Tests and Design of Welded-bar Angle Connections of Precast Floor Elements

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Abstract

Welded floor-to-floor connections are widely employed in dry-assembled precast concrete decks with the main aims to stiffen the diaphragmatic action under horizontal loads and increase robustness of the construction. Bars or slugs are often preferred to plates for their smaller room, better allowance, and cost-effectiveness. Despite welded bar connections are mentioned in many official design codes, their design rules are not therein specified, which was only recently done in ISO Standard 20987:2019 on simplified design guidelines for mechanical connections between precast concrete structural elements in buildings. This standard is the result of EU’s Safecast Research Projects FP7-SME-2007-2 and GA 218417:2009, in the framework of which many structural connections of precast structures were tested and investigated at different scale levels, although specific local testing of welded bar connections was not carried out. Thus, the quoted ISO formulation lacked from direct experimental check. This paper aims at filling this gap by presenting the results of shear tests on floor-to-floor connections made by bars welded to rebar-anchored inclined angles. The tests were carried out considering anchorage rebars having different shape and diameter. Moreover, the validity of the currently available ISO design rule is commented, highlighting a disproportional degree of conservativeness referring to concrete spalling failure. An updated formulation is proposed based on a novel mechanical behaviour model matching the experimental observations and results.

1. Introduction

While these issues are commonly dealt with reinforced concrete topping in cast-in-situ and partially precast concrete buildings, floor-to-floor mechanical connections are alternatively employed in dry-assembled precast concrete buildings to increase the stiffness of the diaphragms when subjected to horizontal actions (Dolce et al. 1994; Tena-Colunga and Abrams 1996; Fleischman et al. 1998, 2005a, 2005b; Cleland and Ghosh 2002; Dal Lago and Ferrara 2018; Dal Lago et al. 2019) and to provide structural robustness (Dal Lago and Gajera 2021). Floor-to-floor welded mechanical connections are preferred to other typologies such as keyed joints (Menegotto and Monti 2005), or bolted steel connections based on plasticity or friction (Dal Lago 2017a, 2018a) to mutually connect elements with thin flanges, such as double-tee or box voided elements, mainly due to the limited thickness available, often as thin as what required to allocate a welded wire mesh with minimum concrete cover (about 50 mm).

Local experimental tests on floor-to-floor connections whose basic configurations are shown in Fig. 1 were carried out mainly with reference to flanges of double-tee elements concerning welded rebars over either exposed rebars [hairpin connection, Fig. 1(a)] or rebar-anchored flat plates as in Fig. 1(b) (Venuti 1970; Pincheira et al. 1998), welded plates over rebar-anchored flat plates as in Fig. 1(c) (Hofheins et al. 2002) and commercial inserts made by welded slugs over wing-bent steel plates as in Fig. 1(d) (Shaikh and Feile 2004; van de Lindt 2007).

A combination of the above was further tested in shear by Pincheira et al. (2005) and Cao and Naito (2009), and under axial loading by Naito et al. (2009). The test results were later summarised in Ren and Naito (2013), where also indications on proper numerical modelling were provided. Naito and Ren (2013) proposed an experimentally based method for the structural assessment of floor-to-floor connectors. Bent-wing steel plates were also tested by Hendricks et al. (2018) specifically considering vehicular loading, including fatigue assessment (Naito et al. 2019). Out-of-plane shear tests on similar connections installed on thick concrete slabs simulating bridge elements were also carried out by Porter et al. (2011).

Full-scale prototype testing of precast decks subjected to seismic loading was carried out by Schoettler et al. (2009) referring to double-tee elements connected with either hairpin with concrete topping, welded bar over stud flat plates, and bent-wing steel plates. Keyed and topped hollow-core decks were tested by Schoettler et al. (2009) under seismic loading and by Angel et al. (2019)
under cyclic loading. Untopped hollow core elements connected with either rebar or flat plate welded connections over rebar-anchored flat plates were tested cyclically by Pang et al. (2020).

This paper presents the results of local shear tests carried out on a specific floor-to-floor mechanical connection made with bar welding over inclined thick angles (Fig. 2) anchored to the concrete flange through a U-

Fig. 1 Basic configurations of floor-to-floor mechanical connections investigated in the past: (a) hairpin, (b) welded rebars over rebar- or stud-anchored flat plates, (c) welded plates over rebar-anchored or stud-anchored flat plates, (d) welded slugs over wing-bent steel plates.

Fig. 2 Basic configuration of floor-to-floor connection made with welded rebar over rebar-anchored thick steel angles: (a) 3D view of the mechanical connection, (b) vertical cross-section in the flange, (c) example of installation into a mould for double-tee elements.
bent rebar pre-connected to the angle through fillet welding in the front length and rear contact points. The use of inclined angles instead of flat plates allows for several advantages, including larger welding surface between anchor rebars and angle, ease of positioning in the mould, temporary support of the rebar to be welded, large tolerance allowance, co-axiality of the connected reinforcement, etc.

This type of connection, although widely employed in the precast industry of some areas, including Italy and Europe, had been rarely investigated regarding the characterisation of its mechanical behaviour. Welded headed stud connections employing thick steel angles (not inclined) were previously tested by Spencer and Neille (1976), although installed in precast thick walls with much larger concrete area than flanges of floor elements.

This specific connection arrangement under consideration was indeed already subjected to testing, although inserted in the large full-scale precast building prototype dry-assembled and tested under pseudo-dynamic seismic loading as part of the Safecast project (Negro et al. 2013; Dal Lago et al. 2018b), where it was applied to both box-section elements with lower flange and double-tee elements. Hence, no specific characterisation of its mechanical behaviour was derived. Nevertheless, the diaphragm behaviour was reported to be very efficient in the lower two stories (Bournas et al. 2013), where the floor-to-floor welded connections were employed, which highlights their efficiency in stiffening the diaphragm. In the same tests, the diaphragm action was reported not to be fully efficient at the roof level, where box-section elements with lower flange were placed distanced and, therefore, floor-to-floor connections could not be installed. In this configuration, indeed typical of large precast industrial buildings (Dal Lago 2017), the diaphragm behaviour only relies upon the roof-to-beam connections, usually made with dowel bars or bolted steel angles (Dal Lago et al. 2017b).

Despite these connections are cited in current codes and popular technical documents, including Eurocode 2 (EN 2004) and fib Bulletin 43 (fib 2008), official rules are provided only in the technical design guidelines resulting from the Safecast project (JRC 2012), recently merged into ISO 20987. This formulation analyses several failure modes, namely welding, plate, anchor loop, pull-out, and spalling. According to these formulae, concrete spalling results often critical for the design shear resistance of the whole connection at study. The experimental investigation shown in the present paper is also aimed at checking whether the relatively low resistance values associated to the ISO 20987 formula for concrete spalling failure is realistic or too conservative.

2. Specimens

Three tests were carried out referring to different characteristics of the embedded connector with the aim to investigate the influence of the rebar diameter and of different rebar bending angle over the shear resistance of the joint. The specimens were couples of concrete plates with plan dimensions of 1700 mm by 400 mm and 60 mm of thickness, hosting two connections at a time distanced by 900 mm. Their details are collected in Figs. 3(a), 3(b) and 3(c). Each of the 6 constructed specimens contained 2 connectors: specimens U6 had a $\Phi 6$ rebar bent at 180° in U shape; the connectors of specimens U8 had a $\Phi 8$ rebar bent at 180° in U shape; the connectors of specimens V6 had a $\Phi 6$ rebar bent at 90° in V shape. Each rebar was welded with a 4 mm electrode to a hot-rolled 80 mm length of a 40 mm by 40 mm equal side S235 steel angle of thickness 4 mm [Fig. 3(d)]. A welded wire mesh with 150 mm square grid of $\Phi 6$ rebars was placed centrally in the flange, and the connection device was placed over the bottom mesh rebars, for both U6 and U8 specimens. The device of V6 specimen was placed in the same position by moving downwards by 6 mm the welded wire mesh, since the V-shaped anchor rebar of the device overlapped with both longitudinal and transverse mesh rebars. All rebars were made with B450A steel grade.

The specimens were manufactured by a precast concrete producer [Fig. 4(a)]. Concrete, prescribed with class C45/55 (nominal cubic strength of 55 MPa), was measured to have cubic compressive strength equal to 69.9 MPa and 67.1 MPa from two standard tests carried out in the day of the first test (tests lasted for about 5 days), after about 3.5 months of hardening.

The welded joint was made in the LPMSC laboratory of Politecnico di Milano [Fig. 4(b)]. The lab operators placed 80 mm long $\Phi 16$ rebar pieces made with B450C grade steel in between the angles with the ribbed part facing top and bottom. Then, they welded the single rebars at the two sides in order to fill the gap between rebar piece and angle, first with a 2.5 mm electrode, and later with a 3.25 mm electrode, both made with steel having ultimate tensile strength higher than 550 MPa.

3. Test setup

Each specimen was subjected to a monotonic in-plane shear test under displacement control at a velocity of 0.02 mm/sec, corresponding to a quasi-static strain velocity range. The test setup is shown in Fig. 5(a). The specimen was placed over cylindrical rollers and restrained by simple contact with 6 rigid metallic braced angles tightened through bolts to the strong floor of the laboratory. 4 rigid angles restrained the lateral displacement and rotation of the specimen and were placed in the transverse direction with respect to the load direction, and 2 of them restrained one concrete plate in front and rear directions. The other concrete plate was pushed by a hydraulic actuator with capacity of 300 kN. The test setup was thus conceived with an eccentricity equal to half of the slab flange width (20 cm), which is equilibrated by a set of statically undetermined restraints, where the two welded connections distanced by 90 cm...
equilibrate the overturning moment by a reaction couple equal to $90/20 = 4.5$ times less than the jack load. These limited forces generate a biaxial compression state of stress in the concrete area located behind the angles of the connection, which is however not mobilised in the failure modes observed at the end of the test. Thus, it is assumed that the unavoidable presence of a load eccentricity did not play a relevant role on the experimental results observed. Concerning the instrumentation, the actuator was equipped with a 300 kN load cell, and several Linear Variable Displacement Transducers (LVDTs) were installed over the specimen, as shown in Fig. 5(b). LVDT-1, embedded into the actuator, measured its stroke and served as controller channel; LVDT-2 and LVDT-3 measured the absolute displacement of the pushed and fixed concrete blocks, respectively; LVDT-4 and LVDT-5 measured the relative sliding of the concrete plates in correspondence of the two welded connections; LVDT-6 and LVDT-7 measured the transverse displacement of the joint at front and rear of one connection, spaced by 300 mm. Each instrument was accurately calibrated and checked prior to the execution of the tests. A picture of the setup ready for testing is shown in Fig. 5(c).

Fig. 3 Details from shop drawings of the tested elements: (a) U6-angle welded to 6 mm dia. rebar bent at $180^\circ$ (units: cm), (b) U8-angle welded to 8 mm dia. rebar bent at $180^\circ$ (units: cm), (c) V6-angle welded to 8 mm dia. rebar bent at $90^\circ$ (units: cm), and (d) bottom detail view of the rebar-anchored angle insert for specimen U6 (units: mm).
4. Test results

The tests results are shown in Figs. 6, 7 and 8 considering monotonic response with absolute displacement values, with relative displacement values, and transverse deformation, respectively. The behaviour of the connections in all configurations is characterised by a peak load attained after a quasi-elastic branch at a relative displacement as low as 1.0 to 1.5 mm, followed by a quasi-linear gentle softening branch. The peak loads were very similar for specimens U6 and V6, which suggests that negligible differences in resistance are introduced by different shaping of the anchor rebar. Moreover, the peak load of specimen U8 was higher than specimens with Φ6 anchor rebar, although the difference is much less than proportional to the rebar area. For all specimens, to the peak behaviour was associated a sudden damage of concrete around the connection (Fig. 9), which can be defined as spalling. The post-peak behaviour of the specimens was different among them. In particular, the anchor rebar of one connection of specimen U6 deformed remarkably in combined shear and flexure with a dowel-like deformation profile, up to failure of the bar by ductile fracture in the section right below the connection with the rear angle side [Fig. 10(a)]. The transverse behaviour was characterised by a small rigid rotation, as noticed by the alternate directions of the readings of the transverse LVDTs [Fig. 8(a)], evolved into a similar opening trend at large displacement.

Fig. 4 Photographs of the specimens: (a) specimens V6 (top) and U6 (bottom) ready for concreting, (b) one connection welded and ready to be tested.
Fig. 5 Test setup: (a) general views, (b) instrumentation, (c) photograph.
A similar behaviour was observed for specimen U8, where the test was stopped after having attained a very large displacement with a clear dowel deformation of the anchor rebar at one side [Fig. 10(b)]. In this case, the progressive inclination of the rebar brought to a negative opening of the joint, with the concrete plates getting closer at large displacements [Fig. 8(b)].

The specimen V6 attained the larger deformation after the post-peak softening branch, probably induced by the ductile failure mode of the anchor rebar, which failed under combined shear and tension [Fig. 10(c)] induced by the decomposition of the shear load, which led to a progressive opening of the joint as a consequence of the tension component [Fig. 8(c)].

Figure 11 shows for each test the initial stiffness of the couple of connections, their resistance, and the slope of the softening branch. By looking at the practically identical relative displacement of the connections and at the rigid deformation of the concrete plates out of the locally disturbed area where the connections were installed, it appears logical to state that the contribution of the single connection in terms of both stiffness and resistance can be evaluated by simply dividing the total values by their number (in this case, by two).

5. Test interpretation and design rules

Observing the test results and the failure mechanisms of Fig. 10, it is evident that the failure mechanism at peak load involved concrete, with the anchor rebar subse-
quently failing with different mechanisms (mixed shear/flexure according to dowel mechanism for U-bent rebars and mixed shear/tension for V-bent rebars), which however occurred in all cases at remarkably lower loads with respect to the peak associated to concrete failure. The formulation proposed in the Safecast project guidelines (JRC 2012) and ISO 20987 for the concrete failure (spalling) of a single connection is re-

![Figure 9 Test results showing photographs of failure modes of specimens: (a) U6, (b) U8, (c) V6.](image)

![Figure 10 Test results showing detailed views of failed connection for specimens: (a) rebar failed for U6, (b) rebars highly deformed for U8, (c) rebar failed for V6.](image)

![Figure 11 Test results showing the strength and stiffness of specimens: (a) U6, (b) U8, (c) V6 (load and stiffness values refer to two welded connections).](image)
ported in Eq. (1).

\[ R_{d,ISO} = 2 \cdot a \cdot h \cdot f_{cd} \]  

(1)

where \( a \) is the lower between the side of the angle (indicated in Fig. 12) or the length of the angle profile \( b \); \( h \) is the depth of concrete mobilised, defined in this ISO 20987 formulation as the lower between twice the concrete cover of the anchor rebar and the flange thickness; \( f_{cd} \) is the design concrete strength in tension. Despite the number of connections in a row is not explicitly present in the above-quoted formulation, it is intended that the strength of \( n \) connections sufficiently distanced in a row could be obtained by multiplying by \( n \) the strength of a single connection.

This formulation is directly dependant on the tensile strength of concrete and, given the anchorage rebar is placed centrally with respect to the flange thickness, to the full thickness of the flange. Both the above parameters are apparently not consistent to describe the behaviour observed experimentally. As a matter of fact, the failure of concrete was related to compressive crushing rather than to tensile or tangential stress. Moreover, the area subjected to compressive failure never involved the full thickness of the flange, where in all observed mechanisms part of the lower half of the flange did not experience damage or failure. Additionally, the tests showed a less-than-linear dependency of the resistance upon the anchorage rebar diameter, parameter which is not included into the above formulation.

An alternative formulation is therefore proposed on the basis of the concrete compressive strength and of the definition of the concrete area involved into the resisting mechanism experimentally observed. Moreover, a formulation is also introduced regarding the evaluation of the shear elastic stiffness of the connections. By analysing the results, the joint behaviour appears to be quasi-elastic up to 75% to 80% of the peak load, followed by a short non-linear branch up to the peak (as shown in Fig. 12).
Fig. 11), and by a post-peak marked softening characterised by a relevant linear decrease of load and a progressively severe irreversible damage to the connection. Thus, the two parameters of stiffness and resistance appear to be sufficient to carry out a traditional elastic design of the connection, whilst more refined models may be implemented after calibration of the experimental curves for sophisticated design/assessment issues.

As anticipated, the resistance of the connection associated to concrete spalling is proposed to depend upon the product of the concrete compression strength \( f_{ck} \) by the concrete area mobilised \( A_c \). Assuming a stress concentration close to the edge of the concrete flange by interpreting the connection as a rigid body acting over a series of springs representing the concrete bed and rotating around a base hinge joint located at a length \( L \) from the edge [Fig. 12(a)], the proposed shear resistance of \( n \) connections in a row associated to concrete spalling \( R_{c,pr} \) may be defined as follows:

\[
R_{c,pr} = n \cdot 0.67 \cdot A_c \cdot f_{ck}
\]  

(2)

The resultant associated to elastic behaviour \( R_{c,pr,el} \) may be defined as follows:

\[
R_{c,pr,el} = n \cdot \frac{A_c}{2} \cdot \sigma_{c,\text{max}}
\]  

(3)

where the latter equation [Eq. (3)] is associated with a triangular linear stress distribution on concrete with maximum stress \( \sigma_{c,\text{max}} \), and the former one [Eq. (2)] to a non-linear parabolic stress distribution of concrete, following the instructions of Section 3.1.7 of Eurocode 2, where due to the unconfined state of the cover concrete volume subjected to spalling, failure is set at the onset of the rectangular branch of the well-known parabola-rectangle constitutive law at a strain \( \varepsilon_{c,2} \) equal to 0.2% for normal performance concrete. The coefficient 0.67 in Eq. (2) is the integral of the parabolic stress-strain formulation given in the above-cited chapter, assuming unitary strength and maximum strain.

The area of concrete mobilised can be defined according to a trapezoidal shape as indicated by the redline hatch pattern in Fig. 12(b), including the whole steel profile, and further simplified into a rectangle (within the blue lines in the same figure) of sides \( L \) and \( h \) as follows:

\[
A_c = L \cdot h
\]  

(4)

where \( h \) is the height of the bottom of the anchor rebar from the top flange surface.

The length \( L \) of the mobilised concrete area depends upon the projected length of the angle profile, to which a contribution due to the anchor rebar is to be added. If considering for the latter contribution a length equal to 4 times the rebar diameter \( \Phi \) for the effective rebar length subjected to dowel action, it can be written as follows:

\[
L = 4\Phi + a \cdot \cos(\theta)
\]  

(5)

Moreover, the height \( h \) of the bottom of the anchor rebar from the top flange surface can be referred to the geometry of the angle and to its inclination as follows:

\[
h = \Phi + (a + t) \cdot \sin(\theta)
\]  

(6)

where \( t \) is the thickness of the flat part of the angle profile.

The design formula can therefore be rewritten as follows, after having introduced the proper material safety coefficient \( \gamma_c \) and referring to the nominal characteristic strength of concrete \( f_{ck} \) (the coefficient \( a_c \) related to the long-term strength of concrete is not included since the shear load in the connection in service is generally null or negligible):

\[
R_{c,pr} = n \cdot 0.67 \left[ 4\Phi + a \cdot \cos(\theta) \right] \left[ \Phi + (a + t) \cdot \sin(\theta) \right] \frac{f_{ck}}{\gamma_c}
\]  

(7)

It is also reminded that, in case of capacity design of the connection under seismic loading, an additional reducing factor related to overstress \( \gamma_c \) associated to the design ductility class of the overall structure may be employed.

The concrete spalling resistance calculated with either the ISO 20987 [Eq. (1)] or with the proposed formulation [Eq. (7) above] is compared with the experimental results in Table 1 referring to either the mean concrete material properties (the mean cubic strength derived from tests was \( R_{cm} = 68.5 \) MPa, so the mean cylindrical strength considered is \( f_{cm} = 56.9 \) MPa) or the design concrete properties associated to a concrete class C45/55 (\( f_{cd} = 30.0 \) MPa). Both mean and design tensile
concrete strengths are taken as per Eurocode 2 for the nominal concrete class C45/55. The mean concrete strength is intended to be used for comparison with experimental tests, and the design concrete strength is intended to be used to determine the safety margin at design stage. The comparison shows that the ISO 20987 formulation, based upon geometry and material properties not fully consistent as previously discussed, provides mean resistance values $R_{cm,ISO}$ which appear to be lower by 44% to 51% with respect to the experimental values $R_{c,exp}$. When referring to the design resistance $R_{cd,ISO}$, the safety margin oscillates between 3.8 and 4.3, which, although positively being on the safe side, appears too conservative and may lead to unsustainable or uneconomical design. Conversely, the strength values $R_{cm,pr}$ originated from the proposed formulation matches with remarkable precision the experimental results, predicting them with an error of less than 1.5%, and providing a sounder design safety margin of about 1.85 when referring to the design values $R_{cd,pr}$.

Figure 13 shows the influence of the different geometrical parameters included into the proposed formulation [Eq. (7)], where they are varied one at a time with respect to a benchmark connection characterised by a 40 mm equal-side angle placed at an inclination $\theta$ of 30° and anchored with a $\Phi 8$ rebar in a fixed-thickness height $h$ of 30 mm. The design compressive strength of concrete $f_{cd}$ is considered equal to 30 MPa. It is intended that possible combinations of $a$, $\theta$, and $\Phi$ may be accomplished with unequal-side angles, where in this case $a$ indicates the length of the cast-in side. It can be noticed that the cast-in angle profile length $a$ and the anchor rebar diameter $\Phi$ influence similarly the resistance with a pseudo-linear trend. By increasing the profile inclination $\theta$, a less-than-linear resistance increase is obtained as a result of the balance between the shorter length $L$ and the predominant longer depth $h$ of the concrete area mobilised.

The proposed elastic stiffness $K_{pr}$ of a row of $n$ connections can be interpreted as the sum of the contribution of each single connection, where the latter can be defined on the basis of the behaviour model previously set for the resistance by considering the concrete springs deforming elastically [triangular stress distribution in Fig. 12(a)], as follows:

$$K_{pr} = \frac{n \cdot K_c}{2}$$

where the stiffness of one concrete side of the connection $K_c$ is defined as follows, and the effective concrete width $m$ was related to a multiplier of the length of the profile $b$ (Fig. 14) grossly calibrated equal to 2.2 on the basis of the experimental results through data fitting:

Fig. 13 Parametric analysis about the influence of geometrical parameters upon the design strength of the connection under investigation: (a) angle profile side length, (b) profile inclination, (c) anchor rebar diameter.

Fig. 14 Width of concrete volume involved in one half of the connection considered for the elastic stiffness proposed formula.
local shear displacement of the connection in the range of 65 to 75 kN, which occurred in correspondence of a non-linear short branch up to peak load of 120 kN/mm up to 75% to 80% of the peak load, followed by a non-linear short branch up to peak load of 65 to 75 kN, which occurred in correspondence of a local shear displacement of the connection in the range of 1.0 to 1.5 mm. Larger anchor rebar diameter led to less-than-proportional larger resistance and elastic stiffness. For all specimens, failure occurred by practically contemporary crushing of the edge concrete (spalling) of the two connections tested in a row. The post-peak behaviour was characterised by a softening quasi-linear branch having stiffness ranging -4.5 to -6.5 kN/mm, about 20 times lower than the pre-peak elastic stiffness, with different post-peak failure mode depending on shape and diameter of the anchor rebar: U-shaped anchor bars deformed in combined shear and tension following a classical dowel action, with the Ø6 rebar failing right at the back of the rear welded connection with the angle and the Ø8 rebar not failing, while the Ø6 V-shaped anchor rebar deformed predominantly under axial strain up to failure, which occurred on the tensioned side only. These different softening post-peak behaviours depending on the configuration of the anchor rebar displayed after the assembly attained a remarkably low level of residual resistance, which makes this detail only marginal for the issue of a safe design. The rebars, however, improve the collaboration of concrete with the steel angle in the pre-peak phase, and their diameter plays a role on the concrete area efficiently mobilised, as shown by the higher resistance and elastic stiffness attained by specimens with larger diameter anchor rebars.

A critical application of the formula proposed in ISO 20987 for similar floor-to-floor welded bar angle connections shows that the formula related to concrete spalling failure is related to geometrical and mechanical parameters which are not fully consistent with the experimental evidence, and that when applied with mean material properties it leads to underestimate by about 50% the actual resistance of the connection, providing a safe-side strength estimation, although deemed to be too conservative. The alternative resistance formula proposed in this paper, based on different geometrical and material strength properties than the ISO 20987, much better fits the experimental results, and may be used for a more realistic and sustainable design approach of new structures or for the assessment of existing structures, providing a sounder safety margin. Moreover, a simplified formula for the estimation of the pre-peak elastic stiffness was also proposed and partially validated against the experimental results, as well as a suggested minimum spacing of consecutive connections installed in a row.

The stiffness and strength formulations proposed in the present paper may be employed for proper simplified numerical modelling and design/check of the connection, respectively, when employed for dry-assembled precast decks with the aim to stiffen the diaphragmatic action under horizontal loads and increase robustness of the construction.

It is finally pointed out that the proposed formulation, despite correctly tackling the physical behaviour of the connection and its stiffness and resistance, is based upon few experimental tests and on an engineering interpretation of the phenomenon. Further experimental or numerical insights may result helpful in confirming or refining it, possibly including considerations about multi-axial load scenarios.

### Table 2. Comparison of experimental stiffness with proposed formulations (values refer to two welded connections (n = 2).)

| Spec. | Rebar bent shape | Rebar diam. [mm] | $K_{exp}$ [kN/mm] | $K_{pre}$ [kN/mm] | $K_{pre} - K_{exp}$ [%] |
|-------|------------------|-----------------|------------------|------------------|------------------|
| U6    | U                | 6               | 176              | 180              | +2.3             |
| U8    | U                | 8               | 235              | 204              | -13.2            |
| V6    | V                | 6               | 169              | 180              | +6.1             |

$$K_c = \frac{E_c \cdot h \cdot L}{2 \cdot m}$$ (9)

$$m = 2.2 \cdot b$$ (10)

In other terms, the proposed formulation defines a sort of equivalent strut having dimensions $L$, $h$, $m$ deforming elastically in the horizontal axis according to the triangular strain/stress profile depicted in Fig. 12(a). The proposed formulation is compared with the experimental results in Table 2, where the longitudinal elastic modulus of concrete $E_c$ equal to 37 kN/mm² according to the nominal properties suggested in Eurocode 2. The proposed simplified formulation is deemed to provide a sound estimation of the elastic stiffness of the connection, although deviations from the experimental results reach almost up to 15% in the worst case, considering that even the evaluation of the experimental stiffness is affected by uncertainty and by the instrument base length.

The effective concrete width $m$ may also be employed as a suggested minimum distance between consecutive connections in a row.

It is reminded that the failure mode currently analysed, associated to concrete spalling, shall be compared with other failure modes possibly affecting the strength of the joint, as correctly pointed out in the ISO 20987 design guidelines, associated to the failure of the welded connection and of the anchor rebar, possibly subjected to multi-axial stress scenarios.

### 6. Conclusions

The monotonic experimental performance of the single floor-to-floor welded bar connection was characterised by a quasi-linear branch with elastic stiffness of 80 to 120 kN/mm up to 75% to 80% of the peak load, followed by a non-linear short branch up to peak load of 65 to 75 kN, which occurred in correspondence of a local shear displacement of the connection in the range of 1.0 to 1.5 mm. Larger anchor rebar diameter led to less-than-proportional larger resistance and elastic stiffness. For all specimens, failure occurred by practically contemporary crushing of the edge concrete (spalling) of the two connections tested in a row. The post-peak behaviour was characterised by a softening quasi-linear branch having stiffness ranging -4.5 to -6.5 kN/mm, about 20 times lower than the pre-peak elastic stiffness, with different post-peak failure mode depending on shape and diameter of the anchor rebar: U-shaped anchor bars deformed in combined shear and tension following a classical dowel action, with the Ø6 rebar failing right at the back of the rear welded connection with the angle and the Ø8 rebar not failing, while the Ø6 V-shaped anchor rebar deformed predominantly under axial strain up to failure, which occurred on the tensioned side only. These different softening post-peak behaviours depending on the configuration of the anchor rebar displayed after the assembly attained a remarkably low level of residual resistance, which makes this detail only marginal for the issue of a safe design. The rebars, however, improve the collaboration of concrete with the steel angle in the pre-peak phase, and their diameter plays a role on the concrete area efficiently mobilised, as shown by the higher resistance and elastic stiffness attained by specimens with larger diameter anchor rebars.

A critical application of the formula proposed in ISO 20987 for similar floor-to-floor welded bar angle connections shows that the formula related to concrete spalling failure is related to geometrical and mechanical parameters which are not fully consistent with the experimental evidence, and that when applied with mean material properties it leads to underestimate by about 50% the actual resistance of the connection, providing a safe-side strength estimation, although deemed to be too conservative. The alternative resistance formula proposed in this paper, based on different geometrical and material strength properties than the ISO 20987, much better fits the experimental results, and may be used for a more realistic and sustainable design approach of new structures or for the assessment of existing structures, providing a sounder safety margin. Moreover, a simplified formula for the estimation of the pre-peak elastic stiffness was also proposed and partially validated against the experimental results, as well as a suggested minimum spacing of consecutive connections installed in a row.

The stiffness and strength formulations proposed in the present paper may be employed for proper simplified numerical modelling and design/check of the connection, respectively, when employed for dry-assembled precast decks with the aim to stiffen the diaphragmatic action under horizontal loads and increase robustness of the construction.

It is finally pointed out that the proposed formulation, despite correctly tackling the physical behaviour of the connection and its stiffness and resistance, is based upon few experimental tests and on an engineering interpretation of the phenomenon. Further experimental or numerical insights may result helpful in confirming or refining it, possibly including considerations about multi-axial load scenarios.
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List of notations

- $\alpha_{cc}$: coefficient accounting for long-term effects on the concrete compressive strength
- $\gamma_v$: partial safety coefficient for the concrete strength
- $\gamma_o$: overstrength factor
- $\varepsilon_{p2}$: limit strain of parabola and rectangle of non-linear concrete constitutive model
- $\phi$: diameter of the anchor rebar
- $\theta$: inclination angle of the angle profile with respect to the vertical axis
- $\sigma$: longitudinal stress on concrete
- $a$: length of the cast-in side of the angle profile
- $b$: length of the angle profile
- $exp$: acronym for experimental
- $f_c$: compressive strength of concrete
- $f_{cd}$: design compressive strength of concrete
- $f_{ck}$: characteristic compressive strength of concrete
- $f_{cm}$: mean compressive strength of concrete
- $f_{cd}$: design tensile strength of concrete
- $h$: depth of concrete mobilised
- $m$: width of concrete mobilised
- $n$: number of connection devices in a row
- $pr$: acronym for proposed
- $t$: thickness of the flat part of the angle profile
- $A_c$: area of concrete mobilised
- $E_c$: longitudinal modulus of concrete
- $K$: shear stiffness of the connections
- $K_s$: stiffness contribution by one concrete half of the connection

- ISO: acronym for ISO 20987
- $L$: length of the concrete mobilised
- $R_s$: spalling resistance
- $R_d$: design spalling resistance
- $R_{m}$: mean spalling resistance

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