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Experimental tests on the shear behaviour of dowels connecting concrete slabs to stone masonry walls

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An expermental investigation has been conducted on the behaviour of dowels used to connect concrete slabs to stone masonry walls in order to transfer horizontal shear forces. A technique for embedding the dowels in the stone block without injection of grouting material or resin has been developed. Special experimental equipment has been designed in order to allow the execution of in situ *load tests on representative ancient buildings. Monotonic loading tests have been carried out on eight specimens with the purpose of measuring both stone block displacement and dowel deformation*

NOTATION

u_1, u_2	Perpendicular displacements of two points of the block at distances x_1 and x_2 from the dowel, respectively
v_3	Displacement of the block in the direction of the load measured at a distance y_3 from the surface of the wall
δ	Displacement of the block in the direction of the load
η	Displacement due to dowel deformation
α	Rotation of the stone block
s	Slip between the steel flange and the masonry wall
F	Transversal load
F_y	Transversal load at the beginning of yielding of the dowel
f_{ys}	Yield strength
ϕ	Diameter of the dowel
l_h	Distance between two plastic hinges in the shank of the dowel
l_m	Distance between the steel flange and the stone surface

1. INTRODUCTION

The structural strengthening of ancient buildings located in seismic zones requires particular attention to the design of floors required to transmit and redistribute seismic actions to the shear walls. Each floor should behave like a diaphragm and needs to be well connected to adjoining masonry walls in order to provide a global box-behaviour to the building when subjected to seismic actions. In fact, while floor strengthening and stiffening are important they are not sufficient by themselves [1,2]. In addition, the connection between the floor and the wall has to be effective and carefully designed. This connection has mainly to resist the shear horizontal action, avoiding slip between the floor and the shear wall as well as the pull-out action when the wall is loaded normal to its plane. In many cases shear action prevails over normal force action. For this reason particular care is here devoted to connections subjected to shear action.

In practice the connection is usually constituted by concrete dovetail elements embedded in the wall and connected to boundary tie-beams of the floor. These elements pass completely through the wall and in many cases are particularly unsuitable for historical buildings having precious façades because of the evident damage they provoke [3]. Moreover these anchorages are often unsuitable because the formation of corresponding holes can damage and weaken the surrounding part of the wall. For this reason dovetail anchorages should be distributed at a distance of at least 2.5 m [2,4], but even then local stress concentrations often occur in the wall causing undesirable cracks. It should be emphasized that this dovetail connecting technique is not supported by many experimental and theoretical investigations and that only a few of its aspects have been discussed in the literature [2,4]. Nowadays, architectural restoration theory tends to adopt lighter connection elements which can be easily substituted at any time without leaving significant damaging marks. Considering all these aspects, a connection made with distributed steel dowels may represent an interesting technical solution. In fact only small drilled holes have to be made in the masonry wall in order to insert the steel rods connecting the tie-beams to the wall. This technique appears particularly suitable for the connection of thin concrete slabs used for strengthening and stiffening wooden floors, as proposed by several authors [5–7]. The boundary tie-beams have such a small depth that the technique of dovetail anchorage is unsuitable because of the concentration of stresses in the slab. In contrast, distributed dowels avoid this problem of stress concentration and can be easily placed in the slab thickness.

Some experimental tests concerning dowel connections under repeated loads have been reported in other research work [8]. Here, the dowels pass through the entire wall so that this technique becomes unsuitable for buildings with precious façades. The experimental results reported are especially focused on the strength evaluation of the connection.

An experimental investigation on the behaviour of dowels has been performed in the present research work. Dowels not passing completely through the wall have been taken into consideration. Particular attention has been devoted to the problem of the anchorage technique and to the study of the interaction between dowel and stone block, as well as between stone block and masonry wall.

2. DOWEL CONNECTION FEATURES

Steel dowel connections between structural elements are commonly adopted in practice, especially to connect R.C. elements or to join steel elements to R.C. structures. This technique usually employs epoxy resin or grouted cement injected in a hole larger than the dowel in order to bond the dowel to the masonry [9]. Such injection can produce some problems in ancient masonry because of the remarkable penetration of the epoxy resin or grouting cement into the masonry voids and, as such, particular devices are required to limit the amount of injected material. Special dowels injected with epoxy resin are at present available and relatively simple to use. However, for historical buildings there are some concerns about the interaction between materials of such different characteristics and, moreover, very little information about the durability of this connection is available.

The technique presented herein is based on the insertion of a smooth steel rod into a calibrated hole in a stone block, without using any gluing material. The precision of the size of the hole is needed to ensure the effectiveness of the connection against pull-out of the rod and to avoid splitting of the stone during rod insertion. Difficulties in obtaining a calibrated hole in the stone block have resulted in using the procedure illustrated in Fig. 1 to embed the rod. The steel rod (Fig. 1a) is forced into a drilled hole with 20–30 strokes of a hammer (mass of 0.5 kg). The hole is obtained by means of an ordinary rock drill using particular drill-bits in order to form a regular calibrated hole. Because the hole diameter is 2 mm more than that of the rod, three steel strips (yielding strength $f_y = 250\ \mathrm{N\ mm^{-2}}$) are also introduced into the hole (Fig. 1b). When the rod is inserted the steel strips bend transversally and assume the shape shown in Fig. 1c. Views of the positioning of the steel strips in the hole and of the steel rod after insertion are shown in Fig. 2a and b.

The strips ensure an effective and controllable pressure

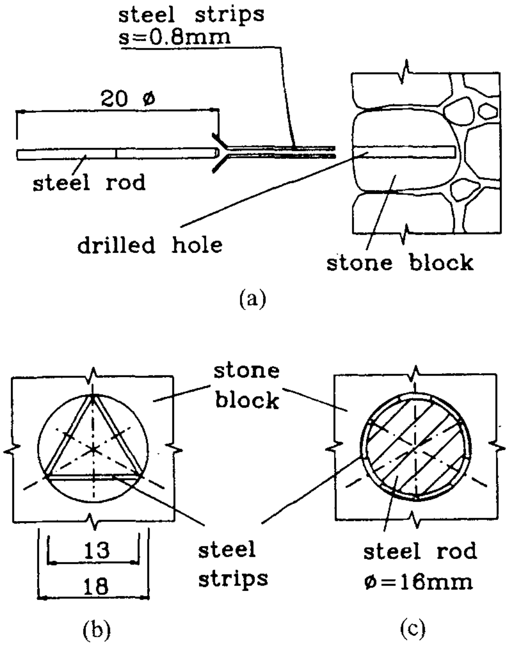


Fig. 1 (a) Procedure for the embedment of steel rod, (b) positioning of steel strips into hole, (c) configuration after rod insertion.

f along the entire rod–hole interface. Using three strips of 13 mm width, the triangular arrangement illustrated in Fig. 1b may be obtained. This steel strip width allows maintenance of the correct position of the strips during insertion of the rod. The thickness and the effective length of the strips adopted were 0.8 and 160 mm, respectively. Some pull-out tests have been carried out in order to verify the effectiveness of this anchorage technique (Fig. 3). The pull-out load obtained was found to vary between 6.5 and 8 kN and a quasi-perfect plastic behaviour of at least 1 cm slip was observed. The technique is quite simple to implement and requires only common equipment. Moreover it seems to be in accordance with architectural restoration theory because removal of the connections is possible and it produces very limited damage in the masonry.

3. EXPERIMENTAL APPARATUS

An experimental investigation of the behaviour of a dowel connection between a concrete slab and a masonry wall subjected to horizontal shear load was carried out on the walls of an ancient building. Particular attention was paid to flexural behaviour of dowels as well as to the local behaviour of the wall. For the sake of simplicity the concrete was replaced by appropriate steel devices that simulate the actual deformation of the dowel in the concrete slab (Figs 4 and 5). Steel devices made it possible to neglect the interlocking effect between the concrete slab and the masonry wall. In reality the contribution of this effect to the shear resistance of the connection may be considerable. However the progressive damage at the slab–wall interface caused by cyclic loads (e.g. seismic

(a)

(b)

Fig. 2 (a) View of steel strips inserted into hole, (b) view of steel rod embedded in masonry wall.

loads) may reduce this contribution to zero after a few cycles [8], so that only the dowel resistance remains effective. For this reason only the dowel effect has been considered in this investigation.

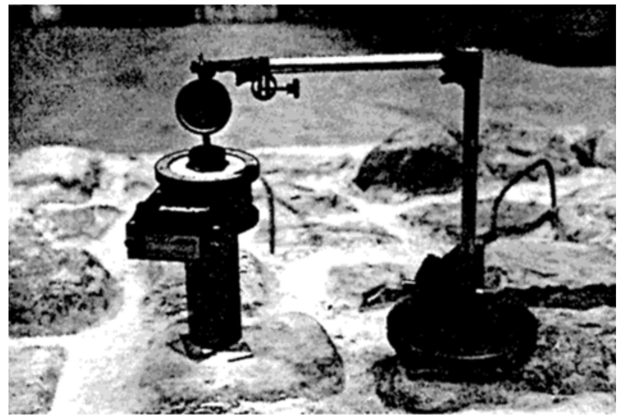
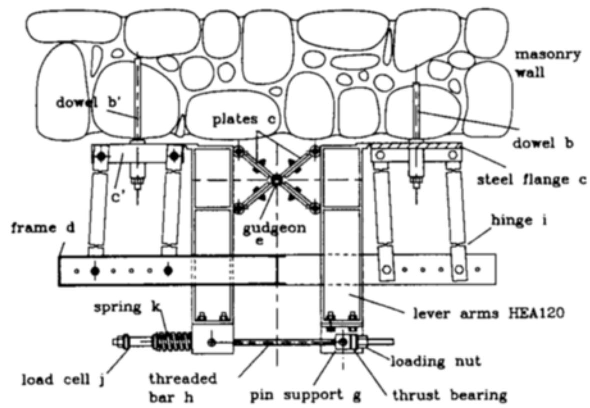
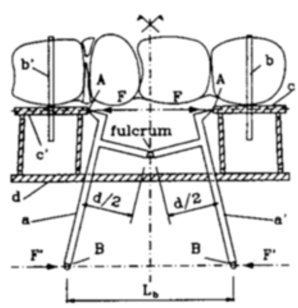
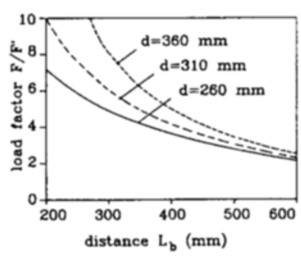


Fig. 3 Arrangement of pull-out test.



(a)





(b) (c)

Fig. 4 (a) Plan of load machine, (b) scheme of lever arm system, (c) curves of the factor F/F' versus distance L_b.

A load machine was designed to test two dowels (b and b′ in Fig. 4a) simultaneously. Since the bearing capacity of the two dowels is unlikely to be equal, the weaker dowel reaches its ultimate slip first. In order to complete the test of the second dowel the steel flange of the first dowel is locked with an anchor bolt.

The load machine was based on the lever system illustrated in Fig. 4b. As shown in Fig. 4a, H-beams (a and a′) form two lever arms hinged at the gudgeon (e). Fig. 4b shows the shear actions F applied to the dowels and the forces F' transmitted by a threaded bar. The relationship between the measured load F', the applied

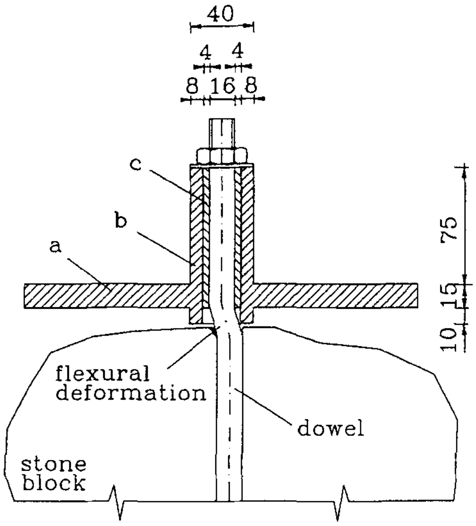


Fig. 5 Detail of steel flange.

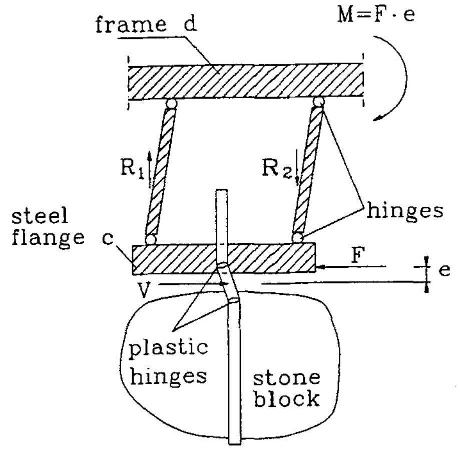


Fig. 7 Scheme showing role of rigid frame.

load F and the distance L_b is non-linear because the remarkable rotation of the arms considerably changes the lever-arm ratio. The ratio F/F' is plotted against the distance L_b in Fig. 4c for three different values of the distance $d/2$ between the fulcrum and the lever-arm axis. The lever-arm apparatus allows the load to be amplified 3–6 times. Values of L_b lower than 200 mm are not recommended (Fig. 4c).

The fulcrum hinge is connected to the lever arms by means of four plates (f) which allow for regulation of the distance between the arms (i.e. variation of the distance d). In this way tests with different distances between the dowels can be carried out.

The value of the load F' was determined indirectly by means of a load cell (j) applied on a threaded bar (h) (Fig. 4b). The load cell is supported by an articulated system in order to avoid bending of the threaded bar. The load machine was designed to produce loads up to 50 kN by screwing the loading nut supported on a thrust bearing. At the other end of the threaded bar a soft steel spring (k) was placed between the load cell (j) and the pin support (g) in order to avoid appreciable load

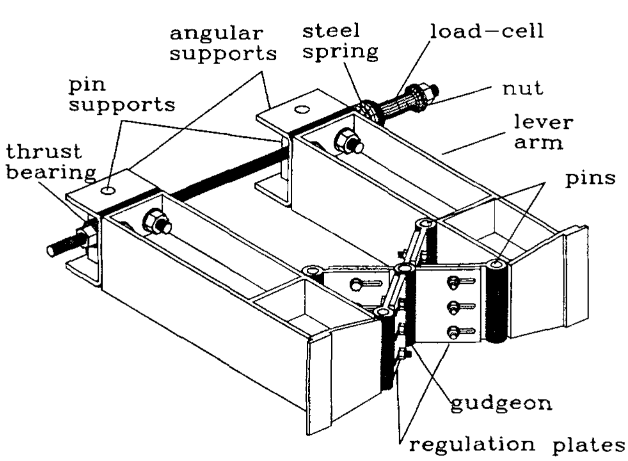


Fig. 6 Isometric drawing of load machine.

decreases at each load step due to time-dependent effects.

Steel flanges (c and c′ in Fig. 4a) were connected to a very stiff frame (d) in order to avoid their rotation. An isometric drawing of the load machine alone is shown in Fig. 6.

The steel flanges were made with a plate (a) welded to a sleeve (b) having an internal diameter of 24 mm (Fig. 5). A second sleeve (c), with an external diameter equal to 24 mm and an internal diameter equal to that of the steel rod, was coupled with the first sleeve (b). A shorter length was adopted for the second sleeve in order to allow flexural deformation of the dowel, thus simulating the actual behaviour of the connector when it is embedded in the concrete slab.

As mentioned previously, the moment due to the eccentricity of applied load F with respect to reaction V (Fig. 7) is equilibrated by a rigid frame (d) that allows only for translation displacement of the flanges. In fact, two pairs of parallel steel bars with hinges at their extremities connect the steel flanges to the frame (Figs 4a and 7). Hinges (i in Fig. 7)) were obtained by reducing the section of the bar (Fig. 4a), instead of using gudgeons that may have lack-of-fit problems in simulating hinge actions.

Both the above-mentioned frame (d in Fig. 4) and the whole load machine were supported by a steel bench (Fig. 8). A system of two carriages was adopted in order to allow relative movements between the machine parts during loading.

The horizontal displacement s of the steel flange (Fig. 9), and the rotation α and translation δ of the stone block were obtained during testing by means of four dial gauges (sensitivity 0.01 mm) arranged as in Fig. 10. Measured displacements u_1, u_2, v_3 are related to α, to δ and the length x_c of the axis around which the block rotates. In fact, the relationships

$$\left.\begin{aligned} v_3 &= \delta - \alpha y_3 \\ u_1 &= \alpha(x_1 - x_c) \\ u_2 &= \alpha(x_2 - x_c) \end{aligned}\right\} \tag{1}$$

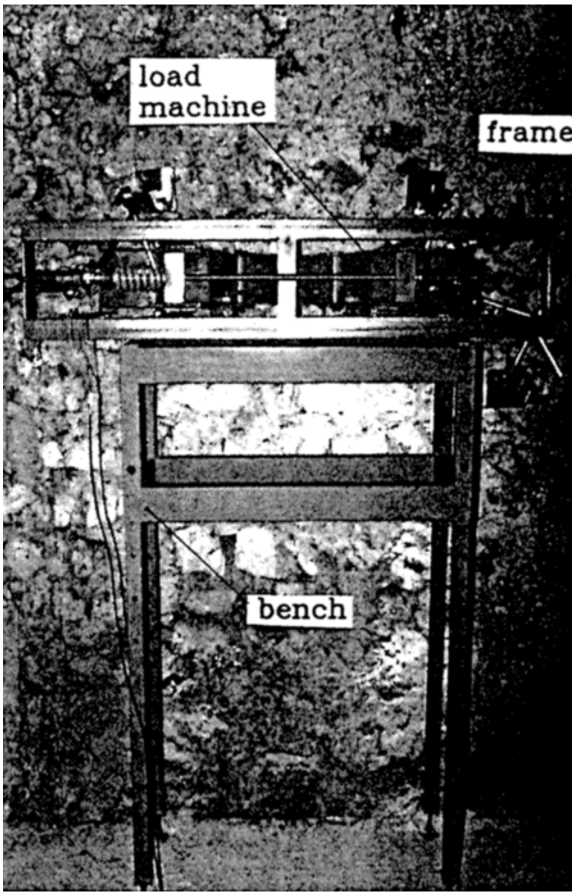


Fig. 8 View of experimental apparatus supported by steel bench.

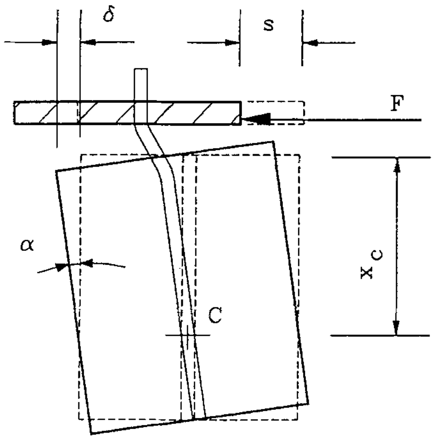


Fig. 9 Displacements of steel flange and of stone block.

give the following expressions:

$$\alpha = \frac{u_1 - u_2}{x_1 - x_2} \tag{2}$$

$$\delta = v_3 + \frac{u_1 - u_2}{x_1 - x_2} y_3 \tag{3}$$

$$x_c = \frac{x_2 u_1 - x_1 u_2}{u_1 - u_2} \tag{4}$$

where x_1, x_2 are the distances of the vertical dial gauges

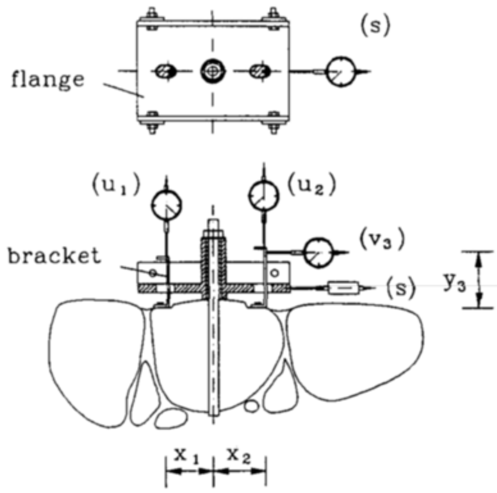


Fig. 10 Arrangement of dial gauges for measuring stone block and steel flange displacements.

from the steel rod axis and y_3 is the distance of the horizontal dial gauge from the wall surface (Fig. 10).

4. TEST PROCEDURE

The experiments were carried out on the walls of a 17th century building. The walls, of about 50 cm thickness, are made with almost-round stone blocks of different kinds of limestone (e.g. sandstone, calcirudite, greywacke, etc.). The majority of the stone blocks have a dimension ranging between 15 and 20 cm. The stone blocks are assembled very carefully in order to avoid joints that are too thick (i.e. 1–2 cm). The mortar used for the joints between stone blocks is made with calcium-silicate sand and hydrated lime as binder.

Stone blocks were chosen in zones where the vertical stress was estimated to be about 0.1 MPa. In the present research work four pairs of stone blocks were tested using smooth steel rods as dowels with a diameter of 16 mm.

First of all the stone was drilled approximately in the centre of its face using a particular drill bit in order to obtain a sufficiently regular hole. The hole diameter was 18 mm in order to insert the steel strips around the dowel, as specified in section 2. Then dowels were forced into the hole using a hammer. About 20–30 hammer blows were needed to insert the dowel a distance of 15 cm.

The test was carried out by increasing, step by step, the slip between the steel flange and the surface of the wall. The load–slip curve for the weakest dowel of each pair of stone blocks was assumed to govern the load procedure. Increments of slip of 0.1 mm were considered up to a total slip of about 2 mm, while increments of 0.2 mm were assumed for higher slip values. Displacements s, u_1, u_2 and v_3 (see section 3) were measured at the end of each loading step. Stabilization of the load was awaited after each slip increment. In particular, a further slip increase was applied when the rate of variation of the load was almost $\frac{1}{10}$ of the initial value. At the beginning of the test the time of load stabilization

Table 1 Splitting and conventional ultimate loads of specimens tested

Test No.	Stone dimensions			Splitting load		Ultimate load	
	L (mm)	D (mm)	H (mm)	F_s (kN)	s (mm)	F_u (kN)	s (mm)
1	18	16	13	19.02	4.37	26.06	10.00
2	13	14	12	11.75	2.10	13.52	3.64
3	22	20	16	20.41	3.88	19.44	5.64
4	21	24	13	–	–	26.06	10.00
5	13	13	13	11.06	1.63	16.58	10.00
6	19	28	10	–	–	32.18	10.00
7	17	20	12	23.58	4.00	24.92	10.00
8	23	16	13	27.59	6.62	26.37	10.00

was about 1 min, while close to the ultimate load it was almost 20 min.

When the weakest dowel reached a slip of about 10 mm, the steel flange was fixed to the wall with anchor bolts in order to complete the test on the other dowel, as already mentioned.

The load F applied to the dowels was obtained from the measured force F' of the machine by means of the curve in Fig. 4c (see also section 3), measuring the distance L_b at each loading step.

5. RESULTS

The results concern eight specimens of dowels (diameter = 16 mm) embedded in stone blocks with the dimensions (defined in Fig. 11) reported in Table 1.

All specimens, except for Nos 4 and 6, were affected by transverse splitting of the stone block. The load causing the splitting was quite different for various specimens. In fact it depends on the type of rock (i.e. with or without layers of softer material in the stone block) and on the dimensions of the block. The experimental results show a certain dependence between splitting load F_s and transverse dimension D of the stone block (see Table 1).

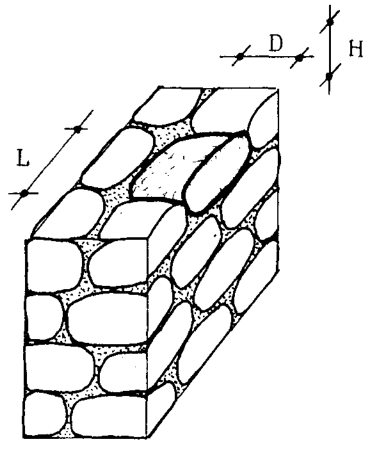


Fig. 11 Stone block parameters.

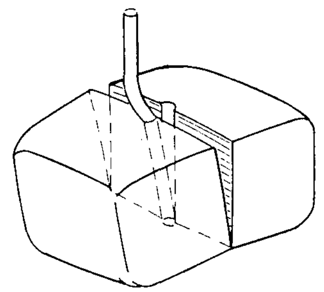
Different kinds of failure were noted for different values of the transverse dimension D of the stone block. In fact specimens Nos 1, 3, 7 and 8, which have a high value of D, show a splitting crack causing a separation of a small part of the stone (Fig. 12b) while in specimens Nos 2 and 5, which are characterized by a small rounded shape and a low value of D, the crack splits the stone in two parts (Fig. 12a).

In almost all specimens tested the splitting load was reached for a slip value ranging between 2 and 4 mm. The experimental tests were carried out for up to 10 mm of slip in order to also investigate the behaviour after splitting of the stone. The tests of specimens Nos 2 and 3 were stopped at a lower value of slip. The loads corresponding to these slips are indicated in Table 1 as the conventional ultimate loads.

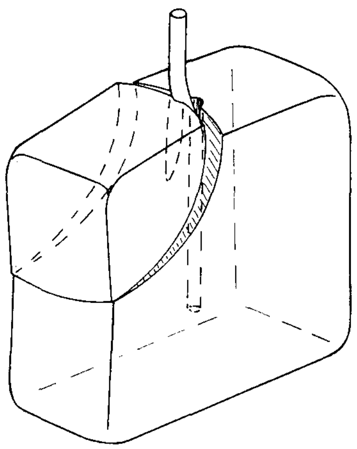
The experimental results concerning the variation of load F versus slip s are plotted in Fig. 13. The onset of splitting is indicated by squares. The propagation of the splitting crack has extended in a sudden way. It can be noted that the curves do not show a decrease in resistance after splitting. On the contrary, a small increase may be observed and substantial ductility is evident after splitting.

Besides the splitting, an important phenomenon of the connection behaviour concerns the beginning of dowel yielding. The curves in Fig. 13 do not clearly show this phenomenon. Actually the slip s is related not only to the displacement η of the dowel but also to the displacement δ of the block. Displacement δ is determined from Equations 2 and 3 while η may be determined as the difference between s and δ. The relationship between load F and displacement η is plotted in Fig. 14. In the figure it is possible to note an appreciable change in the slope of the curves, which probably indicates yielding of the dowel, being the crushing of the rock reached for higher values of the displacement η. The corresponding values of load F_y, displacement η_y and slip s_y are assumed as yielding values and are reported in Table 2 (triangles in Fig. 14).

Two different steel grades were used for the dowels. Specimens Nos 7 and 8 had a yield strength of 365 MPa,



(a)



(b)

Fig. 12 (a) Total splitting and (b) partial splitting of stone block.

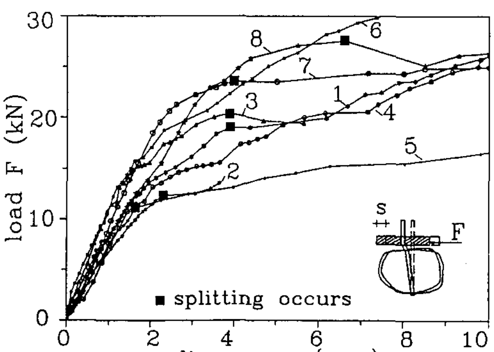


Fig. 13 Load–slip curves for specimens tested.

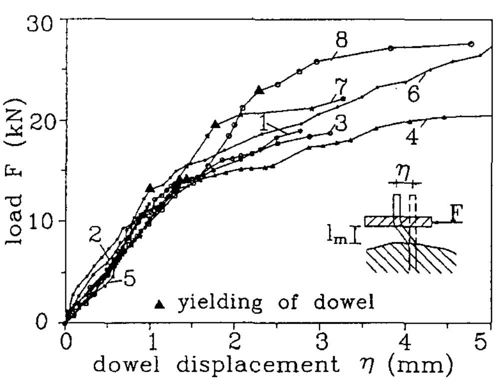


Fig. 14 Load–displacement curves for dowels.

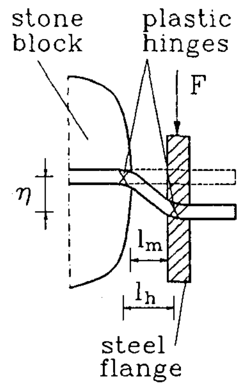


Fig. 15 Geometry of plastic hinges.

while the dowels for the remaining specimens had a yield strength of 265 MPa.

The first branch of each curve in Fig. 14 is approximately linear and very close to the other curves.

In Table 2 the values of the measured free distance l_m between the steel flange (c) and the stone surface (Fig. 15) are also reported. The plastic hinges formed at a distance l_h greater than l_m because of local crushing of the stone [10]. The values of l_h reported in Table 2 were obtained in accordance with the schematic dowel behaviour indicated, assuming ideal rigid plastic behaviour for the steel. In this case the relationship between the yield load F_y and the distance l_h can be obtained theoretically by assuming that the moment due to the load F_y is equal to the plastic moment of the steel rod. In fact, since $M_p = (\pi\phi^3/6)f_{ys}$, the equilibrium equation can be written

$$F_y \frac{l_h}{2} = \frac{\pi\phi^3}{6} f_{ys} \tag{5}$$

and then the distance l_h is

$$l_h = \frac{f_{ys}\phi^3}{3F_y} \tag{6}$$

where f_{ys} is the yield strength, ϕ the dowel diameter and F_y the experimental yield load value indicated in Table 2.

In specimens 2 and 5, complete splitting of the stone occurred before yielding. For specimen 6 the distance l_m was greater than for the other specimens because of the irregular external surface of the stone.

The relationship between the block displacement δ and the applied load F, as well as the relationship between slip s and block displacement δ, are plotted in Figs 16 and 17, respectively. Figure 16 shows a regular shape for the curves and no sudden increase of the stone displacement occurs. Squares indicate the splitting load, where each curve is interrupted because further splitting translation δ makes no sense.

Figure 17 gives important information about the influence of block displacement δ on slip s. The relationship between s and δ is almost linear up to the splitting of the stone block, with the displacement δ being in the range of about 20–30% of the slip s; specimens 2

Table 2 Parameters related to the yielding of dowels

Test No.	Steel of dowel		Yielding of dowel			Hinges distances	
	Yield strength, f_{ys} (MPa)	Ultimate strength, f_{us} (MPa)	F_y (kN)	η (mm)	s (mm)	l_m (mm)	l_h (mm)
1	265	428	14.05	1.41	2.10	12	25.75
2			–	–	–	12	–
3			14.84	1.28	1.56	12	24.38
4			13.14	1.30	2.04	16	27.53
5			–	–	–	12	–
6			15.73	1.30	1.60	12	23.00
7	365	500	19.53	1.75	2.20	13	25.52
8			21.24	2.08	3.03	12	23.46

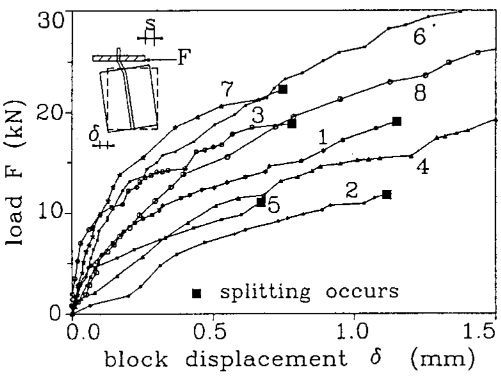


Fig. 16 Load–displacement curves for stone block.

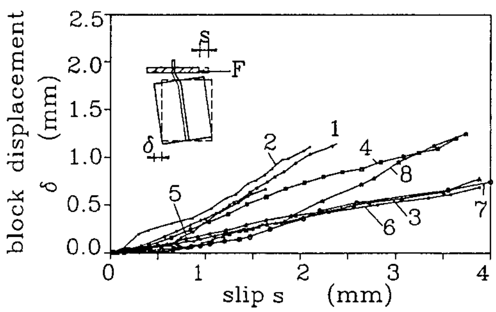


Fig. 17 Relation between displacement of block and slip.

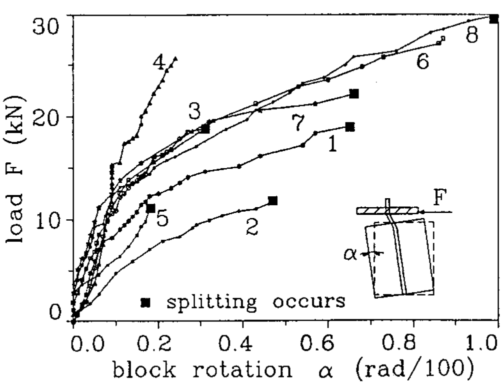


Fig. 18 Load–rotation curves for stone block.

and 5 show a higher percentage because they involve small rounded stones.

To complete the information about the movement of the stone block, the evaluaton of its rotation is needed and can be determined using Equation 2. The relationship between applied load F and block rotation α is plotted in Fig. 18. For specimens Nos 2 and 5 rotations appear earlier than in the other specimens, and they increase faster.

6. CONCLUDING REMARKS

An experimental investigation of the behaviour of dowels connecting concrete slabs to stone masonry walls has been carried out. Experimental tests have been made on eight stone blocks of a wall of an ancient (17th century) building.

Steel rod dowels of 16 mm diameter have been embedded into the stone blocks by forcing each into a calibrated hole having three steel strips between the rod and the internal surface of the hole (section 2).

On the basis of the results obtained the following remarks may be made.

1. Most of the specimens tested reached failure by transversal splitting of the stone block for a slip ranging between 2 and 4 mm (Fig. 13).

2. After the splitting of the stone the load does not decrease but it remains almost constant or it slightly increases. The considerable slip values reached in the experiments (up to 10 mm) demonstrate the considerable ductility of this type of connection.

3. No macroscopic damage to the masonry close to the stone block tested was noted up to the maximum load applied.

4. Yielding of the dowel is not directly evidenced by the experimental curves (Fig. 13) because the slip s of the steel flange depends on the block displacement δ. Curves of dowel displacement $\eta = s - \delta$ against load F show a notable change of slope, which clearly indicates yielding of the dowel (Fig. 14).

5. The relationship between applied load F and slip s of the connection (Fig. 13) is almost linear up to yielding of the dowel. The variation of the slope of the initial branch of the curves for different dowels is quite limited and ranges between 6.5 and 10 kN mm (Fig. 13).

6. The displacement of the block ranges between 20 and 30% of the slip of the steel flange (Fig. 17).

These early results give detailed information on the behaviour of connections between a concrete slab and a masonry wall, even though they are not exhaustive due to the limited number of tests. Other kinds of stone should be tested in further research work; in addition, models and rules for designing adapted dowels for a given use need to be developed in the future.

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RESUME

Essais sur le comportement vis-à-vis de l'effort tranchant de goujons de connexion entre dalles en béton armé et murs en pierre

Dans ce travail nous avons mené une enquête expérimentale sur le comportement vis-à-vis de l'effort tranchant des connexions à goujon, utilisées dans le but de transmettre les actions horizontales (par ex. forces sismiques) entre une dalle en béton armé et une maçonnerie en pierre. La technique adoptée consiste dans l'ancrage des goujons dans la maçonnerie, réalisé au moyen d'un trou calibré dans la pierre, puis d'une insertion en force du goujon. Cette technique présente l'avantage de ne pas exiger des interventions onéreuses au niveau de la maçonnerie, susceptibles d'en réduire la résistance.

Nous avons préparé un appareillage spécifique permettant l'exécution d'expériences in situ. *Pour simplifier l'exécution, l'effet de la dalle en béton a été simulé à l'aide d'une plaque métallique conçue dans ce but. Nous avons donc effectué huit expériences sur la maçonnerie en pierre d'un vieux bâtiment. Durant chaque expérience, la charge subissait de faibles accroissements progressifs et, à chaque accroissement, on enregistrait aussi bien le déplacement relatif entre la plaque métallique et la maçonnerie que les déplacements (translation et rotation) du bloc auquel le goujon était fixé. La plupart des échantillons essayés ont atteint la rupture par cassure transversale du bloc due à des glissements entre la plaque métallique et la maçonnerie, variant entre 2 et 4 mm.*

Après la rupture par cassure du bloc, on n'a pas relevé de brusques réductions de la capacité portante qui, au contraire, est restée constante ou a enregistré une légère augmentation avec l'accroissement du glissement. Nous avons par ailleurs déterminé la charge correspondant à la limite d'écoulement du goujon. En outre, dans les essais effectués, la translation du bloc constitue 20 à 30% du glissement global de la plaque par rapport à la maçonnerie.