Experimental investigation of the shear strength of one-way reinforced concrete (RC) slabs subjected to concentrated loads and in-plane transverse axial tension

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Abstract
This paper presents the results of an experimental campaign carried out to investigate the effects of in-plane transverse tensile forces on the shear strength of linearly supported one-way reinforced concrete (RC) slabs without shear reinforcement subjected to concentrated loads. A total of 5 half scale slabs (1650 × 1650 × 120 mm), subjected simultaneously to different levels of in-plane tensile forces and a concentrated load were tested up to failure. The clear shear span to effective depth ratio \(a_s/d\) was equal to 3 in all tests, to minimize the effects of the direct transmission of the load to the closest support, the so called arching action. As observed, the shear strength barely decreases with increasing values of the tensile force applied, even for values of the external force that cracked the concrete cross-section of the slabs in the direction perpendicular to the span. These observations may indicate that the shear–flexural behavior in the spanning direction is not significantly affected by the membrane forces in the transverse direction.

KEYWORDS
concentrated load, experimental campaign, one-way slabs, reinforced concrete, shear, tension

1 | INTRODUCTION

Reinforced concrete slabs are structural components widely used in both bridges and buildings, due to their great versatility and ease of construction. These structural elements are mainly subjected to bending and shear, but in some cases in practice, in-plane tensile stresses may arise and act simultaneously with the out-of-plane design loads. For instance, this situation may arise in top slabs of box girder bridges subjected to hogging bending moments, decks of tied arch bridges, floor slabs under seismic loads, restrained shrinkage, or thermal effects. The effect of tension on the shear strength of reinforced concrete elements has not been exhaustively studied throughout the last decades, but it is gaining attention within the last years.

1.1 | Previous research works

First widely known studies on the effect of tension on the shear strength of RC elements were carried out in the
United States, after the partial collapse of a military warehouse in 1955, by the Portland Cement Association. Experiments conducted in that investigation, in which rectangular cross-section beams without stirrups were subjected to tension in the spanning direction, showed up to a 50% shear strength reduction for an axial tensile stress of 1.4 MPa. However, tested specimens were not heavily reinforced, having a flexural reinforcement ratio of 0.42%. Due to such a small amount of longitudinal reinforcement, the postcracking capacity was very low and failure occurred soon after first cracks formed. In the next decade, similar experiments using rectangular and T-shaped beams were conducted at the University of Washington. After a series of experiments in beams with and without stirrups, it was recommended that the shear carried by the concrete should be taken equal to zero if tensile stresses exceed $0.33\sqrt{f_c}$ ($f_c$ in MPa). This recommendation was incorporated into the ACI-318 building code released in 1971, in which the concrete contribution to the shear strength was reduced linearly to zero as tensile stresses on the member increase to 3.4 MPa.

At the same time, another investigation carried out at the Imperial College, rectangular beams subjected to different levels of tensile stresses, with a longitudinal reinforcement ratio of 1.46%, showed a moderate reduction on the shear strength with increasing tension, with a 12% decrement for the maximum tensile stress of 2.6 MPa applied. These results started to highlight the importance of the amount of longitudinal reinforcement on the shear strength of RC elements subjected to axial tension. A greater amount of reinforcement leads to narrower cracks and deeper compression head for a particular tension level, which contribute to enhance the shear strength.

In the early 1980s, two experimental campaigns conducted at Cornell University for the US Nuclear Regulatory Commission studied the effects of Biaxial tension on the punching-shear strength of RC slabs (Figure 1a). Both studies concluded that the punching-shear strength was only slightly related to the biaxial tension level, although more flexible behavior was observed for higher levels of tension. Once again, worst results were obtained for the slabs with lowest reinforcement ratios in both orthogonal directions. In the same period of time, Regan presented the results of an experimental campaign on the punching-shear strength of one-way and two-way slabs subjected to unidirectional in-plane tensile forces (Figure 1b). In both cases, an average 15% decrement of the ultimate load for an in-plane tensile stress of 3.95 MPa was reported.

In the late 1980s, as part of the validation of the Modified Compression Field Theory, an experimental campaign was carried out at the University of Toronto, where 24 square panels of 890 mm sides and 70 mm thickness, were subjected to different combinations of in-plane shear ($\nu$) and tension ($\sigma_t$). Panels were reinforced only in the longitudinal direction with a reinforcement ratio $\rho$ of 1% or 2%, depending on the element. It was experimentally observed, as the MCFT had predicted, that panels have considerable postcracking shear capacity provided and that the longitudinal reinforcement ratio prevents excessive widening of the cracks. Panels with $\rho = 1\%$ showed almost no influence of tension on the shear strength up to values of $\sigma_t$ close to 2 MPa, decreasing from that point onwards, until reaching the uniaxial tensile strength of 4.5 MPa. Panels with $\rho = 2\%$ showed an average 20% decrement of the shear strength when $\sigma_t \approx 4.5$ MPa and a 37% when $\sigma_t \approx 7$ MPa, highlighting once again the importance of the amount and distribution (the more uniform along the cross-section depth, the better) of longitudinal reinforcement. Similar trends were observed in another two experimental campaigns carried out several years after at the same laboratory in beam-like panels and square panels, respectively.

More recently, laboratory tests were conducted at the Technical University of Denmark in beams reinforced symmetrically using high performance steel ($f_y = 1027$ MPa) with a longitudinal tensile reinforcement ratio $\rho = 0.88\%$. Tested beams showed almost no influence of tension on the shear strength for tensile stresses up to 7.5 MPa. For this

![FIGURE 1](Set-up of the test carried out: (a) at Cornell University; (b) by Regan)

FIGURE 1  Set-up of the test carried out: (a) at Cornell University; (b) by Regan
level of tensile stress, for instance, Eurocode-2\textsuperscript{14} predicts a total loss of shear strength. The use of high-performance concrete \((f_c > 75\) MPa) has also been studied.\textsuperscript{15} In this case, T-shaped beams were tested under the simultaneous action of shear and axial tension, showing a 33% decrement for longitudinal tensile stresses for \(\sigma_t = 1.81\) MPa when \(\rho = 1\%\) and a 35% of reduction for \(\sigma_t = 2.71\) MPa when \(\rho = 1.51\%\). In both cases the reinforcement was concentrated close to the tension face.

First shear tests carried out in slabs subjected to uniaxial in-plane tension took place at the University of Lyon less than a decade ago.\textsuperscript{16} Slabs were simply supported on their four sides and subjected to a point load close to one of the supports. The external in-plane tension was applied perpendicular to that support (Figure 2). All the tested slabs, reinforced non-symmetrically with a tensile longitudinal reinforcement ratio of \(\rho = 1.22\%\), showed almost no influence of the longitudinal tensile stress for \(\sigma_t\) up to 0.65 MPa. However, a significant average reduction of a 27% was observed for \(\sigma_t = 1\) and 1.2 MPa. These reduction percentages are more or less in line with the tests conducted on non-symmetrically reinforced T-shaped beams mentioned previously. Table 1 presents a summary overview of all these experimental work.

To conclude, within the last few years, thanks to all the research carried out, a couple of theoretical models to predict the punching-shear strength of two-way slabs subjected to in-plane uniaxial or biaxial tension have been presented.\textsuperscript{17,18} Both models coincide in the importance of the amount of reinforcement parallel to the external tension. If the tensile stresses are high enough to cause concrete cracking, low reinforcement ratios may lead to a big loss of the punching-shear capacity caused by premature yielding of part of the reinforcement. However, more research is needed for the case of the shear strength of one-way slabs subjected to in-plane tension.

### 1.2 | Design codes

Shear provisions included in two of the most internationally accepted codes, such as Eurocode-2\textsuperscript{14} and ACI 318-19\textsuperscript{19} are reviewed in this section. EC-2, in section 6.2.2, indicates that the design value for the shear resistance \(V_{Rd,c}\) is given by Equation (1), with a minimum value given by Equation (2), where \(\gamma_c\) is the safety factor for concrete; \(k = 1 + \sqrt{\frac{200}{d}} \leq 2\) is the size effect factor; \(\rho_l\) is the amount of longitudinal tensile reinforcement \((\rho_l \leq 0.02); b_w\) is the smallest width of the cross-section in the tensile area (mm); \(f_{ck}\) is the characteristic concrete compressive strength (MPa) and \(d\) is the effective depth of the cross-section (mm).

\[
V_{Rd,c} = \left( \frac{0.18}{\gamma_c} \cdot \frac{k \cdot \sqrt{100 \cdot \rho_l \cdot f_{ck}^2 + k_1 \cdot \sigma_{cp}}}{d} \right) \cdot b_w \cdot d \quad (1)
\]

\[
V_{Rd,c} = \left( 0.035 k^{1.5} \cdot f_{ck}^{1.5} \right) \cdot b_w \cdot d \quad (2)
\]

The effect of axial stresses on the shear strength is included through the term \(k_1 \cdot \sigma_{cp}\), where \(k_1 = 0.15\) and \(\sigma_{cp} = N_{Ed}/A_c\), being \(N_{Ed}\) the axial load applied in the cross-section, with negative sign in the case of tension, and \(A_c\) is the area of the concrete cross-section. It is remarkable that EC-2 considers that the increment of shear strength due to a compression force \(C\) is equal to the reduction in shear strength due to a tensile force of the same magnitude, despite the response of concrete in one case or in the other one is radically different. Another important aspect of this formulation is that it is intended mostly for beams, where axial stresses are always applied in longitudinal (spanning) direction. However, if these expressions are used to calculate the shear strength of a one-way slab, axial loads may exist in the longitudinal and in the transverse direction. If the latter is the case, it is not clear whether those axial stresses have to be taken into account or neglected. The experimental campaign presented in this paper tries to provide some insight into this issue.

ACI 318-19 provides different sets of equations to calculate the shear strength, depending on the type of element and loading. Chapter 7: “One way slabs” refers to section 22.5 for the calculation of the concrete contribution to the shear strength \(V_c\). Within this section, it is pointed out that the effect of axial tension due to creep and shrinkage shall be considered in calculating \(V_c\), and it is recommended that if there is uncertainty about the magnitude of the axial tension, it is desirable to design shear reinforcement to resist the total shear. For elements without shear reinforcement, and assuming normal weight concrete \((\lambda = 1)\), Table 22.5.5.1 of ACI Code
provides Equation (3) for the one-way shear strength of non-prestressed members, where $\lambda_s = \sqrt{\frac{2}{1+0.04q}} \leq 1$ is the size effect factor; $\rho_w$ is the amount of longitudinal tensile reinforcement; $f'_{c,w}$ is the specified concrete compressive strength (MPa); $b_w$ is the web width (mm); and $d$ is the distance from the extreme compression fiber to centroid of longitudinal reinforcement (mm).

$$V_c = \left[0.66 \cdot \lambda_s \cdot (\rho_w)^{\frac{1}{3}} \cdot \sqrt{f'_{c,w}} + \frac{N_u}{6A_g}\right] \cdot b_w \cdot d \quad (3)$$

In this case, the effect of axial stresses is included through the term $N_u/6A_g$, where $N_u$ is the axial force normal to the cross-section acting simultaneously with the shear force ($N_u < 0$ for tension) and $A_g$ is the gross area of concrete. In the same way as EC-2, ACI-318 considers the increment of shear strength due to a compression force equal to the reduction in shear strength due to a tensile force of the same magnitude. According to the definition of $N_u$, it seems that it is intended for axial forces parallel to the spanning direction, and it is not clear what to do in the case of a one-way slabs subjected to axial tension in the transverse direction.

In this context, an experimental campaign has been carried out at the Laboratory of Technology of Structures and Materials of the Universitat Politècnica de Catalunya (UPC), whose main goal was to identify and quantify the effect of unidirectional in-plane tension forces perpendicular to the spanning direction on the one-way shear strength of RC slabs.

2 EXPERIMENTAL CAMPAIGN

2.1 Specimens tested

A total of five 1650 × 1650 × 120 mm slabs, simply supported on two parallel linear supports, were cast at the Laboratory of Technology of Structures and Materials of the Universitat Politècnica de Catalunya (UPC). The dimensions of the slabs tested in the context of this experimental campaign were conditioned by punching-shear tests previously performed by the authors,20 with the aim of reusing the tensioning system built for that occasion. In order to ease the placement, alignment and removal of the whole set-up within the portal frame where the main hydraulic jack of the laboratory is installed, the point load was decided to be applied at the center of the slabs. This way, one of the supports was moved inwards so a clear shear span to effective depth ratio $a_c/d$ equal to 3 was attained. The opposite support was also moved slightly inwards in order to avoid a flexural failure. The final dimensions of the set-up are and the supporting system are shown in Figure 3.

The specimens were subjected to tension in their plane, only in the direction parallel to the supports, through 10 posttensioning bars 25 mm in diameter and 600 mm in length (Figure 4), partially embedded in two opposite faces (5 bars on each side) connected to the tensioning system. Although most of the tension was transmitted to the concrete by bond, anchors were placed at the embedded end of the bars for greater safety. The value of the external tension applied during the tests is measured in relation to the tensile force producing the cracking of the slab’s cross-section, or $T_{cr} = A_c f_{ct}$, with $A_c$ being the composite concrete and steel area of the cross-section of the slab and $f_{ct}$ the concrete tensile strength. Averaging the values of the tensile strength obtained from characterization tests (Table 2), $T/T_{cr} = 1$ approximately correspond to $\sigma_t = 3.2$ MPa.

The slabs were reinforced with two steel meshes arranged on the upper and lower faces (Figure 5). In the direction parallel to the external tension (transverse direction), all the tested slabs, named A4, A5, A6, A7, and B2, were symmetrically reinforced. On each face,
reinforcement consisted of 12 mm bars spaced 105 mm in type A slabs ($\rho = 0.011$) and 16 mm bars spaced 105 mm in slab B2 ($\rho = 0.02$).

In the direction perpendicular to the applied tension (longitudinal or spanning direction), on the lower face, an assembly consisting of 12 mm bars spaced 100 mm was arranged in all cases, ($\rho = 0.013$), while 12 mm bars spaced 200 mm were arranged on the upper face. Additional reinforcing bars were placed in the vicinity of the posttensioning bar anchors, as a measure of local reinforcement. The tensile reinforcement ratio and effective depth of each type of slab are shown in Table 3. The amount of transverse reinforcement in specimen B2 ($\rho = 2\%$) is higher than usual in practical cases. However, the experimental results of this specimen can be adequate to identify if the increment of the reinforcement parallel to tension significantly affects the structural response and the shear strength.

The slabs were cast using ready-mixed concrete from a local supplier. Normal strength concrete C30/37 with
crushed limestone aggregate with a maximum size of 10 mm was used. To characterize the concrete, standard compression (UNE-EN 12390-3), splitting (UNE-EN 12390-6), and elastic modulus (UNE-EN 12390-13) tests were performed. The specimens were reinforced with deformed bars of B500-S steel. Yield stress and elastic modulus of each type of reinforcing steel (UNE-EN ISO 6892-1) were also characterized. Results are shown in Table 2.

### 2.2 Test set-up

Four out of the five tested specimens were subjected to different levels of in-plane tension, while the control slab was tested without axial force. To introduce the tensile force in the slabs, an auxiliary steel structure, whose plan dimensions were 2500 × 2840 mm, was used. This structure consisted of a rectangular steel frame surrounding the slab (Figure 6a). On sides parallel to the tensile forces, the frame consisted of a single HEB 300 profile, whereas on both perpendicular faces it was formed by two HEB 300 profiles arranged in parallel and slightly separated to allow the passage of bars (extensions) connected to the aforementioned bars partially embedded in the slab (Figure 6b). On one of these faces, a passive anchorage for the tensioning bars was used while they were prestressed from the other end, with the help of force-controlled hollow jacks connected to the bars protruding from the specimens (Figure 6c).

The steel structure was set at the appropriate height with the help of four height-adjustable supports located under each of the four corners of the frame. For safety reasons, once the desired external force value was reached, the bars were anchored and the jacks were disconnected from them. This fact caused a small loss of the axial force during the test that was quantified by the arranged instrumentation. The theoretical loads to be applied and the values measured during the tests are shown in Table 4.

Supporting system comprises two HEB 100 profiles 1650 mm long placed on top of five 20 × 120 mm hinged load cells used to measure the distribution of the reaction at each support (Figure 7). A piece of rubber 120 × 120 × 20 mm was placed over each of the load cells to allow the free rotation of the slab. A 10-mm thick rubber strip was placed on top of the HEB 100 profiles to ensure smooth contact between the supports and the tested specimens. Load cells were supported on two rigid easels arranged in such a way that the distance between the inner face of the loading plate and the closest support was equal to 3d (a_v = 3d).

To introduce the concentrated load, a hydraulic jack anchored in a loading frame, rigidly fixed to the floor
slab, with a maximum capacity of 1000 kN was used. The out-of-plane load was applied to the specimens through a \(145 \times 145 \times 30\) mm steel plate, under which a thin layer of fine aggregate was placed to ensure regular contact with the upper face of the slabs.

2.3 | **Instrumentation**

In addition to the load cells of 50 kN placed underneath the supports, hollow load cells of 500 kN were used in the posttensioned bars (Figure 8a), in both active and passive anchorages, in order to measure the in-plane tensile force applied to each slab. To control the possible yielding of the reinforcement, in both longitudinal and transverse direction, 8 strain gauges were distributed as shown in Figure 8b. To measure the deflection underneath the point load of each specimen, a 75 mm displacement transducer was positioned at the center of the lower face, jointly with the internal linear variable displacement transducer (LVDT) of the loading frame. Finally, 3 LVDTs were placed in the vertical of each support on the upper side of the slabs, to control their descent and then, to subtract their mean value from the displacements measured in the center of the slab.

2.4 | **Test procedure**

First, the tensile force, given in Table 4, was introduced in the slabs and, subsequently, the point load was incrementally applied on its center up to failure of the specimen. In all cases, two levels of the out-of-plane load were previously applied and removed, one up to 40 kN and another up to 80 kN. Both loads were maintained for 100 s and served to accommodate all the elements used in the posttensioned bars (Figure 8a), in both active and passive anchorages, in order to measure the in-plane tensile force applied to each slab. To control the possible yielding of the reinforcement, in both longitudinal and transverse direction, 8 strain gauges were distributed as shown in Figure 8b. To measure the deflection underneath the point load of each specimen, a 75 mm displacement transducer was positioned at the center of the lower face, jointly with the internal linear variable displacement transducer (LVDT) of the loading frame. Finally, 3 LVDTs were placed in the vertical of each support on the upper side of the slabs, to control their descent and then, to subtract their mean value from the displacements measured in the center of the slab.

![Figure 6](image6.png) Tensioning system used to apply the axial force to the specimens: (a) steel frame; (b) extension connected to the protruding bars; (c) jacks placed at the active anchorage

![Figure 7](image7.png) Support system for the tested specimens

| Slab | \(T/T_{cr}\) before disconnecting jacks | \(T/T_{cr}\) at the test beginning | \(T/T_{cr}\) at failure |
|------|-----------------------------------------|-----------------------------------|-----------------------|
| A4   | 0.00                                    | 0.00                              | 0.00                  |
| B2   | 0.50                                    | 0.45                              | 0.43                  |
| A5   | 0.80                                    | 0.73                              | 0.70                  |
| A6   | 1.10                                    | 1.08                              | 1.06                  |
| A7   | 1.40                                    | 1.32                              | 1.30                  |
involved in the test and to check that the instrumentation was properly working. Once the second test load ended and the slab was unloaded, the test up to failure was started, applying the load by a controlled displacement at a rate of 0.005 mm/s.

In support of the experimental campaign, a finite element model using software package Simulia Abaqus was developed, taking as reference previous works carried out by the authors²⁰ and Nana et al.²¹ This model was initially used to predict the tests results and to design the auxiliary structures of loading and support. Once the test on the control slab was performed, the numerical model was calibrated with the obtained result (Figure 9) and used to predict the shear strength of the rest of Type-A slabs under the corresponding value of the external force applied. With respect to Type-B slab, only a single test was performed with a tensile force of \( T/T_{cr} = 0.43 \). Then, since there was not reference slab of type B, the same numerical model was also used to predict the shear capacity of the Type B reference slab, as shown in Table 5. Details of the numerical simulations will be explained in Section 4.

### 3 | EXPERIMENTAL RESULTS

#### 3.1 | Ultimate loads

The main interest of the experimental campaign was to quantify the variation of the shear strength as a function of the tensile force applied to the slabs. Figure 10 shows

| Type of slab | Ultimate load \( (P_u) \) (kN) | Shear strength \( (V_{u_{control}}) \) (kN) | Type of analysis |
|--------------|-------------------------------|---------------------------------|----------------|
| A            | 210.6                         | 127.1                           | Lab. Test      |
| B            | 230.2                         | 138.1                           | FEM            |
the comparison between all the experimental load–
deflection curves obtained. As can be seen, the ultimate
load of all type-A slabs slowly decreases with increasing
levels of the applied tensile force, with a 5.3% reduction
for $T/T_{cr} = 1.3$ ($\sigma_t = 4.2$ MPa). The whole trend is showed
in Figure 16. Regarding slab B2, the ultimate load
obtained was a 2.1% lower than that obtained for its con-
trol slab using the aforementioned FEM, confirming the
trend observed with type-A slabs. Values of the ultimate
load and comparison to the control slabs are summarized
in Table 6. These results indicate that, for the tension
level applied in the laboratory tests, if this tensile force is
applied in the direction perpendicular to the span, the
shear strength seems to be barely affected.

Nevertheless, the external tension affected the stiffness
of the specimens. Slabs A5, A6, and A7 showed similar
response, which indicate that the external force cracked
the concrete, but the reinforcement was able to carry all
the stresses caused by the addition of transverse bending
and tension. This was confirmed by the measurements
provided by the strain gauges placed in both longitudinal
(gauges 7 and 8) and transverse (gauges 2 and 3) rein-
forcement underneath the point load. No signs of yielding
were observed in any of the experimental tests, which dis-
cards the possibility of a flexural failure. These measure-
ments are in good agreement with the brittle failure
observed in the load–deflection curves. Slab B2 represents
an intermediate situation, in which concrete cracks pre-
maturely, but the higher reinforcement ratio in the trans-
verse direction increases the global stiffness with respect
to type-A slabs. It is interesting to note that slab B2 pre-
sented a higher shear strength than slab A4 (type-A control
slab). Some numerical studies\textsuperscript{23} have shown a small but
positive influence of the transverse reinforcement ratio on
the shear strength. However, no significant influence has
been observed experimentally.\textsuperscript{23–25} In the context of the
study presented herein, the FEM predicted a 9.3% incre-
ment of the shear strength for the type-B control slab with
respect to the A4 control slab, and slab B2, tested with a
tension level of $T/T_{cr} = 0.43$ ($\sigma_t = 1.38$ MPa), resisted a
shear strength value 7.1% higher than the control slab A4.

Figure 11 shows the state of cracking at the bottom face
of the slab at failure for all the tests carried out. Auxiliary
lines at 45° with the spanning direction have been sup-
erimposed as a reference. As can be observed, a crack par-
allel to the closest support appeared in all cases. In slabs
subjected to tension, this failure crack was observed to turn
and to connect with cracks parallel to the spanning direc-
tion provoked by the external force applied. Only slab A6
presented a cracking pattern that seems to be a transition
between shear and punching-shear failure, since the crack
parallel to the support quickly turns and partially surrounds
the loading plate. Negative influence of in-plane tensile
forces on the punching-shear strength of two-way slabs
have been reported,\textsuperscript{17,18} and this may have been the case of
slab A6. However, as shown in Figure 10 and Table 6, the
load-deflection behavior and the ultimate load of this slab
were in accordance with the rest of results obtained.

### 3.2 Reactions at the supports

In order to know if the external tensile force affected the
distribution of the reactions at the supports, the measures
yielded by the load cells supporting both HEB 100 profiles
were analyzed and compared with the values obtained for
the control slab. Despite only five load cells were available
for each support, it was enough to capture the global
behavior. Figure 12 shows the distribution of the reaction
at both supports for all type-A slabs tested. It can be seen
that regardless the level of tensile force applied, reaction
distributes more or less in the same way, with highest
values measured at the center of each support and de-
creasing toward the free edges of the slab. An interesting
point is that, despite the clear shear span for the farthest
support is equal to $5d$, it was observed a similar trend for
the reaction distribution than that at the support placed at
$3d$. This observation is in agreement with some existing
studies\textsuperscript{26,27} which conclude that a limit value for the hori-
izontal load spreading must be established in order to
avoid highly un-conservative results when using the sim-
plified formulation of the codes.

| Slab | $V_{u,control}$ (kN) | $T/T_{cr}$ at failure | Ultimate load ($P_u$) (kN) | Shear strength ($V_u$) (kN) | $V_u/V_{u,control}$ |
|------|-----------------|-----------------|-----------------|-----------------|-----------------|
| A4   | 210.6           | 0.00            | 210.6           | 127.1           | 1               |
| B2   | 230.2\textsuperscript{a} | 0.43            | 225.3           | 135.9           | 0.979           |
| A5   | 210.6           | 0.70            | 206.9           | 124.4           | 0.978           |
| A6   | 210.6           | 1.06            | 202.8           | 122.4           | 0.963           |
| A7   | 210.6           | 1.30            | 199.4           | 119.6           | 0.947           |

\textsuperscript{a}Obtained from FEM.
This section presents the numerical model developed jointly with the experimental campaign. In addition to help in the design of the test set-up, the model was also used to contrast the values of the shear strength obtained experimentally, and to perform simulations for values of the tensile force different from those applied in the experimental campaign. The numerical model was carried out with the help of Simulia Abaqus software.\textsuperscript{28}
4.1 | Model generation

Taking into account the existing plane of symmetry at the middle of the slab, parallel to the spanning direction, both in geometry and loading, only half of each slab was modeled, considering the corresponding boundary conditions in the symmetry plane. The numerical model was inspired in a similar model developed by the authors to study the influence of in-plane tensile forces on the punching-shear strength of RC slabs, but also recommendations given in Nana et al., as the use of an explicit quasi-static solution or a linear evolution of the damage parameter, in both tension and compression.

Regarding the type of element used, 8 nodes hexahedral elements with reduced integration (C3D8R) were selected for concrete in order to reduce the computational cost and to avoid obtaining excessively rigid results due to the linear interpolation of the displacements field. For the reinforcing bars, linear beam elements of 2 nodes (B31) perfectly bonded to concrete were used. The chosen size for the concrete elements was 20 mm, so that six elements were arranged along the 120 mm of the slab thickness. The same element size of 20 mm was considered for the reinforcing elements. The supports of the slab were simulated by means of non-linear springs (SPRING A), with a very high stiffness in compression and practically zero stiffness in tension, thus allowing the partial lifting of the slab and avoiding the appearance of undesired tensile reactions (Figure 13a). The introduction of tensile forces was simulated in the model by applying the loads directly to the nodes located in the area of the anchor plates of the posttensioning bars (Figure 13b), while for the point load, a displacement was imposed on the nodes located on the upper face of the slab, under the surface occupied by the load plate (Figure 13c).

The simulations were performed by means of a quasi-static analysis, available in the Abaqus/Explicit package. This kind of analysis has several advantages in comparison to the traditional implicit method when non-linear analyses are carried out. The absence of a global tangent matrix stiffness in the explicit analysis avoids certain convergence problems and in general, it saves a substantial amount of time when performing parametric studies. To avoid dynamic instabilities, the ratio of the kinetic energy generated by the inertia forces to the internal deformation energy must be negligible, which has to be checked after each simulation.

4.2 | Material modeling

To simulate the behavior of concrete, the Concrete Damaged Plasticity model was chosen, which requires the definition of the uniaxial constitutive equations in compression and tension, in addition to the definition of a yielding surface and, optionally, damage parameters for the Young’s modulus also in tension and compression. In compression, a linear behavior of up to 40% of \( f_c \) \((\sigma_c^0)\) was considered. To define the elastic response, only the modulus of elasticity \( E_c \) and the Poisson ratio \( \nu \) are needed. The modulus of elasticity is that obtained in the corresponding characterization test (see Table 2), while for \( \nu \), as Abaqus considers it a fixed value throughout the whole simulation process, including the postcracking regime, a value \( \nu = 0 \) was chosen. To take into account the non-linear behavior in compression, the Hognestad parabolic constitutive equation was chosen (Figure 14a). The values of \( \varepsilon_{c0} \) and \( \varepsilon_{cu} \) correspond to those proposed in table 3.1 of EC-2, with values of \( \varepsilon_{c0} = 0.0020 \) and \( \varepsilon_{cu} = 0.0035 \). As mentioned above, a linear increment for

![Figure 13](image-url)
the damage parameter $d_c$ was selected, with a maximum value of $d_c = 0.9$ corresponding to $\varepsilon_c = \varepsilon_{cu}$ to avoid convergence issues (Figure 14b).

Regarding the tensile behavior, the model proposed in section 5.1.8.2 of the Model Code 2010\textsuperscript{31} has been used, which considers a linear behavior up to cracking stress $f_{ct}$, and a bilinear softening branch governed by the fracture energy of the material (Figure 14c). The value of the fracture energy adopted is given by Equation (4), proposed in Cladera et al.\textsuperscript{32} as a modification of that provided by the Model Code 2010 to account for the maximum aggregate size of concrete, which in this case was 10 mm. A linear variation of the damage parameter in tension $d_t$ was considered, similar to the one in compression, but this time depending on the crack width $w$ rather than on the plastic strain $\varepsilon_{pl}$. A maximum value of $d_t = 0.9$ corresponding to $w = w_c$ was chosen also to avoid convergence issues (Figure 14d).

\[ G_f = 0.028 f_{ct}^{0.28} d_{d_{max}}^{0.32} \] (4)

The behavior of concrete in tension was introduced in ABAQUS through a stress–strain relationship, so it was necessary to transform the stress-crack width given by Model Code (Figure 14c) into a stress–strain curve. For this purpose, the crack opening is divided by the characteristic length of the element ($l_c$), defined as the average dimension of a finite element, in this case the length of an edge of a cubic element. Thus, the softening of the tensile branch, is given by $\varepsilon_{ct} = \varepsilon_{cr} + w/l_c$, where $\varepsilon_{cr}$ is the concrete strain corresponding to the tensile strength, $f_{ct}$.

The values used to define the yielding surface were modified from those proposed in Fernandez et al.\textsuperscript{20} to adjust the simulation of the type-A control slab to the results obtained in the laboratory (Figure 9), and are collected in Table 7. For the steel reinforcement, a bilinear stress–strain diagram was used, considering perfect plasticity. Yield stress and Young's modulus were extracted from the characterization tests (see Table 2).

4.3 | Results

A comparison between the load–displacement curves obtained by the numerical simulations and experimentally is shown in Figure 15. Accurate results were obtained in terms of ultimate load and deflection, despite numerically obtained shear strengths are in general smaller than the experimental ones, with an average difference of 4.46%. Results of the comparison are summarized in Table 8.

Once the numerical model was validated with the experimental results, simulations were performed, for type-A slabs, for other values of $T/T_{cr}$ than those applied at the laboratory, up to a maximum of $T/T_{cr} = 1.5$ (Figure 16). A linear decreasing trend is observed, with a

FIGURE 14 Constitutive relationships for concrete: (a) compressive behavior; (b) compressive damage; (c) tensile behavior; and (d) tensile damage

| TABLE 7 Parameters defined for the concrete damaged plasticity |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| Dilation ($\psi$) | Eccentricity ($\varepsilon$) | $\sigma_{b0}/\sigma_{co}$ | $K$ | Viscosity ($\mu$) |
| 36.0 | 0.1 | 1.16 | 0.8 | 0 |
weak interaction between shear strength in the spanning direction and tensile stresses in transverse direction. For $T/T_{cr} = 1.2$ ($\sigma_t \approx 3.8$ MPa), $P_u/P_{u,T} = 0$ ratio is equal to 0.92. Beyond that point, the numerical model predicts a slightly greater reduction of the shear strength as the tensile force increases, with a 12.4% decrement when $T/T_{cr} = 1.5$ ($\sigma_t \approx 4.8$ MPa). In general, numerical simulations are in agreement with the experimental results. As can be seen, B2 slab properly fits the curve obtained for type-A slabs, which may indicate that for the considered reinforcement ratios, the effect of the external tensile force on the shear strength is similar.

Figure 16 also includes EC-2 and ACI-318 predictions (Equations (1) and (3), respectively) for the reduction of the shear strength with increasing tension. For their calculation, the value of the transverse tensile stress has been used without any further consideration, assuming that longitudinal and transverse normal stresses affects the same way the shear strength. As can be seen, this approach leads to highly conservative results. For example, for $T/T_{cr} = 1.3$ ($\sigma_t \approx 4.2$ MPa), EC-2 predicts a 49.2% reduction and ACI 318 a 75.3% reduction. This percentages are much greater than the 5.3% reduction observed experimentally and 10.8% reduction predicted numerically for the same level of tension. Therefore, in order to obtain closer results to those obtained experimentally and numerically, the value of the transverse tensile stress should be weighted before being used in the current code formulations.

Nevertheless, since the effect of normal stresses is an additive term in the shear strength, this factor may change...
for different values of the effective depth, reinforcement ratio, or concrete compressive strength than those of the specimens tested. This is also the reason why ACI 318 yields more conservative results than EC-2. Both codes add the tensile stresses to the shear strength multiplying $\sigma_t$ by a similar factor (0.167 for ACI and 0.15 for EC-2), but for the geometry of the tested slabs, the ratio between both codes prediction when $\sigma_t = 0$ is $V_{R,EC2}/V_{R,ACI} = 1.37$. Thus, in this case, for the same value of $\sigma_t$, the relative reduction of the shear strength is greater for ACI 318.

The results of the experimental campaign presented herein are more or less in line with the observations made in the only similar experiment available in literature, conducted by Regan, when a 15% reduction on the strength was reported for $\sigma_t \approx 3.95$ MPa. This value is greater than the values obtained by the authors, but it is reasonable since the point load was applied at mid-span and a punching-shear failure occurred. Because of that, part of the failure surface was normal to the transverse tensile stresses and the strength of the specimen could have been more affected. However, in the experimental campaign carried out by Bui et al. on slabs subjected to longitudinal tensile stresses, greater relative strength reductions up to a 27% were obtained for values of around 1.2 MPa. This greater values of strength reduction seems to highlight the different degree of affection between transverse and longitudinal tensile stresses. In any case, the experimental evidence in slabs is scarce and more research is needed in slabs with different geometries, reinforcement ratios, reinforcement dispositions (symmetric or not) and supporting conditions.

To conclude, cracking patterns obtained for different values of the applied external tension are presented in Figure 17, showing a considerable similarity with the experimentally observed crack patterns.

### 5 | CONCLUSIONS

Based on all the experimental work reported throughout the last decades and on the observations made during the experimental campaign presented in this article, the following conclusions can be drawn:

1. Code formulations are, in general, not clear about how to proceed when predicting the effects of transverse tensile forces on the shear strength of one-way slabs. It is probably due to the fact that they are originally intended for beams, where only longitudinal axial stresses may take place in practice. It is not mentioned if transverse normal stresses should be considered in the same way as longitudinal stresses or even if they may be neglected. The comparison between code...
predictions and experimental and numerical results carried out in this study shows that the shear strength is barely affected by the transverse tensile stresses, and neglecting its negative contribution leads only to slightly unsafe results, for the particular case of the specimens tested. On the contrary, if transverse tensile stresses are taken into account, their value should be lessened or very conservative results will be obtained.

2. The decrement in the one-way shear strength due to axial tension can be managed if enough reinforcement is provided to keep crack width under control. Also a uniform or symmetrical distribution of the reinforcement within the concrete cross-section has a favorable effect on the shear strength.

3. For the studied case of one-way slabs subjected to transverse axial tension, no significant reduction of the shear strength has been observed. An approximately linear decrement up to a 5.3% reduction on the shear strength for the highest transverse axial load applied ($\sigma_t \approx 4.2$ MPa) was obtained experimentally. These values are smaller than the relative strength reduction of the shear strength reported in tests on slabs subjected to longitudinal axial tension (27% reduction for $\sigma_t \approx 1.2$ MPa), which highlights the difference in the degree of affectation on the one-way shear strength of transverse and longitudinal tensile stresses.

4. Transverse axial load affects the stiffness of the members due to premature cracking of concrete. If not enough reinforcement is provided in the transverse direction, this fact may lead to a non-fulfillment of the serviceability limit state of deformations and affects the durability of the member due to excessive crack width.

5. The numerical model developed for the non-linear analysis of the tested slabs, calibrated with the experimental results of the tested reference slab ($T = 0$), predicts an approximately linear decrement of the shear strength with increasing axial tension, as observed experimentally, but yields slightly conservative results, particularly for values of $T/T_{cr} > 1.2$. For the aforementioned case of the highest transverse axial load applied at the laboratory ($\sigma_t \approx 4.2$ MPa), the numerical model predicts an 11.8% reduction in the shear strength, compared with the 5.3% reduction observed experimentally.

**NOTATIONS**

- $a_c$: clear shear span. Distance between the inner face of the support and the inner face of the loading plate
- $A_c$: composite concrete and steel cross-sectional area of the slab
- $d_{max}$: largest nominal maximum aggregate size
- $d$: effective depth of the longitudinal tensile reinforcement
- $f_c$: concrete compressive strength
- $f_{ck}$: characteristic concrete compressive strength
- $f_t$: tensile strength of concrete
- $f_y$: yield stress of the reinforcement
- $G_f$: fracture energy
- $T$: external tensile force
- $T_{cr}$: tensile force associated to the section cracking in pure tension
- $V_R$: shear strength
- $w$: Crack width
- $\epsilon_{y0}$: yielding strain of concrete in compression
- $\epsilon_{cu}$: failure strain of concrete in compression
- $\nu$: Poisson’s ratio
- $\rho$: reinforcement ratio
- $\sigma_t$: tensile stresses action on the concrete specimens

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**DATA AVAILABILITY STATEMENT**

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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