We are IntechOpen, the world's leading publisher of Open Access books Built by scientists, for scientists

6,300

Open access books available

172,000

International authors and editors

190M

Downloads

Our authors are among the

154

Countries delivered to

TOP 1%

most cited scientists

12.2%

Contributors from top 500 universities

WEB OF SCIENCE™

Selection of our books indexed in the Book Citation Index
in Web of Science™ Core Collection (BKCI)

Interested in publishing with us?
Contact book.department@intechopen.com

Numbers displayed above are based on latest data collected.
For more information visit www.intechopen.com

Temporary Works in Construction of Bridges Near Third Party Assets

Ganga Kasi V. Prakhya

Abstract

This paper gives a summary of the temporary works and methods applied to enable the construction of bridges near third party assets. The temporary structures have a significant impact on the cost, construction method and construction safety of the supported permanent structures. In the literature, there are many examples of how the temporary works could fail catastrophically and could endanger public life if hazard identification, risk assessment and quality checks are not carried out by competent people or organisations. A brief literature survey of the construction techniques is outlined here. This paper looks at more recent failures and draws some more lessons with a re-emphasis on the use of industry established processes, guidelines for preventing catastrophic events. The paper also describes case histories, where mitigation measures are implemented in order to ensure safety by means of independent checks, monitoring and back analysis.

Keywords: temporary works, monitoring, safety in construction, method statements, risk assessment, back analysis

1. Introduction

Most forms of bridge construction, whether preassembled or cast in-situ will require temporary works on site. These may include, depending on the type of bridge, temporary supports for precast girders or beams, box structures, and temporary staging for cast in-situ construction of the deck, and may also involve specialist operations for complex forms of bridge, e.g., post-tensioning. Many projects focus on optimising the concrete volumes and steel tonnage but a very few focus on how best to integrate temporary works and buildability concepts into the permanent design at the early stages.

Temporary works (TW) are the parts of a construction project that are needed to enable the permanent works to be built. Usually the TW are removed after use—e.g. access scaffolds, props, shoring, excavation support, falsework, and formwork, etc. It may be possible that sometimes the TW an integral part of the permanent works—e.g. props for excavation can be used as part of permanent steelworks, haul road foundations and crane or piling platforms may be used for hard standing or road foundations. In view of safety, it is very important that the same degree of care and attention is given to the design and construction of temporary works (TW) as to the design and construction of the permanent works. As TW may be in place for only a short during the construction phase of the project, there is a tendency to assume they are less important. Lack of care with design, selection, assembly, etc. leaves TW liable to fail or collapse [1]. This places people at risk of injury and can cause the project to be

delayed. Therefore it is important to ensure that the methods, materials, and sequence of construction is thought through with the construction team during early stages and that the risks to the structure as well as the third-party asset owners are resolved.

Where bridges form features of modern infrastructure in densely populated cities and urban areas, the designers will be challenged with site logistics, narrow and busy streets, and third party assets such as railways, underground tunnels, and buried services and it is extremely important in these cases to be more diligent about designing for safety of the temporary works. While the principles can be applied to all bridges, this paper does not cover large suspension bridges on water.

2. Background and UK scenario

Although there were bridge failures in 1950s in USA, Canada and other parts of the world, much of the development in temporary works design in UK was improved when Barton Bridge, Lodden Bridge [2–4] in 1960s and 1970s collapsed during construction and the government was moved to determine whether the industry was in a fit state to manage falsework. Following these major failures in the UK, Professor Stephen Bragg report [4], helped to set the standard for temporary works design and management in the UK. Major failures of UK temporary works have almost disappeared since BS 5975 [5] was published in 1982, following the Bragg report in 1975 into recent falsework disasters in which many lost their lives. In the UK, the design of temporary support trestles would normally comply with the requirements of BS 5975 [6] unless the use of Eurocode 12811 [6] and EN 12812 [7], is stipulated as a contractual requirement. More recently, a Temporary Works forum (TWf) was formed in 2009 [8] in the UK as an independent, non-profit company that operates on a limited cost base. Useful guidance and toolkit and guidance documents are produced by TWf that address the issues of temporary works and applicability of Eurocodes for safer construction in the UK [9]. The general principles of these tool kits generated by the TWf are applicable worldwide.

More recently, the Institutions of Civil Engineers and Institution of Structural Engineers (UK) formed an independent body called SCOSS/CROSS [10] Structural Safety Body in 2005. This Structural-Safety Body is a body devoted to evaluating, and anticipating where possible, trends in the construction industry and issuing warnings where necessary. A confidential (anonymous) reporting system called 'CROSS [10]' broadcasts events which it believes should be more widely known.

While UK standards are developed to a fuller extent to eliminate the catastrophic events in temporary work through the introduction codes, guidance documents [11, 12] we still see that failures occur for various reasons. A publication in ICE Forensic Engineering summarised failures in bridges until 2012 [13] and more than 60 major catastrophic failures across the world have been reported since 2012 to date [14] including the most recent failure in Florida. A few more recent failures are reviewed from the literature survey in this paper to highlight the importance of temporary works.

This paper describes briefly the existing construction techniques of bridges, and reviews more recent failures. There are many types of bridges and but this paper summarises a brief literature survey of current construction methods for precast, prefabricated, and cast-in-situ bridges.

This paper summarises typical scenarios in urban and rural environments and considers a few case histories with a view on how to safely manage the risks associated with the construction of bridges in an urban environment. Most forms of bridge construction, whether preassembled or cast-in-situ will require temporary works on site. These may include, depending on the type of bridge, temporary

supports for precast girders or beams, box structures, and temporary staging for cast-in-situ construction of the deck, and may also involve specialist operations for complex forms of bridge, e.g. post-tensioning. Many projects focus on optimising the concrete volumes and steel tonnage but a very few focus on how best to integrate temporary works and buildability concepts into the permanent design at the early stages.

3. Bridge construction techniques: a brief survey

3.1 Precast bridges

Precast concrete members in bridge systems are appealing because they lend themselves well to incorporating Accelerated Bridge Construction (ABC) methods. In some cases, ABC also includes integral column and cap beam systems for bridges utilising precast concrete girders which have several advantages over structures consisting of steel girders or cast-in-place concrete alternatives as shown in **Figure 1** [15, 16].

For single-span bridge decks, temporary support is normally undertaken directly from the abutment bearing shelf or equivalent and this is relatively straightforward. Beams to multi-span bridge decks may require a more complex temporary support system. If the beams are designed in short lengths until they stitched, they will require a temporary support. A common solution is to provide a temporary trestle support either on a temporary foundation or on the permanent foundation of the piers. Beams are landed on the trestle that supports the bridge deck structure until such time that it becomes self-supporting. These trestles can cost significant sums and involve extensive working at height.

Given the consequences of failure and the difficulty of correcting issues that manifest themselves following landing of bridge beams, a robust temporary support structure is a necessity. Factors of safety (as per permissible stress design, e.g. BS 5975) and design factors (as per limit state design, e.g. Eurocodes [17, 18]) can be used amended by the designer to reflect the above. Safety can also be enhanced by building redundancy into the structure. Unless reasonable calculations are made of the specifics, there should be reasonable margins in the design for uneven distribution of load, accidental load and out-of-tolerance assembly.

Figure 1.
Precast girders with GRP deck panels for in situ deck.

In the case of steel structures, and launching steel bridge girders, we may need temporary trestles on temporary foundations. These trestles will require careful consideration due to the launching operations as their stability under dynamic conditions is of utmost importance.

The trestles are generally designed for dead loads, live loads, moving dynamic loads, notional horizontal loads, secondary forces from mis-alignment and will in some cases have impact loads if they are near a live highway.

3.2 Segmental

Segmental construction is one of the most important developments in construction in the last century and is a very well proven method for delivering durable, long span, and repetitive structures that are both cost-effective and visually appealing.

Segmental concrete construction can be executed in two ways: using precast elements or through cast-in-place construction.

The advantages offered by precast elements are mainly related to fabrication, conducted in a plant that produces more consistency in quality products and where segments can be fabricated in parallel with early field construction activities, thus improving scheduling. The main challenges involved with precast segmental construction lie in the logistics and the setup process between the casting yard and the construction site. This includes a large temporary work system involving specialist materials and jacks, and large movable gantry parts.

Alternatively, cast-in-place construction requires that a substructure be completed prior to fabrication of the superstructure. Cast-in-place segmental construction is used when precast segments are too heavy to be shipped or access to the site is too restrictive, which can occur as spans get longer or bridges get wider.

Construction time is a key factor for projects in urban areas which require lane closures, detours, and traffic interruptions to be minimised. Precast concrete segments are often optimal as they can be built and stored until needed for erection, thus reducing the on-site time of large equipment and construction activities, thus increasing the pace of construction. **Figure 2** shows a multi span bridge over a busy highway built using precast and GRP panels for deck.

The choice between precast or cast-in-place primarily depends upon project size, construction schedule, weight of segments, and site access.

Figure 2.
Precast beams and cats in situ deck with GRP panels for formwork over existing highway—4 span bridge.

3.3 Balanced cantilever construction

The balanced cantilever construction method is used when several spans ranging from 50 to 250 m exist. Bridges using this method can be either precast or cast-in-place. Once the piers are built, they are used as an erection platform for precast segments, or to support a form traveller for cast-in-place segments.

This method can also be easily adapted to irregular and long span lengths, congested project sites, rough and water terrain, rail crossings, and environmentally sensitive areas.

The cantilever method is the preferred method for building cable-stayed bridges. Once segments are installed, they are supported by new cable-stays in each erection stage. Since no auxiliary supports are required, it is both an economical and practical solution for long cable-stayed bridges.

As can be seen from the above, all of the techniques will require sophisticated forms of temporary works to enable safe construction. Despite much regulation and improvement in methods/processes in temporary works design, we are still, unfortunately, experiencing catastrophic accidents in bridges (examples include Barton bridge in Manchester in 2016 failure of lifting systems revealed and a 2019 failure in Norway failure of bolts). The failures of temporary works can be grouped into the following main areas;

- Scaffold collapse
- Failure of the lifting equipment, or temporary bearings
- Insufficient design capacity of a cantilevered arm of the cantilevered construction
- Girder and connection failures
- Design and detailing errors
- Incorrect construction sequence
- Negligence and construction errors

If the temporary structures are located close to third party assets, there are further risks that are very expensive to correct there are explained in the following section. The paper covers bridges on land and in populated areas and will not cover large suspension bridges on water.

4. Hazards from and to third party assets

At bridge construction sites in the urban environment, it is not uncommon to come across: party walls, railways, existing grade listed buildings, existing underground utilities and live highways and riverways. The bridge design and construction will have to address how these asset owners can be protected including the safety of public road and railway users where applicable. The requirements of the asset owners may vary but the fundamental aspect of the construction logistics is to ensure that all assets and owners are protected and public safety is ensured. As per the CDM regulations in UK [18], the design and construction should also address life cycle management and also demolition at the end of design life [19].

4.1 Electricity cables and overhead lines

Injuries are usually caused by the explosive effects of arcing current, and by any associated fire or flames that may result, when a live cable is penetrated by a sharp object such as the point of a tool. Such effects can also occur when a cable is crushed severely enough to cause internal contact between the conductors. Injuries are typically severe, potentially fatal, burns to the hands, face and body. There is also a risk of electric shock.

Inadvertent contact or being in close proximity to overhead electricity lines with equipment such as scaffold tubes, irrigation pipes, metal ladders or vehicles, such as cranes, poses a risk of electric shock. Direct contact with overhead lines is not necessary as electrical current can arc or flashover any gap between the overhead lines and the object. It is therefore important to ensure sufficient clearances for overhead lines are maintained and plant/surcharge loading is restricted from trestle foundations.

4.2 Gas pipes

Damage to underground gas pipes can cause leaks that immediate or time related that may lead to fire or explosion. The ground pressure from trestles or crane mats or outrigger could pose a risk to the stability of the gas pipes and therefore a risk assessment is required from the surcharge loading.

4.3 Water pipes, sewers and drains

Although damage to water pipes is less likely to result in injury, the following may occur;

- A jet of water from a main can injure a person.
- Leaks of water from underground pipes can affect adjacent services and reduce support for other structures.
- Damage to, or removal of thrust blocks can result in sudden loss of containment and the movement of pipe fittings that may travel some distance or cause impact damage. While some sewage is pumped at pressure, sewers are generally gravity-fed, and the main hazards from damage to a sewer are the possibilities of contamination and subsidence.

The type of materials for these water/sewage pipes, will include brick, cast iron, ductile iron, clay or concrete. Some asset owners, will give limitations on the surcharge loading and acceptance criterion for ground settlements or strain, an example of which is shown below from Thames water UK [20]. Current regulations in UK will require a risk assessment before placing any temporary foundations or surcharge on these assets (**Tables 1 and 2**).

4.4 Telecommunications cables, broadband and fibre optics

Damage to telecommunication and TV cables may require expensive repairs and can cause considerable disruption to those relying on the system, especially emergency or essential services. The risks of direct personal injury are normally low, but claims for consequential losses may be substantial.

PIPE TYPE	Diameter (mm)	Allowable Increase in Strain (‰)		Rotation (deg.)
		Tension	Compression	
Brick Sewer (red / yellow / blue brick)	N/A	500	25% of the allowable stress	N/A
Cast Iron Lead-yarn joints	N/A	100	1200	0.1
Ductile Iron (Lead-yarn gasket joints)	N/A	500	700	0.5
Ductile Iron (Rubber gasket joints)	N/A	500	700	2.0
Steel	N/A	450	450	1.5
Vitrified Clay	<125	80	400	0.5
	>125	80	400	See Table 2
Concrete (unreinforced)	<225	20	400	0.5
	225 – 750	40	400	See Table 2
	>750	60	400	

Table 1.
Assessment criteria for existing Thames Water pipeline and sewer assets (reproduced from Ref. [20]).

Diameter (mm)	Rotation (deg.)
< 375	2.0
375 – 750	1.0
750 – 1400	0.5
> 1400	0.3

Table 2.
Maximum rotation for vitrified clay and concrete pipes.

It is important to ensure that surcharge loading from temporary foundation on buried cables is limited.

In addition to the above utility services, the presence of other pipes and cables should be anticipated. These include fuel oil pipes at States housing developments and private/security electricity and telecommunications cables. Risk assessment will be required on a case by case basis to suit to owners' requirement.

4.5 Underground or above ground railway or cable tunnels

Any movement of tunnels if they are underground or above ground (cut and cover) is very critical for the performance and therefore the asset owners will have very stringent requirements for the movement temporary works plant around these structures. Generally clearances need to be maintained between the plant and the assets and this will vary depending on the depth of the asset below the ground. No plant can operate within these zones while constructing the bridges. An example of the Cross Rail exclusion zone in London [20] is shown in **Figure 3**.

Figure 3.
Exclusion zone limits for crossrail underground lines (figure reproduced from Ref. [21]).

4.6 Party walls or other grade listed buildings

These include third party buildings, existing walls of private owners, and grade listed buildings. Generally, if the temporary works for bridge construction works are carried out by the contractor near these assets, he will have to ensure that the damage to either Partywalls or nearby listed grade buildings is minimal. In UK, there are party wall agreements on movements and crackwidths as per Boscardin guidelines [22].

In order to manage the risks from bridge construction operations, it is necessary to prepare a carefully coordinated construction method statement with all the supply chain involved and which includes the sequence, hazard identification, and a risk assessment. A management procedure is required to manage the residual risks on site and contingency measures to mitigate or control the risks.

5. Dealing with hazards: construction method statements, and risk assessments

Catastrophic events in construction are real issues which require proper consideration by all stakeholders, led by directors and senior staff.

These potentially catastrophic events are sometimes referred to as ‘Top Events’. It is appreciated that they can have a disastrous impact on a company’s reputation and well-being and upon society. The process of examining the risk of a catastrophic event requires that a ‘safety case’ is prepared, based upon a safety risk assessment.

CIRIA and HSE UK reports [23, 24] have looked at the risks of ‘Catastrophic Events’ in the UK construction industry and summarised its findings in the report. This report identified the types of events, reason for occurrence, and control measures and included:

- The types of catastrophic event which have occurred or which might occur during construction
- The reasons for occurrence when there have been (or could have been) catastrophic events during construction, including an examination of the underlying factors
- The controls which would contribute to an avoidance of a catastrophic event
- Where the UK construction industry could improve

It was clear that there have been Catastrophic Events with major consequences. Their importance was recognised by the industry, although it is considered that in their day-to-day work few people realised the severity of what might happen if things went seriously wrong.

The key issues proposed in this reports are as follows:

- Issue 1: The industry should recognise that catastrophic events need further attention
- Issue 2: Corporate risk management systems should be improved
- Issue 3: Knowledge, skills and experience of safety risk management should be raised
- Issue 4: Communication and interface management should be improved
- Issue 5: Competence is key
- Issue 6: Effective management of temporary works is crucial to success
- Issue 7: Independent reviews should be employed
- Issue 8: The industry should learn from experience.

Method statements and drawings need to fully detail all aspects of the works. All safety critical items and hold points/permits to load should be identified.

- Low likelihood/high severity items are to be given careful consideration with appropriate monitoring at all levels.
- More invasive questioning and understanding of sub-contractor’s Method Statements and Risk Assessments

5.1 Risk assessments

The terms used in risk assessment vary considerably in the literature. In this report, a harm is defined as an adverse effect on a person. It might be, for example,

a serious illness or injury, but effects on well-being are also taken into account. A hazard is a potential cause of harm to a person, for example a faulty staircase. A hazardous situation exists when a person is exposed to a hazard, for example by using a faulty staircase. The risk associated with a hazard is a function of (a) the likelihood of the hazard causing harm and (b) the severity of the harms or their consequences.

Safe systems of work shall comprise:

- Assess the risks.
- Plan the work - obtain all information relating to the work carried out.
- Check that the plans are accurate.
- Carry out the work in a safe manner.
- At all stages THINK and REVIEW.

The assessment of risk should be considered at all stages of the work, from planning through to final reinstatement. This may be accomplished by the use of formal risk assessments coupled, where necessary, with a work permit system. Risk assessment should include all related work activities and identify training and competency needs as well as the level of supervision required for the risks involved [10].

6. Need for construction: pre trials, tests and other monitoring

Sometimes, it is necessary to carry out trials on site before the bridge structure construction begins. This may take include some mock structures to be built on site to validate all the assumptions in the design. The purpose is to ensure that the sequence of construction can be carried out safely. Examples include test loading on the trestles or surcharge loading, and monitoring of displacements etc. and mock assembly in case of structural steel joints. Eurocodes can be implemented to optimise the temporary works design if suitable testing is carried out.

In the following section, we give more recent literature survey of failures in the temporary works that involved public and third party asset owners and give additional examples of how we have managed the risks (as explained in Sections 3 and 4) during the construction for the projects in which Sir Robert McAlpine and their joint partners were involved. Our recent project also demonstrate how we have used trials on site for back analysis and how monitoring was carried out to improve the confidence in predictions and stakeholder assurances.

7. Survey of lessons learnt from recent failures

7.1 Barton Bridge, Eccles, Feb 1959 & 2016

In the UK the First Barton bridge collapsed in 1959 whilst erecting 4 No, 200 ton steel girders, 80 ft. above the ground [3, 14]. The supporting scaffolding collapsed bringing down the girders and killing 4 men. Sixty men, that would normally have been on the girders, were lining up for their pay at the time. Ironically after 57 years, at the same location in 2016, temporary lifting system failed collapsing a major chunk of the new bridge span across the river as shown in **Figure 4a** and **b**.

7.2 GE 19 bridge on East London Line

GE19 is an 84 m long, single span Warren truss girder bridge on a 3.3% gradient from East to West with a bridge weighed 1300 tonnes post launch and had an In-situ deck on ‘Omnia’ permanent formwork [25]. Minimum clearance of 650 mm to overhead lines. It was found that the bridge had moved longitudinally 38 mm out of position post launch. This necessitated corrective plan jacking that had not been envisaged during pre-planning. At approximately 19:15 hours on 28th May 2008, an hour after work had stopped, the site security guard heard three loud bangs. The Bridge deck had dropped by approx. 200 mm resulting in damage to scaffolding and bearings. Five planks fell onto the live rail below the bridge, ponded water onto the overhead lines and track and the track had to be closed. There were no injuries to any members of the public or employees. PTFE material had been placed on a slope,

Figure 4.
(a) Barton bridge scaffold collapse (1959) and (b) lifting failure of a modern bridge (2016) (figures reproduced from Refs. [3, 14]).

Figure 5.
Temporary bearing.

the vertical load of the bridge generated a horizontal force component. The presence of a second slip plane allowed the wedge of packing between the PTFE surface and bearing surface to be ejected as in **Figure 5**. A similar incident did not occur at the West abutment because the permanent bearings were of a fixed type and hence did not fail (see **Figure 6**).

7.3 Motorway bridge temporary works collapse Colombian Bridge

In March 2019 the temporary works of a motorway bridge near Ancona, Italy, shown in **Figure 7** [26] failed with the immediate collapse of the bridge deck onto cars passing below sadly killing two members of the public [14, 26]. The motorway was subsequently closed. An example of both the human and financial consequences of errors in construction. A design error was blamed for the collapse of the bridge.

7.4 Grayston Drive collapse (Johannesburg)

Two people were killed and 19 injured when the formwork supporting the under-construction bridge collapsed unexpectedly in 2016 shown in **Figure 8** [27]. The initial inquiry showed that some of the site inspection registers for the period just before the collapse were not available [27].

Figure 6.
Bridge launch over railway (reproduced from Ref. [25]).

Figure 7.
Colombian bridge failure (reproduced from Ref. [26]).

7.5 Norway Bridge Collapse 2018

A Hydrogen embrittlement crack has been identified as causing the failure of one of the cable anchor bolts of the Norway Halogaland Bridge shortly after installation shown in **Figure 9** [28]. The initial enquiry showed that that the root cause of the cracking is the hydrogen exposure of the bolts. It is not known if the bolts were exposed to Hydrogen during manufacturing, transportation or at the site.

7.6 Injaka Bridge, South Africa 1998

Incorrect positioning of temporary bearings during incremental launching was identified as the primary cause of the fatal 1998 Injaka Bridge collapse in South Africa [29].

Inexperienced design and construction staff, poor construction quality control and a failure to react to a ‘clear warning that all was not well’ with the structure, led to the disaster. At 300 m long, 14 m wide and up to 37 m above the river bed, Injaka Bridge was a major structure and the consultant and contractor had extensive experience with such incrementally launched post-tensioned structures. The collapse occurred after the contractor had slid out five of the 20, 15 m long sections of the 3 m deep box section deck. The sixth segment was being jacked as the structure collapsed. At that point the concrete deck extended 24.4 m beyond pier 2 with the leading edge of the 27 m long launching nose projecting 7.1 m beyond pier 3.

Figure 8.
Collapse of temporary supports 2018 (figure reproduced from Ref. [27]).

Figure 9.
Hydrogen cracking in bolts (figure reproduced from Ref. [28]).

The primary cause of the collapse was found to have been the positioning of temporary bearings on which the deck structure slid out during construction as shown in **Figure 10**.

Figure 10.
Failure of a multi span bridge (figure reproduced from Ref. [29]).

Figure 11.
Florida Bridge collapse, 2019 (reproduced from Ref. [25]).

Figure 12.
Node 11 failure (reproduced from Ref. [30]).

7.7 Florida Bridge collapse 2019

The initial investigation showed that where the tow truss members meet, at joint 11, there was overestimation of the capacity and under estimation of the loads as shown in **Figures 11** and **12** [25, 30, 31].

8. Case studies of mitigation

Examples of case studies from our recent project experience published by the author in various reports (internal and external) are outlined below. These case studies show how the temporary works design should address the stake holder's expectations in managing risks. Five case histories discussed here show the importance of design, checking, and monitoring and back analysis.

8.1 Case Study 1: Arsenal bridges, London, UK

An example of assets affected by the bridge launch—temporary trestle foundations next to network rail assets, and piling plant and movement of construction plant on the sensitive underground railway tunnels (**Figure 13**).

Assessments were carried out to ensure that the settlements, and movements and surcharge loadings were managed. Monitoring was carried out to gain confidence in the predictions and to ensure that mitigation measures can be placed (**Figure 14**).

8.2 Case Study 2: Gogarburn Bridge, Scotland

The road bridge is on the new spur off the A8 into the new headquarters for the Royal Bank of Scotland. The bridge deck was being erected on temporary trestles, located on either side of the road and in the central reservation. The box girders will be lifted into place in half span sections, each weighing approximately 73 T. Secondary beams span between the box girders to support the deck. The deck was cast on 'omnia plank' permanent formwork. The sequence is depicted in **Figure 15** and the temporary trestles that required design for impact loads are shown in **Figure 16**.

Figure 13.
Bridge launch.

Figure 14.
General arrangement plan and section showing trestles, UG services, tunnels, and network rail.

Figure 15.
Erection sequence of the bridge, trestle.

The arch was being erected in three sections. Each of the two outer pieces was supported by the bearing plate at one end and a trestle on top of the deck at the other.

Finally the centre section was dropped in to complete the arch. The arch was welded together, 25 m above the road. Ten tension rods support the bridge in the centre.

Figure 16.
Temporary trestles near live highway.

8.3 Case Study 3: Motorway M74, UK, Bridge Launch from the temporary trestles

As a part of the M74 completion project in Glasgow, Sir Robert McAlpine Design group, working closely with the site based Joint Venture, has been responsible for many elements of design and checking for the West section of the work [31]. Included in the West section is a 1350 m length of an elevated structure formed of trapezoidal steel girders with in-situ concrete deck and parapets. This section passes over three sections of the overground line which are a part of Network Rails assets; the Paisley Line, the Cook Street Link, and the West Coast Mainline. Parts are also close to a section of the Strathclyde Passenger Transport (SPT) underground line.

With the exception of the West Coast Mainline section, the steel sections were lifted in place using mobile cranes. These sections were supported on temporary trestles whilst they were welded. The section of superstructure spanning over a 65 m wide cutting containing the West Coast mainline was installed by launching the bridge as shown in **Figure 17a**. A 235 m long, 4200 tonne section was formed to the West of the cutting, complete with a section of in situ deck and parapet, and then launched a total of 166 m during a series of short night-time possessions. The

Figure 17.
(a) Launch of the bridge over the live highway and railway, (b) aerial photo of the launch (extract from Ref. [31]).

cantilever distance to the first temporary trestle support beyond the cutting is 89 m. In order to ensure safety to both Public and the existing network rail assets, the temporary works design included the following and was developed collaboratively with the supply chain partners in the joint venture project.

- i. Assessment of the ground was necessary for designing working platforms for all the plant and craneage required for the steel lifts. Outrigger loads from the cranes were up to 4200 kN, with 6000 kN of superlift loadings.
- ii. Assessing the effect of the applied loads on the Scotland Public Transport (SPT) infrastructure.
- iii. Design of frames and anchored rock netting to protect Network Rail assets during piling and pier construction operations adjacent to the railway cutting.
- iv. Design of temporary support trestles and their support piles to take the vertical and horizontal loads applied during the bridge launch operations. The calculated vertical loads were up to 18,000 kN on a single temporary leg under the worst case loading. The design of the trestles evolved as work proceeded as exploratory works were carried out in advance of the piling and services or obstructions were encountered:
One pair of temporary trestle bases was cast adjacent to a Network Rail Retaining and within a main road. The design had to take account of the need to avoid unacceptable movements to the nearby masonry retaining wall.
- v. The six 1200 mm piles to each base had to be installed in locations which avoided impacting on three gas mains, electric cables and a brick sewer.
- vi. Design of additional support walls to take the bridge launch loads on the pile caps under the temporary loading conditions.

Working collaboratively with our supply chain partners, loads on each of the trestles for the launching operations were derived and were used in the analysis of temporary trestles. Trestles foundations included piles where space was restricted and pad foundation where space was not restricted. The analysis for various combinations of normal loads and accidental loads were carried out in ANSYS and typical results from the analysis were summarised in **Table 3**. These were used for monitoring the loads and the movements of the foundations using targets on trestles and foundations.

8.4 Case Study 4: construction of a segmental arch bridge over a railway, Dobwalls Bypass, UK

This case study, published in detail in Ref. 32 by the author, shows how the temporary works were managed by the project team in order to ensure that there was no repeat of 'Gerrards cross failure'. The failure at Gerrards Cross, reported in 2005, demonstrated the importance of controlling the backfill and carefully controlling and monitoring the deflections of the arch bridges during construction. The 87 m long arch unit spanning 15.5 m with a rise of 5.6 m was built on the monolithic principle, which means it acts as one single structure. The radius of the curved track underneath is 500 m, which proved a challenging aspect of the project.

Item	Trestle 1	Trestle 2	Trestle 3
Vertical settlement	8 mm	8 mm	0 mm (under lifting strut). 6 mm elsewhere.
Anticipated lateral movement (at pile cap). Parallel to launch	10 mm	10 mm	9 mm
Maximum lateral movement (worst case soils) at pile cap. Parallel to launch	—	—	21 mm
Maximum lateral movement (worst case) at pile cap. Perpendicular to NR retaining wall	—	—	14 mm
Worst case lateral movement at network rail wall for worst case soils and bridge being pulled back (westwards)	—	—	3 to 5 mm
Total maximum vertical load on support	+5000 kN	34,500 kN	20,000 kN
Total minimum vertical load on support	−4350 kN	—	—

Table 3.
Amber trigger levels for movements and loads on trestles.

The structure was in the form of a short tunnel, a proprietary system by Asset International, comprising pre-cast concrete arch segments springing from an RC slab and upstand, supported on piled foundations. The arch was formed from pre-cast concrete elements using a proprietary arch system with an elliptical cross-section. The two ends of the tunnel consisted of portal sections which are bevelled to follow the slope of the new embankment for the realigned section of road. Since the bridge structure is made up of arch-shaped pre-cast elements, the elements at the ends of the tunnel are cut-off, and no longer form a full arch. These truncated elements, therefore, are connected to each other by means of cast-in-situ reinforced concrete collars to form a monolithic reinforced concrete shell. This monolithic shell includes the two outermost full arch rings which allow all loads acting on one side of the bevel to be transferred to the other side and to the foundations. Working collaboratively with the project team, we have developed a safe system of construction as described below.

In order to predict the behaviour of the tunnels during construction and to advise the construction teams on the methodology, it was necessary to model each stage both in 2D and 3D models. The construction sequence was represented in the analysis by a total of 15 stages as shown in **Figure 18**. Full details of the FE model, assumptions, approach, and the sequence including sensitivity analyses were published in a separate paper Ref. [32] by the author(s). Only extracts from Ref. [32] are presented in this paper.

The construction phases are summarised below.

Phase 1: Establish initial conditions for existing ground and railway embankment;

Phase 2: Place new fill up to the level of the existing railway embankment, and install piles and pile caps;

Phase 3: Construct arch;

Phase 4: Place fill for the new road embankment away from the arch structure up to 75% of the overall height of the arch segments;

Phases 5–13: Place backfill against the arch with a maximum differential between sides of 600 mm, with compaction load of 11.5 kN/m² on the surface of the fill;

Figure 18.

Construction sequence model analysed in the finite difference software (FLAC).

Phase 14: Place final layer of fill over the arch, with compaction load of 11.5 kN/m^2 over the full width of the model on the top surface;

Phase 15: Remove compaction load of 11.5 kN/m^2 on top surface of the model.

It was important to carry out sensitivity studies with respect to soil stiffness, backfill characteristics, interface stiffness, and initial conditions of the arch to establish lower and upper bounds of movements. These sensitivity studies helped us to develop a safe and robust scheme of backfilling sequence and helped us to set new trigger limits for safe construction. Closed-form solutions predict the movements for a fully backfilled scenario however they will not predict the movements for unsymmetrical backfilling on either side of the arches and therefore numerical models in 2D and 3D will give insight to real behaviour. The models developed here in 2D and 3D, therefore, gave insight into the development of the movement throughout the construction process. The power of the modelling is demonstrated by comparing the analytical results with observations on site. The site observations matched well with the numerical predictions from 2D and 3D models as shown in **Figure 19**.

Figure 19.
Movement of the crown of the arch while backfilling sequence is progressed to build the highway bridge over the railway.

8.5 Case Study 5: construction of a hanging building from the truss over railway, Bull Ring, Birmingham, UK

The Bull Ring Redevelopment in Birmingham consisted of demolishing the existing 1960s concrete shopping centre and replacing it with a new one. The northern part of this new complex lies directly above the New Street South railway tunnels, which carry the main lines to London and the West Country through them. During the redevelopment work there was the potential to affect the railway tunnels at various stages of construction.

To maximise the available retail space at the northern end of the development, 2 No. hanging structures were to be constructed to extend the development over the Northern Arm road, with the pedestrian footbridge described above providing the ‘sandwich’ between the hanging structures. To support these hanging structures a structural steelwork bowstring truss is positioned at either end of each structure, spanning across the Northern Arm to carry all other intermediate steelwork, reinforced concrete floor slabs, roof, cladding, etc.

The western hanging structure is supported by trusses T1 and T2, which are supported at their northern end by double columns supported off a reinforced concrete pile cap founded on a cluster of mini piles constructed within the basement of the Rotunda. At its southern end these trusses are again supported by a twin steel column section founded behind the contiguous piled wall and thus forming part of the main development structural frame. The brick arch railway tunnels are not continuous for the full length of the Northern Arm. At its western end the brick arch tunnels give way to a reinforced concrete road bridge, built 1961–1962. This road bridge continues in a westerly direction towards the New Street station junction, noting that immediately northwest of truss T1 the road bridge deck slab is discontinuous with an open section of railway exposed and only protected by a 1.8 m high concrete parapet wall constructed around the opening. Working collaboratively with our supply chain partners, we have developed a safe erection methodology as described below.

Once the fabrication location was established, crane sizes and locations were firmed up, thus allowing a detailed crane analysis to be carried out to produce theoretical outrigger loads, including any redistribution of loads resulting from any crane slewing, jibbing in/out, etc. during the lifts. The Railtrack structures were then assessed under these loads, with feedback to the site team accordingly if the

Figure 20.
Erection of 140 T truss over the railway with a soil cover of 3 m.

structures were likely to be overstressed, with recommendations to relocate cranes, increase outrigger mats, distribute outrigger loads onto twin mats, etc.

Once craneage locations were finalised, craneage layouts were firmed up, structural checks were completed, scaffold layouts finalised and method statements produced. Craneage used for the fabrication of the trusses and associated erection of the temporary works support scaffold varied from truss to truss, but for each truss one of the cranes used for the tandem lift of that truss was utilised as the service crane for the above works. Therefore, for trusses T1 and T2 a 400 T crane was used with an 800 T crane used for trusses T3 and T4. As part of the checks undertaken during the initial piling works to establish constraints for the piling plant, its location, and associated excavations, a considerable amount of design checks using finite element analysis had been undertaken on the brick arch tunnels. To satisfy railtrack/network rail requirements the following monitoring equipment had been installed into the tunnels before works started and included, electro level beam surveys, vibration sensors, tilt meters, and tape extensometers.

Temporary works design checks undertaken were based on theoretical outrigger loads prepared by the crane manufacturers following assessment of the different lifts by the different cranes. On the basis that these outrigger loads did not cause distress to the structures below, it was essential that outrigger loads were checked to ensure that the theoretical maximum loads were not exceeded. Hence use was made of the crane digital outrigger load readout indicator to monitor these loads. The details of the finite element analysis and the assumptions are presented in a full paper in Ref. [33] and only extracts from Ref. 33 are presented in this paper.

During each lift and at various times during the fabrication of the trusses, the tunnel monitoring system PC was attended full time by one of our site engineers to observe any changes, notably deformation movements. A site engineer would also monitor the full time the crane outrigger load indicator during major lifts.

During the crane lifts and during the fabrication of the trusses no discernible deformation to the tunnel was recorded—noting that, when the tunnel monitoring system PC was not being observed full time, the system did activate a telephone alarm once a deformation of 7 mm occurred.

The crane outrigger loads, as observed on the outrigger load indicator, were generally well inside the theoretical figures. On truss T1 lift the 800 T crane on the bridge beams had a maximum outrigger load of 74 T against a predicted value of 106 T. On truss T2 lift the 800 T crane on the tunnel central wall had a maximum outrigger load of 92 T against a predicted value of 98 T.

On the same lift the 400 T crane on the transfer beams spanning across the beams had a maximum outrigger load of 99 T against a predicted value of 120 T. **Figure 20** shows the truss T1 over the tunnels.

Immediately following each lift on the first available track possession/isolation a visual inspection of the tunnel was undertaken and no visible signs of distress and

Figure 21.
Non-linear finite element analysis of soil structure interaction for the loads from erection of the truss over the live railway.

North Tunnel					
	Non-linear analysis		Linear analysis		Observed
	Lateral	Vertical	Lateral	Vertical	Vertical
A	-0.35	-0.68	-0.24	-0.65	-0.7
D	0.05	-1.25	0.29	-1.20	-1.01
G	0.46	-0.64	0.52	-0.66	-0.7

North Tunnel

South Tunnel

South Tunnel					
I	-0.52	-0.35	-0.39	-0.47	-0.9
K	-0.18	-0.98	-0.04	-1.13	-1.3
M	-0.24	-0.37	0.32	-0.49	-0.4

Table 4.
Measured and observed movements of the arch for 70 T rig on the tunnels.

movement were ever observed. Tape extensometer checks were also undertaken and again no significant movement attributable to the crane lifts was ever recorded.

Finite element models were developed in ANSYS (commercially available software) in view of the above. In particular, two-dimensional linear and non-linear global analysis and a three-dimensional non-linear local analysis were carried out. **Figure 21** shows the finite element model. Planar elements were used for two-dimensional analysis whereas solid elements with no tension were used in the three-dimensional analysis.

ANSYS results were calibrated against site observations by making test trails on-site by surcharging pile plant loading on the tunnels and results are shown in **Table 4**.

Analysis method, assumptions, and further details of the finite elements are presented in the detailed paper by the authors in CIRIA report on tunnelling [33]. Only extracts are presented in this paper.

9. Summary

Temporary works are an integral part of the safe construction of bridges and should be designed by competent bodies. Independent checks, and balances are to be in place to ensure that the public and asset owners are protected by the construction method, construction sequence of bridges. This paper summarises more recent failures and draws the conclusion that some of the best practice guidelines as developed in the UK can be adopted outside the UK. Early engagement by the contractor and communication to all the parties involved play a significant role in the safe delivery of the works at the site. Every aspect of the temporary work should be looked into in detail with reasonable margins of safety. Regular monitoring during the construction to ensure that the early warnings are not exceeded and are vital to verify the performance is as predicted and that no unsafe condition is being approached.

Acknowledgements

The author is thankful to Parsons Brinckerhoff for supply of the data and Asset International for sharing site measurements and photos. The author is thankful to our JV team of M74 and Paul Doughty of Sir Robert McAlpine Design Group for sharing data and photographs.

Conflict of interest

The authors declare no conflict of interest.

IntechOpen

IntechOpen

Author details

Ganga Kasi V. Prakhya
Sir Robert McAlpine Ltd, Eaton Court, United Kingdom

*Address all correspondence to: g.prakhya@srm.com

IntechOpen

© 2020 The Author(s). Licensee IntechOpen. This chapter is distributed under the terms of the Creative Commons Attribution License (<http://creativecommons.org/licenses/by/3.0>), which permits unrestricted use, distribution, and reproduction in any medium, provided the original work is properly cited.

References

- [1] Grant M, Pallet P. Temporary Works: Principles of Design and Construction. London, UK: ICE Publishing; 2012
- [2] Bridle R, Sims F. The effect of bridge failures on UK technical policy and practice. Proceedings of the Institution of Civil Engineers: Engineering History and Heritage. 2009;**162**(1):39-49
- [3] Smith D. Bridge failures. Proceedings of the Institution of Civil Engineers: Part. 1976;**1**(60):367-382
- [4] HSE (Health and Safety Executive). Final Report of the Advisory Committee on Falsework. The Bragg Report. London, UK: HMSO; 1976
- [5] BSI. BS 5975: 2008+A1:2011. Code of Practice for Falsework. London, UK: BSI Publication; 1982
- [6] BS EN 12811-1. Temporary Works Equipment. Scaffolds. Performance Requirements and General Design; 2003
- [7] BSI. BS EN 12812: 2008. Falsework. Performance Requirements and General Design. London, UK: BSI; 2008
- [8] Temporary works forum. Toolkits for Design. Available from: <https://www.twforum.org.uk/home> [Accessed: January 2020]
- [9] Hewlett W, Jones A, Marchand S, Bell B. Re-visiting Bragg to keep UK's temporary works safe under EuroNorms. Proceedings of the ICE: Forensic Engineering. 2014;**167**(2):58-68
- [10] Structural Safety Reports on Temp Works. Available from: <https://www.structural-safety.org/> [Accessed: January 2020]
- [11] Soane A. Temporary works toolkit. Part 5: Temporary works failures—What are the common causes? The Structural Engineer. 2017;**95**(1):24-26
- [12] Bennion C. Temporary works associated with precast concrete bridge beam construction. In: Proceedings of the Institution of Structural Engineers, London, May 20. 1987. pp. 37-41
- [13] André J, Beale R, Baptista AM. A Survey of Failures of Bridge Falsework Systems Since 1970. ICE—Forensic Engineering, Institute of Civil Engineers; 2012. 4/12
- [14] Wikipedia—Bridge Failures. Available from: https://en.wikipedia.org/wiki/List_of_bridge_failures [Accessed: January 2020]
- [15] Broughton K, Gill J. Bridge Installation Techniques. London: ICE Publication; 2015
- [16] Khan MA. Accelerated Bridge Construction, Best Practices and Techniques. Elsevier Publication; 2015. ISBN: 97820-12-407224-4
- [17] BSI. BS EN 1990: 2002 + A1: 2005. Eurocode—Basis of Structural Design (Incorporating Corrigendum December 2008 and April 2010). London: BSI; 2005
- [18] BSI. BS EN 1991-1-6: 2005. Eurocode 1. Actions on Structures. General Actions. Actions during Execution. London, UK: BSI; 2005
- [19] HSE Publication. Construction (Design and Management) Regulations (CDM). London: HSE Publication; 2015. Available from: <https://www.hse.gov.uk/construction/cdm/2015/index.htm> [Accessed: January 2020]
- [20] Thames Water Publication. Guidance on Piling, Heavy Loads, Excavation, Tunneling and Dewatering. Available from: <https://developers.thameswater.co.uk/Developing-a-large-site/Planning-your-development/>

- Working-near-or-diverting-our-pipes
[Accessed: January 2020] 2019;**172**(3):226-240. DOI: 10.1680/jbren.18.00027
- [21] Cross rail Information for developers. March 2019. Available from: <http://www.crossrail.co.uk/route/safeguarding/?folder=/10/856/asset/6043> [Accessed: January 2020]
- [22] Gaba A, Hardy S, Doughty L, Powrie W, Selemetas D. Guidance on embedded retaining wall design. In: CIRIA Technical Report C 760D. London: CIRIA Publications; 2017
- [23] Gilbertson A, Kappia L, Boshier L, Gibb A. Guidance on catastrophic events in construction. In: CIRIA Publication C 699. London, UK; 2011
- [24] Preventing catastrophic events in construction. In: Health & Safety Executive Research Report, HSE RR 834. London: HSE Publication; 2011
- [25] Bridge Design & Engineering. 2019;**95**(2nd Q):6
- [26] Bridge Design & Engineering. 2018;**91**:6
- [27] Bridge Design & Engineering. 2018;**92**:6-7
- [28] Bridge Design & Engineering. 2019;**97**:7
- [29] New Civil Engineer. UK: Publication of Inst of Civil Engineers; August 2002
- [30] New Civil Engineer. UK: Publication of Inst of Civil Engineers; January 2020:38-39
- [31] New Civil Engineer. Supplement on M74. UK: Publication of Inst of Civil Engineers; June 2011
- [32] Prakhya G, Hopkin I, Hansford B. Construction of a concrete segmental arch bridge over a railway. Proceedings of the Institution of Civil Engineers—Bridge Engineering.
- [33] McKibbins L, Elmer R, Roberts K. Tunnels: Inspection, assessment and maintenance. In: CIRIA Technical Report C671. London: CIRIA Publication; January 2010