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LOAD CARRYING CAPACITY OF SHEAR WALL T-CONNECTIONS REINFORCED WITH HIGH STRENGTH WIRE ROPES

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ABSTRACT

Traditionally, U-bar loop connections with keyed joints have been used in vertical shear connections between precast concrete wall elements. However, in the recent years, connections with looped high strength wire ropes instead of U-bar loops have proven to be a much more construction-friendly solution. The wire ropes have no bending stiffness and therefore allow for an easier vertical installation of the wall elements. During the last 10 – 15 years, a number of shear tests on plane wire rope connections have been carried out. However, to the best knowledge of the authors, tests on wire rope connections for assembly of precast elements in different planes, such as T- and L-connections, have not yet been published. This paper presents the results of a large test series recently conducted at the University of Southern Denmark to study the shear behaviour of high strength wire rope T-connections grouted with mortar. The test results are evaluated based on the experiences gained from test and modelling of similar plane connections. It is found that T-connections, in contrast to the plane connections, are more prone to fail by rupture of the brittle wire ropes rather than crushing of the joint mortar. Even so, it is found that a ductile post peak response can be obtained for T-connections, especially when so-called double wire boxes are used.

Keywords: Precast concrete elements, shear connections, wire rope loops, mortar connections

1. Introduction

This paper deals with the behaviour of shear connections between precast concrete wall elements. Traditionally, such connections are designed as U-bar loop connections with keyed casting joints. The U-bars are overlapped in the connection, which is usually grouted with mortar and provided with a longitudinal locking bar to enhance the ability of the overlapping U-bar to transfer tension.

In recent years, however, high strength wire ropes have been used instead of U-bar loops (see e.g. Kintscher, 2007; Bachmann & Steinle, 2011). The main reason is that use of wire ropes leads to more construction-friendly connections (the wire ropes have no bending stiffness and therefore make a vertical installation of a wall element much easier). In practice, the high strength wire ropes are pre-installed in so-called wire boxes, see Figure 1. Each wire box either contains one or two looped wire ropes. Wire boxes can be used to connect wall elements in the same plane or, as illustrated in Figure 2, to connect the ends of three wall elements into a so-called T-connection. The wire boxes are embedded in the ends of the wall elements and have an opening towards the connection. The boxes therefore function as shear keys when they are filled with mortar.

During the last 10 – 15 years, a number of experimental programs have been conducted to study the behaviour of *plane* wire rope connections, i.e. connections between two wall elements in the same plane (see e.g. Andersen and Poulsen, 2002; Frederiksen and Madsen, 2011; Hagsten, 2013; Joergensen, 2014). However, to the best knowledge of the authors, there exist no tests on the behavior of wire rope connections between precast elements in different planes such as T-connections and L-connections.

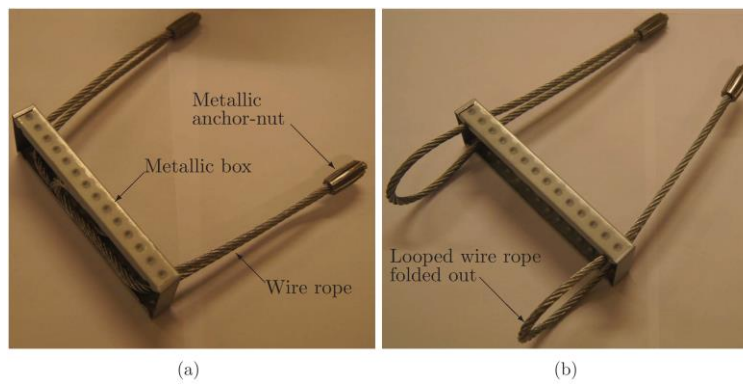


Fig. 1. Picture of a double wire box (a) before and (b) after the looped wire ropes are folded out

This paper presents a large experimental study on the behaviour of looped high strength wire rope T-connections. The tests were recently carried at the University of Southern Denmark. The overall aim of the tests was to investigate how the behaviour of T-connections differs from that of the *plane* connections (which have been the subject of previous studies). In particular, the tests were performed to investigate whether or not T-connections are more prone to fail by rupture of the wire ropes rather than fracture of the joint mortar. From a practical point of view, the later failure mode is preferable since high strength wire ropes have a stress-strain relationship which completely lacks of ductility (Joergensen, 2014). From the previous studies of *plane* wire rope connections, it was found that wire rupture can occur when mortar with compressive strength larger than approximately 60 MPa (Joergensen, 2014) is used to grout the connections. When using normal strength mortar, the tested *plane* connections failed by crushing of mortar and generally displayed a reasonable ductile post peak response.

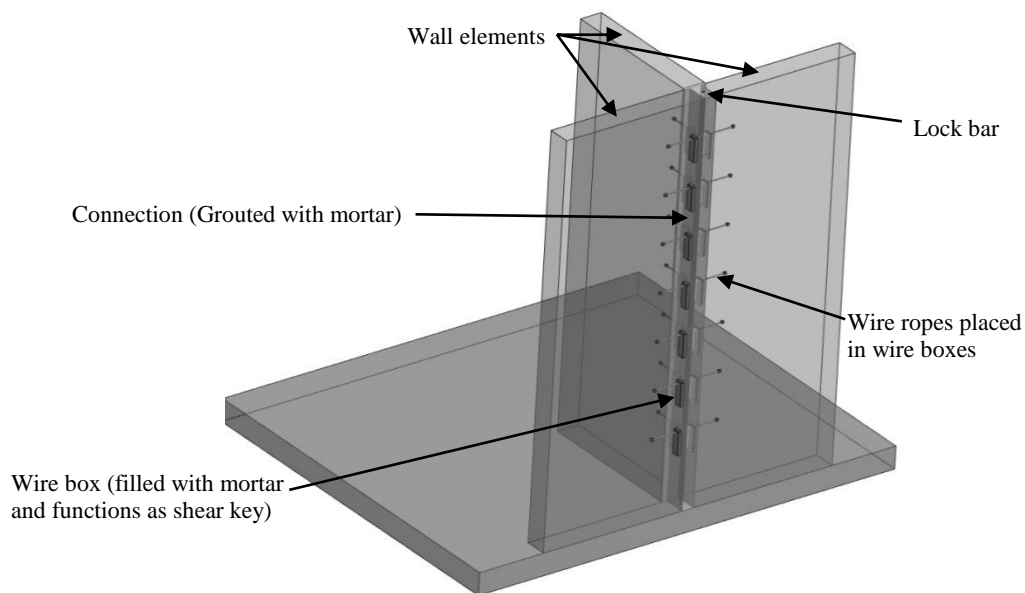


Fig. 2. Drawing of wire loop connection between three wall elements; a so-called T-connection

2. Experimental Programme

The objective of the experimental programme was to study the shear behaviour of connections reinforced with looped high strength wire ropes. The study involves connections between precast concrete elements assembled in so-called T-connections, where the ends of three precast elements are connected to enable mutual shear transfer. Details of the experimental programme may be found in the background documents by Bryndum and Larsen (2016). The programme comprised of 24 test specimens. Since such tests (to the best knowledge of the authors) have not been published before, it was important only to vary one

mechanical/geometrical parameter at a time. In this way important design parameters could be studied. The test programme was designed with inspiration from the parameter variations in the published tests on plane wire rope connections. In this way, the test results of T-connections could be better related and compared to the results of the plane connections.

2.1 Specimen geometry and material data

Figure 3 shows the general layout of the test specimens. Each specimen consisted of three precast elements connected to each other by a joint mortar. The two opposing precast elements were designed as rectangular elements whereas the third precast element was designed as an L-shaped element to enable a test setup, which simulate the so-called shear push-off tests used for plane joints.

Figure 3(c) shows the general layout of the connection. The looped wire ropes from the two opposing precast elements overlap and form approximately a circular core with diameter D_1 . The third precast element was situated with a greater distance to the centre of the connection and thereby needed a larger wire loop. Thus, the loop diameter of the wire rope from the third precast element, D_2 , was larger than D_1 . Before casting of the joint mortar, a longitudinal locking bar was placed inside the circular core, enclosed by the overlapping wire loops. Table 1 summarises the material and geometrical parameters of all tested connections.

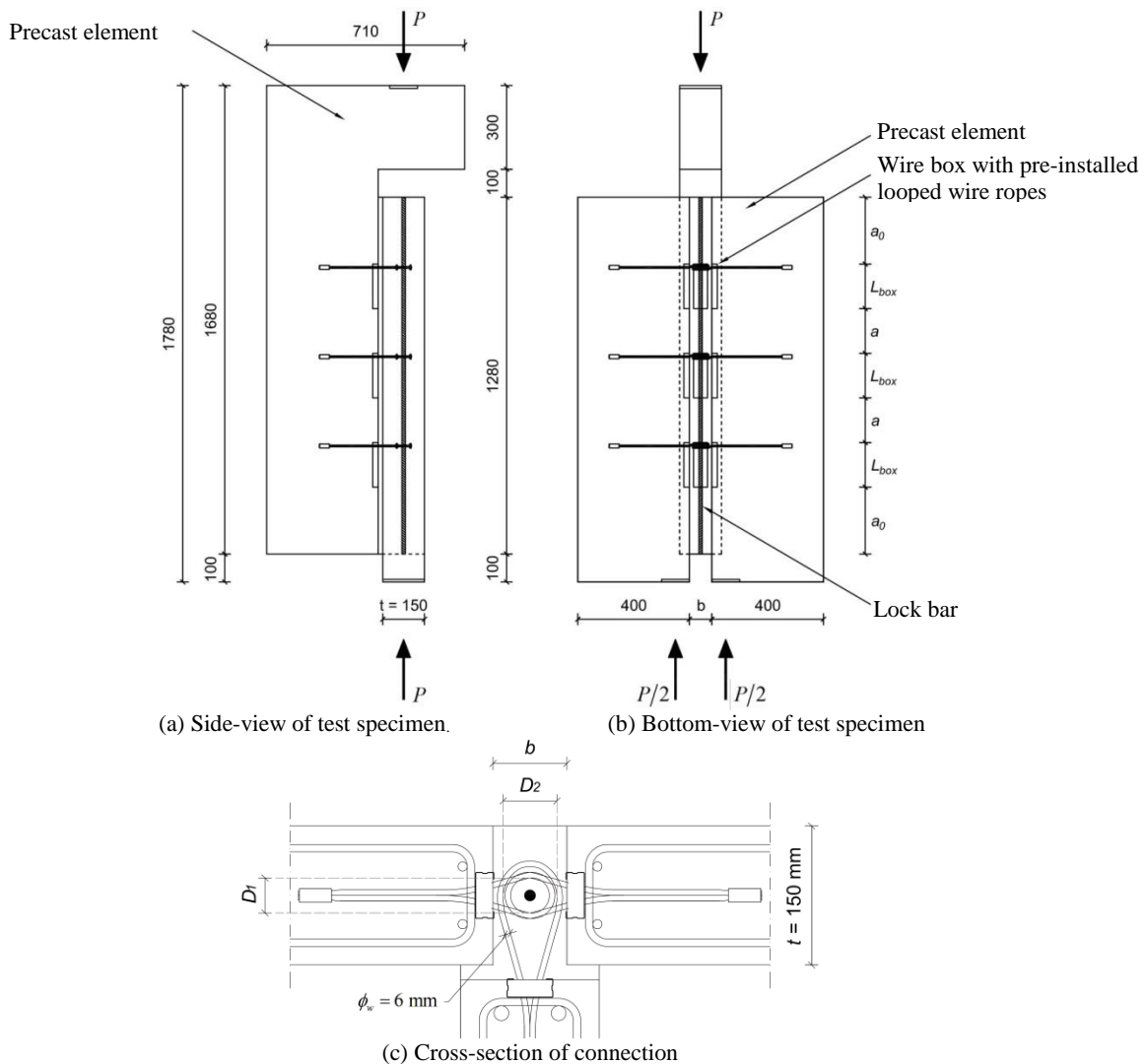


Fig. 3. Layout of the test specimens

Ordinary mortar mixtures were used to obtain three different mortar compressive strengths. The normal strength mortar (tested to approx. 35 MPa) had a maximum aggregate size of 2 mm. The other two mixtures had compressive strength of approx. 68 and 88 MPa and maximum aggregate size of 1 mm. Further details on the mortar mix compositions can be found in the background document by Bryndum and Larsen (2016). Mean values of the uniaxial compressive strength of the joint mortar for each test specimen may be found in Table 1. The uniaxial compressive strength of the precast concrete elements was between 46.2 MPa and 57.0 MPa. Although the strength of the joint mortar for some of the specimens was larger than the strength of the precast concrete, no indication of failure of the reinforced precast elements was observed.

In all test specimens the cross sectional diameter of the wire ropes was 6 mm and the tensile strength of a single wire rope was tested to 31.0 kN in average which corresponds to a strength of 1096 MPa. The tested stress-strain relationships for the wire ropes showed a nearly perfectly elastic behaviour without any yield plateau. The tested yield strengths of the lock bars, f_{yL} , are listed in Table 1. As can be seen, there is a small variation (from 578 to 620 MPa) depending on the diameter of the lock bar.

The test specimens were grouped in five series. Within each series, only one design parameter was varied (see Table 1). The tests were designed as duplicate tests; for example specimens 1A and 1B were designed to be identical. The design of the specimens was varied with reference to specimens 1A and 1B. For example, the specimens in series 2 had approx. the same mortar strength and the same layout as specimens 1A and 1B with exception of the lock bar diameter, which instead of 12 mm assumed the values 0; 6 and 16 mm.

Table 1. Specimen details

Series ID	Specimen ID	n_{wire}	n_{box}	f_c [MPa]	a [mm]	a_0 [mm]	b [mm]	D_1 [mm]	D_2 [mm]	$b_{box} \times L_{box}$ [mm]	ϕ_L [mm]	f_{yL} [MPa]
1	1A	1	3	35.7	160	240	80	38	60	160 x 35	12	585.8
	1B	1	3	34.2	160	240	80	38	60	160 x 35	12	585.8
	2A	1	3	68.6	160	240	80	38	60	160 x 35	12	585.8
	2B	1	3	68.6	160	240	80	38	60	160 x 35	12	585.8
	3A	1	3	88.2	160	240	80	38	60	160 x 35	12	585.8
	3B	1	3	88.2	160	240	80	38	60	160 x 35	12	585.8
2	4A	1	3	31.3	160	240	80	38	60	160 x 35	6	578.5
	4B	1	3	34.2	160	240	80	38	60	160 x 35	6	578.5
	5A	1	3	34.2	160	240	80	38	60	160 x 35	16	619.7
	5B	1	3	35.7	160	240	80	38	60	160 x 35	16	619.7
	6A	1	3	35.7	160	240	80	38	60	160 x 35	0	-
	6B	1	3	35.7	160	240	80	38	60	160 x 35	0	-
3	7A	1	4	31.3	160	80	80	38	60	160 x 35	12	585.8
	7B	1	4	34.2	160	80	80	38	60	160 x 35	12	585.8
	8A	1	2	32.7	160	400	80	38	60	160 x 35	12	585.8
	8B	1	2	32.7	160	400	80	38	60	160 x 35	12	585.8
4	9A	1	3	31.3	160	240	100	46	60	160 x 35	12	585.8
	9B	1	3	31.3	160	240	100	46	60	160 x 35	12	585.8
5	10A	2	3	32.7	160	210	80	40	60	180 x 35	12	585.8
	10B	2	3	32.7	160	210	80	40	60	180 x 35	12	585.8
	11A	2	3	68.3	160	210	80	40	60	180 x 35	12	585.8
	11B	2	3	68.3	160	210	80	40	60	180 x 35	12	585.8
	12A	2	3	88.2	160	210	80	40	60	180 x 35	12	585.8
	12B	2	3	88.2	160	210	80	40	60	180 x 35	12	585.8

n_{wire} is the number of looped wire ropes in each box, n_{box} is the number of wire boxes in each joint surface, f_c is the tested joint mortar compressive cylinder strength, a , a_0 , b , D_1 and D_2 can be seen in Figure 3, $b_{box} \times L_{box}$ is the dimensions of the opening area of the wire box, ϕ_L is the cross sectional diameter of the lock bar, f_{yL} is the yield stress of the lock bar.

In series 3, the number of wire boxes embedded in each joint surface was either two or four instead of three, which was used in the reference specimens 1A and 1B. Series 4 only consisted of one type of specimen. Here, the difference from the reference specimens was the overlapping length of the wires in the two opposing specimens and thus the diameter D_1 . The overlapping length of the two opposing looped wire ropes was 60 mm in specimens 9A and 9B as compared to 40 mm in specimen 1A and 1B. This corresponded to a change of the loop diameter from $D_1 = 38$ in specimen 1A and 1B to $D_1 = 46$ mm in specimens 9A and 9B. With the increased overlapping length it was also necessary to change the width of the connection from $b = 80$ mm in specimens 1A and 1B to $b = 100$ mm in 9A and 9B. The loop diameter of the wire ropes from the L-shaped element (i.e. the element that are transvers to the two others) was $D_2 = 60$ mm in all test specimens.

Series 5 comprised of 6 specimens designed with double wire boxes (i.e. two looped wire ropes in each box). Except from the type of wire box, series 5 is similar to series 1 in the sense, that the uniaxial compressive strength of the joint mortar was varied.

2.2 Test set-up

The specimen configuration was designed in such a way that shear push-off test of the connections could be conducted. The load was applied in a horizontal testing frame, see Figure 4, where the test specimen was placed with the L-shaped element sticking vertically up. The load was applied through an integrated load cell to the head of the L-shaped element. In the other end the two opposing rectangular elements was supported. Each of the two supports consisted of a roller to allow for rotation of the elements. The roller was supported by a steel plate placed on two Teflon plates with grease in between to avoid friction in the support. Additionally steel support and load plates were embedded in the precast elements (see also Figure 3). The two support plates and the load plate embedded in the L-shaped element had an area of 100 mm x 150 mm (note that the thickness of the elements was 150 mm). Thereby the centre of the support plate was placed 50 mm from the edge of the connection.

Figure 5 shows a picture of the test arrangement. On one side of the L-shaped element, the relative displacement between the elements was measured by displacement transducers. On the other side of the L-shaped element, the relative displacement was monitored by use of digital image correlation. Two cameras were used for this purpose. On the bottom side of the test specimen the relative displacement between the two rectangular elements was measured.

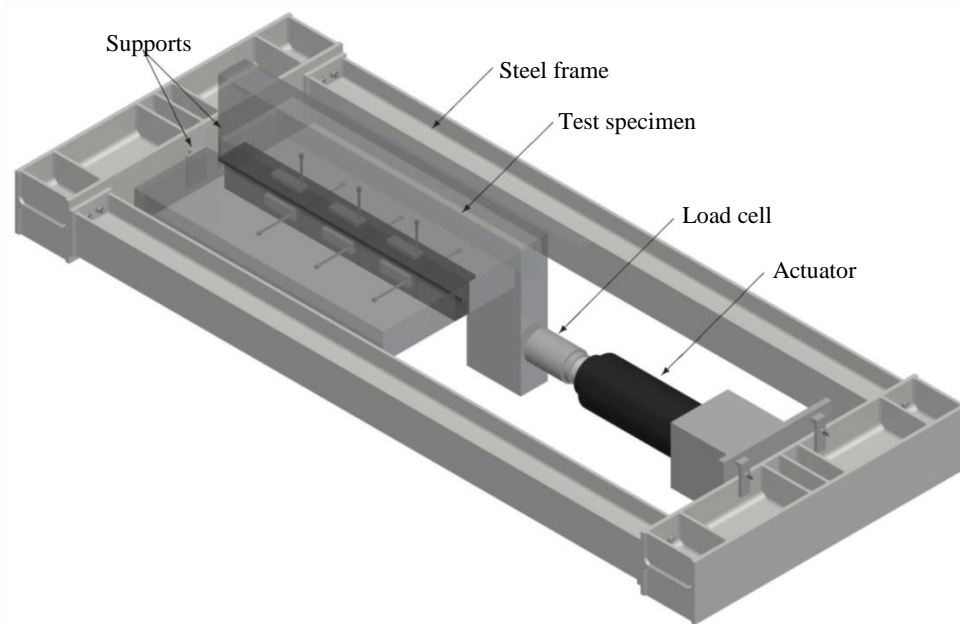


Fig. 4. Illustration of the testing frame with a test specimen installed

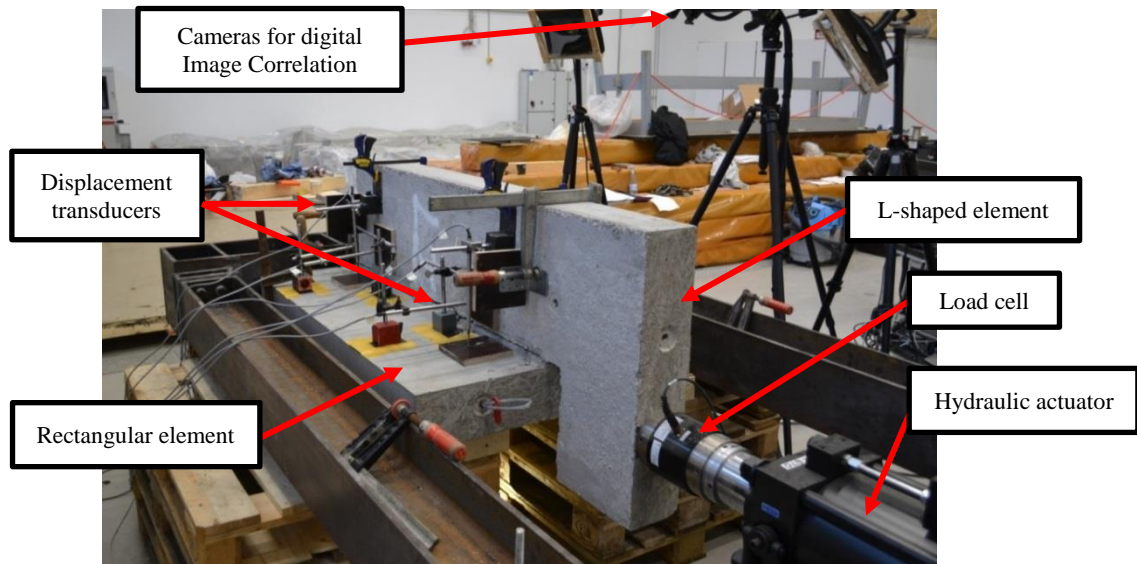


Fig. 5. Picture of a test setup with a test specimen installed

Below the test specimens, a digital video recorder was placed to observe if and when cracking occurred in the joint surface between the rectangular elements and the connection. Specimens 4B and 7B were, however, conducted as upside-down tests so that the L-shaped element was sticking vertically down. For these tests, Digital Image Correlation was used on the top-side which in this case was the two rectangular elements. All tests were displacement controlled by the hydraulic actuator with a speed of 0.5 mm/min until the first crack appeared between the connection and the L-shaped element. This point was easily found since a local peak load appeared at this stage. After appearance of the first crack the speed of the actuator was increased to 2.0 mm/min.

3. Experimental Results

3.1 Tested ultimate load and observed failure modes

The tendencies observed in the published shear tests on plane connections (Andersen and Poulsen, 2002; Frederiksen and Madsen, 2011; Hagsten, 2013) have formed the basis for the examination of the present tests. From the published plane connection tests it was generally found that a 1st peak load occurred when cracking of the joint interface took place. This load is therefore referred to as *the cracking load*. As the shear displacement increases, the load usually drops and increases again until another peak load occurred. This peak is termed *the failure load* because beyond this point, the load-displacement curve displays a descending branch without recovering of the load carrying ability (see Joergensen, 2014). Similar tendencies have also been observed in the present tests. Figure 6 shows, as an example, the 1st peak load, P_{crack} , and the failure load, $P_{u,test}$, as obtained from testing of specimen 11A. Although the 1st peak load is larger than the failure load in many of the tests, the definitions will be used as shown.

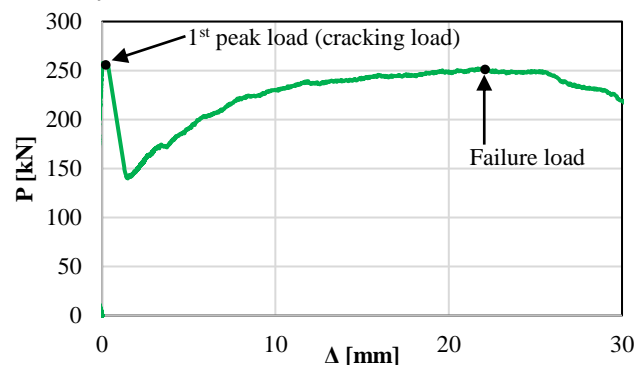


Fig. 6. Load-deformation response with indication of 1st Peak load and failure load

Table 2 summarises the tested 1st peak load as well as the failure load. It appears that the 1st peak load, P_{crack} , is almost identical for most of the tests whereas the failure load, P_u , depends more on the variation of parameters. As indicated in Table 2, wire rupture was observed after testing in many test specimens. However, wire rupture happened at different stages of the tests. In Table 2, “YES” means that the failure load was governed by wire rupture (i.e. rupture of wires took place at the load P_u). On the other hand “NO” means that rupture of wires was not observed at all or if observed, then it was a post-peak phenomenon which took place sometimes after the failure load has been reached.

Table 2. Test results

Series ID	Specimen ID	Variation parameter compared to 1A & 1B	1. peak $P_{u,test}$ [kN]	Failure load $P_{u,test}$ [kN]	Mean of identical $P_{u,test,mean}$ [kN]	Wire rupture (Yes/No)
1	1A	-	267.0	115.0	114.3	Yes
	1B	-	255.9	113.5		No ¹⁾
	2A	$f_c =$	240.8	109.4	119.1	No ¹⁾
	2B	68.6 MPa	246.7	128.8		No ¹⁾
	3A	$f_c =$	265.3	147.8	145.0	Yes
	3B	88.2 MPa	197.8	142.2		Yes
2	4A	$\phi_L =$	269.1	112.5	112.5	No ¹⁾
	4B	6 mm	192.3	-		No ²⁾
	5A	$\phi_L =$	231.1	123.9	151.9	Yes
	5B	16 mm	226.3	179.8		No ²⁾
	6A	$\phi_L =$	255.5	121.3	125.6	No ²⁾
	6B	0 mm	265.0	129.9		No ²⁾
3	7A	$n_{box} = 4$	257.0	162.6	153.8	Yes
	7B		234.2	144.9		Yes
	8A	$n_{box} = 2$	203.2	86.3	87.5	Yes
	8B		240.3	88.7		Yes
4	9A	$b = 100$ mm	254.1	127.5	124.4	Yes
	9B	& $D = 45$ mm	266.2	121.3		Yes
5	10A	$n_{wire} = 2$	254.1	243.6	258.1	No ²⁾
	10B		297.0	272.5		No ²⁾
	11A	$n_{wire} = 2^*$	257.4	252.0	274.0	No ¹⁾
	11B		323.5	295.9		Yes
	12A	$n_{wire} = 2^*$	285.4	268.1	253.0	Yes
	12B		294.7	237.8		Yes

* With exception of the shown variation parameter, Specimen 11 (A&B) are identical to specimen 2 (A&B) whereas specimen 12 (A&B) are identical to specimen 3 (A&B)

¹⁾ Wire rupture appeared sometime after failure load

²⁾ No wire rupture was observed when the connections was opened after test

3.2 Load-deformation response

Figure 7 presents a number of representative load-displacement curves. The displacement is taken as the relative displacement between the L-shaped precast element and the connection. This also corresponds to the relative displacement between the L-shaped elements and the rectangular elements (i.e. the rectangular elements in most tests did not move relative to the joint mortar).

Figure 7(a) shows the response curves for the three test specimens in series 1. The initial part of the response curves is seen to be very similar. It also appears that the failure load increases for increasing compressive strength of the joint mortar whereas the ductility of the connection seems to decrease for increasing mortar strength. It should be mentioned that right after the 1st peak load, the load drops suddenly. The sudden drop of the load caused the entire steel testing frame to elastically retract. Thereby, since the test was displacement controlled by the actuator movement, the elements experienced a sudden deformation. The sudden deformation may have changed the response curve such that it actually drops more than it would have done if the testing frame had been stiffer to accommodate a more correct static displacement control test. However, the authors believe that this only affected the response curve locally.

Since wire rupture has been observed in all specimens in series 1 (when breaking up the joint mortar after testing) it is interesting to study the response curve to see when wire rupture actually happened. For specimen 3B a rapid drop on the response curve is observed immediately after the failure load P_u . This indicates that failure was governed by rupture of the wire ropes. This is further confirmed by the fact that a loud and distinct sound was noticed at the failure load. For specimens 1B and 2B, on the other hand, the response curve appears to be more ductile after the failure load has been reached. This indicates that P_u was not triggered by rupture of the wire ropes. For these tests, the same loud and distinct sound was registered a while after the failure load has been reached.

Figure 7(b) shows the load-displacement response curves for test specimens in series 2. Here, the cross sectional diameter of the lock bar was varied. Generally, the 1st peak load in this series was very similar for all tests. Additionally, it can be seen that the influence of the lock bar diameter is not very significant. However, it is observed that the response of the test specimen without a lock bar is less ductile.

Figure 7(c) shows the response curves for the test specimens in series 3. In this series the number of wire boxes embedded in each joint surface was varied from 2 to 4 boxes. Again, the initial parts of the response curves are similar. After the 1st peak load it is seen that the failure load increases with increasing number of wire boxes. It appears that there is an approximately linear relationship between the number of wire boxes and the failure load. For specimen 8A a rapid drop on the response curve is observed immediately after the failure load. For specimen 7A such a drop on the response curve was observed prior to the failure load. For both tests, loud and distinct sounds were registered at the time of these drops. For specimen 1A, several small drops on the response curve are observed. During the tests, it was noted that a lot of these drops corresponds to a loud sound which indicates wire rupture.

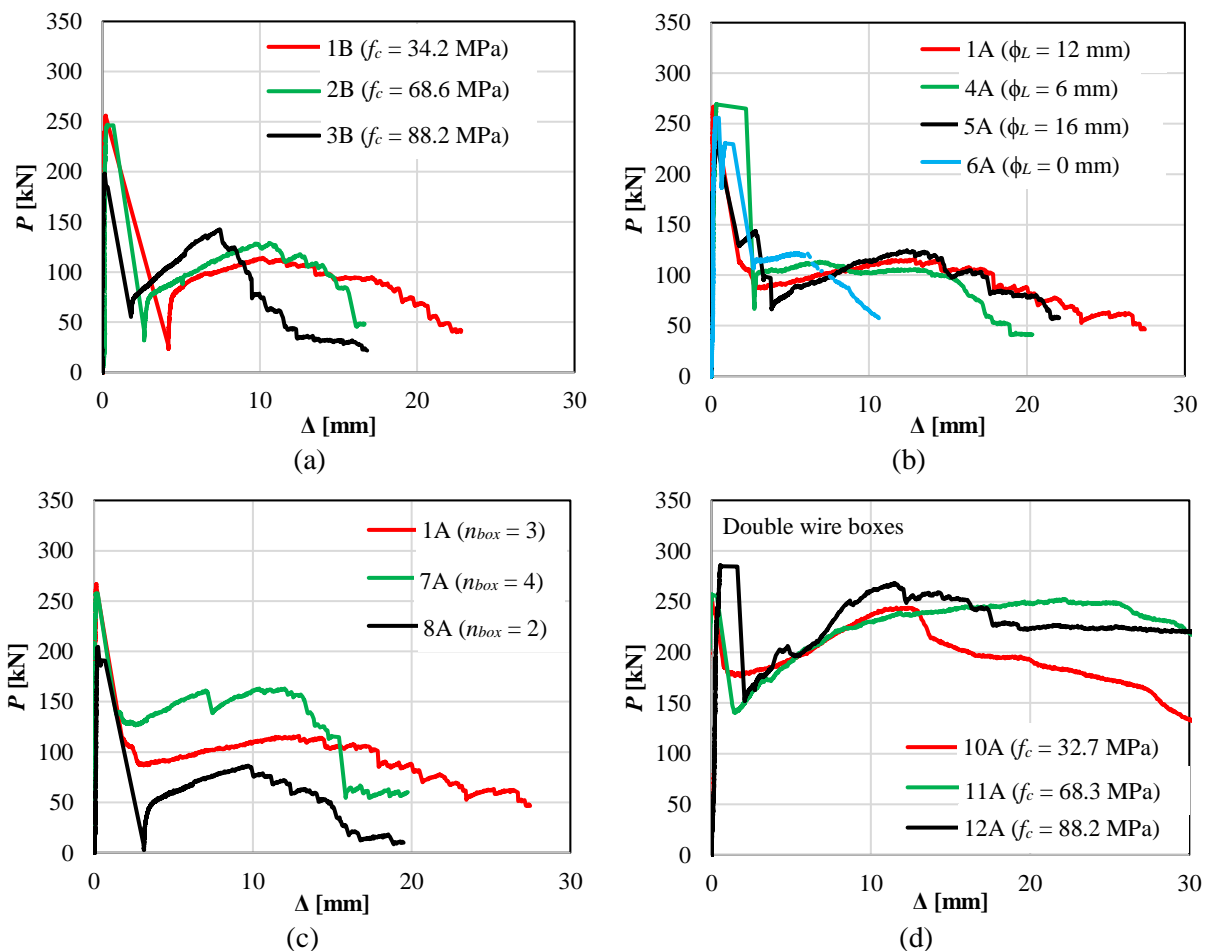


Fig. 7. Typical Load-deformation response curves

Figure 7(d) shows the response curves for the tests with double wire boxes. It appears that these response curves (after the 1st peak load) are more smooth and ductile as compared to the response of connections with single wire boxes. For specimens 10A & 10B, wire rupture was not observed when the specimens were opened up after testing. On the other hand, specimens 11B, 12A and 12B with higher mortar compressive strength experienced wire rupture at the failure load (see Table 2). In this context, it is interesting to note that the ductility observed on the response curves seems to be unaffected by rupture of wire ropes (in fact, Figure 7d shows that specimens 11A and 12A behaved more “plastic” than specimen 10A). The reason for this must be the use of double wire boxes.

4. Comparison with the behaviour of plane connections

Previously, a semi-analytical shear strength model has been formulated for *plane* wire rope connections (Joergensen, 2014). An important requirement for the application of this model was that the shear strength of the connection must be governed by failure of the joint mortar and not as premature rupture of the looped wire ropes. The reason is that since wire ropes fail in a brittle manner then the failure of the connection will also be brittle, which has to be avoided in practice. A more ductile failure mode is obtained when crushing of the mortar is governing. The ductile behaviour is mainly a result of the tri-axial stress condition developed in the mortar.

To avoid premature rupture of the looped wire ropes, a model for calculating the maximum allowable compressive strength of mortar was proposed. This model predicts rupture of wire ropes when the mortar strength exceeds approximately 60 MPa for plane connections with standard wire boxes. However, the model cannot be used to predict premature rupture of wire ropes in T-connections. For the T-connections tested in this study, rupture of wire ropes seems to be topical already at lower mortar compressive strength (wire rupture was observed in series 2 and 3 where the mortar strength was less than 35 MPa). One reason for the difference might be that the joint mortar in a T-connection is confined by three concrete interfaces in contrast to the plane connections, where only two joint interfaces restrain the joint mortar leaving two free surfaces from where failure can propagate. This means that there is better condition in T-connections for developing triaxial stress states which can increase the strength of the mortar and thus making the wire ropes relatively weaker. It is here worth noting that in T-connections with double wire boxes, a pronounced ductile response has been observed even when premature rupture of wire ropes took place.

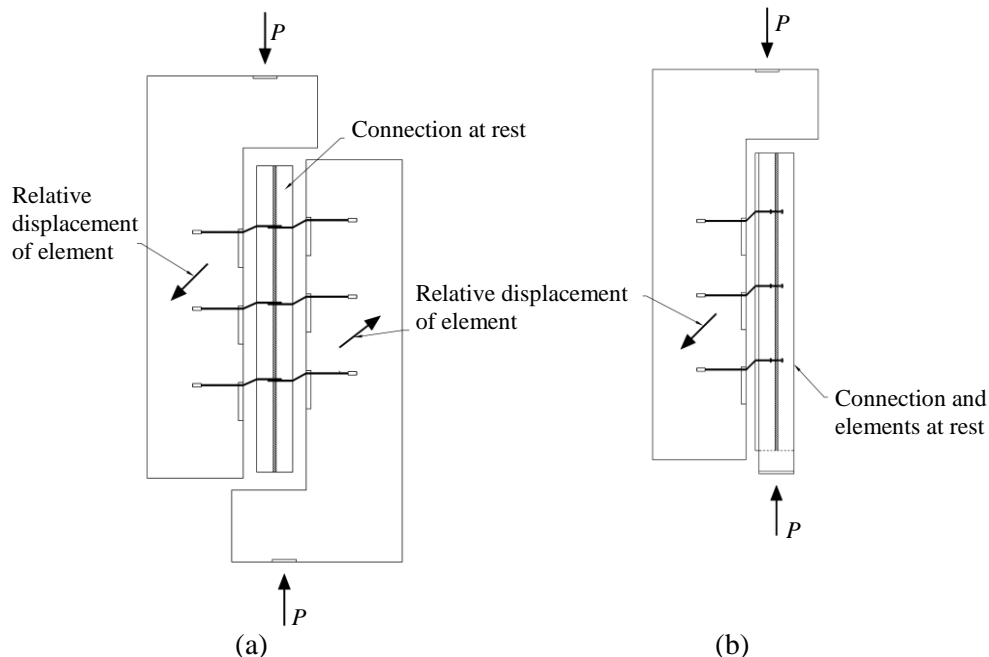


Fig. 8. Contributions to relative displacement for (a) plane connections and (b) T-connections

Another distinct difference between the behaviour of plane and T-connections is the measured displacement capacity, which seems to be smaller for T-connections. As shown in Figure 7 the relative displacement at the failure load is often between 10 to 15 mm in T-connections. For plane connections, on the other hand, the displacement at P_u is in the range of 20 to 30 mm (Joergensen, 2014). Also this difference seems to have root in the way the joint mortar is confined in the two types of connections. In plane connections, relative displacement is observed at both interfaces; see Figure 8(a). In contrast to this, the relative displacement is solely concentrated at the interface of the L-shaped elements whereas the two opposing elements almost always follow the joint mortar as one rigid unit. This effectively means that for the same relative displacement between the precast elements, the kink in the wire ropes and thereby the local deformations in the mortar must be larger in T-connections than in plane connections. This leads to either rupture of wire ropes or crushing of mortar at a smaller relative displacement in T-connections than in plane connections.

Finally, it should be pointed out that the cracking load is larger than the failure load for almost all the tested T-connections. This is at variant with the behaviour of similar plane connections, where the failure load is usually larger than the cracking load.

5. Conclusion

Results of an experimental study on the shear behaviour of high strength looped wire rope T-connections for assembly of precast concrete elements have been presented. The study comprised of 24 test specimens. Such tests have, to the best knowledge of the authors, not been published before. The main findings of the experimental work are:

- Rupture of wire ropes in T-connections takes place at lower mortar strength than in similar plane connections.
- For T-connections with double wire boxes, a pronounced ductile and plastic behaviour have been observed even when rupture of wire ropes seems to have triggered the failure load.
- In contrast to plane connections, the cracking load in T-connections is almost always larger than the failure load.
- T-connections have smaller displacement capacity than similar plane connections.

From the test results, it seems that double wire boxes should be preferred when precast elements are to be assembled in T-connections. For future work, the obtained test results will be used as inspiration when developing models for prediction of: a) the cracking load; b) the failure load; and c) the criteria for premature rupture of wire ropes. In this regards, the previously developed model for plane connections will be used as a basis.

6. Acknowledgements

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