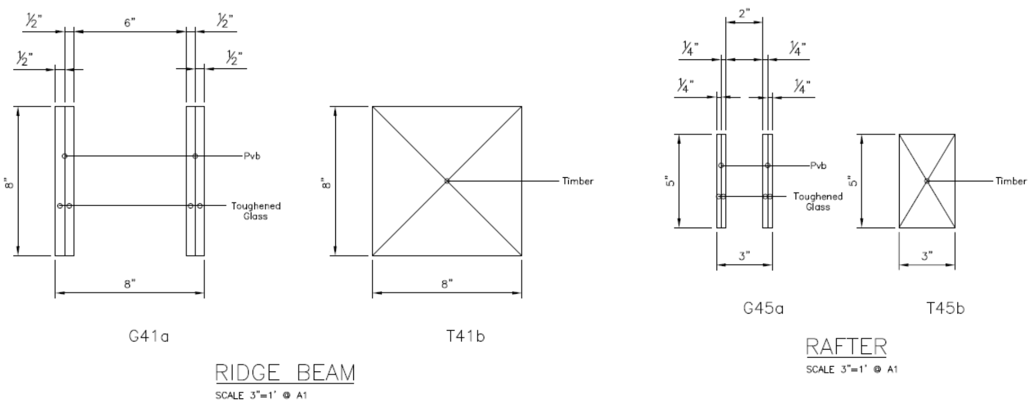
The 5:1 ratio between the design strength of glass and the design strength of timber sets out a general rule for designing glass replacement elements. The size of the timber to be replaced is known and time has shown that the specification was sufficient to carry the loads. Therefore, if the bending depth of all elements is equal, then the width of the new glass element need only be one fifth of the width of the original timber. The required width of the glass beam will be split into two elements and held a constant distance apart, so that the total width matches the old timber element. This arrangement leaves a convenient gap between the two elements that can be used for housing acrylic connections.

Sketch of Typical Rafter Arrangement

The Performance of Acrylic

Acrylic can be combined with glass to unlock the structural and optical potential of both materials. Acrylic can be cast into transparent blocks that transmit over 90% of light and can be machined into useful shapes. Acrylic is relatively soft viscoelastic material and will accumulate surface flaws easily if not treated with a hard coating., Unlike glass it can absorb energy by plastic deformation and thereby exhibits significantly better toughness. Its ability to carry long-term loads depends upon temperature and weathering and therefore the rate that acrylic will creep depends upon the history of the element. Under the right circumstances, the ultimate strength of acrylic is comparable to glass, but it has a significantly lower stiffness and that places considerable limitations on its application.

The mechanical stability of acrylic is particularly susceptible to organic solvents. It is UV stable, but direct sunshine and weathering will cause heat cycles that degrade the structural properties with time. Compromised elements show visible signs of deterioration and distressed elements can be spotted early in the failure sequence. The ability of an element to perform its through life function is both complex and uncertain, and this must be managed by enhanced safety margins and periodic inspection. Under these circumstances the majority of acrylic parts can be safely relied upon to survive beyond their design life.



Shop Drawings Showing New Glass and Original Timber Sections Side by Side

The Performance of Glass Acrylic Composites

Timber elements in the Menokin house form simply supported beams that span between two relatively flexible connection points located on supporting structural elements. The transparent replacement elements will be designed to make the best use of the properties of acrylic and glass. Glass cannot tolerate shock loads or stress concentrations, but acrylic is too flexible to form a significant part of a loaded structural element without deforming significantly.

The bending stress in simply supported beams peaks mid-span and therefore the spanning part of the beam is composed of stiff laminated glass plies, which can resist a significant bending moment. Acrylic can be formed into complex geometries that can meet the needs of irregular joint arrangements and where glass is too hard and unyielding to make contact with itself, acrylic makes a relatively soft and forgiving connection part.

If the connections are detailed correctly, then the bending moment will diminish towards the connection points. Acrylic connection blocks can safely transfer the forces required to hold the elements in place. They may even strain enough to redistribute or release forces, without passing problems from one element to the next, in the same way that a timber frame would accommodate loading irregularities.

Load-Bearing Masonry

The masonry walls of Menokin are the primary support to the floors and the roof. This is why it is said that the characteristic form of construction at Menokin is load-bearing masonry. Many buildings have masonry panels, but that does not mean that they rely on or even require the structural properties of the masonry. Most sizable modern structures rely on a framework of primary columns and beams. Sometimes masonry is used for stability, but the majority of modern masonry is only applied as an infill panel required to create a functional building envelope to keep the rain out and the warmth in.

The load-bearing aspect of the masonry at Menokin is an important part of the value of the building. The construction industry's application of this technology has become so sporadic that most designers are not aware of it's potential and many completely disregard its capacity to do any work at all. This is extremely surprising as the majority of our architectural heritage is constructed from load bearing masonry that has stood for hundreds of years.

The Modern Framework

In the modern construction the difference between success and failure depends upon performance in a competitive marketplace. The market has evolved an unfortunate capacity to select based on short-term profitability. In this environment the advantages of quick execution often outweigh the benefits of a long-term investment, because it is unprofitable in the short-term. The economics of the construction industry mean time is a scarce resource and the assembly time of a masonry panel has become a significant adoption deterrent.

Therefore structural frameworks have become the new tradition in modern construction. They are quick to assemble on site and increasingly subassemblies are manufactured off site, on a production line, in a batch or even continuously. The technology is advanced because it involves the culmination of many specialised processes that rely upon a complicated infrastructure of interrelated industries.

One of the reasons a structural framework is so desirable is because it concentrates the load carrying capacity of the structure into well-defined elements that can be simply understood and manipulated with certainty. In this sense 'advanced' technology has succeeded because it has rationalised a complicated situation into a standardised unit that can be understood and applied by practitioners who have no need for the wealth of information that has lead to its creation.

In many cases it is impossible to meet structural objectives without taking advantage of modern technology, but this does not make it better in every case. Often it is simply more convenient, but the convenience of modern frameworks comes at a price. It is becoming increasingly clear that we cannot afford to pay the price for every structure that is ever built and therefore we need to regain the knowledge accrued over the development of hundreds of years of load-bearing masonry construction.

The Performance of Menokin Masonry

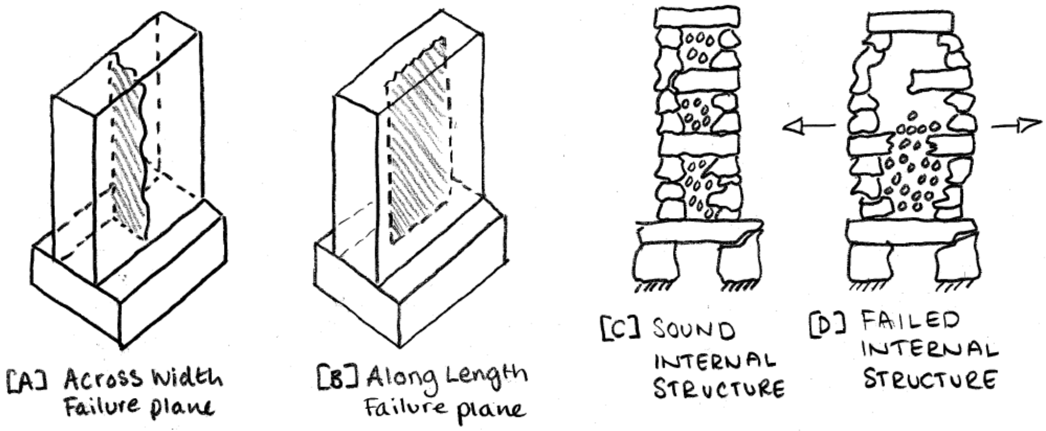
The masonry at Menokin consists of dressed stone, stone rubble, bricks and lime mortar. These units are stacked together in a woven pattern with mortar bed into the joints and the spaces. The units are arranged to insure that the outer surfaces are as vertical as possible. This is easy to achieve with regular shaped bricks, but when irregular shaped rubble is used this practice tends to leads to the formation of two vertical stacks with no bricks common to each wall faces. Therefore larger bricks are placed at regular intervals across the width of wall to tie the two leaves together. As the wall is built any internal space is filled with rubble and mortar.

Integrity of Internal Structure

The performance of a wall depends upon its ability to act as one unit and to remain stable in relations to its foundations. The internal structure of a wall will fail when the force linking the bricks together is overcome. Horizontal mortar joints are held together by gravity. The mortar acts as a cushion, spreading the load to make a positive connection with good resistance to lateral dislocation. Such a load bearing connection is reliable, but it can be over come by the application of lateral forces.

The geometry of the wall is such that a wind load can be amplified at an interface some distance away from the point of application and this will cause the joints to open up. Lime mortar does not provide substantial resistance to tensile forces separating stones, so this force must be resisted by the weight of the masonry above. Therefore it is prudent to put a great deal more mass above any given joint than would be necessary in a structure that has tensile strength. So, for a wall to hold together there must be sufficient compression in the bed joints to overcome unstablising forces and the individual elements must not crumble under that pressure; therefore each element must be load bearing.

Vertical mortar joints will transfer lateral compression in the same way that horizontal bed joints can support load, but when in tension they also rely on gravity, but not directly. The wall is arranged so that any pair of adjacent units has a common unit bearing down on them from above. In this way adjacent bricks are prevented from separating from their neighbours by a series of horizontal bed joints linking the pair together through a common unit above. If the structure is going to hold together then the there must be sufficient load on both stacks of bricks to transfer the shear force through the mortar into the common brick. The common brick must then hold in tension.



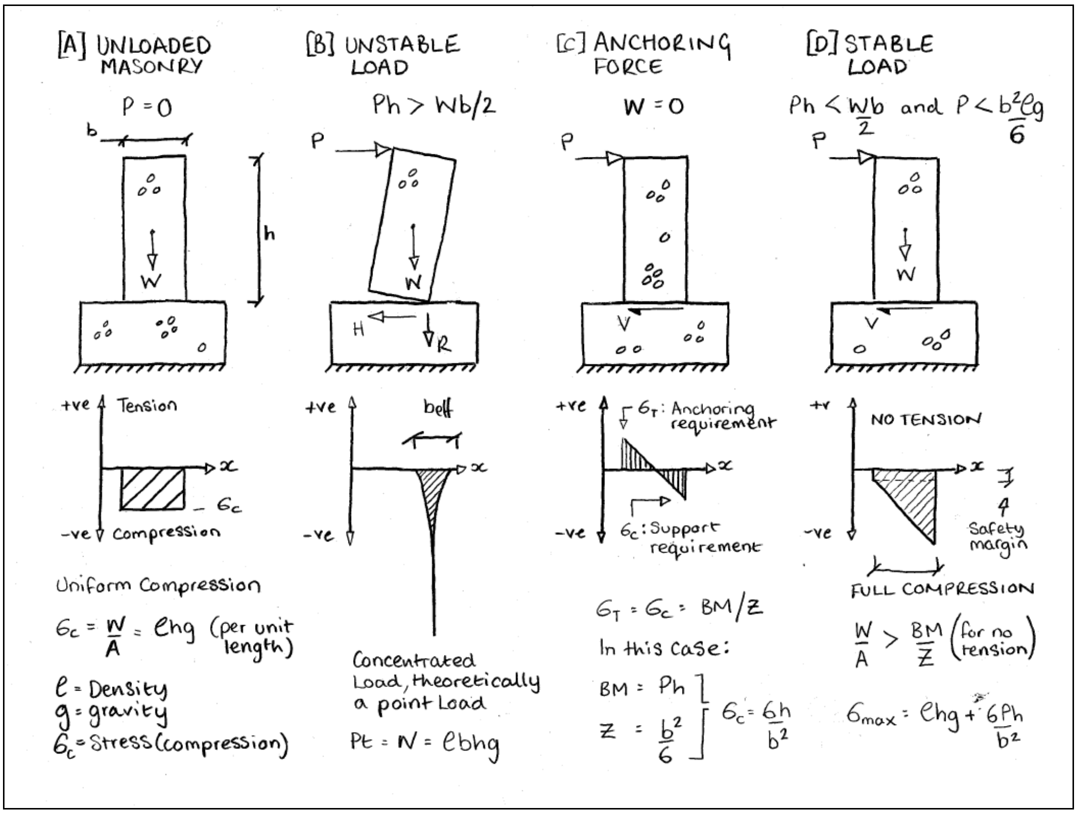
Masonry Internal Failure Mechanics

If a failure plane opens up across the width of the wall, as is illustrated in image [A] then it is only serious if it is caused by one part of the wall becoming unstable in relation to its foundations. Otherwise there is plenty of material down the length of the wall to restrain any further longitudinal deflection and if it is transversely stable then the crack will not grow. A crack in a stable wall is more significant when a failure plane opens up within the length of the wall, as illustrated in image [B]. This situation may occur because transverse tie bricks are sporadic. When a tie brick fails a portion of the wall begins to act like two panels. These new elements are more slender than the whole wall and slender structures are less stable in than chunkier equivalents. Therefore the wall will buckle under a smaller load causing the walls to laterally deflect starting a failure sequence. Debris travels down the new gaps and gets lodged in the mortar causing any further movement to be less easy to reverse. In this situation there may not be any visible signs of distress, until the wall starts to bulge.

Stability of Load Bearing Masonry

A wall finally fails by collapsing. This may be because the internal structure has fell apart though deterioration or because the entire wall has become unstable or a combination of both. A healthy load bearing wall should move by compressing the soft mortar joints. Mortar cushions masonry from high bearing stress. When a wall is regularly subjected to too much bending then the mortar joints will become tensioned and it will deteriorate and/or collapse.

Forces that lift the masonry out of the mortar redistribute the load into concentrated areas. The lime will start to re-carbonate and the mortar starts to crumble. Soon the bricks start bearing on each other with intense loads, grinding away with ever increasing load cycles that intensify according to the decreasing stability of the entire wall. The overall stability of the wall depends on the applied bending moment being less than the stabilising moment. The applied bending moment is a function of destabilising force and the applied leverage. The stability of the element depends on its mass and the position of the centre of gravity.



Unfortunately is difficult to get reliable information about the stability of a wall, because you can't accurately determine the internal structure. Assumptions can be made and when they are, it is a straight forward task to determine the stability and work out how much additional mass is required to prevent the wall from deteriorating any further. However, the answers are only as accurate as the information used to generate them.

Assessment of Masonry (Tim Macfarlane)

The structural analysis to date has been based on experience of JGLCo and DMP, using measurements and architectural and engineering knowledge gained through years of work on load-bearing masonry buildings of this era. No attempt has been made by DMP to analyze the existing structure but this exercise can be carried out at short notice if the Foundation deems this a funding priority. The accuracy of the analysis will depend on the assumptions made about the remaining stone and timber elements and will, in the end, be a judgement call rather than a strict proof one way or the other.

Under JGLCo, the historic structure has been eased back into a stable, vertically-loaded condition using non-invasive techniques. The stabilization program to date has removed fallen structural elements that were pushing walls out of alignment and added to stability by returning loads to their proper vertical alignment and grouting voids in the masonry to cushion and evenly spread the weight.

At the moment the conservation team and DMP have assessed the stability of the structure from a visual standpoint … which from our considerable experience is as reliable as a calculated assessment. However, a full set of calculations for the completed structure will be developed before reconstruction begins. In order to do this full calculated assessment, the foundations will need to be fully unearthed and analysis of the current condition of remaining structural elements completed, as well as the R&D for the proposed conservation approaches being considered – all of which was cancelled by the Foundation before completion in 2008. Thus a complete calculated assessment exceeds what the Menokin Foundation has contracted for to date.

With the “Glass House” project which the Foundation had previously signed onto now being halted, the focus needs to shift from excavation of fallen building elements to a strict focus on the standing elements to make them as stable as possible. However, at this time there are no 2010 contracts with DMP or JGLCo to proceed with engineering, stabilization or conservation.

PART 2

Summary Previous Work

DMPs involvement with the project goes back as far as 2006. In the early months Tim Macfarlane managed the project alone. In May 2007, Mario Claussnitzer became lead engineer and his work continued until May 2008 when he left DMP. Glen Housten worked in parallel with Mario early in 2008 and he took over project management when Mario left.

Ed Lowe, the author of this report, joined Dewhurst Macfarlane & Partners in December 2008 and assumed ownership of all Menokin project work soon after. The majority of this report relates to work conducted by Tim Macfarlane and Ed Lowe. This section has been included to indicate the scope of previous work done, but it is not complete and it would be useful it provision were made for a more systematic assessment of existing engineering information.

In March 2008 Glen Housten investigated suitable tests to assess the structural performance of composite carbon fibre and epoxy resin beams and he proposed five tests to isolate the following structural properties. (Carbon Fibre Tests 06 08.pdf)

1. Tensile strength of epoxy resin bond between carbon fibre and timber.
2. Tensional strength of epoxy resin bond between carbon fibre and timber.
3. Bending Stiffness of carbon fibre beam
4. Bolted connection bearing capacity of hole in carbon fibre Beam
5. Tensile pull-out capacity of epoxy resin bond between steel rod and timber.

In August 2008 Nick Roach was instructed by Glen Houston to start work on the following items for the design of the main house.

1. Determine framing plan
2. Calculate Loads and bending moments
3. Propose innovative repairs
4. Design glass floorboards, roof panels, joists and beams.
5. Scheme design assess stability
6. Design connection details
7. Design foundations for glazing

In November 2008 Glen Housten reported the specification of the elements required for the link passageway. Many of the calculations are still relevant today, but there have since been fundamental changes in the provisions for building stability; the glass portal frames, which act like a rigid ‘goal post’ to stop the building falling over have been replaced by cantilevering fins. This new arrangement removes the deep cross-beam, that was neither elegant nor a useful expression of the proposed glasshouse arrangement.

Link Passage

Summary Link Design Work

Design work on the link structure started in December 2008. Work carried out over this period:

1. Collaborated with the Charles Phillips to resolve the structural and architectural requirements of the building.
2. Scheme Design
3. Engaged specialist fabricator IPIG Ltd
4. Investigated the arrangement for construction feasibility.
5. Performed structural calculations to check the design feasibility.
6. Produced general arrangement drawings.
7. Produced detailed drawings of subassemblies.
8. Produced shop drawings of components.
9. Compiled a schedule of quantities.
10. Produced 3D promotional graphics.

This phase of the project ended in January 2009, after a formal issue of information including drawings, schedule and construction sequence. The items included in the final link passage issue are listed below.

- 001 Foundation Plan
- 002 Ground Level Plan
- 003 Water Table Plan
- 004 Eaves Plan
- 005 Roof Framing Plan
- 006 Roof Panel Plan

- 011 Longitudinal Section
- 012 Transverse Section

- 021 Acrylic Connection Details
- 022 Masonry Interface Details
- 023 Glass to Timber Splice details

- 031 Component Sheet 1
- 032 Component Sheet 2
- 033 Component Sheet 3
- 034 Component Sheet 4
- 035 Component Sheet 5

- Schedule 200109.xls

- 3D construction sequence.pdf

Summary of work done by IPIG – Specialist Glass Fabricator

IPIG were first engaged in November 2008 and they have since contributed to the project in the following ways:

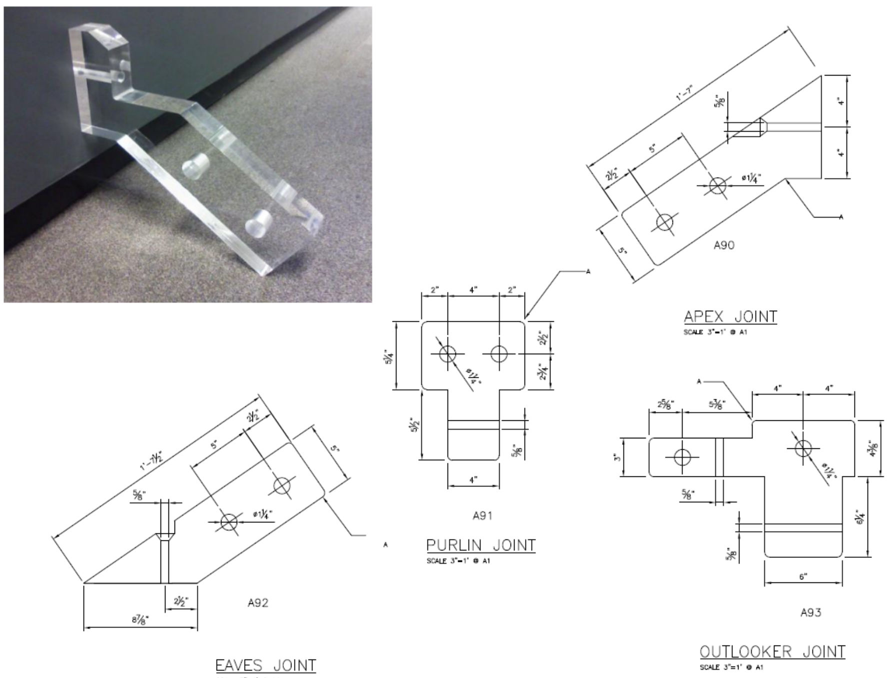
1. They have provided advice to guide the design progress and investigated the feasibility of various elements.
2. They produced a sample of an acrylic connection block.
3. Produced forecast budget from the schedule or materials and drawings.
4. Menokin visit - June 2009: met the team and presented the work of IPIG to the board members.

Budget Forecast - February 2009

This forecast does not include local taxation or import duties. It assumes significant fabrication of subassemblies in the UK and allowance has been made for a single shipment of materials to America.

Cost of Fabrication of Elements:	£280k	$400k
Drawings/Scheduling/Procurement/Admin:	£80k	$115k
Installation*	<u>£120k</u>	<u>$170k</u>
Basic Forecast Cost	<u>£480k</u>	<u>$685k</u>
Including 15% Contingency	<u>£550k</u>	<u>$790k</u>

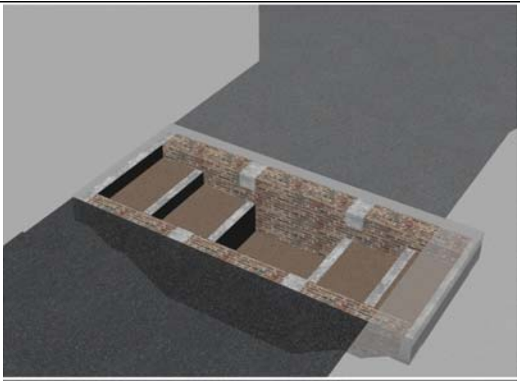
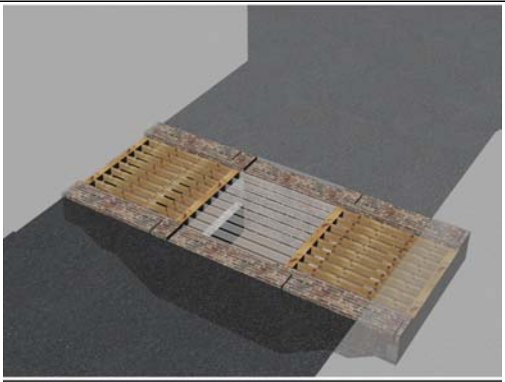
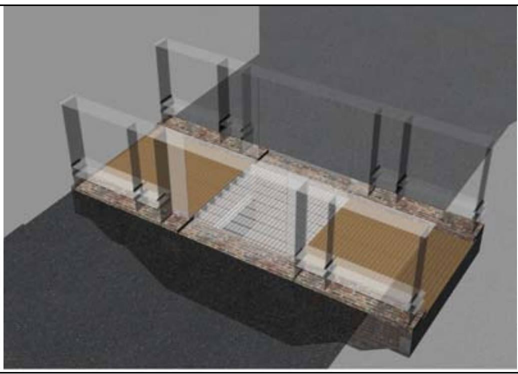
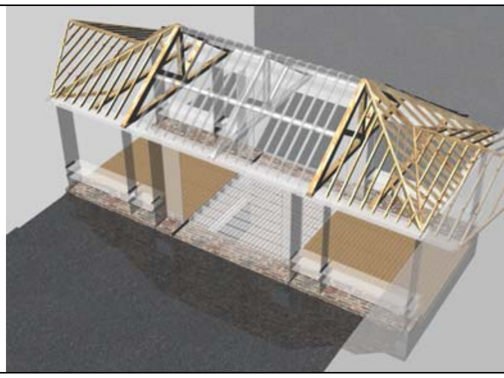
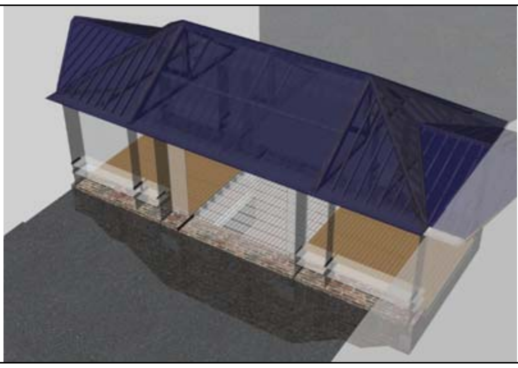
*Installation costs account for a small team of skill artisans and some logistical support.



Typical Acrylic Connection Blocks

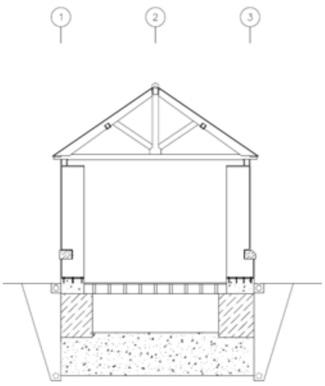
3D Graphics - Structural Layers

A freelance graphic designer – Angie Bessho **-** modelled the link passage in January 2009 under the instruction of Ed Lowe. A summary of the images is presented below.

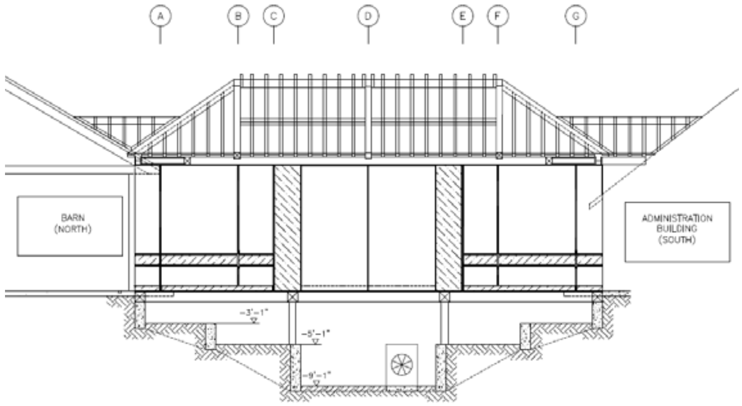
	
[A] Foundations + Retaining Walls	[B] Suspended Floor Arrangement
	
[C] Fins and Shelves	[D] Roof Framing
	
[E] Roof Panels	[F] Historic Masonry

Scheme Design Rationale

The connector passage will be a useful covered walkway between the barn workshop and the administration building. It has been designed as a prototype structure to investigate, develop and test details in preparation for the construction of the main glasshouse. The glasshouse is a square multi-storey building with suspended floors. This arrangement does not have much in common with a long thin single storey passageway. Therefore the design of the walkway has been arranged so that it incorporates as many common features with the main house as possible.

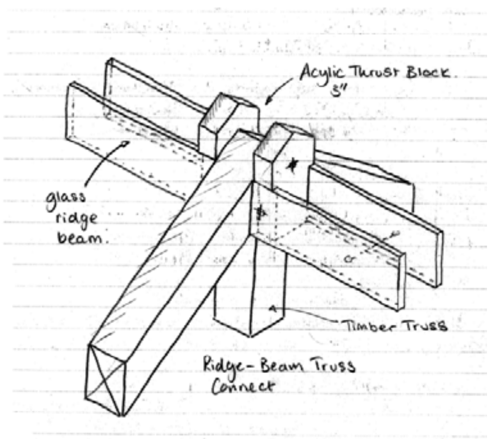


Transverse Section

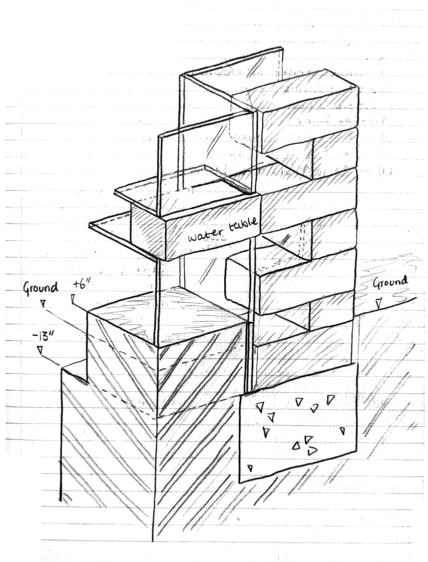


Longitudinal Section

The majority of the link passageway envelope is made of glass, but the structural elements vary between glass, timber and masonry, to capture the wide range of connection details involved in the main building. Toughened glass fins represent the wall depth and brace the glass skin against wind pressure. Stacked stone pillars configured according to the arrangement of the ruin's quoins frame both entrances and this will give an opportunity to investigate and improve the techniques required to cut annealed glass wall panels to fit the irregular profile of surviving masonry.

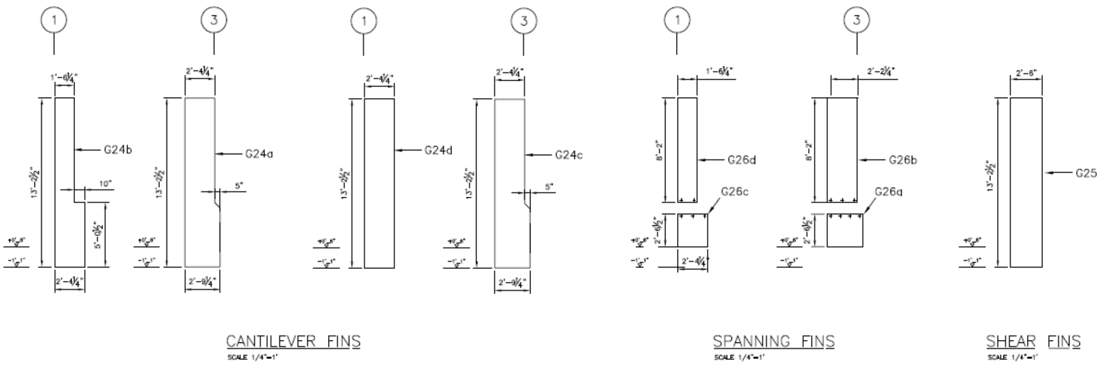


Ridge Beam to Truss Connection



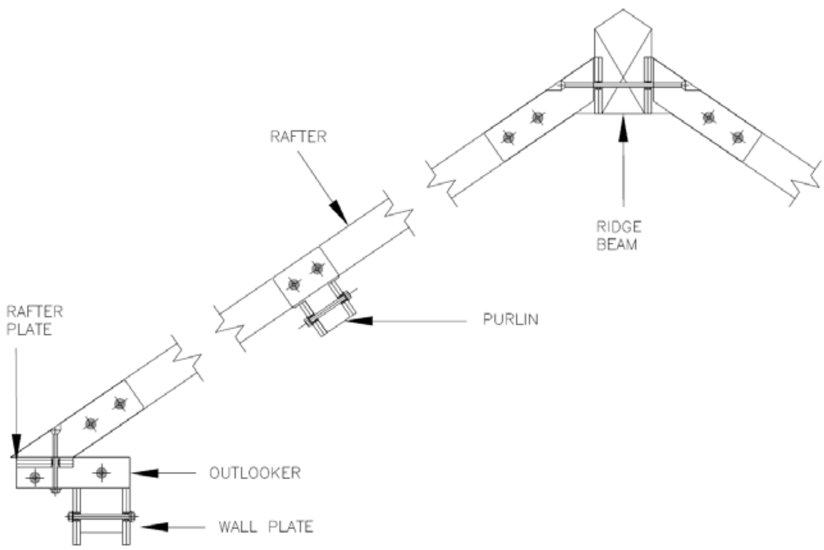
Quoin and Fin Arrangement iwo Entrance

The link passage does not have a masonry core and therefore twelve cantilevered fins secured by concrete blocks embedded into the foundations provide stability. Eight primary fins are orientated to give the building transverse stability and four fins, located behind the quoins, are orientated for longitudinal stability. There are also four secondary fins, similar to those proposed for the main building, which span between the foundations and the wall plate. The fins also support glass shelves which carry the belt course. The secondary fins are separated in two, in order to accommodate a steel connection part which allows the belt course to pass through the fin. This detail is sketched and discussed in the investigation of main glass house.



Shop Drawings of the Various Types of Fin in Elevation

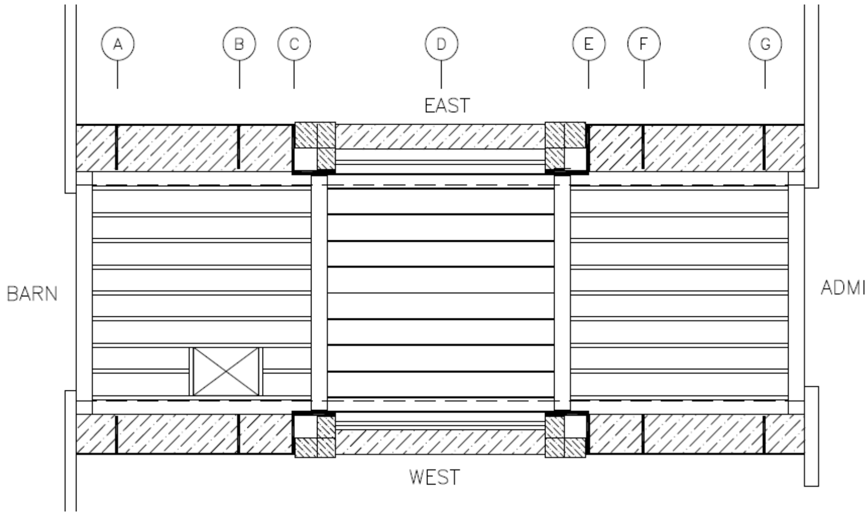
The wall plates span between the fins and support three king post truss frames located on grid lines ‘B’, ‘D’ and ‘F’. The rafter plate bears upon the outlookers that are also supported by the wall plate and positioned in-line with the rafters. The ridge-beam and purlins span between the truss frames in order to give support to the rafters. The rafters carry annealed glass roof panels that may be laminated with a photovoltaic interlayer.



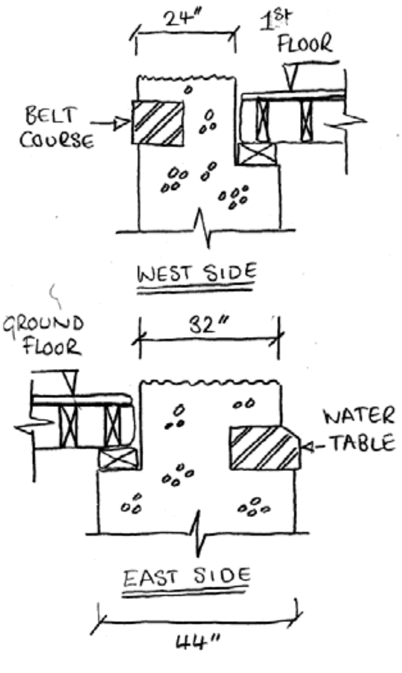
Arrangement of Link Passage Rafters

An 8ft deep basement with a compacted earth floor will be excavated beneath the link passageway, so that the floor can be suspended over, as in the main house. The basement also provides an opportunity to investigate the ground-air exchange tubes that have been proposed in order to ventilate and cool the structure without demanding an unsustainable amount of energy.

Four girders span across the width of the passage separating the floor into three main areas. The end bays are constructed entirely from timber, but the central bay is composed of glass joists spanning between a timber girders and a glass girder. The joists then support glass floorboards. This arrangement has been chosen to capture all possible combinations of timber and/or glass connection details.

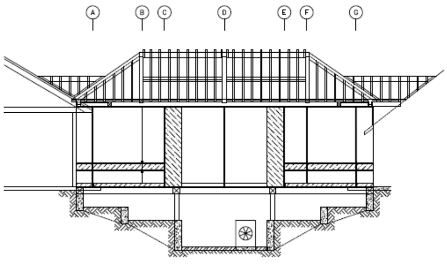


Plan view of Suspending Floor Arrangement

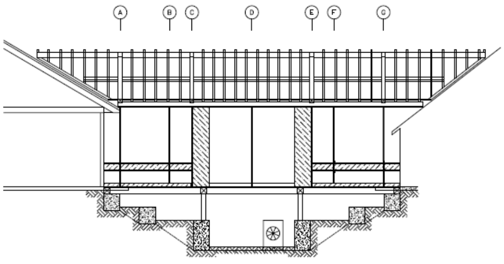


Typical features found on the lower façade, like the 'water-table' course of stone, are expressed on the east elevation of the link passage and typical features of the upper façade, like the belt course, are expressed on the west elevation of the link passage.

The retaining walls of the link basement are designed to recreate the typical arrangements found at the major floor support levels of the ruin and shall be constructed from similar rubble, masonry and mortar. At grade level the east foundation wall steps in from 44" thick to 32" thick, in accordance with the transition from the basement wall to the ground floor walls and the west foundation steps in from 32" thick to 24" thick in accordance with the transition from the ground floor walls to the first floor walls.



Hipped Roof and Cricket



Continuous Ridge Beam

The original design had a continuous ridge beam spanning between the buildings, but it was realised that a continuous roof arrangement does not have many features in common with the main house, particularly in way of the corner details. Therefore the arrangement was altered to shorten the ridge beam in order to include a hipped roof. This is an important modification, because the new arrangement requires details that can satisfactorily frame the interface between two inclined planes and we shall need to develop robust practical solutions for manufacturing and assembling suitable glass components that have either sufficient flexibility or precision to make a neat interface. The arrangement required for the addition of two cricket roofs to manage the drainage of rainwater and these could be constructed from timber, as they do not replicate features of the glasshouse.

Main Glass House

Summary Glass House Design Work

Design work on the Glasshouse structure started in February 2009. Work carried out over this period:

1. Review of original building arrangement from HASB Survey
2. Produced General Arrangement drawings of original structure
3. Review of existing building from photographs
4. Produced General Arrangement drawings of existing structure
5. Collaborated with the Charles Phillips to resolve the structural and architectural requirements of the building.
6. Scheme design
7. Performed structural calculations to check the design feasibility.
8. Produced General Arrangement drawings of proposed structure
9. Investigation into structural reinforcement of damaged timbers
10. Design of Temporary Plexiglas Barrier.
11. Construction of 3D model of geometry (DMP)
12. Support of graphic design process to create promotional graphics (Future Realities)

This phase of the project ended in January 2009, after a formal issue of information including drawings and promotional graphics. The key items delivered for the glasshouse are listed:

Drawings of Original Structure based on HASB Survey:

- E201 Basement Plan
- E202 First Plan
- E203 Second Floor Plan
- E204 Attic Plan
- E205 Roof Plan

Drawings of Existing Structure based on photographic Evidence:

- S211 Basement Plan
- S212 First Plan
- S213 Second Floor Plan
- S214 Attic Plan
- S215 Roof Plan

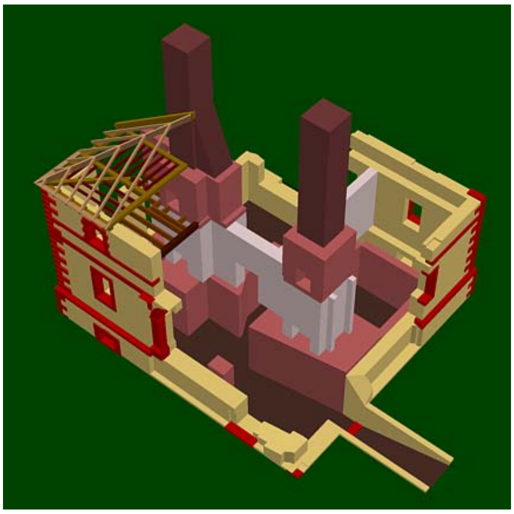
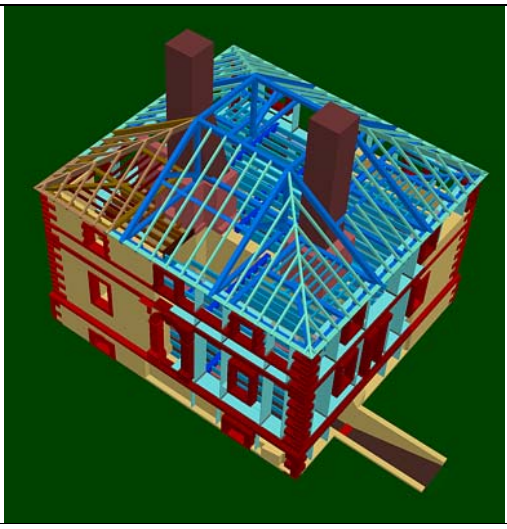
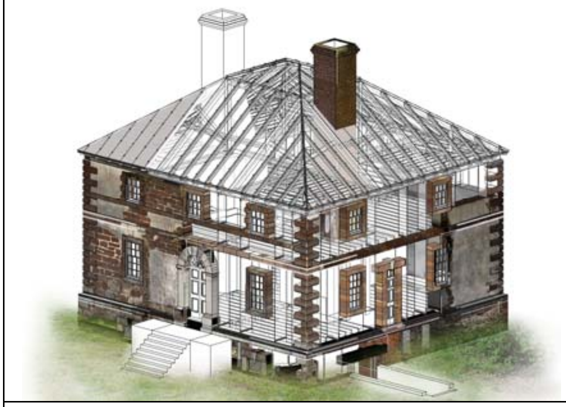
Drawings of Proposed Structure:

- 101 Basement Low Level Plan
- 102 Basement Cill Level
- 103 1st Floor Joist Bearing Level Plan
- 104 1st Floor Plan
- 105 1st Floor Window Level Plan
- 106 2nd Floor Joist Bearing Level Plan
- 107 2nd Floor Plan
- 109 2nd Floor Window Level Plan
- 110 Attic Floor Plan
- 111 Roof Framing Plan
- 112 Roof Cladding Plan

- 121 Elevations

Main Glass House Graphics

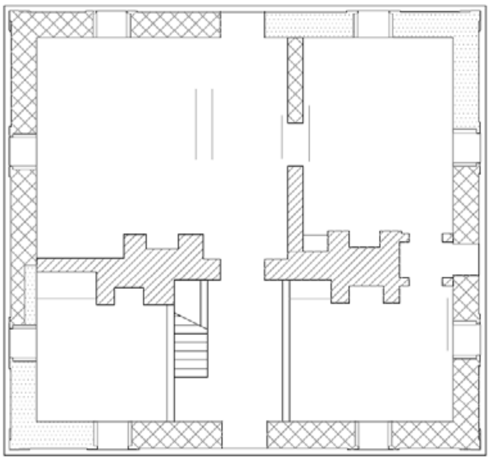
The geometry model was created by Ed Lowe. This was then sent to Future Realities, a graphic design company which applied material textures taken from many photos of the buildings. The illustrative images and the promotional renders are a product of Future Realities.

	
Geometry Model – Existing Structure	*Geometry Model – Proposed Structure*
	
Illustrative Rendering – Full House	*Illustrative Rendering – Close Detail*
	
Promotional Graphics – Day	Promotional Graphics – Night

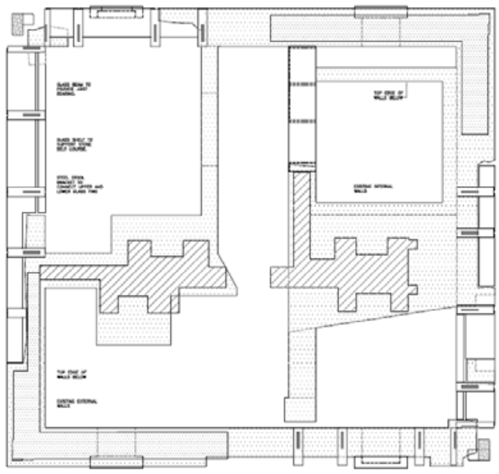
Main Glass House Design Details

The main glass house will be a museum, a display cabinet, a prosthetic repair and hopefully a classic example of good architectural conservation. We have tried to design elegant replacement details that suit these purposes by expressing original structure wherever possible. Currently work has identified the existing structure and determined a proposed arrangement. There has not been enough time to included restored elements in the general arrangement drawings. A useful way to categorise the elements is by the time of construction:

Existing Elements: On site and not requiring repair and reintegration.
New Elements: Modern replacement materials.
Restored Elements: Original elements to be restored.



Existing Structure – Below 1st Floor Joists



Proposed Structure - Below 1st Floor Joists

Existing Floors and Roof

There are very few timber elements still in situ. The roof is mostly collapsed and only the north east quadrant is still standing and shall be preserved under the new glass canopy. Some floor also exists at both levels. This will be stabilised by prosthetic glass elements that splice into the timber elements. Any damage will either be made good by hidden repairs of reinforced where necessary.

Existing Walls

The masonry at Menokin consists of dressed stone, stone rubble, and bricks. The dressed stone forms the architectural components of the building such as the quoins, water table and belt course. The walls are load bearing and the structural behaviour of this arrangement is discussed in part 1 of this report. The outer walls are constructed from load bearing broken stone of irregular size, shape and texture. The rubble wall is protected by a lime render and the stone is held together with lime mortar, which behaves differently to Portland cement. All ruined rubble will not be restored, because it is impossible to tell where it originally came from. A significant effort has been made to restore dressed stone. The central masonry chimney stack was built from regular masonry units, which have not deteriorated as much as the natural stone wall.

Existing Foundations

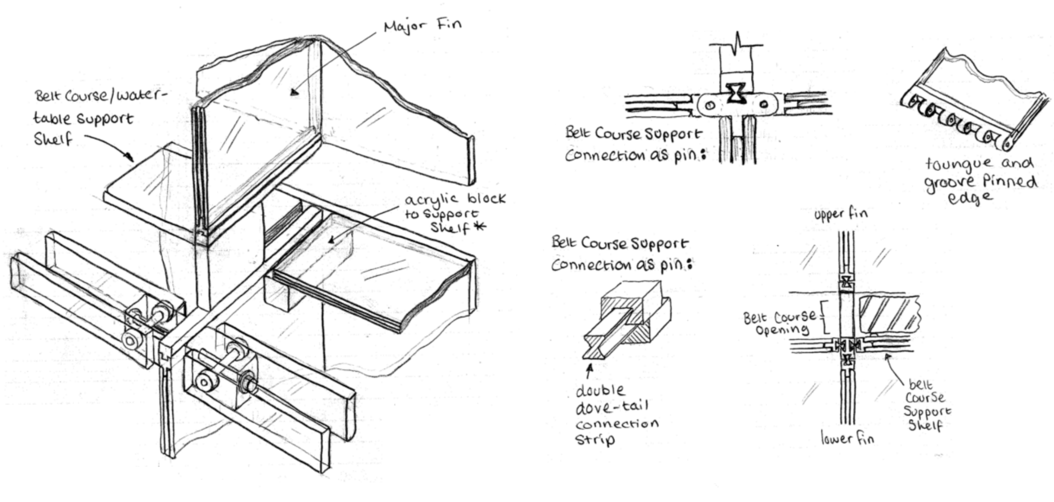
The foundations are in various states of ruin. Much is intact and safely preserved under the walls, other parts have been destroyed to below grade level and some of it is still to be uncovered. Work has been carried out to stabilise the foundations by realigning walls and replacing eroded lime mortar.

The external walls are currently supported on thick basement retaining walls which are founded 8 to 10 feet below grade. In places the wall has collapsed down to grade. The intention will be to construct the glass walls on the existing stone walls and to support the walls horizontally with glass at the ground floor level.

The glass walls will impose a smaller vertical load on the foundations than was experienced in the original stone load-bearing construction and we do not anticipate measurable differential settlement of the foundations. However, our design – using laminated structural glass that will not shatter even if some layers are breached and the joinery detailing that will allow the building to function much like a timber structure – will ensure the glass will not be bothered if there is foundation settlement. (Tim Macfarlane)

New Glass Walls

It is impractical to make the replacement glass walls as thick as the original masonry. A glass enclosure was considered, but it was thought too difficult to clean and it would probably end up requiring environmental control to avoid condensation. Enclosures are formed by the quoin fin arrangement and could be used as air-ducts to usefully avoid this problem. The wall envelope is composed of glass panels supported by glass fins whose depth matches the depth of the original wall. The fins are the primary support to the roof and floors, but the lateral stability of the building relies on the masonry chimney stack, which acts as a structural core. Lateral wind loads are transferred from the wall panels into the fins. The fins pass the load into the floors which form a stiff diaphragm bracing the walls against the core.



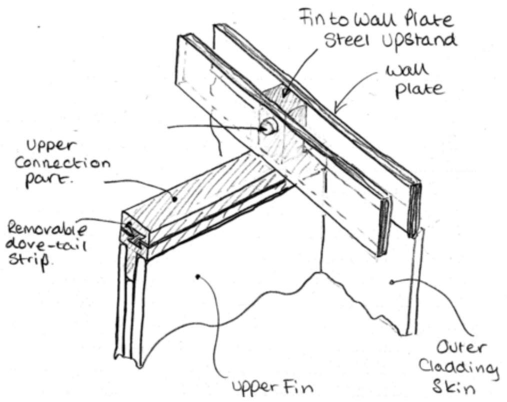
Early Sketch of a floor connection junction.

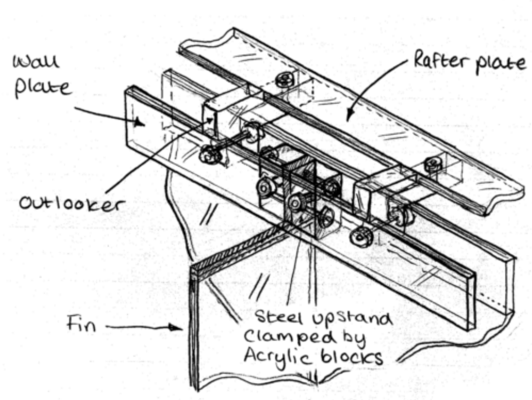
Floor Connection Junction

The floor connection junction is made complicated by the need to restore the belt course in its original position. The belt course passes through the glass fins removing a key part of the load path within the fin. The loads involved cannot be managed by glass or acrylic and so a strong connection part is required. Stainless steel is an obvious choice and it will have to be precision engineered, therefore it has been suggested that we use titanium. Titanium is not structurally necessary, but it may have other benefits. The floor connection junction is critical to the success of the project and a mock up will be required. The junction is not expressed in the link passageway design and I recommend that the arrangement is reorganised to include it.

Wall Plate Connection Junction

The original wall plate was a timber bearing member located upon the top of the wall. It housed the outlookers and truss and provided a level platform for the roof to bear on. Where the walls are represented by glass, a glass wall plate will span between the fins serving the same function as the original element.

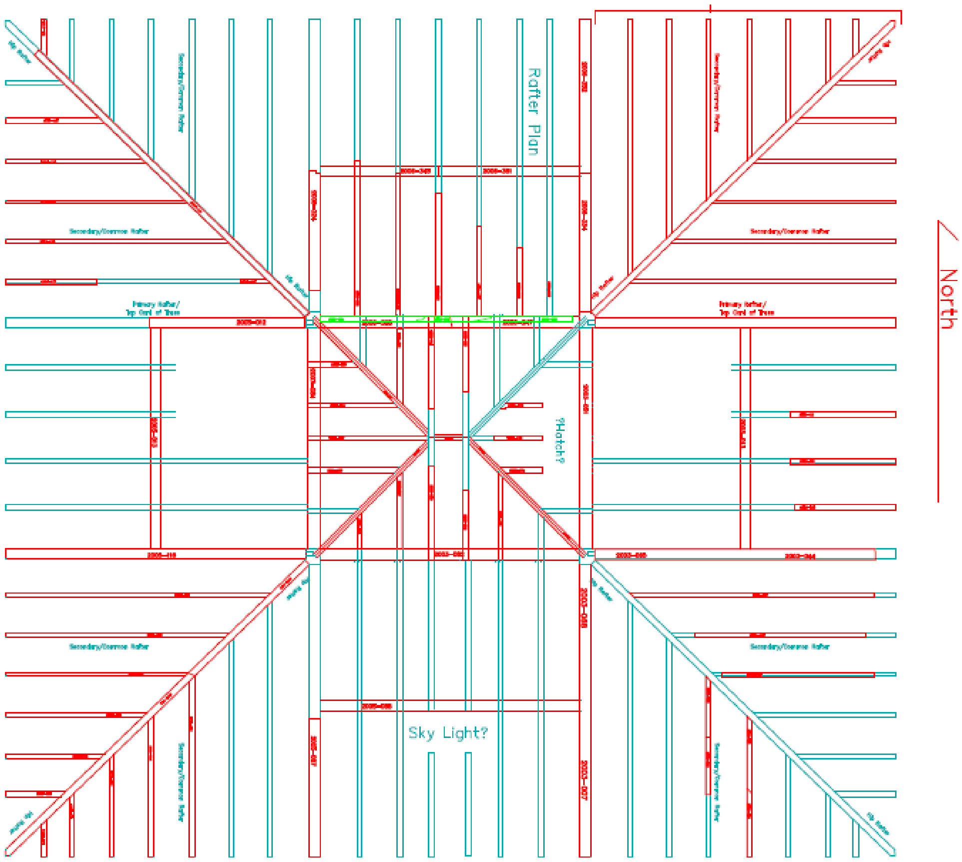




Wall Plate Connection Details

New Glass Roof and Restored Timbers

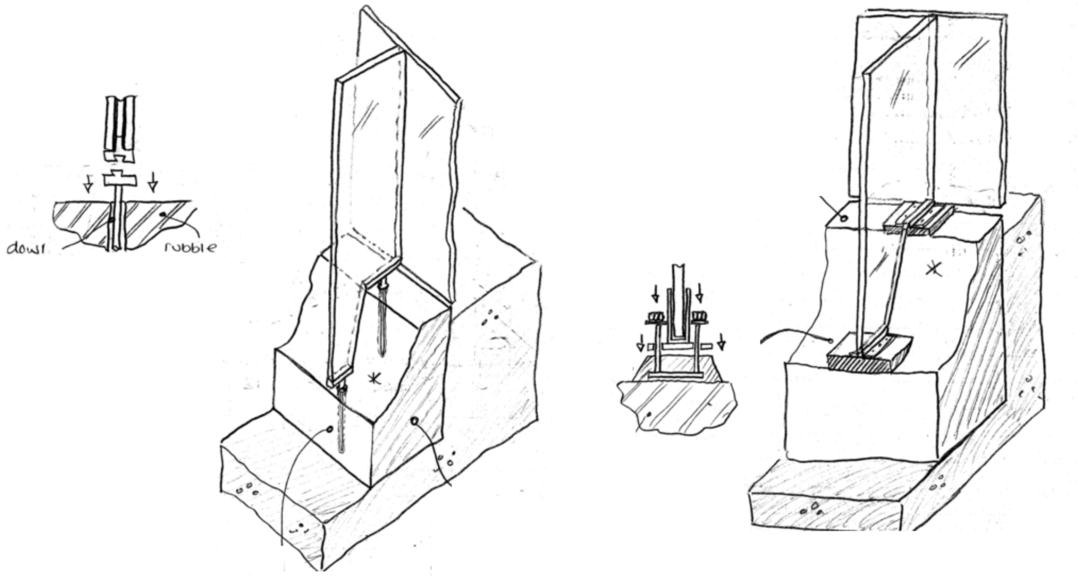
The new roof will be constructed out of any surviving timber elements and supported by new prosthetic elements. The new elements shall be constructed of glass and acrylic composites and these are described in more detail in part one. Splice details have been proposed to create a smooth junction between each material. The main load bearing timber truss elements exist in parts and it is not yet clear how far they can be restored and then expected to carry significant load. This is an interesting area that demands more study.



Roof Framing Plan Indicating the extent of Restorable Timber in Red

Fin Support

The fins will project out of the masonry and need to be held in place, but they do not have to transfer bending moment into the foundation. Therefore it would be wise to release the connection with a notional hinge, so that it cannot deliver twisting actions into the masonry. Two methods of connection have been proposed – see sketch below. The favoured approach is to build a removable concrete plinth on top the wall to bolt the connection detail to. This approach masks some original masonry under the plinth, but it prevents internal masonry form being damaged permanently. An alternative approach would be to embed support bars into the masonry and support the connection above the masonry. I prefer this detail because it is more elegant and clearly expressed, but it damages original material and therefore it has been abandoned.



Potential Fin to Masonry Connection Details