Numerical and experimental investigation on real temporary structures useful for incremental launching bridge construction

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Summary

The study is aimed to evaluate the influence of the construction quality on the buckling load as well as on the bearing capacity of the real sub-structural system provided in order to keep the cantilever moment to a reasonable magnitude, during continuous post-tensioned multi-span bridge casting. The clamping bolt quality as well as the stiffness of the flanged joint of the two steel truss piers, which had collapsed during the "Porcaria bridge" construction, are studied. At first, some laboratory tests were carried out on flanged joint specimens. Later, a numerical investigation was carried out on the real model of the towers by varying the quality of the clamping bolt. The data obtained show how the buckling load is highly influenced by the correct erecting of the tower. In fact, a lower stiffness of the flanged link or an insufficient clamping bolt might produce safety coefficient lower than unity. Besides, the perfect accordance between numerical and experimental investigation gave us the possibility of a more accurate design of the temporary towers.

1. Introduction

The bridge construction techniques commonly used might be different, but the one called "incremental launching method" seems to be the most used. In this method, the segments of the bridge deck are cast in place in short length of between 10 to 30m in stationary formwork located on the part of bridge already erected. This technique is the one used for the "Porcaria Bridge" building. This bridge is characterized by a deck span of 70 mt constituted by by six short units obtained by splitting the roadway according to the symmetry axes, in longitudinal direction, and then, by dividing these elements in three units each, in transversal sense,. Each unit is assembled directly against the previous one by means of a special fabricated structural steel launching nose attached to the leading edge.

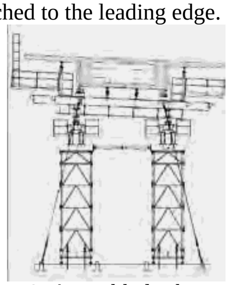

Figure 2. Assembled scheme of temporary piers

Figure 1. Real steel truss pylons in site

During construction of the long spans, temporary piers are provided to keep the cantilever moment to a reasonable magnitude. After launching is complete, and the opposite abutment has been reached, additional pre-stress is applied The temporary piers used, in

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that case, were two steel pylons constituted by modular truss structure. The profiles employed were L100x100x10 for the column, and IPE 80 as beam (fig. 1-2). The columns were finally redoubled and connected to each other by means of a cross stiffening bracket realized by UPN100 profile. Thus, during the bridge construction, the pylons were, continually, taken off and reassembled. The study was started after the collapse of the bridge during a construction phase. The crash, probably, had been caused by the failure of these temporary piers due to some mistakes during the truss assembling phase. In fact, the mapping of the bolted connection carried out on the real pylons had sometimes shown the presence of damaged and deformed elements, but, also often, either the absence of some bolts or their under tightening .
The rotational stiffness was obtained by applying an increasing cyclic load up to the damage of the sample on a laboratory specimens constituted by two modular elements .

Table. 1. Average stiffness from test to be used for numerical investigation

Direction	K (kN m)	$\overline{K}$ [kN·m)
+Y very under tigh.		
Y Real tightening	320,96	320,96
X Real tightening	95,63	95,63
+Y CNR tightening	161,05	176.42
- Y CNR tightening	191,78	
+X CNR tightening	481,88	592.09
-X CNR tightening	702,31	

Besides a specimen in real scale, capable of reproducing the real geometry of the central part of the temporary tower that had collapsed during bridge construction, was built in site and tested under compression. The scope was to evaluate the ultimate load of the provisional structure in respect to the design value. The failure appeared for a load equal to 2100 kN because a local buckling occurred. on the steel columns (fig. 17). The corresponding average displacement was equal to 50 mm.

The comparison between F.E.M. and experimental investigation showed a very good accordance in fact, numerical investigation gave us back the same critical deformed scheme of the experimental test ; corresponding to a buckling load of 1850 kN, very close to the experimental one (2000 kN). The whole tower was, finally, subjected to compression load by varying the quality bolt connection. It can be observed that in presence of a properly made temporary piers (model I) the buckling load under compression is wide greater while in the case of a good bolt tightening (model II) but still far from the fulfilment of the CNR rules, the bearing capacity of the system decreases, although it can be still considered as acceptable. In the case of a little assembling mistake, mainly, regarding the *M16 8.8* bolt tightening quality, a load redistribution occurred with the buckling load very close to the previous case studied. On the contrary, if the assembling mistake regards the *M24 8.8* bolt tightening quality used for flanged connections an excessive decrease of the buckling load is recorded up to 40% less. In other words, the study shows that in some cases the collapse load multiplier, that is equal to the safety coefficient for buckling failure, can be less than the unity. This determines a sudden unexpected collapse of the structure. This is mainly due to mistakes occurring during assembling phase of the temporary towers as the reduced stiffness of the flanged connection missing bolts or missing connection elements.
The load conditions applied to the real model of the tower, in accordance with the ones considered by the structural designer, were also studied but they are the object of other publications.

References

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