ROTATIONAL STIFFNESS OF TUBULAR STEEL-CONCRETE CONNECTON FOR ARCH BRIDGES

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Abstract

The springs of an arch bridge are intended to be entirely clamped and fixed, to ensure the necessary thrust force as well as to resist bending and torsion. In practice, the clamping of a steel tubular arch in a concrete abutment is not perfect and axial end rotations appear especially due to torsion. An alternative of using connectors of various types has been applied to few bridges. The reactions are each resisted by adequate types of connectors. To underpin this type of arrangement, a set of 3 scaled preliminary specimens was tested under torsion. Both strips and headed studs were used as connectors. The test results are linked to the more complex situation of real arch springs. It can explain why during the construction, twice as much as the predicted value of the lateral arch displacement was measured in at least 2 bridge cases. Because of the flexibility of the strip connectors used, the angular rotation almost reaches a value corresponding to free end rotation. In the present state, it may be recommendable for design not to consider complete axial rotational fixity of the arch bridge springs considered.

Keywords: Tubular arch bridge, arch spring connection, torsion stiffness, scaled torsion test.

1. Introduction

The connection of springs to concrete abutments is of paramount importance for the load carrying capacity of arch bridges. At this location the various reactions from the bridge superstructure are transferred to the abutments. The most important reaction is the arch thrust force, together with the vertical reactions. Whether bending moments are transferred to the infrastructure at the arch springs, depends on the use of hinges or clamping. The fabrication of hinges certainly is more complicated than complete clamping of arch springs. In most cases, the concept of the arch spring connections corresponds to clamping and bending moment transfer.

A particular reaction is the torsion moment. Since the torsional capacity and stiffness of circular sections are rather high, large torsion clamping may be expected. As demonstrated further, in some conditions, this torsion stiffness may be rather low, depending on the type of connection being used. Connections with prestressing bars may appear to be an excellent choice to materialize the clamping condition, but failed for torsion in at least the application mentioned below. Therefore, the alternative of using connectors has been considered. In this case as well, the torsion stiffness proved to be smaller than expected. Because of this, a small number of tests was carried out and has given better insight in the issue.

2. Arch spring connection

2.1. Prestressing bars

A fixed connection of steel bridge superstructure to concrete infrastructure immediately reminds the option of using prestressing bars. Since prestressing also implies friction, complete rotation-free connection will be the evident assumption. This type of connection was successful in the case of the steel tubular arch bridge of Merxem Street, reported in (Van Bogaert 2006). In the case of the bridge Woluwe Lane, the twin arches were built from 4 separate elements. This required implementing a procedure to erect and close the steel arches. The finished Woluwe Lane bridge is shown in fig. 1. Two independent tubular arches are equipped with double struts and upper crossbeams, supporting concrete deck slabs. Both arches are also connected by crossbeams, supporting a central lower deck. The 0.40 m thick concrete slabs are fixed to the abutments, thus creating an integral bridge.

Figure 1. Overview of Woluwe Lane Bridge.

After construction of the steel parts, the three heavy slabs had to be cast on site. The dead weight of the concrete acts directly on the steel structure, since no interruption of the subjacent road was allowed. Hence the weight of the concrete being poured is entirely supported by the steel arches. As the design showed that the arches would remain stable, no lateral support was needed in principle. During casting of the concrete slabs, the arches were not fixed in any particular way, the crossbeams of this deck being clamped to both arches. As a result, end bending moments are appearing at the connections of the crossbeams to the arches, together with vertical reactions. The clamping moments introduce torsion in both arches. Hence the latter would rotate about their axis.

During casting of the concrete, lateral displacements of both arches have appeared, varying from 44 to 39 mm, the maximum expected displacement being limited to 24 mm. Obviously, the unsuspected lateral displacements were connected to possible nonlinear behaviour of the slender arches. However thorough GNI-analysis, including a single-wave imperfection of 20 mm showed no particularly larger deformation than the results from linear analysis. This excludes the steel structure as a cause of the unsuspected phenomenon. At this particular stage, an increased torsion flexibility was explained by grease ingress between the steel base plate and the concrete abutment.

2.2. Connectors

After this rather unfortunate experience, alternatives were worked out for more recent cases of connecting steel arch tubes to concrete abutments, the general idea being to attribute a certain type of connector to resist one or several types of reactions. A picture of this principle is shown in Fig. 2. The strips, welded to the 1.6 m diameter tube are equipped with headed studs and resist tensile force due to bending, whereas the rings effectively resist normal thrust compression force. Shear is transferred from the steel tube to the concrete abutment by the interior concrete and limitation of contact stresses. As for the torsion moment, it is resisted by both the welded strips and the headed studs. In the case of Figure 2 the torsion moment was rather low and the torsion stiffness was not a real issue, since a huge end crossbeam counteracts any axial rotation.

The Moerbrug bridge, near to Bruges, is a steel tied arch, with springs located below the chord members. The connection to the concrete abutments is similar as in Fig. 2, a combination of strip and stud connectors. Unfortunately, the connectors are located inside the steel tube. Hence, their resistance is exclusively due to concrete strength. Several causes for the excessive deformations of the arch tops can be considered. One of them is the lack of torsion stiffness. Hence, it was suspected the lack of torsion stiffness, possibly combined with lateral movement of the connection, may be a source of excessive deformation.

Figure 2. Arch spring connectors.

Figure 3. Moerbrug Bridge with temporary bracings.

During casting of the bridge deck, the temporary bracing, connecting both arches was removed too soon. This temporary bracing can be seen in figure 3. Due to the prematurely removal, the concrete casting resulted in lateral displacement of the arch tops by 40 mm. The latter cannot be explained by the missing bracing alone, since calculations show that the lateral displacement should be limited to 21.04 mm. Hence, also in this case the connection showed insufficient torsion stiffness.

3. Scales tests on connections

3.1. Test aims and setup

The aim of the current research is to verify the torsion capacity of a steel tube connected in concrete by 2 different types of connectors. In particular, the strip and headed stud types were to be tested. In addition, it was expected that the tests would give useful indication on how to improve the concept of reinforcement at the connection of the tube and concrete. The 3 tests that were conducted are reported here in a summarized version. More details may be found in (Van Bogaert, Schotte & De Backer 2018). The aim of the test is to apply a pure torque to a steel tube, encased at its lower end in a concrete block and to determine the failure value of the torque, the rotation angle and the failure pattern. Since these tests were intended as a first approximation and mainly to detect what are the failure mechanisms in this type of connection, the samples were scaled and limited to 3. Possibly the results may constitute an introduction to a wider program.

The setup, shown in Fig. 4, consists of a truncated parallelogram concrete slab of 1.2 by 1.2 m and 0.15 m thickness with a minimum of reinforcing rebars. To reduce the torque magnitude and to use limited auxiliary equipment, the tests are a scaled situation of the real situation by a factor of approximately 10. Hence, the concrete itself was to be scaled as well and is in fact mortar.

Vertical UPN 160 profiles of S 235 are encased in the concrete slab and are serving as anchorage points to materialize torsion. Finally, each sample included a vertical steel tube 50/8 S 235 welded to a horizontal UPN 80 S 235 profile. The latter allows its connection by load cells to the vertical UPN profiles and thus the application of 2 equal horizontal forces, which act as a torsion moment at the base of the vertical tube.

3.2. Test without connectors

The first test was intended as a trial and also aimed to detect whether any natural bond would exist between the encased part of the steel tube and the concrete slab. Natural bond would certainly influence the resistance of the connection. In addition, it is unsure whether natural bond would simply add to the connector's resistance or have a more complex influence.

Figure 4. Test setup.

Hence, the steel tube end was encased in the concrete at 50 mm depth. This corresponds to the diameter of the tube. In real structures it is unlikely that a steel tube would be encased to this depth. Consequently, any natural bound would have been a disturbing factor for further testing.

The code (EN 1994-1-1) does not recognize any natural contribution to bond. In the present case the concrete strength obtained just before testing was rather low, since temperature was low with frost overnight. This conducted to results for $f_{ccub150}$ from table 1 and an average resistance of 11.2 MPa. This table also includes the values for tests 2 and 3.

Table 1. Concrete resistance $f_{ccube150}$ (MPa)

Sample	**1**	**2**	**3**
Test 1	**11.4**	**11.0**	**11.2**
Test 2	**17.4**	**16.6**	**17.4**
Test 3	**19.1**	**17.4**	**16.4**

Figure 5. Broken cylinder.

During the test it became evident that natural bond was inexistent. The weight of the equipment of load cells, hooks, turnbuckles and chains made the connection fail immediately. The torsion capacity due to bond was limited to 0.051 kNm. In addition, after removing the steel tube, a cylinder seemed to have broken from the internal concrete, as can be seen in Fig. 5. The torsion capacity of this broken cylinder is very low and can be estimated between 0.004 and 0.008 kNm. Obviously nor the failure of the cylinder nor bond can be of influence to the following tests.

3.3. Test 2 strip connectors

In the second test the steel tube end was equipped with 4 50 mm long strips with a cross section of 8*8 mm. Thus the steel tube reached a depth of 75 mm, since the last 25 mm was also encased in the concrete slab. This corresponds to the arrangement of Fig. 2, without stud connectors.

The connection should have failed for a design value of the torque equaling 0.613 kNm. The latter corresponds to yielding of the strips due to shear, taking into account the yield stress of 235 MPa. However, the yielding does not necessarily correspond to failure. It introduces large deformation and may initiate other failure mechanisms. The graph of Fig. 4 shows the relation of the angular rotation versus torque.

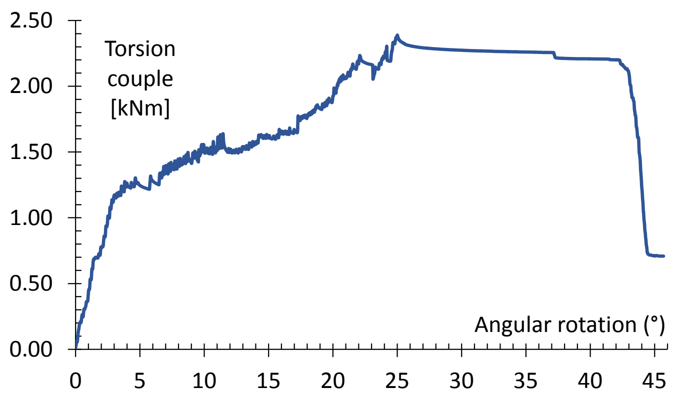

Figure 6. Load-time diagram test 2.

In this diagram several failure steps can be found. Already one may see that after a first linear phase, the curve deviates from the linear part and becomes flatter for a value of the torsion moment of about 1.3 kNm. This demonstrates that a new mechanism must be reached. From a torsion moment of 1.7 kNm the curve again becomes steeper and reaches a maximum for a torque of 2.39 kNm. After reaching the maximum the curve continues first with a horizontal part and rapidly growing angular rotation and finally descends steeply. The conclusion must be that 2.39 kNm is the failure moment.

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Figure 7. Excessive bending of strips.

Figure 8. Prints made by strips.

Some observations were made after this test. As Fig. 7 shows, the strips were excessively bent, although no real sign of cracking or fracture was found at the strip base. This demonstrates that failure of the steel strips alone did not occur during the test. In addition, after removal of the steel tube, the strip ends seem to have been fixed in the concrete, as can be seen in Fig. 8. Apart from some crushing at the edges, there is but little damage to these prints. This might suggest that a bending mechanism of the strips has existed, consisting of a support at the concrete side and a certain degree of clamping at the end of the steel tube

3.4. Test 3 stud connectors

In the 3rd test, an identical tube is equipped with 2 headed stud connectors. The bolts were welded to the tube and were intended to be encased at a depth of 25 mm. A detailed picture of the bolts and tube can be seen in Fig. 9. The total length of the bolts equals 60 mm, including the head of 13 mm. The steel grade differs from classical stud connectors, albeit the value of f_u = 400 MPa is close to the usual 420 MPa.

The predicted value of this connection was based on either the crushing of the surrounding concrete or shear of the connector shaft. As for the torsion moment, the lever arm would be determined by the tube diameter. Thus the predicted value reached 0.98 kNm. As for the previous test, a diagram of the torque versus the angular rotation was determined. The graph is shown in Fig. 10.

Figure 9. Stud connectors.

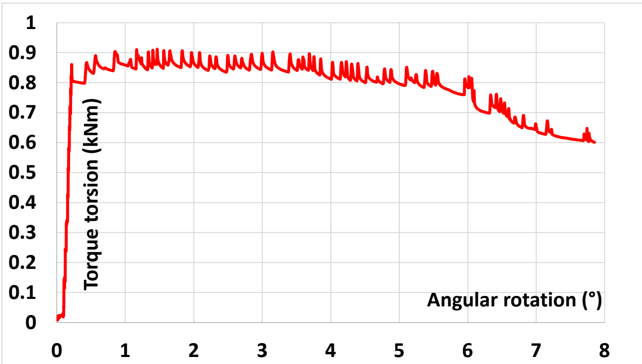

Figure 10. Torque versus angular rotation.

Figure 11. Concrete failure of stud connectors.

In the first part of the diagram a linear relation of the torque with the angular rotation is ending at the maximum value of the torsion moment of 0.825 kNm. This is definitely lower than the predicted value. From there the curve start descending while the rotation increases rapidly. The torsion moment decreases till it reaches a value of 0.7 kNm. During this phase cracks start appearing at the concrete surface and the latter is spalling. After this phase the curve further descends more rapidly and apparently a new mechanism is starting. The rotation can further increase for a lower value of the torque, while the concrete spalling is further increasing. Finally, cracks become general and the torque drops from a value of 0.55 kN at the end of the curve. This is the final failure point. After removal of the spalled concrete the end situation becomes clear as in Fig. 11. The studs are not broken, but heavily bent. There is no apparent cracking of any of the steel parts and thus the concrete part must have failed.

4. Implications of test results for structures

In the diagram of Fig. 12 both curves have been combined and limited to the lowest values of the angular rotation. The difference between both curves is striking, the rotations being totally unsimilar. From the data the specific rotation angle can be determined.

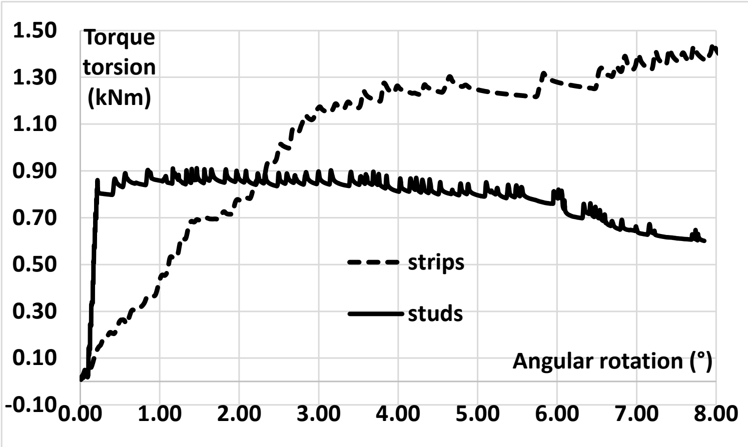

Figure 12. Diagrams for studs and strips combined

Strip connectors are reputed to be of the stiff type and more flexible than studs, which are generally considered as flexible. The diagram proves the opposite for this type application, since the torsion stiffness for the strip connection equals 0.541 kNm²/rad, whereas for the stud connection it reaches 2.868 kNm²/rad. In addition, the values may be compared to GC = 37.6 kNm² of the steel tube, which is respectively 70-times and 13-times stiffer than the strip- and the stud connection.

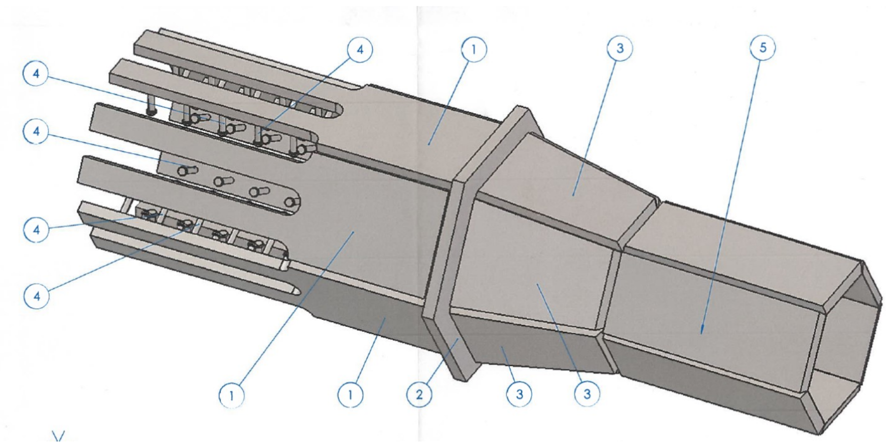

Figure 13. Connection of Moerbrug bridge

Resuming the case of figure 3, its spring connection being shown in figure 13, for the loading case that caused the lateral displacements, the torsion moment equals 130.4 kNm. Should the axial rotation of the arch springs not be prevented, for the same load it takes a value of 0.0026 rad. If the angular rotation of stud connection is scaled to this case by the parameters of stud diameter, active length, concrete strength, tube diameter and number of studs, a value of the rotation of 0.000244 rad is found. However, the torsion may also result in the separation of an internal cylinder as in Fig. 3. If the torque for breaking of this cylinder is also scaled to the bridge size, the failure value corresponds to 15.46 kNm. Hence only the strips and those studs that are located on the strips are active. Again, the flexibility of the strips can be recalculated from the test values and would allow angular rotation of 0.008637 rad, which is larger than for free axial rotation.

These data lead to the conclusion that the internal studs have little contribution to the arch rotational clamping. Hence, the strips encased in the concrete abutment have the largest contribution to the limitation of arch spring rotations, the latter being as high as for no torsional clamping. This indicates that the connection with internal studs may be ineffective for torsion.

5. Conclusions

The 3 tests have demonstrated that excessive axial rotation of springs of arch bridges, connected to concrete abutments can hardly be avoided. Strip connectors show large flexibility, whereas the stiffer stud connectors still show 13-times less stiffness than the base tube. In addition, the connection with prestressing bars was also unable to prevent the axial rotation, due to torsion. Clearly, a wider experimental research may provide larger insight in the torsional clamping of steel tubes in concrete. In the present state, it may be recommendable for design not to consider complete rotational fixity of the arch bridge springs considered.

References

EN 1994-1-1 2004. *Eurocode 4 Design of composite steel and concrete structures - Part 1-1 :General rules and rules for buildings (+AC 2009).* European Committee for Standardisation. Brussels.94-1-1: 2004

Van Bogaert, Philippe, Schotte, Ken & De Backer, Hans (2018). Post failure torsion capacity and robustness of encased tubular arch spring connections. *Proc. 6th Int. Symposium on life cycle Civil engineering. (To be published).* IALCCE

Van Bogaert, Philippe. (2006). Design and Construction of the Merxem Bridge – A Single Tubular Arch. *Proc. 6th Int. Symposium Steel Bridges* (pp. 56–65). ECCS/CECM/EKS.