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CHARACTERISATION TESTING OF MORTAR JOINTS USED IN PRECAST WALL-SLAB-WALL STRUCTURES

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SUMMARY: *As found very frequently in existing precast wall-slab-wall structures in some Northern European countries such as the Netherlands, high-strength mortar is commonly employed to execute both wall-wall and foundation-wall joints featuring no mechanical devices, whereas non-shrink low-strength mortar is used to make up wall-slab connections, replacing fabric felt material under certain circumstances. Material characterisation testing is deemed necessary to establish whether these wet joints are strong enough to allow horizontal forces on a building to be carried through them, thereby preventing the onset of damage in the connectors' sockets as well as the sliding of precast walls and eventually their toppling. In view of this, standard compression and three-point flexural tests have been integrated with bond wrench and triplet tests so as to provide an estimate of the shear force transfer capacity at the base of actual buildings as well as along the boundaries of the connectors' sockets by adhesion. Results from shake-table testing on a full-scale building mock-up – available at the Experiments platform of the Built Environment Data initiative – demonstrate the accuracy of information and analytical relationship derived here, thus making them applicable in numerical modelling efforts.*

KEYWORDS: precast wall-slab-wall structure, wall-foundation connection, wall-wall connection, high-strength mortar, bond wrench testing, triplet testing

1 Introduction

Reinforced precast concrete wall-slab-wall structures, the cast-in-place versions of which are sometimes also named as ”tunnel buildings”, are relatively widespread all over Europe and abroad, meaning that they can be found both in seismic-prone areas and in countries that are not exposed to potential large tectonic earthquakes [1, 2, 3, 4, 5, 6]. Their structural scheme does not feature any beams or columns, with these structures being instead made up of precast wall and slab panels assembled together by means of mortar, fabric felts and steel connectors, the latter option being however opted for the wall-wall connection only. These connection systems have been shown to be uncapable of preventing relative sliding between the precast panels for even low levels of horizontal lateral force in the building, something that renders a structure of

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this type particularly vulnerable to earthquake loading [7, 8].

Particularly in countries such as the Netherlands, where precast wall-slab-wall structures abound, the abovementioned structural deficiencies stem from a combination of reasons, since the country has never been historically prone to hazard from natural seismicity or tectonic earthquakes and, consequently, the design philosophy for these structures, mostly employed for housing, is based on the gravity+wind loads-only design rationale, with designers and contractors being motivated to opt for such an approach and concepts by the lack of seismic design provisions. The fact that induced seismicity in the region of Groningen (i.e. north of the Netherlands) started being observed because of reservoir depletion due to gas extraction is the spark that brought into question the seismic resistance of structures there, calling for experimental testing campaigns on full-scale buildings [7, 8] and components/materials [9, 10], as part of a much wider research programme for seismic risk assessment of the Groningen region [11]. Worthy of mention is also that interested readers can find all experimental data related to the abovementioned testing campaigns openly archived and available at the Experiments platform of the Built Environment Data initiative.

The material characterisation testing presented in this paper falls within the same framework and aims at characterising the mechanical properties of mortar, which is commonly employed in the interfaces underneath the wall panels and sometimes also in the wall-slab interfaces in replacement of felt [8]. To the authors' knowledge, similar testing has neither been found for Groningen specific structures nor in other studies involving such structural typologies (see e.g. [2, 3, 4, 5, 6]). Within this in mind, the results of bond wrench and triplet tests, meant to accompany standard compression and three-point flexural tests, are described in what follows, with a view to provide mechanical properties and analytical relationship that may be employed to estimate (i) the maximum shear force that can be transferred at the base of a building, between the foundations and the precast walls, through joints that merely consist of high-strength mortar, and (ii) the maximum adhesion that can be mobilised along the boundaries of the connectors' sockets by the same mortar. These main objectives notwithstanding, the same information may be used towards the numerical modelling of these building components, thus adding further weight to the undertaken characterisation testing and corresponding recommendations.

2 Precast technology for wall-slab-wall structures and their base wall joints

As can be gathered from Figure 1, these precast wall-slab-wall structures, which are mostly employed for housing in the Groningen region and more in general in the north of the Netherlands, consists of low-rise, single-unit or multi-unit buildings, typically single-storey or two-storey houses that are highly-standardised in terms of both building system and construction process [12]. According to Dutch building practice, these houses feature precast floors, precast party/gable walls and precast load-bearing walls in the longitudinal and transverse direction, with the installation process of both wall and slab panels that is initially crane-driven and then handled manually by the builders such that the panels are finally set in place.

As an example, Figure 2 shows the installation of the first-storey shear/stability and transverse walls of a full-scale two-storey house tested at Eucentre [8]. Particularly noteworthy is the absence of steel rebars protruding into/from the walls (i.e. neither starter rebars are present at the base of the ground-floor walls, nor rebars pass through the floor to connect the first- with the second-storey walls), meaning that the base wall connections are simply high-strength mortar joints. In the construction of these structures thus, firstly, the walls are seated onto 2-3 cm thick plastic spacers placed in-between their base and the foundation (or the hollow core slab), and the panels are hold as upright as possible before steel diagonal props are installed temporarily to shore them up. Then, the steel connectors are lodged into the sockets that form once the two adjacent and the orthogonal wall panels are set in place. Finally, the base wall and wall-wall joints are executed, filling-in the interspace at the base of the walls and the connectors' sockets with high-strength mortar. After such operations are completed, the slab panels are settled onto the walls using fabric felts as interface material and the process is repeated for the upper storeys.

Figure 1: Typical precast concrete houses [12]

Figure 2: Construction details concerning the installation of precast panels and the execution of mortar joints – adapted from Brunesi et al. [8]

However, it is rather common that the floor-to-wall connections are turned into wet joints made up in this case of non-shrink, low-strength mortar. Indeed, imperfections of the wall panels, together with excessive pre-bending in the hollow core slab ones, may hamper the possibility of having them fully in contact with each other, in which case it is customary practice to remove the felts, whenever possible, and to restore the continuity by filling-in the gap between the precast wall/slab panels with the same mortar used for the other structural interfaces [12, 8].

These key construction details, which are inevitably quite unfamiliar to building practice in countries prone to tectonic earthquakes, are collected in Figure 2a-e, whereas interested readers are referred to [8, 13] for more exhaustive information on the full-scale specimen subjected to five shake-table test runs of progressively increased intensity.

As can be gathered from Figure 3, where design drawings are collected for the sake of completeness and clarity, the full-scale building specimen was a two-storey, single-bay structure with a precast system that features two half-span long stability walls, one per each floor, placed perpendicular to transverse walls, each of which consisted of two 2.00 x 2.66 m precast panels. The plan dimensions of the specimen were 4.0 x 5.5 m, meaning that the 250 mm-thick floors consisted of four hollow core slab panels placed side-by-side and spanning 5.5 m to the lateral walls. A 50 mm-thick concrete screed was provided alongside a floor steel mesh consisting of 10-mm rebars spaced at 150 mm. All precast wall panels were 120 mm thick and were provided with a centreline-placed steel grid Ø5/250 of ribbed rebars; it is noted, however, that the latter is not particularly relevant for the seismic behaviour of this type of structures, given their rocking-and-sliding-dominated response [8]. It is instead the behaviour of the 2 cm-thick, poorly-cured, high-strength mortar base wall joints that becomes thus crucial for such a response mechanism, this being the reason behind characterisation testing effort described in the subsequent sections of this manuscript.

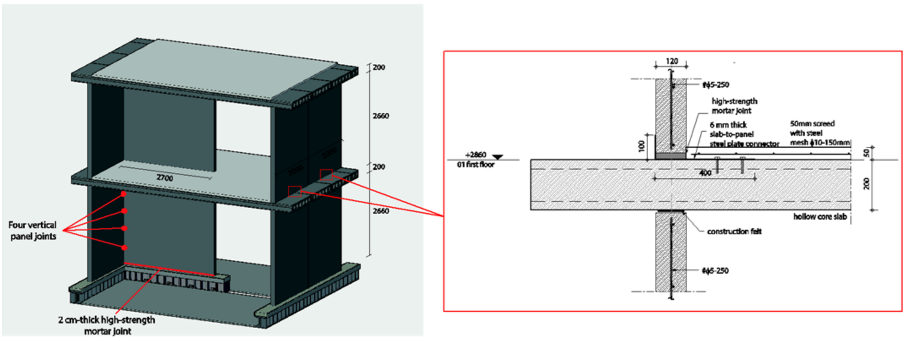


Figure 3: Schematics of the full-scale building specimen tested in dynamic fashion by Brunesi et al. [8]

The design dead and live loads were simulated with support of eight and six concrete blocks on the first and second floors, spaced equidistantly along two lines per each storey, as can be retrieved from Brunesi et al. [8]. The blocks were fixed to the slabs and had dimensions equal to 35 x 60 x 94 cm, for a total specimen's weight of approximately 510 kN. Dynamic test runs on the building specimen were executed through sequencing accelerograms selected by Crowley [14] based on Groningen probabilistic seismic hazard assessment (PSHA) results available at the time of the test. Readers are referred to Bourne et al. [15] for PSHA results, to Crowley [14] for details on input selection, and to the Experiments platform of the Built Environment Data initiative, where the seismic input used for testing can be downloaded, together with full-scale building test data presented in Section 4.

3 Laboratory testing

The experimental activities described in what follows involve standard compression and flexural-tensile strength tests presented as background information, along with bond wrench and triplet testing of mortar. Pivotal aspects and assumptions made for the material characterisation testing campaign carried out at the Eucentre laboratory (Pavia, Italy) are presented, splitting the treatment into three sub-sections in order to report them and key results in a systematic manner.

3.1 Compressive and flexural-tensile strength tests

Standard compression and three-point flexural testing of mortar samples was carried out, as shown in Figure 4, which presents the setup for each type or set of tests. Tests were undertaken according to EN 1015-11 [16], which sets standards for testing procedure and preparation of the prismatic samples to test. All mortar samples, whose dimensions are 160 x 40 x 40 mm, were cured following the recommendations of EN 1015-11 [16] and were tested in force-controlled mode, at a constant load rate. It is worth mentioning that three series of tests were performed to characterise the mortar (i) for the wall base joints, (ii) for the two- and three-way panel connections, and (iii) for the floor-to-wall joints, with the latter being made up of non-shrink, low-strength mortar such as that used for the other structural interfaces in this type of buildings (e.g. interfaces between two slab panels or between two adjacent transverse wall panels). Twenty-four mortar samples were prepared in total, meaning that three samples were prepared per each mortar supply.

(a) (b)

Figure 4: Photographs of the setup for (a) compression and (b) three-point flexural testing of mortar samples

More specifically, six mortar specimens were prepared during the gap-filling intervention, three for the first-storey slab gap and other three for the second-storey slab gap. In addition to them, eighteen further samples were prepared and tested for the high-strength mortar employed to execute the base wall wet joints and the wall-wall ones, nine per each, thus keeping the same proportion between mortar supply and number of samples.

With the above twenty-four specimens, seventy-two tests could actually be carried out, two-thirds of which were compression tests and the remaining third were flexural-tensile ones, because after the execution of the flexural-tensile test on each one of the twenty-four rectangular specimens, these resulted to be split into two almost identical parts that could then be subsequently tested in compression. For practical reasons, the samples were placed in horizontal position, implying that a decision was taken not to treat/disturb it after three-point flexural testing in order to smoothen surfaces (whenever possible) and test prisms in vertical position. As recommended by EN 1015-11 [16] in clause 9.1(b), stiff plates made of steel were used to distribute the applied load.

The compressive strength (f_c) was computed as reported in Eq. (1):

$$f_c = \frac{P}{A_{st}}, \tag{1}$$

where P is the applied compression force and A_{st} is the area of the 40 x 40 mm^2 steel plates used to carry out the compressive tests (see clause 9.1(b) in EN 1015-11 [16]).

The flexural-tensile strength (f_{ft}), or indirect tensile strength (sometimes also named as modulus of rupture), can be calculated according to Eq. (2):

$$f_{ft} = \frac{3Pl}{2A_{sa}b}, \tag{2}$$

where A_{sa} and b are the area and width of the prismatic sample, respectively, and l is the distance between the axes of the two rollers onto which the sample is placed. The latter was set equal to 100 mm, in line with EN 1015-11 [16] – see Figure 2 of that standard.

Table 1 reports all test results, whereas Figure 5 shows them graphically, differentiating the mortar types depending on their use/purpose.

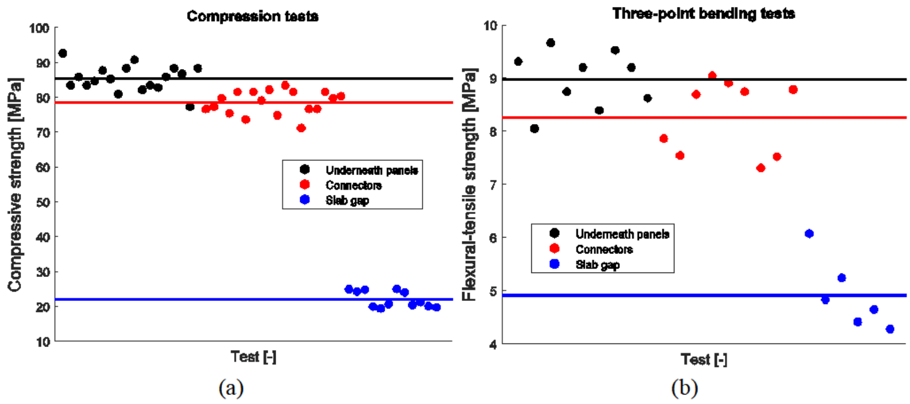


Figure 5: (a) Compressive and (b) flexural-tensile strength values for each type of mortar

The results summarised in Table 1 seem to indicate a relatively moderate degree of variation in the resistance values of the mortar employed for gap-filling purposes. A flexural-tensile strength value of about 5.38 and 4.45 MPa was obtained, on average, for the mortar used at the upper and lower slab gaps, respectively. Relatively low values of coefficient of variation (CoV) can also be computed from these results, particularly if those pertaining to upper and lower slabs are considered separately (i.e. 4 and 12%). The average compressive resistance of the mortar used for wall-wall connections and joints at the base of the precast wall panels is 78.5 and 85.4 MPa, respectively; the average flexural-tensile strength of the former type is 8.26 MPa, whilst that of the latter one is 8.96 MPa. In both cases, the CoV associated with the flexural-tensile resistance is higher than that corresponding to the compressive one (6-8% versus 4%).

Table 1: Results of compressive and flexural-tensile strength tests of mortar underneath the precast panels, for the two- and three-way (vertical) panel joints, and for gap filling in the floor-to-wall connections

Label	Type	Flexural-tensile strength [MPa]	Compressive strength [MPa]	
			Sample 1 of 2	Sample 2 of 2
P1_P	Underneath panels	9.31	92.58	90.74
P2_P	Underneath panels	8.05	83.39	82.16
P3_P	Underneath panels	9.66	85.84	83.39
P4_P	Underneath panels	8.74	83.39	82.77
P5_P	Underneath panels	9.20	84.61	85.84
P6_P	Underneath panels	8.39	87.68	88.29
P7_P	Underneath panels	9.52	85.22	86.76
P8_P	Underneath panels	9.20	80.93	77.25
P9_P	Underneath panels	8.62	88.29	88.29
	Mean [MPa]	8.96	85.41	
	CoV [%]	6.0	4.3	
P1_C	Connectors	7.86	76.64	74.80
P2_C	Connectors	7.54	77.25	83.39
P3_C	Connectors	8.69	79.71	81.55
P4_C	Connectors	9.04	75.41	71.12
P5_C	Connectors	8.90	81.55	76.64
P6_C	Connectors	8.74	73.58	76.64
P7_C	Connectors	7.31	81.55	81.55
P8_C	Connectors	7.52	79.09	79.71
P9_C	Connectors	8.78	82.16	80.32
	Mean [MPa]	8.26	78.48	
	CoV [%]	8.4	4.3	
P1_G	Slab gap (2nd storey)	6.07	24.89	24.95
P2_G	Slab gap (2nd storey)	4.83	24.28	23.97
P3_G	Slab gap (2nd storey)	5.24	24.77	20.36
P4_G	Slab gap (1st storey)	4.41	19.93	21.21
P5_G	Slab gap (1st storey)	4.64	19.37	20.05
P6_G	Slab gap (1st storey)	4.28	20.66	19.74
	Mean [MPa]	4.91	22.02	
	CoV [%]	13.4	10.5	

Somewhat surprisingly, very high compressive and flexural-tensile strength values are obtained, the former ones being close to or even higher than 80 MPa and the latter ones being higher than one-tenth of the compressive resistance counterpart in the overwhelming majority of the cases. Even when the flexural-tensile strength values listed in Table 1, as obtained according to EN 1015-11 [16], were to be converted into direct tensile strength values assuming conservative reduction factors of up to 1.5 [17, 18], data would still be indicative of high tensile resistance. Should this lower bound approach be considered, as for instance recommended in [17, 18], direct tensile strength estimates would indeed range between 5.36 and 6.44 MPa for mortar for wall base joints; and between 4.87 and 6.02 MPa for mortar for wall-wall vertical joints. Estimates of direct tensile strength are thus approximately 6.0-8.5% of the compressive strength.

3.2 Bond wrench tests

As anticipated in Section 3, a set of bond wrench tests on concrete-mortar specimens was also undertaken such that the bond strength of this material, which in this test type forms the horizontal bed-joint of a two-concrete-bricks specimen, could be characterised. Bond wrench testing was executed in accordance with EN 1052-5 [19] to characterise mortar for wall-wall vertical joints, namely that employed to execute the connectors' sockets. Although this type of test is based on a well-known and established approach, it is mostly applied to the case of masonry structures (amongst others see e.g. Graziotti et al. [20]). As such, some key aspects of both the experimental setup and the testing procedure are reported hereafter, whilst the reader is referred to Section 3.2.2 for the full set of experimental results and statistics that were obtained.

3.2.1 Experimental setup and procedure

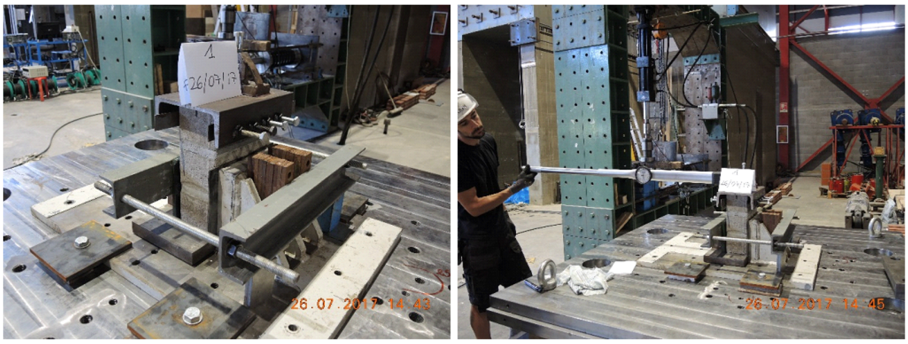
Figure 6 shows the type of specimen used for these tests and the test rig, whereas Figure 7 presents schematics that clarify further the specimen's geometry and rationale behind the testing method. The specimen consists of three 195 x 95 x 80 mm concrete bricks connected, two by two, by a bed-joint. The mean depth of the specimen is 95 mm and the mean width of the tested bed-joint is 195 mm. The thickness of the concrete bricks is 80 mm (t_b = 80 mm). All specimens were prepared and cured following the recommendations specified in EN 1052-5 [19], meaning that concrete-mortar units were firstly checked for linear alignment. Then, excess mortar was struck off and the specimens remained compressed and undisturbed until testing.

As can be gathered from Figure 6b, the test brick, namely the top one, is subjected to a bending moment and a compressive force, due to the application of two forces [19], whilst the remaining part of the specimen is clamped. The bond strength (f_{wi}) was calculated as:

$$f_{wi} = \frac{F_1 e_1 + F_2 e_2 - \frac{2}{3} d_s \left(F_1 + F_2 + \frac{W}{4}\right)}{Z}, \tag{3}$$

where d_s is the mean depth of the specimen, Z is the section modulus of the projected plan area of the failure surface, e_1 is the distance from the applied load to the tension face of the specimen, e_2 is the distance from the centre of gravity of the lower and upper clamp from the tension face of the specimen, F_1 is the applied load, F_2 is the weight of the bond wrench, and W is the weight of the concrete unit pulled off the specimen and any adherent mortar.

Note that $Z = b_j d_s^2/6$, where b_j is the mean width of the bed-joint tested. Thus, in this case, Z = 293312.5 mm^3 because d_s = 95 mm and b_j = 195 mm. For the sake of completeness, it is worth specifying also that e_1 = 298 mm, e_2 = 1327 mm, and F_2 = 122.5 N.



(a) (b)

Figure 6: Bond wrench testing: (a) example of concrete-mortar test specimen and (b) apparatus for base clamping and application of loads

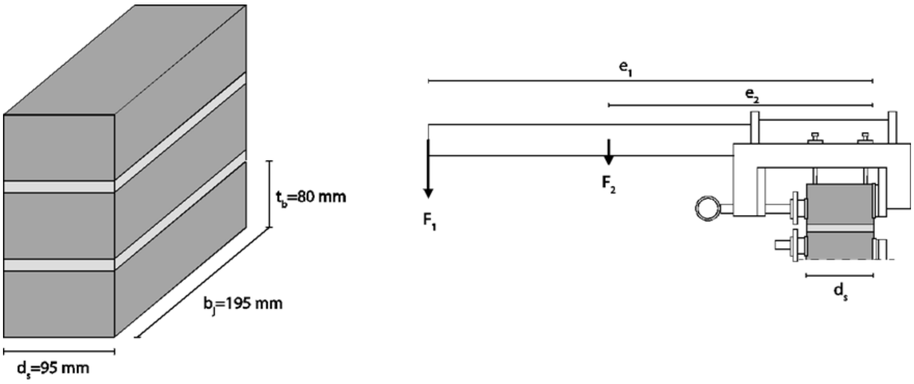
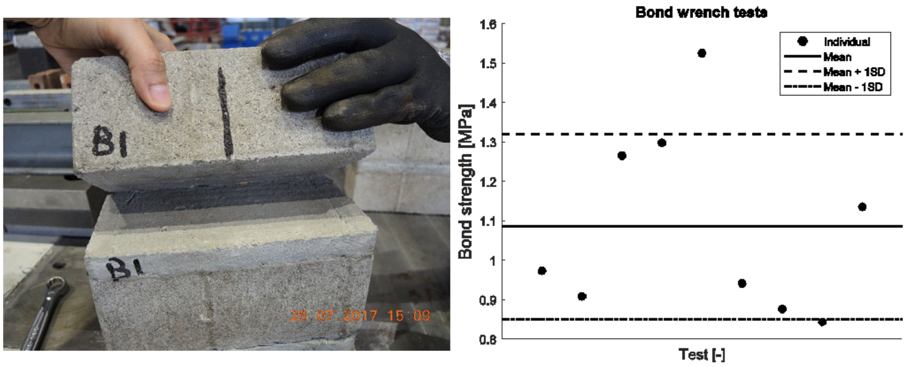


Figure 7: Schematics of concrete unit for bond wrench testing and points of application of F_1 and F_2



(a) (b)

Figure 8: (a) Observed failure mode in one of the bed-joints tested (i.e. bed-joint #01, A1-type) and (b) bond strength values for all bed-joints tested (i.e. individual and statistical values)

Test results are summarised in what follows.

3.2.2 Test results

Figure 8a shows the failure mode observed in one of the bed-joints tested, whilst all bond strength values are collected in Figure 8b, along with the statistics obtained from the undertaken testing (i.e. mean and mean plus/minus one standard deviation). As reported in Table 2, all specimens failed according to the A1- or A2-type of failure described in the EN 1052-5 [19], and no intermediate modes of failure were observed. The bond strength was found to range approximately from 0.843 to 1.525 MPa, and the mean value of this parameter was computed to be 1.085 MPa, as can be seen in Figure 8b. Furthermore, the standard deviation and CoV are 0.235 MPa and 21.6%, respectively, both of which values indicate a relatively high degree of variation.

Table 2: Summary of bond wrench test results

Bed-joint	Weight	Applied Force	Bending Moment	Bond strength	Way of failure
	[N]	[N]	[Nm]	[MPa]	
#01	30.02	203.47	270	0.973	A1
#02	31.97	188.39	250	0.908	A1
#03	36.68	271.29	360	1.265	A2
#04	30.53	278.82	370	1.298	A1
#05	37.12	331.57	440	1.525	A2
#06	31.09	195.93	260	0.941	A1
#07	30.61	180.86	240	0.876	A1
#08	31.45	173.32	230	0.843	A1
#09	31.01	241.15	320	1.135	A1

3.3 Triplet tests

In addition to previous characterisation tests aimed at assessing the bond strength of mortar, nine further triplet specimens, such as the one shown in Figure 9, were prepared and were each tested under three different levels of axial load so as to estimate the cohesion and frictional resistance of this material, something mandatory in reflection of the fact that, as discussed previously, in several existing buildings of this type the precast wall-elements are simply resting on their concrete foundations without any starter rebars that protrude from the foundation into the wall panel. This implies that the entire lateral-force resisting system of these precast wall-slab-wall structures relies only on the shear-friction resistance of mortar, as indeed confirmed by the shake-table tests undertaken by the same authors (Brunesi et al. [8]), whereby it has been showcased how damage in this type of buildings is inevitably associated with the base wall wet joints.

Comprehensive information on the specimens, each of which consisting of three concrete blocks and two mortar layers in-between them, as well as on the setup and rationale behind these tests can be found in the following Section. Note that concrete-mortar specimens were prepared, cured and tested complying with EN 1052-3 [21].

Figure 9: Example of concrete-mortar triplet specimen installed in the testing apparatus

3.3.1 Experimental setup and procedure

Figure 10 presents a photograph of the testing apparatus and an enlarged view of one of the triplet specimens, disassembled at the end of one sequence of tests.

(a) (b)

Figure 10: Friction testing of triplets: (a) test setup for application of vertical and horizontal forces and (b) triplet specimen at the end of the test

The concrete blocks were 195 x 95 mm^2 and had thickness equal to 80 mm. The mortar layers were approximately 2 cm thick each, meaning that they were assumed to have the same thickness as that of high-strength mortar joints at the base of similar buildings in practice [8]. After each specimen was assembled, a uniformly distributed mass was applied on top of it in such a way that the triplet could be kept compressed and undisturbed until testing [21]. Triplet tests were all executed after 28 days of curing (EN 1052-3 [21]) through the following procedure.

The specimen, placed in the testing apparatus between two steel plates, is supported by roller bearings, and a compression force, chosen to be representative of the range of axial load values present in the walls of existing structures of this type (quantified below), is then applied to the two lateral faces of the triplet. To do so, the specimen is post-tensioned by means of four steel rods before the vertical force is applied to the central concrete block through an actuator in force control (Figure 10a). As shown in Figure 10b, each triplet specimen was instrumented using six displacement transducers such that the time instant corresponding to the activation of sliding could be identified, thus leading to the estimate of the shear force resisted by the mortar.

Within this experimental test framework, the horizontal/axial load is uniformly distributed on the faces of the specimen and is kept constant during each single test run. Three axial load levels, corresponding to compressive stress (σ) of 0.2 MPa, 0.6 MPa and 1.0 MPa, were selected for each triplet specimen, with the sequence of their application being also varied from one test to the other. For a given compressive stress, the shear stress (τ) can be calculated as:

$$\tau = \frac{F_S}{2A_l}, \tag{4}$$

where F_S is the vertical shear force measured during the tests and A_l is the lateral surface of the specimens parallel to the vertical shear force, the latter being equal to 195 x 95 mm^2 in this specific case.

3.3.2 Test results

Figure 11a presents the vertical shear force-pseudotime response of triplet specimens #01, #02 and #03, the testing of which was undertaken imposing increasing levels of horizontal/axial force (F_N) that correspond to 0.2 MPa, 0.6 MPa and 1.0 MPa. Response plots such as the one shown in Figure 11a allow the drops in strength corresponding to the activation of sliding (see Figure 10b) to be identified, leading to the couples (σ, τ) and (F_N, F_S) summarised in Table 3. For the sake of completeness, the vertical force-pseudotime-history response of the other specimens, namely triplet test #04 to triplet test #09, are shown in Figure 11b and Figure 11c.

Table 3: Summary of monotonic-friction triplet test results

Triplet label	Axial load [kN]	Axial stress [MPa]	Shear force [kN]	Shear stress [MPa]
#01	3.7	0.201	14.7	0.398
#02	3.5	0.190	20.3	0.547
#03	3.9	0.209	20.8	0.561
#04	11.8	0.639	32.8	0.887
#05	12.0	0.649	30.8	0.832
#06	11.8	0.637	25.2	0.681
#07	18.6	1.001	55.0	1.483
#08	18.3	0.986	52.5	1.416
#09	19.2	1.038	54.3	1.464

Plotting of the couples (σ, τ) rendered apparent that the relatively simple Coulomb's law can be used for a readily interpretation of the obtained results. In fact, the shear strength τ of the mortar bed-joints depends on three parameters that are the cohesion τ_0, as well as the static friction coefficient μ and the transversal compression σ. The cohesion (or adhesion stress) contributes to the resistance only if the mortar bed-joints are not cracked, whilst the friction force acts also after the formations of fractures, for as long as the concrete blocks and the mortar are in contact. According to Coulomb's law, τ is linearly dependent on the imposed axial compression, leading to the following regression curve:

$$\tau = \tau_0 + \mu\sigma = 0.202 + 1.161\sigma \quad - \quad R^2 = 0.905. \tag{5}$$

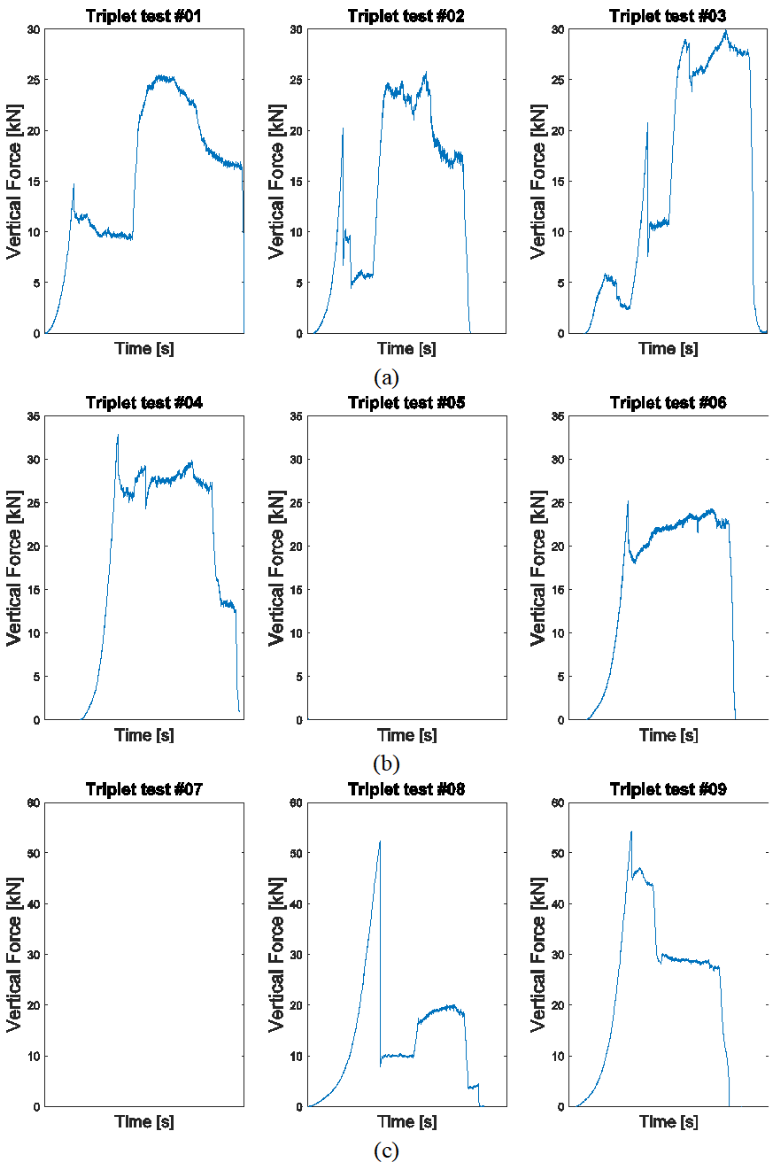


Figure 11: Vertical force-pseudotime response of triplet specimens for different axial load values and loading sequence: (a) σ = 0.2, 0.6, 1.0 MPa, (b) σ = 0.6, 1.0, 0.2 MPa, and (c) σ = 1.0, 0.6, 0.2 MPa

This regression curve is presented in Figure 12, together with all (σ, τ) couples. It is worthwhile to mention that these couples refer to the maximum value of shear strength of mortar, which is given by both cohesion and friction force, as the specimen is uncracked before the test is performed.

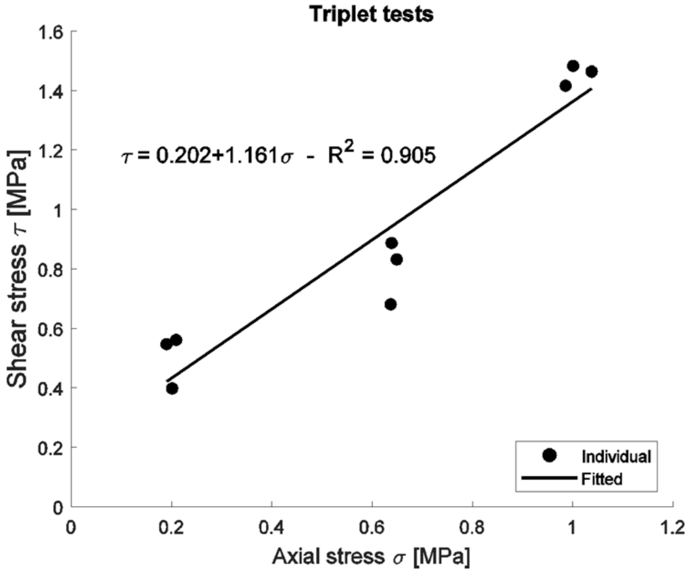


Figure 12: Axial stress-shear stress couples from triplet monotonic tests and closed-form analytical expression from regression analysis

It can be concluded that the proposed analytical expression shows a good fit with the obtained set of experimental data, given that the coefficient of determination of the regression R^2 is equal to 0.905. The cohesion τ_0, namely the Y-intercept of the linear regression curve, is 0.202 and the coefficient of friction μ, indicating the slope of the linear regression line, is 1.161.

4 Discussion and observations

Not surprisingly, the closed-form analytical expression given by Eq. (5) can be used to predict the shear capacity of base wall wet joints made up of high-strength mortar. Concerning this, and also with regards to implications for the numerical modelling of existing precast wall-slab-wall buildings featuring this type of wall-to-foundation connection, the reader is referred to Figure 13, where the experimentally-driven analytical capacity estimates are superposed on the shake-table test results of a full-scale two-storey building subjected to test runs of progressively increased seismic intensity [8]. Dynamic test data are openly available and archived at the Experiments platform of the Built Environment Data initiative (under the following Dataset Digital Object Identifier: https://doi.org/10.60756/euc-uaty26g318).

More specifically, Figure 13a compares the analytical prediction with the resistance actually mobilised in the building during test run 100%, following which intensity shaking the high-strength mortar joint at the first-storey stability wall turned out to be cracked for more than half the wall's depth (i.e. 60% approximately), as can be seen from Figure 14. It is needless to note that the analytical prediction was obtained by multiplying the shear stress estimated through Eq. (5) by the area of the cracked portion of the base wall joint, with the latter being computed as the product of the wall thickness times 60% of the wall depth.

Eq. (5) is thus proven accurate, as it underestimates the base shear resulting from shake-table testing by approximately 5%, and the same consideration can also be made in reflection of the comparison presented in Figure 13b, which collects analytical estimates and experimental results corresponding to the subsequent test run, namely that for the 150% seismic intensity. Indeed, the same Eq. (5) was employed to undertake the calculations, assuming in this case that the wet joint at the base of the shear wall is fully cracked along its depth. This assumption turns out to be conservative, as a similar discrepancy of about 5% can be calculated from analytical and test data (see Figure 13b).

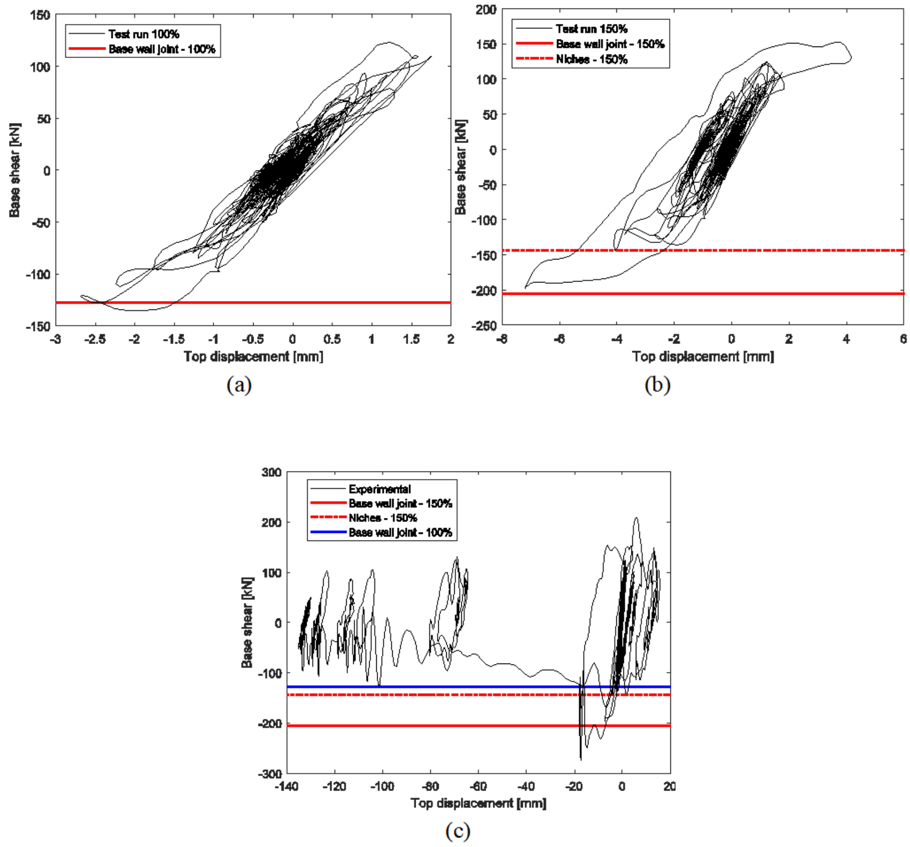


Figure 13: Comparison between analytical capacity estimates and results from incremental shake-table testing: (a) test run 100%, (b) test run 150%, and (c) all runs and damage mechanisms

Figure 14: Development of horizontal cracks in the high-strength mortar joint at the base of the first-storey stability/shear wall after test run 100% [8]

To corroborate further the above assumption, Figure 15 highlights the main damage patterns at the end of test run 150%, with the full-depth horizontal cracks in correspondence to the shear wall being the most notable occurrence (Figure 15a). Furthermore, Figure 15b shows an example of the cracks developed all along the boundaries of the connectors' sockets, four in total. The resistance associated with this latter mechanism can be predicted as well, by computing the product of the mean bond strength of mortar, as obtained from bond wrench testing (Section 3.2), and the lateral surface of the four panel niches. Figure 13c compares the three capacity estimates and reaffirms their accuracy, with respect to the sequence and extent of damage mechanisms exhibited by the full-scale building specimen, thus adding further weight to the recommendations presented here and the testing campaign described in this paper.

(a) (b)

Figure 15: Damage mechanisms/patterns at the end of test run 150% (Brunesi et al. [8]): (a) full-depth cracks in the base wall joint and (b) example of cracks along the boundaries of the connectors' sockets

5 Conclusions

Wet mortar joints, with no mechanical connection devices, can be found very frequently in existing precast wall-slab-wall buildings in some Northern European countries, not necessarily prone to hazard from natural/tectonic seismicity but some of which have recently started being exposed to induced seismicity phenomena, thus motivating the characterisation testing campaign into which this paper has delved. In addition to standard compression and three-point flexural testing, bond wrench and triplet monotonic-friction tests were carried out, as also called for by observations stemmed from incremental dynamic shake-table testing that involved a full-scale building specimen representative of this class of structures [8].

The material characterisation test results described in this work allowed the development of closed-form expressions that can be used to analytically estimate the maximum horizontal lateral force that may be carried through a building's high-strength mortar joints. Not surprisingly, the test results also evidenced how these wet joints are not sufficiently resistant to prevent damage, in spite of the fact that very high compressive and flexural-tensile strength values were obtained for the mortar employed for the base wall joints or to execute the connections between the wall panels. Put simply, cohesion and frictional resistance of this high-strength mortar cannot be high enough to meet the unfeasible force demand that concentrates in joints lacking mechanical connection, under moderate-high levels of shaking, and the same applies to the bond strength that can be mobilised at the mortar-concrete interface before cracking.

In conclusion, on the one hand, the set of material properties and analytical expressions derived in this paper were found to lead to resistance estimates that are well aligned with the findings from shake-table testing on a building meant to resemble many characteristics of this structural typology and, however, on the other hand, the same properties and estimates laid bare the need for connection detailing more akin to common earthquake engineering practice to be used in these structures, when they are expected to be exposed to seismic action of non-negligible intensity.

Lastly, it is noted that all experimental data processed and presented in this paper are directly tabulated herein and/or openly archived at the Experiments platform of the Built Environment Data initiative, to which interested readers are referred.

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Data availability statement

All experimental data processed and presented in this paper are directly tabulated herein and/or openly archived at the Experiments platform of the Built Environment Data initiative, to which interested readers are referred.

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