PERFORMANCE OF BRIDGE FALSEWORK STRUCTURES UNDER EXTERNAL HAZARDS

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Abstract: Bridge falsework systems are one of the most common temporary structures used during the construction of concrete bridges. In this paper the results obtained from numerical studies of a selected structural system made of steel are presented. In particular, risks associated with bridge falsework systems will be analysed for the most relevant external hazards: (i) concrete casting loads; (ii) wind loads; (iii) ground settlements. For each considered hazard scenario, the resistance of the system is calculated and analysed based on deterministic analyses. From the results, relevant practical information is obtained which can reduce the risk associated with the design and operation of bridge falsework systems.

1. Introduction

The present paper concerns bridge falsework, commonly used during the construction, and rehabilitation to the retrofit of bridge structures, in particular those using the Cuplok® systems, see Fig. 1.

Fig. 1: Example of bridge falsework Cuplok® systems

The present paper contributes to a better knowledge about the structural behaviour and resistance of bridge falsework systems.

To study the behaviour and resistance of bridge falsework systems the finite element analysis program ABAQUS® was used. Since the joint elements available in ABAQUS® are unable to model the non-linear analytical model derived from joint tests results, in particular the stiffness and resistance degradation with loading cycles, a new joint element was developed to simulate the behaviour, resistance and failure of several types of joints present in bridge falsework systems. Therefore, this paper starts with a summary of the details of joint modelling and the formulation of the new joint element. Afterwards, the verification procedure is briefly presented.

Finally, the behaviour and resistance of bridge falsework will be analysed for some of the most relevant external hazard scenarios: (i) concrete casting loads, including dynamic effects and local overloads; (ii) wind loads, with varying values according to the construction phases of the bridge relevant to the falsework system, *i.e.* assembling of the falsework, casting of the concrete and curing of the concrete; (iii) ground settlements; (iv) combined effect of actions of different nature.

2. Development of numerical models

2.1 Joint analytical constitutive models

Several types of joints exist in bridge falsework Cuplok® systems, the most common being: (i) beam-to-column joints (*aka* standard-to-ledger joints or Cuplok joints), (ii) column-to-column joint (*aka* spigot joints), (iii) brace-to-ledger joints, (iv) top and bottom boundary joints (*i.e.* forkhead and baseplate joints).

Different numerical modelling techniques are available to simulate these types of joints: from the more complete 3D joint modelling using solid elements to the simple spring-like joint modelling. In this paper phenomenological models are preferred. The analytical models used for Cuplok® joints, spigot joints and forkhead joints have been derived from the experimental tests presented in [1,2]. The analytical model consists of a multilinear fit to the experimental tests results. The complete set of models is presented in [1].

Due to paper size limitations only the constitutive model of the bending behaviour of the brace joints is presented here. From the six degrees of freedom available at the brace joint, *i.e.* between a brace element and its supporting element (either a standard or a ledger element), only the degree of freedom associated with the displacements along the longitudinal (axial) axis of the brace element was included explicitly in the analytical model.

The analytical model for the axial axis is as follows:

Compressive and Tensile forces (Monotonic loading):

$$\delta \in \left[-\delta_4, \delta_4\right] \Rightarrow F^i = F^{i-1} + \left(\delta^i - \delta^{i-1}\right) \times k$$

$$\delta \in \left]-\infty, -\delta_4\right[, \left]\delta_4, +\infty\right[\Rightarrow F^i = \delta^i \times k_{res}$$

$$\delta \in \left[\delta_1^-, \delta_1^+\right] \Rightarrow k = k_1^+ = k_1^- = k_1$$

$$\delta \in \left[-\delta_2, \delta_1^-\right[, \left]\delta_1^+, \delta_2\right] \Rightarrow k = k_2^+ = k_2^- = k_2$$

$$\delta \in \left[-\delta_3, -\delta_2\right[, \left]\delta_2, \delta_3\right] \Rightarrow k = k_3^+ = k_3^- = k_3 \quad \text{with} \quad \delta_1^+ < \delta_2^+ < \delta_3^+ < \delta_4^+$$

$$\delta \in \left[-\delta_4, -\delta_3\right[, \left]\delta_3, \delta_4\right] \Rightarrow k = k_4^+ = k_4^- = k_4$$

$$k_2 > k_3 > k_4,\ k_2 > 0{,}5 \times k_1$$

Unloading (bilinear model):

$$\textit{If}\ F^i < k_{ZERO} \times \delta^i \Rightarrow k = k_{ZERO} = \max\left[1/100 \times \max\left(k_L\right), k_{res}\right],\ F^i = k_{ZERO} \times \delta^i$$

$$\textit{Else}\ k = k_U > \max\left(k_L\right),\ F^i = F^{i-1} + \left(\delta^i - \delta^{i-1}\right) \times k$$

All degrees of freedom were considered to work in isolation. Therefore, no interaction between degrees of freedom was considered in the analytical model.

The degrees of freedom associated with shear displacements were considered to be linear elastic with a large stiffness value, k, equal to 1×10^6 N/mm. In the case of the degree of freedom associated with torsion rotations a flexible joint with a linear elastic stiffness given as an input parameter of the analytical model was used; in general, a value equal to 50 kN.m/rad was adopted for the torsional axis. Finally, the two bending rotations degrees of freedom were considered to be free to rotate.

2.2 Finite element types

2.2.1 Bridge falsework main elements

The main elements of bridge falsework systems are standards (including jacks), ledgers and braces. All these elements were modelled using second-order beam elements with six degrees of freedom per node using Timoshenko beam theory (Abaqus® B32 element), suitable for finite strains and large rotations problems. The sections of the different parts of the elements were included in the elements definitions.

2.2.2 Formwork system

The formwork system was considered made of plywood beams positioned in an orthogonal mesh on top of the bridge falsework system, and of plywood panels to which the construction loads were applied to. All the beam elements were modelled using second-order beam elements (Abaqus® B32 element), and the plywood panels were modelled using first order reduced integration shell elements with six degrees of freedom per node and second-order accuracy and enhanced hourglass modes control using thick shell theory (Abaqus® S4R element), also suitable for finite strains and large rotations problems.

2.2.3 Joint elements

For all other joints a spring-like finite user element was developed through Abaqus® UEL subroutine. The spring user element is made of three nodes, labelled node 1, node 2 and node 3, respectively, each with six degrees of freedom: three displacements and three rotations. The first two nodes are coincident and were used to control the constitutive behaviour of the user element and each one belonged to a different element (beam element or another user element). The third node of the user element is the second node of one of the beam elements attached to the user element and the distance between the second and third node of the user element is non zero. This third node is used to determine the initial directions of the x, y and z axis of the local coordinate system of the user element. The details of the user finite element are given in [1].

2.3 Materials

Only one type of material was considered for each system: (i) steel for the bridge falsework system and (ii) plywood for the formwork system. Ground was not explicitly modelled.

The steel was modelled by an isotropic elastic material with loading rate independent Young's Modulus, E, and Poisson's coefficient, υ, with isotropic plasticity and isotropic hardening. A linear damage evolution model was also considered by reducing the internal forces of the element linearly as a function of plastic deformation values larger than the deformation value at tensile strength. When the Ramberg-Osgood relationship parameters were available this model was preferred.

Plywood was modelled as orthotropic elastic material with loading rate independent isotropic plasticity and isotropic hardening. A linear damage evolution model identical to the one used for steel was considered. See [1] for details.

2.4 Validation and verification of numerical models

In order to validate the numerical models developed to study the behaviour of bridge falsework systems several parameters were analysed. Different mesh densities, numerical solvers and joint models were compared, and the ones that performed better were selected. A 50 mm length was selected as the reference finite element size. See [1] for details.

After being validated the numerical models were verified by comparing the numerical behaviour with the behaviour measured in 18 full-scale tests performed at the University of Sydney in 2006, and published in [3]. The statistical analysis of the ratio between the recorded maximum load and the numerically predicted value is presented in Table 1. It can be observed that the developed numerical models can match the experimental resistance with a better precision and accuracy than the previously developed numerical models.

Table 1: Statistical results.

	Previous work [4]	Present work
Average ratio	1,012	1,003
Standard deviation of the ratio	0,100	0,057
COV of the ratio	0,098	0,057

3. Bridge falsework numerical analyses

Different models were considered in this section. Unless noted otherwise, the models considered resemble the structures A2 and A4 tested in the Sydney University (referenced in this section as Models A2 and A4, respectively), see [4]. Both structures display a grid frame of three-by-three bays with a constant nominal bay width of 1829 mm in both directions, with three lifts with equal nominal lift height of 1,5 m and 600 mm of jack extension height. The bracing configuration of Model A2 is represented in Fig. 2. Bracing arrangement is the same in each bay in each direction. Model A4 is braceless.

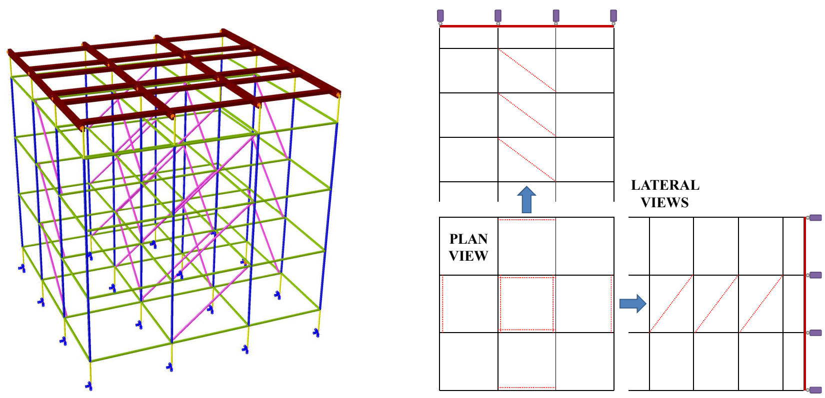


Fig. 2: Structural layout of Model A2

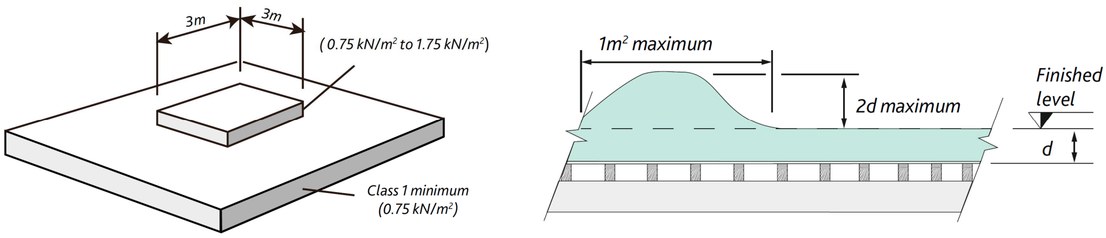
In all cases, the cross-section geometrical characteristics as well as the material properties of the various elements which make the falsework system are identical to the ones used in the structures tested in the Sydney University. The standards were made from cold-formed circular steel tube section (CHS) with a nominal yield stress of 450 MPa. The cross-section had a nominal external diameter of 48,3 mm and a wall thickness of 4 mm. Ledgers were made of steel with nominal yield stress of 350 MPa, also of CHS with a nominal external diameter of 48,3 mm and thickness of 3,2 mm. The telescopic brace elements, CHS with outer tube cross-section of 48,3 mm × 4,0 mm and inner tube cross-section of 38,2 mm × 3,2 mm, were connected to the ledgers by hook joints. The nominal yield stress of the brace elements steel was equal to 400 MPa. The adjustable jacks were made of 36 mm diameter threaded steel rods with nominal yield stress equal to 430 MPa. The rectangular baseplates were 180 mm wide and 10 mm thick with nominal yield stress equal to 250 MPa [4].

3.1 Concrete casting action

For single span concrete bridges, the bridge decks when casted in situ can be concreted in a single operation, starting from one end or from the middle of the span. For continuous span concrete bridges, alternative casting methods can be used involving construction joints at one fifth of the span length.

Concrete can be placed either by skips or by pumps. The latter is nowadays the most used method for placing concrete on bridge decks. Concrete casting loads consist in a combination of dead loads and variable loads. The former consists on the weight of the fresh concrete plus the weight of the reinforcing steel. The latter consists on the weight of the workers, tools and equipment, plus allowance for heaping of the concrete and loading dynamic effects.

The self-weight of the fresh concrete and of the reinforcing steel can be considered equal to 26 kN/m^3. Fig. 3(b) illustrates the possible local heaping of the concrete during concrete placing and the unfactored load values to account for this variable load specified in BS EN 12812 [6] (Fig. 3(a)). The loads specified in BS EN 12812 only allow for concrete to be dropped by no more than 1 m height and also a heap height not greater than three times the depth of the slab (subject to a maximum imposed load of 1,75 kN/m^2), applied to a maximum area equal to 1 m^2 as shown in Fig. 3(a) [7].



(a) *In-situ* concrete construction loads [5] (b) Heaping of concrete [6]

Fig. 3: Casting construction loads

The dynamic effects of concrete placing are complex. Based on the values published by [7], the equivalent dynamic load for skips is in the range of 0,8 to 1,6 kN. For pumps, as the rates of flow of concrete are much lower than the ones when using skips, the equivalent dynamic load is estimated not to be greater than 0,5 kN.

Several numerical models were developed to test if under a number of different scenarios the concrete casting could be a critical hazard to the safety and performance of bridge falsework structures [1]. Therefore, two different concrete placing methods were analysed combined with various local concrete heaping values (from zero to two times the slab thickness), slab thickness (0,25 m to 1,5 m) and number of casting layers (one to ten). Table 2 presents the models characteristics.

In all models, pumps were used to place concrete and a 0,5 kN equivalent dynamic load was therefore considered, associated to concrete blocks representing a 1 m^2 formwork area. Also, the stiffness of the poured concrete was considered negligible, thus not contributing to the load distribution to the formwork system. The only loads considered were the ones associated with the concrete casting action itself: (i) weight of the fresh concrete, (ii) local concrete heaping and (iii) equivalent dynamic loads. If collapse of the falsework system was not attained when the slab was fully casted, the thickness of the slab was increased until collapse state was reached.

Table 2: Summary of different model characteristics used to analyse concrete casting actions*

Model-ID	Structure (s)	Concrete placing method (c)	Local heaping height (h)	Slab thickness (t)	Number of concrete layers (l)
s-c-h-t-l	A2, A4	1, 2	h = 1: None h = 2: 1×slab thickness h = 3: 2×slab thickness	0,25 m, 0,5 m, 1,0 m, 1,5 m	1, 2, 5, 10

*Example of Model A2-1-1-0,25-1: Model A2 with concrete placing method #1, with no local heaping height, a slab thickness equal to 0,25 m and the number of concrete layers is equal to one

From the results obtained for the various numerical models developed, see Table 3, it can be observed that the most influencing factor related with the concrete casting action is the local concrete heaping height. However, only considerable unrealistically high values (*e.g.* two times the slab thickness) lead to an important degradation of the maximum pressure value that the system can resist. All other variables, *i.e.* concrete placing method, slab thickness and dynamic effects, seem to have a very small influence on the resistance of the falsework system. In conclusion, local concrete heaping height should be controlled and its value should be limited, in particular for falsework systems which are subject to loads close to their resistance capacity, especially for thin slabs.

Table 3: Results of the models developed to analyse concrete casting actions

Model	Maximum pressure (N/mm^2)
A2 reference	0,03909 (0,0%)
A2-1-1-0,5-1	0,03906 (-0,1%)*
A2-1-3-0,5-2	0,03800 (-2,8%)*
A2-2-3-0,5-2	0,03795 (-2,9%)*
A2-2-3-0,5-5	0,03788 (-3,1%)*
A2-1-1-1,0-1	0,03900 (-0,2%)*
A2-2-2-1,0-2	0,03793 (-3,0%)*
A2-2-3-1,0-2	Collapse was reached for t=51,2s
A2-2-3-1,0-5	Collapse was reached for t=67,8s
A2-1-1-1,5-2	Collapse was reached for t=98,8s
A2-2-1-1,5-2	0,03900 (-0,2%)*
A2-2-1-1,5-10	0,03900 (-0,2%)*
A4 reference	0,01401 (0,0%)
A4-2-3-0,25-2	0,01391 (-0,7%)**
A4-2-3-0,5-1	0,01395 (-0,5%)**
A4-2-3-0,5-2	0,01382 (-1,3%)**
A4-2-3-0,5-10	0,01354 (-3,4%)**

*Relative to A2 reference model resistance

**Relative to A4 reference model resistance

3.2 Wind action

Wind is another important action to be considered in the design of bridge falsework structures. Wind action is always present in any given instant of time with a certain direction and intensity which are both complex to characterise, to predict and involving a large uncertainty. During the operation of bridge falsework structures wind may play a critical role in any of the following important phases: (i) during assembly of the falsework system, (ii) during the casting of the concrete and (iii) after concrete has been placed but before the concrete has hardened to a degree where it can resist the applied actions by itself, including the wind action.

Traditionally, wind action is specified in design standards. For bridge falsework, BS EN 12812 [5] specifies the following requirements:

- Assembly phase (referred as phase p1): Maximum wind velocity;
- Concrete casting phase(referred as phase p2): Working wind velocity;
- Phase before concrete has hardened (referred as phase p3): Maximum wind velocity.

Wind velocity was determined from BS EN 1991-1-4 [8] complemented by BS 5975 [9], see [1] for details. The design load value was obtained by multiplying the characteristic value by a partial factor equal to 1,5. For the falsework elements a design value equal to 0,071 N/mm was obtained. For the working wind velocity the load values were calculated considering a wind pressure equal to 200 N/m^2 [8].

Several numerical models were developed to test if under a number of different scenarios the wind action could be a critical hazard to the safety and performance of bridge falsework structures, see Table 4. Wind action was only considered in one direction: the direction of the collapse mode of Models A2 and A4 under vertical loads.

Wind action was combined with concrete casting loads in phases p2 and p3. In phase p2 only the working wind velocity was considered for all models. As a simplification, during phase p2 the concrete casting loads were modelled as a uniform load distributed over the entire formwork surface which value increased until the weight of a reference slab thickness was attained. For Model A2 a 0,5 m reference slab thickness was considered whereas for Model A4 a 0,25 m reference slab thickness was considered.

Table 4: Summary of different model characteristics used to analyse wind actions*

Model-ID	Structure (s)	Wind action during assembly phase (p1)	Wind action after concrete casting is finished (p3)	Spigot pins present? (sp)	Baseplate anchor bolts present? (ab)
s-p1- p3-sp-ab	A2, A4	p1 = 1: Maximum wind p1 = 2: Working wind	p3 = 1: Upper limit wind + reference slab p3 = 2: Working wind+ upper limit slab	sp = 1: No sp = 2: Yes	ab = 1: No ab = 2: Yes

*Example of Model A2-1-1-1: Model A2 with maximum wind during phase *p1* and wind as leading action during phase *p3*, with no spigots and no anchor bolts

In phase p3, either the weight of the slab was increased until collapse was reached maintaining the wind action equal to the working wind, or the wind action was increased until collapse was reached maintaining the thickness of the slab equal to the reference slab thickness.

It was also analysed the effects of incorporating anchor bolts at the baseplates and pins at the spigot joints to add resistance against uplift loads to the systems. The anchor bolts considered were made of steel, had 10 mm nominal external diameter and the steel had an ultimate tensile strength equal to 600 MPa. One anchor bolt was positioned at each corner of every baseplate, separated 100 mm from each other, making a total of four anchor bolts per baseplate. It is assumed that the anchor bolts resistance is equal to the tensile resistance of the anchor bolts. The pins considered were also made of steel, had 8 mm nominal external diameter and the steel had a yield strength and an ultimate tensile strength equal to 400 MPa and 500 MPa, respectively.

From the results obtained, see Table 5, it can be observed that the most influential factor related with the wind action is the occurrence of the maximum design wind velocities.

Table 5: Results of the models developed to analyse wind action

Model	Maximum wind load on falsework (N/mm)	Maximum concrete pressure on formwork (N/mm2)
A2 reference		0,03909 (0,0%)
A2-1-1-1-1***	0,03749	
A2-2-1-1-1	0,17658	
A2-2-2-1-1		0,03659 (-6,4%)*
A2-1-1-2-2	0,14598	
A2-1-2-2-2		0,01558 (-60,2%)*
A2-1-1-1-2	0,08281	
A2-1-1-2-1***	0,03787	
A4 reference		0,01401 (0,0%)
A4-1-1-1-1	0,03408	
A4-2-1-1-1	0,07930	
A4-2-2-1-1		0,01345 (-4,0%)**
A4-1-1-2-2****	0,07105	0,00344 (-75,5%)**
A4-1-1-1-2***	0,06215	
A4-1-1-2-1***	0,03726	

*Relative to A2 reference model resistance

**Relative to A4 reference model resistance

***Collapse occurred during phase *p1*

****Collapse occurred during phase *p2*

High values of wind action lead to a significant degradation of the resistance of the system. This is particularly true for braceless falsework systems (Models A4). However, even the occurrence of working wind velocities had an impact on the system resistance, in particular for braced falsework systems see Model A2-2-2-1-1 for example. This is justified because wind action subjects spigot joints to larger rotations, thus larger bending moments, and spigot joints are a weak link of bridge falsework structures. Therefore, collapse occurs for lower concrete pressures than the ones obtained when wind action is not considered.

It was also possible to conclude that in braced systems (Models A2), including pins at the spigot joints and anchor bolts at the baseplates had a significant beneficial effect on the system's resistance when compared with the option of not using these components, see models A2-1-1-1-1 and A2-1-1-2-2 for example. Therefore, if high wind velocities are forecasted one option to increase structural resistance is to use brace elements and anchor bolts at the baseplates.

3.3 Ground settlements

Ground settlements are also another potential critical hazard which deserved an in-depth analysis. Due to the low robustness of bridge falsework systems, see [1], any imposed load redistribution may not find the required force redistribution capacity driving the system to collapse.

In this study, various hazard scenarios were considered using the already presented Models A2 and A4. In all scenarios, the ground settlement action was applied as imposed displacements at bottom node(s) of bridge falsework models – a limit case scenario considering the ground has no stiffness. The settlements were combined with concrete casting loads as follows: during the concrete casting phase, differential ground settlements were increased until reaching a target reference value (10 mm or 100 mm) which coincided with the end of the concrete placement. In all models, the concrete casting loads were modelled as a uniform load distributed over the entire formwork surface which value increased until the weight of a reference slab thickness was attained. For Model A2 a 0,5 m reference slab thickness was considered whereas for Model A4 a 0,25 m reference slab thickness was considered. Afterwards, concrete pressure was increased until collapse was reached.

Several numerical models were developed to test if under a number of different scenarios the differential ground settlement action could be a critical hazard to the safety and performance of bridge falsework structures, see [1]. In particular, localised and widespread ground settlements were considered, see [1].

Additionally to the cuplok joint (connecting falsework beams to columns) characteristics taken from the tests results reported in [1,2], an alternative scenario was considered in which the cuplok joint exhibited large looseness (0,03 rad, approximately four times the tests average value) and small initial stiffness (1 kN.m/rad, more than ten times lower than the tests average value), to analyse how the structure can accommodate the ground settlements in these circumstances.

Analysing the results, see [1], it can be concluded that there is a noticeable (negative) sensitivity of the considered bridge falsework structures resistance, and of its variability, to the possibility of differential ground settlements. Even for residual differential ground settlements (*e.g.* 10 mm) it was found that there is a critical scenario where a localised residual differential ground settlement can generate a significant reduction (20%) of the resistance capacity of the tested falsework structures. For higher settlement values, the critical scenario changes from a localised occurrence to a more widespread occurrence, with an increase of the negative impact on the system's resistance, which could represent less than 50% of the resistance of the reference system.

It is also possible to conclude that the resistance of stiffer falsework systems seems to be more sensitive to differential ground settlements. When comparing the results for type-A2 models (braced systems) with the results for type-A4 models (unbraced systems), the reduction of the resistance in the former models can reach values up to 50% whereas in the latter models the maximum reduction is approximately half of this value, against the resistance of the reference system.

In addition, the influence of the presence of large looseness at the cuplok joints was also analysed. This scenario can occur due to application of a deficient lock procedure of these joints (André 2014). The result is a large drop on the system's resistance. However, these systems, with increased looseness at the cuplok joints, are relatively less sensitive to differential ground settlements than the original systems with lower looseness. This can be justified because the presence of significant looseness at the cuplok joints helps the system to accommodate differential ground settlements with less induced strains than the ones that occur in a system with smaller looseness at the cuplok joints.

3.4 Combined effect of actions

The previous sections concentrated in studying the potential impact of the application of external actions of three different natures, in the safety and performance of bridge falsework structures, taking as application examples cases of simple structural systems. However, up until now the combined effect of these three different actions was not considered. Several numerical models were developed to determine how much penalising combining the three different actions would be to the safety and performance of bridge falsework structures with respect to the results of each action applied in isolation [1].

From the results obtained, see [1], it could be concluded that the combined effect of external actions is more severe than the effect of each action applied in isolation: up to 70% reduction of the resistance was observed due to the combined effect of external actions when compared to the case of single action application. Therefore, bridge falsework systems must be designed accounting for all foreseeable actions and their concomitant values.

4. Conclusions

In this paper a summary of the development, validation and verification of advanced numerical models of bridge falsework was overviewed. From these models it was possible to study several relevant hazard scenarios through deterministic analyses. It was found that:

1. Concrete casting loads, including dynamic effects and local overloads can be considered important only for thin slabs supported by falsework structures which do not exhibit a large safety margin;
2. Wind loads, on the other hand were found to be critical loads since they can reduce, in some cases drastically, the resistance of the falsework. In particular, strong winds can overturn the bridge falsework structure when it is still unloaded during the assembly phase. Various solutions were analysed and it was found that including proper bracing and anchor bolts at the falsework baseplates was the most efficient solution to prevent collapses in early phases due to strong wind action. Of course, assuming that the foundation is properly designed and prepared;
3. The effects of differential ground settlements were also analysed. It was demonstrated that even a small value of isolated differential ground settlements could reduce by more than 10% the resistance of the system. It was also found that stiffer systems are more sensitive to differential ground settlements than more flexible solutions because the latter can accommodate the imposed displacements without straining significantly the structure. However, excessive looseness at the joints can reduce considerably the resistance of the system;
4. The combined effect of different actions should always be considered during the design of bridge falsework systems, unless demonstrated that it is not relevant. Reductions of more than 50% on the resistance value were observed when compared with the isolated action application of construction loads vertical pressures;

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