Historic England

The Structural Conservation of Ruins

Structural assessment of ruined masonry walls



Summary

Ruined masonry structures form a small but important part of our historic estate. The issues that a structural engineer may encounter when working on a ruined structure are quite different from those encountered when working on roofed buildings of any age. An engineer who is asked to provide advice on such structures should be familiar with historic methods of construction, primary mechanisms of decay and the philosophy of historic building conservation. Numerical analysis of ruins should be approached with caution and carefully considered, especially where modern design codes are used as the basis for assessment. Where required, structural repairs should be sensitive and designed to safeguard the significance of the structure.

This publication aims to provide consulting engineers who are not familiar with historic ruins with the tools necessary to make informed engineering decisions and specify sympathetic repairs. This publication does not aim to be an exhaustive design guide and does not cover every construction typology that may be encountered. The Historic England Practical Building Conservation series provide a comprehensive and practical reference for professionals involved in repairing historic buildings and should be referred to alongside this guidance for further information on mortars, stone and brick.

This document has been prepared by the Historic England Structural Engineering Team. This edition published by Historic England October 2026. All images © Historic England unless otherwise stated.

Please refer to this document as:

Historic England 2026 *The Structural Conservation of Ruins.* Swindon. Historic England.

HEAG333

HistoricEngland.org.uk/advice/technical-advice

Front cover: The conserved remains of St Nicholas' College in Wallingford Castle Meadows, Wallingford, Oxfordshire. © Historic England Archive.

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1. Introduction

Ruined structures form an important part of our architectural and cultural heritage. They reveal information on how past societies lived, as well as historic construction methods and typologies. Today, many of these structures are conserved, preserved and maintained as ruins to be visited (Figures 1 and 2).

A building can fall into ruin when it is no longer in use, by deliberate destruction or following a catastrophic event such as a fire. As the building decays it loses elements which provide protection such as roofs, windows and external finishes. The loss of structural elements and consequential altered load paths mean that structural issues arise that are different to those encountered in an in-use, roofed building.

This publication focuses on the structural defects which can occur in ruined, traditionally built, solid masonry walls, methods of surveying and assessment, and the design of suitable structural repairs. It is primarily aimed at structural engineers working on masonry ruins. Roofed buildings, retaining structures, arches and vaults are not covered in this document, although they may be present at ruined sites.

There are numerous examples where historic ruins have been successfully incorporated into new buildings. This demands a very careful and considered approach to balance the needs of building users while conserving significance. Such matters are beyond the scope of this publication.



Figure 1: Witley Court in Worcestershire, a Grade I listed country house damaged by fire in 1937 and preserved as a ruin.



Figure 2: The church at Lindisfarne Priory, Holy Island, Northumberland, was built from red sandstone between the 11th and 12th centuries and ruined after it fell into disuse following the dissolution of the monasteries.

In Europe, interest in ruins as a place to visit and appreciate re-emerged in the 18th century. Today these structures may appear to have been left untouched since they were abandoned, but the majority have been stripped of invasive vegetation, altered and repaired several times.

Typical masonry construction

The type of construction and materials used can affect the behaviour of a wall and the type of defects that are likely to occur. Further information can be found in sections 2 and 5.

The following materials are commonly found in ruined structures:

Stone

The composite stone wall is constructed from two facings of mortared masonry encapsulating a rubble stone core set in mortar (Figure 5). This construction was introduced by the Romans and saw a resurgence from the medieval period onwards (Figure 3). Later, walls became thinner with larger openings. In some cases, brick or rubble stonework was used for the inner skin with ashlar or finely finished stone on the outer skin.

The building stones used around the UK depend largely on what was available locally, and their distribution reflects the underlying geology. For some high-status buildings, particular types of stone

might be brought in from further afield. The physical and chemical composition of building stones varies widely and affects their properties and weathering behaviour. Further information on the distribution, sources and use of building stones in England can be found in Historic England's *Building Stones Database for England*.

Flint was a commonly used building material in some parts of the UK (Figure 4). Flints are almost entirely composed of silica and form as small, irregular nodules within chalk and limestone. It is an extremely hard material, difficult to cut and is typically minimally worked. Flint facework is produced by 'knapping' – striking at an angle with another flint to shear the flint along natural lines of weakness.

Flint walls are often used in composite construction, featuring facings of coursed flint with quoins of worked stone and a core of unworked flint and other field stones, along with a substantial amount of mortar.

It was common for traditionally built stone masonry structures to be finished with lime-based finishes such as plasters, renders and limewashes. This provided additional protection against weathering.



Figure 3: Totnes Castle, Devon, featuring a stone keep constructed from limestone and sandstone.



Figure 4: West Tower of Bishops Waltham Palace, Hampshire, constructed from flint.

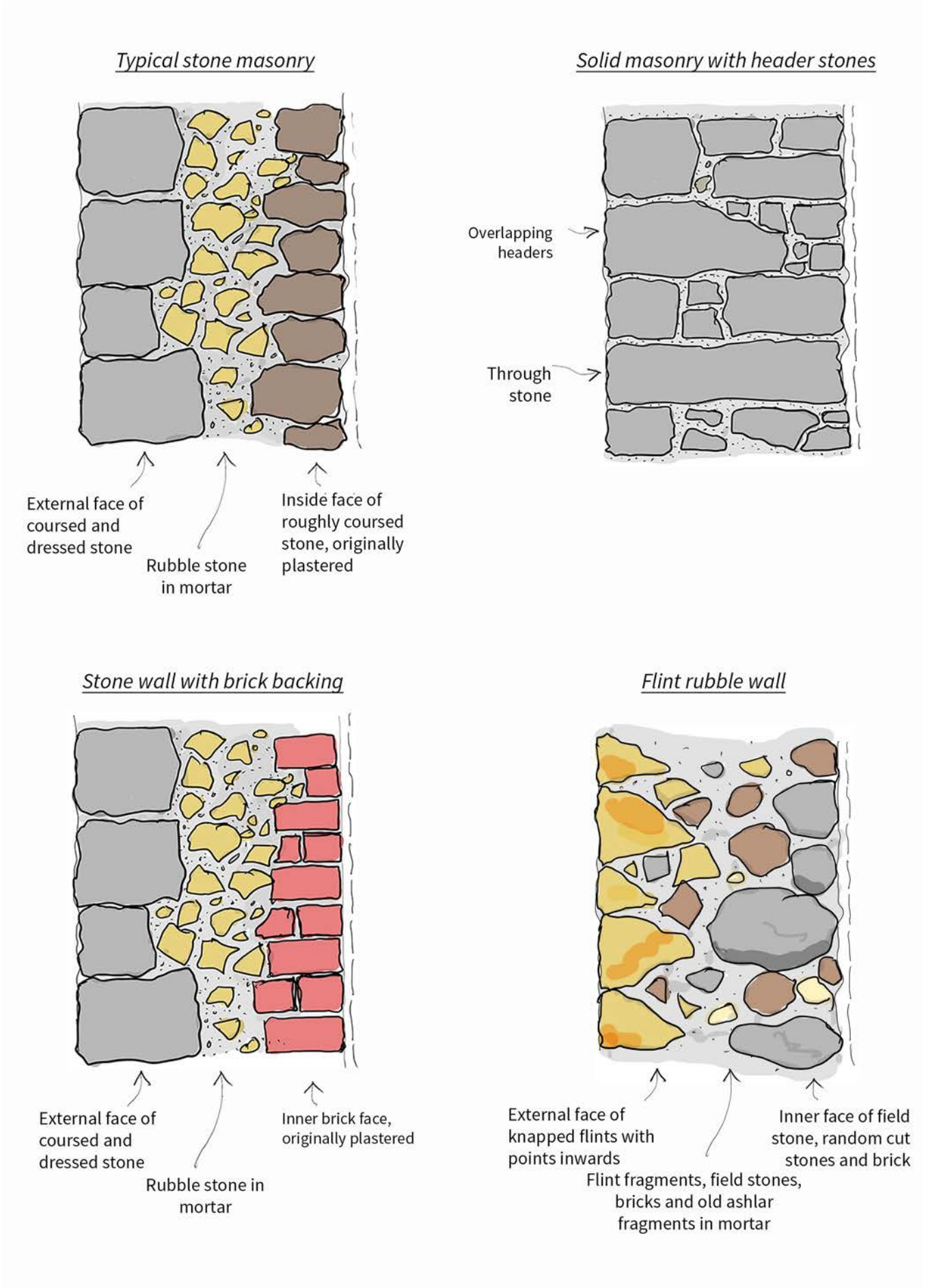


Figure 5: Examples of solid masonry wall construction.

Brick

Brickmaking was introduced to Britain by the Romans (Figure 7), but it was not until the 12th century that it resumed on a significant scale. Its popularity grew throughout the Tudor period and became increasingly common following the Great Fire of London in 1666 and the subsequent Royal Proclamation (Figure 8). Cavity wall systems began to be developed in the late 19th century. Iron or mild steel ties or stoneware bonding bricks were used to tie together the two brick leaves in early cavity walls. Brickwork was also used to form quoins and dressings around openings in rubble and flint masonry.

Historic solid brick wall construction may not be fully bonded, unlike modern solid wall construction. There may be only a few headers tying the bricks together or none at all. This can lead to one face bulging or detaching from the wall (Figure 6). From the late 17th century, it became common practice to build the exterior face of a wall as a separate, half-brick (112mm) thick leaf to economise on the use of high-quality facing bricks.

Historic bricks can vary in dimension and material properties from modern bricks. Some bricks were very soft to allow shaping by ‘rubbing’ or carving.



Figure 6: A bulging brick wall face with few ties between the inner and outer skin.



Figure 7: Brick lacing courses within a flint and sandstone Roman wall at Pevensey Castle, East Sussex.



Figure 8: Kirby Muxloe Castle Gatehouse, Leicestershire, constructed from brick.

Mortar

Mortars are formed by mixing a binder with aggregates and, in some cases, additives (admixtures). Until the late 18th and early 19th century, most mortars used in Britain were based on raw materials derived from local geological resources. Lime and/or earth (clay) were the principal binders. After this date, cementitious mortars became more widely used. Lime and earth-based mortars have different material properties and behaviours from cement-based mortars (refer to section 5). The retrospective use of cementitious mortars on ruins originally built with lime or earth mortar can accelerate decay where the repair mortar is incompatible with the existing masonry or unsuitable for its location and function (refer to section 2 for more detail).

Aggregates used in historic mortars included subsoil, sand, and crushed stone or brick. Additives change the properties of mortar, for example by modifying setting times or allowing the mortar to set in the presence of water. Pozzolans including trass (volcanic ash) and finely crushed brick are examples of additives sometimes found in historic mortars.

Earth mortars

Earth mortars are common in historic construction; they are generally found in the core of the wall with the faces pointed in lime mortar or rendered. Earth mortars are particularly vulnerable to moisture and have negligible tensile strength.

Lime mortars

There are many types of lime mortar, but they can be broadly categorised by binder type as air limes, hydraulic limes and gauged limes.

Air limes or non-hydraulic limes are produced by calcining (burning) pure limestone (calcium carbonate) free from clay or other impurities. The resulting quicklime (calcium oxide) is then ‘slaked’ by adding water to produce slaked or hydrated lime (calcium hydroxide), which may take the form of putty or dry powder, depending on the amount of water used. Air limes harden initially by drying, then by a chemical reaction with carbon dioxide in the air, in a process known as carbonation. Air lime mortars harden slowly and remain somewhat deformable (ductile under compression). They are weaker and less durable than hydraulic lime and guaged mortars and will not harden if immersed in water. Evidence suggests that ‘hot mixing’ (i.e. combining quicklime and aggregates before slaking) was a common method of preparing lime mortars in traditional construction.

Hydraulic limes are produced from limestones that naturally contain silica and alumina, principally derived from clay minerals, together with other impurities such as iron and magnesium. During calcination (burning), some of the calcium oxide reacts with the silica and alumina to form calcium silicates and aluminates. The remaining uncombined calcium oxide is referred to as ‘free lime’. The hydraulic quicklime is normally slaked with a controlled quantity of water, sufficient to hydrate the remaining free calcium oxide and produce a dry hydrated lime powder, but not sufficient to initiate significant hydration of the silicates and aluminates. When water is added during mortar mixing, the calcium silicates and aluminates hydrate to form solid, interlocking crystals of calcium silicate hydrate and calcium aluminate hydrate, producing the initial hydraulic set. Further strength develops through continued hydration and the gradual carbonation of the remaining free lime. Because setting occurs through hydration rather than carbonation alone, hydraulic limes are able to set under water (hence the term ‘hydraulic’) or in saturated masonry.

Hydraulic limes are also generally stronger and more frost resistant than air limes, although performance depends on the mortar, its maturity, moisture content and exposure. These properties are influenced by the degree of hydraulicity, which depends on the reactive clay content of the

limestone used as the raw material. Higher clay contents result in a greater proportion of hydraulic compounds and a lower proportion of free lime, while lower clay contents result in a less hydraulic binder with more free lime available for carbonation. The remaining uncombined 'free lime' acts in the same way as air lime and hardens gradually by carbonation, contributing to the long-term development of strength.

During the 19th century, hydraulic limes were classified as 'feebly', 'moderately' and 'eminently' hydraulic according to their speed of set under water. Natural ('Roman') cements are closely related hydraulic binders produced from limestones containing considerable quantities of reactive clays.

Modern pre-bagged Natural Hydraulic Lime (NHL) is supplied in three strength categories: NHL2, NHL3.5 and NHL5. The numbers represent the value (in N/mm^2) of the 28-day cube strength of mortar below which 5% of all possible test results would be expected to fall. These categories are not equivalent to the 19th- century classifications. The strength ranges of NHL2, NHL3.5 and NHL5 mortars may overlap in practice, depending on factors such as mix design, aggregate selection, curing conditions and age, and NHL mortars continue to gain strength long after 28 days, such that their final strength may be significantly greater than the stated values. Modern NHL mortars are not always compatible with historic buildings because they may be significantly stronger and less porous and permeable than the mortars used in the original construction, depending on the mix design, aggregate and curing conditions.

Gauged limes are lime mortars which have been 'gauged' (mixed) with small amounts of Portland cement or pozzolans, such as ash or crushed bricks, to improve setting time, increase strength or improve durability. Depending on the type and proportion of the addition, this may reduce permeability and flexibility. The presence of hydraulic or pozzolanic additions (such as volcanic ash, crushed brick, trass, or pulverised fuel ash) enables the mortar to set in the presence of water. The rate of setting is influenced by the specific properties and quantity of the addition used. These properties may be desirable in more exposed locations. Gauged lime mortars were commonly used in the 20th century for bricklaying and are found widely in 20th century repairs to monuments. Their composition and proportions varied according to the materials available, the required strength and the conditions of exposure.

The mix proportions varied according to strength requirement and exposure, and a typical mix used in brickwork historically was 1:1:6 ratio of cement: air lime: aggregate.

Other structural materials

Masonry walls often contain fixings and elements comprising other materials, such as:

- Bonding timbers: Longitudinal timbers built into brick courses, found in late 17th to mid-19th century walls. Their primary purpose was to hold brickwork together while the lime mortar gained strength
- Lintels above doors and windows: typically timber, although precast concrete or steel can be found in later buildings
- Iron or bronze cramps (sometimes set in lead) to provide restraint or fix stones units together, and fixtures such as pintles (Figure 9)
- Slate damp proof courses
- Iron, steel or ceramic cavity wall ties (Figure 10).





Figure 9 (left): An iron cramp used in stone construction. Corrosion has caused the stone's surface to spall.
Figure 10 (right): An iron wall tie used in early cavity wall construction.

Foundations

Geotechnical engineering is a modern discipline, and in the past ground conditions would not have been analysed in the same way as they are today. Historic buildings were typically built on shallow, spread foundations and it was common for settlement due to consolidation of soft ground to occur. This settlement could occur seasonally over many years, or shortly after construction. Rotation of lower masonry lifts due to settlement during construction was often rectified the following season as the next lift was built on top (Figure 12). Timber piled foundations were occasionally used as an alternative to shallow spread foundations in very poor ground, for example adjacent to a river or watercourse (Figure 11).

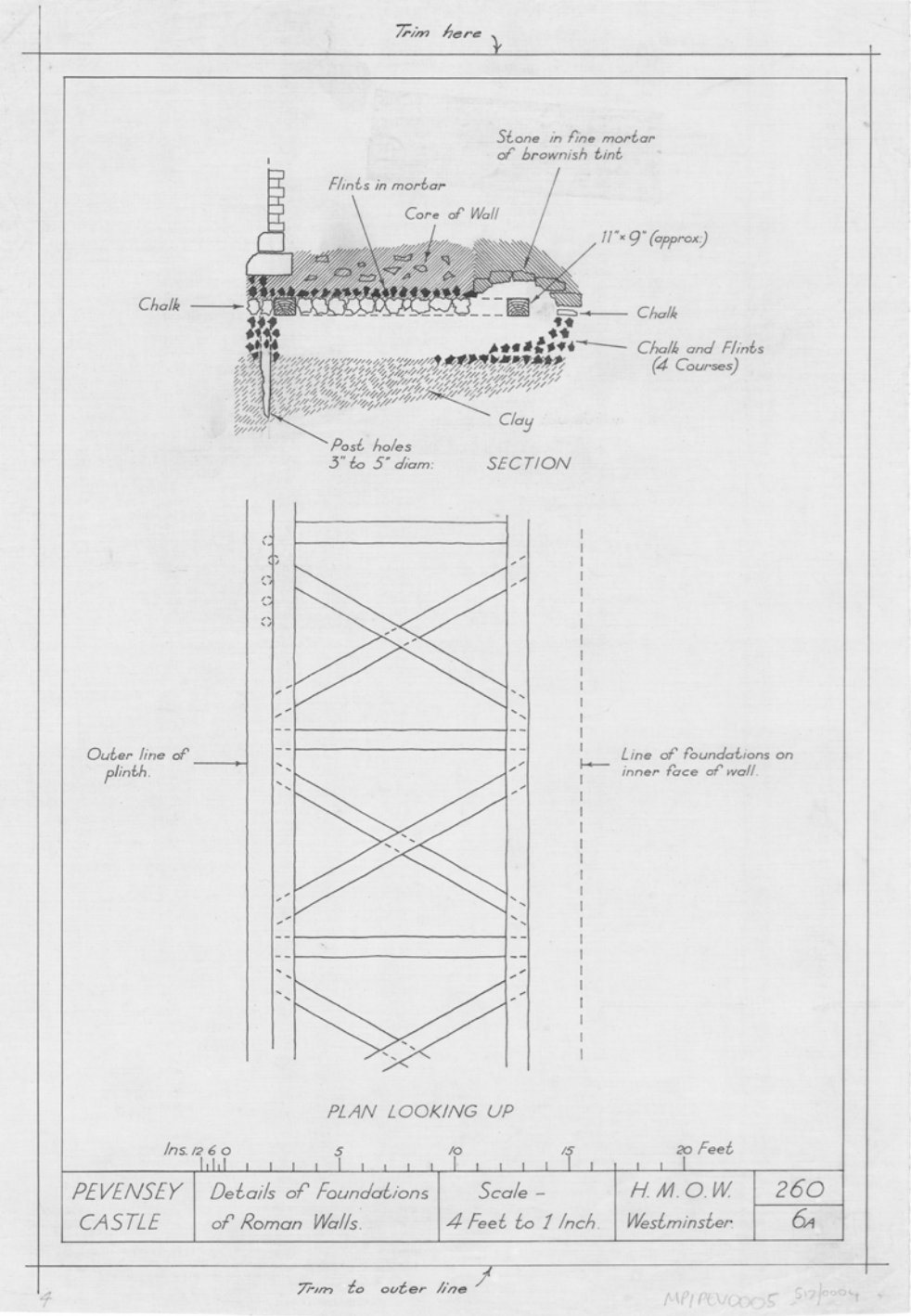


Figure 11: Drawing of the timber-piled foundations of the outer bailey wall at Pevensey Castle, East Sussex.



Figure 12: The tower of Temple Church, Bristol began leaning during construction. The lean was rectified during the construction of the upper levels.

Historic Repairs and Alterations

It is important for an engineer working on ruined structures to understand both the original construction and subsequent interventions that may have been made, as these can impact how the structures behave and the defects that may develop.

Typical repairs carried out on ruins may include:

- Rebuilding of unstable sections and wall tops, often using cementitious mortar (Figure 16);
- Installation of soft capping on wall heads (typically in the late 20th century);
- Construction of stone or brick buttresses;
- Concrete underpinning (Figures 14, 15);
- Re-pointing, often using cementitious mortar;
- Grouting, often using cementitious mortar;
- Introduction of stainless steel or phosphor bronze bars to support overhanging masonry;
- Jacking leaning walls back to vertical;
- Reinforced concrete stitching beams across cracks, hidden within the masonry wall;
- Reconstruction of collapsed sections of wall;
- Reinforced concrete beams above openings or ring beams around wall tops, hidden within the depth masonry wall (Figure 17).

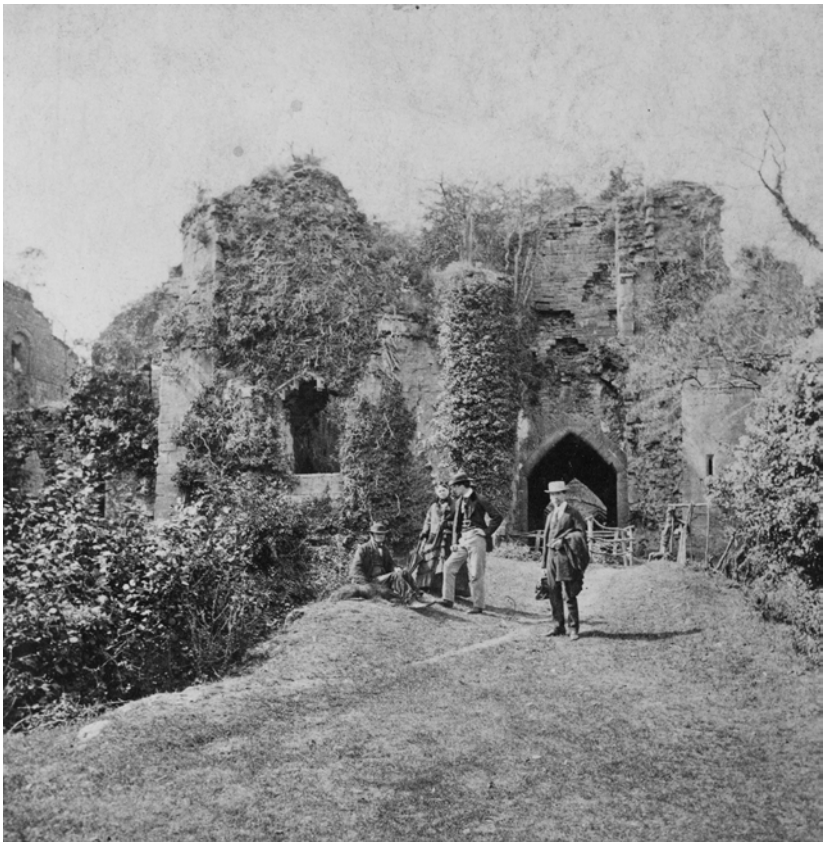


Figure 13: Photograph of Goodrich Castle in 1914 prior to a programme of vegetation removal and repairs.

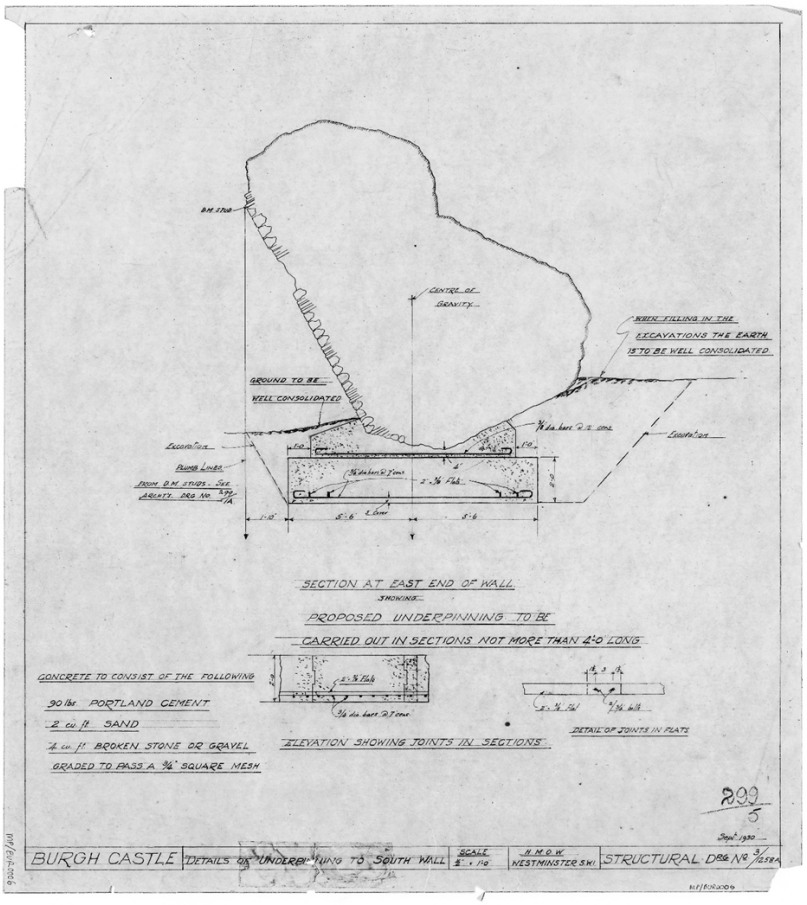


Figure 14: The underpinning of a section of wall at Burgh Castle, Norfolk, in 1930.





Figure 15 (left): Installation of a reinforced concrete ring beam at the base of Cow Tower, Norwich, in 1954.
Figure 16 (right): Consolidation of wall tops of Cow Tower, Norwich in 1954.

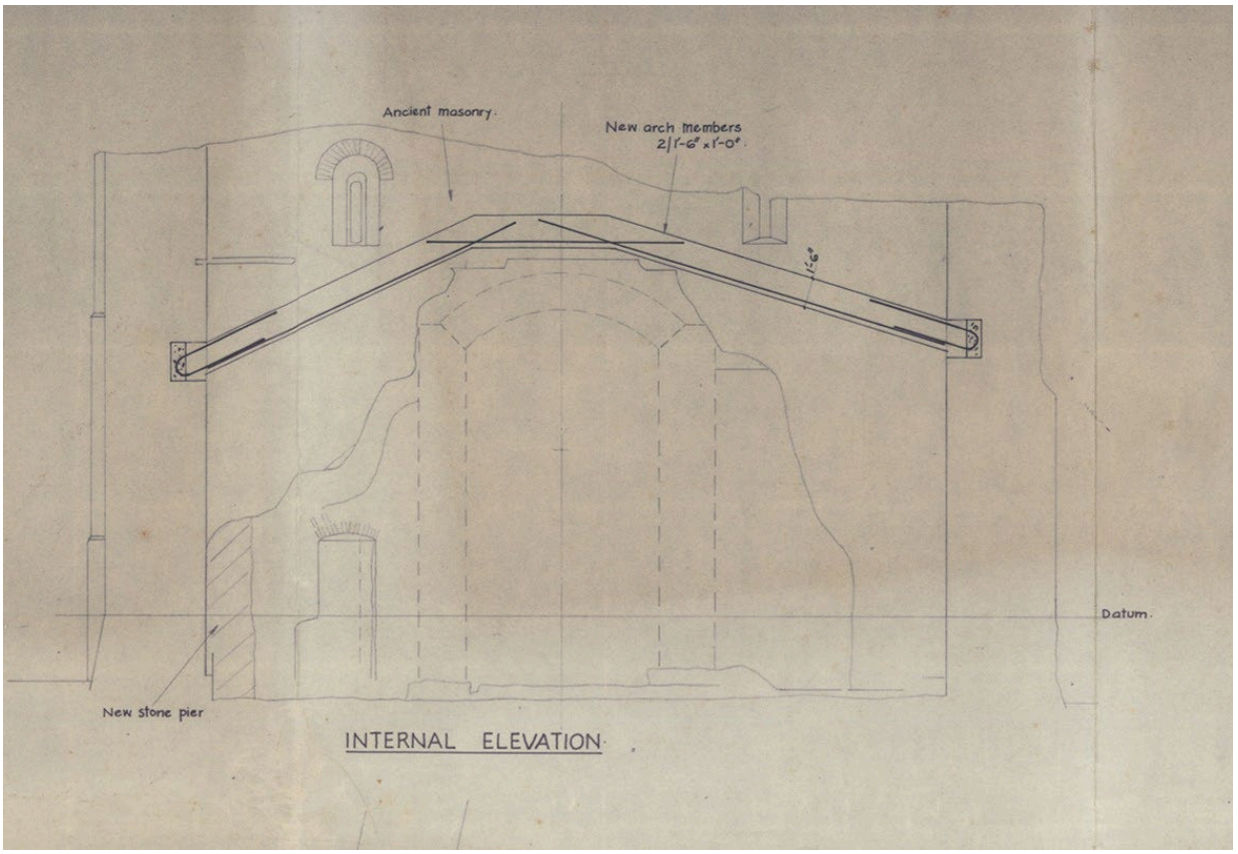


Figure 17: Drawing showing proposed installation of a reinforced concrete beam above an opening in the keep at Wolvesey Castle, Hampshire, in 1966.

2. Deterioration and failure

Ruins are, by definition, structures which have already undergone some deterioration or failure. They are no longer complete and lack the basic environmental protection of a roof or windows, which are afforded to buildings in use. This leaves them vulnerable to further damage and deterioration.

It is useful to understand why the structure became a ruin in the first place. There may be a combination of factors such as:

- **Change in political or religious culture** for example, the dissolution of the monasteries led to the ruination of a huge number of abbeys and priories (Figure 19).
- **War, conflict or disaster** for example, many fortifications, some in use since the medieval period, were damaged or destroyed during the Civil War and World War II (Figure 18).
- **Redundancy** buildings which no longer had a viable use and were left to deteriorate slowly over time. This could be due to de-population through pandemics or social changes; the buildings perhaps became too expensive to maintain, no longer met requirements, or the activity they were used for ceased.
- **Stone robbing or reuse** stone was often taken from redundant structures to save the labour-intensive job of sourcing new stone for building.



Figure 18: Bridgnorth Castle in Shropshire was ruined during the English Civil War.



Figure 19: Furness Abbey in Cumbria underwent extensive dismantling for stone repurposing after dissolution in 1537.

It is important to understand the causes of deterioration, as this may have a bearing on the structural condition.

Ongoing decay and deterioration can often be attributed to a combination of environmental, human and structural causes including:

- water ingress;
- wind;
- vegetation;
- load path alteration;
- ground conditions;
- climate change;
- previous interventions;
- lack of maintenance;
- vandalism;
- pollution;
- poor original construction/detailing;
- animals.

Water ingress

This is one of the leading causes of ongoing deterioration in masonry ruins. Originally, a wall would have been protected by a roof or copings, appropriate mortar pointing or a coat of render or limewash (Figure 20). Without these protective elements, water can enter freely, percolating downwards and washing out fines and binder material which can erode the structural integrity of the wall. Many historic walls were built with earth mortar cores that are susceptible to softening and erosion when exposed to prolonged moisture ingress. Washout of the core material often leads to the formation of voids, reductions in internal bonding, face separation and local instability.

Wet walls also decay by soluble salt precipitation. As salts crystalise at drying fronts they expand and break up the face of the stone. Repeated cycles of recrystallisation can cause extensive damage. This is particularly noticeable at the bottom of walls in the gritting zone along roads, and on the leeward (sheltered) face of coastal ruins.

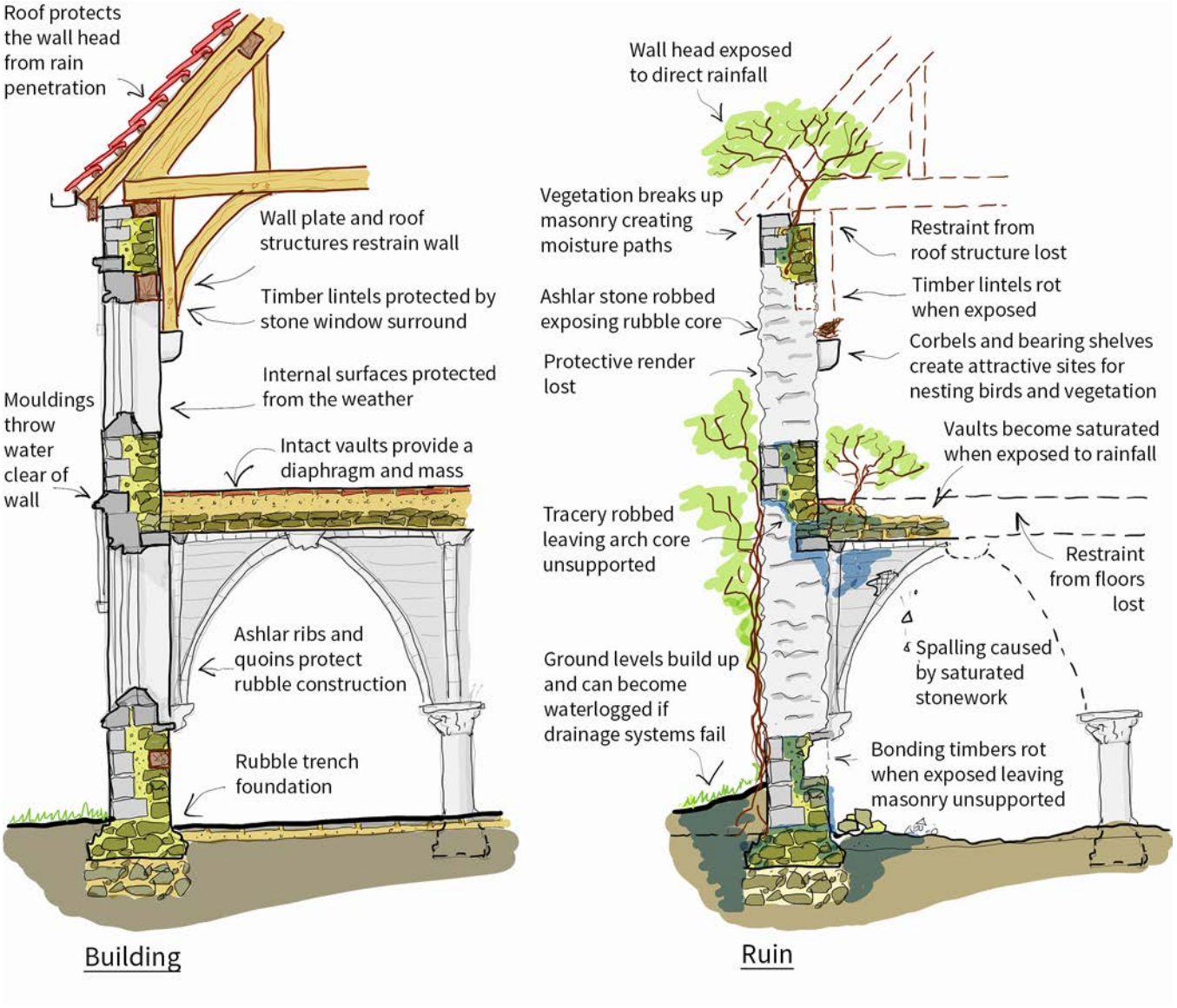


Figure 20: Sketch comparing different levels of exposure to the elements experienced by a building in use and a ruin.

In the winter, alternate cycles of freezing and thawing of water in a saturated wall can lead to the displacement of masonry, potentially leading to collapse (Figure 21). Freeze-thaw cycles can also cause progressive damage to the surfaces of masonry units and mortar joints.

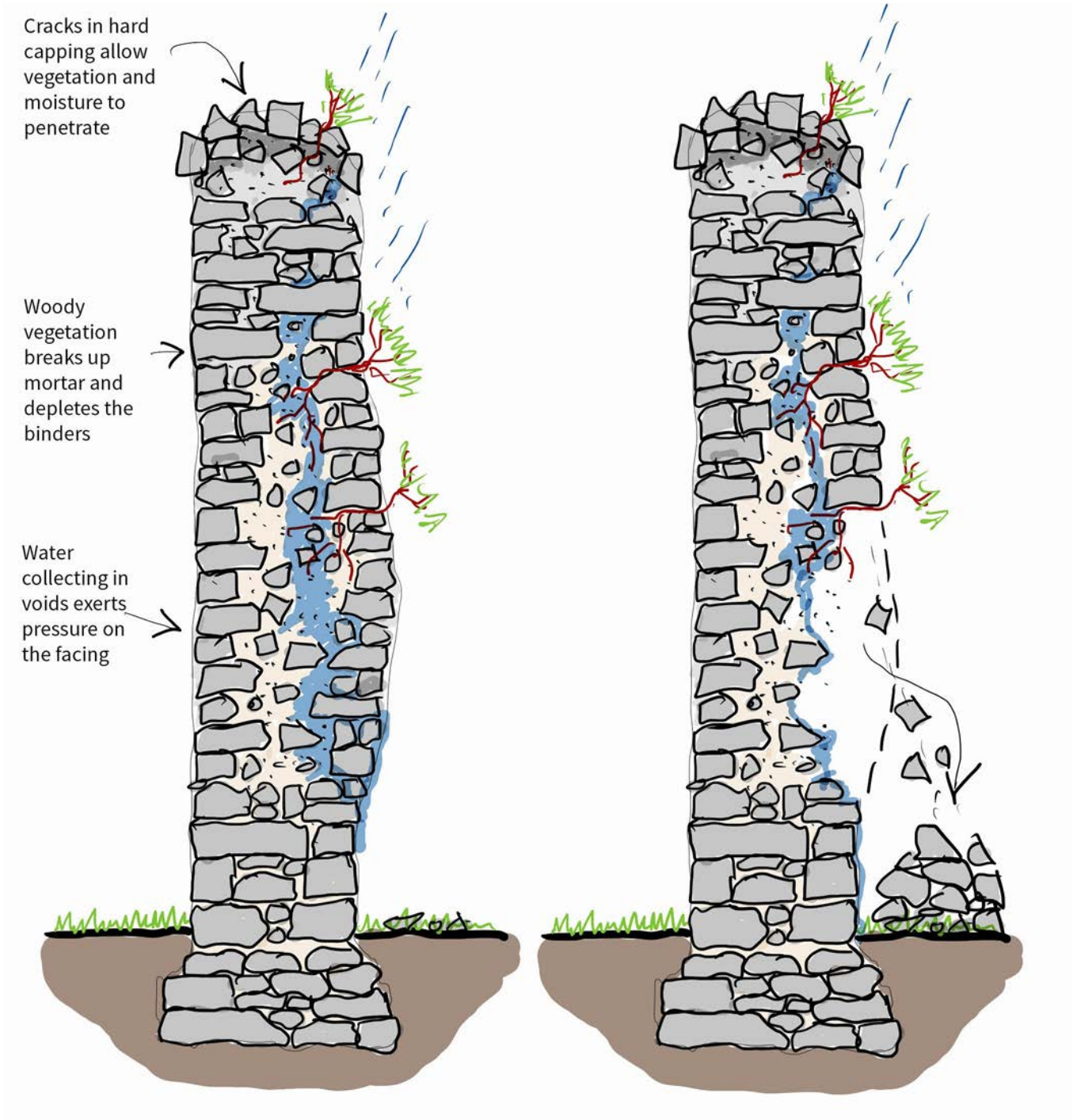


Figure 21: Sketch showing how water ingress can lead to the wall face bulging and ultimately collapsing.

Wind

Localised wind effects can cause accelerated salt-crystallisation erosion of stonework, especially on exposed sites or near the coast. The geometry of the structure may create vortices which accelerate erosion in a small area. Larger surfaces can be affected by high winds, especially where wind is funnelled through openings. Porous and soft stones are particularly vulnerable (Figure 22).

This type of erosion mainly affects the surface of the stone, but in some cases the erosion will be severe enough to undermine and destabilise the stonework above and can be considered as causing structural defects.

Wind loading can also be a direct threat to the stability of tall, slender and fragile ruins.



Figure 22: Wind driven erosion (alveolar decay) to stonework at Goodrich Castle in Herefordshire.

Vegetation

Wet masonry can often be identified by the presence of vegetation (Figure 23). Seeds may germinate in voids, open joints and on ledges. Woody plants such as red valerian (*Centranthus ruber*) can cause significant damage as their deep root systems can displace masonry.

It is not unusual on poorly maintained ruins to see mature trees growing from ledges and wall heads, usually surrounded by displaced stones or localised areas of collapse.



Figure 23: Established vegetation to the wall head at Wigmore Castle, Herefordshire.

Established vegetation may lead to a build-up of organic matter on top of or within the wall. This in turn may create a more favourable environment for new vegetation to grow, which can accelerate a cyclical process of deterioration. Woody plants growing within or close to a wall may also cause damage to the fabric of the building as they sway in the wind.

Woody stems plants or trees growing on, in or against a wall need to be identified and their growth assessed to determine if they require management, to prevent any damage or associated jacking force of the roots (Figures 24, 25 and 26).



Figure 24: Vegetation cut back at wall face and treated.



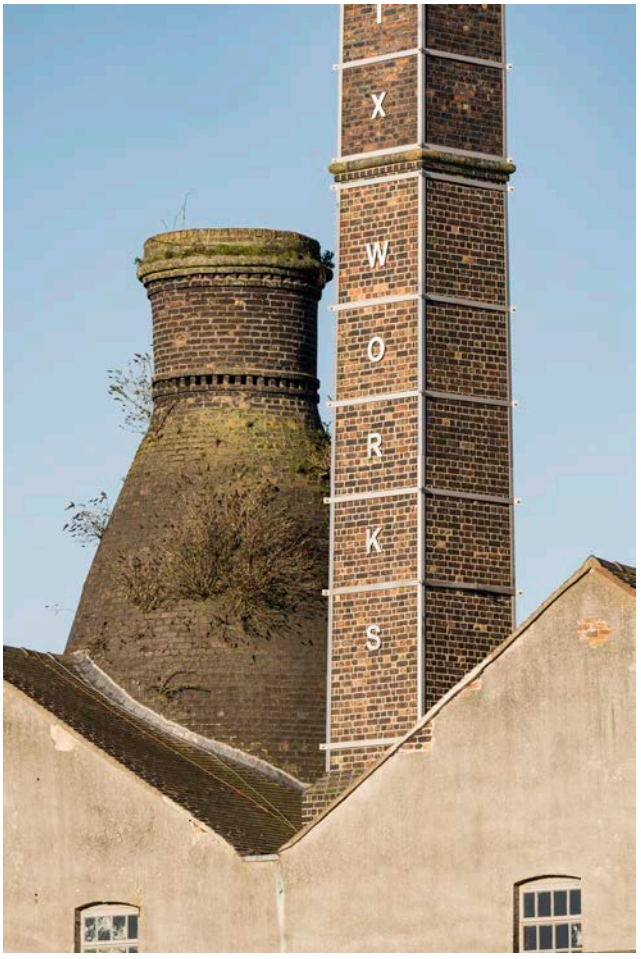


Figure 25 (left): A ruinous building with a tree growing inside it.
Figure 26 (right): Vegetation rooted in a bottle kiln.

Load path alteration

Once there is deterioration or material loss to a masonry ruin, load paths may be changed. Typical examples would be the loss of a supporting return wall, the loss of an arch or lintel, or the collapse of a floor or roof structure (Figures 27 and 28). This leads to redistribution of loads onto other parts of the structure, or previously supported areas losing lateral or vertical support.



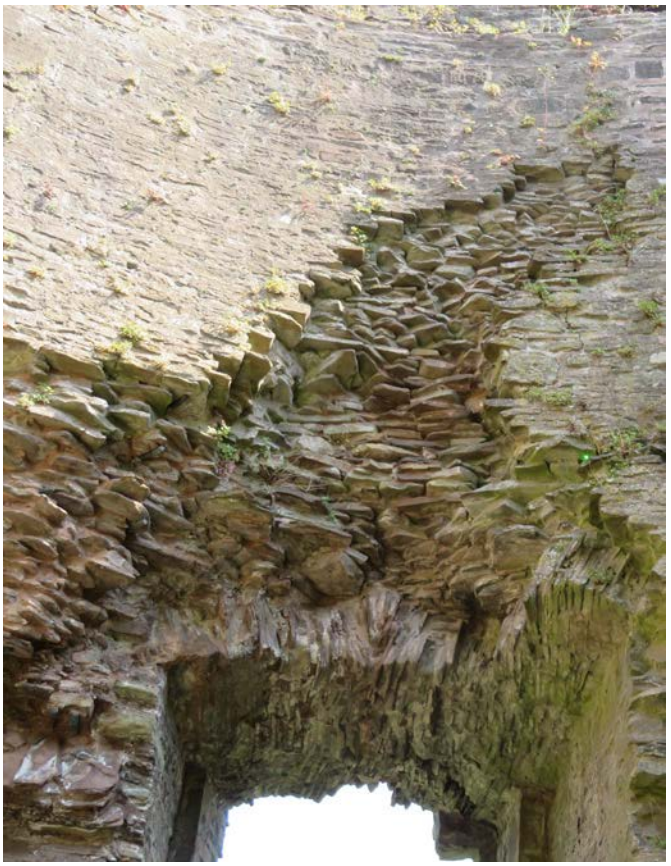


Figure 27 (left): Loss of masonry over openings due to loss of supporting lintels at Goodrich Castle, Herefordshire.
Figure 28 (right): Loss of masonry over failed stone arch at Longtown Castle, Herefordshire.

Once support is lost, or areas become subject to additional destabilising loads, the utilisation ratio of the remaining structure becomes greater. The vulnerability of these areas to further movement and deterioration increases.

Ground conditions

Ground levels rise or fall around a masonry ruin for many reasons. It may be the simple build-up of organic matter over the years, presence of tree roots, or areas of collapsed masonry (Figure 30). Ground levels may fall due to intentional removal of spoil, for example when cutting tracks or paths, or can be caused by burrowing animals or soil erosion (Figure 29).



Figure 29: Undermining of a wall base by badger activity.



Figure 30: Build-up of collapsed masonry behind a wall can increase moisture content within the masonry and increase lateral pressure on the wall.

Where ground levels increase on one side of a wall, lateral pressure will be exerted on the structure. Damp ground in direct contact with the wall will also allow water ingress and increase moisture levels. Where ground levels fall, the base of the wall risks being undermined.

The structure can also be affected by changes in the water table and moisture content of the ground, especially in shrinkable soils. Trees influence the uptake of water in the ground; when they grow, the surrounding ground may shrink as the tree takes up water from the soil. Similarly, when they die or are felled, the ground may swell as water is no longer absorbed by the tree. Other factors which may influence the water table are: failure of drainage systems, intensive agriculture and development in adjacent areas. Nearby structures with shallow foundations could be at risk of cracking or instability.

Saturation of the ground may lead to softening of the sub-soils and a reduction in bearing capacity. In some cases, the bearing capacity may become lower than the applied load and settlement can occur as a result.

Timber piles generally have a long design life as long as they remain fully submerged. However, if the level of the water table drops and the piles dry out or if they are subjected to cyclical wetting and drying, they will decay rapidly which may cause differential settlements and/or localised loss of support.

Climate change

Climate change is leading to more extreme weather, including longer periods of dry weather and more severe storm events with higher levels of rainfall within short periods of time. This can affect masonry ruins both above and below the ground.

More intense and frequent weather events may exacerbate existing defects, such as increased water ingress through cracks and open joints, accelerating erosion of the wall and core, and making collapse and loss more likely (Figure 31).



Figure 31: Collapse of an internal face of masonry with earth mortar core at Chichele College, Northamptonshire.

Higher rainfall and wind loading directly increase the rate of weathering and erosion of ruins, especially those which are not well maintained. Erosion and weathering can also affect the ground and lead to slope instability. Changes to weather patterns can affect the growing patterns of vegetation which may cause further deterioration of the ruin. On coastal sites, more frequent storm events may accelerate coastal erosion and result in the loss of vulnerable masonry ruins (Figure 32).



Figure 32: An aerial view of Reculver Towers in Kent, a site at risk of loss due to climate change.

Previous interventions

Previous alterations and interventions may have a detrimental effect on masonry ruins. A common example is 20th century re-pointing with incompatible mortars which can lead to the accelerated deterioration of the masonry.

As cementitious mortars are generally more brittle than lime mortars, they are more susceptible to cracking under thermal and structural movement, creating an entry point for water into the wall core. Cementitious mortars are also typically much less permeable than lime and earth mortars, so, where used inappropriately, any absorbed moisture may become trapped within the wall, causing deterioration to both the core and the masonry face. The potential build-up of weight in the core due to water and saturated masonry can lead to bulging facework - the cement 'straight jacket' effect.

Cement grouting to fill voids in the cores of ruined walls was a common method of consolidation in the early 20th century. It can trap water within the wall and is often a source of harmful soluble salts.

Twentieth century alterations might also include re-profiling of the wall head, or alterations to enable access, both of which may alter loadings on the ruin, affect water runoff or lead to accumulation of vegetation.

Decay of structural materials within the wall from either previous repairs or as part of the original construction can cause ongoing damage. For example, expansive corrosion of iron cramps or ties will crack surrounding masonry units and decay of longitudinal timbers can result in brickwork above being dislodged or collapsing.

Lack of maintenance

Ruins can deteriorate even when maintained, especially if the stone is soft or particularly vulnerable to wetting cycles or atmospheric conditions. However, inadequate maintenance will accelerate deterioration and exacerbate any existing issues, resulting in an avoidable loss of historical fabric (Figure 34).



Figure 33: Sawley Abbey in Lancashire is a managed site with poor quality stone, which continues to deteriorate.



Figure 34: Lack of maintenance leading to accelerated weathering and deterioration.

3. Initial assessment

Why carry out a structural appraisal?

The purpose of a structural appraisal is to review the condition of the wall and assess any risk of deterioration, loss or failure.

Structural surveys of ruins are normally carried out by structural engineers, appointed by the building owner or manager. The structural survey may be commissioned in response to recommendations made during regular planned surveys by a surveyor or architect.

A structural engineer may also be involved if there are safety concerns, for example, noticeable movement or cracking, or a partial collapse.

The experience and competency of the structural engineer appraising or specifying repairs to a ruin should be proportionate to the significance of the structure, as a minimum the engineer should be a chartered structural engineer or civil engineer who specialises in structures. Chartered engineers are bound by their institutions code of professional conduct, to undertake only those tasks and accept only those appointments for which they are competent.

More significant heritage assets require an engineer with experience and competency of both ruins and conservation, the latter is most easily demonstrated by also being on the Conservation Accreditation Register for Engineers (CARE)[36].

How to carry out a structural appraisal

Before deciding whether an intervention is needed and how a structural repair should be approached, the structural engineer needs to know as much as possible about the construction, the condition of the structure, and any historic changes made to the ruin. This can be carried out as a collaborative exercise with other professions working on the project.

Each site is different and will require an individual approach. This section sets out a series of steps which may serve as a useful starting point.

1. Desktop study

A desktop study can help the engineer understand how the form and condition of a ruin has changed over time and what repairs may have been carried out. It can show whether cracks and other defects are recent or longstanding.

Information on changes to ground levels, tree locations and geological conditions should be gathered where available.

Sources of information include:

- historic survey drawings, photographs and maps;
- photographs and artistic drawings;
- drawings and specifications for earlier repair works;
- previous condition surveys, monitoring and structural reports;
- conservation management plans and heritage statements;
- records of archaeological investigations;
- guidebooks and history books;
- borehole records and geological maps;
- previous and current tree surveys;
- hydrological surveys.

Some of this information may be available from the ruin owner or custodian. Other material can be found in local and national archives and online resources. These include:

- Historic England, Historic Environment Scotland and Cadw archives;
- Historic Environment Record of Northern Ireland;
- Historic England's Research Report Database;
- Historic England's Historic Environment Records;
- The National Archives;
- Royal Commission on the Ancient and Historical Monuments of Wales;
- Archwilio: Historic Environment Record of Wales;
- British Geological Society's Geology Viewer.



Figure 35: Archive photo showing a view of the garderobe in the curtain wall at Eynsford Castle in Kent before works carried out in the 1950s.



Figure 36: Archive photo showing a view of the garderobe in the curtain wall at Eynsford Castle in Kent after works carried out in the 1950s.

2. Initial visual survey

An initial site visit should be carried out to gain an understanding of the condition of the ruin, the surrounding site, and the possible causes of any structural issues. This will inform whether further monitoring and surveys are necessary and if calculations are required to assess stability.

An initial visual survey is typically carried out from ground level, using binoculars or a camera to look at details high up on the wall. A drone can be helpful to check the condition of the wall head or high-level masonry (Figure 37).

Basic dimensions of the wall and measurements of any leaning sections can also be useful at this stage. Observations can be recorded on measured drawings (if available) or photographs.

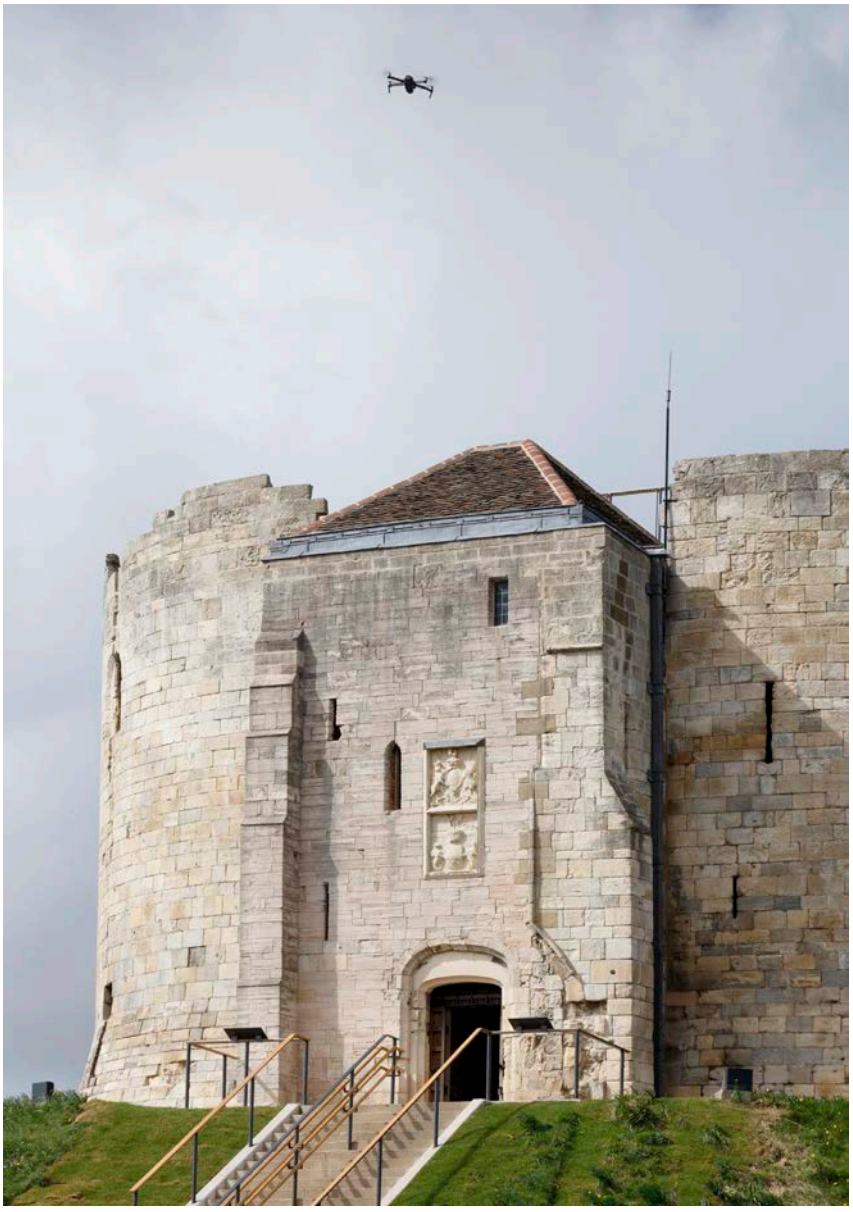


Figure 37: A drone survey of Clifford's Tower in York to inspect the condition of the wall tops.

Unstable walls
If there is any concern about the stability of a ruin, an initial survey should be carried out from a safe distance. Temporary propping may be required to allow a closer inspection, for which relevant consents should be obtained. High-resolution photography or drones can be used to safely gather information.

Any concerns about stability should be raised immediately with the owner or custodian, and measures should be taken to protect the public.

Demolition
If an asset is proposed to be demolished additional guidance can be found on the IStructE website:

Guidance Note: Assessing Historic Assets that are Proposed to be Demolished or Partially Demolished.

This note, authored by CARE in conjunction with the IStructE and ICE, is intended to provide clear guidance for engineers who are requested to inspect and report on historic assets.

Identifying structural issues

The following questions are a good starting point when carrying out a survey:

- Is the wall leaning, and can this be measured using a plumb-bob or surveying equipment?
- Does the wall face appear damp?
- Does the wall face appear to bulge?
- Is there overhanging or unsupported masonry?
- Is there fallen masonry around the base of the wall?
- What is the condition of the masonry units and mortar?
- Are there any surviving surface coatings, such as render, plaster or limewash?
- Are there any embedded timber or metal elements in the wall? What condition are they in, and what is the condition of the adjacent masonry?
- Are there cracks in the wall? What do the crack patterns indicate?
- Are areas of cracking or collapse old or recent? Visual observations on weathering, the presence of vegetation, lichen growth and accumulation of debris can be used to gauge how recent the deterioration may be.
- Has the condition changed when compared to previous surveys and photographs?

Identifying possible causes

The following questions can help to understand what has caused structural issues:

- What is the condition of any previous repairs?
- What is the wall capping construction and what condition is it in?
- Are there trees nearby? What size and species are they?
- Is there vegetation growing on the wall tops or faces?
- What are the ground conditions?
- Is any drainage visible on site?
- How is rainwater shed from the walls?
- Has there been any change to the ground levels around the wall?
- Has there been any change to the hydrology?
- Has the loading on the wall changed?
- Have the restraint conditions changed? For example, recent loss of roof or return walls.

3. Further surveys, monitoring and structural analysis

Following the initial survey, the engineer needs to decide what the next steps should be. Would more detailed investigations (intrusive or non-intrusive) yield useful information? Is a period of monitoring needed? Refer to section 4 for more information on monitoring and surveying techniques.

Structural analysis may help to establish whether the wall is at risk of failure due to sliding, overturning, bending, or shear. Refer to sections 5 and 6 for more information on failure mechanisms and material behaviour.

4. Design and structural repairs

If the steps above show that the ruin is structurally unsound, then a sensitive structural repair should be developed by the design team, in consultation with the Inspector of Ancient Monuments or Conservation Officer and any other relevant statutory bodies.

Often the ruin will be structurally sound but undergoing slow decay. In this case less intrusive repair works such as capping repairs, masonry consolidation and repointing are appropriate. Unless the structural engineer is experienced and confident in the specification of fabric repairs these repairs should be specified by a building surveyor or architect experienced in conservation.

Refer to sections 7 and 8 for more information on types of structural repairs and when they may be appropriate.

4. Surveying and monitoring

Why survey or monitor ruins?

A simple visual survey may not be sufficient for an engineer to be confident that a ruined asset has structural integrity. There are many reasons to employ more thorough survey and monitoring techniques during the investigation phase of a project. Monitoring can also be used during remedial work and post-completion to assess the effects of any interventions on the ruin. Before beginning work, contact the relevant national or local body to understand what consents are required.

The most common reasons for surveying or monitoring a ruined structure are:

- to provide accurate dimensional information for analysis and to define or specify remedial work;
- to review the general condition of the structure;
- to better diagnose and understand the cause of defects;
- to confirm whether movement is ongoing or historic;
- to differentiate between normal seasonal, diurnal and/or tidal variations and progressive movement;
- to provide a point of comparison for asset managers;
- to check for in-plane or out-of-plane movement;
- to check for specific defects such as voiding or hidden ironwork, which may cause damage or collapse if not addressed;
- to demonstrate whether an apparently defective structure is still performing adequately.

Survey techniques

Common non-intrusive techniques that can be used to assess ruins are:

- Close-up visual and tactile surveys using access equipment as required. Access may be via rope, cherry picker, tower scaffold, scaffold, crane basket, or ladder (Figures 38, 39 and 40).
- Measured surveys to define geometry and identify leaning walls or bulging facework (Figure 41).
- Archival research to look for evidence of historic work and defects (Figure 42).
- Cover meter surveys to identify embedded metal.
- Ground penetrating radar (GPR) to determine the location and condition of buried timber or embedded metalwork within the walls; the location of voids within the masonry core; the presence of narrow voids suggesting delamination or the location of below-ground voids in the area surrounding the structure which have the potential to cause instability. The results of GPR can be difficult to interpret in complex composite walls due to the heterogeneous nature of the construction. They require expert analysis and should be reviewed alongside other evidence.
- Basic checks using hand tools, measuring verticality with plumb-bobs, tapping to identify delamination.





Figure 38 (left): Rope access survey at Kenilworth Castle, Warwickshire.
Figure 39 (right): A structural inspection by cherry picker at Witley Court, Worcestershire. © English Heritage.





Figure 40 (left): Tower scaffold for inspecting high level masonry.
Figure 41 (right): Setting up for a measured survey of a site.

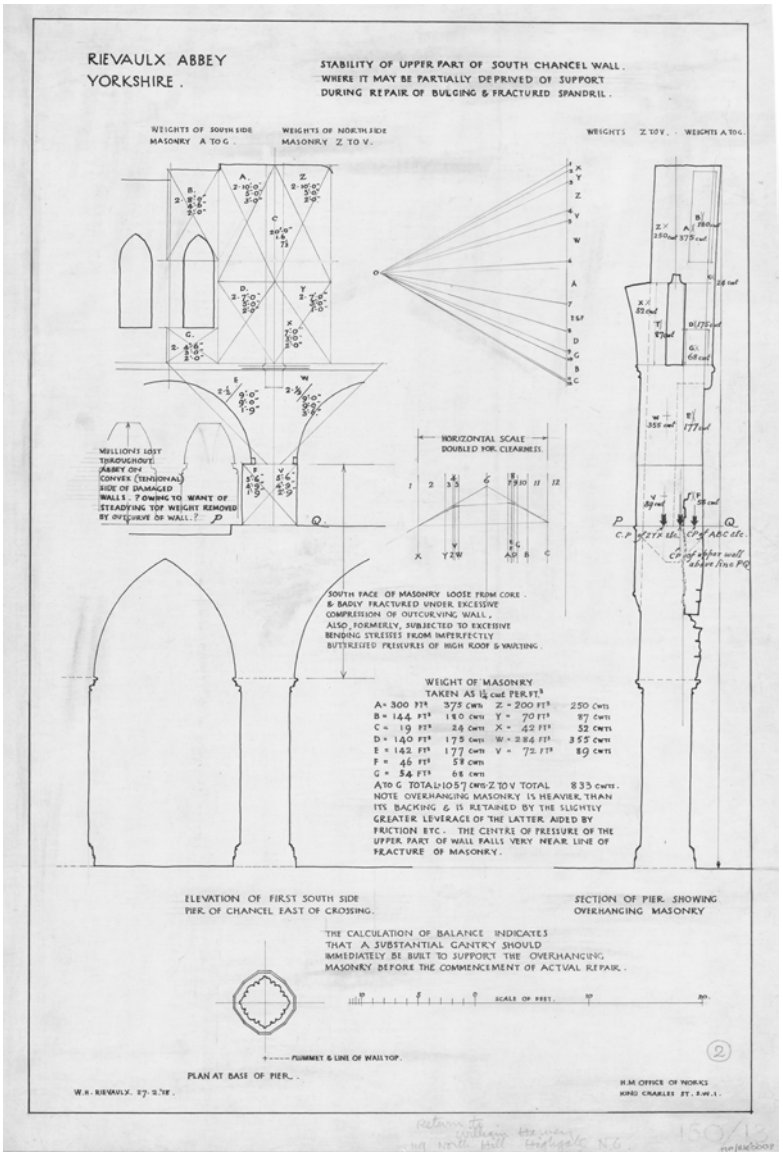


Figure 42: A calculation for historic interventions at Rievaulx Abbey in North Yorkshire found in the Historic England Archive.

Measured surveys, cover meter surveys and ground penetrating radar are specialist surveys that may be carried out either by the structural engineer themselves if they have the capability, or by a specialist. If a specialist is used, interpretation of the results will still be required by the project engineer to gain a holistic understanding of the site.

Intrusive Surveys and Material Testing

Material testing is sometimes needed to understand the properties of the masonry units and mortar. This is not covered in detail here but may be required in areas where the compressive strength of the masonry unit and mortar is critical. Analysing the composition of existing mortars may help inform mix design for remedial work. The strength of mass rubble core walls relies on the strength of the whole matrix rather than individual elements. This is discussed further in section 5.

In some cases, intrusive investigations, such as trial pits or localised opening-up, can provide useful information on wall build up and condition, foundations and bearing strata. Intrusive investigation should only be carried out where needed because of the potential damage or disturbance to the historic fabric and buried archaeology. Consents from the relevant national or local body should be sought.

Movement monitoring

What is the purpose of movement monitoring?

Monitoring can provide useful data and prevent unnecessary intervention. The scope and purpose of monitoring must be clearly defined at the start to ensure that the results are useful and relevant.

Different techniques can be used depending on access constraints, budget and the nature and accuracy of information required. Techniques suitable for recording changes in geometry over many years may not be suitable for assessing whether an isolated crack is continuing to open. The cost of monitoring varies greatly depending on the technique used; the cost of more sophisticated monitoring can be significant.

Some key questions include:

- What is the monitoring for? Is it necessary? Will it change the outcome?
- What is the required accuracy? Does a crack need to be monitored to the closest millimetre or are general trends sufficient? The accuracy must be sufficient for potential failure mechanisms to be identified and addressed as quickly as possible, to minimise ongoing deterioration.
- What is the required frequency of readings? Can this frequency be reduced if an initial period of regular readings gives confidence in the rate of movement?
- What degree of normal seasonal, diurnal or tidal variations is expected? The accuracy and frequency of reading must be sufficient to detect movement outside of these trends.
- Are there any access restrictions? How accessible is the area that needs to be monitored? Is a ladder or any other form of access equipment necessary?
- Is the structure safe and would any potential failure mechanism be ductile? Are extra precautions required to install or read remote monitoring?

Types of monitoring

High-precision target monitoring

Set positions can be monitored using a total station from established survey control points. Positions are defined using cross headed screws, grouted in levelling sockets with ball-headed bolts or semi-permanent prisms (Figures 43 and 44).

Accuracy: +/- 2mm

Advantages:	Disadvantages:
• Simple to set up for a professional and experienced surveyor, • Very accurate, • Can be directly compared via spreadsheet requiring no specialist skills, • Small data size.	• Requires skilled surveyors and high-end equipment to properly establish the markers and monitor over time, • Only provides data on specific monitoring positions; site wide issues may be missed, • Points may become lost between surveys or may be subject to vandalism, • Loss of accuracy between multiple set ups required to pick up lots of points, • Monitoring positions often need to be fixed to the structure, • It could be a long time before a defect is picked up depending on the interval between surveys.

3D laser scanning

The ruin can be scanned producing a three-dimensional point cloud which can be used to obtain overall geometry or detect overall trends in building movement with low accuracy.

Accuracy: depends on the accuracy of the equipment and site constraints such as vegetation. There may be variability in accuracy between wall tops and the surrounding ground.

Accuracy: +/- 5 to 10mm

Advantages	Disadvantages
• Records geometry and any high-level changes, • Can use drones or robotic scanners if site is dangerous or inaccessible (Figure 45), • Can pick up large areas easily and quickly, • Can use data to produce elevations and sections.	• Lower accuracy than monitoring discrete targets, • Specialist surveyor required, • Loss of accuracy between multiple set ups required to pick up lots of points, • Requires specialist software to process and compare movement between point clouds, • Large data size, • Grass and vegetation has to be cut back prior to each survey to prevent the accuracy of results being compromised, • It could be a long time before a defect is picked up depending on the interval between surveys.

Remote monitoring

Remote tiltmeters, crack meters and vibration monitors can be installed to monitor small changes in the width of cracks, inclination or vibration which could indicate early signs of structural failure (Figures 46 and 47). Proprietary devices can be bought or hired off the shelf. Bespoke systems can be built to be site specific.

Accuracy: +/-0.03 degrees

Advantages	Disadvantages
• Data can be viewed in an online portal, • Trigger levels and alerts can be set, • Can be programmed to take readings in real time or at a desired frequency.	• High initial cost, particularly if custom built, • Ongoing cost for the monitoring portal, • Maintenance visits are necessary to change the batteries. This will be more difficult if the site is not safe, • Devices may be subject to vandalism, • Relies on mobile phone signal.

Photogrammetric comparison

Photogrammetry can be used to produce a three-dimensional model. Models can be produced at agreed intervals and compared visually, ground markers could be installed for ease of comparison.

Accuracy: low

Advantages	Disadvantages
• Low cost, • Can be undertaken by drone if the site is unsafe, although a specialist company with appropriate permissions may be required in an urban environment.	• May be subjective or difficult to accurately compare, • No precise information.

Tell tales, crack-width gauges or callipers, plumb-bobs, laser levels

Cracks and inclination can be monitored by hand using basic hand tools (Figures 48, 49 and 50).

Accuracy: +/- 0.5mm for digital callipers/ laser levels, lower for plumb-bobs and highly dependent on weather conditions.

Advantages	Disadvantages
• Low cost to set up; local volunteers can be trained to take readings, • Allows quick analysis of stability which can be used to decide whether other techniques such as laser scanning are required.	• Relies on the capability of the surveyor; results may vary if the surveyor or instrument changes, • Labour intensive; the surveyor must make regular visits to site, • Repeated monitoring visits may be expensive.

Satellite monitoring

Interferometric Synthetic Aperture Radar (InSAR) may be used to detect changes in the position of the ground (or building) surface. InSAR is a monitoring technique that uses radar imagery to measure ground movement.

Comparing the images produced at different times can determine whether the subject has moved away from or toward the satellite during the period between data drops. Coverage and noise can be calculated to define the accuracy of the data for a specific site.

Accuracy: mm/yr

Advantages	Disadvantages
• No need for operatives to travel to site; reduced carbon footprint/cost • Useful for monitoring large areas over long periods of time; historic data is available for many sites. • Information is updated every 6 to 12 days • Free open data for a coarse grid, a tighter grid of points available commercially.	• Information must be processed by a specialist survey company; it is not an off-the-shelf product, • InSAR may be too coarse for individual assets, • Does not work everywhere, • Only measures movement in the line of sight between the satellite and ground, • Picks up overall movement of the asset and may help to identify the cause of damage but does not pick up damage to asset such as cracks opening up.



Figure 43: Prism targets either side of a crack at Longtown Castle, Herefordshire.



Figure 44: High precision reflectorless monitoring of the motte at Totnes Castle, Devon.



Figure 45: Laser scanning a dangerous building using a robotic dog scanner.

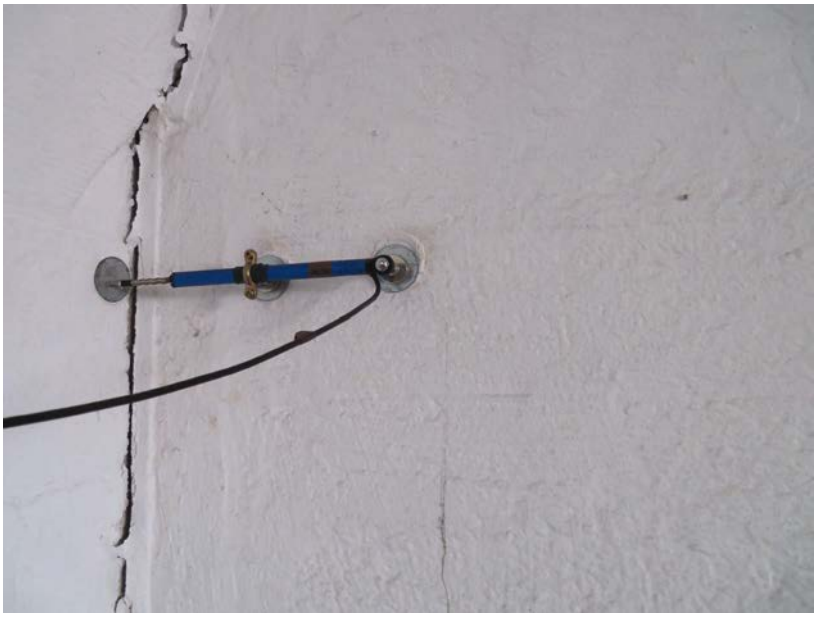


Figure 46: Remote crack monitor measuring any change in crack width.



Figure 47: Remote tilt monitor measuring any change in rotation of a damaged wall.

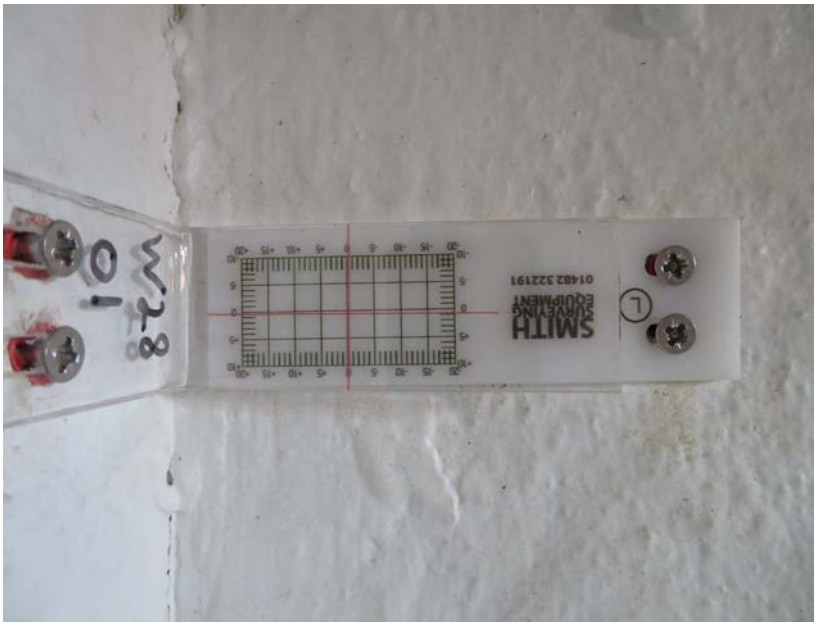


Figure 48: Tell-tale to monitor a crack at the junction of two walls.



Figure 49: Demec studs and electronic vernier callipers used to monitor movement in a cracked masonry wall.

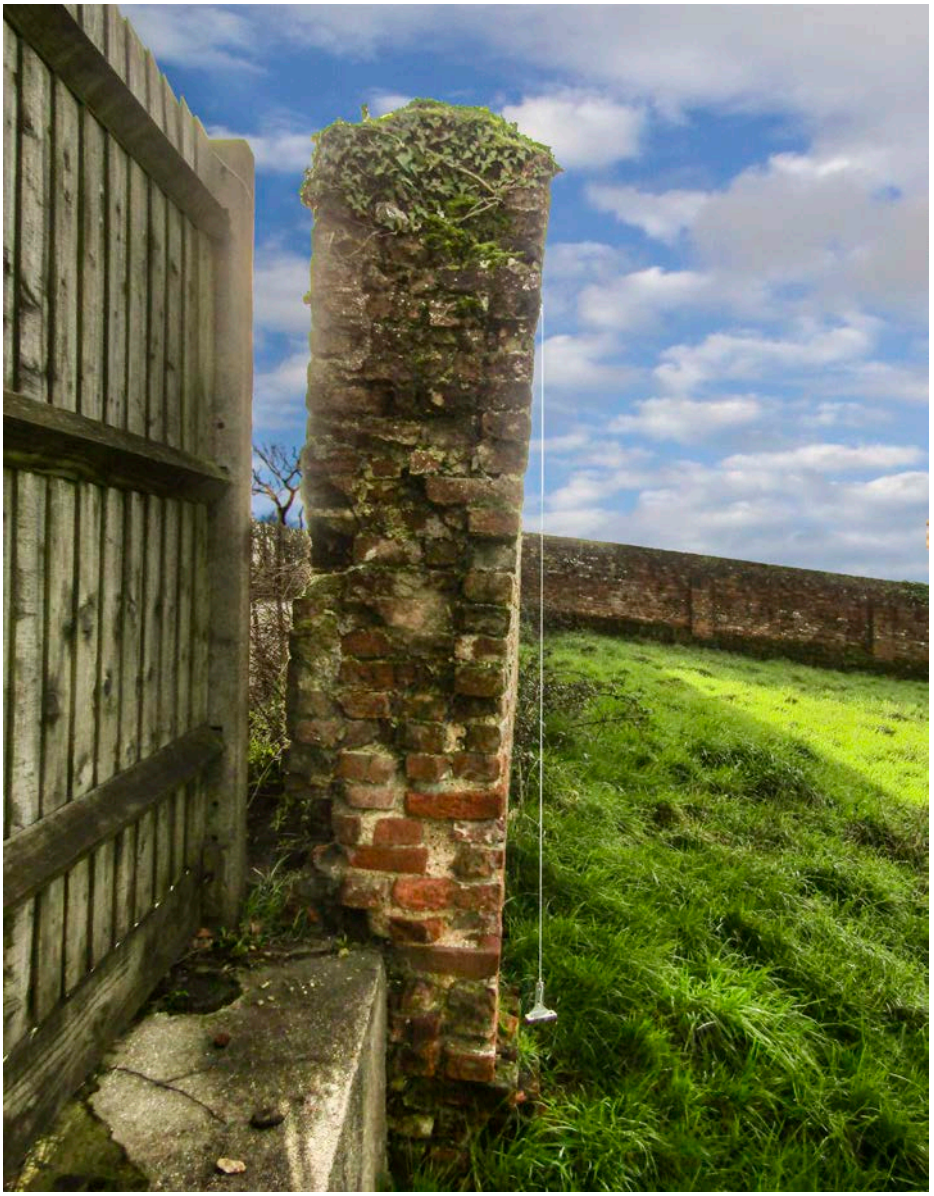


Figure 50: Plumb-bob used to measure the inclination of a leaning masonry wall.

Considerations when setting up monitoring

Ruined structures differ from new structures in many ways. They often remain in the same ownership for many decades, and this can be advantageous in relation to monitoring, as the records will remain with a single organisation.

Ideally, a single individual should retain responsibility for coordinating, filing and reviewing the monitoring data. This is particularly crucial when monitoring takes place over a long period of time. It is very easy for records to get lost and monitoring appointments to be forgotten. The file format of the data should also be considered at the outset, particularly if the results need to be accessible to future generations.

Ruined structures are often scheduled or have some other form of statutory protection. Consent may be needed to attach prisms or any other semi-permanent monitoring points to the structure, and advice should be sought from the local Inspector of Ancient Monuments or local authority conservation officer.

The ongoing cost of the monitoring should be understood from the outset and allowances made in future budgets if required. Initial set up is likely to be the main cost burden but there will be costs associated with site visits, post processing of data after each visit and maintenance of remote monitoring units.

The need for trigger levels or alerts should be carefully considered as these add cost and complexity to the monitoring set up. If trigger levels and/or alerts are set, an action plan should be prepared, including the named contacts who will receive the alert from the remote devices and what will happen if the trigger levels are reached. The action plan should consider:

- whether the site will need to be closed;
- whether adjoining land or rights of way will need to be closed;
- what happens if a trigger level is breached out of hours?

If local authorities and neighbouring landowners need to be involved in the event of a trigger level breach, they should be engaged at an early stage.

5. Engineering properties of lime-mortared masonry

The structural behaviour of traditional masonry structures is primarily governed by their shape and weight, i.e. the physical routes for possible thrust lines (compression-only load paths) to be contained within the masonry by some acceptable geometric margin of safety.

Section 6 covers structural stability of traditional masonry ruins and suitable methods for assessment. Occasionally a structure may be of such borderline condition, or it may need to accept such a significant change in loading, that the engineer requires more detail about the physical and mechanical properties of the masonry material.

Structural composition

The form of masonry construction varies from building to building. Joint thickness, and the associated volume of mortar, vary according to the local vernacular methods of construction (Figure 51). The quality of mortaring also varies, and in rubble-cored walls the core may even be built dry or left unfilled and can vary in quality. The core may range from constructively packed 'snecks' or cut masonry units in horizontal courses, deliberately locked together by the mason, through to 'end tipped' construction in which rubble and mortar are tipped into the space between the inner and outer skins of masonry.

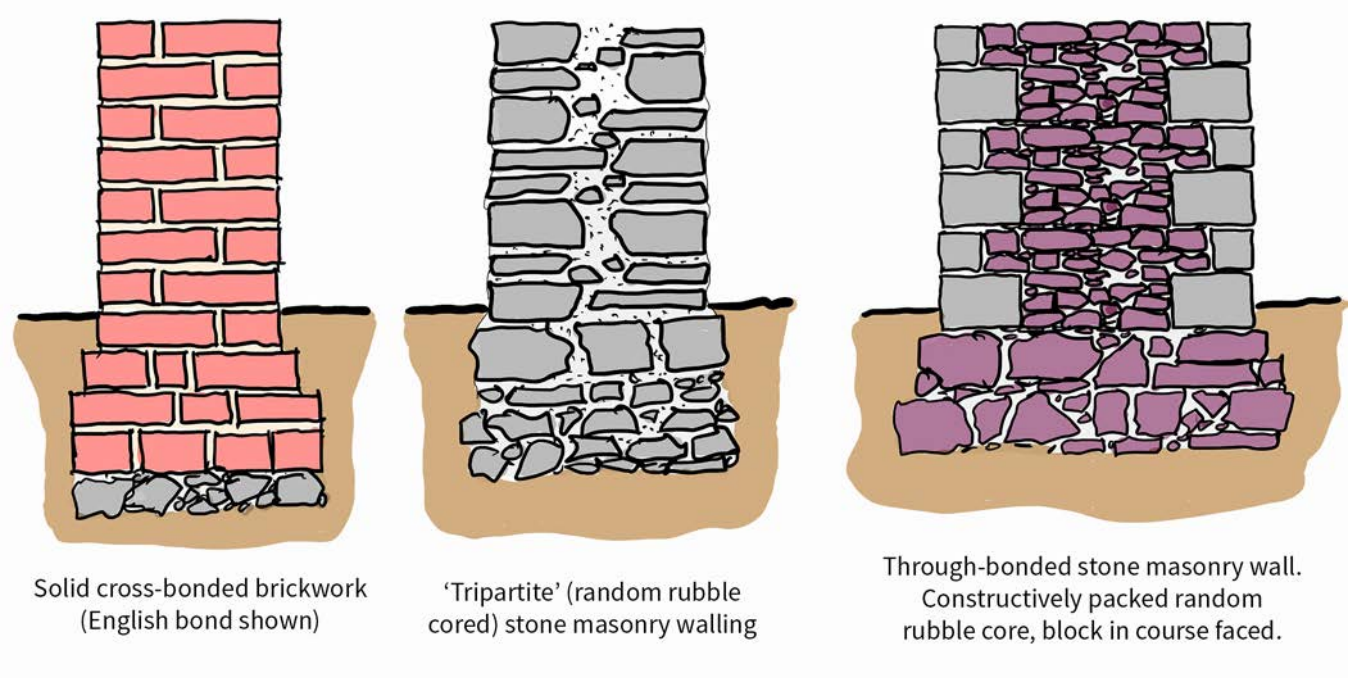


Figure 51: Some common masonry wall types encountered in the conservation of masonry ruins.

Most masonry ruins are constructed using traditional lime mortar, earth mortar or lime stabilised earth mortar to bed and bond the masonry units. The mortar may appear cementitious if it has been surface pointed with a modern material.

Practical experience and analysis of data compiled by the Scottish Lime Centre Trust (SLCT)[3] suggests that most historic lime mortars in traditional buildings are of air (non-hydraulic) lime or feebly hydraulic binder type and are rich in binder amount. Typical slaked lime to aggregate proportions were generally around 1:1.5 parts by volume.

Lime-rich mortars have exceptional bond intimacy with the masonry units, giving a good seal and hydraulic contact but a low mechanical pull-off strength compared with cement mortar. This means that traditional masonry cannot support flexural tensile stress across the joints. This is a very important distinction between the behaviour of traditional and modern masonry structures. In practice it means that the engineer needs to find compression-only load paths through the masonry.

Material properties

Traditional stone masonry typically employs masonry units of very high strength, with comparatively very weak lime, earth or earth-lime mortar. Brick masonry generally also follows this trend, although stone tends to exceed the strength of brick at the extreme end of the spectrum.

Physical properties for common masonry units are given in Figures 52 and 53;

Type of clay brick	Compressive strength, f_b (N/mm^2)
Extruded	20-200
Machine moulded	3.5-175
Engineering	≥ 70
Commons	2-20
Flettons (normal)	15-28
London stocks	7-20
Calcium silicate bricks	7-35 (commonly 14-28)
Concrete bricks	7-40

Figure 52: Indicative values of compressive strength of bricks (Adapted from *Appraisal of existing structures*. With permission from the Institution of Structural Engineers. ©IStructE Ltd. [Note: source material probably originated from: Everett, A. *Materials*. London: Batsford, 1970]).

Type and location	Density (kg/m^3)	Compressive strength, f_b (N/mm^2)	Water absorption (% of dry weight)
Granites			
Devon and Cornwall	1800-2650	140-200	0.30 +
Cumbria and Scotland	2600-2900	170-250	0.19-0.24
Sandstones			
Derbys., Glos. and Notts.	2200-2600	40-65	3.5-5
Lancs., Shrops. and Staffs.	1950-2350	25-40	Not available
Northumberland	2150-2350	35-60	5-10
W. Yorks., Durham and Scotland	2200-2750	40-70	2.4-2.8
Limestones			
Avon (Bath) and Devon (Beer)	2200-2400	17-35	7-15
Cumbria and N. Yorks.	2400-2700	80-100	Not available
Glos. (Cotswold)	1800-2300	16-120	1-4
Leics. (Clipsham) and Lincs.	1800-2200	20-35	4.5-10
Portland	2000-2300	35-55	4.5-10

Figure 53: Indicative values of physical properties of common building stones.
(Adapted from *Appraisal of existing structures.* With permission from the Institution of Structural Engineers. ©IStructE Ltd. [Note: source material probably originated from: Everett, A. *Materials*. London: Batsford, 1970]).

The Geological Society has compiled a database of building stone in the UK which is available as open access via the Lyell Collection, giving density, compressive strength and durability indicators of a wide range of stones used across the British Isles[4]. The BRE British Stones Glossary[5] provides further information.

Values for the strength of lime mortar vary wildly in the literature; for air lime (non-hydraulic) mortars, compressive strength values from 0.5-6.87 N/mm^2 are quoted. Many studies quote strength values at very young ages. For air lime mortars, strength increases with carbonation and because this takes place over many years, such studies do not provide reliable information about ultimate mortar strength. Guidance from Hendry (1990)[7] and IStructE (2010)[8] assume the strength of lime mortar to range between 0.5-1.0N/mm^2. It is worth noting that samples cured under ideal laboratory conditions are likely to vary in compressive strength from mortars cured in field conditions.

As understanding of traditional masonry construction continues to advance, these assumptions appear increasingly pessimistic, although depending on the formula used to estimate the strength of the masonry composite, high strength values may still be estimated for assessment purposes.

Aggregate type, shape and particle-size distribution, and water-binder ratio strongly influence early strength of air lime mortars. Those made with oolitic limestone sands can achieve far higher strengths than those made with silicate sands, sometimes even exceeding the strength of mortars made with moderately hydraulic binders[9]. Angular and well-graded aggregates generally impart

higher strength than rounded or evenly graded ones. Stiffer mixes, including dry slaked, hot mixed mortars achieve early strength and become load bearing faster.

Figure 135 in the appendix presents a summary of compressive strength values for different mortars found in literature, this information is summarised in Figure 54 (below).

Hydraulic mortars harden by a combination of dewatering, an early chemical set (hydration of calcium silicates and aluminates), followed by carbonation of the available lime in the binder[19]. With increasing hydraulicity, there is an increase in mortar strength and brittleness, and a reduction in bond intimacy at the mortar-masonry unit interface. These properties may not be compatible with traditional and historic masonry substrates[20]. Increasing hydraulicity can also densify the material microstructure, reducing fabric compatibility.

For general purposes, a reasonable strength estimate for undetermined lime mortar encountered in a masonry ruin is around 1.0N/mm^2 or 1.5N/mm^2, where binder richness and fine sharp aggregate can be visually identified. For weak, friable mortar, the lower value of 0.5N/mm^2 would be appropriate.

It should be noted that these are uniaxial compressive strengths. As part of a masonry structure, the comparatively weak lime mortar can support stresses far higher than its uniaxial compressive strength in isolation. This is due to the triaxial confinement of the weak, deformable mortar joint by the strong masonry units[21].

Triaxial compression tests on air lime mortars have demonstrated a major increase in compressive strength and a plastic, ductile behaviour with relatively small confinement pressure, compared with the weak, brittle behaviour of the same mortar under uniaxial tests. This provides clear evidence of this point[21]. There is rarely any well-reasoned engineering justification for the use of high strength mortars for the care and repair of historic masonry.

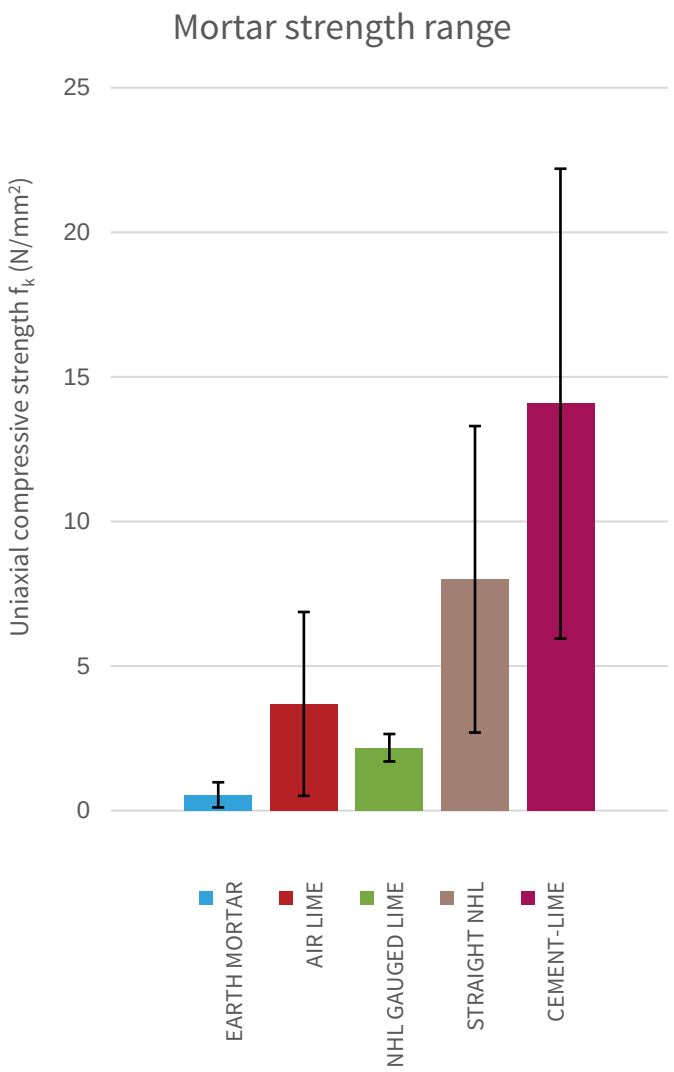


Figure 54: Indicative range of uniaxial compressive strengths from literature for different types of mortar. Median values are shown, drawing on the sources quoted in the appendix to this section.

The masonry composite

The strength of the masonry composite is a function of the respective uniaxial compressive strengths of the masonry units, the mortar, the thickness of the mortar joints, the shape of the masonry units, and the cross-sectional bond detailing[21].

The Institution of Structural Engineer's publication, Appraisal of Existing Structures[8] notes that there is no simple way of assessing the compressive strength of masonry in situ; for the assessment of structural masonry, design codes do not necessarily need to be used, provided a logical justification for whatever assumptions or judgements are made instead (section 6.2.5)[8]. This may include a review of the load history of a building or part of a structure, evaluated against its structural pathology.

Eurocode 6 (BS EN 1996-1-1:2022)[22] gives several options for the strength of masonry to be determined. In the assessment of existing masonry structures, many of which may be listed

buildings or scheduled monuments, physical testing of the material will not necessarily be possible and will be the exception rather than the norm. This normally means that the structural engineer must assess the strength of the masonry by studying the physical composition of the ruin, and relating direct visual observation to published strength data (in a similar manner to the engineering assessment of historic steelwork, for example).

Eurocode 6 (BS EN 1996-1-1:2022)[22] gives the following expression for the calculation of masonry strength:

$f_{m,k} = K \bullet f_b^{\alpha} \bullet f_m^{\beta}$ (Equation 1)

For stone masonry, K is given as 0.45, α is given as 0.7, and β is given as 0.3. For brickwork, K ranges from 0.35-0.55 for general purpose mortar, and higher for thin layer mortar joints (cl.5.7.1.3)[22]. Reduction factors apply to rubble masonry construction which further reduce the compressive strength of the masonry composite. It should be noted that Equation 1 is based on a database of slender wall panel and wallette testing constructed with cementitious mortars which have little correlation with traditional masonry structures.[13] To estimate the strength of traditional masonry built with lime mortar, the following formula may be more realistic:

$f_{m,k} = 0.56 \bullet f_b^{0.53} f_m^{0.5}$ (Equation 2)

A traditional rule of thumb for the design and assessment of structural masonry is that the strength of the masonry composite is one-tenth the crushing strength of the stone.[23]

An estimate of the strength of both the masonry units and the lime mortar must be made. In general, traditional masonry is very strong, although engineering judgement cannot be dispensed with and a realistic assessment of the masonry units found in a given structure should be made by the engineer.

Strength curves for masonry built using lime mortar using the optimised formula (Equation 2) are given in Figure 55. For masonry units in the 40-100N/mm^2 strength range, and lime mortar values around the upper limit of the non-hydraulic strength range (~1.5N/mm^2) the strength of the masonry composite is around one-tenth that of the stone, which agrees with the traditional rule of thumb.

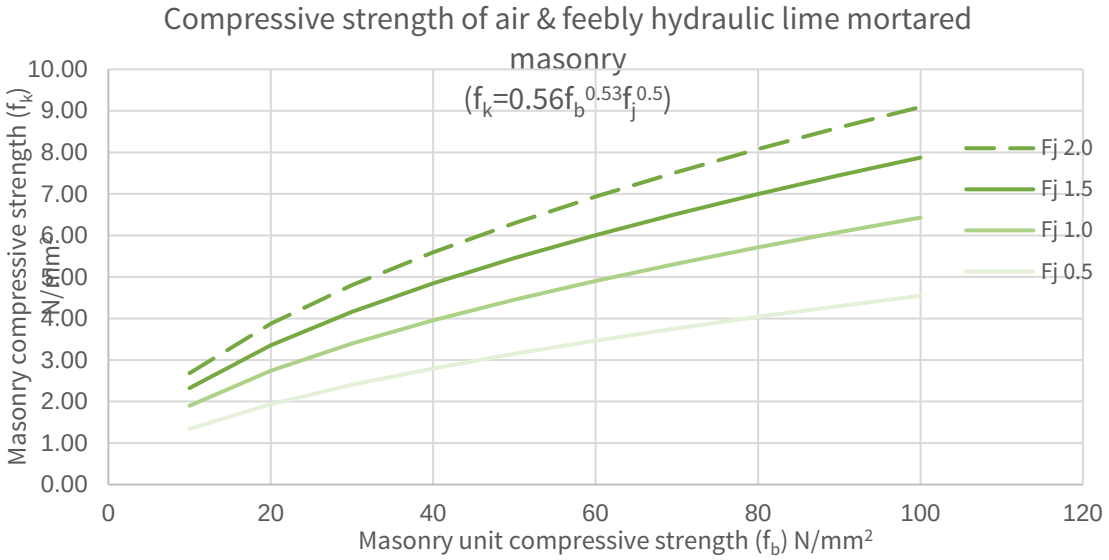


Figure 55: Compressive strength of masonry composite using the optimised formula advanced by Costigan et al. (2015)[13] to estimate the strength of structural masonry built with lime mortar.

This formula still includes assumptions, which may not apply to the specific masonry under assessment. Standard joint thicknesses are assumed (of the order of 10-12mm, although not explicitly stated), and the effect of unit size is not captured by the formula, instead requiring the 'normalised compressive strength' of the masonry unit to be determined by the engineer.

For thin jointed masonry, significantly higher strengths can be achieved for masonry made with the same materials: masonry built with 5mm thick joints is around 1.6 times the strength of the same masonry built with 10mm thick joints, and for tightly jointed ashlar work the strength of the masonry composite approaches that of the stone unit[7].

For thicker, rubble core walls and random rubble construction, further reductions in strength apply to accommodate the effects of cross-sectional bond limitations and the higher proportion of mortar relative to the masonry units. The impact of the quality of the core work is demonstrated by Mundell et al[24] which shows tightly packed drystone retaining walls can accommodate significantly more displacement than poorer constructions due to tensile strength developed through the interlocking of their stones.

Engineering judgement needs to be employed in choosing which formula to apply for which masonry element in question, as wildly different strength estimates can be generated by the different equations.

Material testing for assessment

Stone and Brick

Analysis of masonry units starts with visual identification of the stone or brick type, with reference to published strength data for similar masonry units (Figures 52 and 53).

Further clarification of the type(s) of masonry units present in a structure can be achieved by minimally intrusive sampling and analysis by an experienced analyst via optical microscopy on the freshly broken profile to clarify visual identification, progressing to petrography if necessary. The findings could be compared to published data of similar stone/brick types.

Where there remains significant doubt as to the material strength, physical testing may be needed. Specimens should be tested in the direction that they are loaded in practice; at least five results should be obtained, and the mean strength adopted for estimating unit strength. Specimens should not be less than 90mm diameter and standardised to the 'normalised' compressive strength of a 200mm cube. Cores of h/d ratio of 1.0 correspond to a 200mm cube.

Where the building is formally designated (i.e. Listed or Scheduled), the appropriate consent will be required for intrusive physical testing, the rationale for which will have to be clearly and convincingly justified by the structural engineer. In large engineering structures, physical testing on a statistically relevant scale is essentially impossible.

Mortar

Analysis of mortar at the most basic level starts with a close visual inspection of the mortar(s) present in the masonry. A freshly broken cross-section often reveals more than the elevational surfaces which have been subjected to weathering and organic growth.

Minimally invasive intrusive testing of the masonry materials may be undertaken to help clarify structural properties for assessment and inform the design of repairs. However, the mortar cannot

realistically be tested for strength in situ, and obtaining samples large enough to submit to laboratory strength testing is highly unlikely. Small specimens may be extracted and analysed to reveal their physical and chemical composition, including:

- aggregate type, shape and grading
- binder type
- binder-to-aggregate mix proportions.

These properties can be readily determined by a combination of acid digestion and x-ray diffraction techniques. Further insight into the mortar's physical composition can be achieved via petrographic examination and porosimetry by an experienced materials analyst.

Having identified the mortar type, the engineer can then estimate its structural properties by comparing it to published data (for example, Figure 135 in the appendix), and/or preparing mortar specimens of matching composition in test moulds. These specimens can be subjected to traditional compressive strength testing to evaluate compressive strength, stiffness and density.

It is important to keep in mind that replicating the original mortar is only desirable where the performance requirements, exposure and environmental conditions remain unchanged; this will rarely be the case with a ruined structure.

Stability and strength in perspective

In this review of material properties, it is important to appreciate that masonry buildings normally fail by falling over, not by crushing failure of the material. Traditional masonry buildings can exhibit serious or dangerous structural conditions – occasionally on the point of collapse – while continuing to maintain very low internal stresses compared with the strength of the masonry material (Figure 56).

Unless the building is founded on rock, it is obvious that the formation soil will typically be the weak point, especially when eccentric bearing pressure profiles are involved, such as with leaning walls, piers, and arch structures (Figure 57.) Visualising traditional masonry structures as upside-down weighted cable structures demonstrates this: a tension system is only as strong as its weakest part.

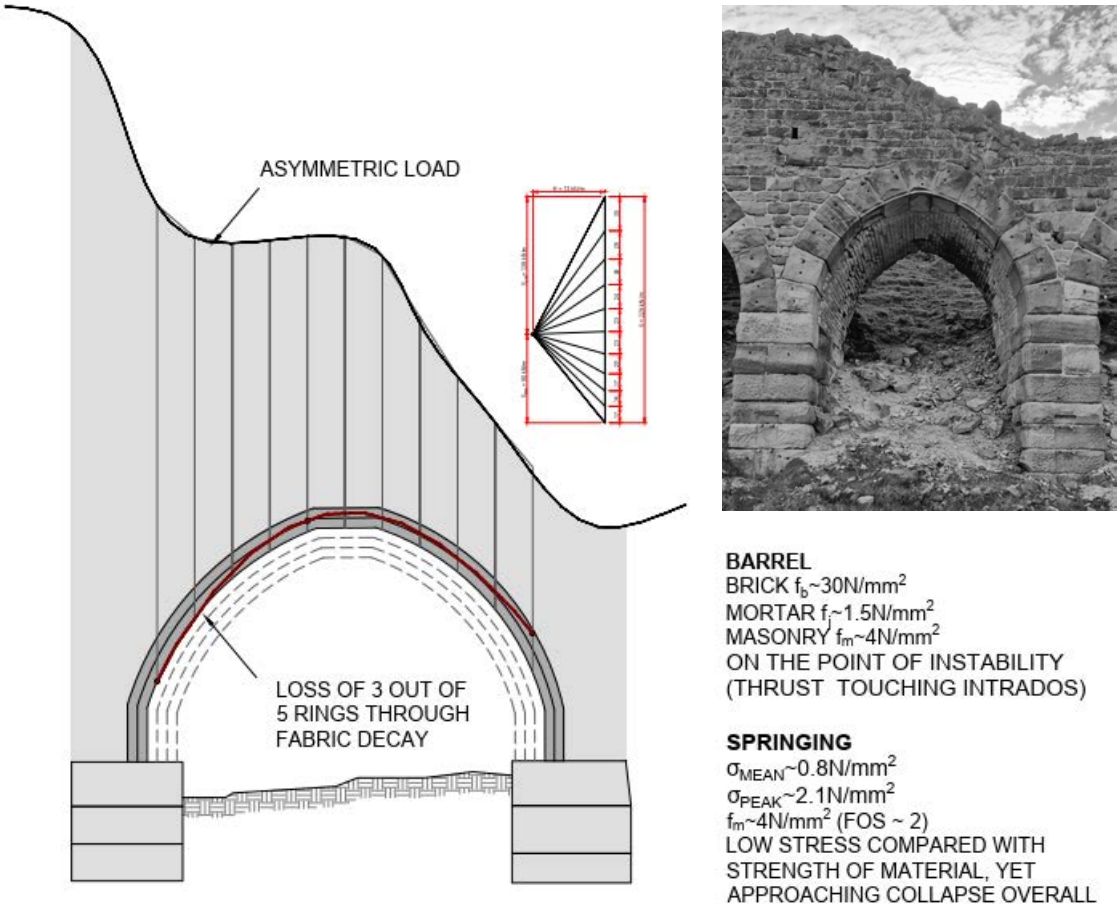


Figure 56: Extract of a structural assessment of a ruined kiln approaching collapse, still maintaining low internal compressive stresses compared with the strength of the material. © David Wiggins.

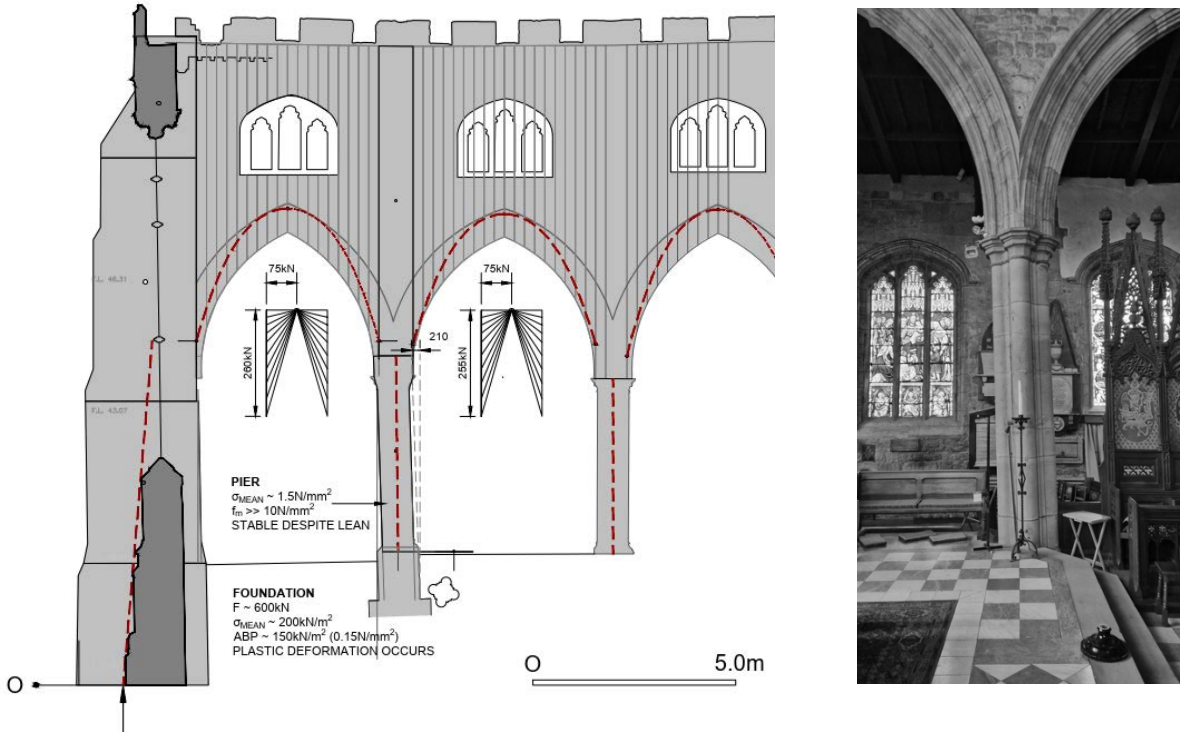


Figure 57: Extract of a structural assessment of a deformed pier in a spreading arcade, illustrating the order-of-magnitude difference in strength between the ashlar pier and the clay formation soil upon which it bears.
© David Wiggins.

Structural and fabric pathology

Structural and fabric are not separate. The building exists in its entirety while at the same time responding to structural and environmental actions.

Ruined structures have generally lost the restraint of floors and roofs, so that walls are often free-standing or mutually buttressed by remaining cross-walls. The steady percolation of rainwater through these exposed wall-top joints can degrade the masonry bond over time and reduce mutual restraint, leaving one or several parts of the building very slender or potentially unstable. For the ongoing care and structural conservation of masonry ruins, it is important to manage water ingress to prevent deterioration of these stability-critical restraints.

Decay of the masonry fabric, while generally slow, may already be at an advanced stage in the case of ruins. Any significant loss of section to a wall or structural element will reduce the geometric factor of safety between thrust-line and the edges of the masonry, and by weakening the remaining fabric, require an increasingly thick zone across which the thrust must be spread to prevent crushing (Figure 58).

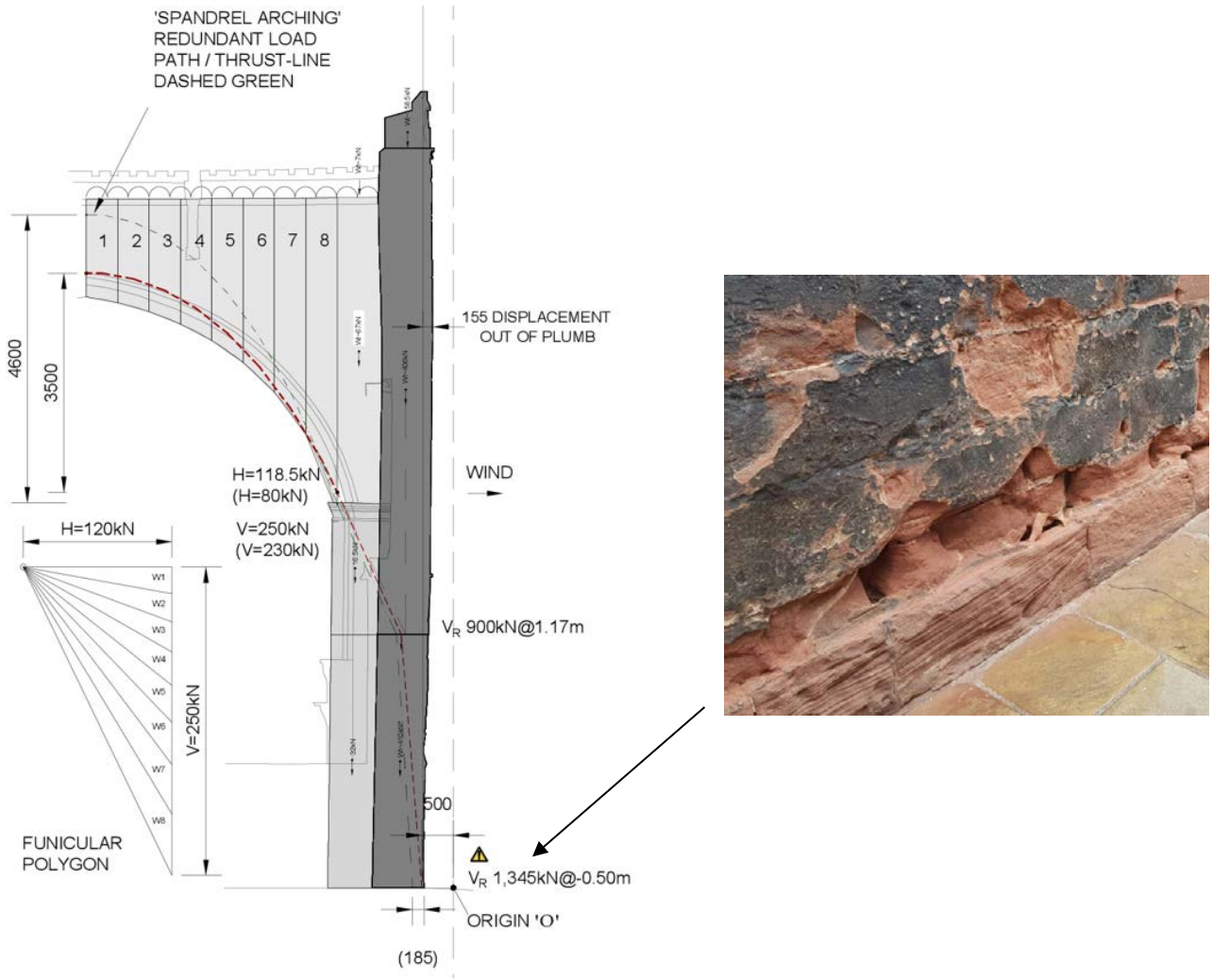


Figure 58: Extract of structural assessment of a badly buttressed gable, suffering extensive fabric decay at its base, eating into the very limited reserve of masonry (geometric factor of safety) to contain the thrust from the arcades.
© David Wiggins.

In advanced cases, structural indent repairs will be required to replace heavily decayed or missing masonry units and mortar, reinstating the loss of structural support. Decay in UK masonry is primarily driven by soluble salt precipitation, which occurs at the drying fronts (often around the bottom of walls and on leeward faces of coastal ruins).

To effectively manage the precipitation of salts, it is essential to keep mortar joints tightly filled with traditional lime mortar and/or earth mortar, depending on the particular substrate. The exception to this general rule is in the treatment of wall tops in ruins.

Specification of repair mortars for structural conservation

It should always be remembered that the performance requirements for a repair mortar in a ruined structure are likely to be significantly different from those of the original mortar. Additionally, a range of repair mortars may be needed to meet differing performance requirements in different parts of the structure. Therefore, a simple like-for-like replacement of an original mortar may not be suitable.

While the bedding and core mortar in a masonry ruin will normally be of earth, earth-lime, air or feebly hydraulic lime mortar, an accurate match of the same mortar today will not harden and become durable if used in areas where the mortar is liable to stay wet. This is especially true of wall tops.[25] The Smeaton Project by English Heritage found that the only durable mortar for use on exposed wall tops was 1:1:6 cement-lime mortar.[26] However, it is equally known that the same cement-lime mortar causes accelerated decay of the masonry units, and is incompatible with wall sides. It is now generally appreciated that the masonry units in exposed wall tops/weathering details are essential ‘wearing surfaces’, of much more finite life expectancy than the masonry below which they shelter. Accordingly, conservative repair to these aspects of masonry buildings should focus on maintaining effective function, rather than absolute preservation of all fabric.

To illustrate the difference in context, Figure 59 presents an arrangement with an exposed wall top consolidated with a waterproof (but not sacrificial) mortar. Below this hard protection the wall sides are then repointed with a wicking traditional lime mortar to draw water and salts out of the body of the wall below. It should be noted that this is an example used to illustrate the principle only and other options for the capping of exposed walls are available.

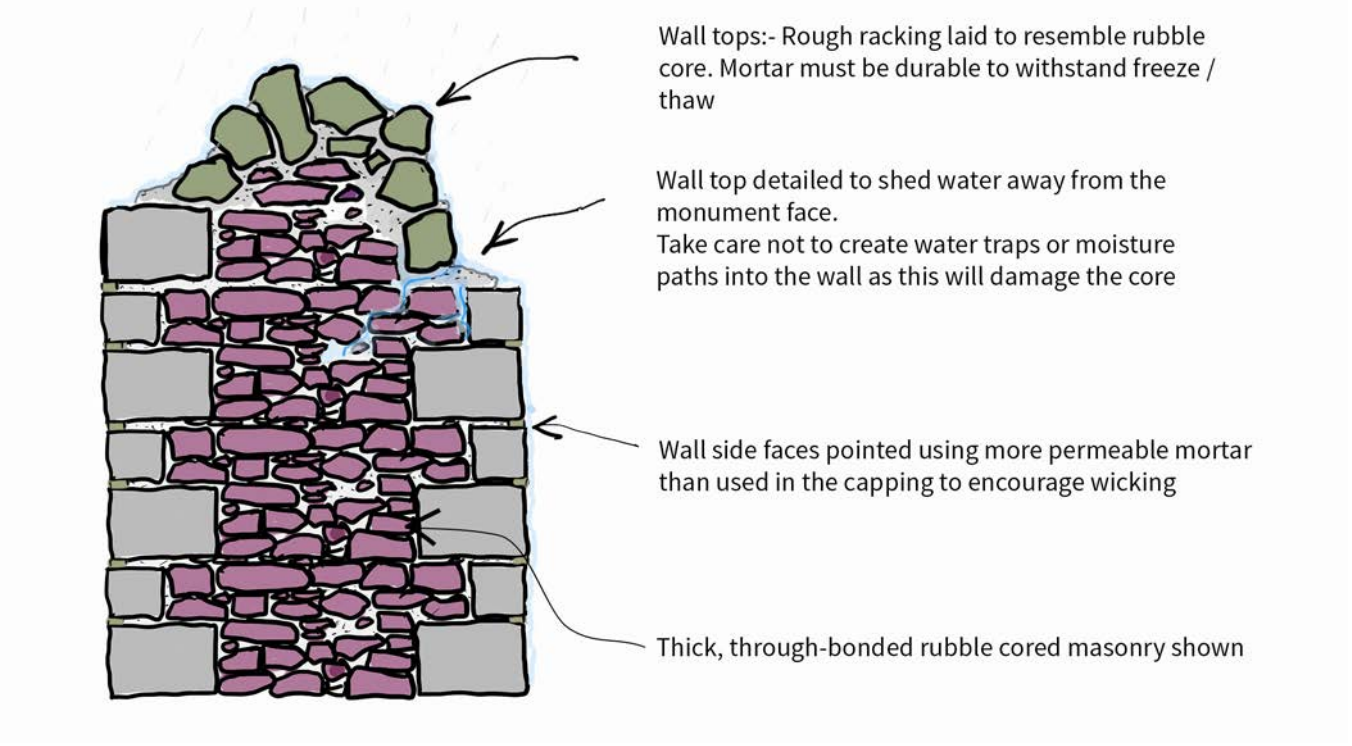


Figure 59: Example of detailing the masonry for structural conservation, with attention to context of different mortars for different contexts but also highlighting the significance of weathering details.

Neither mortar is suitable for the other's purpose. It is important to be honest as to the functional requirements and purpose of a mortar: for hard capping, the goal is to create a durable waterproof 'lid', which diverts water off the monument. Such a mortar cannot wick water and therefore cannot behave sacrificially to preserve the masonry units.

Sacrificiality of lime mortars

Traditional lime mortar wicks water from masonry walls by capillary drying. The mortar has a microstructure which is well suited to this process[1]. The lime in the binder is responsible for the development of this special pore structure, assuming that the lime type is air or feebly hydraulic and the mix is lime-rich. High proportions of porous aggregates can strongly influence the pore structure (Figure 61).

The outcome of this wicking behaviour is that soluble salts are gradually transported out of the masonry units through the mortar joints (Figure 60). Salts are the principal agent of decay in masonry buildings in the UK[2]. When in solution salts are harmless, but at the long-term drying front they crystallise and can damage the fabric.

The sacrificial weathering of traditional lime mortar is a direct consequence of this wicking behaviour. Thanks to its fine microstructure, air lime mortar makes a very effective poultice to a wide range of traditional masonry materials.

After decades or centuries of drawing harmful salts and moisture out of the walls, the mortar may have worn back into the depth of the joint, much like the tread on a worn tyre. Periodic repointing is needed to replace missing structural material and reinstate the full wall profile for the purpose of carrying load. At the same time the 'depth of tread' for the future sacrificial protection of the masonry units is refreshed so it can continue to stave off decay.



Figure 60: A pronounced visual example of the sacrificial weathering phenomenon associated with traditional lime mortars, which normally goes unseen, as the mortar draws the harmful salts out of the masonry over time.

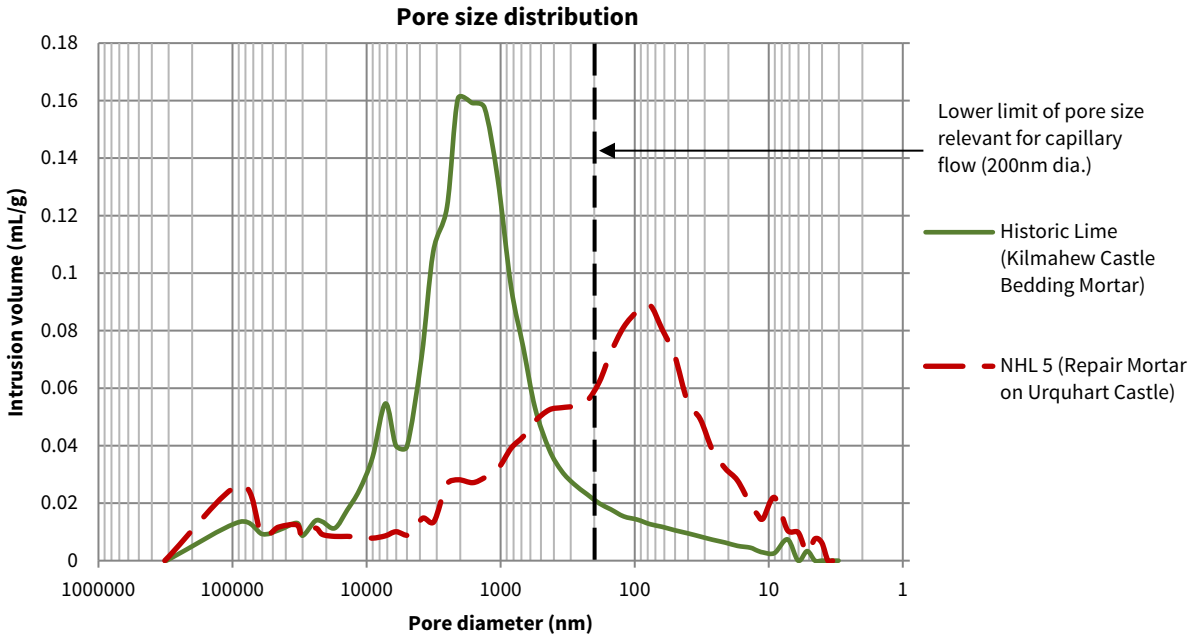


Figure 61 Comparison of pore size distributions of traditional hot-mixed lime mortars found in historic structures (green solid line) with modern NHL repair mortars (red dashed line). There is significant difference between the two mortars. The historic mortar binder is air lime, extremely porous and highly conducive to capillary drying, whereas the modern mortar is of denser microstructure, which reduces capillary flow line.

6. Numerical assessment

Applying modern structural design codes to historic ruins is not straightforward, as already demonstrated in section 5. Where the wall loading is environmental and where the structure has not substantially changed for a significant period, is not leaning significantly and has been well maintained, stability analysis is not necessarily required. However, there will be some instances where a stability assessment will help make informed choices about conservation of a ruin (Figure 62).



Figure 62: Isolated pier following demolition of bridge deck. © Paul Evans.

Mechanisms of failure

The most common forms of failure of a freestanding wall are unravelling (Figures 63 and 64) and face collapse (Figure 21). As previously discussed, these result from poor maintenance or unsympathetic historic repairs and are often triggered by water build-up in the wall core. This is difficult to assess numerically but where surveys show signs of deterioration, repairs will be required.

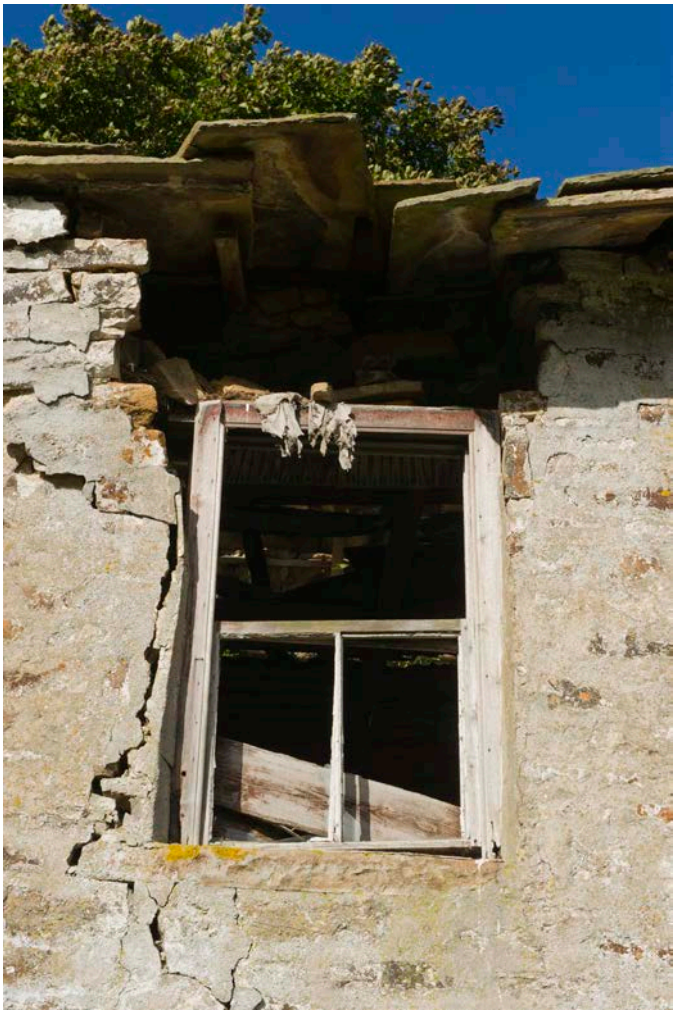


Figure 63: Unravelling of masonry in an unoccupied, unmaintained house.



Figure 64: Unravelling of freestanding wall.

The mechanisms of failure most suited to numerical assessment are overturning, sliding (shear) or bending. Of these, the most likely form of failure of a laterally unrestrained, coursed masonry wall is parallel to the bed joint. This appears as horizontal cracking, generally at low level or just above a change in thickness. Sometimes, it may appear as crushing of masonry on the face of the wall.

Tension stresses occur if the centre of load falls outside the 'middle third' of the wall section; since mortar is relatively weak in tension, this may lead to cracking as the load redistributes. The wall does not become unstable until the centre of load falls completely outside the section.

Where a ruin is laterally restrained by a return wall, buttress or pier, shear failure may occur at the interface with the returns, particularly if the two are not well bonded. Long wall panels might suffer bending failure perpendicular to the bed joint at mid span, or a combination of failure modes may lead to failure parallel to the bed joints at mid span combined with diagonal cracking nearer returns.



Figure 65: Ruinous wall buttressed between openings by steel frame.

The current code of practice makes the characteristic flexural strength perpendicular to the bed joint (horizontally) approximately three times greater than that parallel to the bed joint (vertically). In the case of random rubble walls and, to a lesser extent, dressed natural stone walls with core work, it intuitively feels that the wall should be more isotropic than this, enabling more lateral load to be carried vertically, where it can more readily be accommodated due to self-weight reducing the flexural tension from the lateral load. This intuition is reinforced by the work of Mundell et al[27] which shows that unbound, brittle drystone retaining walls are ductile and have tensile strength through the interlocking of their stones.

It should be borne in mind by the engineer that the standing ability of a wall is not necessarily a one-dimensional problem and the ability of a wall to span in two directions i.e. laterally within itself, should be considered where appropriate.

Loading

The loadings which might be applied to a ruin are:

- self-weight of the ruin;
- environmental loads (e.g. wind);
- lack of verticality;
- variable actions; where no floors or roofs remain, this is generally not significant.

Wind loading

Using the Midland's as an example, the Met Office description for the regional climate[28] notes: 'Wind speed is sensitive to local topographic effects and land use. Places sheltered by hills, or in extensive wooded or urban areas will have lower wind speeds and fewer days of gale.'

Whilst Eurocode 1 – Action on Structures – Wind Actions (BS EN 1991-1-4)[29] does make some allowances for topography, in a rural location (where ruins are often located) it does not allow for land use or shielding as there is a perception that these may change over the design life of a building. However, in a protected heritage setting, these are less likely to change, and there is merit in measuring the wind speed and direction directly rather than relying on the code of practice, which provides a more conservative assessment.

The velocity can be compared to the Met Office data for the locality (taking care to compare either the hourly wind speed or gusts) to ascertain the correct allowances for the site and extrapolating to adjust the fundamental value of basic wind velocity ($v_{b,0}$ in BS EN 1991-1-4[29]) to a site and height-specific value, this time taking care to use the ten-minute mean windspeed. The relationship between gust length and velocity was defined in the 1960s by the Durst curve.[30]

The age of many ruins and length of time they have stood as ruins means it is likely that they will already have experienced higher wind loads than current design codes specify, calculated on a 50-year return period. They are therefore proved in service and should not need numerical analysis in the absence of change.

Lack of verticality

Lack of verticality of a ruin may be due to the original construction being built out of plumb, partial settlement of foundations or other influences such as jacking by tree roots.

When the centre of gravity is in the middle of a wall it means that the stress at the base of the wall (or bearing stress) from self -weight is rectangular. As the centre of weight moves away from the wall centroid the stress block shape changes from rectangular to trapezoidal, reducing to a triangular profile when the centre of gravity is one sixth of the wall width away from the centre.

As the centre of gravity moves into the outer third of the wall, tension cracking occurs in the mortar and the width of the triangle reduces. The peak stress at the compression face increases to accommodate the width reduction. The wall stability becomes more sensitive to changes in applied load or ground conditions as the centre of weight moves closer to the compression face of the wall. When the centre of weight goes beyond the face of the wall, stability is lost and in the absence of stabilising action it is at risk of toppling (Figure 66).

In ruins, the eccentricity can be measured directly; the induced overturning and restoring moments can be calculated using this rather than using the design contingency outlined in cl. 7.3 of BS EN 1996-1-1[22] and cl. 6.1.2 Recommendations for the design of masonry to BS EN 1996-1-1 and BS EN 1996-2 (PD6697)[32].

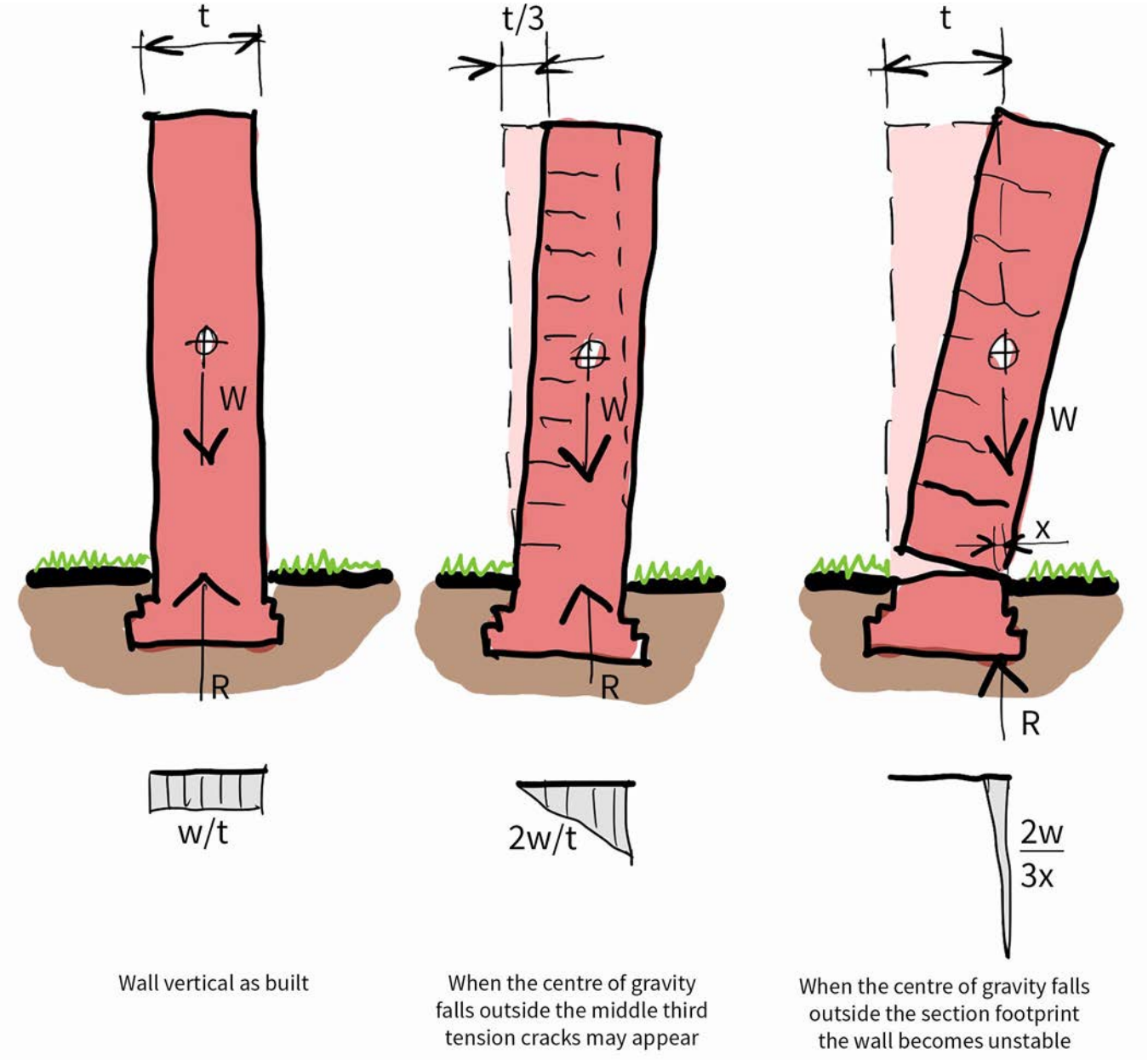


Figure 66: Distribution of stresses at the base of a leaning wall.

Ultimate limit state assessments

Ultimate limit state assessments are used to demonstrate that a structure will not fail under the extremes of potential loading. The ultimate limit state must be satisfied to ensure the safety and stability of the structure. In modern structural analysis, partial material factors and partial safety factors are applied to material capacities and loading to ensure that the probability of failure remains low.

Eurocode 6 BS EN 1996-1-1[22] is the current code of practice for the design of reinforced and unreinforced masonry structures. When considering the analysis of a freestanding masonry wall the 'tilting' or 'toppling block' approach for stability may be better suited than the standard formulae for bending and shear failure provided in BS EN 1996-1-1[22]. This approach shown in Figure 67 is described in more detail in section 36.5 of Design guide 10[33]. This approach has been updated to align with BS EN 1996-1-1[22] in Figure 5.24 of the Manual for the design of plain masonry[34], although the latter manual lacks the narrative of Design Guide 10.

A more detailed assessment considering interlocking of the masonry units can be undertaken by discrete-element modelling, but the software required for this type of modelling is currently expensive and only specialists or larger engineering practices (who more generally use it for geotechnical analysis) tend to have access to it. Therefore, unless there is likely to be a significant saving or reduction of intervention from the use of discrete-element modelling which would cover the cost of the analysis, potentially by a specialist sub-consultant, it is likely to be beyond the scope of most project briefs and budgets.

Usually, the critical ultimate limit state for ruins is when wind load is the lead variable action. However, most ruins have been standing in a ruined state for considerably longer than the design return period of 50 years used in modern wind loading codes. If a numerical analysis is demanded, and if by calculation, the ruin lacks sufficient assessed capacity to withstand the theoretical applied loading, then a back calculation can be made to ascertain both the calculated 'return period' that can be withstood and/or the maximum wind speed. This wind speed can be compared with historical data, ensuring the same duration for wind velocity or gusts as discussed above.

This will help the client assess whether a managed approach to risk can be adopted, for example by closing the site when high winds are expected. These precautions may be necessary anyway because of risks from flying debris and the increased risk of falls in high winds.

Another option is to combine wind monitoring (discussed above) with monitoring the plan movement of the wall tops, to ascertain real-life loading and movement, eliminating assumptions about wind speed and materials.

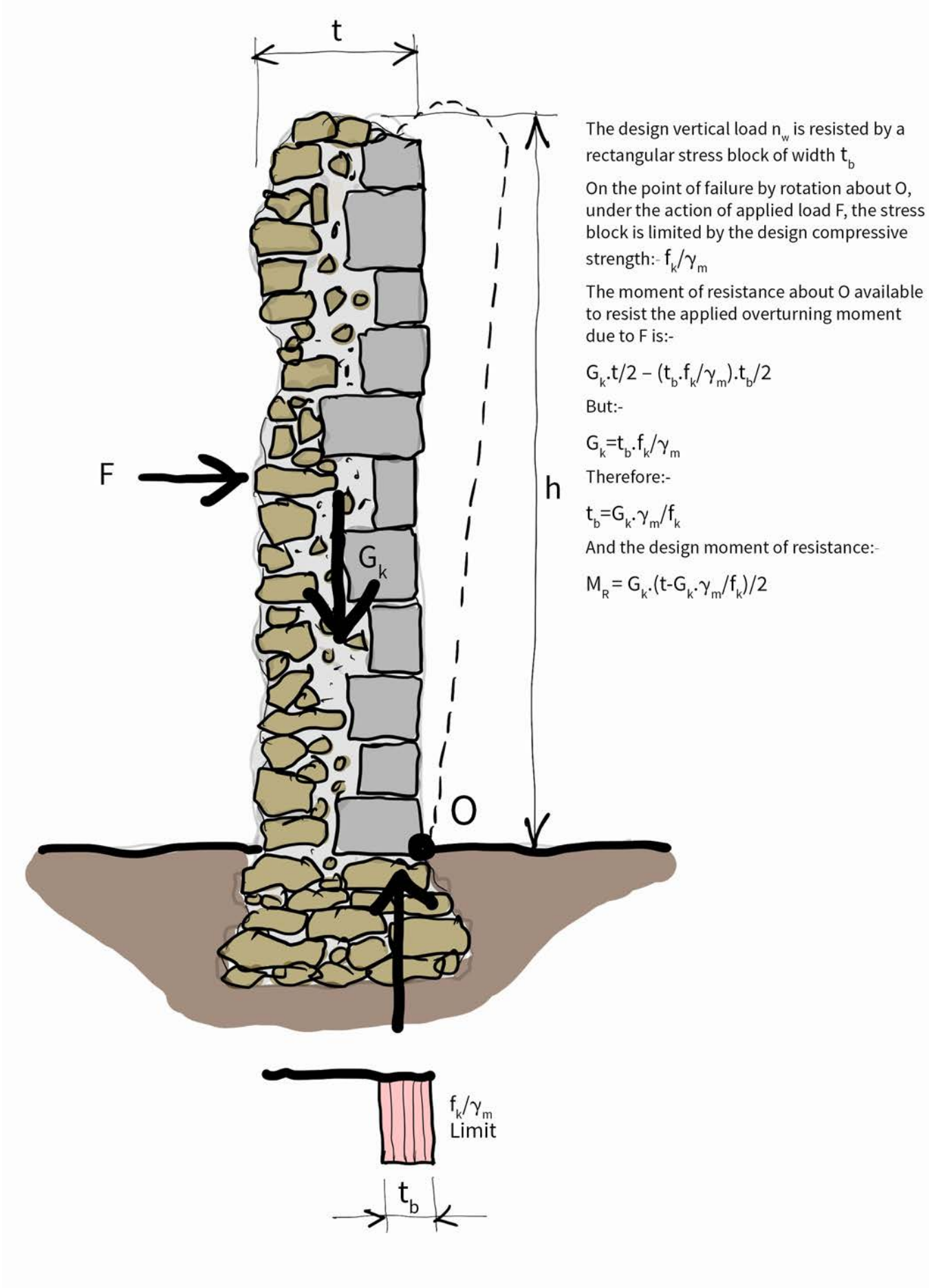


Figure 67: Design moment of resistance (M_R) of a free-standing wall without flexure.

Material factors of safety

Material factors of safety are applied by engineers during the design of structures to take account of the potential uncertainty in material properties and construction quality.

Section 4.6.3 of The Appraisal of Existing Structures[8] states: 'with sufficient onsite testing work the engineer may review the [material]… partial safety factors.' For many ruins, where budgets are constrained, this approach – while likely to be beneficial – is unlikely to be followed and the advice in section 5 should be adopted to ensure the selection of realistic material properties.

Section 6.2.5 of The Appraisal of Existing Structures[8] goes on to state that γ_m includes allowances for deviation from the design, future cutting of chases and raking out of joints prior to pointing. With a ruin of known thickness, it should be possible to assess whether it has historically been chased (and whether it is likely to be in the future) and the reduction to γ_m if the ratio of chase to wall thickness is less than the assumptions described in this clause, ensuring there still remains sufficient residual allowance for re-pointing.

Serviceability limit state assessments

Serviceability limit state assessments are used to consider whether the structure will crack or deflect under normal in-service loads. The serviceability limit state is most relevant to buildings which are in use and often key to user comfort.

Ruins are, by definition, not 'in service' so it is not usually necessary or helpful to undertake serviceability checks.

In the unlikely event that serviceability checks are felt necessary, BS EN 1996-1-1[22] cl 9.2 states 'in unreinforced masonry structures, the serviceability limit state for cracking and deflection need not be considered separately when the ultimate limit state has been satisfied… provided its dimensions are limited [to those] obtained in Annex E.' With a ruin, there is a real possibility that the dimensions are outside those stated in annex F and may need to be assessed by other means.

Lateral loading on freestanding ruinous masonry is predominantly instantaneous. Creep does not need to be considered, except when considering verticality, earth pressure and the jacking force of roots.

Overall factors of safety

When assessing existing ruins through calculation careful consideration must be given to an acceptable overall factor of safety. On a managed site a factor of safety of one may be acceptable providing regular inspections are carried out and site staff are on hand to notice and report any changes. However, on an unmanned, remote site which is visited by tourists but inspected less regularly by building professionals or asset managers a higher factor of safety may be deemed necessary. Where a structure has a factor of safety approaching one, this should be clearly communicated to the building owner or asset manager along with the associated risks so that an informed decision on the need for intervention can be made.

7. Repair methods

What to consider when specifying a repair

When considering an intervention, the principles of conservation[35] should be observed. Through detailed survey, monitoring and calculation it may be possible to prove that no work is required, justifying the cost of additional upfront work by an experienced conservation engineer [competency is discussed at the beginning of section 3].

If a structural repair is needed, several points must be considered:

- The repair material must be compatible with the original structure and not cause further damage.
- Using an alien material may be preferable if it reduces loss or disturbance of historic fabric but should be applied with caution and only where the material has been proven in practice.
- The repair should be reversible or re-treatable wherever possible.

An engineer must understand the cause of damage before specifying a repair. Implementing an inappropriate repair without establishing the cause of damage could exacerbate the deterioration, possibly in areas which have so far been unaffected.

The type of repair chosen may depend on the significance of the structure, and on which aspect of the structure has most heritage value. For example, the significance may lie in the fabric itself or it may lie in its current form. The local Conservation Officer or Inspector of Ancient Monuments will be able to advise on this.

Approach and conservation principles

Ruins are often of great historical and archaeological significance. They may be designated as Scheduled Monuments, providing protection under law. Other protective designations may also apply, including Listed Buildings, Registered Park and Gardens, Sites of Special Scientific Interest and Registered Battlefields.

There is legislation protecting ancient monuments in all jurisdictions of the UK. In England and Scotland, the key legislation is the Ancient Monuments and Archaeological Areas Act 1979, in Wales this is the Historic Environment (Wales) Act 2023 and in Northern Ireland this is the Historic Monuments and Archaeological Objects (NI) Order 1995. Further information can be found on the website of the relevant national body.

Before selecting or designing a repair, the heritage values and significance of the ruin, as a whole and in its details, and the impact that repair options will have on them, should be understood. Options should be proportionate to the identified risk and should consider whether the causes of deterioration can first be removed, reduced or managed. The likely benefits, harm, uncertainties and future maintenance implications of each option should be compared before a preferred approach is selected. Decisions should be reasonable, transparent and adequately documented.

The aim when working on a ruin should be to protect and stabilise it to slow further deterioration. The presumption under the legislation is that whilst some deliberate change may be acceptable, the monument will be handed down to future generations in much the same state, or better, than it was found.

Minimum intervention is the primary aim of good conservation engineering, and this must always be kept in mind. If a ruin has remained stable and without defects under its current applied loading, then it is not usually necessary to prove its capacity by structural calculation.

To avoid potentially harmful interventions, any structural defects should be investigated thoroughly to understand the cause(s) before any repairs are carried out.

Using compatible repair materials will help prevent future damage to and maintain the significance of the structure. Typically, stones and mortars should be matched to the existing materials unless there are changes to the exposure and condition of the masonry which may require alternative materials to be selected. Where new materials, such as metal or concrete, are necessary they should be chosen so as not to corrode or react with the ruin.

It should be noted that Scheduled Monuments are exempt from most Building Regulations[31] in England and Wales. Where ruins are not scheduled, repair or like-for-like replacement can be made without applying for Building Regulations approval. Advice should be sought from the relevant building control body if there is any uncertainty.

Any intervention must obviously maintain or enhance the stability of the structure. Any deviation from the regulations, in terms of access for example, should be discussed with the building owner or manager to ensure the consequences are properly understood.

Further information on Historic England's approach to repair can be found here:
historicengland.org.uk/advice/technical-advice/buildings/principles-of-repair-for-historic-buildings

Further information on Cadw's approach to repair can be found here:
cadw.gov.wales/advice-support/historic-assets/conservation-principles-action

Further information on Historic Environment Scotland approach to repair can be found here:
historicenvironment.scot/advice-and-support/your-property/looking-after-your-property/repair-of-traditional-buildings/

Further information on Northern Ireland's Department for Communities approach to repair can be found here:
communities-ni.gov.uk/articles/repair-and-maintenance-guidance

Consents

Any work on a Scheduled Monument is likely to need consent; this includes intrusive investigations, fixing monitoring equipment to walls, and repair works. Early pre-application engagement should be sought with the responsible authority. For English Scheduled Monuments contact the local Historic England Inspector of Ancient Monuments, and refer to the National Heritage List for England online to find out what protections apply to the site.

For further information on Scheduled Monument Consents in England refer to: historicengland.org.uk/advice/planning/consents/smc

For Scheduled Monuments in Wales, contact the local Cadw Inspector of Ancient Monuments and refer to Cof Cymru (National Historic Assets of Wales) online to find out what protections apply to the site.

For further information on Scheduled Monument Consents in Wales refer to: cadw.gov.wales/advice-support/historic-assets/scheduled-monuments/scheduled-monument-consent

For monuments and listed buildings outside of England and Wales, refer to the relevant national body's website.

Listed buildings will also require consents and the Local Planning Authority's Conservation Officer should be contacted at the start of the project.

Repair techniques

The following section provides examples of repairs for common defects in ruinous structures. Generic repair techniques should only be used after proper consideration of the unique site-specific conditions of any defect.

Structural repairs should never be carried out in isolation. Water and vegetation are the most common causes of structural defects in ruinous structures. An architect, building surveyor or structural engineer experienced in building conservation should review the structure holistically and advise on any remedial measures. Wall capping, consolidation or mortar repairs should be carried out in tandem to prevent ongoing deterioration and recurrence of the same defects.

Support for overhanging masonry

This may be the replacement of a support which has decayed or deteriorated in condition or the introduction of support following localised collapse.

Repair options include:

- Stainless steel bar/plate/beams that may be hidden or exposed. A hidden repair requires good record keeping, preventing unnecessary future intervention, but an exposed repair may have a greater visual impact (Figure 68). Phosphor bronze may also be specified, provided there is certainty as to its properties to ensure it is sufficiently stiff and does not result in staining of the historic fabric from corrosion products.



Figure 68: Stainless steel plate repairs at Farleigh Hungerford Castle, Somerset.

- Timber lintels and plates are common on the formerly internal face of ruined walls; in an exposed environment such timbers will be subject to decay (Figure 69). Timber lintels may be replaced like-for-like, possibly incorporating leadwork to slow deterioration. In areas that are difficult and costly to access or where decay is likely to happen more quickly due to shade or driving rain, they can be replaced with precast concrete.



Figure 69: Inaccessible timber lintels at Kenilworth Castle, Warwickshire.

- Corbelling new masonry is an alternative to providing support with a new or alien material. Stone is often more durable than steel or timber, but the visual impact may be greater (Figure 70).



Figure 70: Corbelled masonry repair at Bindon Abbey in Dorset.

- Pinning or securing loose or overhanging stones can be done with stainless steel helical ties, wires or threaded bars. Fixings are usually secured to the stones using grout or resin, so such repairs are not easily reversible (Figures 71 and 72).

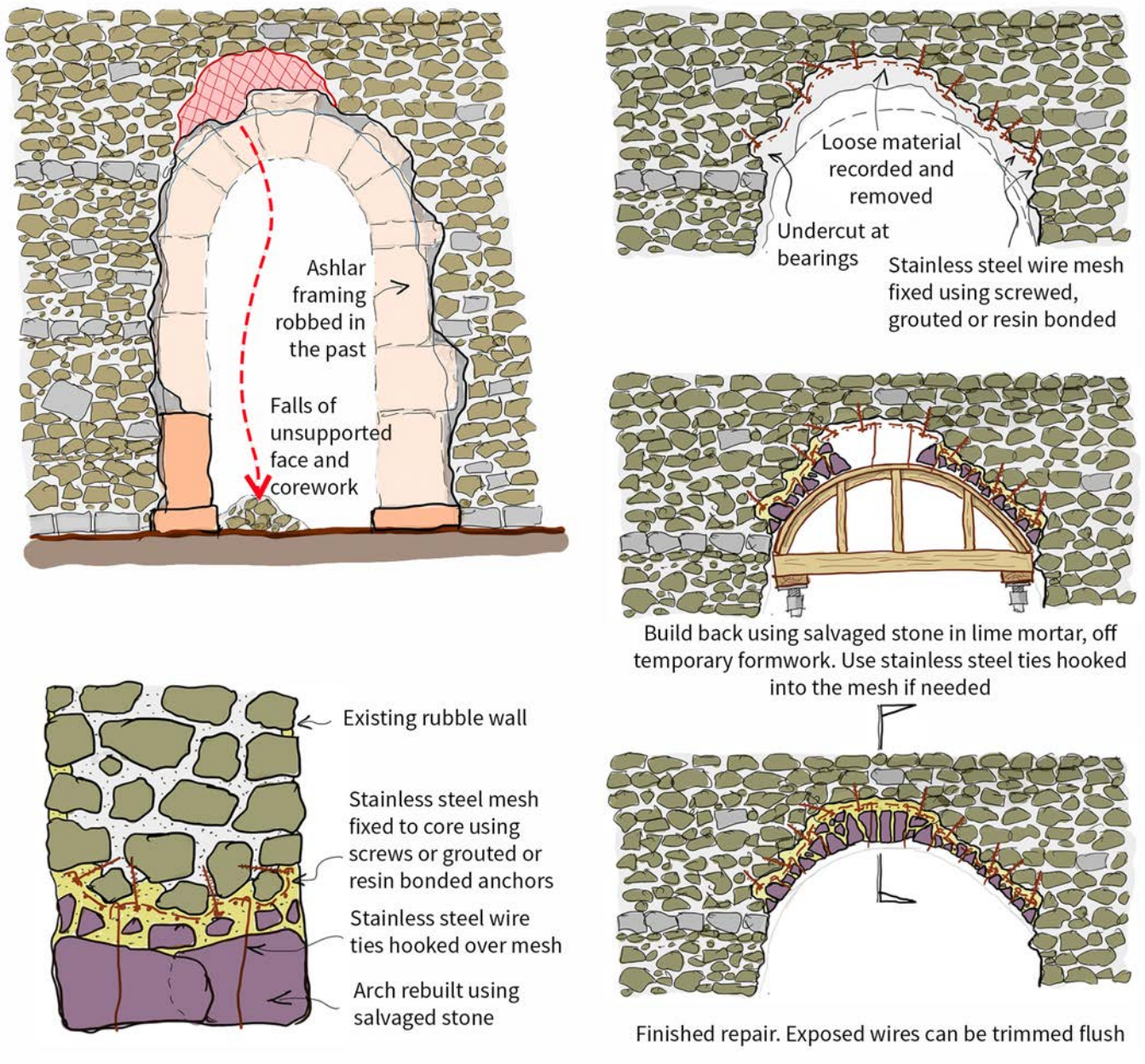


Figure 71: Overhanging flint masonry repair.

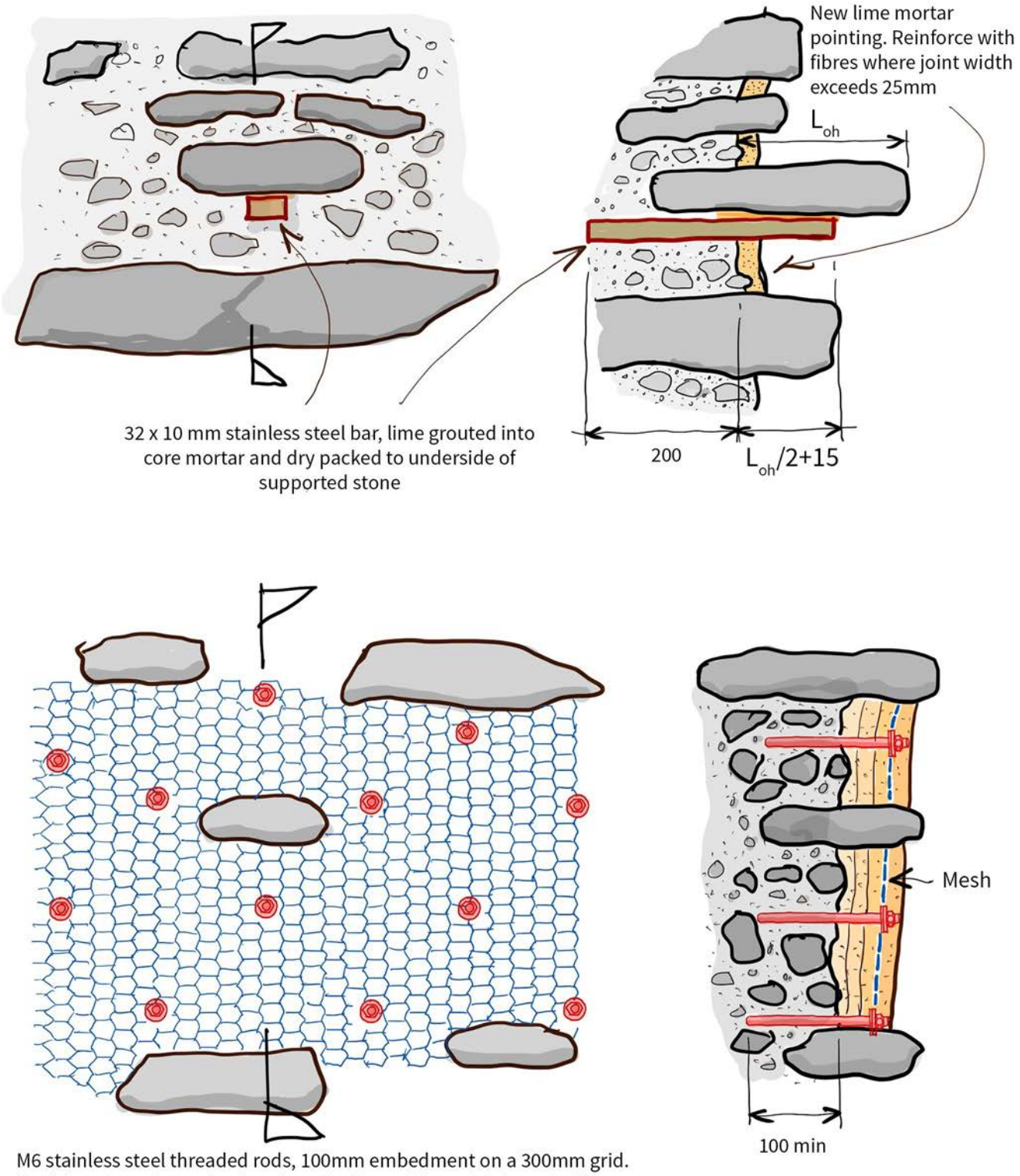


Figure 72: Overhanging stone masonry repair.

Bulging or separation of facing masonry

Ruined masonry walls are often made up of two coursed facing skins and a rubble masonry core. Where the facing masonry has separated due to voiding of the core material, a repair may be required to reinstate its connection (Figure 73).

Repair options include:

- Tying facing stones: proprietary sock anchors or helical bars may be used to stitch the facing stones. Ties are often installed in a grid at regular centres.
- Grouting: grouting of the core with an appropriate lime-based grout may be used to fill voids, reinstate connection and prevent further movement. Stitching of the facing stones or temporary support may be required to resist the lateral forces imposed by grouting. Care should be taken when flushing through with water prior to grouting if internal surface finishes are present.
- Introducing new bonding stones: this is a more invasive repair than using proprietary stitching as historic fabric will need to be locally broken out and rebuilt to incorporate new stones.
- Local dismantling and rebuilding.



Figure 73: Masonry collapse of one face.

Leaning walls

Section 6 sets out methods of assessing the stability of freestanding walls. If this assessment shows that the wall is unstable the cause of the movement should be identified and, if appropriate, remediated before the wall is stabilised.

Repair options include:

- Jack the wall back into position: this may be an option for isolated free-standing walls and may need to be undertaken in conjunction with underpinning or foundation work.
- Take down and rebuild: if the form of the remaining structure is more important than the historic fabric, taking the wall down to a safe height and rebuilding may be the least intrusive option (Figures 74 and 75).
- Return wall ties: masonry anchors, concrete elbows or local rebuilding and toothing in may be used to introduce or reinforce restraint from perpendicular walls.
- Add structure to allow the wall to span horizontally: the wall may be reinforced with masonry anchors or helical bar to allow it to span horizontally as well as vertically. Head restraint may be added to the top of a wall to provide additional support between returns. Ring beams can be constructed across the wall head between points of restraint to allow the wall to span vertically.
- Add buttresses: buttresses may be added in masonry, concrete or steel. Buttresses are likely to have a greater visual impact than other repair methods but have been commonly used on ruins for many years (Figures 65, 76 and 77).





Figure 74 (left): Leaning wall at Tintagel Castle island ward, Cornwall.
Figure 75 (right): Wall at Tintagel Castle island ward, Cornwall, following rebuilding.



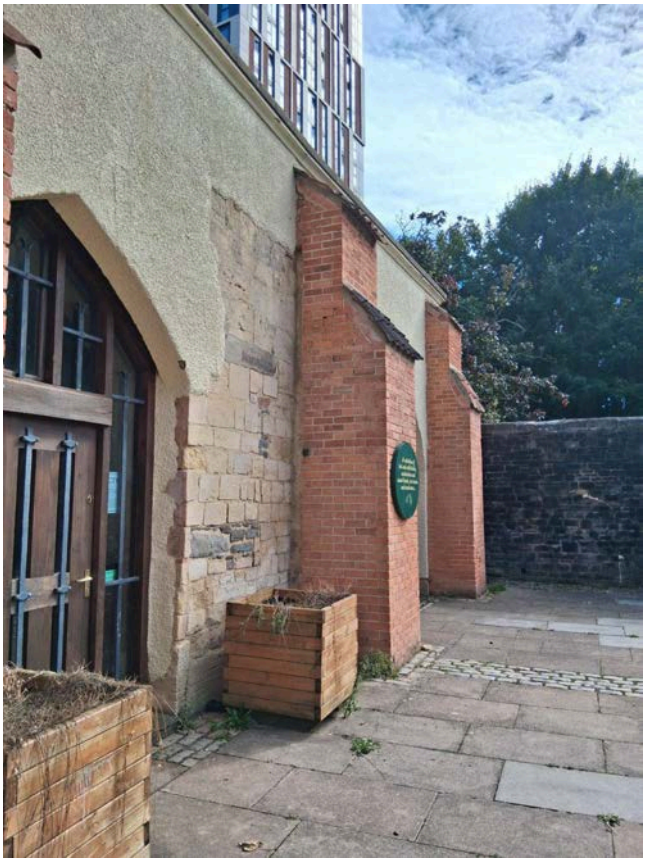


Figure 76 (left): Historic stone buttresses at Lindisfarne Priory, Northumberland.
Figure 77 (right): Brick buttresses added to historic stone wall of the Vaulted Chamber, Bristol Castle, Bristol.

Crack repair

Masonry has little tensile strength so cracks usually indicate that the structure is experiencing tension perpendicular to the line of cracking (Figure 78). Cracking can occur for many reasons, and any repair will be specific to the type and direction of movement. Repairing cracks without establishing the cause of distress is likely to result in failure of the repair and may even lead to accelerated damage to the structure.

Repair options include:

- Repoint with no further intervention: if movement monitoring demonstrates that the cracking is not progressive or the cause of the movement has been remediated and the scale of the movement that has already occurred does not adversely affect the building, it may be possible to repoint the crack without any further intervention.
- Tying or stitching with helical bar: if masonry units are fractured, the crack has reduced the ability of the wall to span or further opening may allow water into the core, stainless steel helical bars can be used within the mortar joints to add tensile resistance (Figure 79). This type of repair may be more difficult to implement in ruined masonry which may not have consistent coursing. Helical bars are flexible and are suited to situations where they may only need to act in tension. They are not suitable where bending strength may be required.
- Tying with masonry sock anchors: if the scale of the crack is such that a significant intervention is required to reinstate the ability of the wall to span, masonry sock anchors can be installed across the crack within the thickness of wall.

- Local rebuilding: cracked units can be removed and replaced to reinstate the structural action of the original masonry. This will result in a greater loss of historic fabric than other methods of stitching.



Figure 78: Historic cracking to the keep at Longtown Castle, Herefordshire.



Figure 79: Masonry crack stitching with helical bar at Framlingham Castle, Suffolk.

Temporary repairs

In some cases, an urgent temporary repair may be necessary to prevent an immediate collapse. Examples include scaffolding or shoring to prevent the loss of facework on a bulging wall or lorry/ratchet straps to hold unstable masonry or a leaning wall in position (Figures 80, 81 and 82).

Safe working access should be considered at an early stage in the project. This can involve installing temporary works and, in some cases, local deconstruction of unstable elements before repair works can begin.



Figure 80: Temporary shoring to a bulging masonry wall.



Figure 81: Shingle used to prevent failure of an overhanging masonry corbel at Hurst Castle, Hampshire.



Figure 82: A temporary scaffold frame erected to support overhanging facework.

In England, if the ruin is a Scheduled Monument there is a provision within The Ancient Monuments (Class Consents) Order 1994 to allow works which are urgently necessary in the interests of health and safety to be carried out without applying for Schedule Monument Consent under 'Class 5', provided that the works are limited to the minimum measures immediately necessary (such as temporary propping) and notice and justification is provided to the Secretary of State as soon as is reasonably practicable[37]. The local Inspector of Ancient Monuments at Historic England will be able to provide further advice.

This does not apply to scheduled monuments in Wales, where carrying out unauthorised works to a scheduled site (i.e. without consent, or not in accordance with any conditions attached to consent) is a criminal offence. However, the Historic Environment (Wales) Act 2023 allows a defence of unauthorised works where they are taken to address urgent health and safety needs. They must be limited to the minimum measures immediately necessary to secure health and safety, and notified to Cadw, together with a justification of measures taken, as soon as reasonably practicable.

Specification of repair mortars for structural conservation

Mortars used for the structural conservation of ruins should be designed for the particular application and site conditions. Selection should take account of:

- type, condition and significance of the masonry and existing mortars;
- function and extent of the repair;
- exposure and moisture pathways;
- wall geometry, wall-head treatment, drainage and water-shedding details;
- required properties of fresh and cured mortar;
- compatibility with adjacent materials;
- appearance;

Mortar design should also take account of the practical constraints on the works to be carried out, including:

- availability of required materials and skills;
- access;
- season and weather conditions;
- temporary protection to facilitate curing and controlled drying;
- programme and cost;
- available resources for future inspection, maintenance and review.

Material analysis of existing mortars can be helpful in characterising binder type, binder/aggregate ratios and aggregate type and grading. Results should be interpreted by someone with appropriate knowledge and experience of historic mortars. It should always be remembered that the performance requirements for a repair mortar in a ruined structure are likely to be significantly different from those of the original mortar. Additionally, a range of repair mortars may be needed to meet differing performance requirements in different parts of the structure. Therefore, a simple like-for-like replacement of an original mortar may not be suitable.

Sacrificial mortar

The purpose of repointing is not just to replace missing structural material and reinstate the full wall profile for carrying load, but it is to achieve this structural function whilst at the same time refreshing the 'depth of tread' for the future sacrificial protection of the masonry units to prevent continued decay.

The importance of using compatible mortars in structural masonry conservation cannot be over-stated.[19] Using incompatible mortars (for example, impervious wall top mixes on wall sides) causes accelerated decay and can lead to loss of the masonry units. This is because heavily hydraulic binders are microstructurally dense, non-wicking, and typically exhibit poor bond intimacy (further preventing water transport); these factors together displace the harmful drying front into the masonry units, where salts then precipitate and damage the fabric (Figure 83).



Figure 83: The consequences of using an incompatible mortar on a traditional masonry structure. Note the salt accumulation in the bricks, which are severely decayed, while the mortar protrudes from the weathered face.

8. Major structural interventions

This guidance is not intended for use on major structural interventions. Major interventions need a bespoke design to satisfy specific requirements, all of which will need to be agreed with the Inspector or the local planning authority as appropriate.

What constitutes a major intervention cannot be rigidly defined but encompasses techniques such as underpinning with piles or with concrete footings, permanent propping with steel or concrete, and any other repairs with modern materials that make significant changes to the historic fabric.

The aim of this guidance is to prompt designers to explore whether a major intervention is needed and appropriate, or whether it could cause more damage to the historic structure than allowing part of that structure to gracefully subside or collapse (Figure 84).

Archive evidence shows that many historic ruins were repaired or partially rebuilt in the late nineteenth and twentieth centuries. Some of the repairs were very successful and go unnoticed today, having become part of the history of the ruins themselves. Other interventions were less successful and are now regarded as misguided. In all cases there is a risk that observers are misled into interpreting features less than a century old as being much more ancient.

Ruins are, by definition, in a state of dilapidation, decay or arrested collapse. Some are so valuable that any risk of damage from a major structural intervention is far outweighed by the loss that would occur if the ruin collapsed. Some ruins pose a risk to the public or to neighbouring properties and in these cases, there may be no alternative to carrying out invasive repairs to make them safe.

Sometimes however we run the risk of creating a pastiche of historic fabric wrapped around a modern structure, solely for the sake of maintaining appearance. In those cases, we must question whether it is better to allow the ruin to continue its path of ruination provided it can do so safely.

These are not questions for a designer to answer in isolation. Wherever major interventions to ruins are being contemplated, the Inspector of Ancient Monuments or Conservation Officer should be consulted. Advice should be sought from other heritage professionals such as archaeologists and building historians in order to develop a holistic picture of the ruin and its setting, and to plan for its future management.

Sometimes ruins are brought back into occupation; if carried out sensitively the results can be spectacular and award winning (Figure 85). The principles of conservation philosophy should always be followed; such interventions should be reversible, bearing in mind that the ruin is usually far older than the design life of the new building. These projects should be honest and legible, use compatible materials and minimise damage to historic fabric. The design of any new elements should reflect and respect the history and significance of the ruin and its setting.



Figure 84: Major structural interventions are often controversial. They are best developed by engineers working in conjunction with other heritage professionals.



Figure 85: The ruined Banqueting House (Grade I) at Whitby Abbey was converted into a visitor centre in 2019 by inserting a new steel frame and concrete floors.

9. Checklist

Is the site a Scheduled Monument, Listed Building, Locally Listed or other heritage structure?

Repair works, temporary support and some monitoring systems will require consents. Make contact with the relevant local authority and Historic England (or equivalent national body) early to discuss proposals.

What record information is available about the site?

Owners, archives and online resources can help the engineer understand how the form and condition of a ruin have changed over time and what repairs may have been carried out. Refer to section 3 for possible sources of information.

Can an initial visual survey be carried out safely?

An initial visual survey is often carried out from ground level to gain an understanding of the condition of the ruin, the surrounding site, and the possible causes of any structural issues. If there is any concern about the stability of a ruin, this should be carried out from a safe distance. Temporary propping may be required to allow a closer inspection, for which relevant consents should be obtained.

What further surveys should be carried out?

A visual survey may not be sufficient for an engineer to be confident that a ruined asset has structural integrity. Other methods such as GPR, cover meter surveys and measured surveys may provide useful information. Refer to section 4 for further information. Refer to section 4 for further information.

What are the factors driving the decay/ deterioration of the wall?

Common causes include vegetation growth, water ingress, failure of previous interventions and load path alteration. Refer to sections 1 and 2 for further information.

What structural issues are visible?

Surveys may show structural cracks, leaning or bowing walls or unsupported masonry.

Are there concerns are ongoing structural movement?

Movement monitoring is useful to understand whether movement is ongoing, historic or seasonal. Refer to Section 4 for information on different types of monitoring.

Are there concerns about stability that require calculation?

Stability analysis may be required if the wall is leaning significantly, or the loading or support conditions have recently changed. Refer to section 6 for further information on calculations.

Are structural repairs required?

Through detailed survey, monitoring and/ or calculation it may be possible to prove that no work is required. Any repairs specified should be in line with conservation principles. Refer to section 7 and 8 for further information.

10. Case Studies

Bishop's Waltham Palace

Bridget Drake-Wilkes

Bishop's Waltham Palace was constructed in the 12th century as part of the original residence of the Bishop of Winchester. It was remodelled in the late 14th century and was ruined in the Civil War. Bishop's Waltham Palace has been a scheduled monument since 1934 and in guardianship of the state since 1952. Today it is operated by English Heritage as a free-to-access site. This case study looks at the north end of the west range, which housed the kitchen.



Figure 86: An aerial view of the Bishops Waltham site looking west.

The standing remains are random rubble, predominantly with flint "masonry units", with most square cut stones (at wall corners and openings) robbed since the building was ruined.



Figure 87: The south wall of the kitchen viewed from the south.



Figure 88: The east wall of the kitchen viewed from the east.

Structural concerns as to the adequacy of walls above unformed openings had been identified during a quinquennial inspection, with concerns as to the risk to the public of falling material. I was asked in 2018 to comment on these areas so that any engineering requirements could be incorporated into an upcoming, planned consolidation project.

Approach – south wall

As well as inspecting the ruin, we were in a fortunate position to have access to records of the site since it came into guardianship in 1954. Figure 89 shows that when the site came into guardianship brick arches existed above some of the larger openings. The brickwork was removed to enable the ruin to be better read as “unravelling”.

At the south wall opening (Figure 87) inspection of the arch soffit from the ground suggested not much material had been lost over the intervening 50 years and that whilst it would benefit from a fingertip inspection when the scaffold was erected it was anticipated that just a light de-scale of loose mortar and consolidation using lime mortar would be required.

At the east wall opening (Figure 89) the archive photos showed that the extent of facing stone had reduced since 1964 and inspection of the arch soffit from the ground suggested historic masonry repairs had been undertaken. The soffit appeared to be in poor condition with stones having limited embedment and cracks visible at the mortar bed. Again, a fingertip inspection was recommended when the scaffold was erected, and it was anticipated that just a light de-scale of loose mortar and consolidation using lime mortar would be required.

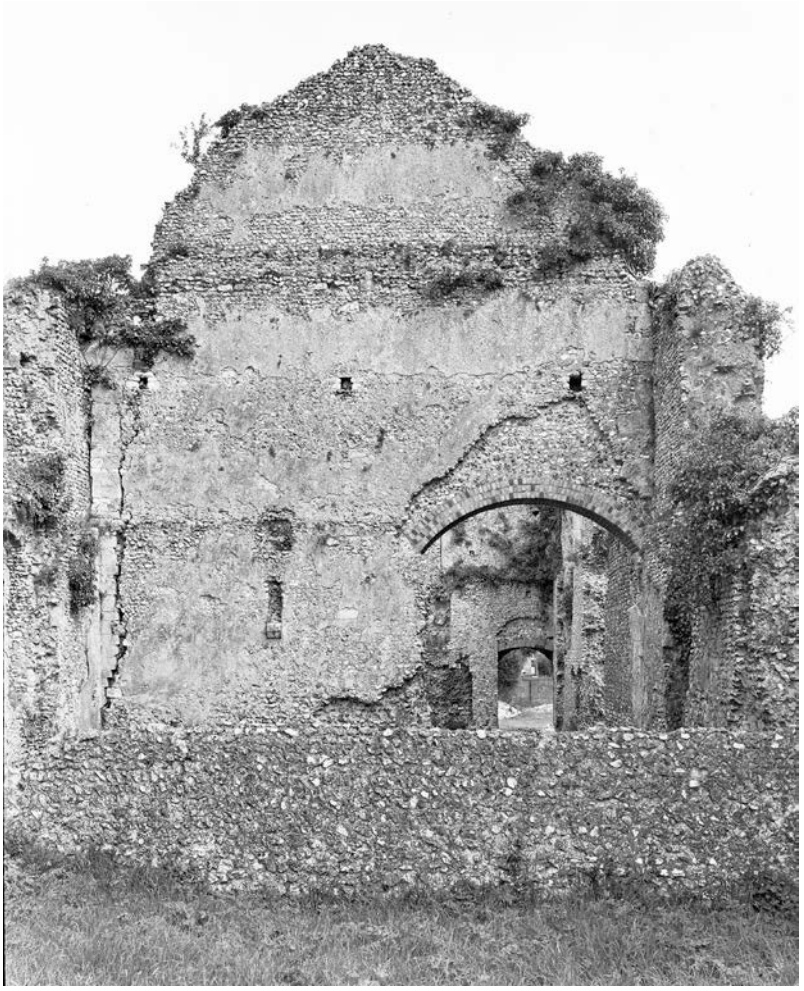


Figure 89: The south wall of the kitchen viewed from the south, with the brick arch in place (1956).



Figure 90: The east wall of the kitchen viewed from the east (1956).



Figure 91: The south wall of the kitchen viewed from the south shortly after the brick arch has been removed (1960).



Figure 92: The east wall of the kitchen viewed from the east (1964).

Conclusions and proposals

When originally constructed, it is likely that the opening had stone quoins or similar to form the opening. These historically have been robbed leaving an arch formed in flint core-work. Whilst globally the arches in core-work are stable, they are vulnerable to exposure to the elements. They are vulnerable to driving rain, saturation due to defects in the facing and wall head above, thawing after freezing causing damage to the pore structure of the mortar and local spalling of the mortar surface, which in time leads to individual stones being inadequately supported and falling from the arch soffit.

No numerical assessment was undertaken because there were no concerns about the overall stability of the walls.



Figure 93: The opening in the east wall of the kitchen viewed from the east prior to works.



Figure 94: Detail of figure 93.

Following fingertip inspection of the south wall arch soffit, light scaling and consolidation was appropriate.

On closer inspection from the scaffold of the east wall arch soffit, all exposed stonework was loose and the earlier mortar repairs had failed. There was a high risk of falling masonry from the soffit, posing a danger to visitors. The following options were considered:

- Do nothing – unacceptable due to the risk of falling material injuring people below
- Close off the area below – eliminate people from harm
- Descale – accepting further loss of historic fabric on a cyclical basis
- Descale and repoint– with this approach there is a risk that raking out sound mortar would destabilise individual flints increasing the risk of further loss of material but would create a dense surface of mortar reducing the frequency of future descaling
- Reinforced reinstatement – mechanically fix mortar and loose stones to the core to enable them to be held in position until mortar has cured
- Reinstate lost fabric – with centering to form the arch profile of the 1950s and the rebuild arches to be reinforced with metalwork
- Install new lintels – requiring pockets to be cut out and infill between the lintel and the current core profile
- Install a modern brickwork arch – as had existed elsewhere on the site in the early 20th century.

The selected approach was a reinforced reinstatement, using a methodology similar to shown in Figure 70, but without centering. The specification was 3mm stainless steel screws with washers, with 50mm embedment into the core mortar and 5 to 10mm projection, to enable stainless steel tying wire to be secured to the fixings in the zone between the washer and the core mortar to create a wire mesh, mortar was then worked into the mesh and individual flints below tied off the mesh to provide support until the mortar cured. An over render was then applied where the shape of the stones dictated this was the best solution.

This approach retained the maximum amount of historic structure, was visually unintrusive and was anticipated to reduce the rate of future unravelling of the wall.



Figure 95: An opening in the east wall of the kitchen viewed from the east on completion of works.



Figure 96: Detail of figure 95.

The contractor modified the specification so some of the fixings were 100mm long and some were 6mm diameter in places to get sufficient load capacity to hang the flints, but in general the methodology worked well. Once the mortar has cured, the project team were confident you could cut out the tying wire, but we felt it would be prudent to leave it in as it would not be visible from ground level. We also felt by leaving the wire in position, it would enable someone undertaking a fingertip inspection in the future to identify and understand the repair.

Canterbury Castle Keep

Sarah Tattersall

Canterbury Castle comprises a substantial stone keep, constructed in the late 11th century to replace an earlier post-conquest motte and bailey castle nearby. It is situated in the centre of Canterbury just within the medieval city walls, which follow the line of earlier Roman walls of Durovernum. Today the castle is in the ownership of Canterbury City Council and is a Scheduled Monument.

Following construction of nearby Dover Castle in the 12th century, Canterbury Castle came to be used as the county gaol. It suffered significant damage over the centuries, falling into ruin by the 17th century and used as a storage facility by the Canterbury Gas Light and Coke company in the 19th century. Canterbury City Council purchased the castle in 1928 and carried out a campaign of repairs to restore the ruins in order to open the castle to the public.

The keep walls are constructed from a rubble stone core set in lime mortar. Bands of flint and stone remain evident and there are localised areas of dressed stone. Much of the stone facing has fallen or been robbed for use elsewhere, leaving the inner rubble core, or later campaigns of flint repair visible. In addition to flint, a number of stone types are present including Kentish Ragstone, Caen, Thanet Sandstone and Chalk. Various mortars are evident, including 20th century repointing that utilised a hard cementitious mortar.

The walls measure 26.5m (87ft) by 23m (75 feet) externally and are now approximately 13m high, having been reduced in height in 1831. They are typically 2.7m (9ft) thick and capped in rough-racked flintwork, set to a gentle fall to each side.

Two internal cross walls originally existed running north-west and south-east but only their foundations now remain. All internal floor structures are lost, although evidence of joist pockets remains in several walls. Putlog (scaffolding) holes are evident to the walls in the form of small rectangular indents, which have later been infilled with slates set back into the masonry.

Remains of former fireplaces with stone set in a herringbone pattern are evident, along with remains of spiral stairs. There is a well within the curtain walls.

There are many window openings at high level with arched stonework. These have lost much of their earlier stone dressings revealing flint or with rough arches in ragstone.

The castle was closed in 2018 due to concerns over falling flints. A structural engineer was appointed as part of an architect-led design team with a brief to repair the castle walls and enable its safe re-opening to the public.



Figure 97: View of the castle prior to repair works. © Mr David E Horwood. Source Historic England Archive.



Figure 98: View of castle with scaffold to exterior. © Sarah Tattersall.



Figure 99: View of castle from scaffold to interior. © Sarah Tattersall.

Approach

The design team benefitted from a perimeter scaffold that had already been installed by the client to both sides of the wall. This allowed for initial removal of vegetation and full access for a visual inspection of the masonry, followed by subsequent hands-on checks to establish the extent of loose masonry. An ortho-photographic survey was also undertaken, which formed the basis of a background on which to mark repairs.

In conjunction with this, an archaeological assessment was undertaken to aid understanding of the various phases of construction, alteration and repair.

Mortar analysis was undertaken by a specialist utilising samples from each phase in order to establish the mortars used in the original construction and subsequent interventions, to aid archaeological understanding and to inform the performance and visual requirements for suitable repair mixes. This comprised a visual inspection under microscope, preliminary testing including carbonation, presence of calcareous aggregates, apparent water permeability and reaction with dilute hydrochloric acid, following by chemical dissolution analysis to establish the aggregate type, grading and binder-aggregate ratio.

An ecological assessment identified the presence of bats utilising areas of the keep. This confirmed the presence of bats roosting on one elevation and within the staircase and meant that no access was permitted to these areas between during November to March, which had significant programme implications and affected the order works were undertaken.

Vegetation such as valerian, buddleia, ivy and sycamore were removed by cutting back growth to the surface of the masonry, to aid inspection. These plants were causing damage to the capping and wall faces and the roots were removed to the extent practicable, or treated as part of the later repairs to prevent future regrowth.

Intrusive investigation works were undertaken ahead of the repairs. Several areas of failed capping were lifted, to better understand the wall construction and to establish the integrity of the wall core below. A standard metal detector confirmed the presence of localised metal reinforcement that had been inserted in prior 20th century repairs.

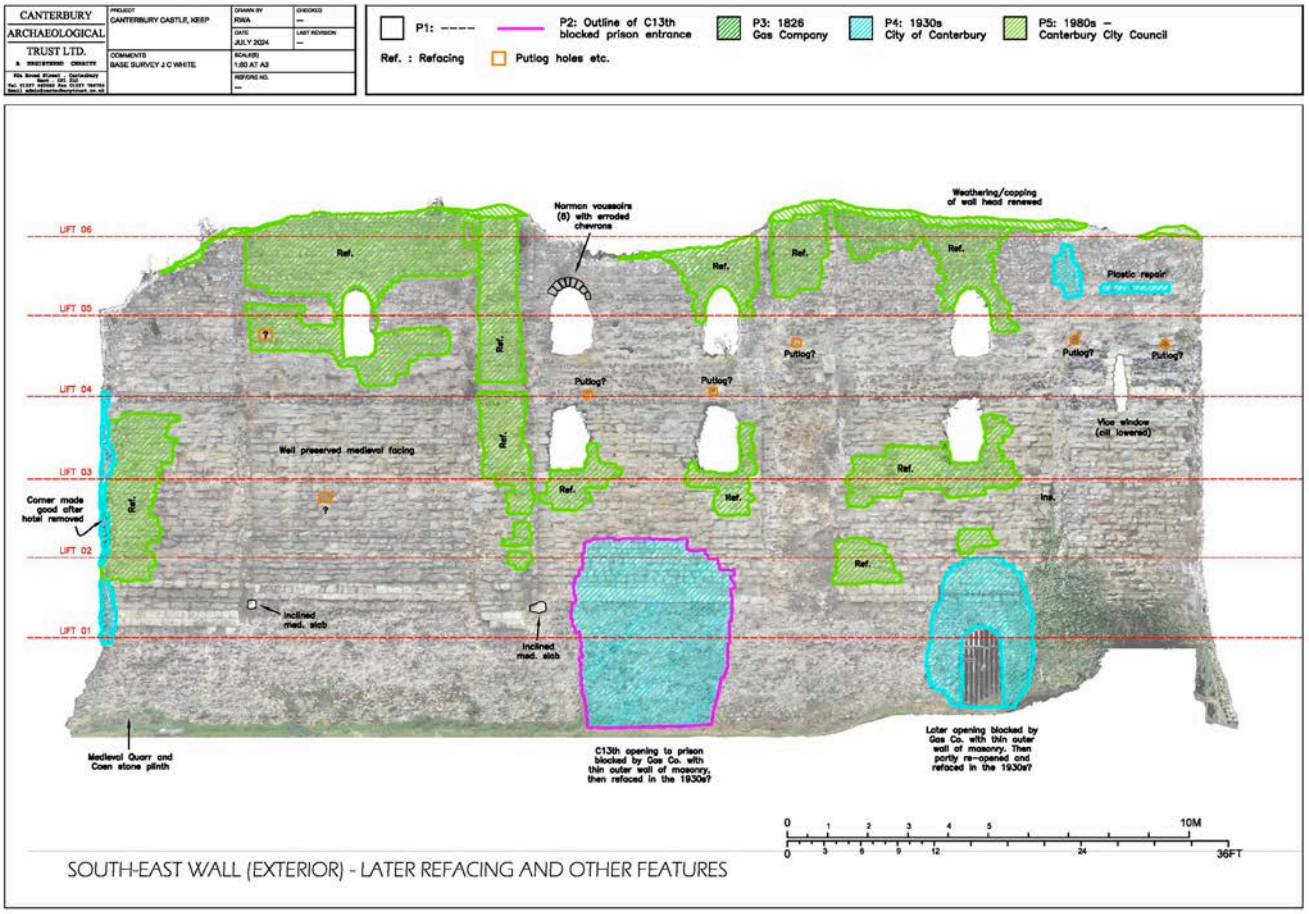


Figure 100: Archaeological analysis phases of building for the south-east wall. © Canterbury Archaeological Trust.

Condition Survey

The findings from the condition survey are summarised in this table:

Condition Survey

Capping:

Localised degradation was evident, resulting from the use of poor quality mortars used in 20th century campaigns of repair, very shallow falls and weak points from construction joints, where vegetation had embedded. The 20th century repair mortars appeared generally utilising a relatively strong cement/lime blend and in some areas to the capping this had been worked over again with a further layer of rough racking utilising a hydraulic lime, with shallow flints that became loose when the mortar degraded.

The edge of the near-horizontal capping where it met the vertical walls was poorly-detailed, with flints at risk of falling from high level.

High level loose masonry was evident from the walls, due to continued weathering and erosion of mortars.

Vertical cracking was occurring at weak points such as the interface between the perimeter walls and remains of the cross-walls, as a result of wooded vegetation growth.



Figure 101: Typical capping, with minimal falls and vegetation growth. © Sarah Tattersall.



Figure 102: Joint between wall and capping poorly detailed, with loose and falling flint. © Sarah Tattersall.

Arched openings:

Unsupported flintwork above openings, leaving flintwork or rough Ragstone arches above without proper support, due to the dressed stone arches having been wholly or partially lost and ongoing loss of flints from the underside of the arch.



Figure 103: Partial remains of an arched opening. © Sarah Tattersall.



Figure 104: Arched opening, with dressed stone arches and jambs lost. © Sarah Tattersall.

Quoins:

Unsupported high-level masonry on a corner, caused by the quoins having been lost or robbed from below. The quoins help tie the two walls together; where they were missing the flint walls had begun to “unzip” and pull apart.



Figure 105: Degraded Caen stone quoins. © Sarah Tattersall.



Figure 106: “Un-zipped” flint walls at corners where quoins are missing. © Sarah Tattersall.

Bulged area: Localised distortion adjacent to the well. Anew opening was cut through the wall in the 19th century to provide a doorway and weakened the wall as a result.	 Figure 107: Bulging area above the well. Areas with vertical cracking and loose masonry. © Sarah Tattersall.

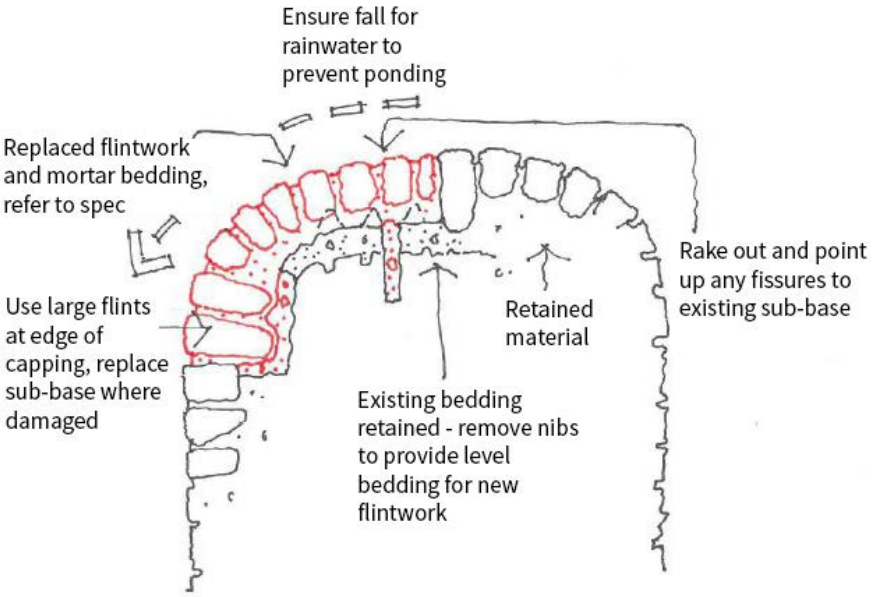
Proposed Repairs

The walls have remained standing for over 900 years. In this time, they have been subjected to the demands of the weather, robbing of stone and various uses, or lack of use. For their age they generally remained in a fair condition, but with defects evident that required timely intervention.

Proposed repairs were based on the principle of minimum intervention in order to reduce the risk of high-level falling masonry and allow the site to re-open to the public. This was coupled with advice to the client of the need for continued periodic inspection and maintenance on completion of the repairs, utilising rope access. Planting of areas of the ground within the keep will also be used to persuade visitors away from the walls.

The overarching philosophy was to add additional structure to provide support to previously unsupported stone or flint as far as possible as a clear intervention, rather than to remove or alter historic fabric. Where existing stone could not be retained, this was replaced on a like-for-like basis, utilising Kentish Ragstone for random rubble and Caen for dressed stone, set in lime mortar. The approach was developed by the design team with early and ongoing engagement with Historic England inspector and engineer.

The repairs are summarised in this table:

Capping: It was not within the scope of the project to re-work the entire capping due to budgetary constraints and many areas were sound, so the approach was undertaken to ensure that all of the edge details, junctions and abutments were properly weathered, with long flints utilised to tie in the flintwork at the edge, supplemented by stainless steel ties where necessary, although where possible the use of inert masonry ties was preferred. A hosepipe test was undertaken to ensure that the water ran off of the capping and any areas of ponding were re-profiled.	 Figure 108: Wall heads following repairs to the capping. © Sarah Tattersall.  Figure 109: Sketch of wall capping repairs. © Purcell Architecture.

Arched openings:

This was the most challenging aspect of the project and with very limited evidence of the original stonework remaining it was agreed that it would not be appropriate to insert new stonework.

A profiled Grade 316 austenitic stainless steel arched bar was inserted in order to support overhanging flintwork where necessary, and rubble ragstone utilised where necessary to reinstate arching action.

For the partial arches, stainless steel rods were inserted to provide a tie to restrain the top of the overhang and encourage it to act as a corbel, set into a new ragstone.



Figure 110: Completed arch repair using a profiled stainless steel bar. © Sarah Tattersall.

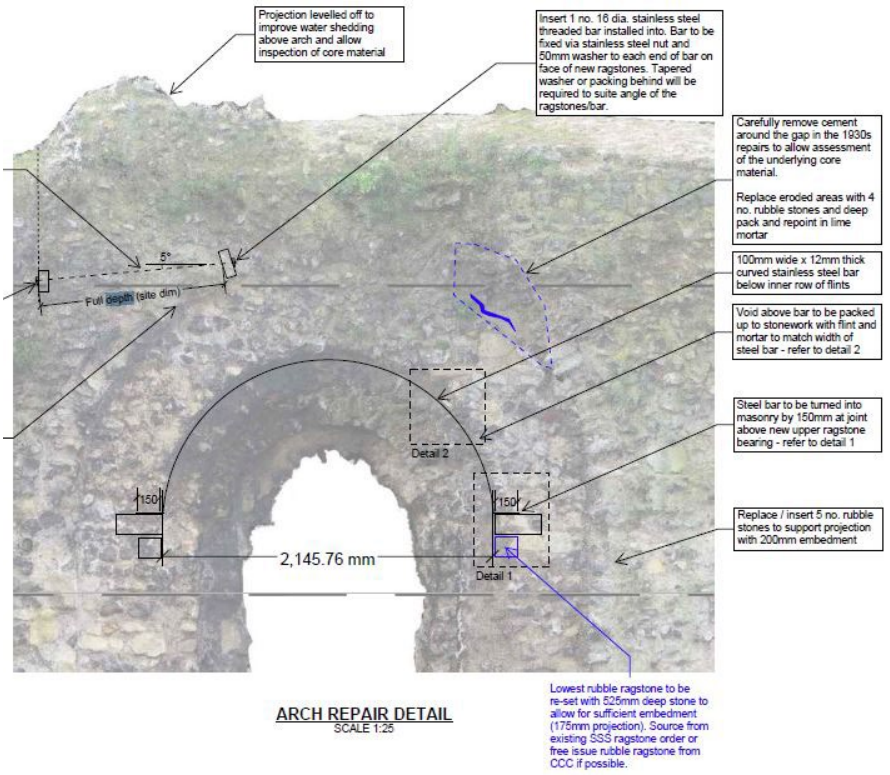


Figure 111: Drawing showing repairs to arches on the Keep walls. © The Morton Partnership.



Figure 112: Repair of a partial arch using with stainless steel anchors. © Sarah Tattersall.

Quoins:

Where possible, existing quoins were retained and these were supplemented with new Caen quoins. The stone at the base was set deep into the wall, with stainless steel threaded bars and the flint below built up with long flints, toothed into the wall.



Figure 113: Repair to quoins, with stainless steel rods concealed behind. © Sarah Tattersall.

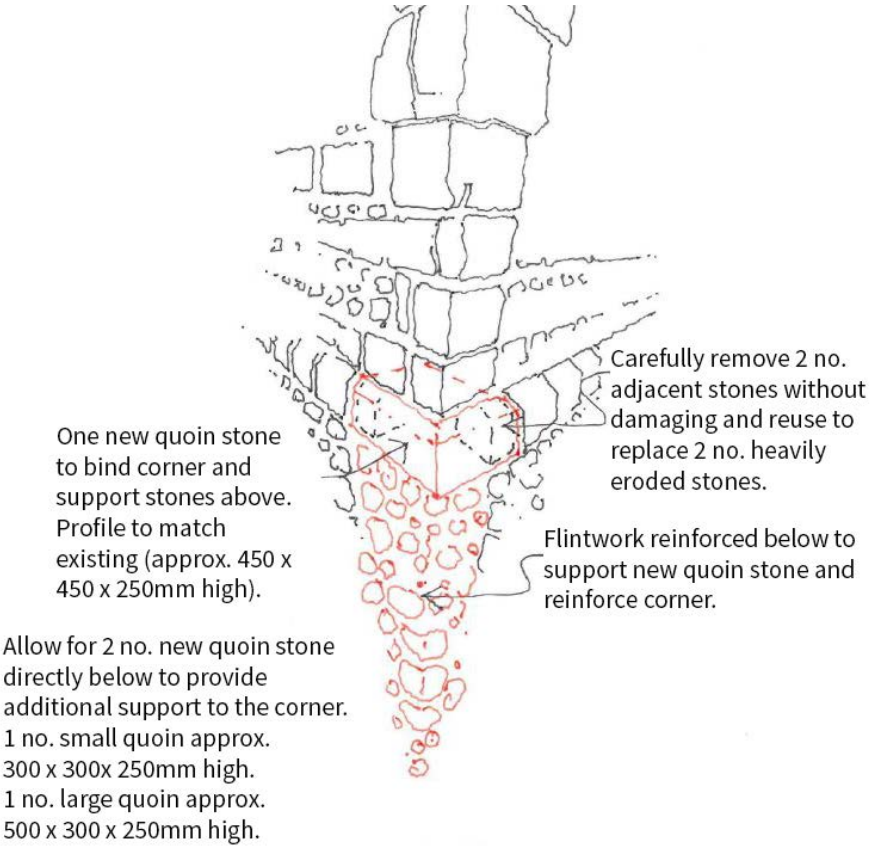


Figure 114: Sketch of corner repair. © Purcell Architecture.

Bulged area:

A new buttress was constructed to restrain the bulging wall, profiled in a curve around the well and utilising the base of an existing rubble wall.

Stainless steel helical bars were inserted horizontally across the cracks behind pointing and overhanging rubble areas that could not be adequately stabilised were covered by stainless steel mesh.

The team had explored other alternatives including inserting proprietary anchors into the wall, however this would necessitate infilling an existing door opening, which was not considered practical or desirable.



Figure 115: A new buttress was constructed to restrain the bulging wall. © Sarah Tattersall.

Outcomes

The repairs to the castle walls have now been substantially completed. Localised areas of masonry repair work remain and the scaffold still stands, due to tight ecological restrictions with the presence of bats observed on the northern side.

As part of a wider landscaping project, a new steel staircase will be constructed, threaded alongside several of the keep walls, allowing high-level access to the top of the wall, with views across Canterbury towards the cathedral.



Figure 116: Ground investigation with archaeological monitoring, to inform the next phase of works. © Sarah Tattersall.

Penrhyn Quarry Hospital

Alex Evans

The ruined remains of Penrhyn Quarry Hospital located near Bethesda in North Wales are a Scheduled Ancient Monument and are within the UNESCO World Heritage Site of The Slate Landscapes of North Wales. Penrhyn Slate Quarry has remained in constant use for over 200 years and at one time was the largest manmade excavation in the world, meeting the international demand for roofing slate. Penrhyn Quarry Hospital was built in 1842 for the quarry workers and contained three wards, a surgery and waiting room. It is noted for being a very early adopter of anaesthetic when an amputation was carried out under ether in 1847. The hospital is one of only three constructed in North Wales for quarry workers and constitutes an important part in the history of slate quarrying.

While ongoing and active quarrying has led to the loss of many physical areas, Penrhyn Quarry Hospital is one of the relict areas which remain intact. The hospital ruins remain to a substantial height. The interior is partly filled with collapsed rubble. The upstanding walls are constructed in stone masonry, largely rubble slate blocks. There are no remaining floor or roof structures. Most of the external elevations survive to full height.

The ruin comprises masonry walls built using slate blocks interspersed with other types of local stone. There is a central two storey block subdivided by slate masonry walls, including the main central cross wall which is where the significant structural interventions were focussed. There are single storey wings to the north-west and south-east of the central block.

Prior to the project the ruin was in a poor condition. There was extensive vegetation including mature trees growing in and around the walls. The ruin had suffered significant deterioration from weather exposure and water ingress, and the gable peak of the central cross wall to the central block was leaning and starting to disintegrate at the head.

Most of the lintels were originally timber, and these had decayed leaving numerous areas of unsupported masonry. In some locations the loss of the lintels had led to the localised collapse of the masonry above. Due to the leaning high-level gable and areas of unsupported masonry, establishing safe access needed to be planned carefully. Some areas could not be accessed initially until some stabilisation work had been undertaken to make all areas safe.



Figure 117: Unsupported openings and failed lintels. © Grace & Howe Consulting Engineers Ltd.



Figure 118: Cross wall to the central block. © Grace & Howe Consulting Engineers Ltd.



Figure 119: Leaning gable over cross wall. © Grace & Howe Consulting Engineers Ltd.



Figure 120: Gable over cross wall. © Grace & Howe Consulting Engineers Ltd.

Approach

Vegetation including mature trees were first cleared from the monument to enable access and to facilitate survey work. A photogrammetric survey was undertaken which was used to mark areas of concern and deterioration. Scaffolding was erected in safe areas to enable close access to the monument for close inspection and to undertake further clearing work. In order to assess the lean to the central dividing wall accurately, a measured survey was undertaken to provide measured 2D elevations and sections at various positions along the wall. The area under the leaning gable could not be scaffolded or accessed due to the risk of falling masonry, so propping could not be installed and stabilisation work could only be undertaken from one side of the wall.

Using the 2D survey information, a numerical analysis was undertaken by hand calculation on the leaning wall considering information such as the measure of lean, effect of openings and restraint conditions. It was then possible to determine at what level the wall needed to be taken down to in order to make it safe to rebuild.

Once the taking down work had been completed from the safe side, access could be established to both sides of the wall in order to undertake remedial work.

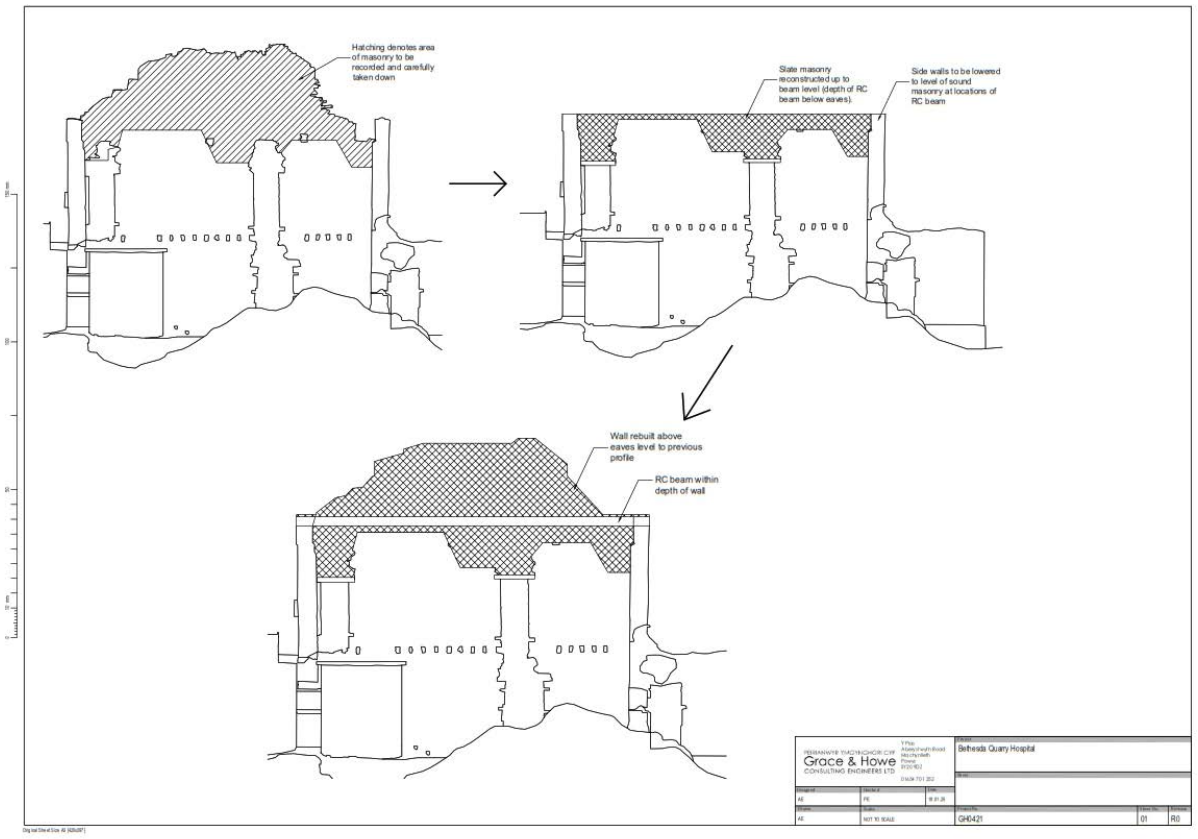


Figure 121: Drawing of sequencing of stabilisation works. © Grace & Howe Consulting Engineers Ltd.

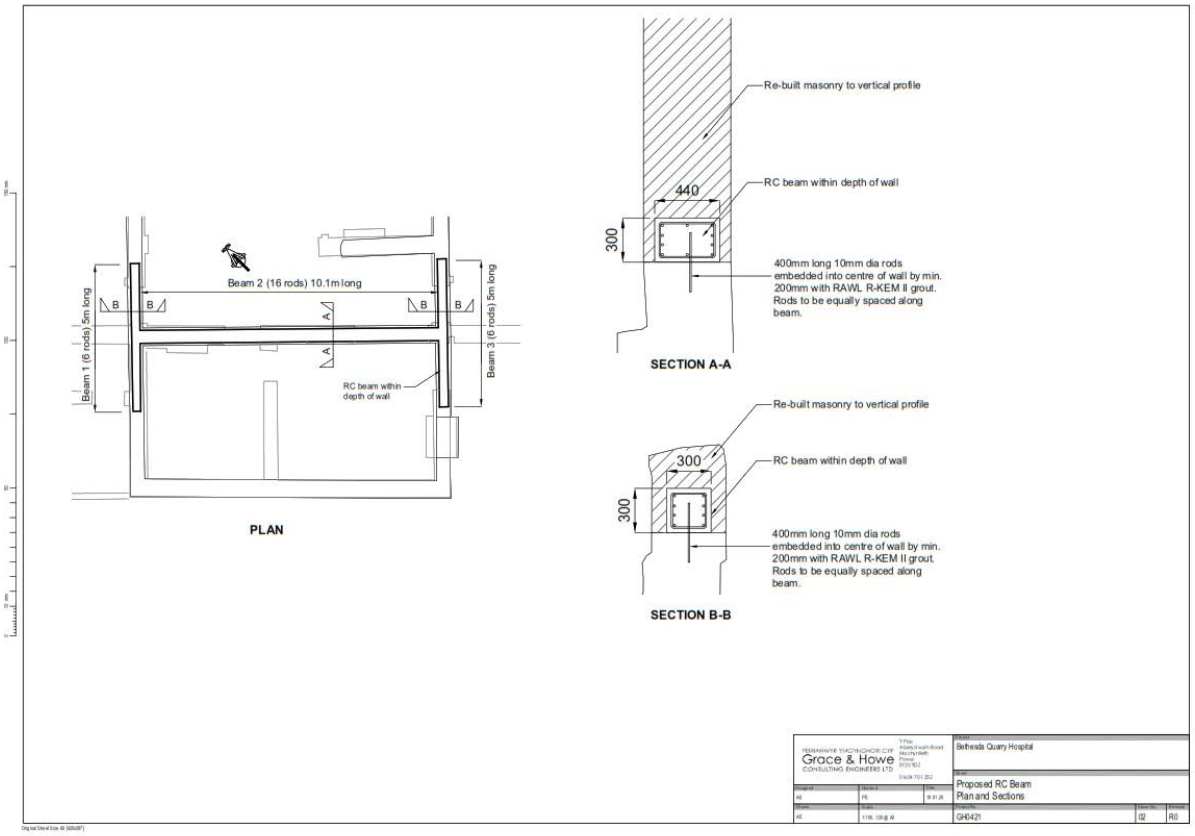


Figure 122: Plan and section of proposed H-beam. © Grace & Howe Consulting Engineers Ltd.

Conclusions and proposals

The main cause of decay was water ingress into masonry. This had caused direct deterioration of the pointing, core and finishes through washout, weathering and freeze/thaw cycles. It also had secondary effects in creating favourable conditions for the establishment of vegetation in the masonry which led to dislodging and deterioration of stonework. The water ingress and environmental exposure also caused the decay of the timber elements, mainly comprising lintels.

The project had to balance the conservation of the structure with making the areas safe in the long term, firstly for contractors and then unsupervised public access. There was also the likelihood that another opportunity for any significant conservation works would not take place for a significant period of time, and that many areas were high level and difficult to access.

Interventions were discussed with the conservation architect and Cadw within the context of conservation principles such as minimum intervention, reversibility and honest repair. As in many cases, the interventions which best fitted one principal were aften at odds with another. Options were considered for stabilising the upper part of the cross wall such as introducing a propping structure to one side on the direction of lean, but the temporary works to make the area safe would be significant and not without risk. It would also require introducing new structure which could confuse the 'reading' of the ruin. Given the deteriorated condition of the top of the wall, at least some rebuilding would be required to stabilise the masonry even if it was propped. The silhouette of the gable was considered to be an aesthetically significant and defining element of the ruin, so rebuilding and strengthening was considered the most appropriate solution. The rebuilding would retain the architectural significance and incorporating strengthening works would provide longevity and ensure that the same leaning defect did not develop over time once work was complete.

The decision was taken to replace deteriorated or missing lintels with precast concrete lintels for both durability and accessibility reasons. The change to being in an external environment had caused the timber lintels to decay, and the option of installing timber lintels with a DPC (damp proof course) or lead liner above to mitigate against water ingress was discussed with Cadw and the architect. It was felt that the high-level access issues would be restrictive for any further maintenance and the precast concrete option would provide more certainty for longevity. There were slate facing sections to the original external face in front of the timber lintels which were kept in place, and when viewing the ruin from the outside the visual impact of the concrete lintels would therefore be minimal.

A permanent restraint system was designed for the leaning wall which provides lateral support at the original eaves level, and also to the upper part of the wall edges where there had been parting from the side walls. The restraint is based on a reinforced concrete ring beam, but because of the presence of chimney stacks it was modified into a H shape on plan, with a reinforced concrete beam running across the head of the leaning wall and returning along both directions of the side walls at each end. The beam was fixed down into the top of the wall with stainless steel dowels to ensure that lateral load from the wall was fully transferred into the reinforced concrete beam and distributed to the side walls. The beam sits within the depth of the wall and is faced on both sides with slate slips so the concrete is not visible. The triangular gable section was then rebuilt above the new beam to re-establish the profile of the ruin. Just below eaves level, anchors were installed through the side walls and into the cross wall to re-establish restraint to the edges where the cross wall had moved.

Missing and decayed lintels were replaced in precast concrete, sized to suit the gap left by the timber sections. Wall tops have been capped with mortar or coping stones, and this will significantly reduce the water ingress into the masonry which was a significant factor in the deterioration.

Outcomes

Following completion of the works, the ruin is now stable. The main central wall still has a slight lean, but by analysis and the application of additional support the long-term stability has been achieved with the minimum amount of taking down. The form of the ruin has been maintained with the distinctive gable outline remaining as a significant feature.



Figure 123: New precast concrete lintel to internal face with new capping over the head of the wall. © Grace & Howe



Figure 124: Original slate external lintel retained in place. © Grace & Howe Consulting Engineers Ltd.

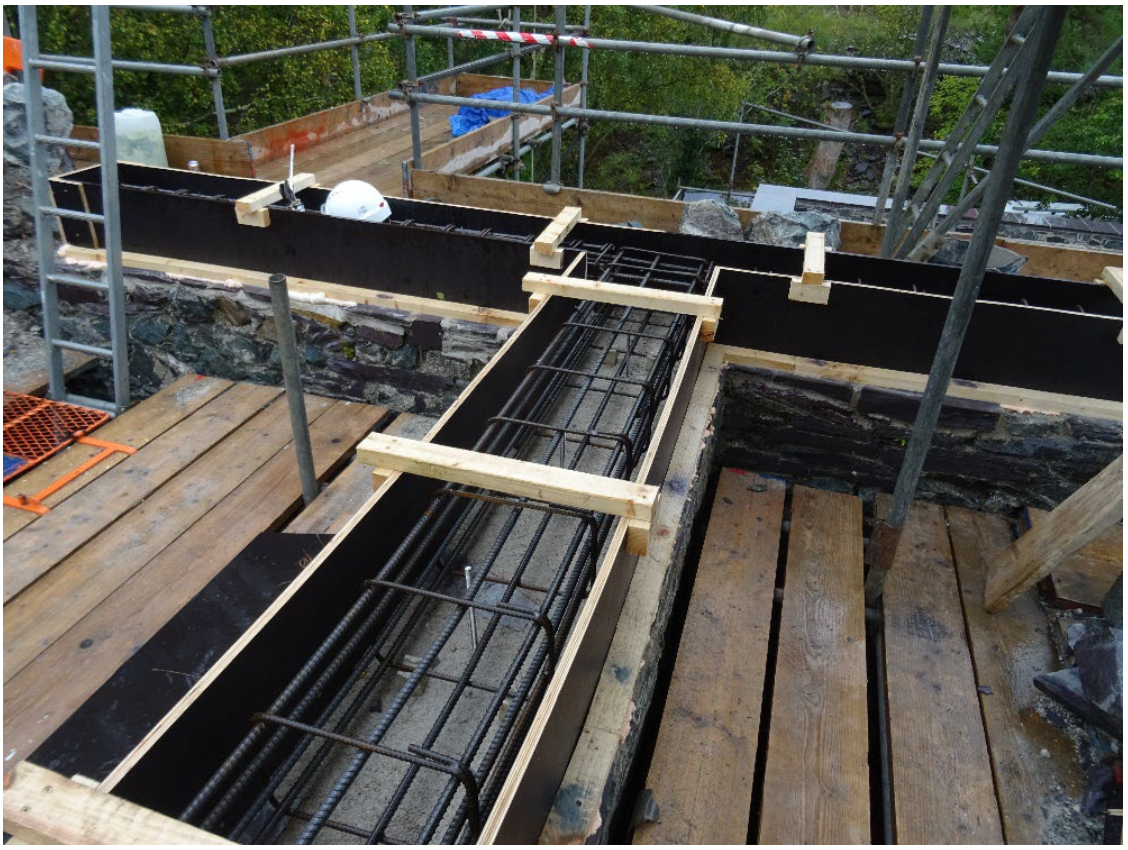


Figure 125: Installation of new reinforced concrete beam to wall head. © Grace & Howe Consulting Engineers Ltd.



Figure 126: Reconstruction of the gable over the central cross wall. © Donald Insall Associates Ltd.

Rebuilding Stone Buttresses

David Wiggins

A set of stone buttresses are to be taken down and re-built because of extensive stone decay.

The upper 12m height of each buttress is to be rebuilt, using magnesian limestone to match the original stone units. Mortar is to be a lime-rich, non-hydraulic, hot mixed lime mortar to match the original bedding mortars. This should give a technically compatible repair.

Air limes develop strength slowly, so the designer was asked to review the implications on construction and temporary support.

Materials testing

The compressive strength of the masonry units f_b was provided by the quarry at 60-75N/mm^2. Density of the stone is 2.4Mg/m^3.

The original mortar was analysed by wet chemistry, X-ray diffraction and petrography to determine the mix composition and binder type. The slaked lime to aggregate ratio is 1:1. Aggregate are a blend of limestone dust and river sand.

Mortar specimens were made following this mix composition, and tested to determine the uniaxial compressive strength. Mortar strength was found to be 1.25N/mm^2 at 28 days. At 360 days the strength reached 3.0N/mm^2 with a density of 1.6Mg/m^3. At 360 days the mortar was fully carbonated. Additional crushing tests on couplets of the mortar and stone to be used gave an average compressive strength at 3 days of 3.6 N/mm^2.

Calculations

The buttresses were assessed against BS EN 1996-1-1:2022 (Eurocode 6 - Design of masonry structures) and the UK National Annex

Basic data

ρ	22	kN/m^3	Density of the composite masonry construction
f_b	60	N/mm^2	Normalised compressive strength of the stone masonry units
$f_{m,28}$	1.25	N/mm^2	Mean compressive strength of masonry mortar at 28 days
$f_{m,360}$	3.0	N/mm^2	Mean compressive strength of masonry mortar after 1 year
γ_m	2.6		Material factor of safety (table NA.1, Masonry category II, execution class 1)
K	0.45		Constant used in the calculation of the compressive strength of masonry
α	0.7		NA2.5. Constant used in the compressive strength calculation for masonry
β	0.3		NA2.5. Constant used in the compressive strength calculation for masonry

Design compressive strength of the masonry unit

The design ultimate compressive strength of the masonry unit is calculated as follows:

At any age	$F_d = K.\ f_b^{\alpha}.\ f_m^{\beta} / \gamma_m$	Eq 5.1 BS EN 1996-1-1:2022
At 28 days	$F_{d,28} = 0.45 \times 60^{0.7} \times 1.25^{0.3} / 2.6$	= 3.25 N/mm^2
At 1 year	$F_{d,360} = 0.45 \times 60^{0.7} \times 3.0^{0.3} / 2.6$	= 4.23 N/mm^2

Building at a rate of one 0.5m lift per day it will take 24 days to complete the 12m high rebuild. At this point the compressive stress in the masonry at the base is as follows:

Weight of the buttress	$G_k = \rho\ .\ b\ .\ t\ .\ H$	= 22 x 2.0 x 0.75 x 12	= 396 kN
Factored ultimate compressive stress	$f_{c,d} = \gamma_L.\ \rho.\ H$	=1.35 x 22 x 12/1000	= 0.35 N/mm^2
Unfactored service compressive stress	$f_{c,k} = \rho.\ H$	= 22 x 12/1000	= 0.26 N/mm^2

From the calculation above there is no risk of the lower courses crushing as the buttress is raised. Utilisation is only 10% of the ultimate capacity.

Costigan[13] et al tested the various strength models for compressive strength of masonry construction based on the strength of individual mortar and stone / brick units. This study concluded that the model used in BS EN 1996 provided the most accurate predictions across all mortar types. Unfortunately, all the models tested, including that in BS EN 1996, significantly overestimated the early age strength of masonry built using air-lime mortars.

Until further research has been carried out, strength predictions for air-lime masonry less than six months old should be treated with caution. In this case the testing of stone / mortar couplets gave extra confidence.

Wind load

After 28 days the temporary supports are to be removed, leaving the buttress to resist wind loads un-aided. Check overturning:

Weight of the buttress	$G_k = \rho.\ b\ .\ t\ .\ H$	= 22 x 2.0 x 0.75 x 12	= 396 kN
Wind load acting on the buttress	$w_k =$		= 2.75 kN/m
Design moment at the base of the rebuild	$M_{o,d} = w_k\ .\ \gamma_L.\ H^2/2$	= 2.75 x 1.5 x 12.0^2/2	= 297 kNm
Restoring moment from the buttress deadweight	$M_{R,d} = Gk\ .\ b/2$	= 396 x 2.0/2	= 396 kNm > 297 OK

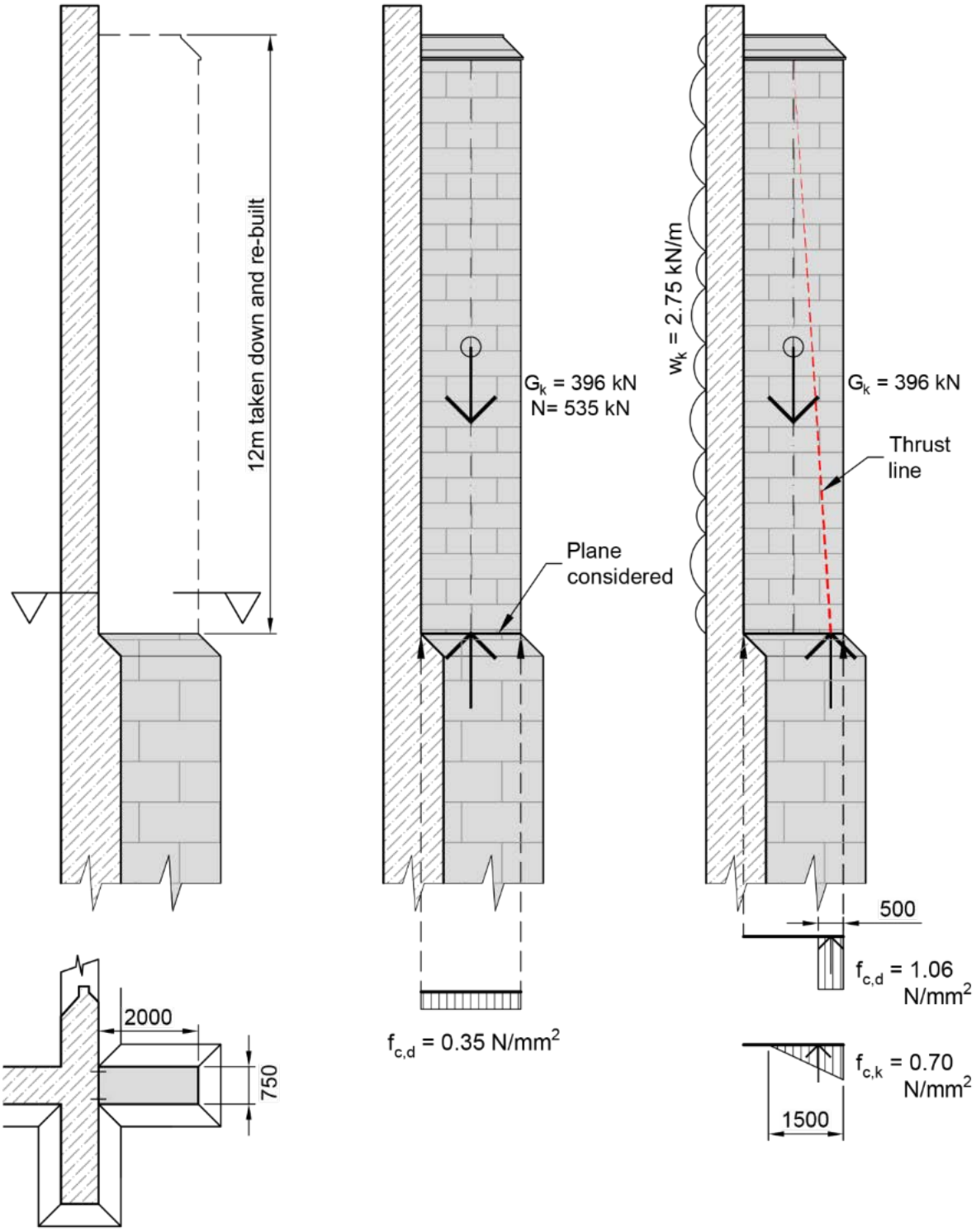


Figure 127: Buttress elevation with load paths.

Check stresses in the buttress:

The buttress will work as the 'web' of a T section when acting as a vertical cantilever, provided that it is effectively bonded to the wall. For the purpose of this check however, the contribution of the main wall is ignored. Using factored ultimate loads:

Eccentricity of reaction from centroid of buttress	$e_{R,d} = M_{o,d} / G_k$	= 297 / 396 x 1000	= 750 mm

The maximum stress in the masonry at the bottom of the buttress can be calculated using a rectangular stress block (note in the wind load combination the load factor for permanent load g_L = 1.0):

Depth of rectangular stress block	$x_d = b - 2.\ e_{R,d}$	= 2000 – 2 x 750	= 500mm
Maximum compressive stress	$f_{c,d} = \frac{g_L . G_k}{(t . x_d)}$	$= \frac{1.0 \times 396 \times 1000}{(750 \times 500)}$	= 1.06 N/mm^2

A triangular stress block is used for the unfactored service loads.

Design moment at the base of the buttress	$M_{o,k} = w_k . H^2/2$	= 2.75 x 12.0^2/2	= 198 kNm
Eccentricity of reaction from centroid of buttress	$e_{R,k} = M_{o,k} / G_k$	= 198 / 396 x 1000	= 500 mm
Depth of triangular stress block	$x_k = 3(b/2 - e_{R,k})$	= 3(2000/2 – 500)	= 1500mm
Maximum compressive stress	$f_{c,d} = \frac{2 . G_k}{(t . x_k)}$	$= \frac{2 \times 396 \times 1000}{(750 \times 1500)}$	= 0.70 N/mm^2

From the calculation above there should be no risk of the masonry crushing due to wind loads. Utilisation is only 33% of the ultimate capacity.

Buckling considerations

When the buttress is in compression under either permanent or wind load it acts like the web of a 'T' section. It gains its restraint about its weaker axis from the main body of the wall.

The slenderness ratio of the buttress b/t = 2000/750 = 2.7. At this slenderness there is no risk of the buttress buckling.

Summary

The predicted masonry strength is ample to support its own weight at 28 days, however strength predictions for air-lime mortars less than 6 months old should be treated with caution. The predicted masonry strength is also ample to resist wind loading at 28 days.

We can conclude that traditional air lime mortar is suitable for the major structural repair proposed, although its performance should be reviewed as construction progresses.

Witley Court Stable Block South Wall

Bridget Drake-Wilkes

The south stable block historically consisted of a ground and first floor with roof over. Following a fire to the Main House in 1937, the house fell into disuse, the south stable block was demolished, with the exception of the large south wall [39].

Witley Court has been a scheduled monument since 1970 and a Grade II* registered park and garden since 1986. The stables and coach house (including the south wall of the stable block) are Grade II listed since 1984, when the orangery (Grade II*) and the rear wall of Louis XVI's court (Grade II) to the south were also listed.



Figure 128: Looking southwest at the stable block south wall within the wider ruin.

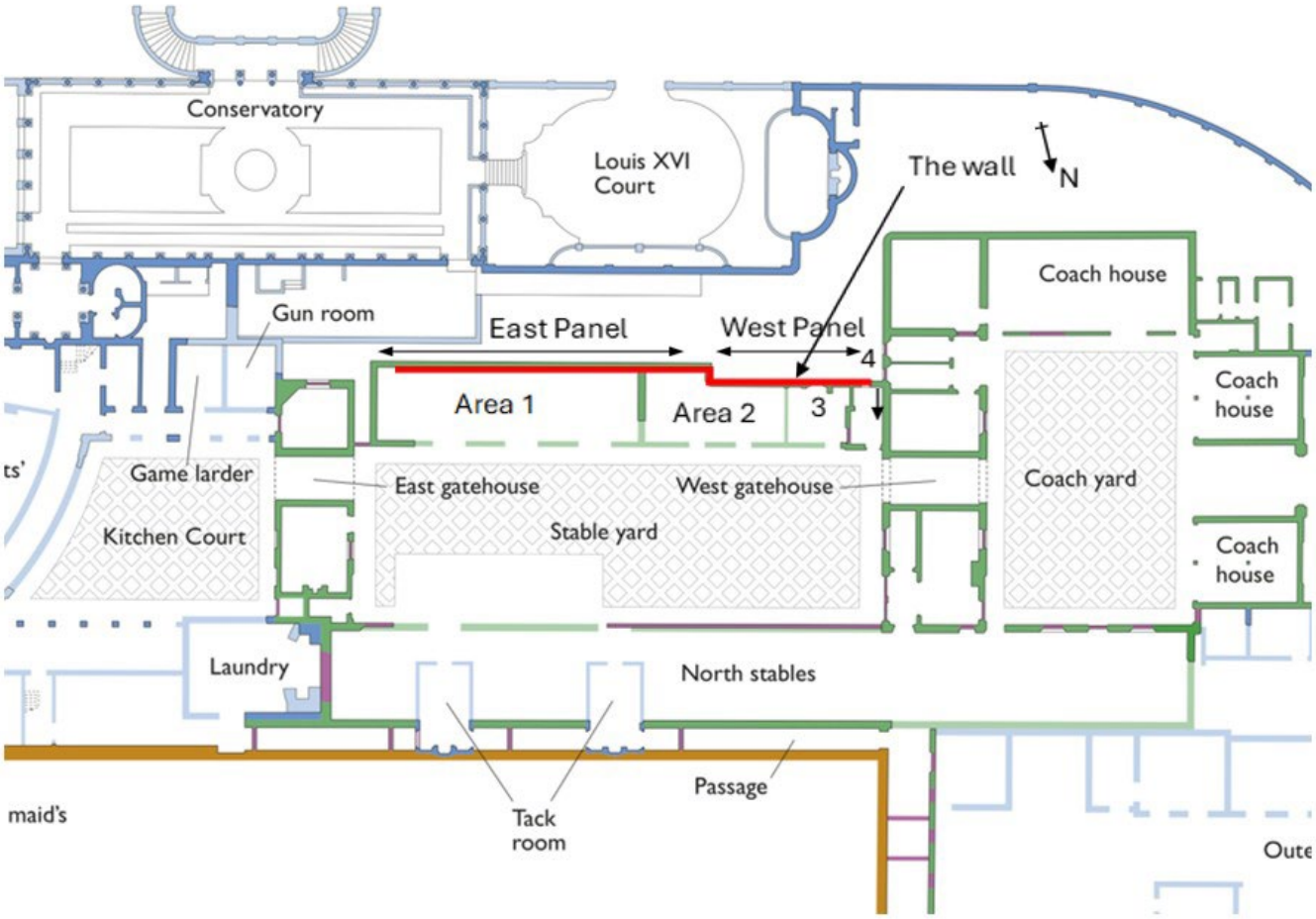
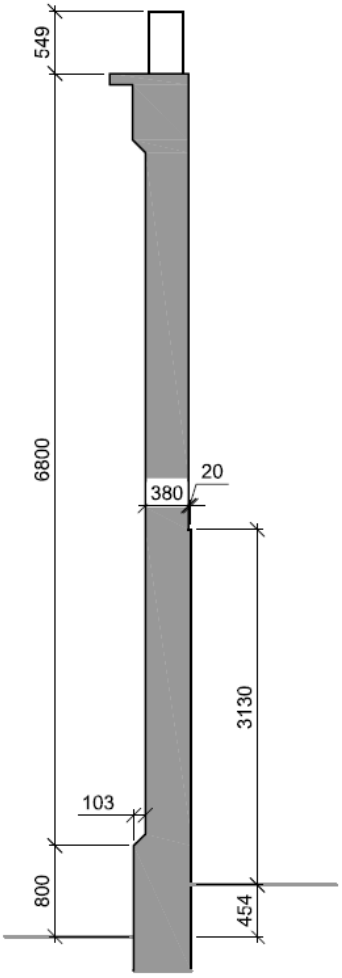


Figure 129: Detailed plan of the wall. © English Heritage.

In 2002 the south wall of the stable block, which by that time was in a parlous state was consolidated, its adequacy for wind loading following consolidation was unable to be justified to the contemporaneous standard (CP3: Chapter 5: Part 2 [40], CP111 [42]and CP121 [43]). So “temporary” scaffold buttressing was installed which still remains and is inspected monthly.

By 2015, the boards supporting the kentledge and the sole plates of the scaffold had been renewed several times and there had been a loss of corporate knowledge regarding the purpose of the structure. Historic England’s structural engineering team were requested to provide an update on this area, to ascertain whether the shoring was still required, what the issues are and whether a more appropriate, sensitive longer term solution could be installed.



The wall is of brick construction with lime mortar, approximately 35m long and 7.60m high on its south elevation (excluding the balustrade approximately 600mm high). Ground level to the north elevation is approximately 450mm higher. The wall at high level is 380mm wide with a 20mm step in the north elevation at historic ground floor ceiling level and a brick plinth to the south elevation increasing the wall width to approximately 500mm over the bottom 800mm.

The area either side of the wall is closed to the public and whist it is accessible to staff and volunteers; it is understood that nothing is to be stored near the wall and they generally have no need to go near the wall.

Figure 130: Cross section through the south wall of the stable block (looking west).

Approach

When inspected in 2015 the wall was in good condition and appeared vertical by eye. It was assessed to the current approved codes of practice Eurocode 1-Actions on Structures-Wind Actions (BS EN 1991-1-4)[29] Eurocode 6- Design of Masonry Structures- General Rules for reinforced and unreinforced masonry (BS EN 1996-1-1)[22].

Based on engineering judgement the following assumptions were made in 2015:

Brickwork

- openings in the wall were assumed not to be significant
- The bricks were clay with water absorption >12%
- The mean compressive strength of the masonry units was – 7N/mm^2
- The mortar strength was 4N/mm^2 (enhanced due to the duration the mortar had been in-situ)
- Category of manufacturing control – Category I
- Class of execution control – Class 1.

Wind loading

- Probability of exceedance –1 in 50 years (standard for buildings)
- Dynamic factor – 1.04 (1.00 is normally recommended for masonry but due to significant height of freestanding wall a higher value has been taken)
- Wind loading on the top balustrade section (where existing) has not been considered.
- Although the national annex does not allow a displacement height in country terrain, given that Witley Court is scheduled and a grade II* registered park and garden, the wall is grade II listed and is substantially shielded by the grade II* orangery and the grade II listed rear wall of Louis XVI's court to the south and a retaining wall to the churchyard to the north. It was felt that the setting of the site was not going to change and therefore the sheltering to the height of the adjacent walls was permitted in the assessment.

The calculations in 2015 assessed the wall without any temporary propping, assumed elastic design and did not permit tensile stress in the wall. The following two scenarios were considered:

1. The wall as a freestanding cantilever of 1m unit width
2. The wall acts as two panels as shown on figure B (with no consideration of the triangular masonry buttress that had been reinstated as part of the 2002 works.

In scenario 1 only the top third of the wall can withstand the 1 in 50-year wind load and the base of the wall had the capacity to resist only 25% of the wind load.

In scenario 2 the slenderness of the wall as the wall was found to be adequate to Figure F.3 of Eurocode. The wall was determined to be able to withstand the 1 in 50-year wind load from the north and the west panel can withstand the southern wind load. However, the east wall could only resist 61% of the wind load and the central return 42%.

Conclusions and reflections

Without a measured survey of the wall, more detailed analysis (such as discrete element modelling), testing of the materials and onsite real time logging of wind load and displacement it was determined that as a minimum the scaffold propping was required to remain in-situ with monthly inspections. The following permanent options were presented to eliminate the need for cyclical inspection and periodic renewal of timber elements:

- Remove scaffold – reducing long term ongoing costs, with a heavy coping the risk of unravelling is low but, the area adjacent would need to be cordoned off during high winds.
- Install stainless steel anchors vertically through the wall into the bedrock below, these would be designed to enable the wall to cantilever vertically under high wind loads.
- Construct a permanent reinforced concrete of steel frame to provide buttressing. Although lighter than a wall solution this would either require kentledge or a piled solution to resist overturning away from the frame the in this archaeologically sensitive location.
- Construct reinforced concrete or masonry walls to act as buttresses to adequate height. With adequate length and stiffness this may not require kentledge or a piled solution to resist overturning, however piles maybe required due to excessive weight in this archaeologically sensitive location.

- Construct a new roof at original eaves level to the stable block, with columns on the front line of the original building (to hold the roof up) - this would act as a diaphragm to transfer load to the existent return walls. Without enclosing the space beneath the roof, the columns would need to be piled to resist uplift on the roof again in an archaeologically sensitive location.
- Reinstate the stable block.

If I were re-running the analysis I would now utilise the toppling block approach (as I still don't have access to discrete element modelling), but without the onsite real time logging of wind load and displacement I suspect it would be unlikely to demonstrate the adequacy of the east panel to the southern wind load.

Masonry Conservation of a Free-Standing Ruin

David Wiggins

A ruined castle keep was found on structural appraisal to have very poor fabric pathology, exhibiting deeply eroded mortar joints, widespread frost damage, failure of prior fabric repair campaigns. This lead to loosening of the masonry units and posed a risk to public safety and the continued stability of the building itself.

A study of the wall tops informed of a rough racking consolidation campaign some 50 years ago, but this detail allows vertically falling rainfall to pool and form a film of water on the wall head which clings to the masonry and pours down the sides of the walls, driving long term elevated dampness of the masonry fabric, and contributing to “damp tower” syndrome.

The root cause behind the persistent failure of prior repointing campaigns with hydraulic lime mortars was diagnosed to be the intrinsic deficiency in weathering detailing to the hard capped ruin. Original bedding mortar was analysed and determined to comprise a binder rich airline hot mixed mortar.

The engineer needs to design an appropriate repair mortar with which to undertake structural rebuilding and fabric repointing activities. This is an elevated site and the high pre-existing water content of the substrate needs to be recognised in the design of the repair. Carbonation kinetics will be challenging.

Mortar performance requirements

Wall tops mortar: a good seal against free water ingress, good bond intimacy, high compressive strength and freeze thaw durability, tolerably moderate stiffness modulus (not too brittle) to sustain bond intimacy over time.

Wall sides mortar: appropriate compressive strength to service the stress in the wall, a comparatively low stiffness modulus, giving the mortar the ability to behave in a ductile, plastic manner under triaxial confinement; high microporosity to establish wick-action and poultice salty water out of the wall, and in doing so sacrificially preserve the masonry units.

Indicative repair mortar by context		
Context	**Constituents by Volume**	**Commentary**
Wall tops	1 part white cement 1 part lime hydrate 5 parts grit sand	Hard wearing, durable mortar, practically waterproof for minimising free water ingress into the wall below, while maintaining a good seal around the mortar-masonry unit interfaces thanks to its lime content. Unsuitable for wall sides: not sacrificial (will accelerate the decay of stones/bricks by displacing drying front into the masonry units)
	1 part natural cement 2-3 parts grit sand	Durable, exceptionally strong mortar, completely waterproof and extremely fast setting (minutes). Limited life expectancy of bond intimacy at the mortar/masonry-unit interface owing to its comparative brittleness. Unsuitable for wall sides: not sacrificial (will accelerate the decay of stones/bricks by displacing drying front into the masonry units)
Wall Sides	1 part quicklime 2 parts limestone dust	Traditional lime-rich aerial lime mortar. Dry-slaking the quicklime to a hot hydrate allows mixing with a minimum of kneading water to achieve a very stiff, sticky mix for bedding and pointing work that is very quickly load bearing with a good hardening rate when dry. Strength and hardening rate can be significantly enhanced by using microporous limestone aggregate in heavy dosings. Unsuitable for wall tops or any location liable to sustained wetness: not frost resistant in saturated conditions.
	1 part quicklime 1 part NHL 5 1.5 parts Portland stone dust 1.5 parts grit sand	Gauged moderately hydraulic hot-mixed lime mortar with good hardening rate, and higher strength and earlier frost resistance, but slower carbonation rate compared with the air lime mortar. Durable wall sides mortar. Unsuitable for wall tops or any location liable to sustained wetness: not frost resistant in saturated conditions.

Fig. 131: Indicative structural repair mortars for different fields of application

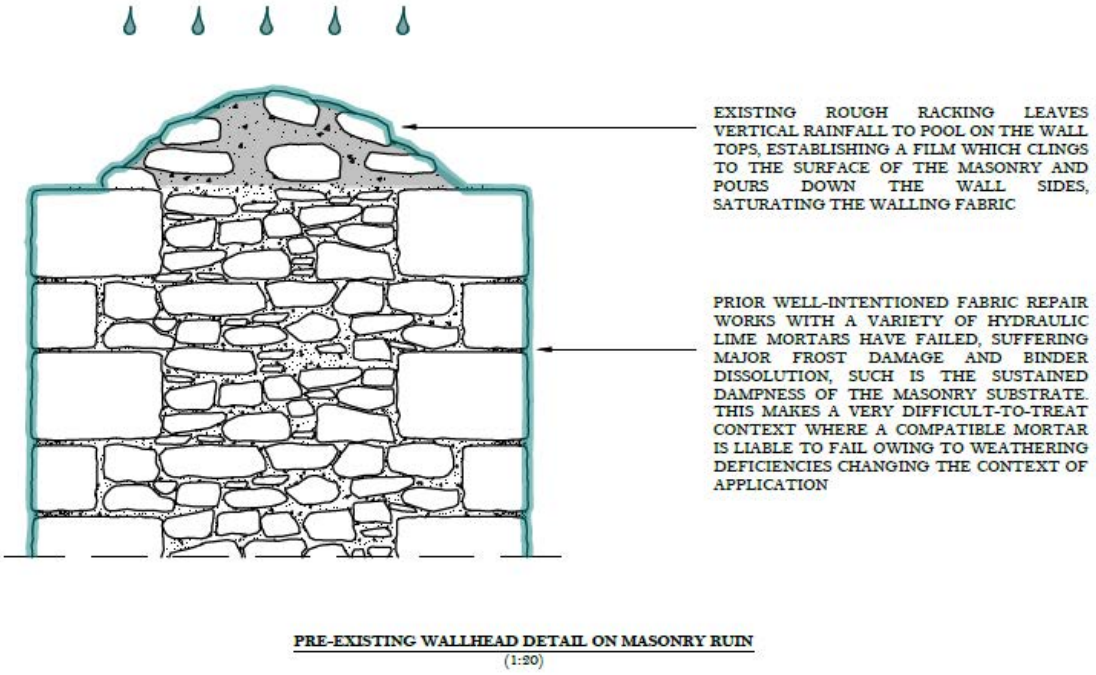
For this problem, the 1:1:5 mix was selected for the wall tops application. For wall sides, below the new lead weathering, the gauged hot-mixed lime mortar was used with heavy dosing of Portland limestone dust to improve early and long term strength development, as well as raising the overall capillary microporosity of the mortar. Despite the hydraulic gauge, the 1:1 slaked lime to aggregate ratio gave a good match with the lime richness of the original mortar; the gauge allowed the development of early strength while the stored water already in the substrate was drawn out by the repair.

Weathering detail improvement

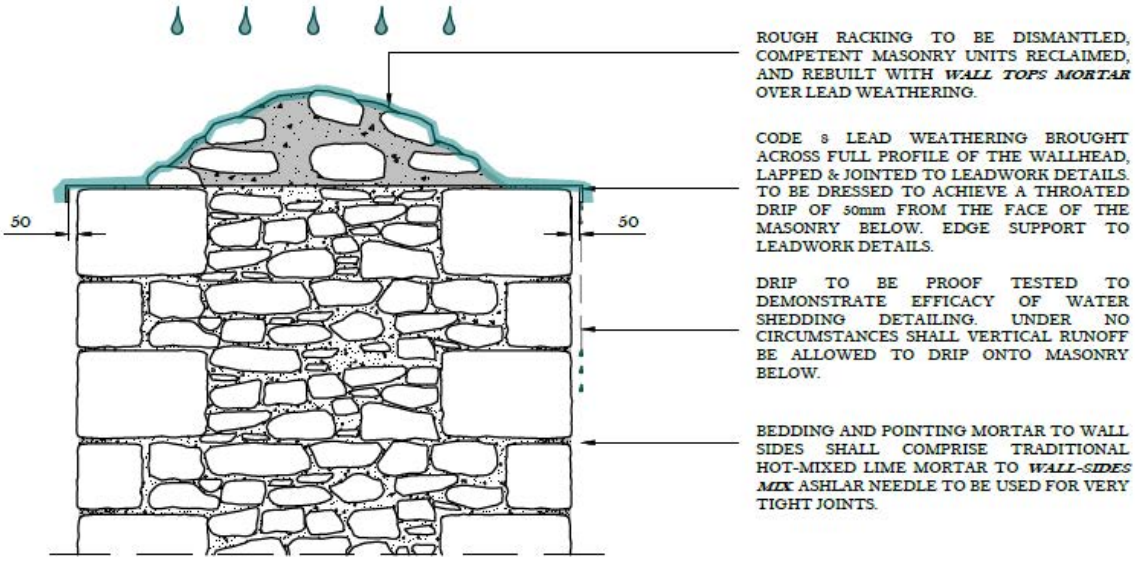
The pre-existing context of the masonry exposure was one of sustained dampness, liable to saturated freezing conditions, which was proven to fail despite hydraulic mortars being used for previous repairs. Without changing the exposure context for the repair mortar, this essentially precludes the use of carbonate binders, which need to be tolerably dry to harden.

Before designing a compatible mortar to mimic the lime richness and physical properties of the original mortar, corrections to the wall top weathering detailing are required to intercept all vertical rainfall, and the drip it clear of the monument.

This detailing correction now allows the engineer to work with two contexts: wall tops liable to saturated conditions; wall sides liable to driven rainfall.



PRE-EXISTING WALLHEAD DETAIL ON MASONRY RUIN
(1:20)



CORRECTIVE INTERVENTION TO WALL-TOPS WEATHERING
(1:20)

Fig. 132 & 133 (top & bottom): Drawing showing pre-existing wallhead detail, where vertical rainfall pooled on the wall top, clung to the wall sides, and saturated the masonry fabric over time, precluding the effective use of lime-rich repair mortars (130,top); Revised wallhead detail, introducing a waterproof detail with a throated drip edge to physically remove the rainfall from the monument, allowing a lime-rich mortar to be used on the wall sides, no longer liable to saturation (131, bottom) © David Wiggins



Fig. 134 Completed assembly of the reinstated rough racking, after the grouting of the wallhead with hot lime grout, creation of a lead weathering brought out to project from both faces of the wall sides with welted drip-edge (left); beading of rainwater on flush testing of the welted drip-edge proved the efficacy of the intervention, allowing lime-rich hot mixed mortars on the wall sides (right).

11. Appendix

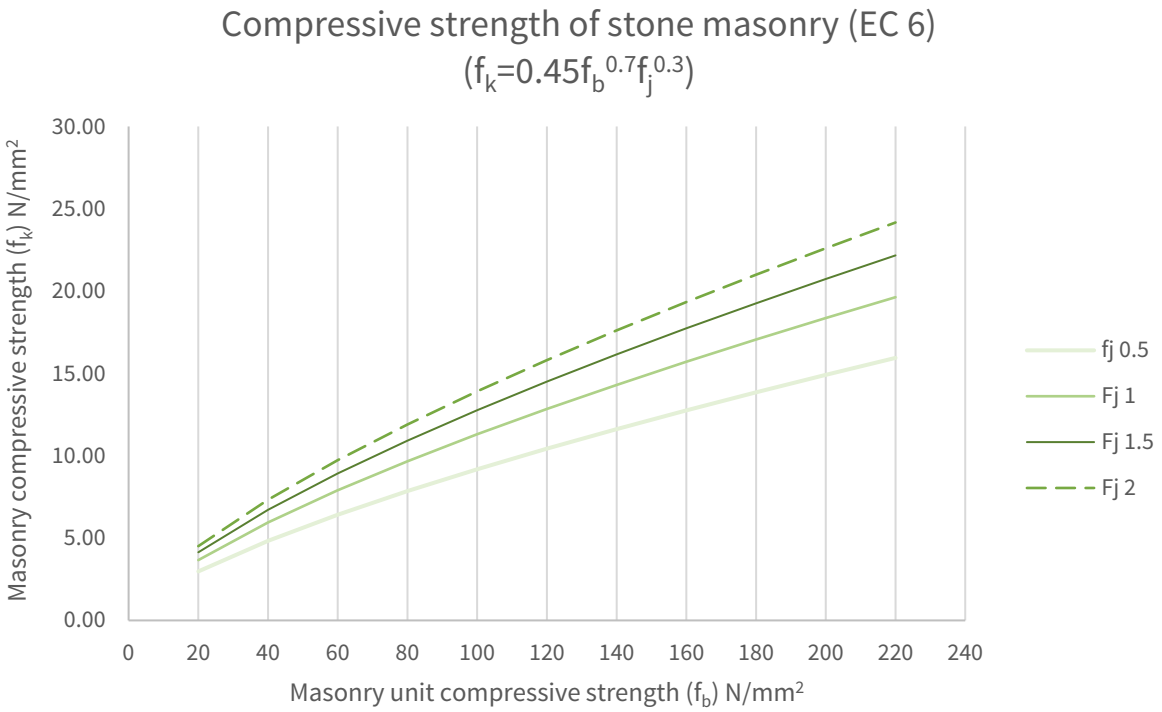
Indicative values of uniaxial compressive strength of earth, lime and cementitious mortars

Mortar composition and age	Compressive strength (N/mm^2)	Reference
Earth mortars		
Clay-based earth mortar at 90 days, no fibres; varying amounts of lime stabilisation	0.11-0.39	Gomes et al. (2018)[10]
Clay-based earth mortar at 90 days, no fibres or lime stabilisation	0.51-0.98	Gomes et al. (2018)[10]
Air limes		
CL90 lime putty (1:3 B;A, silicate sand aggregate @ 1 yr)	0.51-1.94	Lawrence, 2006[9]
CL90 quicklime (@1:3 QL/Agg i.e. ~1:1.5 slaked lime to aggregate, @90 days)	1.0	Margalha et al. (2011)[11]
CL90 putty (@1:3, @ 50 days)	1.25	Drougkas et al. (2016)[12]
CL90 dry hydrate (@ 1:3, @1yr)	1.4	Costigan et al. (2015)[13]
CL90 quicklime, micronised (@1:2 QL/Agg i.e. ~1:1 slaked lime to aggregate, @360dys)	1.63	Margalha et al. (2011)[11]
CL90 dry hydrate (@ 1:3, with limestone aggregate, post carbonation, 2yrs)	2.65	Boffill et al. (2019)[14]
CL90 dry hydrate (@ 1:3, with silica aggregate) (post carbonation, 2yrs)	1.25	Boffill et al. (2019)[14]
CL90 dry hydrate (@ 1:1, @1yr)	3.0-5.5	Lanas & Alvarez (2003)[15]
CL90 dry hydrate (@ 1:1, @1yr)	3.0-5.5	Lanas & Alvarez (2003)[15]
CL90 lime putty (1:3 B;A, oolitic sand aggregate @ 1 yr)	3.18-6.87	Lawrence, 2006[9]
Gauged air/NHL lime mortars		
CL90 quicklime / NHL 3.5 blend (@1:1:6 QL/NHL/Agg i.e. ~1:3 slaked lime to aggregate, @ 180dys)	1.7-2.5	BLFI (2014)[16]
NHL 3.5/Putty blend (0.5:0.5:3 @ 2yrs)	2.65	St Astier (2006)[17]
Natural hydraulic lime (NHL) mortars		
NHL 3.5@ 1:3 (post carbonation)	9.7	Domede et al. (2009)[18]
NHL 2 @ 1:3 (1 yr)	2.7	Costigan et al. (2015)[13]
NHL 3.5 @ 1:3 (1 yr)	10.0	Costigan et al. (2015)[13]
NHL 5 @ 1:3 (1 yr)	13.3	Costigan et al. (2015)[13]
NHL 2 @ 1:3 (2 yrs)	2.75	St Astier (2006)[17]
NHL 2 @ 1:2 (2 yrs)	4.25	St Astier (2006)[17]
NHL 3.5 @ 1:3 (2 yrs)	3.97	St Astier (2006)[17]
NHL 3.5 @ 1:2 (2 yrs)	8.63	St Astier (2006)[17]

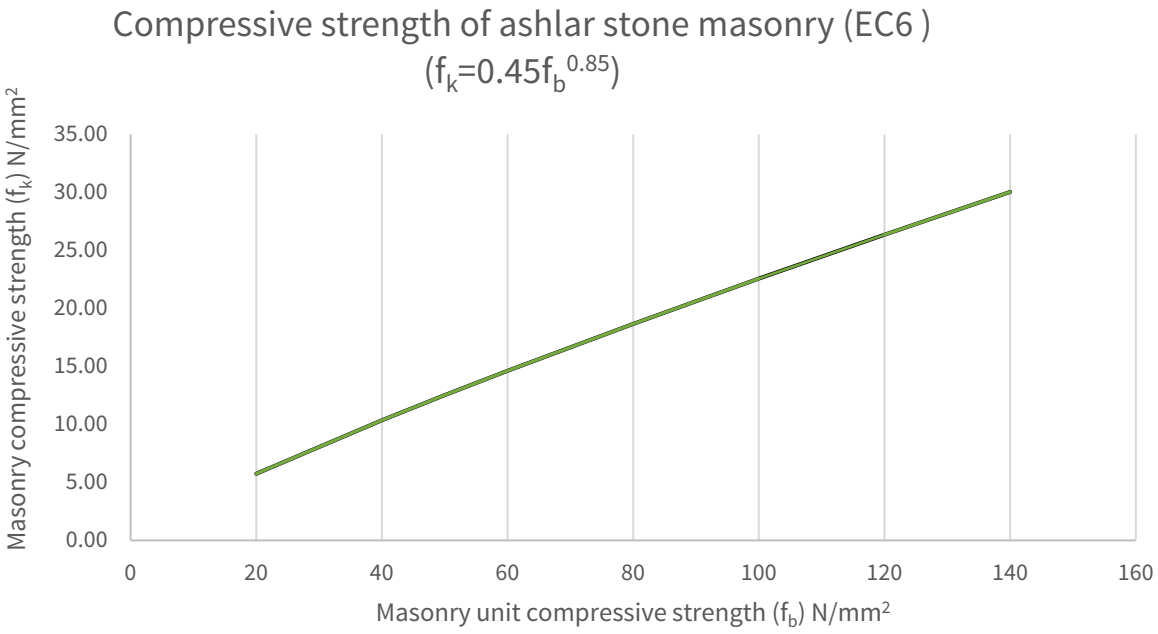
NHL 5 @ 1:3 (2 yrs)	7.8	St Astier (2006)[17]
NHL 5 @ 1:2 (2 yrs)	10.8	St Astier (2006)[17]
Gauged cement lime mortars		
Cem/Lime M6 1:0.5:4 (6 mths)	22.2	Costigan et al. (2015)[13]
Cem/Lime 1:1:6 (2 yrs)	8.5	St Astier (2006)[17]
Cem/Lime 1:2:9 (2 yrs)	5.95	St Astier (2006)[17]

Figure 135: Selected earth, lime and gauged mortar compressive strengths as reported in literature.

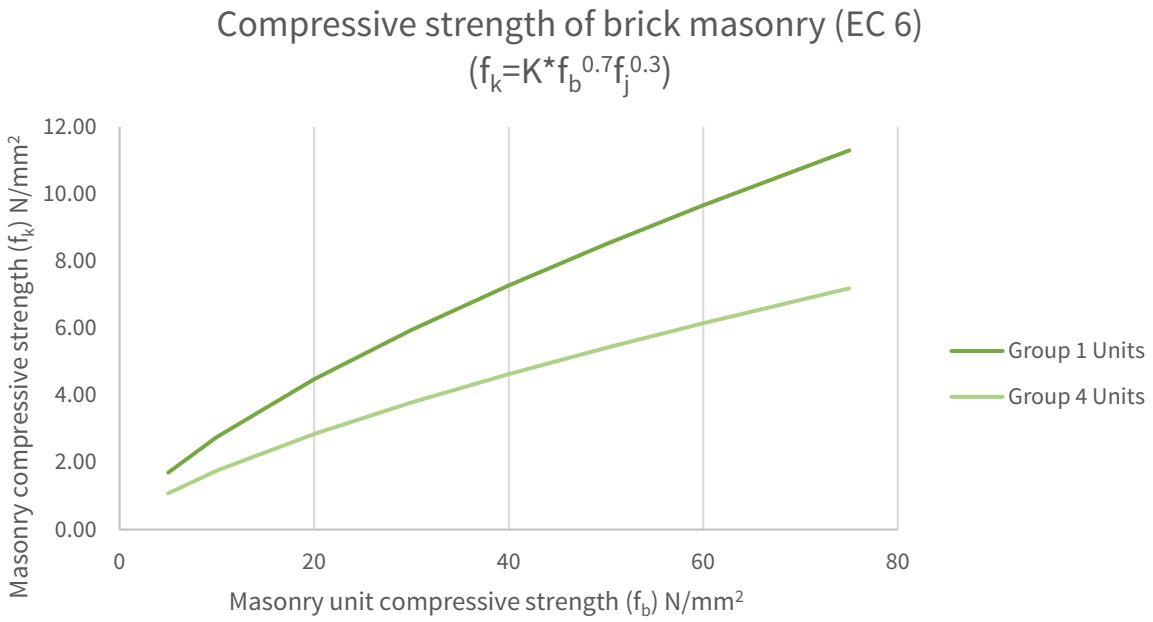
Strength curves for selected masonry composites



*See notes on applicability of this expression in EC6.



*See notes on thin layer mortar masonry in EC6. Expression adapted for use with stone masonry. f_b limited to 50N/mm², giving f_m 12.5N/mm². Ashlar masonry highly unlikely for igneous/metamorphic rock types. Care should be taken to ensure the masonry is through bonded, not simply ashlar faced rubble or brickwork.



*For general purpose mortar of f_j 1.0N/mm² uniaxial compressive strength.

12. Glossary of terms

Acid digestion – a method used to determine the composition of mortar by dissolving the binding component with hydrochloric acid leaving behind the insoluble aggregate.

Air lime mortar – a mortar made with a pure lime binder which cures through the process of carbonation.

Capillary drying – the removal of moisture from a porous material by evaporation at an exposed surface, accompanied by the movement of liquid moisture through the connected pore structure by capillary action towards the drying surface.

Calcination, calcining- Reduction or oxidation of ores by heating in a kiln at an appropriate temperature. Calcination of relatively pure limestones requires temperatures between 800C and 1000C to drive off carbon dioxide and water, producing calcium oxide.

Carbonation- In fresh, non-hydraulic lime mortar exposed to the air , the conversion reaction of carbon dioxide in air with carbon hydroxide (slaked lime), yielding calcium carbonate (and water). Air limes set by carbonation. Carbonation of free lime in hydraulic limes after the initial hydraulic set enhances the mechanical strength and durability.

Cementitious mortar – a mortar made with a cement binder.

Composite masonry wall – a masonry wall constructed from different types of masonry unit.

Consents – the formal approvals required before work on a listed building or scheduled monument can be carried out.

Corrosion – a natural process that occurs when metals and other materials chemically or electrochemically react with their environment which leads to alteration and, in some cases, progressive deterioration or loss of material.

Cover Meter – device used to identify the presence of embedded ironwork.

Decay – the state or process of deterioration.

Discrete-element modelling – a numerical technique used in structural engineering to simulate the behaviour of complex structures by splitting them into discrete elements.

Distress – signs of deterioration.

Earth mortar – a mortar comprised of local natural subsoil and clay particles often used as a bedding mortar in traditional buildings.

Eminently Hydraulic - a historic classification for lime containing a relatively high proportion of reactive clay constituents, producing a stronger and more rapid hydraulic set, with a lower proportion of free lime available for carbonation. It does not correspond with modern NHL strength classification.

Failure – loss of capacity of a structure - either at ultimate limit state in terms of supporting its applied loads or serviceability limit state in terms of fulfilling its intended function.

Feebly Hydraulic – a historic classification for lime containing a relatively low proportion of reactive clays giving a limited hydraulic set alongside continuing carbonation. It does not correspond with modern NHL strength classification.

Gauged Lime Mortar – a lime-based mortar with the addition of small quantities of cement, pozzolan or other material to modify its setting and performance.

Ground Penetrating Radar (GPR) – a non-destructive survey technique which uses electromagnetic waves and the analysis of their reflections to detect changes below the surface of a material.

Heritage Value – the aesthetic, historic, cultural, social or spiritual importance or significance of a building, structure or site.

Hot-mix lime mortar – a mortar made by slaking quicklime and aggregate together with water, either added separately or already present in the aggregate. When mixed, the water reacts with the quicklime and generates heat. The mortar may be used while still hot or after it has cooled and, where appropriate, matured.

Hydraulic Lime Mortar – a mortar made with a hydraulic lime binder which cures through hydration of its hydraulic components and gradual carbonation of its free lime.

Hydraulicity- In a building lime, the property of setting primarilary bychemical reaction with water rather than by carbonation, the degree of hydraulicity depending on the quantity of reactive silicates/aluminates present and the temperature of calcination

Indent – a deep recess in a masonry wall caused by erosion of the masonry units.

Interferometric Synthetic Aperture Radar (InSAR) - a remote sensing technology using satellites which can be used to measure changes in ground elevation.

Jacking force – a compression force applied between or within masonry units by a corroding ferrous material or vegetation.

Lime mortar – a mortar made with lime as the principal binder, mixed with aggregate, water and, in some cases, a pozzolanic material or other addition.

Listed Building – a building included on the National Heritage List for England and given statutory protection from anything that affects the special or architectural interest through the Planning (Listed Buildings and Conservation Areas) Act 1990.

Oolitic limestone – a sedimentary rock composed primarily of ooids, which are small, spherical grains formed from calcium carbonate.

Phosphor Bronze – a copper alloy with tin and phosphorus used in historic building repairs.

Pinning of stonework – process of re-attaching loose outer facing stones back to the main body of a masonry wall.

Porosimetry – an analytical technique used to determine a material's pore structure.

Poultice – a material applied to the surface of another material which draws out contaminants and staining.

Pull-off strength – the strength of a fixing or material in tension.

Quicklime – Calcium Oxide (CaO), a whiteish, odourless, alkaline substance which is obtained by heating limestone. It is highly reactive with water, forming calcium hydroxide and releasing considerable heat.

Return period – the reoccurrence interval or average time between events.

Reversible repair – a repair which could be later undone without any damage to the historic fabric.

Ruin – the remains of a building which has fallen into disrepair.

Sacrificial weathering – a process where certain materials are intentionally used to protect others.

Scheduled Monument – a nationally important archaeological site recorded on the National Heritage List for England. Scheduled Monuments have statutory protection against any changes with the aim of passing them down to future generations in the manner in which they were found.

Serviceability Limit State – a design scenario covering situations where the building may not collapse, but cannot be used for its intended purpose such as excessive deflection or vibration.

Silicate – a salt derived from a silicic acid.

Sock anchors – a method of repairing masonry structures using a steel or stainless steel anchor fixed with resin or grout contained within a sock to prevent resin or grout leaching into the structure.

Soluble salt precipitation – deposits of solid salts which were previously in solution on the surface of masonry, concrete or plaster left behind when water evaporates.

Thrust lines – an imaginary line which represents the path of compression load through a structure.

Total station – an electronic or optical instrument used in surveying, used to measure the distance of an object from the instrument along with horizontal and vertical angles.

Triaxial Compressive Strength – the maximum axial stress which a material can withstand whilst subject to a constant confining pressure.

Tripartite vault – a type of architectural vault formed by the intersection of three ribs or segments.

Uniaxial Compressive Strength – the maximum axial stress which a material can withstand when loaded in unconfined conditions.

Wallette – a small wall panel built specifically for the purpose of research or testing.

X-ray diffraction – an analytical technique used to identify crystalline phases in a material sample from their characteristic diffraction patterns.

13. Glossary of symbols

α	Power used for obtaining characteristic compressive strength of masonry.
f_b	Normalised mean compressive strength of masonry unit.
f_m	Compressive strength of the mortar, which may be taken as equal to the mortar strength class.
f_k	Characteristic compressive strength of masonry.
$f_{m,k}$	Characteristic compressive strength of mortar.
G_k	Characteristic value of permanent action.
K	Constant used in the calculation of compressive strength of masonry.
t_b	Width of rectangular stress block.
$v_{b,0}$	Fundamental value of basic wind velocity.
β	Power used for obtaining the characteristic compressive strength of masonry or an enhancement factor for concentrated loads.
γ_m	Partial factor for material property, also accounting for model uncertainties and dimensional variations.

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Assessing historic assets that are proposed to be demolished or partially demolished [online resource] https://www.istructe.org/resources/guidance/care-assessing-historic-assets-for-demolition/

16. Acknowledgements

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Historic England gratefully acknowledges the support of Cadw and the CARE (Conservation Accredited Register of Engineers) Panel in the production of this guidance.



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Publication date: October 2026 © Historic England
Design: Historic England