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Extending the service life of Swiss bridges of cultural value

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Bridges of high cultural value and aesthetic quality deserve respectful treatment and construction interventions must balance these assets with the severe requirements of utilisation. This is particularly relevant to structural engineers and bridge owners involved in rehabilitation or modification interventions. This paper presents, by means of eight bridges in Switzerland, examples of how interventions can be performed with adequate respect to cultural value. It is argued that the preservation of cultural value may go hand in hand with socio-economic, environmental and technical requirements following the principles of sustainable development. These requirements are met through the application of advanced structural engineering methods specific to existing structures. Extending service life not only adds value to bridges – it also leads to an appreciation of the art of structural engineering and the identity of the engineers themselves.

1. Introduction

With the exception of structures with recognised historical and technical importance, bridges have yet to find adequate consideration as objects of high cultural value. The intervention of bodies for the preservation of monuments has been limited in the domain of bridges, focusing mainly on riveted steel or masonry bridges built prior to the twentieth century. Only in exceptional cases are bridges from the twentieth century considered as structures of high value. Examples of these in Switzerland include the bridges of the world-famous Swiss engineer Robert Maillart who was active between 1900 and 1940. The determining factor may be that most bridge engineers are not educated to recognise cultural value and aesthetic qualities of such structures and in particular fail to acknowledge them as contemporary monuments.

Consequently, many bridges of high cultural value have already been subjected to interventions based purely on technical criteria without any consideration given to cultural value. Because of this, many of the less well-known, but nevertheless valuable, bridges have been defaced. For example, a bridge’s appearance can be disfigured by adding new structural elements, removing or modifying details or erasing structural age indicators, thus damaging the bridge’s identity and historical features.

To prevent further loss of cultural value, bridge engineers, owners, preservation authorities and the public need to be encouraged and empowered to give adequate esteem and importance to bridges. This paper presents, by means of examples, how interventions on bridges can be performed with adequate respect to their cultural value. It is argued that the preservation of cultural value may go hand in hand with socio-economic, environmental and technical requirements following the principles of sustainable development. These requirements can be met by application of advanced structural engineering methods for existing bridges.

2. Dealing with bridges of high cultural value

2.1 Objectives of protection and scope of action

Bridges are built to serve several generations. As part of the transportation infrastructure, bridges add value to the public economy. Therefore, there is high interest in economic performance while providing unrestricted utilisation (e.g. without limits on traffic loads). Also, the safety of individuals and society needs to be considered in a well-balanced manner according to the bridge’s significance within a given transportation system. A bridge is designed for a specific traffic type and its conversion to a different use is questionable.

The continued and contemporary use of a bridge is central and must be guaranteed. This may, however, cause conflict with the conservation of the bridge. While the primary structural elements shall be preserved for the longest possible extent, elements of equipment are subject to wear and require periodic renewal. Their conservation would often make contemporary utilisation impossible and, consequently, elements of shorter
lifespan such as kerbs or pavement may therefore be adapted or even replaced.

Cultural value is preserved when sustainable development principles are followed. This implies preservation of a bridge's features, substance and appearance as well as its relation to the overall appearance of the location and surrounding landscape. As a consequence, any intervention on a bridge needs to be economical, must respect the environment and resources and be socially and culturally compatible when considering current needs and requirements. The challenge for a structural engineer is to demonstrate that the bridge’s real load-carrying capacity is sufficient for modern traffic needs and that only minor interventions are necessary to re-establish and/or improve durability.

2.2 Structural engineering in the domain of existing structures
The contemporary approach to existing structures is based on an inherent methodology that essentially includes collecting detailed in situ information about the structure. The controlling parameters are determined more precisely and, for example, the structural safety of an existing bridge can be proved using so-called updated values for actions (loads) and resistance. In this way, it can often be shown that an existing bridge may be subjected to higher load effects while meeting the safety requirements, thereby avoiding intervention.

This methodology has evolved and been successfully applied over the last 20 years. However, it has not yet been fully adopted by many structural engineers, possibly because there are no codes on existing structures available for engineers to rely on. As current codes do not address the major issues of existing structures, their application is fundamentally wrong and often leads to unnecessary interventions. A change of paradigm is needed in the structural engineering community to clearly distinguish between codes for new and existing structures. For this reason, the Swiss Society of Engineers and Architects (SIA) recently released a series of codes for existing structures (Brühwiler et al., 2012).

2.3 Types of intervention
There are two basic types of intervention on bridges – rehabilitation and modification. The objective of the former is to restore structural safety and serviceability of a bridge for a given service life under constant criteria of functionality and utilisation. The objective of modification, on the other hand, is to transform the functional properties of an existing bridge in response to a foreseen increase of utilisation requirements (e.g. higher traffic loads or a wider driving surface).

Accordingly, the function and utilisation of a bridge determine the type of intervention needed for its preservation. When performing interventions on bridges of high cultural value, the following two questions regarding bridge aesthetics are raised.

- Does the original character – the structural form, structural details and surface colour and texture – need to be preserved?
- Shall the bridge's appearance be intentionally modified by the intervention? In other words, will the intervention be visible?

In principle, the basic concept of an intervention should be evident and understandable. The original character should be preserved in rehabilitation interventions. In modification interventions, however, the appearance of the structure may change according to the needs.

3. Examples
The principles outlined in Section 2 are illustrated by the following examples of interventions to extend the service life of bridges of high cultural value in Switzerland.

3.1 Railway bridge over the Rhine
Designed by the famous German railway engineer Robert Gerwig (1820–1885) and constructed in 1858, this bridge is the oldest riveted railway bridge still in service in continental Europe (Figure 1). The riveted wrought-iron girder is typical of the nineteenth century railway construction era in central Europe. Accordingly, the bridge’s cultural value is obviously high (Brühwiler and Hirt, 2010).

In 1999, the bridge was upgraded for modern railway traffic of the Zurich metropolitan area. To estimate the remaining fatigue life of the bridge structure, a detailed examination was

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Figure 1. Railway bridge over the Rhine between Waldshut (Germany) and Koblenz (canton of Aargau, Switzerland)
performed using realistic past and future traffic models as well as advanced knowledge of the fatigue behaviour of riveted wrought-iron details. It was proven that the bridge structure was safe and its service life could be considerably extended for the foreseen passenger train traffic (Keller et al., 1995).

More recently, the bridge was again examined in view of a long remaining service life of 100 years and considering increasing future passenger train traffic. Sufficient fatigue safety and structural safety was again confirmed. To guarantee the bridge’s durability over the next 100 years, future rehabilitation works, including replacement of the open railway track (to be performed in 2015), were shown to be by far more cost effective and economical than bridge replacement (an option that was often and is still chosen in such situations).

3.2 Dorénaz road bridge over the Rhone
This three-span reinforced concrete (RC) girder bridge (Figure 2), constructed in 1933, was designed by Alexandre Sarrasin (1895–1976), a well-known Swiss engineer who specialised in RC (Brühwiler and Frey, 2002; Habel and Brühwiler, 2009). The simplicity and harmony of forms and the economical use of building materials are among the remarkable characteristics of this bridge design. The structural details of the piers contribute to the bridge aesthetics. This bridge has been recognised as an important landmark of attractive concrete bridges in the initial phase of widespread application of concrete constructions.

Dorénaz Bridge was rehabilitated in 1999 to meet the needs of unrestricted modern road traffic. Detailed examination using updated models of actual structural resistance and traffic actions provided the basis for an intervention limited to invisible strengthening of the deck slab to restore its structural safety (Bailey et al., 1999). To extend durability, local repairs of steel rebars damaged by corrosion leaving visible marks as age indicators were performed and all concrete surfaces were treated using corrosion inhibitors. This rehabilitation improved the durability and the load-carrying capacity of the bridge, while the original character of the bridge was preserved for a long remaining service life with unrestricted traffic.

3.3 Gueuoz road bridge
This structure, with an arch span of 99 m, was built in 1933 and is the most famous bridge designed by Alexandre Sarrasin. It is characterised by its slender and bold concrete structure assembled with linear members that form a light arch and a U-shaped girder that provides structural stiffness. In 1994, a new steel–concrete composite bridge was built parallel to the original bridge to accommodate increasing traffic needs (Figure 3). The concrete bridge structure showed a similar type of damage as found on Dorénaz Bridge and its rehabilitation in 2005 followed the same basic concepts. The concrete bridge can now be used for one-lane traffic during maintenance work on the new bridge.

3.4 Schwandbach and Rossgraben Bridges
In 1933, Robert Maillart (1872–1940), the most famous Swiss concrete bridge builder, designed and built two bridges close to one another. Schwandbach Bridge (Figure 4(a)) is a deck-stiffened arch that carries a curved roadway. Rossgraben Bridge (Figure 4(b)), with its arch span of 82 m, is the second largest open-box three-hinge arch structure after Maillart’s world-famous Salginatobel Bridge. Both RC structures showed satisfactory performance in terms of load-carrying behaviour and durability. Therefore, after more than 70 years of being in service, relatively very little rehabilitation was necessary to restore their durability. Detailed examination of the structures...
showed that the load-carrying capacity was sufficient for future traffic loads.

Rehabilitation works performed in 2005 comprised

- improving the entire water drainage system
- placing a waterproofing membrane on the deck slab
- locally repairing zones where steel rebars were damaged by corrosion
- protecting the exposed concrete surfaces with in-depth hydrophobic treatment (i.e. allowing for impregnation of the concrete up to a 20 mm depth).

This low-impact and cost-effective rehabilitation brought no visible change to the structures yet significantly improved the service life of these very high cultural value bridges.

3.5 Javroz road bridge

In 1950, Henri Gicot (1897–1982) designed Javroz Bridge to replace a riveted arch bridge. The design of this rather slender structure is influenced by Sarrasin’s arch bridges. The cultural value and aesthetic quality of this bridge are attributed to its integration into the landscape and its elegance and transparency (Figure 5).

Due to a predicted increase in future traffic demand, the bridge had to be widened by more than 3 m. This led to a visible intervention performed in 1999 and 2000. The bridge deck was widened symmetrically by cantilever slabs and strengthened by external post-tensioning. To improve the durability of the bridge, Gerber (halving) expansion joints were removed, which led to a modification of the static system. Furthermore, corrosion damage on the steel rebars was repaired and the entire deck slab was protected with a waterproofing membrane. All works had to be performed while maintaining one lane of traffic (Brühwiler, 2002).

The characterising lines of the bridge are defined by the strong arch and the kerb. Therefore, dimensioning and detailing of the kerb of the widened deck slab were carefully carried out to obtain (optical) equilibrium with the strong arch. The aesthetic appearance of the structure was thus slightly improved.

3.6 Schwarzwasser Bridge

The deck of this road bridge, built in 1882, had to be replaced and widened in 2005 to respond to the requirements of future traffic needs. With an arch span of 112 m, this bridge is one of the most impressive nineteenth century riveted bridges in Switzerland (Figure 6). It was designed by the Swiss steel bridge engineer Beat Gubser (1836–1882) and built by the steel construction company G. Ott & Cie from Berne. The bold appearance and elegance of the structure, as well as its
importance for steel construction in Switzerland, justify its high cultural value.

To accommodate a wider traffic surface, the concrete deck slab had to be replaced by wider prefabricated elements. Detailed examination of the riveted wrought-iron structure showed an almost sufficient load-carrying capacity for future higher traffic loads. Only minor strengthening of the arch had to be performed to improve structural capacity. The original character of the bridge was preserved, it was adapted to cater for future traffic needs and its service life was largely extended. The prefabricated deck elements allowed for a rapid construction process and traffic restrictions during the works were thus very limited. The intervention cost was significantly lower than the estimated cost of bridge replacement.

3.7 Bessières Bridge
Bessières Bridge, located next to the old town of Lausanne, was designed by the well-known railway engineer Jules Gaudard (1833–1917) with detailing by the architect Eugène Jost (1865–1946) and built in 1910 by the ‘Atelier de construction mécanique’ in Vevey (Figure 7). With a span of 81 m, the bridge consists of five steel arches resting on massive natural stone masonry piers. Rehabilitation works were performed in 2003. The rehabilitation project involved

- repair of corrosion-damaged steel rebars in the RC deck slab
- replacement of the waterproofing membrane on the top surface of the deck
- renewal of the corrosion-protective coating on the steel structure
- redesign and installation of new and higher bridge railings (Brühwiler and Gue, 2003).

Special attention was paid to the choice of colours and structural details in order to respect the bridge’s original character. The bridge, with the Cathedral in the background, provides a postcard view of the old city of Lausanne. In 2006, a new bridge for the Lausanne Metro line was built underneath the arch bridge. This bridge follows the axis of Bessières Bridge and crosses through its masonry piers – an original urban ‘bridge landscape’ was created!
4. Conclusions

Construction interventions on bridges of high cultural value must meet stringent utilisation requirements in tandem with their cultural value and aesthetic quality. The examples presented in this paper highlight that most bridges have a certain cultural value that needs to be recognised and respected. There are still many ‘undiscovered’ and ‘ignored’ bridges built over the last 60 years, which deserve similar treatment to those presented here. Structural engineers and bridge owners need to be more aware of these aspects when conducting rehabilitation or modification interventions.

Structural engineering with the ultimate goal of limiting construction intervention to a strict minimum is intertwined with the interest of preserving monuments and limiting costs to bridge owners. There are no ‘old’ bridges – there are bridges that provide adequate performance and those that do not. Extending the service life of bridges by following the approaches presented in this paper will allow continuous utilisation of existing structures rather than their replacement. This approach is clearly in agreement with the principles of sustainable development. Finally, extending service life means not only giving value to the bridges themselves, but also appreciating the identity of the engineers and the art of structural engineering.

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Construction engineers are increasingly faced with tasks in which a balance must be struck between historic structural designs and modern use requirements. Although they usually master these challenges, a thorough understanding of historic structures and conservation objectives is not widespread. However, such an understanding is key for proper handling of buildings with high conservation significance. In the case of the reconstruction of the Neues Museum in Berlin, Germany, the most important responsibility of the engineers was to recognise, at an early stage, that purely theoretical approaches would not deliver the intended result and that all parties involved in the project needed to engage in the discussions, which also had to be facilitated. Key services to be provided included the development of verification concepts, planning and designing the required tests in a cross-disciplinary approach and coordinating the activities with the supervising authority.

1. Introduction

The Neues Museum in Berlin, Germany was built from 1841 to 1859 under the management of Friedrich August Stüler, a pupil of Karl-Friedrich Schinkel. Although designed with a solid appearance to the outside observer, the building’s interiors were a compelling combination of the latest cast-iron technology and ancient lightweight construction techniques. Even the particularly well-organised and systematic building process in the shell construction phase might serve as a good example for some sites of today. However, shortly after its opening, the museum building began to show settlement damage. This settlement activity persisted and reached a depth of 40 cm at the most unfavourable location.

In 1943, during the Second World War, the central staircase hall was bombed out. The north-west wing, the Egyptian courtyard and the south-east dome suffered the same fate in 1945. Some parts of the building were left fully exposed to the elements for over 40 years. The significance of the remaining fabric was recognised in the mid-1980s, prompting a decision to reconstruct the Neues Museum; however, significant portions of the damaged building sections had to be demolished.

Initial work to stabilise the foundations and the building fabric had begun even before the reunification of Germany. Planning and design of the reconstruction commenced in 1998 and the first construction work started in 2003. The structural work was completed in 2008 and the museum reopened in October 2009.

This article describes some of the aspects relevant to the work of the construction engineers who dealt with the historic fabric in the course of planning the reconstruction. Nowadays, a single engineer or even a group of engineers is no longer capable of mastering such tasks alone. This type of project requires a number of experts from various fields.

Some historic structures are unsuitable for a formal evaluation solely based on generally accepted verification methods or modern codes and standards. For this reason, experimental methods had to be considered at an early stage in order to support the structural stability analysis required for the historic elements (Steffens, 2001). The verification methodology was defined at the concept planning stage and implemented experimentally in close cooperation with Prof. Steffens, Ingenieurgesellschaft mbH (PSI), Bremen.

2. Problems with the foundations and their restoration

Neues Museum is located on an island in the middle of the River Spree, which runs through the heart of Berlin (Figure 1).
The ground is characterised by unconsolidated Holocene inclusions in the otherwise sound Pleistocene bedrock, which is typical of its location in the Berlin–Warsaw glacial valley. Of all the buildings on Museum Island, Neues Museum is particularly affected by this inhomogeneity. The structurally stable ground has a downward gradient from the south-east to the north-west corner, where it lies approximately 25 m below the bottom edge of the foundation (Figure 2).

This situation was known when the original building was constructed – an attempt was made at resolving the issue by inserting a wooden pile foundation. However, since the length of the wooden piles was limited, the north-west area rested on a ‘floating’ pile foundation (Figure 3), which began to cause damage to the building shortly after its opening in 1855 due to uneven settlement. This damage required subsequent repair. The situation was exacerbated by the fact that the groundwater level was lowered several times for extended periods during construction of nearby buildings. As a result, the pile foundation and top wooden grid became infested, which triggered irreversible rotting of the grid and the pile heads.

2.1 Restoration of the foundations
A pile-supported foundation slab in the area of the former north-west wing and the new foundations for the adjoining walls had been cast prior to the reunification of Germany. After reunification, Ingenieurgruppe Bauen was commissioned with continuing this work.

The first task was to assess the structure that had been inserted to transfer loads from the masonry to the new piles. Part of the remedial work carried out before reunification was the insertion of a ‘high-tech’ structure comprising stainless steel block connectors and prestressed steel ties; however, this placed high peak loads on the historic brickwork (Figure 4).

Ingenieurgruppe Bauen then proposed a ‘robust’ design (Figure 5) that required less drilling in the original building fabric, enabled the use of standard structural steel and reduced the amount of stress transferred to the historic brickwork. The relevant structural and economic factors were carefully considered, and the new robust design was chosen for the remaining walls.

Small composite bored piles were used (consisting of steel pipes and reinforced concrete) that had been classified as a special

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Figure 1. Aerial photograph of Museum Island, Neues Museum (provided by Planungsgruppe Museumsinsel)

Figure 2. Levels of the structurally stable ground, approximately 3.5 to 25 m below the bottom of the foundation (courtesy of Ingenieurgruppe Bauen)
design in East Germany and which were constructed as tubular piles with a national technical approval after the reunification of Germany. From 1990 to 1994, 2568 piles in lengths ranging from 9 to 32 m were inserted, corresponding to a total length of approximately 50 km.

3. Settlement measurements and building stabilisation
A comprehensive series of measurements was carried out simultaneously with the foundation work in order to monitor movements of the building during these activities and to
stabilise it in the event of any disruptive movements due to drilling. After the reunification of Germany, these measurements were systematically analysed with a view to stabilising the building fabric. These measurements are being continued using a correspondingly adapted measurement programme.

A comparison of the values measured at significant points during the construction work yielded the following findings.

- Drilling work inevitably increased the rate of settlement for a certain period.
- The settlements could be considered ‘accelerated’ phenomena that would have occurred over time in an uncontrolled manner.
- The movement patterns of certain parts of the building that occurred due to damage previously caused by the inhomogeneous ground were revealed even more clearly by the drilling work. Remedial measures for stabilising the building were derived from these patterns.
- After the foundation slab had been installed, identical settlements were measured for the foundation slab and the walls, showing that the load-transfer structure is effective.
- Secondary settlements that occurred after hardening of the foundation slab were minor. Differences in settlement between neighbouring measuring points remain within reasonable limits and can be compensated by the building fabric.
- Reconstruction work should aim to maintain the achieved uniformity of settlement and deformation.
- Minor local damage caused by differences in settlement cannot be excluded.

3.1 Stabilisation of the building fabric

While the stabilisation work was being planned and executed, there was still no concept for the final reconstruction of the building. For this reason, planning and implementation followed the principle of stabilising the respective existing condition while avoiding any restrictions for reconstruction. Furthermore, as far as reasonably possible, all new work should be either reversible or reusable for the reconstruction.

One immediate remedial activity was to consolidate cracks that posed a risk to the structural stability of the building. The following work was performed once settlements had ceased.

- Settlement-induced gaps between individual shear walls and building sections were closed.
- Detached stiffening walls were reincorporated in the overall structure.
For reasons of building physics (weather protection), as many as possible of the smaller, structurally less significant cracks were also consolidated.

In a final step, the walk-on stability of the surviving ceilings had to be examined and secured for the interim period (Eisele and Seiler (1999) give a comprehensive report on this topic).

4. Aspects of structural design during reconstruction

As a result of the industrial revolution in England, the Neues Museum was one of the first prestigious buildings in Berlin in which the use of iron became a distinct feature in both structural and architectural terms. Because of the issues that arose from the unstable ground, dead loads had to be minimised and masonry ceilings built on ‘clay pots’ were thus used.

Given the history of the building, most parts of the surviving original ceilings were found to be in astonishingly good structural condition. However, real problems arose in some of the floor bays where weather exposure had damaged the fabric or where significant sagging occurred in central sections due to the removal of lateral supporting members.

The client requested that all ceilings should be upgraded to fulfil current requirements so that they could withstand service loads of up to 5 kN/m\(^2\) in accordance with DIN 1055. Estimates and information given in the original building documentation give rise to the assumption that load-bearing capacities of 200 kg/m\(^2\) (2 kN/m\(^2\)) were considered sufficient at the time of construction (Hoffmann, 1846). The structural design aimed to preserve the historic structures as a technical monument and to return them to their original purpose, as undisturbed as possible, while also responding to the client’s requests. The remaining original structures took priority so that the exhibition layout could be adjusted to the specific situation as and when required.

Structurally supporting members, such as large sections of the iron structures, were evaluated using modern verification concepts. The investigations revealed that some of these structures provided significant load-bearing reserves for their original purpose.

4.1 Assessment of the load-bearing capacity of clay pot ceilings

The concave design of the ceiling areas was based on an ancient technique that used cylindrical, completely closed, hollow clay ‘bricks’ laid in gypsum mortar, which were referred to as ‘clay pots’ in the project documents. This method provides an extremely low weight per unit area at remarkably high load-bearing capacities (Figures 6–8).

The clay pots were manufactured with only 7–10 mm thick walls and lids. They were used as ‘bricks’ for various purposes, again held together with a gypsum mortar. Among other features, this construction method was used for the calotte ceilings consisting of transverse arches and brickwork pendentives interspaced with spherical domes of around 4–5 m in diameter (Figure 6), barrel vaults with 5–6 m spans (Figures 7, 9 and 10) and vaulted ceilings between steel beams.

Figure 6. (a) Calotte ceiling consisting of clay pots. (b) Condition prior to reconstruction (courtesy of Ingenieurgruppe Bauen)
4.1.1 Structural analysis

In the initial step, the loads acting on the existing clay pot ceiling systems (calotte ceilings, barrel vaults and vaulted ceilings) were estimated on the basis of preliminary calculations. For this purpose, assumptions had to be made regarding material characteristics such as the modulus of elasticity, load-bearing capacity and stiffness of the system. All reference documents found in the literature were used. The analyses of a historical load test performed on a single-bay vault (Hoffmann, 1846) were compared with the results of a calculation using a 3D finite-element (FE) model. Parametric studies carried out for structurally determinate subsystems were used to vary the derived assumptions for the material characteristics and to analyse the results of the calculations.

The basic findings from this work were as follows.

- The highest loads acting on the clay pot structures are in the barrel vaults.
- Due to the relatively low dead loads, the effects of loads that vary from bay to bay (‘kinematics’) must also be considered for the barrel vaults because they form multi-bay systems.
- The design can be interpreted as a combination of two load-bearing effects: a diaphragm effect in the plane of the pot lids and a more flexible honeycomb structure in the cross-section.
- The service loads that the ceilings may potentially withstand cannot be determined without supplementary experimental tests.

4.1.2 Experimental tests

The findings of the structural analysis were used to develop the following programme for estimating the load-bearing capacity of the clay pot ceilings.

- Preliminary load test on an existing four-bay barrel vault to identify the system.
- Basic tests on small specimens with associated numerical analyses.
- Load test on a newly built sample vault.
- Development of an appropriate computation model and structural verifications for all clay pot systems.
- Confirmation of the computation model, and thus of the load-bearing capacity, using load tests in selected locations.
- Inclusion of the plaster floor forming part of the historic ceilings in the load-bearing capacity analysis and evaluation of its contribution to the ceiling areas to be reconstructed.

4.1.3 Preliminary load test on an existing four-bay barrel vault (Figure 9)

A four-bay barrel vault with spans of approximately 6 m was chosen for the preliminary load tests. Two bays were alternately loaded in order to analyse the stiffness of the overall system, deformation behaviour, the modulus of
elasticity of the clay pot mortar matrix, torsion of the transverse arches, continuity effect and biaxial structural effect.

Mobile load frames were used to generate service loads that varied from bay to bay.

Online measuring equipment and an associated acoustic emission analysis (AEA) were used to ensure that the testing was non-destructive. The system for a subsequent FE calculation was derived from the measured deformation and deflection values.

4.1.4 Basic tests on small specimens

In addition to conserving and repairing the existing ceilings, two of the originally seven bays of a barrel vault system were to be complemented by five new clay pot floor bays replicating the original design. This meant that the clay pots and the gypsum mortar mixes had to be selected so as to ensure that their mechanical properties and appearance largely corresponded to the original materials. To achieve this goal, laboratory tests were conducted on about 30 small specimens of various ‘pots’ and mortars in different combinations and load cases. An AEA of these small specimens was performed for calibration purposes.
The materials ultimately used for the reconstruction work were selected on the basis of these tests. The tests and associated calculations with a refined model based on an elastoplastic material law were used to prove the existence of a redundant structural system. After failure of the rigid outer shell (i.e. pot lids breaking away), the load shifts to the honeycomb structure formed by the pot walls and the mortar within the cross-section; this is associated with a corresponding increase in deformation.

4.1.5 Sample vault
Since the original vault could not be used for limit load tests, a new full-scale two-bay sample vault was constructed within the building. This structure also provided an opportunity to test specific techniques and work sequences. In the first test series, the effect of a partially load-bearing plaster layer was investigated. A supplementary load-bearing plaster floor was included in subsequent testing. The structure was loaded to failure in the course of the second series of measurements and included the structural effect of the plaster. The results complemented the values calculated for the existing and newly designed floor bays (Figures 10 and 11) and ultimately confirmed that the load-bearing capacities were sufficient for museum use.

4.2 Assessment of the load-bearing capacity of the limestone columns
Columns made of various natural stones (sandstone, limestone, marble) were used in the central axes of the individual exhibition rooms (Figures 6 and 12). In addition to their dead loads, the columns are subjected to higher service loads resulting from museum operation and, to a certain extent, to increased loads due to the construction of an additional floor to accommodate building services and installations under the roof. The condition of the individual column sections is significant because various weather conditions had affected the materials (which were not necessarily suitable for outdoor use) during the long period of neglect as an unprotected ruin. Thus, columns with Carrara marble capitals and bases and limestone shafts (Figure 12) had to be examined more thoroughly. Most
of the columns are still in their original positions. Two undamaged columns and the fragments of four broken columns had been put into storage.

The shafts consist of ‘Pyrenean marble’, which is a limestone of the ‘marbre campan melange’ variety. This material is not homogeneous; its external surfaces show veins in various colours and orientations (Figures 12–14). The original documents revealed that remedial work was necessary even before their original installation in order to repair transport damage.

Initial ultrasonic and georadar measurements delivered reference values that made it possible to classify the existing items. However, even at this stage, theoretical verifications alone were insufficient because the internal load-bearing behaviour could not be captured adequately by an idealised model. The degree of interlocking and geometrical shape differ from joint to joint, which is why they are not accurately reflected in the calculations. Again, it appeared useful to combine theoretical verifications with experimental tests.

Specially prepared ‘drums’ made of column fragments salvaged in the course of the demolition work in the 1980s were subjected to concentric and eccentric loads. AEA was used for a more accurate determination of their load-bearing potential and both non-destructive and limit load tests were carried out (Figure 13). In the next step, a mobile test rig was used, again in an AEA-controlled setting, to subject the two column shafts to concentric and eccentric service loads that were increased by inclusion of a safety factor (Figure 14).

Although sufficient load-bearing capacity was found for concentric loading, the permissible load had to be limited for even minor eccentricities owing to the inhomogeneous structure of the limestone. Conversely, this meant that the entire structure would withstand the actual loads if eccentricities could be kept minor during load application.

In the next series of tests, further column shafts identified as particularly critical in the course of the preliminary examinations were successfully tested in situ in a vertical position using appropriately modified loading devices (Figures 15 and 16). Despite the reduction in load-bearing capacity detected for eccentric loading, the existing geometric inhomogeneities in the columns were measured on site and directly applied to the computational verifications. A comparison with the eccentricities reached in the tests using specific load scenarios then made it possible to calculate the permissible load-bearing capacity with sufficient accuracy.

4.3 Reinforcement of cast-iron girders with CFRP sheets

In keeping with the trend of the time, the original building was constructed with many cast-iron components. This is unproblematic wherever such structural members are loaded mainly in compression. However, cast-iron sections were also used as ‘joists’, between which the infill brickwork could be inserted very easily to form vaulted ceilings. These joists are subjected to bending with a marked tensile bending zone, which is why they also had to be verified for the service loads required by modern use patterns. The tensile strength of the original cast-iron amounts to only 30–40% of its compressive strength. Inhomogeneities caused by the manufacturing process (shrinkage cavities) weaken the tension zones, thus leading to a dangerous susceptibility to brittle failure.
Approximate calculations using design methods specified in the literature (Frey and Käpplein, 1993) resulted in theoretically permissible service loads between 1.0 and 2.0 kN/m² assuming undamaged structural components – quite optimistic given the history of the building. These values were insufficient if the building is to be used as a museum.

Historic cast iron cannot be welded and bolted steel fish plates were not an option owing to the shape of the elements. One possible solution was to reinforce the iron with carbon fibre reinforced plastic (CFRP) sheets in the tensile zone (Figure 17). This method is widely known in concrete construction and complies with conservation requirements because mechanical interventions in the member to be reinforced are not necessary. In addition, installation of the reinforcement is reversible.

The first step was to obtain basic information on the behaviour of cast-iron beams. New beams were cast for this purpose. In our ‘high-tech’ society, it is not at all easy to produce grey cast iron to the lower quality standard equivalent to the original condition. The beams were partially damaged prior to testing and subjected to experiments to determine their load-bearing capacities in the reinforced and non-reinforced condition (Figure 18).
On the basis of the test results, a realistic design method was developed that proves a sufficient degree of post-reinforcement safety, even for previously damaged cast-iron beams. The reinforcing CFRP sheets were selected and dimensioned on this basis (Figure 19): they are 1.4 mm thick and 50 mm high.

For verification purposes, on-site confirmation tests were performed in selected trial bays after installation of the reinforcement. These tests provided impressive proof of its effectiveness. The specified service load of 5 kN/m² was thus confirmed for all historic ceilings.

Works by Eisele et al. (2004) and Eisele (2006) give a more detailed account of this section.

5. Summary
Increasingly, construction engineers need to strike a balance between historic structural designs and modern use requirements. Although these challenges are usually mastered (virtually anything is possible from a purely technical point of view), thorough understanding of historic structures and conservation objectives is not widespread. However, such an understanding is key to successful treatment of buildings with high conservation significance.
Figure 17. (a) Ceiling above the vestibule; panelled ceiling consisting of clay pots between cast-iron girders after reinforcement, partly relined and (b) ceiling section and detailed section with CFRP sheet reinforcement underneath historic soffit lining made of zinc (dimensions in mm) (courtesy of Ingenieurgruppe Bauen).

Figure 18. Fracture patterns of notched beams, notch size approximately 4 mm. Fracture close to the notch at the point of load introduction (courtesy of Ingenieurgruppe Bauen).
In the case reported in this paper, the most important responsibility of the engineers was to recognise, at an early stage, that purely theoretical approaches would not deliver the intended result and that all parties involved in the project – such as the client, user, architect and required experts – needed to engage in the discussions, which also had to be facilitated. Key services that needed to be provided included the development of verification concepts, the planning and design of the required tests in a cross-disciplinary approach and coordination of all the activities with the supervising authority.

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- Test engineer: Dr.-Ing. Hartmut Kalleja, Berlin.
- Experimental structural testing: Prof. Dr.-Ing. Steffens, Ingenieurgesellschaft mbH (PSI), Bremen.

REFERENCES


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The construction of the Kiev Suspension Bridge 1846–1853

John Vignoles MA, CEng, MICE
Civil Engineer, retired

In 1846, the Tsar Nicholas I of Russia commissioned Charles Blacker Vignoles to build a fixed crossing over the River Dnieper at Kiev. Opened in 1853, the resultant suspension bridge, involving the construction of foundations in the fast-flowing river, was the largest multispan suspension bridge in Europe at the time. In the paper, the author, a direct descendant of C.B. Vignoles, describes the design, procurement and construction of the bridge between 1846 and 1853, drawing on information from Vignoles’ journals and letters, and also from other contemporary documents. He also makes use of material derived from recently discovered sketches and progress photographs taken by John Cooke Bourne and Roger Fenton – this was one of the first times photography had been used specifically to record construction progress.

In December 1846, Vignoles returned from a 2-month break to find advice from Colonel du Plat that Tsar Nicholas I was inviting designs for a bridge crossing the River Dnieper at Kiev, to be submitted in St Petersburg by the end of January 1847. He decided to submit a proposal, a decision that was to change his life forever.

The River Dnieper at Kiev was slow-moving and sluggish in summer, and frozen in winter. It was a main highway for barge traffic, mainly timber but also other commodities. During the summer, when the river was free-flowing, the 2200 ft (670 m) wide river could be crossed by a bridge of boats. In contrast, during the winter, the river could only be crossed over the ice. Problematically, there was no permanent crossing available all the year round, as the bridge of boats could not survive the intermediate freeze/thaw period, when the river currents could be extreme.

As Vignoles was later to explain in an article in the London Times, ‘...it became a necessary condition that the number of piers of any bridge to be built there should be the fewest possible, with the largest openings between them. Hence it seemed most natural that, within the given limit of expense, the principle of a suspension bridge should be preferred.’ (The Times, 1850)

Vignoles was familiar with a number of suspension bridges in England (see Table 1). He knew that important decisions would need to be made, in particular the system of deck suspension, whether with wire ropes or wrought iron link chains, and the make-up and stiffness of the deck, whether in...
iron or timber. Failures had occurred on both the Menai and Montrose bridges, requiring significant reconstruction, and reports of these had been reported in the *Proceedings of the Institution of Civil Engineers* (Maude, 1841; Rendel, 1841).

In the few days available to prepare a preliminary design, Vignoles worked in a number of different directions. He consulted various technical papers at the Institution of Civil Engineers (ICE); and, considering that wrought iron links were preferable to a wire rope suspension system, he wrote to various ironmasters concerning the provision of the links for the suspension chains. He discussed the strengthened Montrose Bridge deck with J.M. Rendel, who had rebuilt it in 1840. He also consulted with W. Tierney Clark, the designer for the Budapest Bridge, which was presently under construction, with whom he had worked many years before in 1823/4 on the Hammersmith Bridge design (Smith, 1991).

In a space of 2 weeks, Vignoles produced a memorandum of the principles involved and an abstract of quantities to send to his Warsaw partners. He commissioned a proposal document from Charles Cheffins, an experienced publisher of engineering drawings and documents, whose establishment of expert draughtsmen and lithographers was accustomed to producing proposals at speed for parliamentary submissions (Cross-Rudkin et al., 2008). To accompany the proposal, he commissioned John Cooke Bourne, the illustrator of the London and Birmingham and Great Western Railways, to make a watercolour sketch of the crossing.

On the morning of 3 January 1847, we read in Vignoles’ journal that he: ‘...received finally the Geometrical and perspective Drawings of the Kiev Bridge magnificently finished and mounted in the manner required and enclosed in a splendid Box.’ (Vignoles, 1847).

It remained only to pay Mr Cheffins for the work and to make arrangements for his office manager Mr Cummins to conduct business in his absence.

### 2.3 The proposal

Vignoles describes the bridge layout as follows:

‘...[The bridge] has four principal openings, each of 440 feet, and two side openings of 225 feet each, and also a passage of 50 feet on...’
the right shore, spanned by a swivel bridge, opening for the passage
of the steamboats and other river craft. There are therefore five
suspension piers in the river, one mooring abutment on the left
bank, another mooring abutment on the Kiev side of the stream
(which, on account of the passage for boats beyond it, is actually an
island of masonry in the river), and an abutment for the swivel
bridge on the right bank. Each of these have required a cofferdam
of unusual size, particularly the two last mentioned.…’

‘…the platform is suspended by chains, all on the same horizontal
plane, two on each side of the road; the footpaths project beyond
the chains and are carried by cantilevers round the piers exteriorly,
so that the foot passengers are completely separated from the
horsemen and carriages…’ (The Times, 1850)

Figure 2 shows the layout of the proposed suspension bridge,
with the Budapest Bridge shown at the same scale. The piers P1
to P5 are founded in cofferdams D2 to D7. The suspension
chains are moored in foundations D2 and D8. The swivel
bridge is mounted on abutment D1.

Figure 3 shows the bridge deck arrangement, and Figure 4 the
arrangement of the swivel bridge, including foundations.

The philosophy behind the deck or platform construction (see
Figure 5) may be found in a technical report from the
American Commissioner to the Great Exhibition of 1851,
which, from the writing style, was probably written by
Vignoles himself.

‘…The [four] chains are composed of broad, flat links, twelve feet
long, and weighing about four hundred weight each. The tie rods
which hang from the chains at each side are two inches in diameter,
and are immediately connected to the girders which support the
platform.

‘The manner in which the platform is constructed is the chief
novelty which has been introduced in their structure, and
consists in a judicious combination of iron and wood, the object
being to obtain a light and stiff platform. Two kinds of girders
are adopted here, namely: trussed girders and tension girders;
the trussed girders are chiefly composed of wood, and are
deeper than the tension girders, which latter are rendered rigid
by tension bars. One set of chains supports the trussed girders
and the other set supports the tension girders; and these occur
alternately. The additional depth of the trussed girders is for the
double purpose of stiffening the platform and supporting the
foot-paths which are outside of the chains. The trussed girders
are connected underneath, at each end, by longitudinal ties
which run the whole length, and [by] the balustrades which
separate the carriageway from the footpaths; they act con-
jointly with the ties underneath in checking any tendencies to
undulation, the girders also being braced diagonally to prevent
any side play.’ (Riddle, 1852)
Figure 2. Layout of Kiev Suspension Bridge (1 ft = 1 foot = 0.3048 m)

Figure 3. Bridge general arrangement – plate 45 from Practical Draughtsman (Johnson, 1869) (1 ft = 1 foot = 0.3048 m)
The development of the deck design was clearly influenced by the earlier bridges.

The Menai Bridge deck alterations introduced articulated deck bearers, to ensure that the load was shared equally between the four suspension rods (see Figure 6), and the timber kerbs were increased in size to provide some form of longitudinal continuity (Maude, 1841).

For the Montrose reconstruction, Rendel adopted a timber deck, using tensioned cross-beams supported on alternate chains. To provide adequate strength, one cross-beam in six was of a trussed...
construction, and the structure was further stiffened by longitudinal timber framing (see Figure 7) (Rendel, 1841).

For the Budapest Bridge, Tierney Clark used cast-iron cross-beams, stiffened longitudinally with iron trusses (see Figure 8) (Clark, 1852). There are two pairs of suspension chains, one above the other. The deck cross-beams are hung by twin hangers from alternate chains. A particular innovation at Budapest, which would also be adopted by Vignoles at Kiev, was the use of fully rolled chain links, in which the eyes were part of the same rolling (see section 3.5, below) (Howard, 1849).

3. Design submission and procurement

3.1 Proposal submission: January to March 1847
On 3 January 1847, Vignoles set off for St Petersburg to submit his proposal, accompanied by two of his sons, Hutton aged 24, and Henry aged 20, travelling via Stuttgart, Vienna and Warsaw. In Warsaw, he met Colonel du Plat and the Evans...
brothers, and discussed the form of their partnership. After a pause of several days, while they established the budget and firmed up the wording of the proposal, the journey recommenced, reaching St Petersburg on 1 February.

On 12 February, Vignoles was introduced to Count Kleinmichel, the Minister of Public Works, who asked that he prepare copies of the design for the bridge showing the depth of water at various seasons and also the transverse section of the river. In typical style, while waiting to see the Tsar, Vignoles also sketched outline designs for crossings of three strategic rivers that he had encountered during his journey from Warsaw.

On 16 February, he showed all the plans to the Tsar, who was sufficiently impressed with the proposal for Kiev to ask him to prepare a full design, and promised that a letter would be written to that effect.

With that understanding, by 10 March, Vignoles had returned to London to organise a firm design, stopping in Warsaw to sign an agreement with his partners and with a Polish contractor, Mr Blomberg, who would undertake the construction. He had sent Hutton Vignoles to Moscow and then Kiev on a provisional reconnoitre for materials.

On 7 April 1847, Vignoles received the promised letter giving official acceptance of his proposal for the Kiev crossing at an estimated price of approximately £239 000 (1 670 000 roubles), approximately £20 million in today’s money.

3.2 Design development: April to June 1847

Back in London (see Figure 9) Vignoles’ next step was to work on the structural design, his personal responsibility, and he approached the task with his usual verve and enthusiasm. He made visits to Montrose, Shoreham and Hammersmith bridges;
he visited iron works to look at raw material for the chain links; he looked at steam pile driving machines; he investigated the cost of shipping freight to Kiev from Liverpool via the port of Odessa; and he discussed architectural details. Finally, he commissioned John Cooke Bourne to draw a revised proposal.

He also set up a design office at Preston to prepare drawings for the foundation construction, using the services of William Coulthard, who had been associated with Vignoles for a number of years as both engineer and contractor, and was also a close friend of the family. In this work, Coulthard would be assisted by his son, William Robson Coulthard, by Hutton Vignoles, now back from Kiev, and by John England; young engineers who, after working on the bridge construction at Kiev, would all proceed to successful engineering careers (Cross-Rudkin et al., 2008).

On 16 June 1847, the final design proposal was at last complete, and a fresh portfolio prepared for submission to the Tsar. The signed portfolio, entitled ‘Dessins Detaillés du Pont Suspendu a Kiev’ and signed ‘Charles Vignoles, Ingenieur Civil Juillet 1847 No 4 Trafalgar Square London’ is today in the Scientific Technical Library of the St Petersburg State Transport University (see Figure 10).

3.3 The site visit and design approval: July to September 1847

In July 1847, Vignoles returned to Warsaw with the final design. He then visited Kiev for his first view of the site itself—up until now, he had been reliant on descriptions made by others. The journey involved over 80 h travelling in each direction by coach on unmade roads for a visit of 2 days. While there, he commissioned searches for timber and stone, and also for a foundry that could make piling shoes.

Back in Warsaw, 8 days later, Vignoles consulted with his partners about the financial agreement. Significantly, as it was to turn out, the risks and rewards rested with Vignoles and Colonel du Plat, who was the senior partner in the venture. The Evans brothers declined to take a share in the risk, but agreed in all other respects to assist him, and to endeavour to get Mr Blomberg either to continue on the original terms or to take an extra fee for doing the work as the contractor.

Vignoles then returned to St Petersburg, where he was to remain for 6 weeks negotiating the design with the 30-man bridge commission appointed by the Tsar. Various dimensional changes were requested, and the suspension calculations were challenged, with negotiations going round in circles. It soon became clear to Vignoles that no-one was prepared to take responsibility for approving the works. Frustrated by the delays, at the beginning of September Vignoles managed to arrange a meeting with the Tsar, who asked him to confirm personally that his calculations were satisfactory, and to take account of the requested dimensional changes. This he agreed to do.

On 22 September, Vignoles met the Tsar again, this time on site at Kiev, and the various amendments were agreed. Eventually, at the end of September, Vignoles received a signed contract. Vignoles writes in his diary ‘Thus after 9 months of tedious discussions this affair of the Kiev Bridge was brought to a satisfactory conclusion.’
3.4 Works contract

Returning to England via Warsaw, Vignoles agreed the terms of the contract with Mr Blomberg to supply the labour force and construct the works at Kiev, including the procurement of locally available materials of timber and stone. All ironwork and temporary works machinery would be procured by Vignoles in the UK, with payment direct from Russia. The principal quantities involved are listed in Table 2. Before leaving Warsaw for London, Vignoles had a Power of Attorney drawn up to enable Blomberg to order and mobilise the work; he also engaged a Polish engineer to go to Kiev and carry out a preliminary survey of the site.

3.5 Procurement and mobilisation: October 1847 to February 1848

On 7 October, Vignoles arrived back in England and immediately started to make preparations for the start of work on site in the new year.

On 16 October, a contract was placed with John Musgrave and Sons, Globe Works, Bolton to supply the temporary works ironwork. The first shipment was loaded at Liverpool and shipped out to Odessa in December, followed by a second shipment in January. In all 22 shipments of permanent and temporary materials would be shipped from England (see Figure 11).

Also in October, Vignoles spent 3 days in Ireland consulting on the mathematics of the design with his friend and colleague, Mr Thomas F Bergin MRIA, the respected manager of the Dublin and Kingstown Railway, and his nephew Edward Whiteford, who was studying mathematics at Trinity College Dublin. For an independent check, they approached Thomas Romney Robinson FRS, who had been Astronomer Royal at the observatory at Armagh since 1822. As Professor Robinson would accept no fee, Vignoles presented him with a Whitworth slide-lathe. This is a typical example of Vignoles’ ability to build a useful and competent network of advisers.

The chain procurement had involved numerous discussions and letters to ironmasters. In 1845, Mr Thomas Howard, of the King and Queen Iron works at Rotherhithe on the Thames, had devised a method of rolling suspension chain links that avoided the need for separately welding the eyes. The works was presently engaged in making 5000 links for the Budapest bridge (Howard, 1849). Vignoles arranged for sample links to be rolled from raw material by Thornycrofts, a Wolverhampton foundry working under licence from Thomas Howard. Full-size tests were carried out on these links at Fox Henderson’s works in Birmingham. The tests demonstrated that the link capacity increases with the pin diameter, leading Vignoles to adopt a pin of 5 in (12 cm) diameter as opposed to the 4-5 in (11 cm) pin used at Budapest. Sir Charles Fox later described the tests in a paper to the Royal Society (Fox, 1865). The link drawings were issued on 18 December 1847 to a number of ironmasters, and finally on 23 January 1848 we find Vignoles checking the contract with Fox Henderson for the supply of rods, pins and chains. The quantity of individual links was said to extend approximately 17 miles (27 km) in length. Each individual link
Exhibited (at the request of the Directors) in the Crystal Palace, Sydenham, 1854

**MODEL**

**KIEFF SUSPENSION BRIDGE**

Recently erected across the

**RIVER DNIEPER**

Near the

**CITY and FORTRESS of KIEFF**

By order of

**HIS IMPERIAL MAJESTY NICHOLAS I, EMPEROR RUSSIA**

**CHARLES VIGNOLES, FRAS, MRIA, ENGINEER**

First stone laid 9 September 1848 – Opened 10 October 1853

The model by Jabez James, A Inst CE, Mechanical Engineer

Scale of model one inch to eight feet – equal to 1/96th (0.0104166) of real length

The suspension chains and bolts executed by Messrs. Fox, Henderson & Co., London Works, Birmingham

The other iron works executed by Messrs. Musgrove [sic] & Sons, Globe Works, Bolton-le-Moors

Principal dimensions etc.

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extreme length, 854 yards – nearly half a mile or…...</td>
<td>2562 ft</td>
</tr>
<tr>
<td>(366 Russian sagenes = 776 French metres)</td>
<td></td>
</tr>
<tr>
<td>Extreme breadth, 17½ yards or.....</td>
<td>52 ½ ft</td>
</tr>
<tr>
<td>(7½ russian sagenes – 16 French metres)</td>
<td></td>
</tr>
<tr>
<td>Spans</td>
<td></td>
</tr>
<tr>
<td>Each of the four large openings from centre to center of the river suspension towers or piers.........</td>
<td>440 ft</td>
</tr>
<tr>
<td>Each of the two side openings, from centre of the suspension tower to the face of the abutment...........</td>
<td>225 ft</td>
</tr>
<tr>
<td>Swivel bridge opening in the clear...</td>
<td>50 ft</td>
</tr>
<tr>
<td>Clear waterway at highest floods...</td>
<td>2140 ft</td>
</tr>
<tr>
<td>Height of platform above ordinary summer water-line...</td>
<td>30 ft</td>
</tr>
<tr>
<td>Greatest rise of floods (after the melting of the snows in the spring) above the ordinary summer water level........</td>
<td>20 ft</td>
</tr>
<tr>
<td>Greatest depth of water in the channel at summer level...</td>
<td>40 ft</td>
</tr>
<tr>
<td>Ditto at highest floods..............................</td>
<td>60 ft</td>
</tr>
<tr>
<td>Extreme height from deepest foundations to the top of caps of the suspension towers or piers, ..................</td>
<td>112 ft</td>
</tr>
<tr>
<td>Breadth of portals piercing the suspension towers........</td>
<td>28 ft</td>
</tr>
<tr>
<td>Height of ditto..................................</td>
<td>35 ft</td>
</tr>
<tr>
<td>Chord of chains of large openings, clear of piers........</td>
<td>416 ft</td>
</tr>
<tr>
<td>Versed sine of ditto.............................</td>
<td>30 ft</td>
</tr>
</tbody>
</table>

Total weight of all the platforms and rods, clear of the piers (42 lb per square foot)........................... 2350 tons

Weight of chains and pins, suspended clear of the piers.................. 1076 tons

Total weight of the four chains and pins only.................................. 1578 tons

Minimum section area of the four chains, including pins and overlaps..... 328 sq in

Total weight of iron of all kinds used in the works........................... 3500 tons

Total quantity of timber used, including temporary works............... 500 000 cu ft

Total quantity of brickwork and masonry and concrete in the works........... 1 500 000 cu ft

Proof load per available square foot of suspended platform........... 63 lb

Total proof load calculated to be laid on the bridge for testing... 2350 tons

Actual load laid on the bridge for the test, 60 000 ft³ of sand, which being much wetted by heavy rain during the test weighed 1 cwt per cubic foot, being equivalent to the weight of 50 000 infantry soldiers or about........ 3000 tons

TOTAL ABSOLUTE EXPENDITURE about £432 000 sterling

Table 2. Statistics: transcription of 1854 brochure
would be tested using a proving machine developed for the purpose.

On 10 December, Vignoles arranged for a first financial drawdown from his bankers and had discussions with Colonel du Plat regarding his share of capital for the finances. Although a form of stage payments had been agreed with the client, responsibility for initial payment rested with the partners as was common at the time, and cash flow would always be a concern. Before leaving London, Vignoles would arrange a Power of Attorney for his office manager, Mr Cummins, who was based at 4 Trafalgar Square, London, to manage his affairs in his absence.

On 15 December, the foundation drawings were completed. The Preston office was closed and the design assistants travelled to London to join the party for Kiev. William Coulthard (senior) was to remain in England on a salaried basis, to provide a design service to the site as required.

Also in December, Mr Frost, who Vignoles had just appointed as river works engineer, was asked to investigate and procure some piling engines that he had seen working at Perth; and on 16 January 1848, 2 weeks before departure for Kiev, Vignoles travelled up to Newcastle to visit the Nasmyth steam pile drivers at work on the foundations for the Newcastle High Level Bridge. The river cofferdams, which were needed to permit the foundation construction, would require a significant number of timber piles to be driven into the bed of the river.

At the end of January 1848, 4 months after signing the contract, Vignoles set off to Kiev, where he intended to establish his own residence, as he had 10 years previously at Dinting Vale when working on the Woodhead Tunnel on the Manchester to Sheffield Railway. He took with him his daughter, Camilla Croudace (who was estranged from her husband), to keep house for him.

On 19 February 1848, the site team assembled at Kiev. Vignoles was to be chief engineer; with Hutton and Henry Vignoles, John England and William Robson Coulthard as assistant engineers. As well as Mr Frost and his son, there were two mechanical engineers, Mr Coulishaw and Mr Bell, and Mr George Pemberton who was a blacksmith. Mr Dacre White was to be responsible for the shipping import at Odessa. Mr Whiteford, the maths scholar from Trinity College Dublin, joined the team as general assistant. The contractor’s agent was Mr Schweitzer, and a Captain Kirchenpauer was construction engineer.

Also in the team was John Cooke Bourne, who had provided the preliminary design illustrations. As resident artist, he would be responsible for producing record drawings and sketches and photographs (calotypes) as required. According to Vignoles’ journal, at the close of each season a set of these
photographs was placed in an album for the client. It is understood that an album for the 1849 and 1850 seasons is in the library of the St Petersburg State Transport University.

A founder member of the photographic society, Vignoles was an early and enthusiastic advocate of the use of photography to record construction, using the latest methods available. In 1852, he would invite Roger Fenton to travel to site to take stereoscopic photographs of the chain erection process. In 1853, he would buy a new camera from Mr Delamotte, which Bourne would use to great effect. An album of Bourne's photographs for the 1853 season is in the National Museum of the History of the Ukraine in Kiev (Hannavy, 2004).

3.6 Construction programme
The construction programme, intended and achieved, is shown in Figure 12, and follows a linear process with working time governed by the seasons of the river. It shows that Vignoles anticipated that the cofferdams would be constructed in the first season (1848), and the foundations would follow in the next (1849). Once the foundations were out of the water, 2 years would be required for the piers and deck construction. This meant that the bridge was expected to be handed over in September 1851, 4 years after signing the contract, before the start of the winter season. As the achieved programme shows, however, there were significant delays to the river works and foundations, which would result in a 2-year delay in bridge completion until September 1853.

4. Construction 1848
4.1 Initial site works
The layout of the site works is shown in Figure 13. On arrival in Kiev, the team found as expected that it was still winter and the river was frozen over. This enabled them to work relatively dry-shod surveying the line of the bridge, establishing foundation positions, and drilling down to prove the ground. To their satisfaction, they found hard clay on the right bank under the heights. The remainder of the river bed was sand, which would lead to problems with the water-retaining cofferdams at each foundation. Much time was spent preparing plans and sections of the river, and deciding the bridge alignment. Vignoles appears to have carried out much of the direction of the works himself, and his journals are full of detail during this period.

Work also proceeded to establish the construction camp on the north or left bank and to set up and source stone and timber for the works. The course of the supply railway from the heights was also determined. This incorporated an inclined plane, or manually operated cable railway, which was surveyed and constructed by the end of the year, and would be instrumental in conveying materials to the work.

Once the river had thawed sufficiently, piling for both the temporary bridge supports and for the cofferdams started. The temporary bridge was built to carry a railway across the works, with pile bents at 54 ft (16 m) centres, and sidings at each cofferdam. There were seven cofferdams in number, some in deep water and some in shallow. The basic construction was of two concentric rings of timber piles, with the space between filled with puddle clay. The two rings were tied together with iron straps, and the interior would be propped with a system of timber struts and walings to take the pressure of the water. Figure 14 is a sketch by Bourne showing pile installation proceeding in one of the foundations on the left bank.

Vignoles approached the work on site with his customary optimism. This is evident from a letter he wrote on 16 May 1848 to Robert Stephenson (Cross-Rudkin et al., 2008),

![Figure 12. Planned and achieved construction programme](image)
congratulating him on the successful lift of the first Conway tube, and requesting his assistance with a parliamentary hearing on the North West Railway (Lancaster to Settle), as he could not leave Kiev at that time. He writes:

‘The very large work I am engaged in here for the Emperor of Russia has nothing novel in it to the English Engineers. I have been busily engaged in matters which in this country require all my personal care and superintendence, but I am happy to say that except in magnitude, there are no difficulties to be encountered. It is chiefly a question of time and I am fortunate in having got an experienced and honest contractor. Of course, everything except Brick, Stone and Timber must come out from England, even leather and cordage, but I trust my arrangements to carry on everything smoothly and at present the season is favourable.’ (Vignoles, 1848)

The first difficulties were, however, just around the corner. In mid-June cholera began to make its appearance. On 29 June, he notes in his journal ‘much concerned to find that 100 men had left in consequence of apprehensions of the cholera, though epidemic on the decline.’ The same day Vignoles noted that some of the temporary bridge piles had been broken during the night by barges and rafts travelling down the river. The contractor was reduced to borrowing soldiers from the garrison to man the piling engines. Sadly, on 2 July, Mr Whiteford, the engineering assistant from Ireland, was taken ill with cholera and died the same day.

Despite these difficulties, Vignoles returned to London in July, for a couple of months, leaving his son Hutton in charge, with instructions how to proceed. He took with him sufficient drawings and sketches to enable him to commission a large-scale model (1:96) of the bridge from Mr Jabez James, a mechanical engineer who specialised in model-making, which he wished to present to the Tsar as an illustration of his bridge-building abilities. While in England, he took the opportunity to inspect the progress on the manufacture of the suspension chain links.

When Vignoles returned to Kiev in October, he found that considerable progress had been made in his absence – the temporary bridge was in use, the construction yard was prepared on the heights and the construction workers’ camp on the other bank. He remained there during the next 6 weeks and his journal is full of examples of his leadership.
On 18 October, after inspecting the completed deep water cofferdams, he noted in his journal that: ‘The deep water cofferdams appeared to be well made and first trials of the engines and pumps showed that with a few alterations and more experience we should be fully in condition to meet all difficulties of water.’ The water level at the time was 96 ft (29 m) above datum, significantly lower than it would be the following spring.

A month later (see Figure 15), there was a difficult moment when the river was partially frozen, which perhaps hinted at further problems for the next season. The river currents increased, the last river barges of the season swept down out of control and carried away a portion of the temporary bridge, much to Vignoles’ fury. A few days later, Vignoles describes in his journal how, by dint of laying straw on the river surface, the remaining gaps between the ice floes were sealed and the works were closed down for the winter. Before leaving for London, he instructed the team to design and instal ice-breakers on the upstream side of each cofferdam and temporary bridge support in anticipation of the end of winter thaw.

4.2 Forward planning
Back in England in December, Vignoles found that the procurement of the chains was progressing, and the construction methods for the piers were being developed by William Coulthard. Meanwhile at Kiev, the engineers were set to work detailing the stones for the piers, and scheduling the timbers for the bridge deck to be put aside for seasoning.

4.3 Interim payments
In January 1849 came the first of many concerns regarding cash flow. Payments from Russia were not arriving at the required rate – he had had to make an advance of £10 000 to Mr Blomberg in the summer – and to keep the flow of supplies and materials from England, Vignoles decided to take out a personal loan of £15 000 from his solicitor. This decision was not taken lightly, but without it the work could not proceed, and he was determined not to let the production stall over something as basic as cash flow.

5. Construction 1849
When he returned to Kiev in the spring, Vignoles hoped that the works could proceed smoothly, but that was not to be the case.

5.1 Setback on the riverworks
The sequence of events is shown in Figure 16. In April 1849, as the river began to thaw, large ice floes collided with the piers of the temporary bridge causing damage to some spans, which had to be replaced. Then, to make matters worse, in May, the melting of the snow upstream caused the river to rise to a high flood level. A number of large timber barges were wrecked on the bridge works, causing yet more damage. The river continued to rise and 2 days later, with the water at 113·5 ft (34 m) above datum, a height of 19 ft (6 m) more than the previous summer, cofferdam D4 was swept away and cofferdams D3 and D5 were severely damaged.

In an extensive report to Colonel du Plat on 11 May Vignoles writes that: ‘...part of dam no 4 shot up vertically like a whale about 25 to 30 ft, and was carried away 2 versts downstream...’ ‘...soundings show a water depth indicating 10 to 14 ft of scour...’ (Vignoles, 1849a).

Vignoles kept matters under control, securing cofferdams D3 and D5 by sinking barges on top of them as ballast. After taking soundings for scour and inspecting piles salvaged from the wreckage of cofferdam D4, he gave instructions to the site staff to redrive the cofferdams with three concentric rows of longer length piles, and to fill between the walls with puddle clay.

Two days later, Vignoles left for England via Warsaw for a typically busy few weeks. He attended the ICE when Thomas Howard presented his paper on rolling bars for suspension bridges (Howard, 1849); he visited the Britannia Bridge tubes at Bangor, which had been installed by Robert Stephenson while he was out of the country; and he visited Coulthard to discuss the swing bridge designs, which were to incorporate wrought iron beams.

While at Bangor, Vignoles discussed the problems of river scour with I.K. Brunel, Robert Stephenson and William Cubitt, among others. He also consulted with Herr Hubbe, the Hamburg Port Authority engineer, who advised the use of fascine mattresses. After viewing an installation in Holland, Vignoles appointed two Dutch engineers to come to Kiev and

Figure 14. Piling in a foundation with temporary bridge beyond – J.C. Bourne c. 1848 (Ironbridge Gorge Museum Trust)
reinforce the river bed against scour around the cofferdams with willow mattresses. In the first part of August he was present in Ireland for Queen Victoria’s visit to Dublin. On 16 August, he was back in London leaving for Warsaw via Hamburg and Berlin, and was finally back in Kiev on 5 September.

By the end of October 1849, the cofferdams were securely remade at last, the river bed reinstated and the fascine mattresses laid (see Figure 17). However, the project was now effectively a year behind schedule.

5.2 Inclined plane experiments
In October 1849, Vignoles carried out a series of experiments to see if the operation of the inclined plane could be automated, using empty wagons as counterweight. The incline consisted of two gradients of 1 in 14 and 1 in 4, separated by a curve in plan. To overcome the friction in the pulleys, it was shown necessary to use five loaded wagons to recover one empty one. Vignoles notes in his journal:

‘I conclude that the number of waggons and the number of men to work the plane from the top of all is too great… we have to… place [the large wheels] on the top of the steep part of the plane [1 in 4 gradient] and work the upper part by letting the loaded waggons down the 1 in 14 with the brake and pull up the empty waggons with horses; paving between the rails.’ (Vignoles, 1849b)

5.3 Erection of scale model in St Petersburg
In December 1849, Vignoles went to St Petersburg to supervise the erection by Mr Jabez James and his colleagues of the 1:96 scale model, which he had commissioned the year before. The 26 ft (8 m) long model, complete in every particular, was installed in the Winter Palace and presented to Tsar Nicholas I by Vignoles on his ‘name-day’. To celebrate the model’s completion, the article describing the works was published in The Times on Wednesday 2nd January 1850, and copied to the Mechanics Magazine 3 days later (The Times, 1850). The model is still in the Museum of Railway Transport in St Petersburg and has thus outlasted the bridge itself.

Vignoles commissioned a duplicate copy of the model, which was exhibited adjacent to one of Stephenson’s Britannia Bridge at the 1851 Great Exhibition (Great Exhibition, 1851). In 1854 it was placed in the permanent exhibition at the relocated Crystal Palace in London (where it was destroyed in a fire in 1866). The text of The Times article of 1850 was used as the basis for the brochure narrative (Vignoles, 1854).

6. Work on site 1850 and 1851
Early in 1850, there was more concern about expenditure. It was decided to abandon the domestic set-up as it was too expensive, and to send his daughter Camilla home. The site establishment would then be run as part of the office, and the finances managed accordingly.
When the thaw of 1850 came, work could once again restart on the cofferdams. Now the construction plant came into its own. There were two static steam engines, one on each bank, and five mobile ones. The cofferdams were pumped down, the leaks plugged, and attempts made to bottom them out with a mixture of clay puddle and concrete. The cement for the concrete was made in the yard, where there were eight large roasting ovens, capable of producing 500 ft$^3$ (14 m$^3$) of cement each day (Vignoles, 1854).

During the summers of 1850 and 1851 the work dragged on and the project became further behind schedule. There were big problems with water leaks in the bases of the cofferdams, and much working time was lost because of saints days and holidays. As a consequence, there was little progress during the summer of 1850 with the bases of D3, D4 and D5, and at this time Vignoles took personal charge of work on site in an effort to speed progress. As a result of his efforts, the bases of the shallow water foundations, in dams D6 and D7, were concreted before the winter of 1850 closed in, and work on the bases to piers P4 and P5 could at last begin (see Figure 18).

In October 1850, potential disaster struck again when the main contractor, Mr Blomberg, died. Fortunately for
Vignoles, the agent on site, Mr Schweitzer, agreed to carry on the work.

In the Spring of 1851, work recommenced on the deep water foundations, and by the end of the season much progress had somewhat belatedly been made (see Figure 19). The deep water cofferdams were finally sealed by mid-September, the foundations completed, and the piers (P1, P2 and P3) were built to above flood level. Piers P4 and P5 had by then been completed up to arch level, which enabled Vignoles to indicate the position of the suspension chains, when the Tsar visited the site, by arranging to hang ropes between P4, P5 and the abutment.

Work could then continue through the winter on completing the portals to piers 1, 2 and 3.

Figure 20 is a drawing ‘in colour by candlelight’ by Bourne showing the construction of a pier in October 1851. Points of interest are the centering of the arch, the saddles for the chains and the temporary works platforms. At the right of the picture can be seen the temporary bridge.

7. Chain lifting and deck construction 1852 to 1853

7.1 Chain lifting

Commencing in June 1852, the suspension chains were now erected on a span-by-span basis (see Figure 21). This involved positioning trestles at points along the span and using block and tackle to winch the chains up to level. A fair amount of supervision was needed for this, and there are frustrated remarks in Vignoles’ journal about the time it took and the skills required. The deep water chains would be lifted during the winter while the river was frozen, using trestles placed on the ice.

In August 1852, the photographer Roger Fenton travelled to site with Vignoles, bringing his stereoscopic camera equipment with him (Hannavy, 2004).

Three photographs by Fenton record the chain erection process. Figure 22, which is taken from a larger photograph, shows chain erection starting on pier number 1. The inclined plane can be clearly seen in the background.
Figure 23 shows chain erection on pier numbers 4 and 5. Figure 24 shows details of the chain erection process. The last two are composite images prepared from photographs in the collection of Dom Pedro II of Brazil at the National Library in Rio de Janeiro.

7.2 Deck construction 1853
Once the chains were in place, the deck construction could follow unimpeded by the river (see Figure 25) and the project could at last be brought to a successful conclusion.

There was again concern about cash flow. To cover outstanding payments, Vignoles used a life insurance as security to raise a further £17 000. He brought a significant amount in gold coin with him personally to pay the workforce, when he travelled to site in August 1853, this being the simplest way of providing it.

7.3 Acceptance trials
At the end of September 1853, as part of the acceptance trials, the bridge was test-loaded (see Figure 26). The bridge spans...
were loaded with 60 000 ft$^3$ (1698 m$^3$) of sand, which was brought in by the barrow-load (see Figures 27 and 28). Before the spans were fully unloaded a heavy rainstorm ensued, which soaked the dry sand, increasing the effect of the load considerably. Although being alarmed at the possible effects of this, Vignoles was relieved to note that the bridge stood the strain successfully ‘with a load equal to 40,000 men’ – revised to 50 000 men in Table 2 – and that ‘there was no permanent deflection in the bridge platform nor any stretch in the chains’.

8. Completed bridge

8.1 Opening

Finally, on 10 October 1853 the bridge was opened by Grand Duke Nicolas, the third son of Tsar Nicholas I. There was a full ceremony of blessing and crowds thronged the bridge (Figure 29).

A week later, Vignoles disbanded his establishment and left Kiev never to return. A skeleton staff remained to deal with outstanding items, carry out the maintenance, and draw up the final account, finally returning to England in 1855.

8.2 Accounts

Vignoles published an impressive set of statistics to accompany the scale model in the 1854 Great Exhibition at the Crystal Palace (see Table 2). This gives the final cost of the bridge project as approximately £432 000 (Vignoles, 1854).

Unfortunately for Vignoles, given the huge quantity of personal funds he had invested in the project, the account was not settled until after the end of the Crimean War. He spent some months in St Petersburg pursuing outstanding payments both for his unfulfilled designs and for the bridge itself. In January 1858, he received a final payment in part settlement of his claims. In February 1858, there is a note in the journal regarding settling the accounts of Mr Palmer, his solicitor, Mr Coulthard, his design assistant, and Mr Bourne, his photographer. In March 1858, he notes that he has: ‘sent to Douglas Evans at his request a discharge in full… signed by
me, releasing him from further [liability] for all sums on Kieff a/c which had passed through his hands since 1848’.

8.3 Legacy
Notwithstanding the difficulties in its construction, the Tsar Nicholas 1 Bridge stood for many years (see Figure 30). However, in 1920 the bridge deck was destroyed by the retreating Polish army, and the crossing reverted to a bridge of boats. In pictures of the destroyed bridge willow tree growth from the fascine mattresses can be clearly distinguished.

A new deck was constructed at a higher level using the original piers, which were still valid. This bridge was destroyed in 1941. The foundations can just be made out at low water alongside the modern metro crossing.

9. Conclusion
Back in London, free from the burden of such an ambitious project, Vignoles concentrated on overseas railway work – on the Rhine; by Lake Geneva; in Bahia in Brazil, where he was represented by his son Hutton; in Spain; and in Poland.
Figure 24. Details of chain erection – from a photograph by Roger Fenton (att), 1852 (Acervo da Fundação Biblioteca Nacional, Brasil)

Figure 25. View of deck under construction – J.C. Bourne, 1853 (collection of National Museum of the History of the Ukraine, Kiev)
Test A – 60,000 cu ft wet sand equals 3000 tons

Test B – 400 tons on span P4 - P5 remainder unloaded. No significant deflection

August 1853 – Deck completed

Test A

<table>
<thead>
<tr>
<th></th>
<th>300 tons</th>
<th>600 tons</th>
<th>600 tons</th>
<th>600 tons</th>
<th>600 tons</th>
<th>300 tons</th>
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<tbody>
<tr>
<td>Test B</td>
<td>0 tons</td>
<td>0 tons</td>
<td>0 tons</td>
<td>0 tons</td>
<td>400 tons</td>
<td>0 tons</td>
</tr>
</tbody>
</table>

September 1853 – Test loading

10 October 1853 – Opening ceremony

Figure 26. Completion testing 1853 (1 cu ft = 1 cubic foot = 0.0283 m³; 1 ton = 1.01604 metric tonnes)

Figure 27. Test loading – sand barrowed in over the swing bridge – J.C. Bourne, 1853 (collection of National Museum of the History of the Ukraine, Kiev)
Figure 28. Test loading – sand being spread over the main bridge deck – J.C. Bourne, 1853 (Acervo da Fundação Biblioteca Nacional, Brasil)

Figure 29. Completed bridge – J.C. Bourne, 1853 (private collection)
He continued his interest in photography, and when in London he found time to attend ICE discussions on many subjects; also his name lives on in the flat-bottomed rail that bears his name. But perhaps surprisingly, given the scale of the project, although details of the Kiev Bridge construction were alluded to in various papers to the ICE and also to the British Association for the Advancement of Science (Vignoles, 1857), the promised technical paper, which at one time was planned for 1855, never materialised. As Vignoles commented during a discussion (ICE, 1867) ‘procrastination is the thief of time’ – the only published piece of writing being The Times article, the exhibition catalogues and the 1854 brochure. Part of the purpose of this paper is to set the record straight.

The story of the building of the Kiev Bridge brings out the character of Charles Blacker Vignoles in many ways. His determination to take on such an ambitious venture, his perseverance to see it through, his willingness to take risks with his own money, his ability to stimulate and enthuse his team of assistants, his pride in his workmanship, his wide knowledge and interest in every aspect of engineering, his showmanship and feeling for publicity – these qualities are all typical of the man and indeed of the age. Similar examples are to be found time and again in his journals, demonstrating that he was indeed a worthy member of the engineering fraternity. This was recognised when he was elected President of the ICE from 1869 to 1871.

His life’s work complete, Vignoles died in 1875 at the age of 82 years. The Kiev Bridge survived him for 45 years.

Acknowledgements
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MANUSCRIPT SOURCES
In preparing the paper, the author made extensive use of Vignoles’ unpublished journals and diaries, which are in The British Library. These are listed in the Additional Manuscripts collection catalogue as

- Vignoles Diaries (four volumes comprising Add. ms 58203 to 58206) Vol. III 1846; Vol. IV July–November 1851.

Vignoles’ unpublished letters are held in the Portsmouth Records Office as Letters of Charles Blacker Vignoles (ref. 1072A).
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The safety of masonry buttresses

S. Huerta

The vault is the main element in most historical buildings. Masonry vaults exert an inclined thrust that must be resisted by a substantial mass of masonry: the buttress. The buttress system assures the safety of the whole construction. Most traditional structural design rules addressed the problem of buttress design. Today, an architect or engineer assessing the structural safety of a historical construction needs to estimate the safety of the buttress system with accuracy. This is not an easy matter. Among other possible failures, a buttress may fracture under certain conditions with a substantial loss of stability, it may show a certain leaning or it may be separated from the wall. Furthermore, buttress systems are complex structures – a combination of walls and counterforts, flying buttresses, etc. – made of different types of masonry, and their assessment cannot be handled in an abstract way. This paper outlines the development of buttress design since around 1700 to explain the main approaches used and to provide a historical context. The paper then goes on to summarise the state-of-the-art in modern masonry buttress analysis and to discuss estimations of safety.

1. INTRODUCTION

The vault was the main element of monumental architecture for around two millennia until around 1900. Masonry vaults and arches exert, inexorably, an inclined thrust that must be resisted by a substantial mass of masonry or buttress. The buttress system thus assures the safety of the construction; a vault may collapse without serious consequences for the whole building. However, because failure of the buttress system always leads to catastrophic collapse, the safety of vaulted masonry buildings lies in the buttresses. An understanding of this problem may involve an architectural or construction historian trying to understand the structural ‘logic’ of some buttress forms, and an assessment of the structural safety of a historical construction requires an accurate estimation of the safety of the buttress system.

However, this is a neglected topic. Antiquarians and then medieval archaeologists and architectural historians focused their attention on vaults. For historical engineers and architects, the problem was to evaluate the vault thrust and the theory of masonry arches and domes developed during the eighteenth and nineteenth centuries tackled almost exclusively this problem.

The vault thrust, used to check the stability of a buttress, was considered an exercise of simple statics. However, modern limit analysis allows a more comprehensive analysis of the theory of masonry structures and sheds new light on the study of the safety of vault–buttress systems.

The aim of this work is to draw attention to the buttress, its design and safety, the logic (or lack of logic) of some forms and the possible approaches to its understanding. The paper is addressed to both historians and practitioners, and to anyone interested in reaching a deeper understanding of masonry architecture. The approach is historical and begins by offering an outline of the development of buttress design in order to single out the main issues regarding buttresses and the way they have been resolved in different epochs. Modern architectural historians and architects or engineers working in restoration need an understanding of these problems: first, to complete the historical overview and, second, to gain knowledge about a monument without which any intervention would be deemed to be arbitrary.

2. TRADITIONAL BUTTRESS DESIGN

Old master builders were well aware of the importance of the buttress system. Before the science of statics was sufficiently developed (say, at the end of the seventeenth century) the only possible approach was the use of empirical structural rules. The approach was not entirely unscientific as each building that stood safely for many years was a successful experiment. The rules codified the sizes of the main structural elements, the depths of buttresses, the thicknesses of arches or ribs, the thickness of walls, etc. Most of the rules that have survived refer to buttress design, and this indicates the importance assigned by the master builders to the crucial problem of deciding the form and size of the buttress for a certain vault or vault system.

The rules were specific to each structural type: rules for designing the buttresses of light Gothic vaults could not be applied to the heavy Renaissance or Baroque barrel vaults of later centuries. This matter has been studied in detail elsewhere (Huerta, 2004). To understand the nature of the design rules, two rules, one Gothic and other stemming from the Renaissance, are now considered.

2.1. A Gothic design rule

Gothic design rules were of two types: geometrical and arithmetical. In both cases, the objective was to decide the depth...
of the buttress as a fraction of the span. In late Gothic German manuals of the early fifteenth century, simple fractions are used to decide the main elements (walls, buttresses and rib vaults) and geometrical procedures are then used to define the forms (imposts, mouldings, etc.). In Germany, France and Spain there is indirect documentary evidence of the use of several geometrical rules. These rules survived in the late Renaissance and Baroque stonecutting manuals that followed the tradition of the medieval stonemasons.

The geometrical rule most cited is represented in Figure 1(a). It appeared first in the lost manual of Baccojani, Germany, c. 1550 (Müller, 1990), in the manuscript of Martínez de Aranda, Spain (c. 1590) and was published for the first time in France in the treatise by Derand (1643) (it is usually incorrectly attributed to Blondel who also published it in 1675). It was then published in many stonecutting and architectural manuals until the twentieth century (see, for example, Cassinello 1964). The rule addresses the problem of designing the buttress of a Gothic cross-vault. The profile (elevation) of the transverse arch is used to generate the form and size of the vault (spatial). However, it is remarkable that the height of the buttress is not considered. After the seventeenth century, this rule was misinterpreted as a rule to obtain the dimensions of the buttress for an arch or barrel vault whose intrados was the arc of a circle, but Derand is explicit about its Gothic origin and, besides, the proportions of the
buttresses obtained by the rule are Gothic. Of course, the rule was only a rough guide to the designer to be interpreted only by a master, not to be applied blindly.

2.2. A Renaissance rule

Renaissance vaults were usually barrel vaults (sometimes with lunettes). The outward thrust of a barrel vault is much greater than a Gothic vault (typically, its weight might be twice that of a Gothic vault of the same plan). Gothic rules were useless and new design rules were developed, based mainly on observations of Roman ruins and also, perhaps, on inspection of Romanesque churches. As the profile of the vault was always semicircular it was not necessary to consider the form of the vault; a simple fraction of the span was used. The rule stated that the buttress should have a depth between one-third and one-half of the span, as is cited in many architectural manuals (Figure 2). Again, the designer would decide in each case what the precise dimensions should be.

The Spanish architect Fray Lorenzo de San Nicolás presented in his treatise of 1639 a detailed account of the application of the rule (San Nicolás, 1639). He considered three types of vault made of stone, brick with radial joints (half brick thickness, \( \approx 150 \text{ mm} \)) and timbrel vaults made by setting two shells of flat bricks, breaking the joints (typical thickness 100 mm). He also considered two types of buttresses – a continuous wall and a wall reinforced with counterforts. His exposition is so systematic that it can be summarised in tabular form (Table 1).

Table 1 shows that, in common with Italian Renaissance design rules, the buttress (wall plus counterfort) should have a depth of at least one-third of the span. The Gothic rule gave a depth/span ratio of 1/4 (for a semicircular transverse arch) or less. This discrepancy is enormous, as it should be considering the difference between both structural types. Of course, the transition between the two types led to some structural disasters. The Renaissance mason, educated in the medieval tradition, would have considered the stereotomy of the ‘modern’ Renaissance vaults trivial, but would not have known how to determine the precise size of the buttress. In Spain, where Gothic architecture continued to dominate until the eighteenth century, there is documentary evidence of this problem. The architect García Berruguilla (1747) made a comparison of the two rules and remarked that many ruins and disasters stemmed from this discrepancy (Figure 3).

Evidence of the same type of problem came to light in a small Spanish church. Construction of the church began with a Gothic presbytery in around 1650 and was finished in 1699 with a nave covered by a barrel vault (Figure 4(c)). The ignorant master builder used the same buttress to the ‘modern’ nave (a) with the result that can be seen in (b). The nave had to be assured by a scaffold and additional larger buttresses had to be added to the already greatly distorted vault in around 1700 (Huerta and López-Manzanares, 1997).

3. SCIENTIFIC BUTTRESS DESIGN

At the end of the seventeenth century, the science of statics was sufficiently well developed to attempt scientific design of vaults and buttresses. The matter of vault analysis and design has been the subject of numerous publications (an excellent outline is provided by Heyman (1972)). This paper concentrates on the hitherto neglected matter of buttress design and makes reference to vault theories only when necessary.

<table>
<thead>
<tr>
<th>Type of buttress</th>
<th>Wall (uniform section)</th>
<th>Wall thickness</th>
<th>Wall plus counterfort</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone vault</td>
<td>1/3</td>
<td>1/6</td>
<td>( \geq 1/3 )</td>
</tr>
<tr>
<td>Brick vault, radial joints</td>
<td>1/4</td>
<td>1/7</td>
<td>1/3</td>
</tr>
<tr>
<td>Brick, timbrel vault</td>
<td>1/5</td>
<td>1/8</td>
<td>1/4</td>
</tr>
</tbody>
</table>

Table 1. Depth of buttress as a fraction of the span. Arithmetical rules of Fray Lorenzo de San Nicolás (1639) for buttress design for barrel vaults with lunettes, constructed using three different materials (Huerta, 2004)
3.1. The French school
Philippe de La Hire was the first to attempt buttress design using statical calculations (La Hire, 1712) (Figure 5(a)). To do this he needed to estimate the vault thrust, but his objective was to obtain the depth of the buttress. La Hire assumed that the vault breaks at a certain point (le joint de rupture) where the thrust of the upper part of the vault acts at an inclined angle, approximately tangential to the curve of intrados (Figure 5).

La Hire did not fix the position of the joint of rupture nor, explicitly, the direction of the force. This made the procedure inadequate for practical use, implying some trials to find the worst position. It was Bélidor (1729) who transformed La Hire’s idea into an engineering design procedure. Bélidor located the joint of rupture on the intrados equidistant from the impost and the crown; the thrust acts through the centre of the joint and is normal to the plane of joint. In this way, calculation of the arch thrust can be made using a parallelogram of forces (Figure 6(a)).

Bélidor was aware of the usefulness of his method and applied it to many practical situations, even in complex buildings (Figure 6(b)). Of course, the buttress design for a given vault involved the solution of a second-order equation, and Bélidor gave the mathematical solution for many cases. As always this occurs when centres of gravity are involved, the algebraic expressions were somewhat frightening, but to any French engineer this was not a problem (this was not, however, the case for normal architects and master builders who continued applying empirical rules).

La Hire and Bélidor considered the buttress as solid, as a monolith. This may seem surprising since they knew that buttresses were built using discrete stones, but the study of general equilibrium was basically correct. They considered equilibrium at the boundary of the buttress, with the overturning moment of the vault thrust balanced by the moment due to the weight of the buttress. The buttress so obtained was, then, in perfectly balanced equilibrium with the vault and was, therefore, critical and unsafe. Bélidor ensured that the results of his design calculations would be safe by recommending that the buttresses be built a few inches deeper. The fact is that the results based on equilibrium calculations using statics agreed well with the expected results derived using traditional design rules and the observation of existing constructions. We now know that this is because the vault thrust calculated was not the actual thrust in the collapse situation, but much more unfavourable (mainly because of the inclination): the ‘wrong’ thrust incorporated a margin of safety. Bélidor did not know this, but he knew that the method gave good practical results and he did not enquire further into the problem of safety. Bélidor also studied the case of compound buttresses: a wall
reinforced with counterforts. He continued to treat the wall–
counterfort system as a monolith, taking moments with respect to the external border of the counterfort. This was too optimistic, but again Bélidor considered that his calculations gave reasonable and reliable results.

It was French engineer Audoy who was the first to design buttresses using the correct vault thrust (Audoy, 1820). He rediscovered Coulomb’s theory of 1773, which had remained in oblivion for 50 years, and applied it to the calculation of vault thrusts. In the second part of his *Mémoire*, he addressed

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Figure 4. Church of Guimarei, Spain: (a) the hypothetical original design with Gothic buttresses; (b) current state with the new buttresses (drawing to scale representing the actual deformations); (c) plan showing the phases of construction (Huerta and López-Manzanares, 1997)

Figure 5. La Hire’s procedure for designing the buttress for a simple vault: (a) original drawing; (b) Heyman’s interpretation (Heyman, 1998a, b)
the matter of buttress design. The depths obtained were this time clearly critical (Figure 7) and Audoy was compelled to study the problem of safety. He considered that one way to attack the problem was to multiply the vault thrust (the horizontal thrust at the crown) by a factor. But, how could this factor be determined? He knew that the factor of safety should take into account many different aspects that would be almost impossible to express in mathematical terms. He then decided to be pragmatic: the incorrect theory of La Hire/Bélidor had been in use for a century, giving good practical results. The factor should be chosen such that the buttress depth would be the same as that obtained using La Hire’s method. He used an incorrect theory to calibrate the results of a correct theory – a true engineering approach. In this way he arrived at a numerical value for the factor of safety of 1.9. The procedure was to multiply the vault thrust by this factor and to use the higher thrust to determine the size of the buttress. This buttress would then be safe for the working value of the imposed loads.

This was the French approach to buttress design during the whole of the nineteenth century: the overturning moments, multiplied by a factor, the coefficient de stabilité, were made equal to the stabilising moment produced by the weight of the whole buttress. Again, the consideration of the buttress as a monolith is implicit.

Figure 6. Bélidor’s examples of buttress design calculations: (a) the typical case; (b) a complex building (Bélidor, 1729)

Figure 7. Critical buttresses calculated by Audoy (1820)
Audoy also considered the compound buttress, and was critical of Belidor’s monolithic assumption. It was unrealistic to assume that the whole weight could be mobilised around the border of the counterfort. He proposed considering the counterfort plus the adjacent wall rotating about the border of the counterfort, and the wall between them rotating about the border of the wall. This would lead to a much thicker buttress system. In general, this part of Audoy’s Mémoire was ignored by subsequent French engineers who persisted – all through the nineteenth century – in considering the compound as a monolith.

3.2. The English school

The French theory considered the arch in a collapse state – imaginary in the eyes of La Hire and Belidor, real after Coulomb and Audoy – and then multiplied the calculated horizontal thrust by a factor of safety to design the buttress. What happened inside the masonry of both the vault and the buttresses (the internal forces) was not considered. English analysis of arches and vaults began with Robert Hooke who, in 1675, made the crucial statement ‘as hangs the flexible line, so but inverted will stand the rigid arch’. This was Hooke’s solution to the problem of finding the ‘true … form of all manner of arches for building, with the true buttmen necessary to each of them’, which he included as an anagram, among others, in a postscript to his Description of Helioscopes (Hooke, 1676). There was no explanation, but there is indirect evidence that Hooke considered that the inverted arch could be prolonged inside the abutment, as is shown in one of the preliminary designs made in collaboration with Christopher Wren for the dome of St. Paul’s Cathedral (Heyman, 2003; Huerta, 2006).

Hooke’s assertion was completed by Gregory a few years later in 1697 ‘… none but the catenaria is the figure of a true legitimate arch … and when an arch of any other figure is supported, it is because in its thickness some catenaria is included’ (Heyman, 1998a). This is a very powerful statement. However, it was ignored and English engineers and mathematicians dedicated all their efforts to calculating mathematically the form of the ‘curve of equilibrium’, which had to coincide with the intrados (or centreline) of the arch. Of course, the pull of the chain would be the thrust of the arch and, in theory, by the end of the eighteenth century English engineers were in a good position to estimate the correct size of buttresses. In fact, nothing of the sort happened. It is fascinating to see how, for example, Hutton, after having struggled with the complicated mathematics of different curves of equilibrium, was unable to give a reasonable estimate of the buttress size for the simplest case of a bridge and had to revert to a modified version of Belidor’s approach (Hutton, 1812).

It was not until Thomas Young (1817) freed the ‘curve of equilibrium’ from the straitjacket of having to follow the shape of the intrados that an advance was possible. Young defined the concept of ‘line of thrust’ as that which ‘… represents, for every part of a system of bodies supporting each other, the general direction of their mutual pressure’. Any deviation from the intrados (or centreline) of the arch was made possible by the effect of friction, as Young explicitly stated. This crucial contribution by Young was ignored by his contemporaries and remained so until recently (Huerta, 2005).

Henry Moseley (1835) is usually credited with inventing the concept of line of thrust and, indeed, he also presented a complete mathematical theory of arches and buttresses. He first considered the problem of the buttress, in his article of 1838 in a highly abstract way. However, in Mechanical Principles of Engineering and Architecture (Moseley, 1843) he paid great attention to the problems of buttress design with a view to solving practical design problems and studied in detail the transmission of forces inside the masonry itself (Figure 8(a)). In doing so, he improved on the French approach, which only guaranteed the stability of the buttress with respect to its base and ignored the possibility of failure at any other joint in the arch (which can, indeed, be the case in a buttress of varying section).
Moseley studied the form of the line of thrust through a rectangular buttress (Figure 8(b)) and realised that a buttress of infinite height may, nevertheless, have a finite thickness. In fact, this had been discovered by the French engineer Danyzy in 1732, who realised that this property justified the use of safe geometrical design methods that did not consider buttress height (Huerta, 2004). This discovery is, however, usually attributed to Moseley.

Moseley also studied the safety of arches and his approach was completely different from the French approach. Being well aware that the safety of every joint depended mainly on the position of the line of thrust (centre of resistance) he defined his 'modulus of stability' as ‘the nearest distance which the line of resistance [line of thrust] approaches the extrados’ (Moseley, 1843). Moseley is here considering safety as a matter of geometry. It is true that, for usual arch forms, the danger is that of overturning, since the possibility of a sliding failure is concentrated only in a very few cases, for example when the point of application is near the top of the buttress, a particular case that Moseley studied.

Moseley made every effort to explain the usefulness of his theory and provided many examples. However, the complicated formulae at which he arrived (sometimes occupying several lines) must surely have discouraged many engineers and architects. Like many of his contemporaries, Moseley was interested in the rationality of Gothic forms. He studied the high strength of pointed arches (Figure 9(a)) and also investigated the form of a Gothic stepped buttress with (nearly) constant stability throughout its height (Figure 9(b)).

Finally, Moseley explained how to deal with compound buttresses such as the one shown in Figure 9(c). He reduced the problem to that of a continuous wall extending the material between the counterforts and obtained a continuous wall with two different specific weights. He then applied the same considerations as for a simple buttress. This was equivalent to considering the wall-plus-counterforts system as a monolith, since it is the line defined by the external boundary of the counterforts that is considered as the axis of overturning.

W. J. M. Rankine exploited all the consequences of Moseley’s contribution and proposed a complete theory of masonry buttresses (Rankine, 1858). Rankine stated the two conditions of stability of a plane joint as follows.

The obliquity of the pressure must not exceed the angle of repose.

The ratio which the deviation of the centre of pressure from the centre of figure of the joint bears to the length of the diameter of the joint traversing those two centres, must not exceed a certain fraction, whose value varies, according to circumstances, from one-eight to three-eighths.

He named the first ‘stability of friction’ and the second ‘stability of position’. Rankine then adopted the same geometrical approach of safety as Moseley, but defined the position of the line of thrust relative to the dimensions of the joint, fixing the maximum deviation from the centre of the joint as \( q_t \), where \( t \) is the length of the joint. The parameter \( q \) serves as the means of defining the safety required. Rankine stated that this ratio should be obtained through a consideration of strength of materials, but then added


nevertheless, an approximation to that position can be deduced from an examination of the examples which occur in practice, without having recourse to an investigation founded on the theory of the strength of materials.

He then discussed possible values of \( q \) and established, as a lower limit, the value usually adopted for retaining walls: for British engineers \( q = 3/8 = 0.375 \) and for French engineers \( q = 3/10 = 0.3 \). Rankine went on to remark


In the abutments of arches, in piers and detached buttresses, and in towers and chimneys exposed to the pressure of the wind, it has been
found by experience to be advisable so to limit the deviation of the centre of pressure from the centre of figure, that the maximum intensity of the pressure, supposing it to be an uniformly varying pressure, shall not exceed the double of the mean intensity.

This means that the point of application of the thrust must lie within the core of the cross-section, so that all the stresses must be compressive, and Rankine gave a table relating the shape of the section to the parameter \( q \), which defines the limiting size of the core that ensures safety, as explained above. The minimum value is for a circular solid section where \( q = 1/8 \). It should be noted that here Rankine mixes the notion of a geometrical factor of safety with a limiting value of strength. In fact, the problem is not the occurrence of certain tensile stresses or even of the value of compressive stresses, but the 'stability of position'. There may be a stability condition that is dangerous despite very low compressive stresses: the compressive stress resultant may be near the border with low stresses. The middle-third rule for rectangular sections is popular because it appears to be a strength condition, compatible with the 'elastic' approach; in fact, the consideration of no tensile stress leads to a strict geometrical condition. It is this geometrical condition that assures safety, whether the engineer or architect is aware of this or not.

Rankine was also interested in the intrinsic geometrical properties of buttresses related to stability. For this, he invented the moment of stability (Figure 10(a)) as a geometrical property of the buttress, independent of the force or forces it resists, which he defined thus.

The moment of stability of a body or structure supported at a given plane joint is the moment of the couple of forces which must be applied in a given vertical plane to that body or structure in addition to its own weight, in order to transfer the centre of resistance of the joint to the limiting position consistent with stability.

The mathematical expression for a horizontal joint is

\[
M_s = W (q \pm q') t, \quad q' t \text{ being the distance from the vertical line passing through the centre of gravity of the masonry above the joint to the centre of the joint C. The value of the property denoted by } \pm \text{ depends on the position of this vertical line, to the right (+) or the left (−) of C (Figure 10(b)). Rankine then explained that the moment of stability is in fact (see Figure 10(b))}
\]

the moment of the couple, which, being combined with a single force equal to the weight of the structure, transfers the line of action of that force parallel to itself through a distance equal to the given horizontal distance of the centre of resistance from the centre of gravity of the structure.

While this is all simple statics, Rankine's approach permits the engineer to consider the relative benefits of certain forms of buttress from the point of view of stability. Figure 11 shows several buttress profiles. If the moment of stability of the rectangular buttress (Figure 11(a)) is taken as 1, then the other buttresses (with the same volume) have moments of stability of (b) 1.71, (c) 1.63 and (d) 2.18 (profile (d) corresponds to Veðthéuil Church (e)). The efficiency of the Gothic stepped buttress is thus demonstrated but, of course, it complicates the process of building and the need of maintenance to prevent the ingress of rainwater.

Rankine went on to state that a structure formed from a series of masonry blocks would be safe if, at every joint, the two conditions of stability of friction and position were satisfied. This can be done in the most convenient way by drawing the line of thrust (Figure 12(a)). He then went into the detail of obtaining this curve, both geometrically (predating by 8 years the graphical design methods published by Culmann) and analytically, for the buttresses of buildings (i.e. slender buttresses subjected to the action of concentrated loads (Figure 12(b)). He first developed the general equation for the line of thrust and then studied several practical cases. The exposition is rigorous and worth studying even today.
As for the compound buttress, Rankine, like Audoy, considered (implicitly) the buttress ‘divided’ into two parts: the ‘counterforted wall’ having the full thickness of the buttress and the wall between the counterforts. He computed the moment of stability as the sum of the moments of stability of both parts (Figure 12(c)). He then calculated the thickness of the equivalent uniform wall (i.e. a continuous wall having the same stability) and compared the volume of masonry in both cases. He concluded that ‘there is a saving of masonry (though in general but a small one) by the use of counterforts’. In fact, Rankine’s assumption is equivalent to considering that there is no connection at all between the two parts of the buttressed wall and so gives an unnecessary margin of safety. Rankine also treated a number of other forms of masonry construction, including the buttressing of a groined vault (Figure 12(d)).
3.3. Design and analysis of masonry buttresses and buildings c. 1900

In the last quarter of the nineteenth century, both approaches – the French approach using the coefficient de stabilité, which served mainly to check the stability at the base of the buttress, and the English geometrical approach – coexisted. The main problem was to obtain the vault thrust; verification of the safety of the buttress was made afterwards.

The analysis and design of complete buildings proved to be a much more complicated affair than, for example, the design of masonry bridges. Masonry bridges usually have a barrel vault and the abutment walls are rectangular buttresses (solid, or hollow with spandrel walls). In the case of a building, say a neo-Gothic church, the system of vaults, walls perforated by great windows and triforia, flying buttresses and external buttressing formed a complex structure that had to be analysed if any numerical results were to be obtained. This was rarely done apart from very particular cases. In general, the traditional geometrical rules continued to be used to establish overall dimensions and the stability was then checked using one of the above mentioned methods.

Karl Mohrmann, an architect of the Hannover school, set himself the task of explaining the behaviour of complex Gothic buildings using the conventional approach to the theory of structures. His additions to the third edition of Lehrbuch der gotischen Konstruktionen (Manual of Gothic Construction) (Ungewitter and Mohrmann, 1890) contain a detailed statical analysis of all the elements of a Gothic structure. His approach was to look for a reasonable state of statical equilibrium among the infinitely many possible in such highly hyper-static structures. With regard to buttresses, Mohrmann added a complete chapter on the form and depth of buttresses (Form und Stärke der Widerlager). This chapter runs to 50 pages and includes many figures, tables and plates; it constitutes by far the most complete study on buttresses ever written. He also added substantial notes and drawings on buttresses in a chapter on churches in section and elevation (Die Kirche im Querschnitt und Aufriß). He studied churches with one nave, hall churches and churches with three naves of different heights, and in each case made calculations of the stability of the buttress system.

Mohrmann was fundamentally concerned with the problems of statical equilibrium (i.e. of the possible paths or lines of thrust of compressive forces that could explain the behaviour of the buttress system as a whole). He began by studying the simple buttress considering the shape of thrust lines and the possibility of failure in sections other than the base. As shown in the left-hand drawing of Figure 13(a), he was concerned with the equilibrium of an intermediate pier; in the right-hand sketch he drew attention to the possibility of failure at three different sections (I, II, III). To ensure safety at each joint, he accepted Rankine’s condition that all the stresses must be compressive (i.e. the thrust must be contained within the core of the cross-section). Figure 13(b) illustrates different families of planes of joints in order to construct the line of thrust.

However, the emphasis through all Mohrmann’s chapters and subsequent notes is always on statical equilibrium. For example, he studied the problem of hall churches with adjacent vaults of different spans and how to equilibrate the thrusts so that the loads may enter more or less vertically into the piers (Figure 14). For analysis of complex buildings, he used the equilibrium approach extensively. He first divided the structure into several fundamental elements or ‘blocks’ (main vaults, aisle vaults, flying buttresses, walls and triforia, buttresses, etc.). He then studied the conditions of equilibrium for every element, ‘assembled’ the blocks in equilibrium and then checked that, at every joint, the thrusts were contained within the masonry. Figure 15 shows a selection of the illustrations of different plates indicating the level of detail of his analysis. However, it appears than Mohrmann was not satisfied with the study of global equilibrium; he also addressed some particular problems.
not previously researched. Figure 16 shows one of these problems: the effect of diminution of section in a triforium and how the loads may pass through the masonry. Among his many other examples are the stability of slender mullions in windows and the statics of Gothic spires. Mohrmann’s additions form a complete manual on the statics of Gothic churches.

Mohrmann is singular for both his extension and depth, but he was not alone in using statical equilibrium with graphical statics to analyse the stability of masonry buildings. In Europe around 1900, it was current practice among architects and engineers to use this approach when analysing many structural problems; the work of Pierre Planat in France deserves particular mention (Huerta, 2008).

However, for the theoreticians deeply steeped in the assumptions of classical elastic theory, this equilibrium approach was considered, at best, a gross approximation of ‘actual’ structural behaviour. For them, answers could only be found by solving the three sets of equations for equilibrium, elastic material and compatibility. Those equations had been long established (for example by Navier in 1826), but they were impossible to solve for masonry structures (Heyman, 1998b). Nevertheless, the conviction that the only way to find the ‘real’ state of the structure was by solving the elastic equations was considered indisputable – an article of faith, in fact.

4. THE TWENTIETH CENTURY: LIMIT STATE ANALYSIS

The equilibrium analysis of Mohrmann hinted at the heart of the problem. The buttress system forms part of a complex hyperstatic structure and therefore its internal forces are part of a complex spatial system of equilibrium. The usual distinction between a vault, which thrusts, and a buttress, which counter-thrusts, is useful for simple systems (as in bridge design or the case of a single nave church) but, in the case of a complex church, overall equilibrium needs to be considered. Heyman (1967–1968) noted how consciously and freely Mohrmann handled the different possible equilibrium solutions. Indeed, Mohrmann proposed a simplified method to obtain a satisfactory value for the thrust of a flying buttress (Figure 15(c)). Given a certain value for the vault thrust (which, in fact, varies between close limits), he assumed (arbitrarily) that the horizontal component of the thrust from the flying buttress \( B \) is horizontal and acts at a certain height. Taking moments about the centre of the pier, it is possible to calculate the value of \( B \) and then to check the passage of the thrust through the masonry of the buttress system (if this is not the case, it is easy to make another trial). Mohrmann made no statements about the elastic properties of the material or about compatibility conditions. This was intolerable for engineering scientists, but any architect or engineer would have felt that this was a valid way – indeed the only way – to handle the infinitely many equilibrium solutions. In fact, the example of Figure 15(c) is deliberately simple – a far cry from the real case of Beauvais Cathedral with its complex system of flying buttresses, intermediate piers and a third, hidden, horizontal ‘flying buttress’ in the form of a wall over the transverse arches of the aisle vault (Figure 17(a)). Benouville made a statical analysis of Beauvais in the concours he made to become architecte diocesain (Benouville, 1891) (Figure 17(b)). His analysis (épure de stabilité) was presented without...
explanation as a well known and usual procedure (some of his results have been discussed by Heyman (1967–1968); the force polygons represent the equilibrium between certain parts or 'blocks' of the structure). It should be stressed that Benouville presented one solution of equilibrium of internal forces within the masonry. He did not claim that this was the 'true' or 'actual' state of the structure and it is evident that it would not be difficult to find other safe equilibrium states.

Figure 15. Selection of drawings from Ungewitter and Mohrmann (1890) showing Mohrmann's method of analysis of complex structures: (a) flying buttresses; (b) transmission of thrust through a nave pier; (c) simplified method of obtaining an adequate value for the thrust of the flying buttress

Figure 16. The stability of a triforium wall subject to inclined forces (Ungewitter and Mohrmann, 1890)
What is, then, the actual state of a masonry structure? How can its safety possibly be ascertained if there are doubts about the true state of internal forces?

The solution to these problems came in 1966 with publication of a milestone paper in the theory of masonry structures (Heyman, 1966). In his paper ‘The stone skeleton’, Heyman (1995) realised that if the material satisfied certain conditions – namely an infinite compressive strength, zero tensile strength and with sliding at the joints impossible – then the whole theory of masonry structures could be rigorously incorporated within modern limit state analysis (LSA) (or plastic design) of structures. These same assumptions had already been made explicitly by many nineteenth century engineers (for example by Moseley in 1843) and they are logical and easy to check. The first, infinite compressive strength, although unsafe in theory (since no real material has infinite strength) is true enough in practice as the actual stresses in masonry buildings are very low. (Benouville had been surprised to find a stress of only 1.3 N/mm² in the nave piers of Beauvais Cathedral, which support the world’s highest Gothic vaults.)

If the material masonry structure obeys the three equilibrium conditions, the yield surface is formed by two straight lines and the condition that must be satisfied at every joint is that the internal force – the thrust – should be contained within the masonry. (The yield surface plots the relationship between the stress resultants \( N \) (normal force) and \( M \) (bending moment) at the limit state of the joint. The limit for the eccentricity of the load is half the depth of the section \( h \), therefore the straight lines \( Nh = \pm M \) define the permissible region for the values \( N \) and \( M \).) When the thrust touches the boundary, a hinge is formed. It is this possibility of forming hinges that is crucial to translating the fundamental theorems of LSA from steel to masonry (an extraordinarily imaginative and lucid explanation is given by Heyman (2008)). From these theorems, the most ‘fundamental’ is the safe theorem: if it is possible to find a set of internal forces in equilibrium with the external loads that satisfies the yield condition, then the structure is safe (will not collapse). The crucial point is that this ‘set of internal forces in equilibrium’ need not be the ‘actual’ state in the structure. In fact, the question as to what is the actual or real state in a hyper-static structure (in any material) is nonsensical. Minute changes in the boundary conditions produce enormous changes in the set of internal forces in equilibrium with the loads. This matter has been discussed in depth by Heyman in many publications (e.g. Heyman, 2005) and need not be repeated here.

So, it turns out that Mohrmann’s static equilibrium approach is entirely valid. The first task of the analyst in studying a complex buttress system is to find a reasonable equilibrium state that could serve to answer the problem in question. General questions of the type ‘Is a cathedral (which has stood for six centuries) safe?’ are unnecessary. There must be some evidence of damage or danger to trigger the expensive process involving expertise and, because there is no universal, standard way of...
approaching the analysis of historical masonry structures, the expert analyst will focus the analysis precisely on the problem or problems observed.

This review of the history of buttress design has permitted us to single out the principal problems regarding their analysis and estimations of structural safety. However, some final comments can be made, within the frame of LSA, which will serve to complete the picture and define the concept of safety and the analysis of compound buttresses more precisely.

4.1. Geometrical coefficient of safety
One of the main conclusions of the application of LSA is that the safety of masonry structures is only a matter of geometry and it turns out that Rankine’s approach of constraining the position of the thrust to a certain region within the section is completely correct. Huerta and López-Manzanares (1997), however, considered it more convenient to use a different way of determining the location of the thrust. This consists of dividing the half-diameter AC by the distance from the point of application of the thrust to the centre of C (\(x\)) (Figure 18(a)). This method obtains a geometrical coefficient of safety, \(c_g\) of \(t/2x\) (\(= t/2g\)), which represents the fraction of the central area of the buttress that may contain the thrust. In this way, a geometrical coefficient of 2 means that the thrust is located at the boundary of the middle central half of the buttress, a coefficient of 3 means that it is at the boundary of the middle third, and so on. Different sections will have different geometric coefficients of safety will be a minimum in the overturning direction of the buttress. In Figure 18(b) this takes place at the base.

This definition of the geometrical coefficient of safety has the advantage of being similar to the geometrical factor of safety defined by Heyman (1969, 1982) for masonry arches. For bridges, Heyman suggests geometrical factors of safety of around 2. The value of 3 used by Rankine when considering the compressive stress in the whole section is more restrictive, but still seems low when compared with factors actually observed in many Gothic buildings. It is not uncommon to find geometrical coefficients of 4 or higher for rectangular buttresses. To discuss actual values of the geometrical coefficient of safety \(c_g\) it is necessary to study:

(a) the actual collapse load of a masonry buttress; and
(b) the influence of the buttress leaning on the safety of the entire vault system.

4.2. Collapse of an isolated buttress
One of the best approaches to estimate the safety of a structure accurately is to calculate its collapse load. In all the previous analyses it has been implicitly considered that the buttress would not collapse. If it were to collapse, it is assumed it would do so at one of the joints between the sections into which the buttress is imagined to be divided (for example, joint C in Figure 13(a)). However, although it is useful to consider the buttress as a system of rigid blocks in order to study the internal forces, the fact is that the buttress is formed from stones or bricks bonded together with weak mortar. If a buttress is caused to overturn at a certain boundary (say at the base) it will probably fracture and a masonry wedge will remain at the base. The weight of this wedge would not have been taken into account for the computing of its moment of stability and, therefore, the calculated collapse load would represent an upper bound. This was well known by nineteenth century engineers and the history of the interest in this phenomenon has been treated elsewhere (Huerta, 2004; Huerta and Foces, 2003).

It was again Heyman who first studied the possibility of fracture in thick walls and towers made of nonmonolithic masonry. In 1992, he showed how to approach the problem and calculated the form of the surface of fracture in the collapse of a leaning tower (Heyman, 1992). Shortly afterwards, in 1993, an unpublished manuscript from around 1800 by the Spanish engineer Joaquín Monasterio was discovered (Huerta and Foces, 2003). Monasterio had also studied the formation of fracture surfaces in buttresses (Figure 19) and considered that in buttresses made of ashlar masonry, the fracture would be
determined by the form and size of the blocks (Monasterio, c. 1800).

In collaboration with John A. Ochsendorf, this problem was revisited in 2001. When conducting research under Heyman at the University of Cambridge, Ochsendorf used a numerical approximation to obtain the surprising result that the fracture of a rectangular buttress was planar. This elegant demonstration of planar fracture was made known to the author in a private communication (Heyman, 2001) and in Ochsendorf’s doctoral thesis (Ochsendorf, 2002). It was published in Spanish and English (Ochsendorf et al., 2003, 2004) and, more recently, Ochsendorf and Lorenzi (2008) have contrasted previous studies of the fracture of buttresses with numerical models of block systems.

The statics of fracture of certain buttresses is extremely simple. Once the point of origin of the fracture has been located, the line of thrust must pass through the boundary of the core (the middle third for a rectangular section). The angle of the plane of fracture can then be obtained by means of Equation 1 (Figure 20(d)), which establishes the equilibrium of the inferior masonry wedge

\[
(W_0 + W_c)(t/3) = Hh
\]

where \(W_0\) and \(H\) are the components of the resultant force above section AC, \(W_c\) is the weight of the masonry wedge that remains on the base (considering 1 m breadth is \(W_c = (1/2)(ht)\); \(y\) being the specific weight of the masonry), \(t\) is the depth of the buttress at AB and \(h\) is the height of the wedge. Algebraic manipulation yields

\[
\tan \alpha = \frac{h}{t} = \frac{W_0}{H - (ht/6)}
\]

Let us consider an actual masonry buttress subject to a concentrated load \(F\) of magnitude inferior to the actual collapse load. The value of \(F\) being known, it is a simple matter to draw the thrust line, dividing the buttress in hypothetical blocks separated by horizontal joints. For the case where there is no fracture, the thrust at the base will pass through point F in Figure 20(b). In general, at section AC, a fracture will begin to form when the stress at the inner face of the buttress falls to zero, or when the line of thrust reaches the boundary of the middle third. At point B there is an abrupt change of curvature of the line of thrust and the curved line of thrust (above section AC) becomes a straight line (below section AC). The wedge CGH will separate from the buttress and the angle of fracture can be calculated by means of Equation 2. (It should be noted that the angle of fracture depends only on the loads above section AC.) As a result, the thrust at the base becomes displaced from point E to F, and the geometrical coefficient of safety experiences a reduction of around 20%. This is a considerable reduction. The collapse load \(F_c\) can be calculated by statical considerations.

Figure 19. The earliest known consideration (Monasterio, c. 1800) of the fracture of buttresses and its effect on the stability of the vault–buttress system within vault theory (Huerta and Foce, 2003)
(Ochsendorf et al., 2004) giving a value around 30% less than the collapse load for the monolithic buttress.

With regard to the safety of actual buttresses, the possibility of fracture establishes a minimum value for the geometrical coefficient of safety: to prevent formation of a fracture, the thrust should fall within the core. This is a geometrical condition. For buttresses of rectangular section, it is the ‘middle third’ (c₉ = 3). For columns or nave piers of circular or nearly circular section, it is the ‘middle fourth’ (c₉ = 4). If the pier has nearly the section of a square rotated by 45° and the overturning moment acts along the diagonal of the square, it is the ‘middle sixth’ (c₉ = 6). (This much more restrictive value for nave piers may help explain the gross deformations often observed in the transverse sections of Romanesque and Gothic churches.)

At first sight it would appear that Rankine’s condition is correct for the design of building buttresses since it avoids the formation of any fracture. However, this is not the case, as will be shown later.

4.3. The effect of leaning

The buttresses of historical architecture are seldom perfectly vertical. A leaning of 0.5–1° is very common; the walls of the church of Guimari (Figure 4(b)) have had an inclination of around 1.5° for the last three centuries. The effect of leaning on the moment of stability of a slender buttress may be important. Consider a prismatic buttress (of rectangular section) of depth t, breadth 1 m and height h = x.t. The moment of stability will be proportional to h.t/6. An inclination of x would make this moment proportional to h.t/6 – x(h/2)]. Therefore, the leaning will reduce the moment of stability by a factor (1 – 3.x²). For x = 0.5°, this means a reduction of 15%; for x = 1°, a reduction of 30%.

The leaning has another effect on buttresses that support masonry vaults. The vault will suffer, as a consequence of a symmetrical leaning of x by both buttresses, an opening at the level of the imposts (vault springings) of the order of 2(hx).

As a second case study, consider a theoretical case in which the buttress is a continuous wall supporting a semicircular barrel vault (Figure 21). The vault has a span s = 7 m and a thickness of s/20; the buttress has been designed with a depth t = 1.6 m. A first analysis, considering the buttress as monolithic, gives a geometrical coefficient of safety of c₉ = 3–6 at the base. There is no danger of fracture and, apparently, the buttress is safe enough. However, this dimension gives a depth/span ratio of 1/4.3, which is much less than the 1/3 recommended by the Renaissance empirical design rule and, also, by Fray Lorenzo (see Table 1). (The dimensions correspond roughly to the chapel of the Paço de Antequeira, Rois, Galicia, built in the eighteenth century; the vault collapsed in the 1980s and expertise to rebuild the vault was required (Huerta et al., 1997).)

Let us explore the consequence of a leaning of 1° [in the Rois chapel the inclinations were between 0.5 and 1.4°]. Since the buttress has a nonuniform profile, let us consider, as above, a...
slenderness of 6. This will reduce the moment of stability by 30%. This ‘new’ distorted arch would then crack and adopt the form shown in Figure 21(b), where the deformations have been exaggerated to show the geometrical parameters. There is a simple relationship between the horizontal displacement $d_h$ due to leaning and the vertical drop $d_v$ of the crown. A simple quadratic equation may be written, assuming that the hinges will not change and that the distance AB remains constant during the movement. In this case $d_h = 160 \text{ mm}$ (i.e. a total opening of 320 mm or $s/22$) and $d_v = 260 \text{ mm}$. The original horizontal thrust $H$ increases by 20% in the distorted vault.

It is now possible to check the stability of the inclined buttress for the new thrust. It turns out that the geometrical coefficient of safety has been reduced to $c_g = 2.3$. This implies, theoretically, that there would be a cracking of the buttress at the base and a further reduction of the geometrical safety to $c_g = 2.1$. The situation is far from favourable as any increase in the vault thrust would give rise to further leaning of the buttress and possible collapse. (It should be noted that the buttress would not overturn at this leaning as its weight would pass within the base during the ‘snap-through’ collapse of the vault; this is the reason why, in ruins, the walls are still standing though the vaults have collapsed.) In fact, this is what happened at Rois (Figure 22). At an indeterminate date in history, the chapel had been given added counterforts (not bonded to the wall) and this apparently stopped the movement. However, abandonment of the building in the 1960s and the entry of water, etc. provoked the collapse that began in the 1980s with the fall of a central part of the vault and culminated in the 1990s with collapse of 70% of the vault. A contemporary master builder from the beginning of the eighteenth century would have objected to the thinness of the walls. However, a modern architect and engineer would have considered the initial design safe because the buttress had been designed using design criteria even more conservative than those recommended by Rankine and subsequent authors, and it had a geometrical coefficient of safety of 3.6.

The conclusion is clear: the problem is not simply the overturning of the buttress (as in retaining walls or dams). The design should be ‘generous’ in order to provide for unavoidable settlements and leanings. This means higher coefficients of safety and a greater depth/span ratio. The reader may check without difficulty how a buttress with a depth of 1/3 of the span (2.3 m instead of 1.6 m, the Renaissance rule) would have withstood the actual leaning and consequent increase of the vault thrust. Indeed, most probably, the inclinations would have been much less and the problem then nonexistent. Only the final empirical experiment – the building itself – gives a correct indication as to the ‘true’ value for the geometrical coefficient of safety. In the author’s experience, values of 4 or 5 (or even much higher) are not infrequent in well-preserved historical buildings.

4.4. Compound buttresses

The matter of the compound buttress – a wall reinforced with counterforts – is much more complex. The actual construction of the structure is of such importance as to make meaningless any calculations that do not take it into consideration. The thick walls usually have outer skins of more or less regular masonry and an inner core of rubble masonry. The method of bonding with the counterforts may vary significantly (Figure 23) and the nature of the rubble masonry may also vary with height – at Guimarei Church (Figure 4) the masonry of the core was very good at the springings of the vaults but very poor (‘earth mortar’ at the foot between the counterforts).

The behaviour of a compound buttress can be explained with reference to the several methods developed to calculate the
moment of stability of a buttress formed by a wall with counterforts. Figure 24 shows three modes of collapse (Huerta and López-Manzanares, 1996):

(a) the monolithic assumption where the whole system rotates around the boundary of the counterfort;
(b) the Audoy/Rankine assumption that considers, maybe too conservatively, that there is no connection between the buttresses (wall plus counterfort) and the wall between them; and
(c) a counterfort simply added to an existing wall.

If we take the moment of stability of case (a) as \( M_a = 1 \), then \( M_b = 0.5 \) and \( M_c = 0.3 \). An intermediate case between (a) and (b), where a part of the wall is bonded to the buttress, is also possible; for example, if the length of wall attached at each side of the buttress is assumed to be half the thickness of the wall, then \( M_d = 0.6 \).

It may also be the case that an upper part of the wall contributes to the stability through existing arches over the windows or the formation of ‘relieving arches’ inside the masonry (see Figure 25). All this, again, depends on the quality and state of the masonry, the movements that the structure has suffered, the nature of possible internal cracks, etc. A better understanding can only be found in the empirical rules and in the critical study of existing buildings. The assumption that part of the wall is well bonded to the buttress, so that it may collaborate in the stability, appears rational. The key question is the length of well-bonded wall that should be assumed.

Fray Lorenzo de San Nicolás gave detailed rules (Table 1) that were used by many Spanish architects in the design and building of parish churches until well into the nineteenth century and these churches now constitute a large body of experimental data and a resource for detailed study. The author studied the case of a stone barrel vault with a thickness 1/20th of the span and made calculations for its geometry as specified by Fray Lorenzo (Figure 26(a) and (b)). (The actual vaults of this time had lunettes, but they were small and did not affect the calculations.) The masonry of the walls, buttress and vaults is assumed to have uniform specific weight. The total length of wall bonded to the buttress is given by \( z_1 \). The calculations were made assuming a geometrical coefficient of safety \( c_g \) of 4. It may be seen from the curves in Figure 26(c) that a ratio of depth of buttress to span of 1/3 or higher corresponds to values of \( z_1 \) between 0-5 and 1, which sounds very reasonable. A better bonding would increase the value of \( c_g \) correspondingly.

5. CONCLUSIONS

In the middle of the nineteenth century Viollet-le-Duc wrote (Viollet-le-Duc, 1854–1868):
Figure 24. Three possible modes for the collapse of a compound buttress (Huerta and López-Manzanares, 1996).

Figure 25. Possible contribution of a wall to the stability of buttresses through relieving arches.
The skeleton yields or resists ... following the needs and the place ... it seems to be alive, because it obeys opposed forces and the stability is only acquired through the equilibrium of those forces.

translated as

Le squelette cède ou résiste ... suivant le besoin et la place ... il semble posséder une vie, car il obéit à des forces contraires et son immobilité n’est obtenue qu’au moyen de l’équilibre de ces forces.

The stability and safety of masonry architecture is a matter of geometry. The traditional geometrical design rules are of the correct form and represent the distillation of centuries of experience which have proved to be, even today, a reliable indication of actual factors of safety. A modern LSA of masonry leads to the same geometrical conclusions. Furthermore, LSA liberates designers from the ‘elastic straightjacket’ and permits them to concentrate on the crucial aspects of statical equilibrium.

The analyst’s task is not to find the actual equilibrium state of a masonry structure – which is an impossible task – but to find, among the infinitely many equilibrium solutions, one that would permit them to answer the question or questions posed by the problem at hand. The study of a buttress should be considered in relation to the vault system it abuts. Again, the analyst should evaluate the situation and apply the corresponding simplifying assumptions.

The first requisite of a masonry buttress is to withstand the thrust of arches or vaults and wind loads. However, the buttress should also be so designed as to avoid significant deformations in the vaults. Due to the height of most masonry buttresses this imposes a severe restriction on the position of the line of thrust at the base, with the result that geometrical coefficients of safety for buttresses are much higher than those for arches (usually around 2).

The usual geometrical coefficient of safety of 3 (the ‘middle third rule’, which avoids any tensile stresses in the buttress) appears to be insufficient to ensure the stability of structures over many centuries, as has been proven with much historical architecture. To arrive at this conclusion it has been necessary to combine historical research into how buildings were designed with the theoretical advances of modern LSA. This may be the most important conclusion: a historic masonry structure cannot be properly understood without a historical background of the building. Such buildings represent an invaluable ‘experiment in progress’ and should be regarded with respect and admiration.

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Figure 26. Study of the stability of a buttress system as presented by Fray Lorenzo (1639): (a), (b) geometrical parameters; (c) stability (depth of buttress to span) for different heights and bonding of the wall–buttress system (Huerta, 2004).
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The growth of Hull as a British port at the time of William Wilberforce (1759–1833) saw the construction of docks and dock bridges similar to those in London, Plymouth and Liverpool. These structures now form an essential part of Britain’s heritage and are rightly protected under listed buildings and conservation regulations. However, they remain at risk, especially if left unused. For several years, Hull City Council has been developing a conservation policy for its nine listed bridges and a comprehensive historical audit was thus required. This paper looks back into the history of Hull Docks and the development of cast-iron swing bridges between 1800 and 1850. The paper describes the design of Hull’s cast-iron Wellington Street Swing Bridge as part of a scheme for restoration of the bridge to working order. When the bridge was originally built, it would have been required to carry horse-drawn traffic that would probably have weighed no more than 5 t in total and carried commodities at walking pace, thus minimising the impact factor. Its new role, to stimulate community living and regeneration, is to offer pedestrian and cycle use with occasional use by vehicles weighing up to 7.5 t. The original bridge design may have been based on a three-pinned arch, although this is just speculation. The recent design by consulting engineer Pell Frischmann considered the bridge’s articulation; this is described together with other parameters that influenced analysis of the cast-iron structure.

1. INTRODUCTION

The fabrication of cast-iron swing bridges would originally have been left to skilled workers. Present-day foundries cannot produce castings of such magnitude in one piece, largely because the technology is redundant as superior materials and modern fabrication methods are available. This made the restoration of Wellington Street Swing Bridge in Hull, UK more problematic. The decisions that had to be made needed to balance various issues. In effect, a decision was required as to whether to return the bridge as close to its original condition as possible or to a version of its later working existence.

In returning Wellington Street Swing Bridge to use, there was inevitably some conflict between appearance, health and safety issues, modern engineering practice and preferences, and what constitutes restoration. These are the types of issues addressed by the so-called Venice charter. The bridge has been much altered since its construction in 1846 and is now a Grade 2 listed structure, meaning it has special architectural and historical interest. Some alterations were necessitated through wear and tear of original components; other changes were required because of altered uses and demands made upon the structure, or because of technological changes that have improved efficiency.

2. HISTORY OF CAST IRON SWING BRIDGES

Cast iron was first used in the construction of swing bridges in London. In notebooks housed in the National Library in Edinburgh, John Rennie (1761–1821) suggested the use of cast-iron swing bridges for London Docks in an entry dated 4 April 1803 and the decision to use cast iron was noted on 16 April 1803. Although cast-iron swing bridges were erected in London Docks around this time, the earliest bridges were actually made from timber. William Jessop (1745–1814) designed wooden bridges for the Caledonian Canal (Figure 1), although they were in fact eventually built in cast iron. On wooden bridges, the timber trusses forming the bridge leaves could have been distorted to create the arch. However, this was more difficult for cast iron bridges, particularly when they increased in size. For example, when double carriageways were introduced in the 1840s, angled bearing pads were used to provide stability when the bridge was moved and horizontal ones were used as arch bearings when open to road traffic. In order to maintain a sliding fit for the bearing pads, it is probable that the bridges were balanced about a point towards the roller track/bearing blocks closest to the navigable passage and not about the central bearing. Until around 1850, the central bearing did not carry any vertical load, and bearing clearance may have been such as to allow the bridge to tilt slightly when loaded, allowing the bearing pads to become active.

In 1840 the railway arrived in Hull, terminating at the corner of Kingston Street and Railway Street close to Humber Dock. To cope with increasing trade through the port of Hull, two new docks were proposed (with Acts of Parliament passed in 1844 and 1845). The opening of the railway swiftly led to the building of Railway Dock. This is not shown on the proposal dated 1840 (Figure 2) but was built off the north-west corner of Humber Dock, close to the goods terminus; it opened in 1846. It was necessary to widen access for the growing number of steam vessels using the docks, so two new swing bridges spanning the improved dock entrances were built at the same time. This involved replacement of the original Wellington Street Swing Bridge, a creation of John Rennie who just 40 years earlier had designed Humber Dock. He is commemorated on a plaque on
the eastern side of Humber Dock Street (Figure 3). Construction of the second proposed new dock, dock basin and timber pond shown on the plan followed and, on completion in 1849, was named Victoria Dock.

The construction of Hull’s docks and swing bridges during the early Victorian period was carried out under the direction of John Bernard Hartley (1814–1872). Hull’s swing bridges are virtually identical with the one built in 1842 over the passage...
between Salthouse and Canning Docks in Liverpool. Any slight variations in dimensions can probably be ascribed to casting methods of the time. For instance, large patterns were necessary and these may have been constructed in sections for assembly during moulding, resulting in slightly different overall dimensions. His father’s (Jesse Hartley) design for Clarence Dock Bridge in Liverpool (1834) and Canada Dock entrance bridge (c. 1840) are forerunners of the same basic design.

Jesse Hartley (1780–1860) was born in Pontefract; his father was Bernard Hartley, surveyor of bridges to West Riding from 1797. Jesse Hartley worked in Ireland until 1818 when he returned to England to become bridgemaster for the Salford Hundred in Lancashire. In 1824 he became dock engineer at Liverpool, probably the world’s most important dock engineering position at the time and a post he retained until his death in 1860. Hartley’s work can still be seen today and among the bridges he built are those at Ferrybridge (1804) and Castleford (1805–1808).

John Bernard Hartley was trained as an engineer by James Walker (1781–1862), although he must have learnt much from his father. He was certainly working with him in the 1840s when he was retained as consulting engineer for Hull Docks during the construction of Railway and Victoria Docks. Afterwards, he continued to work in Liverpool until a year after his father’s death. He retired due to ill-health, but continued to advise engineers until he died in 1871.

The engineer Ralph Walker produced the first drawing for a cast iron swing bridge in 1800 as part of the design stage of the earliest London Docks (Figure 4). Wooden swing bridges were already widely used on canals at that time, although they were usually single leaf. Walker’s design was for a double-leaf bridge. When the bridge was used by road traffic, the two outer ends of the leaves were locked together and bearing pads were fitted to the abutments close to the edge of the bridge passage. When the bridge came under load, as a vehicle passed over it, the locked outer ends of the leaves would descend slightly and this would...
cause the ironwork of the bridge to press against the bearing pads (thrust blocks), forming an arch and thus increasing the bridge’s load-bearing capacity. The one major difficulty today is to understand how the bridge moved to create an arch.

Distortion of the bridge girders is improbable, so it is more likely that clearance of the rollers and central bearing allowed movement. While turning, fixed bearing pads could cause the bridge to jam or, because of the close tolerances required, were vulnerable to breakage. It was possibly to overcome this problem that John Rennie, in a drawing that probably dates from 1803, suggested the use of a moveable wedge operated by gears. This system survives on the cast iron swing bridge at Leith Docks in Edinburgh, and it was probably used in London. In later designs, such as Wellington Street Swing Bridge, fixed bearing pads were continually under load, the bridge sliding across several pads as it turned, and further horizontal pads took the load when the bridge was closed and in use by road traffic.

Over the next 50 years, the design was developed for bridges spans up to 13.7 m. With the introduction of steam ships in the 1850s, the width of ships using docks increased and passages of 18.3 m and over were required. Due to its limited tensile strength, cast iron was unsuitable for structures spanning such widths and from the 1880s, wrought iron and steel were used in the construction of dock opening bridges. Most cast iron swing bridges were subsequently replaced, though a few survived on smaller docks, such as Humber and Railway Docks. Originally manually operated, hydraulic power was used from the late nineteenth century and, in the case of Humber Dock, conversion to electric power was undertaken in the late twentieth century.

Wellington Street Swing Bridge was developed from numerous swing bridges constructed prior to 1846. Appendix 1 gives a chronology of some early large swing bridges; few have survived, most having been removed during dock or road improvements. Cast iron swing bridges similar in size to those in Hull have survived in Liverpool, Leith and Plymouth. The bridge at Liverpool’s Albert Dock is still in use, but there are increasing problems with balance and consequent ease of operation. The Leith Bridge is now fixed as there is no water-borne traffic. None survive in London Docks.

Cast iron swing bridges can be divided into two types depending upon how the weight of the bridge is supported. On the earliest examples, the load is taken by wheels or rollers that run on a large-diameter circular track under the bridge; Wellington Street Swing Bridge is of this configuration. On a few cast-iron swing bridges built around 1850, the load is supported by both the central bearing around which the bridge turns and the roller track. On some later wrought iron and steel swing bridges, wheels at the inner end of the bridge act to stabilise the bridge when in motion.

3. WELLINGTON STREET SWING BRIDGE

The grade 2 listed bridge bears great similarities to Ralph Walker’s original 1800 design (Figures 4, 5 and 6). Constructed in 1846, it is a single-span, double-leaf (two swinging portions), rim bearing, balanced asymmetric cast iron swing bridge following the principles of a three-pin arch. A three-pin arch forms under application of live load. Thrust blocks at the foot of the main girders provide two of the pins and the shear keys at the nose connection provide the third. The structure spans a lock basin between the Humber estuary and Hull Marina. The double-leaf superstructure comprises six cast iron girder trusses with a timber deck and metal handrails.

The superstructure is supported on a wrought and cast iron slewing mechanism, a form of rim bearing. The rim bearing comprises a centring pintle, 31 tapered roller bearings, cage plates, spokes and upper and lower roller tracks, all of which form the slewing mechanism. For each leaf of the structure the centring pintle serves the purpose of holding the slewing mechanism in place and providing the point of rotation. The roller cages and spokes provide lateral restraint to the roller bearings ensuring a circular moving path. The rollers travel between upper and lower plates known as the roller tracks.
The substructure comprises Yorkshire stone retaining walls that are shaped forming the lock basin; a recess is positioned at each side of the lock into which the bridge deck leaves neatly sit. It is thought that the masonry was built onto a timber raft or framework that was founded on timber piles, although this has not been confirmed by site investigation.

The original form of the structure was manually operated. Over time, the operating method progressed to a hydraulic driven system and finally an electric driven system. Up to the time of the refurbishment, the timber deck had been retained, although its condition was extremely poor.

The outer girders of dock swing bridges, particularly those alongside the passageway for shipping, are extremely vulnerable to shock loading, either from vessels hitting the bridge or from the bridge striking objects during turning. Structures built from cast iron are prone to damage due to the brittle nature of the material and Wellington Street Swing Bridge was no exception. The outer girders had been fractured on several occasions, and were heavily plated. Evidence of cracked ribs was found in the main girders; this damage had been repaired using iron or steel plates bolted over cracked frame elements. This in itself was considered an important feature, indicating the reason why such bridges and structural cast iron were superseded. To return the bridge to operation, the tracks and rollers required restoration—a difficult job as the dock level had been raised such that the track is occasionally under water.

One problem in the restoration of historic load-bearing structures open to the public is that work today is undertaken to rigorous modern safety standards. Modern structures are engineered to design specifications that ensure a fitness for purpose of that specific structure. (The standards and specifications used throughout the assessment and design of Wellington Street Swing Bridge are listed in Appendix 2.) Joints and bearing surfaces have to be of high standard and accuracy. Older structures were often over-engineered, with fastenings aligned on site and modifications made as necessary. Historically, during erection, the framework created by the girders and spacers did not have to be completely square and ties ran through clearance holes. In some cases, such holes have been enlarged, either during original construction, or during subsequent repair. The modern method of assembly assumes that the side frames are an integral part of the whole bridge structure and that all faces need to mate squarely. This requirement puts great emphasis on the structural integrity and strength of the outer girders, something that Wellington Street Swing Bridge girders did not have.

Several approaches are possible in a process of restoration and the various methods result in slightly different outcomes. When preserving an industrial object, there are three main possibilities that should be considered. The adoption of one of these options will then dictate the methods to be used in preservation.

(a) Option 1 is to conserve the object as an example of its type. Today, this means keeping the object in its ‘as-worked’ condition, just as it had finished working. Previously, the object may have been returned to its original as-built condition.

(b) Option 2 is to maintain the object, using traditional materials and methods, in order to preserve and record the traditional skills used in maintenance and operation, as well as preserving the object itself in a condition similar to that when in use.

(c) Option 3 is to maintain the structure using modern methods and materials in such a way as to be able to operate the object regularly and safely.

Wellington Street Swing Bridge could have been restored using option 1, which would restore its appearance, although it would not have been suitable for operation. Considerable work would have been needed just to stabilise the structure and it would have been difficult to ensure long-term sustainability. Furthermore, too much work had already been undertaken to make such an approach a viable way forwards from a heritage, practicality and economics standpoint. Therefore three possible ways for restoration to working condition were available. First, to reuse the existing outer girders, thereby retaining the original appearance and form of construction (option 2); second, to fabricate new outer girders (option 3); and third, to cast new outer girders (option 3).

Wellington Street Swing Bridge is not unique; there is a similar bridge nearby, at Railway Dock, which still has its early hydraulic operating machinery. Wellington Street Swing Bridge has been converted to electric operation. With two similar bridges exhibiting different approaches to operation, here too there are possibilities for interpretation. Another cast iron swing
bridge, slightly more modern in design, is preserved less than a mile away at the former entrance to Alexandra Dock. There is also the slightly smaller and earlier, although basically similar, cast iron swing bridge at Liverpool’s Albert Dock.

Many stakeholders had an interest in the structure, including Hull City Council, English Heritage, Hull City Build, Hull Planning Department, Humber Archaeology and British Waterways (operators of Hull Marina). Each stakeholder had their own views on the development of the works, for example

(a) English Heritage required minimal aesthetic impact to the structure and retention of major elements.
(b) Hull City Council favoured ease of operation and maintenance
(c) City Build required a cost-effective solution
(d) Humber Archaeology focused on conservation of history.

All views were taken into consideration and options were developed, discussed and discounted through close discussion with the relevant parties. It was decided that the best approach would be to refurbish the structure in line with option 3, preventing the structure from remaining in disrepair and out of use. However, due to the grade 2 listed status of the bridge, some features of the structure had to be retained. These included the cast iron rail kerbs (Figure 7) as they were an integral part of the original aesthetics of the structure, the hinged handrails that could be folded back clear of the lock side during operation and also many original major structural elements. Generally, it was considered desirable to retain the overall appearance of the bridge. Furthermore, other features around the bridge, including mooring bollards and original sections of handrail that were of historic value, added to the general setting and were worthy of preservation.

4. ASSESSMENT

In 1999 Pell Frischmann was commissioned to undertake an inspection of Wellington Street Swing Bridge and to provide an assessment of its strength. The inspection highlighted some obvious and not so obvious defects in the bridge. Briefly, the defects comprised

(a) poor condition of the decking
(b) significant damage to the outer main girders (girders 1 and 6), with numerous repairs present in the form of plating
(c) severe damage to upper support ring of the structure
(d) damage to the nose tongue-and-groove sections
(e) plastic deformation to both upper and lower cast iron roller paths
(f) kentledge of loose rubble poorly compacted and retained
(g) damage to the gear rack at the tail of the east leaf.

Through studies of historical information and discussions with the operators, it was determined that the east pintle was probably broken. This was borne out by the fact that the structure moved out of alignment during a trial swinging operation. The uneven swing of the structure meant that the two halves of the structure did not connect properly when in the ‘open to traffic’ position. It is thought that the plastic deformation observed to the upper and lower roller paths was as a result of the pintle breakage, exacerbated by imbalance in the structure due to insufficient weight of kentledge. This highlighted the need to ascertain the correct weight of kentledge required.

The assessment was based on the BD21/97 assessment of highway bridges and structures.\(^1\) The structure was analysed using a simple static distribution for loading. A two-dimensional (2D) frame model was used to determine the level of forces within the numerous members of the bridge. Two separate models representing open and closed conditions were considered—a cantilever situation and a three-pin arch. The cantilever was analysed for dead loads only and the three-pin arch was analysed for a combination of dead and live loads.

As the structure is of cast iron construction, permissible stress criteria were used to analyse the bridge elements with limiting stresses (as defined by BD21/97) of 154 N/mm\(^2\) in compression and 46 N/mm\(^2\) in tension. The critical element of the structure was found to be the top boom (uppermost horizontal member of the truss) of the main girders. This was rated a zero live load capacity (i.e. only capable of taking a dead load).

The structure was not considered to be of strategic importance at the time and, due to the limited capacity the bridge, was taken out of service and left in its ‘parked’ position, that is open to boat traffic.

5. FEASIBILITY STUDY

In 2004 Pell Frischmann was commissioned to investigate the feasibility of bringing the bridge back into use and to develop refurbishment options with a view to taking these through to construction. The catalyst for the works was the regeneration projects taking place in Hull, in particular around the fruit market area. The structure was now situated on a strategic link, providing key access across the Hull Marina lock entrance.

The client’s aspirations for the structure were 7.5 t assessment live loading (ALL) with pedestrian loading or crowd loading alone; the structure was to be easily operated and maintained, and the refurbishment works were to be carried out within a defined budget. The preferred option needed to be developed to a stage whereby a planning application could be made.
As part of the feasibility study, it was felt that a more detailed examination of the structure should be undertaken with the intention of categorising types of repair required. The condition of the bridge had not deteriorated appreciably in the intervening years and was verified to be as detailed in the initial assessment made in 1999. Repairs were thus still required to the upper support ring, outer main girders, decking and the nose tongue-and-groove sections.

The opportunity was taken to involve other engineering disciplines with a view to determining the condition of and repairs required to the mechanical and electrical elements. At this stage it was not possible to examine the condition of the pintle without removing the superstructure. The rollers were in good condition, although the roller paths exhibited plastic deformation. The thrust blocks were in poor condition and required a degree of repair. In general the rack and pinion was in fair condition; small elements of the rack required replacement on the tail of the east leaf. The spokes and roller cages were in poor condition and required replacement. The electrical drive system also needed to be replaced as it was outdated. The ducting was not inspected at this stage of the works.

To develop the feasibility study, several assumptions were made about the structure’s intended behaviour. It was assumed that the structure would behave as a three-pin arch when subjected to vehicular loads. Gaps would be present under dead load conditions, at the interface between the leaves of the bridge and the thrust blocks at the springing of the arch. Upon application of live load, the gaps would close to form the arch. Deck loads (dead and live) were assumed to be applied at top boom modal positions to prevent bending in the cast iron elements. The deck was intended to be non-composite with the main girders.

At this stage the suspected broken pintle was considered and reasons for the breakage were developed. It is assumed that the original function of the pintle was to carry minimal horizontal forces and maintain the position of the main girders relative to the roller ring. While the reason for breakage could not be absolutely defined, as mentioned earlier it was considered likely that it had broken due to poor balance of the structure, although another cause may be poor contact between the thrust block and nose connection under load. This consideration highlighted the need to ensure that the structure would behave in its intended manner when recommissioned.

The 1999 assessment was based on a conservative form of analysis employing a 2D frame model. It was felt that improvements in bridge capacity could be made by utilising a more complex sophisticated model. In addition, all works were to be carried out in accordance with the revised assessment of highway bridges and structures, BD21/01.\(^15\) However, on formulation of a three-dimensional (3D) computer model with a replacement timber deck, the results showed no significant improvement. It was considered that the timber deck was not offering sufficient stiffness to distribute loads between girders. Sensitivity analysis on the stiffness of the deck soon indicated that greater capacity could be obtained with the use of stiffer elements representing the deck and this became the method for developing strengthening options.

Consideration was given to several deck options before modelling began. These included a composite steel and concrete deck. Preliminary calculations ruled out this form of deck as it would have too large a dead weight and a significant visual impact not in keeping with the listed status of the structure. Steel and aluminium decks spanning between the girders were also considered, but the lack of distribution from the edge girder to the internal girders under pedestrian loading and the extremely poor condition of the edge girders meant that these options could not be carried forward. It was therefore decided that comparisons between steel and aluminium decks spanning over the internal girders and cantilevering over the edge girders would be carried out, therefore isolating the edge girders from picking up live load and hence making them become a cosmetic feature of the bridge.

A condition factor of 0.9 representing the state of the structure was applied to enable a true comparison of results with the original assessment. In both options a shear key was considered to act between leaves, enabling transfer of vertical forces.

In conclusion, the limitation for the capacity of the structure was based on the performance of the main girder top boom. The original timber deck offered limited load distribution and hence insufficient load sharing across the main girders; therefore timber decking was discounted. Aluminium decking provided a capacity similar to the steel deck, leaving a choice between the two. Hull City Council expressed a preference for the steel deck.

Utilising the above approach, main girder strengthening was kept to a minimum and limited to the external girders. Because the external girders were effectively only supporting their own dead weight, the repairs required were principally cosmetic. Options for repair included stitching, plating over and plate bonding. To keep the aesthetic appearance of the structure, the decision was made to maintain the original repair concept of additional plating.

6. DESIGN

Through the development of the options, the model evolved into a 3D frame representing the main girders with a finite element slab representing the bridge deck. The Lusas finite element analysis\(^16\) package was used for these purposes. Two models were created to represent the open position (Figure 8) and the closed position (Figure 9). 3D engineering beams (BMS3s) were used to represent the main girders and deck beam elements. The deck plate was represented as a 3D thick shell (QTS4s and TTS3s) with an appropriate mesh created when the model was complete. A pinned connection was created between the main girders and the finite element slab, ensuring the deck was not composite with the main girders. This avoided the introduction of torsional load effects from the deck. This was achieved by creating a moment release at the base of a dummy element, representing the eccentricity of the deck. Braking and traction loads were transmitted through the deck by way of an anchor at the tail of each leaf attached to the kentledge system. These loads are in turn transmitted through the turning ring into the substructure below.

Discrete supports were assigned to each individual roller position with additional supports provided for both the thrust blocks and the pintle. Due to the nature of the model, specific supports could be active or inactive during analysis, allowing...
Figure 8. Model of single leaf

Steel orthotropic deck
Kentledge
Spider

Figure 9. Model of full structure

Steel orthotropic deck
Kentledge
Spider
the behaviour of the structure to be easily manipulated. It was noted that under certain load conditions some supports were lifting off and that approximately only half of the bearings were utilised under the original dead load conditions.

An estimation of the weight of the existing kentledge was made. Following this, the exact weight of kentledge was determined to balance the structure under dead load conditions. Nominal unfactored loads were used for this purpose. The kentledge was formed from cast in situ reinforced concrete and steel plates, which provided the opportunity to fine balance the structure during the commissioning process. The existing cast iron kentledge base plates were retained as they were in good condition. In total, some 14·2 t of reinforced concrete and 13 t of steel plate were utilised as kentledge in each leaf.

The existing shear key, a tongue-and-groove system built into the nose casting of each leaf, was to a low-tolerance fit and allowed movement of each leaf under live load. It was considered that this would not provide the middle pin of the arch configuration without great modification. A new shear key system was thus developed for incorporation into the new deck. The new shear keys (Figure 10) transfer vertical loads by mechanical interlock upon application of live load. The interlock occurs when a nominal gap of 1·5 mm between the male and female sections is closed under live load. In total, four shear keys were built into the deck, one above each internal main girder. A degree of adjustability was added into the shear keys by allowing removal or installation of varying depth shims to account for construction tolerances. Properties were assigned to the joint to allow the transfer of forces between the two elements of the model at the level of the main girders.

The thrust blocks were utilised when the reaction shoe on the foundation came into contact with the lower element of the main girder, creating a point of reaction. No contact between the main girders and thrust blocks was assumed for the dead loads present on the structure. For live loading (pedestrians and HA loading), contact of the thrust blocks was considered active for the four internal girders and inactive for the external girders. This ensured that the external girders were not subject to live loading due to their condition.

A single lane of Highways Agency (HA) loading adjusted to represent 7·5 t vehicles (in accordance with BD21/01 15) was considered across the centre of the structure over a 3·65 m wide lane. In combination with HA loading, pedestrian loading of 5 kN/m² was limited to the footways above the edge girders. As a separate load case, crowd loading was also considered at 5 kN/m² applied over the entire structure. No accidental wheel loading was taken into consideration for the edge girders as it was presumed that appropriate protection would be provided to prevent this occurring.

Bespoke handrails were designed to current load requirements while maintaining a similar overall appearance to the originals. As the original lock side rails were foldable, this feature had to be retained. To avoid accidental or malicious folding of the rails, they were secured with locking bolts; removal of the locking bolts is required if the rails need to be folded (e.g. tall ship events utilising the marina facilities).

To prevent bimetallic corrosion of the steel decking, a system of natural rubber packers was placed below each transverse deck element, located at the top boom nodes. In addition, a system of clamps was provided to prevent ‘lift off’ of the deck.

7. CONSTRUCTION

The construction contract was based on a traditional form of procurement with monies set aside as contingency for the risk of unforeseen defects that may have become apparent during dismantling of the structure. The contractor elected to remove...
the superstructure from site and transport it by road to their
works, where, in the controlled environment of a workshop, the
bridge could be entirely dismantled. Prior to this, the bridge had
to be fully surveyed and partially dismantled to create suitably
sized pieces for transportation. The contractor’s programme was
planned so that restoration of the surrounding on-site features
was carried out whilst the bridge was away from site.

The existing roller wheels were re-bored and re-profiled for use
on the structure. The east side refurbished tapered roller bearing
assembly is shown in Figure 11.

The two outer or edge girders were found to be more slender
than the inner four girders; this is perhaps understandable
because they were intended to only carry the footway, but had
not been previously recognised. Some damage was identified
during the survey, but other cracks were found on dismantling.
During disassembly, the second outer girder to be removed was
being manipulated by overhead crane when, instantaneously
and without warning, the girder fractured across its full width
and fell to rest on the ground. This risk had not been foreseen
and, in hindsight, the incident could have been very serious had
it happened while the girder was suspended high in the air;
fortunately, only the girder sustained damage. An investigation
was carried out along with a metallurgical examination of the
fractured cast iron to determine the cause of the incident and
help develop proposals for future handling of the girders.

The girder (overall length approximately 14-5 m) had fractured
at a distance of 6-0 m from one end, not at mid-span as might
have been expected. The fracture surfaces show a brittle fracture
mode (Figure 12) typical of grey iron that has been subjected to
tensile overload and/or bending. Two features were apparent
and may have contributed to the failure—an over-sized bolt hole
in the top flange from where the fracture appears to have
originated and two inclusions (foreign matter) very close to the
surface.

A second fracture occurred as a consequence of the first. This
fracture was a direct consequence of the corner of the girder
hitting the floor after the girder broke in two. There was
significant evidence of gross porosity and graphitic or selective
corrosion defects on the surface, which suggests extremely poor
integrity and dubious mechanical strength.

Figure 11. Refurbished roller bearing assembly

Metallurgical tests on two samples were undertaken to establish
the material’s properties. Chemical analyses of the samples were
very similar, the test results generally classifying the material as
grade 12 according to BS 1452.17 Phosphorous levels, however,
were significantly higher than anticipated, with significant
implications and effects on microstructure and mechanical
properties. Phosphorous levels in excess of about 0.2% in this
grade and wall thickness will promote brittleness due to the
formation of a phosphide eutectic, which was clearly visible in
the microstructures. This high phosphorous level is not too
surprising given the age of the bridge; iron ore from local high-
phosphorous sources would probably have been used rather
than imported Swedish iron ore, which was renowned for high
purity and low phosphorous levels. Early nineteenth century
iron founders did not fully understand the importance of
phosphorous, nor did they have the slag technology to be able to
control it. The iron was probably made in a cupola furnace with
little or no refining treatments.

Mechanical properties derived from testing confirmed the
material to have low tensile strength (89 MPa) and low
elongation (1%), giving further explanation as to why the girder
had fractured so readily and unexpectedly. The recorded
hardness value of the girder was 197 HB (Brinell hardness); it
was noted that grade 12 grey iron listed in BS 1452 specifies a
minimum tensile strength of twelve tons per square inch (185
MPa) with a corresponding hardness of 205 HB.

Wider visual examination of the girder identified historical
damage amounting to gross section failure (evidenced by a
‘fishplate’ repair) and one partial crack across the weakest point
of the flange. Close visual inspection revealed that this very old
fishplate repair took place early in the life of the bridge, perhaps
even during construction when difficulties of handling the
girder would have first been encountered.

The investigation concluded that the method of handling the
girder placed the ‘weakest’ features on the tensile side when
subjected to bending, which would not have been obvious at the
time. For all subsequent lifting operations, two girders were
clamped together to provide more stability and a rigid item for
safer handling.

As a consequence of the incident and the resulting broken outer
It was considered that the introduction of new parts would not necessarily put the historical integrity of the bridge at risk. Many alterations and additions were already in place as the bridge had been repaired and adapted for new regulations and methods of operation during the present restoration scheme and (less sympathetically) at other times in the past. The plating of existing cracks in the outer girders and elsewhere on the bridge had also created changes to the original structure. New outer girders would enable the bridge to be rebuilt using modern methods and thus comply with modern safety standards.

The question of how the new outer girders should be made needed addressing. In theory, casting new girders would be closer to the original and therefore superficially a more attractive option. However, in practice, it is now impossible to cast large objects in one piece and this capability was researched by the contractor with no success. Casting three pieces and then joining them together is a different system to that used originally and the original material (grey iron) is not available. The result would, therefore, lack integrity. It was considered that the simpler option of fabricating a new outer girder from steel plate was possibly the best and most honest option; it could be similar to the original in terms of general design, but lacking the detail decoration on the original. Close up, it would thus be easy to identify the replacement parts whilst not significantly altering the overall look of the bridge. This is in keeping with current best practice adopted for architectural and artefactual restoration works, which requires modern alterations and additions to be apparent.

After consultation with the local conservation officer, approval was granted for the manufacture of four new girders to replace the original outer girders and an addendum to the original listed building consent was submitted. Four new outer girders were fabricated from steel using the latest techniques and equipment.

In the contractor’s workshop, all parts of the bridge that were to be retained were stamp marked for identification purposes before being thoroughly blast cleaned, surveyed and inspected for damage and repaired. Finally, the application of a protective system prepared the components for assembly, whether on site or in the workshop. This process revealed much. The tails of the main girders were in poor condition and numerous ‘blow holes’ could be seen (Figure 13). These holes had developed during casting of the beams and were concealed behind the loose kentledge. As this particular element of the structure was not highly stressed, a simple repair of putty-filling the holes was undertaken prior to re-painting.

After the initial set back of breaking the girder, the remaining works progressed largely according to plan and new components were manufactured to replace decrepit parts that could no longer provide a working solution. Essentially, the main structure remains original with the new steel deck supported on top. Mechanically much of the rotating mechanism needed replacement to meet the demands of regular use, although parts were restored where possible (e.g. the geared drive rack). New drive motors were fitted into the existing housings and an all new electronic control system was installed.

Figure 14 shows the structure being lifted into position; only on very close inspection can the new outer girders be seen to be different from the cast iron originals. The rail kerbs needed to be re-profiled for bolting onto the steel deck plate as these had previously protruded through the timber deck. The deck was finally waterproofed using a sprayed applied membrane and topped with a non-slip coating.

8. CONCLUSIONS

The maintenance of Hull’s historic bridges has been managed successfully over recent years with significant funding being made available at the appropriate times. In part, this can be
attributed to the historical audit of each listed bridge and to the development of an action plan, though there can be no doubt that the future of any listed structure can be best secured when there is a demand for its continued use, as in the case of Wellington Street Swing Bridge.

The replacement of such an historic structure with a modern equivalent asset was considered, but the feasibility study found in favour of the original bridge. The cost of refurbishment also proved reasonable and beneficial to the client. Based on a recent asset valuation exercise, the bridge cost around £6100 per square metre of deck, which is considered to represent good value for a moving bridge. At the same time this solution provides a sustainable long-term future for one of the last remaining bridges of this type.

The nine listed bridges in Hull form an interesting group and provide an insight into the development of bridge technology over the last two centuries. Although not comprehensive, the group value cannot be overlooked and indeed it may be enhanced by the future inclusion of more bridges. The completed Wellington Street Swing Bridge is shown in Figure 15.

**APPENDIX 1. EARLY SWING BRIDGES**

1793 Selby Swing Bridge designed by William Jessop and John Carr; opening span 10-7 m.10

1796 William Jessop suggests the use of swing bridges rather than lift bridges for the Gloucester & Berkeley Canal.4

1801–02 Blackwall entrance lock, West India Docks, London; 11-6 m wide, double-leaf wooden swing bridge, iron rollers on 3-7 m diameter circle.11,12 Designed by William Jessop using plans drawn up by Ralph Walker in 1800. The original design was for cast iron, though wood used in practice.

1801–02 Limehouse entrance lock, West India Docks, London; 11 m wide, double-leaf wooden swing bridge, iron rollers on 3-7 m diameter circle.

1804–05 City Canal entrance locks, London; width 13-7 m, double-leaf wooden swing bridges supported on twenty iron rollers between plates; engineer William Jessop; ironwork supplied by William Bailey, James Ward and William Crawshay.

1807–13 Caledonian Canal swing bridges similar to those at West India Docks. Engineers William Jessop and Thomas Telford. Cast iron was used instead of wood for the bridges at ‘Muirtown and elsewhere’ from 1813.4

1808 Cast iron swing bridge, similar to those in London, erected in Hull; builders Aydon & Elwell, Shelf Ironworks, Bradford.13

1808–09 Cast iron swing bridge erected at Queens Dock, Liverpool, similar to that in Hull; builders Aydon & Elwell, Shelf Ironworks, Bradford.13

1810–11 Cast iron swing bridge erected at Kings Dock, Liverpool; builders Aydon & Elwell, Shelf Ironworks, Bradford.13

1815 Cast iron swing bridge erected at Old Dock, Liverpool; builders Aydon & Elwell, Shelf Ironworks, Bradford. It was moved to Brunswick Dock when the Old Dock was in-filled in 1826.13

1820 Cast iron swing bridges erected at Princes and Georges Docks, Liverpool; builder William Hazledine.13

1832 Two cast iron bridges for Liverpool Docks built by Hird, Dawson, Hardy & Field of Low Moor, Bradford.13

1842 Three cast iron swing bridges for Albert and Salt House Docks, Liverpool, to be built by Haigh Ironworks, Wigan. Albert Dock bridges have a single carriageway; the Salthouse Bridge has a double carriageway.13

1843 Cast iron swing bridge for Georges Dock, Liverpool, to same specification as Salthouse Dock, built by Haigh Ironworks.13

1846 Cast iron double-carriageway swing bridge over south entrance to Georges Dock, Liverpool, built by J. & E. Walker.13

1846 Cast iron swing bridges, similar to that at Salthouse Dock, Liverpool, erected in Hull by Haigh Ironworks.13

1848 Cast iron swing bridges for Alexandra Dock, Hull, built by Beecroft, Butler & Co., Kirkstall Forge, Leeds.13

1853 Five cast iron swing bridges built for Wapping Docks, Liverpool, built by Beecroft, Butler & Co., Kirkstall Forge, Leeds.13

**APPENDIX 2. CODES USED IN THE ASSESSMENT AND DESIGN OF WELLINGTON STREET SWING BRIDGE**

BD21/97 The Assessment of Highway Bridges and Structures
BD21/01 The Assessment of Highway Bridges and Structures
BS 648: 1964 Weights of Building Materials
BS 5400-3: 2000 Steel, Concrete and Composite Bridges. Part 3: Code of Practice for Design of Steel Bridges
BS 8500-2: 2002 Concrete (complimentary British Standard to BS EN 206-1 Part 2: Specification for Constituent Materials and Concrete)
BS 8666: 2005 Scheduling, Dimensioning, Bending and Cutting of Steel Reinforcement for Concrete
PD 6484: 1979 Commentary on Corrosion at Bimetallic Contacts and its Alleviation
BS 2573-1: 1983 Rules for the Design of Cranes. Part 1:
Specification for Classification, Stress Calculations and Design Criteria for Structures

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REFERENCES

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