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Blast Studies on Bridges – A State-of-the-Art Review

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ABSTRACT: Blast events can lead to critical injuries along with heavy casualties in addition to disastrous structural failure, thereby giving rise to detrimental economic and social impacts, both domestically as well as internationally. Bridges, an integral component of a vibrant transportation network, are not only vulnerable to accidental incidents, but also susceptible to deliberate attacks. Blast engineering regarding civil infrastructure, although very crucial in this modern era, has only received rapidly evolving interest in recent time, and many areas in this field, including most of the aspects regarding bridges, demand intensive attention. A state-of-the-art review of the previous blast research projects concerning bridges is presented in this paper. The simplistic and advanced theoretical and practical investigation strategies employed are explained, and the distinct assumptions and justifications adopted are highlighted. The blast consequences observed on the individual bridge elements considered are described together with the subsequent impact on the overall integrity of the corresponding bridge systems. The mitigation tactics proposed are discussed, with major emphasis on the functions delivered as well as the inherent effectiveness. Further enquiries to be carried out for the future development of this engineering discipline are also identified.

Keywords: Bridges, blast investigations, blast effects, blast retrofitting, blast design

1 INTRODUCTION

Blast-related issues have become extremely important for civil infrastructure, and no longer confined to petrochemical and military facilities, as proven from the history of this modern era. Blast incidents can happen under accidental or intentional circumstances, which are both unpredictable since human behaviour is involved. These blast events could cause critical injuries along with heavy casualties in addition to disastrous structural failure, thereby giving rise to detrimental economic and social impacts, both domestically as well as internationally.

Bridges are an integral component of a vibrant transportation system. The intrinsic function of a bridge is to afford convenient and efficient access to destinations separated by geographical terrains and artificial obstacles. Apart from that, a high-profile bridge constructed with cutting-edge technology as well as remarkable aesthetic appeal also serves as a regional and national landmark (e.g. Sydney Harbour Bridge, Australia). Thus, bridges are amongst the attractive targets for deliberate attacks with notorious intentions. Additionally, bridges are also susceptible to accidental explosions, owing to the diverse categories of users.

Blast engineering regarding civil infrastructure has only received rapidly evolving interest in recent time. More research is being conducted to advance the theoretical and experimental investigation technology, as well as to enhance the level of understanding of the blast implications on multistorey buildings, bridges, industrial structures and public facilities. Blast solutions, which consist of retrofitting options, for existing provisions; design guidelines, for future services; and deterrent measures, which aim to hinder blast occurrence and lower blast severity, are under constant development. Nevertheless, many areas in this field, including most of the aspects regarding bridges, still demand intensive attention.

A state-of-the-art review of the previous blast research projects concerning bridges is presented in this paper. The simplistic and advanced theoretical and practical investigation strategies employed are explained, and the distinct assumptions and justifications adopted are highlighted. The blast consequences observed on the individual bridge elements considered are described together with the subsequent impact on the overall integrity of the corresponding bridge systems. The mitigation tactics proposed are discussed, with major emphasis on the functions delivered as well as the inherent effectiveness. Further enquiries to be carried out for the future development of this engineering field are also identified.

Blast studies broadly differ in terms of objective, problem encountered, methodology, expertise and technology required, as well as budgetary restriction and schedule constraint. Consequently, the research findings derived under the discretion of separate investigations, which are either valid exclusively to the specific issues under consideration or justifiable generically to much broader aspects, are not necessarily subject to direct comparison and should always be interpreted independently and perceived as either mutually reinforcing or disagreeing to a certain extent, while absolute generalisation should, however, never be overly enforced.

2 BLAST INVESTIGATION TECHNIQUES

Blast effects on bridges can be generated through the coupled methods, i.e. fluid-structure interaction (FSI) simulation and the experimental approach, whereby the interaction between the blast wave and the structure is taken into account, and the uncoupled methods, whereby blast loads are estimated separately before being imposed either statically or dynamically on the targets.

A summary of the blast studies concerning bridges is given in Table 1.

2.1 *Coupled methods*

In the FSI procedure, the structure is modelled as a multi-degree-of-freedom (MDOF) system with Lagrangian elements, while the air and the explosive are explicitly defined using Eulerian elements. The recent development of the arbitrary Lagrangian-Eulerian (ALE) formulation, which permits multiple substances within a single element, has resolved the shortcomings of the Lagrangian approach and the Eulerian approach in computational fluid dynamics (CFD).

In blast trials, only the features of interest are usually presented (the prototypes are often scaled down from the original sizes), and this is acceptable as long as sufficient details are provided to fully capture the desired effects and attain realistic boundary conditions from the reaction frames (Ray 2006). The publication of sensitive data is usually restricted on account of security concerns.

2.1.1 *Numerical simulation*

Cimo (2007) created a two-span simply-supported composite girder bridge (a two-dimensional model), using ANSYS AUTODYN. A faster computation speed was attained by remapping the blast pressures obtained from the one-dimensional analysis onto the two-dimensional space. Unfortunately, the numerical outputs were only recorded up to the instant when the computation was prematurely terminated because of an unexpected interface-overlapping error.

Son (2008) modelled a steel orthotropic deck and a composite girder with MSC.Dytran. Different levels of axial loads were assigned to replicate the distinct conditions in the earth-anchored suspension bridge, the cable-stayed bridge and the self-anchored suspension bridge. The nodal displacements recorded away from the blast impact were taken to represent the global behaviour of the deck and the girder. The subsequent heating effect was ignored in the study.

With the aid of actual design drawings, Deng and Jin (2009) simulated a cable-stayed bridge with ANSYS AUTODYN. The blast pressures derived from the one-dimensional analysis were remapped into the final three-dimensional domain, and fine meshes were created only for the air in the immediate vicinity of the structure, as an attempt to reduce the processing duration.

A three-span simply-supported composite girder bridge model was developed by Yi (2009), using LS-DYNA. Without modelling the explosive, the blast pressures estimated from ConWep were applied directly onto the receiving faces of the ambient elements. The desired mesh sizes were selected by reproducing the test data reported by Magnusson and Hallgren (2004).

The study undertaken by Son and Lee (2011), which was related to an existing cable-stayed bridge, involved a concrete-filled composite pylon and a hollow steel pylon. The FSI simulations were implemented in MD Nastran SOL700. Only the tower and the steel orthotropic deck were extracted for the blast simulation, while the absence of the cables was compensated by introducing axial compressive forces and allowing only horizontal translation at the longitudinal ends. A total of four Euler subdivisions were formed, in order to govern the flow movement activated upon structural failure.

2.1.2 *Experimental procedure*

The investigation programme devised by Ray (2006) was comprised of a series of ½-scale blast experiments and parallel numerical simulations involving

Table 1. Blast studies involving bridges.

Literature	Investigation technique				Bridge system	Bridge component
	Coupled		Uncoupled			
	Test	FSI	Static	Dynamic		
Winget et al. (2005)				•	Simply-supported	Concrete pier, prestressed girder
Ray (2006)	•				Suspension	Steel tower
Cimo (2007)		•			Simply-supported	Composite girder
Mahoney (2007)			•		Simply-supported	Prestressed & composite & truss girders
Suthar (2007)			•		Suspension	Steel truss girder
Zheng (2007)				•	Simply-supported	Concrete pier
Fujikura et al. (2008)	•			•	Simply-supported	Composite pier
Islam & Yazdani (2008)			•		Simply-supported	Concrete pier, prestressed girder
Matthews (2008)				•	Simply-supported	Prestressed girder
Son (2008)		•		•	Cable-supported	Steel orthotropic deck, composite girder
Deng & Jin (2009)		•			Cable-stayed	Steel truss girder
Tokal-Ahmed (2009)				•	Simply-supported	Prestressed girder, concrete pier
Yan & Chang (2009)				•	Cable-stayed	Steel girder & cable, composite tower
Yi (2009)		•			Simply-supported	Composite girder, concrete pier
Zhou (2009)			•	•	Simply-supported	Concrete pier, composite girder
Tang & Hao (2010)				•	Cable-stayed	Hollow tower & pier, box girder
Fujikura & Bruneau (2011)	•			•	Simply-supported	Composite pier
Son & Lee (2011)		•			Cable-stayed	Hollow steel pylon, composite pylon
Williams & Williamson (2011)				•	Simply-supported	Concrete pier
Williamson et al. (2011a)	•				Simply-supported	Concrete pier

steel towers, which focussed on the following parameters: charge weight, span length of steel plate, compressibility of retrofitting matter, axial load intensity, presence of clip angle, degree of fragmentation, steel quality and type of fastener. Only individual steel components were included in Phase I; complex partial models of the steel towers were incorporated in Phase II.

A total of six ¼-scale circular concrete-filled steel tube columns, capable of withstanding both blast and seismic action, were tested by Fujikura et al. (2008). The reaction frame setup ensured a pinned condition at the top, whereas the base was secured with the steel plates and the vertical channels embedded within the foundation. The exact scaled distances adopted to produce the elastic reactions, the maximum deformations and the ultimate failures were, however, not revealed.

Matthews (2008) reported two blast tests which involved full-scale prestressed concrete girders. The reaction structures at both ends were intended to restrain uplift resulted from the below-girder explosion, but treated as simple supports in the above-girder blast scenario. Blast intensities just enough to induce ultimate responses were generated, and this ensured a meaningful impact assessment.

Fujikura and Bruneau (2011) conducted blast experiments on two non-ductile reinforced concrete piers surrounded with steel jackets and two ductile reinforced concrete piers, which were designed to withstand seismic action. Adequate spacing was ensured between the piers along the reaction frame so as to avoid blast wave interference. No instrumentation was installed, however, because of the difficulty with data measurement in close-in detonations.

The blast trials reported by Williamson et al. (2011a) involved highway bridge piers. Using ½-scale specimens, 10 small standoff tests and 6 local damage tests were commissioned to investigate the blast consequences on the piers designed to withstand gravity, seismic or blast action, by relying on the following variables: cross sectional shape, length-to-depth ratio, type of transverse reinforcement, volumetric reinforcement ratio, splice location on longitudinal reinforcement, standoff distance and charge weight. The reaction frame was intended to provide a pinned condition at the top, whereas the fixed connection at the bottom was achieved by grouting the large footing to the reaction slab.

2.2 *Uncoupled methods*

Blast load estimation is usually performed with the aid of empirical equations as well as theoretical approaches. Alternatively, the first principles incorporated within hydrocodes, e.g. Air3d, can also be employed. The blast action is imposed either uniformly or in a non-uniform manner on the targets.

In an equivalent static analysis where the inertial effect and dynamic material properties are ignored, dynamic blast loads are converted into equivalent static loads; in a dynamic analysis, an actual structural configuration can either be idealised as an equivalent single-degree-of-freedom (SDOF) system where a lumped mass supported by a weightless spring is affected by an axial force that brings about a displacement that corresponds to the deflection at the critical location of the actual arrangement under the original loading, or represented as a MDOF system based on finite element analysis (FEA) or frame analysis.

2.2.1 *Static analysis*

Mahoney (2007) applied the blast loads obtained from A.T.-BLAST on a long-span simply-supported prestressed concrete girder bridge, a three-span simply-supported composite girder bridge and a three-span cantilever truss bridge, all simulated in SAP2000. The arbitrary blast setups were decided with the aid of Monte Carlo simulation. The consequence assessments were conducted by referring to the structural damage indicated by the performance of the plastic hinges, along with the possible amount of downtime and casualties.

A suspension bridge model was created by Suthar (2007) in SAP2000. The blast loads taken from A.T.-BLAST were treated as equivalent static loads. The progressive collapse analysis was conducted by relying on the formation of plastic hinges at the top and bottom chords of the trusses.

Islam and Yazdani (2008) modelled a two-span simply-supported prestressed concrete girder bridge with STAAD.Pro. 50% and 30% of the largest peak pressures given by A.T.-BLAST were exerted as uniformly distributed static loads on the units closest to the explosions and along the adjacent members respectively.

Zhou (2009) produced a two-span simply-supported composite girder bridge model, using ANSYS. After the blast impact on a selected point was determined from A.T.-BLAST, the pressure drop at another location at the instant that corresponds to the arrival time recorded was computed, assuming a bilinear pressure profile, as consistent with the adjustment proposed by McClendon (2007), to account for the actual quasi-exponential decay. Then, the average pressure was uniformly distributed across the affected area.

2.2.2 *Dynamic analysis*

Winget et al. (2005) investigated the blast response of a four-span simply-supported prestressed concrete girder bridge, by relying on the SDOF analyses conducted using SPAn32. The reduction in cross sectional area due to local damage was calculated from the empirical equations developed by Marchand and Plenge (1998). The linear wave profile, which preserved merely 80% of the original impulse on a square pier, was adjusted to account for the clearing effect so as to obtain the impulse on the corresponding circular pier, which was then used to scale the pressure history for the square pier, as generated from BlastX.

In the study conducted by Zheng (2007), the blast action determined from A.T.-BLAST was imposed on the reinforced concrete piers simulated with ANSYS. For simplicity, the cross sections of the piers were discretised and transformed into equivalent I-sections with negligible web thicknesses.

Fujikura et al. (2008) devised a simplistic approach to predict the blast reaction of the concrete-filled steel tube columns. The kinetic energy delivered by an impulse was assumed to be fully converted into internal strain energy, ignoring spalling and breaching. A reduction factor was introduced to incorporate close-in blast impact, clearing effect and the influence of strain rate. The blast pressures were calculated from A.T.-BLAST, while the impulse variations along the columns were derived from BEL.

Matthews (2008) built a model of a simple-span simply-supported prestressed concrete girder bridge with ABAQUS. The segments near the supports were assumed to behave in an elastic manner. The deck was also assigned a prestress action, owing to the difficulty in integrating the slab after the introduction of the prestress forces in the girders. The blast inputs were imported from BEL.

Tokal-Ahmed (2009) erected a two-span simply-supported prestressed concrete girder bridge with ELS, and claimed that, under extreme loading conditions, the applied element method (AEM) is preferred over the discrete element method (DEM) as well as the finite element method (FEM). Nevertheless, the underestimation of the numerical results had been acknowledged, since only incident pressures can be generated within ELS. A linear blast distribution (where peak pressure was assumed to vary linearly with distance), together with a correction factor, were proposed for the calculation of the uniformly distributed blast pressure in a SDOF analysis.

Yan and Chang (2009) devised a probabilistic methodology to assess the blast vulnerability of cable-stayed bridges. Stage I predicts the reliability indices of the individual bridge components, which are dependent on the given limit states, through stochastic FEA and the first-order second-moment (FOSM) method; stage II estimates the probability of progressive collapse under the damage inflicted at the component level, by relying on the event tree approach. An imaginary cable-stayed bridge was created for demonstration purpose.

Tang and Hao (2010) constructed a cable-stayed bridge model with LS-DYNA. The main span and the side spans were designed as composite hollow girders and prestressed concrete box girders respectively. The blast pressures for scaled distances above and below 0.067 m/kg$^{1/3}$ were generated using A.T.-BLAST and ANSYS AUTODYN respectively. Owing to the massive scale of the bridge, smeared modelling of rebars, together with linear elastic properties, were commissioned in the segments that were not anticipating blast destruction and the mesh sizes were gradually increased from the blast vicinity onwards.

Fujikura and Bruneau (2011) described a moment-shear interaction model to estimate the direct shear capacity which was expressed as the sum of the cohesion and frictional resistance along the shear interface. Only the compressive region of the pier was of interest, while the contribution from the reinforcing steel was ignored.

By relying on LS-DYNA, Williams and Williamson (2011) developed numerical models to capture the spalling damage on the reinforced concrete piers tested by Williamson et al. (2011a). The blast inputs were extracted from CFD simulation. The qualitative validation was completed by matching the response shapes as well as the damage and crack patterns; the quantitative verification was accomplished by comparing the peak residual displacements. Interestingly, the strain rate behaviour of concrete was deliberately ignored, since strength enhancement was believed to be attributed to inertial restraint and non-homogeneous deformation but not the intrinsic characteristic of the material itself, agreeing totally with Schwer (2009).

3 BLAST EFFECTS ON BRIDGES

3.1 *Decks*

Owing to the close proximity of the explosive, an above-deck explosion will normally inflict local damage on the deck. A steel deck might fracture and suffer plastic deformation (Son 2008), while compressive crushing could happen on a concrete deck (Tokal-Ahmed 2009). During a below-deck blast event, the deck could separate from the girders if the shear fasteners are inadequately designed, according to Winget et al. (2005) and Yi (2009).

Deck failure, albeit usually not devastating to the survival of the entire bridge (Matthews 2008; Tokal-Ahmed 2009), could be beneficial instead, since the dispersion of confined waves will limit the harm imposed on the girders (Winget et al. 2005). Nevertheless, compromising a deck system with a supportive role to the overall structural integrity (e.g. the torsional strength of a curved trapezoidal steel bridge) is unacceptable (Roberts et al. 2003; Williamson and Winget 2005).

3.2 *Girders*

During a below-girder blast incident, the girders in a simply-supported bridge will mostly experience flexural failure (Gannon 2004; Tokal-Ahmed 2009). Nevertheless, shear failure is also possible (Islam and Yazdani 2008). Upward loading is indeed harmful to the girders which are normally intended to withstand predominantly downward loading, and the situation could be worsened if prestressed members are involved (Williamson and Winget 2005). The girders might also endure transverse deflection aside from vertical deformation, thereby suffering biaxial distortion (Tokal-Ahmed 2009).

The girders might undergo rigid body translation or uplift because of the upward action. Apart from that, simultaneous movement of the abutments and the piers due to the horizontal wave could also happen. Consequently, if inadequate seat width is provided, the girders will collapse during the recovery phase after the upward pressure has diminished (Winget et al. 2005; Tokal-Ahmed 2009).

A blast wave generated below the deck will travel towards the ground and compress the surrounding air. The reflected wave assumes faster velocity in this heated medium and might eventually merge with the incident wave approaching the girders, giving rise to a much greater impact. However, if the explosive is situated higher above the ground, the girders might encounter the incident wave before the arrival of the reflected wave, and therefore sustain less damage (Winget et al. 2005). Wave reflection arising in the restricted spaces between the girders will prompt pressure amplification, according to Winget et al. (2005) and Cimo (2007).

The blast implications associated with a blast source placed on top of the deck are comparatively less severe (Tokal-Ahmed 2009). An above-girder explosion will most likely impose merely local impact on the girders, in spite of the bending and shear failures reported by Islam and Yazdani (2008) and Zhou (2009). The deck might also act as a defensive barrier for the girders (Matthews 2008).

It is unlikely for a truss girder with excessive redundancy to incur total collapse (Suthar 2007), unless significant loss of units prevailed (Gannon 2004). However, the functionality of a truss element could be adversely affected by the lateral action as well as the reversal of stress induced within (e.g. a tensile chord might buckle), as suggested by Williamson and Winget (2005).

A blast wave will infiltrate into the interior of a hollow girder through a surface fracture (Son and Lee 2011). The stress waves transmitted from an affected area could also contribute to the damage on the adjoining sections. Tang and Hao (2010) detected major destruction of the base of the concrete box girder, but merely minor failure at the bottom of the hollow steel girder. Box girders are also vulnerable to internal explosions (Williamson and Winget 2005) whereby confined pressures, which bring about higher impulses (Ray et al. 2003), are generated.

The axial forces within a girder arrangement assume a significant role in governing the global response of a cable-suspended bridge. Son (2008) claimed that a cable-supported bridge with enormous axial forces along the girder system (e.g. a self-anchored suspension bridge) will show an inferior blast performance. The blast deformation of a steel orthotropic deck could be magnified by the destabilising P-Δ effect under the presence of the compressive action, and this might lead to subsequent collapse of the entire span. With a composite girder, the axial load initially received by the concrete slab will be transferred to the steel girders which might buckle and give way eventually.

3.3 *Piers and towers*

An explosive situated near the base of a column has a great potential to provoke shear failure, which is the dominant failure mechanism for a wide range of blast scenarios (Fujikura and Bruneau 2011; Williamson et al. 2011a). However, flexural failure could also be a major concern (Winget et al. 2005; Yi 2009).

The blast distribution on a column might vary with time because of the wave reflection beneath the deck. Winget et al. (2005) noticed that the peak pressure was initially recorded at the position corresponding to the charge height, but pressure build-up arose on top of the column over time because of the restriction effect. Nonetheless, the first peak pressure, which arrives earliest with the highest magnitude, will usually dictate the blast response (Williamson et al. 2010).

Concrete spalling and breaching due to a near-field explosion will drastically reduce the capacity of a column (Williamson and Winget 2005; Yi 2009). Interestingly, local damage could also be seen on the side covers of the column, in addition to the front and rear faces, and this is noticeably distinct from the local effect normally seen on a wall (Williamson et al. 2010).

Detonations triggered simultaneously at the opposite sides of a column are expected to inflict much greater harm in comparison with the equivalent explosion that only impacts on a single face, and the relative effect could differ by "as much as a factor of three", according to Winget et al. (2005).

Gravitational load (up to the balance point on the axial load-moment interaction diagram) will promote the flexural capacity of a column (Winget et al. 2005). A column is usually competent to withstand the gravitational and blast loads transmitted via the spans (Williamson and Winget 2005; Winget et al. 2005), even though such extreme action may prompt the column to buckle (Tokal-Ahmed 2009).

Upward blast loading will give rise to a net tensile action within a column that is rigidly linked to the girders, and this may prompt the steel reinforcing bars to fracture, as observed by Yi (2009).

The blast outcome on a hollow structure strongly depends on the P-Δ effect, aside from the presence of intermediate units. Son and Lee (2011) demonstrated the demolition of the steel pylon under the influence of the P-Δ effect. Tang and Hao (2010) noticed that the protected face of the concrete tower was not subjected to blast demolition, owing to the absence of interior units for stress wave transmission. However, the destruction of the rear wall of the concrete pier was not only evident, owing to the existing central wall, but also more violent, since concrete is weaker in tension.

For simply-supported bridges, eliminating the piers will definitely bring the loaded spans to fall, as observed by Tokal-Ahmed (2009) and Yi (2009). Bulson (1997) claimed that, in order to induce collapse, it might be better to "shake down the piers" than to "shoot up the superstructure". Hao and Tang (2010) found that the cable-stayed bridge did not crumble even though the pier and the tower suffered substantial blast damage, unless their cross sections were completely destroyed.

3.4 *Cables and anchorage regions*

Williamson and Winget (2005) claimed that the cables in suspension bridges and cable-stayed bridges are resilient to blast action. Prompt blast wave clearing around the flexible cables with small exposed areas and rounded profiles enables equilibrium to be reached faster. Moreover, multiple cable failures are usually required to collapse a span. The hangers of suspension bridges might break under contact detonations, but not without the use of unique charge shapes.

The anchorage precincts might be susceptible to blast impact (Williamson and Winget 2005). Tang and Hao (2010) noticed that the demolition of the concrete deck had resulted in the loss of the anchorage points for several cables.

4 BLAST COUNTERMEASURES

The two major objectives to be accomplished are threat and consequence mitigation. The former aims to inhibit the occurrence and control the viciousness of blast instances while the latter intends to minimise the detrimental implications on the structures.

4.1 *Risk assessment and management*

A risk assessment and management methodology has been presented by Williamson and Winget (2005) to address blast issues in a macroscopic perspective.

The risk assessment process consists of the following steps:

a) determination of importance level, depending on the inherent functions of the bridge
b) definition of threats, by taking into account internal problems (e.g. structural deficiencies) as well as external factors which include human causes and natural hazards
c) evaluation of vulnerability, by allowing for both strategic and tactical issues in the estimation of the probability of occurrence of the worst-case scenarios
d) identification of consequences, in order to evaluate the severity of the detected risks

The consequences that are deemed unacceptable must be mitigated in accordance with the risk management procedure outlined below:

e) identification of potential countermeasures
f) selection of feasible solutions, with the aid of a cost-benefit analysis, by factoring in financial, environmental, social and technical constraints
g) implementation of the preferred options, by utilising available resources
h) evaluation of the effectiveness of the mitigation schemes
i) continuous monitoring of the performance (if the anticipated outcome is attained) or reiteration of the entire procedure (otherwise)

4.2 *Security measures*

The security measures that can be delivered to alleviate blast threat broadly fall into four major categories (Roberts et al. 2003; Williamson and Winget 2005):

– strategic site layout, which involves practical landscaping (e.g. clearing of overgrown vegetation provides better visual coverage to the surrounding environment)
– information and access deterrence, in order to frustrate attack plots (e.g. prohibiting unauthorised admittance to the interior of a tower and removal of sensitive data from accessible websites)
– regular monitoring and surveillance, to enable prompt discovery of suspicious individuals and actions (e.g. installation of CCTV cameras at prominent spots and good lighting setups)
– efficient planning and coordination, to ensure prompt reactions during blast incidents (e.g. emergency response plans)

Security plans may be customised flexibly to attain the desired security levels which are dictated by the contemporary blast threat levels which, in turn, alter constantly with time (Williamson and Winget 2005). The security practices could be employed as short-term relief while long-term solutions are under development (Roberts et al. 2003).

4.3 *Standoff distance*

The best way of attaining the preferred level of blast protection is through the provision of sufficient standoff distance (Tokal-Ahmed 2009). Increasing the standoff distance even by “as little as few inches” (Williamson et al. 2010) could immensely benefit the survival of a structure. Interestingly however, Winget et al. (2005) realised that increasing the standoff distance up to a particular range alleviated the blast impact to a considerable extent, but the relative advantage derived beyond this point was not substantial.

Nevertheless, satisfactory standoff distances are often difficult to be attained (Winget et al. 2005), since prohibiting full access to certain bridge components could be impractical or impossible. Under permissible circumstances, vehicle barriers and security fences could be erected (Williamson and Winget 2005). Defensive armour could be placed at a slight distance ahead of the target, and the intermediate space could be filled with energy-absorbing substances (Tokal-Ahmed 2009). Furthermore, offering sacrificial layers is also possible, but the associated debris impact should not be neglected (Williamson et al. 2011b).

However, it must be emphasised that relocating a charge to a farther location might not always end up with a positive outcome, as the loading mode involved might be altered. The unification of incident and reflected waves could happen at a larger bridge clearance (Winget et al. 2005), and greater harm could be resulted when local impacts are unfavourably substituted with global effects (Hao and Tang 2010).

4.4 *Structural geometry*

A circular column with a curved surface will attract less reflected pressure and enable the blast wave to clear more swiftly, in contrast to a rectangular column with a flat surface (Williamson et al. 2011b). Under an identical explosion, the blast loads exerted on a round and a square column could differ up to 34% according to Williamson et al. (2010), whereas Fujikura et al. (2008) suggested a 55% decrease, as opposed to the 20% reduction adopted by Winget et al. (2005).

Nonetheless, rectangular columns are not necessarily less favourable. A square column is 27% more massive per unit height in contrast to its counterpart of similar size (i.e. the width equals to the diameter), and therefore incurs smaller shear demand under dynamic loading (Williamson et al. 2011b). Besides, a round column of the same cross sectional area also possesses lower blast endurance (Zheng 2007). Further studies are needed, however, to draw clearer conclusions regarding the relative merits of these two shapes (Williamson et al. 2011b).

4.5 *Structural dimension*

A structure with greater mass is likely to exhibit stronger blast resilience, since smaller velocity will be resulted, thereby reducing the energy that needs to be dissipated through strain (Dusenberry 2010). Longer girders, which are usually bulkier and more flexible, could afford greater deformation (Winget et al. 2005). Larger columns possess higher shear capacities, and therefore able to sustain less damage albeit trialled under smaller scaled distances (Yi 2009; Williamson et al. 2010).

However, tall columns should be avoided if possible or braced at least (Zheng 2007), as stiffness and flexural resistance are inversely correlated to effective height (Gannon 2004). Also, deck thickness should be kept minimal because blast transmission will cease following deck failure, thereby limiting the harm confronting the girders which happen to be more crucial (Gannon 2004).

Williamson et al. (2010) recommended a minimum diameter of 30 in. (0.76 m) for circular columns to counter close-in detonations. The current methods used to estimate the local damage on walls are not suitable for columns (Davis et al. 2009).

4.6 *Material characteristics*

The use of high strength steel will not bring any benefits, unless adequate energy-absorbing capacity is assured (Williamson and Winget 2005). Son (2008) observed that a thinner plate fractured with larger displacement shortly after the explosion, although having greater strength. This was accredited to the early yielding and the greater ductility of the lower grade steel, aside from the lower flexibility of the thicker plate designed to have similar moment capacity. However, if the member size is kept constant, higher strength steel is definitely more advantageous (Gannon 2004).

Judging from the failure modes, Yi (2009) noticed that the concrete piers with higher strength were more superior, albeit observed to be more brittle because a constant reinforcement quantity was adopted. Zheng (2007) also suggested that the blast endurance of a column is proportional to concrete strength. Tokal-Ahmed (2009) recommended the use of slurry-infiltrated fibre concrete (SIFCON), slurry-infiltrated mat concrete (SIMCON) and engineered cementitious composite (ECC) for enhanced ductility.

Concrete is more appropriate than steel when close-in detonations are concerned (Mays et al. 2009). The rebound of a concrete member is less profound because cracking brings about internal damping. Complex stress combinations could arise in steel members which are comparatively more slender (hence more prone to local instability), and the ultimate capacities are also difficult to be estimated.

4.7 *Retrofitting options*

Malvar et al. (2007) as well as Buchan and Chen (2007) acknowledged the capability of fibre-reinforced-polymer (FRP) in enhancing the blast resistance of timber, concrete, steel and masonry structures. Nonetheless, the limitations of FRP have also been highlighted. FRP does not possess enough strain capacity to meet the strain demand during a close-in detonation, and needs to be shielded from flying fragments (Malvar et al. 2007). Silva and Lu (2007) and Tanapornraweekit (2010) proved that strengthening solely the tensile side of the concrete slabs was inadequate, but Wu et al. (2009) observed improvement even though only the compressive face was protected. FRP promotes the shear capacity of a column, but might not be credible in prohibiting concrete breaching (Winget et al. 2005).

A steel jacket shows better competence in preventing acute loss of cross section. It also improves the flexural rigidity of a structural member. Although a minor gain in the shear capacity is also achieved, it is often inadequate to satisfy the increasing shear demand, since now less energy is consumed by means of flexural deformation (Winget et al. 2005). Such claim is consistent with the test results reported by Fujikura and Bruneau (2011). Thus, Winget et al. (2005) suggested that steel jackets could be used in conjunction with FRP wraps in order to exploit the benefits of both materials.

Fujikura et al. (2008) found that the concrete-filled steel tube columns exhibited notable ductility and minimal local damage. Son and Lee (2011) noticed that the concrete-filled steel tube pylon suffered smaller lateral deformation and survived the blast incident, since the associated P-Δ effect was less significant.

Son (2008) presented a strengthening concept known as the “Fuse System”, which aims to improve the blast performance of steel orthotropic decks. The surrounding weak and strong elements are represented with steel plates thinner and thicker than the deck respectively. The weak elements are expected to give way under blast action so that the damaged section is isolated from the remainder of the deck, whereas the thick elements contain the impact within the affected region.

4.8 *Detailing and connections*

In order to counter reverse loading, continuous top and bottom reinforcing bars should be provided in the deck slabs and the girders, and undraped tendons are preferred over draped versions in prestressed members (Williamson and Winget 2005). Additionally, shear fasteners should be carefully prescribed so that the required composite behaviour is attained (Winget et al. 2005).

The ductility and the shear capacity of a column are governed by the transverse rebars incorporated within. Williamson et al. (2011b) pointed out that the minimum amounts of transverse reinforcement required along the total length of a circular and a rectangular column, which are precisely 50% more than the seismic provisions stipulated in AASHTO (2010). The feasibility of other volumetric ratios apart from those considered above has not been evaluated on account of the budgetary and schedule constraints of the experimental programme (Williamson et al. 2011b).

Williamson et al. (2011a) showed that the blast resilience of the columns severely depreciated following the loss of anchorage for the discrete hoops due to concrete spalling. Thus, the spiral reinforcement, which ensures better concrete confinement, is more attractive.

Nonetheless, the blast performance could still be enhanced by augmenting the bend and the length of the hooks to achieve sufficient anchorage. A 90° hook with an extension equals sixfold the nominal diameter of the rebar, *d*, is the standard requirement in AASHTO (2010). The experimental work undertaken by Bae and Bayrak (2008) demonstrated that an extension of 15*d* was more attractive than that of 8*d* for a hook with a 135° bend. For more stringent protection against severe blast action, the length of the 135° hook should not be less than the larger of 15*d* and 10 in. (≈ 0.25 m), as suggested by Williamson et al. (2010).

Splices used to link longitudinal steel bars should be avoided whenever feasible, since concrete breaching is possible if an explosion occurs near the splices. Williamson et al. (2011b) advised that the lowest splice position should be at least 3.6 m above the ground or the deck (by assuming the height of a typical truck bomb to be 1.8 m). Alternatively, splice length could also be raised in order that the rebars remain anchored to the concrete. However, no recommendations have been offered to define the acceptable splice lengths, owing to lack of pertinent data.

Local buckling of steel members may be prevented with the aid of stiffeners (Roberts et al. 2003; Williamson and Winget 2005). Likewise, lateral bracings could also be attached to girders and columns, as an attempt to improve their flexural capacities (Gannon 2004; Winget et al. 2005).

Enlarging the abutments and extending the expansion joints could avoid possible loss of seating (Roberts et al. 2003). Hinge and cable restraints, which commonly serve in earthquake-prone regions, may also be useful (Williamson and Winget 2005). Tokal-Ahmed (2009) described a fixed connection, where the shear studs embedded within the girder are welded to the plate which is held firm to the abutment or the pier with the anchors.

4.9 *System redundancy*

Both operational and structural redundancy ought to be taken into account (Winget et al. 2005). The former (e.g. sound traffic management) ensures prompt reactions during the aftermath of a blast event while the latter offers alternative load paths for the transfer of action from the units that are dismissed.

Strong connections and continuous rebars are vital for the containment of blast effect (Williamson and Winget 2005). Additional girders and cables could also be delivered if possible (Roberts et al. 2003). Aside from erecting extra columns, energy-absorbing walls could also be installed either between the columns or as the primary support (Tokal-Ahmed 2009).

The auxiliary cable system developed by Tan and Astaneh-Asl (2003) to prohibit progressive collapse of steel buildings could be adapted for use in bridges, as advised by Tokal-Ahmed (2009). The supplementary cables could be placed in the intermediate diaphragms and the vertical stiffeners (along the longitudinal direction) of the concrete and steel girders respectively. This secondary function is only activated if the columns are removed, and the gravitational load will be transferred to the anchorage locations behind the abutments or beneath the spans.

4.10 *Performance-based design methodologies*

Performance-based blast designs should be implemented in a multi-hazard perspective, taking into account a diverse range of potential risks and consequences whilst factoring in available resources together with their possible allocation, when defining the performance objectives which consist of design events and performance levels (Whittaker et al. 2003), making full use of the prescriptive standards as a baseline (Thompson and Bank 2007). The design schemes must be capable of accommodating complicated modifications in a highly flexible manner.

The design criteria corresponding to various protection levels, usually commence with “superficial damage” (or equivalent) and terminate with “collapse prevention”, vary from different sources depending on the focus (Winget et al. 2005; Mahoney 2007; Son 2008; Yi 2009), and are normally qualitative for global integrity but quantitative for individual components.

5 FURTHER RESEARCH

Current research efforts focussed chiefly on the simply-supported bridge, particularly on the pier which tends to be the most critical and vulnerable component (Williamson and Marchand 2006). More studies devoted to the key elements of suspension bridges (e.g. main cables and saddles), cable-stayed bridges (e.g. cables and anchorage zones), arch bridges (e.g. tension ties) and truss bridges (e.g. compression chords and connections) ought to be undertaken (Roberts et al. 2003).

The enquiry of the blast implications on prestressed bridge components has attracted very little interest. Good understanding of the role of the bonding effect between the prestressing tendons and the concrete under reverse loading should be established. Reasonable estimation of the loss of prestress as a result of concrete disengagement is also very important (Winget et al. 2005).

Little is known about the combined outcome derived from blast, impact and fire, which are usually dealt with separately. At this stage, it is uncertain if any relevant work has been undertaken for such purpose. However, it is difficult, if not impossible, to assess the individual contribution from each loading, and the need for more sophisticated approaches is inevitable.

Another topic that deserves attention is the assessment of the residual capacity of a damaged member, which will ultimately lead to a more sensible emergency reaction plan. Bao and Li (2010) and Wu et al. (2011) proposed empirical formulae to estimate the residual capacities of reinforced concrete columns subjected to blast impact. Similar effort could also be extended to other bridge components in the future.

Blast failure indicators are crucial for proper quantification of blast outcomes. The ductility limits reserved for ordinary structural members need to be adjusted to suit bridge units (Williamson and Winget 2005). Blast assessment criteria appropriate for connections, cables and anchorage spots are yet to be decided. However, damage parameters for local responses could be difficult to be defined.

Some of the descriptions of the defence tactics are conceptual and qualitative in nature. So far, no attempts have been made to quantify the degree of protection offered by the security practices. The viability of the wall arrangement suggested by Tokal-Ahmed (2009) needs to be affirmed, since larger reflected pressure is to be expected on a flat surface. Apart from that, installing additional columns might not be beneficial, since less gravitational load will be attracted to each individual column, and this will lead to a lower gain in the flexural capacity.

The constructability of the hardening tactics should not be neglected. Augmenting the transverse reinforcement ratio promotes the shear capacity of a column, but renders the placement of the longitudinal rebars problematic and invites the formation of concrete voids. Alternatively, mechanical couplers at splice locations, fibre-reinforced concrete, concrete-filled steel tubes as well as retrofitting options may be considered (Williamson et al. 2010).

Comprehensive evaluation of the relative effectiveness of the various protective measures should be strongly encouraged, as this facilitates logical prioritisation of the solutions as well as direct designation of the most appropriate fortification scheme to a given project. For example, Williamson et al. (2010) recommended that the following critical variables ought to be dealt with in such order of effectiveness: standoff distance, structural geometry, amount of transverse reinforcement, type of ligature and anchorage, and splice location on longitudinal reinforcement.

FRP is amongst the most popular materials used for blast protection. Buchan and Chen (2007) claimed that some fundamental aspects, such as the bonding characteristic of FRP, the replacement criteria for FRP and the substrates, and the relative merits of different composite types are still not well understood.

The potential of other retrofitting matter should also be evaluated. The advantages of ultra-high strength concrete (UHSC) have been affirmed by Ngo (2005), Wu et al. (2009) and Yi et al. (2012), but its practicality as a hardening option for existing bridges remains unclear. Polyurea is useful for blast protection (Chen et al. 2008; Hrynyk and Myers 2008), but its bonding behaviour and strain rate properties associated with various chemical compositions should be thoroughly explored. De-bonding has been reported by Ackland et al. (2013), but not Raman (2011).

Blast experiments ought to be regarded as the most reliable source of knowledge especially for those relatively unexplored areas. However, only restricted amount of blast tests related to bridges have been reported. More practical work is required to realistically capture the blast response of bridge structures, to expand the database available for validation use, and to initiate further enquiries.

Most of the analytical studies have not been verified by experimental results. The test data collected by Fujikura et al. (2008), Fujikura and Bruneau (2011), and Williamson et al. (2011a) was used solely to devise simplistic analytical and design methods. Explicit calibration of the parameters in FSI simulations should be encouraged, as this could benefit other situations with little or no supporting data.

Some finite element modelling strategies commonly serve in blast simulation should be carefully revised. Beam elements are acceptable for cables during axial static loading, but not under direct blast impact. Anchorage points are often oversimplified with direct links that are not reflective of the exact situations (Son 2008; Tang and Hao 2010). Connections are usually either treated as interactive or shared nodes, or simply replaced with boundary conditions, and this might not be always true because unrealistic stress concentration could arise.

Concrete spalling is often replicated by deleting the subjugated elements (Williams and Williamson 2011), even though this violates the conservation of mass (Tang and Hao 2010). However, no standard rules are available for the determination of the desirable criteria to invoke erosion, and the most commonly selected parameters are maximum principal strain and tensile strength.

The implementation of risk and reliability analysis is strictly lacking. Only the FOSM method has been attempted by Yan and Chang (2009), while the application of higher order and sampling techniques awaits further demonstration. Perhaps the greatest challenge lies in obtaining the appropriate performance functions, especially when significant local effects arise.

The blast-resistant design guidelines for highway bridge piers published by Williamson et al. (2010) covered three design categories: scaled distance (Z) > 1.2 m/kg^3 (where no blast protection is required), 1.2 m/kg^3 $\geq Z >$ 0.6 m/kg^3 (where seismic solutions are deemed adequate), and lastly $Z \leq$ 0.6 m/kg^3 (which calls for stringent fortification). However, similar need of other bridge components is still not specifically catered for.

The transferability of the current blast design standards (intended mainly for buildings and petrochemical facilities) to the context of bridges should be evaluated. The structural units in a building are normally shielded by the façade, while bridge components, in general, may be accessed with greater convenience, thereby anticipating more profound blast consequences aside from more diverse loading scenarios.

6 CONCLUSIONS

Extensive blast investigations have been conducted on towers, piers and girders, whereas fewer efforts have been dedicated to cables, anchorage points, decks and connections. The simply-supported bridge, which is known unanimously to be the most common bridge type, has received the most intensive attention, while restricted number of works related to cable-suspended bridges have also been published.

Both coupled as well as uncoupled methods have been applied, not only in a deterministic style, but also in a probabilistic manner. Although dynamic analysis (both SDOF and MDOF) has expectedly exhibited strong popularity, the fact that static analysis has also been attempted in numerous works in spite of its potential drawbacks is indeed surprising. FSI simulation was preferred over the experimental approach.

Blast incidents on bridges may be broadly categorised as above-deck and below-deck explosions. Deck failure usually does not pose devastating threat to the overall integrity of a bridge, unless its strength and stability are compromised. A below-girder explosion is more destructive, owing to its more widespread impact, its capability to provoke uplift and the pressure amplification observed in the restricted spaces. A truss girder often enjoys excessive redundancy, but could suffer from lateral action and stress reversal. For columns, shear failure is more common than flexural failure; local damage could also be detected on the side covers; and axial buckling and tensile pulling are possible as well. Blast wave infiltration, stress wave transmission and significant P-Δ effect are also responsible for the demolition of hollow structures. Cables are believed to be resilient to blast impact, as opposed to the anchorage regions.

Threat and consequence mitigation can be achieved by means of: flexible security practices, sufficient standoff distance, practical structural geometries and dimensions, desirable material properties, effective retrofitting tactics, proper detailing and connections, as well as adequate system redundancy. A risk assessment and management framework can be devised to deal with blast issues in a systematic manner. Performance-based design principles are also applicable to blast engineering.

Further enquiries should emphasise on innovative development of new techniques, along with critical review of the current methods, for blast investigation and protection. Additionally, robust expansion of the current state of knowledge regarding the blast response of bridges is also vital.

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