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Experimental and analytical study for retrofitting with thin concrete overlay

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Abstract

The retrofitting of existing reinforced concrete structures is of paramount importance due to aging structures, changes in design codes, and the need to enhance resilience against seismic events. This paper addresses this critical issue by presenting the results of an experimental campaign focused on evaluating the shear performance of old-to-new concrete interface, considering the application of a thin (5 cm) concrete overlay. Both monotonic and cyclic shear tests were conducted to assess the behavior under static and seismic loading conditions. Mechanical anchors were installed into the existing concrete as connectors. The study investigates the influence of different concrete types, i.e. normal-weight and lightweight concrete, used for the overlay. The experimentally obtained shear strength values were compared with predictions derived from established design codes and guidelines: Eurocode 2 (old and new versions) and EOTA TR066. The aim of the comparison was to understand if the available codes are able to predict the experimental behavior when using an overlay with limited thickness and a lightweight concrete. TR066 results conservative but able to predict the failure mode, while the new Eurocode 2 gives the best prediction of resistance, although it does not cover very short embedment depth and seismic action. Lastly, the evaluation of the reduction coefficient α_{seis} is made to account the behavior of the connectors under seismic action.

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Keywords: Concrete overlay; retrofitting; anchors; cyclic shear tests.

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1. Introduction

Aging of structures, coupled with evolving design standards and increasing load demands, presents a significant challenge to the integrity and serviceability of existing reinforced concrete (RC) structures. For this reason, in the last decades, the research has been devoted to the developing of rehabilitation strategies to extend their service life and ensure safety (Gkournelos et al., 2021). Concrete overlays have emerged as a promising approach for enhancing the performance of existing RC structures (Krstulovic-Opara et al., 1995, Fernandes et al., 2017, Raza et al., 2019, Abellán-García et al., 2024). It consists in casting an additional concrete layer (overlay) on existing structural elements (e.g., slabs, columns or beams) to improve their behavior, thereby creating a new interface between concretes cast at different times. The capacity for shear stress transfer at the interface determines whether the reinforced concrete element's cross-section works monolithically or decoupled.

When the concrete interface alone is not sufficient to guarantee adequate load transfer, it becomes necessary to provide additional transverse reinforcement. In such instances, stress transmission relies on three primary mechanisms: (i) adhesion, stemming from the chemical and physical bond between the two concrete layers; (ii) friction, which engages when stress is applied perpendicular to the interface, either externally or from the confining action of the reinforcement; and (iii) dowel action, a mechanism that necessitates appropriate concrete cover and embedment depth to achieve full efficacy. Several aspects influence the interface's shear resistance, including the concrete's compressive strength, the reinforcement ratio, and the surface roughness.

The application of concrete overlay is very common for the retrofitting of bridge decks (Krstulovic-Opara et al., 1995, Abellán-García et al., 2024), for columns jacketing (Raza et al., 2019) or strengthening of floor systems. However, in case of applications on existing buildings, the thickness and weight of the overlay should be limited as much as possible to avoid significant increase of loads on the structure as well to limit the dimensions.

This paper addresses the topic of concrete interface by presenting the results of an experimental campaign specifically designed to evaluate the shear performance of old-to-new concrete interfaces. The study focuses on a practical retrofitting scenario involving the application of a thin (5 cm) concrete overlay to existing concrete elements. Normal-weight and light-weight concrete were used for the new overlay. Mechanical anchors were installed into the existing concrete as connectors. Both monotonic and cyclic shear tests were conducted.

The key objective of the research is to critically evaluate the predictive capabilities of current international design codes and guidelines when considering a limited thickness for the overlay and a light-weight concrete. To this end, the experimentally obtained shear strength values are compared with predictions derived from several established standards, including the old Eurocode 2 EN 1992-1-1:2014 (European Committee for Standardization, 2014), its updated version EN 1992-1-1:2023 (European Committee for Standardization, 2023), and EOTA TR066 (European Organization for Technical Assessment, 2020), which specifically addresses post-installed shear connections. The evaluation of the parameter α_{seis} according to the EOTA EAD 332347 (European Organization for Technical Assessment, 2021) approach is also performed to take into account the behavior of the connectors under seismic action.

2. Experimental research

2.1. Materials and test setup

A total of ten specimens were tested considering monotonic and cyclic tests. The geometry of the specimens was chosen according to EAD332347-00-0601-v01 (European Organization for Technical Assessment, 2021) with an extra expanded polystyrene (EPS) sheet with thickness of 50 mm in order to be able to cast an overlay with a thickness of 50 mm. Each specimen was made of two concrete blocks, cast separately at least one month apart. The first block (blue in Fig. 1a) was cast first to simulate the existing concrete using C20/25 concrete (cubic compressive strength 28.4 MPa); after the casting, the interface surface (500x200 mm) was artificially roughened with the use of an electric chisel with different bits. The roughness of the interface was measured with the sand-patch method following the prescription of the EN13036-1:2010, with values between 2.87 and 3.05 mm for the different specimens. The second block (green in Fig. 1a) represents the added concrete and it was cast later using C20/25 ready mix normal concrete (cubic compressive strength 31.5 MPa) or LC25/28 lightweight concrete (cubic compressive strength 33.5 MPa). In the roughened area, three screw anchors with diameter 10 mm (Deutsches Institut für Bautechnik, 2022) and a nominal embedment depth in the existing concrete $h_{nom,ex}$=55mm were installed with a spacing of 170 mm.

The test setup was defined according to the EAD 332347-00-0601-v01 (European Organization for Technical Assessment, 2021) as shown in Fig.2. The "old" concrete block was fixed and the load was applied on the "new" block at the level of the new-old block interface. The slip was monitored via four displacement transducers (average value of the four transducers) applied on both side of the specimen (Fig. 2) and on both "front" and "back" (side of the jack) position. The tests were monitored with the MOOG system which acquired the measurements of the load cell and of the displacement transducers.

The monotonic tests were performed by pulling the new block until failure. The cyclic tests were conducted with increasing slip levels (three cycles at each shear slip level): ±0.05 mm; ±0.1 mm; ±0.2 mm; ±0.4 mm; ±0.7 mm; ±1.0 mm; ±1.50 mm; ±2.0 mm; ±3.0 mm.

A total of 10 tests were performed, 4 static (S) and 6 cyclic (C). The static tests were performed on 2 specimens with normal-weight concrete (NC) and 2 with light-weight concrete (LC). The cyclic tests were performed on 3 specimens with NC and 3 with LC. One static test with LC (S10-2- LC) will be not considered in the following because the specimen was not properly casted and the interface between the old and the concrete appeared discontinuous.

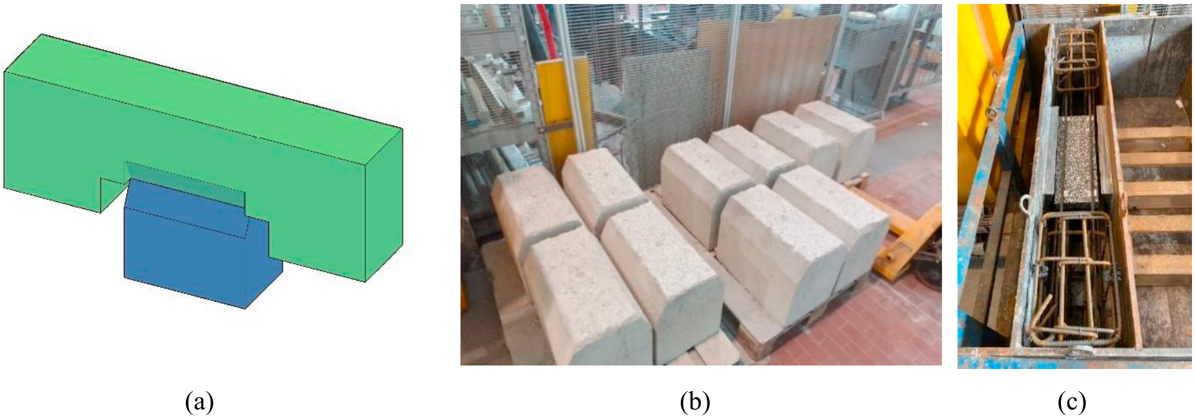

(a) (b) (c)

Fig. 1. (a) Specimen geometry; (b) "old" block; (c) "new" block before casting with EPS sheet.

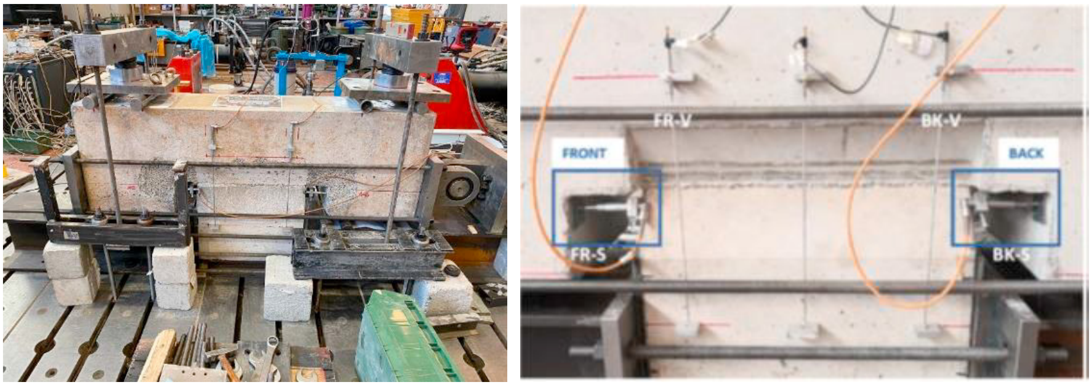


Fig. 2. Left, test setup. Right, front and back transducers.

2.2. Results

A summary of the results is reported in Table 1 in which, for each group of tests, the average values of maximum load (with the respective coefficient of variation), maximum slip, and slip at the peak load are reported. The average cubic concrete strengths of old and new concrete are reported too. In case of monotonic static (S) tests, the LC and NC tests are grouped together due to the limited number of specimens and the fact that the results were quite similar.

Table 1. Test results.

Test	Max load [kN]	CoV [%]	Max slip [mm]	Peak load slip [mm]	Concrete strength [MPa]	
					old	new
S10 LC/NC	144.24	15.5	8.27	3.00	28.4	31.5/33.0
C10 LC	121.7/-123.6	23.4/36.2	+1.67/-1.57	+1.36/-1.29	28.4	31.5
C10 NC	99.8/-133.2	16.7/13.8	+3.33/-3.67	+2.06/-2.16	28.4	31.5

In all tests, the first crack appeared at the interface between old and new concrete (red line, Fig. 3). Then an airline crack appeared at the top of the concrete overlay (interface between concrete and EPS panel, blue line, Fig. 3). Finally, a diagonal crack on one side of the specimen appeared (green line, Fig. 3) due to the high stress concentration induced by the reaction of the external constraints (Cattaneo et al., 2021). The final failure was usually associated with concrete cone failure in the concrete overlay, and in some cases (test C10-2 LC and C10-4NC), also the pull-out of the anchor in the existing concrete close to the diagonal crack was observed (Fig. 4).

Figure 5 reports the load-displacement curves of monotonic and cyclic tests. An unexpected result was observed both in monotonic and cyclic tests: lightweight concrete showed better performances (in particular in terms of stiffness) than ordinary concrete. Unfortunately, no additional specimens were available to confirm this behavior. Moreover, although the values of the peak are similar for both types of concrete, the peak load is reached at larger displacement in normal weight concrete with respect to lightweight concrete.

Figure 6 shows the peak values at first, second and third cycles vs slip for each specimen. Although the values of the peak are similar for both concretes, the peak load is reached at larger displacement in normal-weight concrete than in light-weight concrete. The same behaviour was observed for monotonic load (see diamond dots in Fig. 6a).

Fig. 3. Damage of the specimen after test.

Fig. 4. Pull-out of the anchor and concrete cone failure of the overlay.

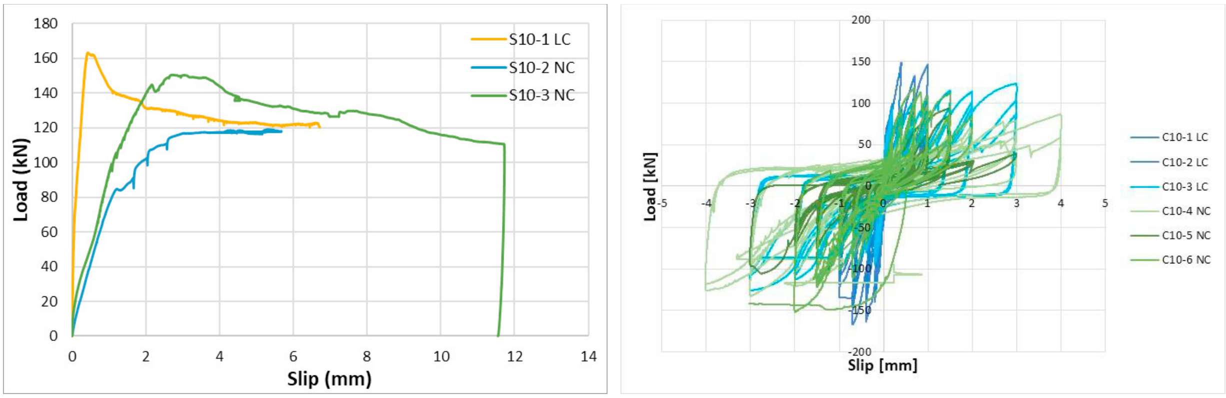
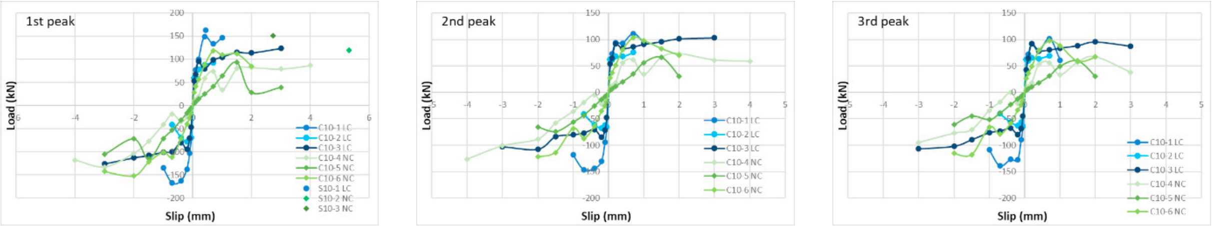


Fig. 5. Load-displacement of monotonic (left) and cyclic (right) tests.



(a) (b) (c)

Fig. 6. Peak load vs slip for the (a) first, (b) second and (c) third cycle.

3. Code provisions

Various international design codes and guidelines provide methodologies for assessing the shear resistance associated to the interface, often differing in their underlying assumptions and empirical formulations.

The core formulation proposed by the old Eurocode 2 (European Committee for Standardization, 2014) for the design shear stress resistance, τ_{Rd}, is based on a combination of cohesion and friction :

$$\tau_{Rd} = cf_{ctd} + \mu\sigma_n + \rho f_{yd}(\mu \sin\alpha + \cos\alpha) \leq 0.5\nu f_{cd} \quad (1)$$

where c is the cohesion and μ is the friction, both depending on the roughness of the surface and whose values are given by the code, f_{ctd} is the design tensile strength of concrete, σ_n is the normal compressive stress across the interface, ρ is the reinforcement ratio, f_{yd} its design yield strength, α is the angle of inclination of the connector and f_{cd} is the design compressive strength of concrete. An upper limit is imposed to prevent concrete crushing in the shear plane. The code assumes a sufficiently anchored shear reinforcement and, for this reason, steel yielding is considered as the decisive failure mode. Therefore, the case where the full anchorage of the reinforcement cannot be ensured is not covered.

The updated version of Eurocode 2 (European Committee for Standardization, 2023) kept the fundamental concept of combining cohesion and friction for the interface shear resistance, but recalibrated the values of the coefficients in order to obtain a more accurate design, considering also non-fully anchored bars. Indeed, two different formulations have been proposed for different reinforcement configurations. When the interface is without reinforcement or if the required reinforcement across the interface is anchored for $\sigma_{sd} = f_{yd}$ (Y), the design shear stress resistance at the interface may be taken as:

$$\tau_{Rd} = c_{v1}\sqrt{\frac{f_{ck}}{\gamma_c}} + \mu_v\sigma_n + \rho f_{yd}(\mu_v \sin\alpha + \cos\alpha) \leq 0.30\nu f_{cd} + \rho f_{yd}\cos\alpha \tag{2}$$

where the coefficient c_{v1} (cohesion) and μ_v (friction) have been recalibrated, and f_{ck} is the lowest compressive strength of the concretes at the interface.

If yielding of the required reinforcement crossing the interface is not ensured (NY), due to insufficient anchorage, the shear stress resistance is calculated as the combination of mechanical interlock, friction and dowel action. It may be taken as:

$$\tau_{Rd} = c_{v2}\sqrt{\frac{f_{ck}}{\gamma_c}} + \mu_v\sigma_n + \kappa_v\rho f_{yd}\mu_v + \kappa_{dowel}\rho\sqrt{f_{yd}f_{cd}} \leq 0.25 f_{cd} \tag{3}$$

where c_{v2}, k_v, k_{dowel} are all coefficients which depend on the roughness of the interface as defined by the code. The interface reinforcement should be anchored for a stress of at least $0.5f_{yd}$ with a minimum length of embedment of 8ϕ if no other methods of anchorage than by straight bars are applied. For interface reinforcement with α=90° and an embedment length of at least 8ϕ, but anchored for a stress lower than $0.5f_{yd}$, Equation (3) may be used with $c_{v2}=\mu_v=k_v=0$, i.e., only the contribution of dowel action is taken into account.

The EOTA Technical Report TR066 (European Organization for Technical Assessment, 2020) has been specifically developed to address the interface design. It provides a formulation which allows to design interfaces with shear connectors provided with an anchorage length smaller than the one required for full anchorage and considers the properties of engineered connectors in terms of material ductility, cross section geometry and pullout resistance. Consequently, the steel stress of the shear connector calculated from the design resistance under tension is used instead of the yield strength. The design shear stress resistance at the interface according to the TR066 is:

$$\tau_{Rd} = c_r f_{ck}^{\frac{1}{3}} + \mu\sigma_n + \mu\kappa_1\alpha_{k1}\rho\sigma_s + \kappa_2\alpha_{k1}\rho\sqrt{\frac{f_{yk}}{\gamma_s}\frac{0.85f_{ck}}{\gamma_c}} \leq \beta_c\nu\frac{0.85f_{ck}}{\gamma_c} \tag{4}$$

where c_r, μ, κ_1, κ_2 are all parameters related to the roughness of the surface and whose values are given by the code, β_c and ν are coefficient related to the concrete strength, $\alpha_{\kappa1}$, $\alpha_{\kappa2}$ are coefficients given in the European Technical Assessment of the connector, σ_s is the steel stress associated to the relevant failure mode.

The mentioned formulations are used in the following to predict the values obtained from the experimental research. The safety factors will be set equal to 1. The analytical results are given in Table 2, in which the different terms of the formulations associated with the different resisting mechanisms are reported, together with the shear stress resistance (τ_{Rd}) and the ultimate shear force (V_u).

Table 2. Analytical prediction according to the different codes.

Code	Adhesion	Friction	Dowel action	τ_{Rd} [MPa]	V_u [kN]
EC2:2014	0.987	0.926		1.913	191.3
EC2:2023 Y	0.729	0.926		1.655	165.5
EC2:2023 NY	0.389	0.463	0.279	1.131	113.1
EC2:2023 NY ($\sigma_{sd} < 0.5f_{yd}$)	-	-	0.279	0.279	27.9
TR066	0.287	0.081	0.229	0.596	59.6

It should be highlighted that the different specimens had the same geometry, roughness, reinforcement configurations and compressive strength of the old concrete block (the only difference is the type of concrete of the new block). For this reason, only one value is given for each formulation.

The reference mean experimental values to consider for the comparisons are V_u = 144.2 kN in case of static load, and V_{1max}/V_{1min} = 110.7 kN/-128.4 kN in case of cyclic load. Eurocode 2:2014 (and the new version of the Eurocode 2:2023 in the yielded (Y) configuration), considering the yielded connector, strongly overestimates the load. The new version of the Eurocode 2:2023 in the non-yielded (NY) configuration and with a steel stress higher than $0.5f_{yd}$ is the one which better predict the experimental value, even if the experimental configuration does not fulfill the minimum embedment length required (8ϕ). TR066 results very conservative, even if it is able to predict the correct failure mode of the shear connectors, i.e. the concrete cone failure.

Based on the experimental data and considering the approach which is able to predict the observed failure mode, i.e., TR066, it seems not necessary to introduce a corrective coefficient to account for the limited thickness of the overlay since the analytical results are still very conservative.

In case of lightweight concrete, no significant effect on the behavior was evident from the experimental evidence. However, according to the existing literature (Palieraki et al., 2022), a coefficient α_{LC} = 0.75 is recommended to be applied on the adhesion/interlock and friction components, but not on the dowel action. The same coefficient is also suggested by the ACI 318 (ACI 318-19, 2019) for the pullout verification of post-installed anchors.

4. Behavior under seismic action

The evaluation of the parameter α_{seis} is made according to the draft of EAD 332347-00-0601-v01(European Organization for Technical Assessment 2021) to take into account the behavior of the connectors under seismic action. According to the formulation, in order to assess the seismic performance, the following values must be evaluated: the average peak resistance of monotonic tests ($V_{um,mon}$); the peak resistance in the loading direction (first cycle) where the maximum resistance is recorded, measured in each test ($V_{u,cyc,1}$); the peak resistance in the loading direction (first cycle) where the absolute maximum/minimum resistance is recorded, measured in each test ($V_{u,max,1}$ and $V_{u,min,1}$); the peak resistance at the first and third cycles calculated averaging the peaks recorded in the two loading directions, measured in each test ($V_{u,ave,1}$ and $V_{u,ave,3}$). Threfore, the reduction factor for seismic cyclic loading α_{seis} can be evaluated as:

$$\alpha_{seis} = \beta_{cv,seis} \cdot \alpha_{seis,1} \cdot \alpha_{seis,2} \cdot \alpha_{seis,3} \tag{4}$$

$$with\ \ \beta_{cv,seis} = min\left\{\beta_{cv,seis,1} \cdot \beta_{cv,seis,2} \cdot \beta_{cv,seis,3}\right\} \tag{5}$$

where $\alpha_{seis,1}$ is the reduction of peak resistance due to cyclic loading, $\alpha_{seis,2}$ is the reduction due to non-symmetric response in the two loading direction, $\alpha_{seis,3}$ is the reduction due to in-cycle degradation, $\beta_{cv,seis,1}$ is the reduction factors due to large coefficient variation of $V_{u,cyc,1,I}$, $\beta_{cv,seis,1}$ is the reduction factors due to large coefficient variation of $\alpha_{seis,2}$, $\beta_{cv,seis,1}$ is the reduction factors due to large coefficient variation of $\alpha_{seis,3}$. For further details see (European Organization for Technical Assessment 2021).

The factors obtained from the experimental data are summarized in Table 3. The value of the seismic reduction factor for the thin overlay results in line with the value reported in the ETA of the anchor (Deutsches Institut für Bautechnik 2022) in case of thick overlay.

Table 3. Evaluation of α_{seis}.

$\alpha_{seis,1}$	$\alpha_{seis,2}$	$\alpha_{seis,3}$	$\beta_{cv,seis,1}$	$\beta_{cv,seis,2}$	$\beta_{cv,seis,3}$	α_{seis}
0.77	1.00	0.71	1.00	1.00	1.00	0.54

5. Conclusions

An experimental and analytical study focused on the retrofitting of existing reinforced concrete structures through the application of a thin (5 cm) concrete overlay has been presented. The experimental research evaluated the shear performance of the old-to-new concrete interface under both monotonic and cyclic loading conditions, also investigating the influence of using lightweight concrete for the overlay.

From the experimental campaign, a typical failure mechanism emerged, starting with cracking at the old-to-new interface, followed by a crack near the EPS panel, and finally a diagonal crack on the "old" block side. The final failure was generally associated with concrete cone failure in the overlay. An unexpected result was observed, with lightweight concrete showing better performance in terms of stiffness compared to ordinary concrete, although with a similar peak load reached at larger displacements for normal-weight concrete.

The comparison of analytical predictions obtained with different design codes (Eurocode 2, old and new versions, and EOTA TR066) show considerable variability, with TR066 and Eurocode 2:2023 (non-yielding scenario, NY) predicting more conservative shear resistance and ultimate force values compared to Eurocode 2:2014. Indeed, for the studied application, yielded (Y) steel assumption resulted not realistic nor on the safe side. TR066 seems to be very conservative, although it can predict the observed failure mode (cone failure in the overlay). EC2:2023 (non-yielding scenario, NY) is the formulation which better predict the experimental values, although the correct value should account for a steel stress lower than $0.5f_{yd}$. Moreover, the embedment length of at least 8ϕ is not fulfilled in the studied case. Lastly, according to experimental data, a coefficient of $\alpha_{seis} = 0.54$ should be applied in order to take into account the seismic behavior of the connectors.

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