IN-SITU TEST OF THE SHEAR STRENGTH AND DEFORMABILITY OF AN 18TH CENTURY STONE-AND-BRICK MASONRY WALL

P.F. SHEPPARD, Research Engineer

Institute for Testing and Research in Materials and Structures (ZRMK), Ljubljana, Yugoslavia

ABSTRACT In the paper a description is given of how the shear strength and deformability of a load-bearing, stone-and-brick masonry wall in an 18th Century building in Ljubljana were determined by means of an in-situ test. The tested wall element, of dimensions 1.60 x 0.80 x x 0.45 m, was loaded horizontally at mid-height until symmetrical diagonal cracking occurred. It was then repaired by full cement-grouting, and retested to failure. The results obtained are compared with those obtained in tests of similar wall elements in the laboratory.

1. INTRODUCTION

Following the 1976 Friuli Earthquake and the 1979 Montenegro Earthquake, extensive research was carried out at ZRMK Ljubljana, by means of full-scale and model tests in the laboratory, into the load-carrying capacity and deformability of stone-masonry walls and buildings, subjected to combined, vertical and horizontal loading. The object of this research work, whose results have been presented in references [1] and [2], was to determine the seismic resistance of typical stone-masonry walls and buildings, and to provide a basis for the assessment of the efficacy of methods proposed for the repair and strengthening of such structures. These tests were supplemented, in 1983, by the in-situ test of a stone-and-brick masonry wall described in this paper.

Apart from the mitigation of earthquake damage, there has, in recent years, been increasing interest, both in Yugoslavia and elsewhere, in the revitalization of older urban and rural centres of cultural and historical importance. For instance, in the capital of the republic of Slovenia, Ljubljana, systematic and organized revitalization of the old part of the city centre has been going on since 1976. Other towns in Slovenia where the process of revitalization has begun on a larger scale include Maribor, Ptuj, Novo mesto, Piran, Škofja Loka and Kamnik. All these towns are situated in seismic regions where earthquakes of intensity VII to IX on the MCS scale can be expected.

When carrying out revitalization projects, which involve the investment of large funds, it is essential to make a realistic estimate of the seismic resistance of the buildings concerned and to design, if necessary, effective anti-seismic measures which will ensure the safety of the building and its inhabitants in the case of a strong earthquake. For this reason it was decided that a programme of tests of the typical, stone-and-brick masonry walls of the buildings which make up the oldest part of Ljubljana's city centre be carried out. These buildings date mainly from the 17th and 18th centuries, although some are even older. It is interesting to note that many of the oldest houses were destroyed in an intensity IX earthquake which hit central Slovenia in 1511, whereas a considerable number of badly-damaged houses had to be demolished after the intensity VIII Ljubljana earthquake of 1895.

In the text which follows, an account is given of the testing of a typical stone-and-brick masonry wall, in both original and strengthened state, which was carried out in-situ inside a building in the old part of Ljubljana's city centre. A description is also given of the other on-site and laboratory tests which were carried out in parallel with the in-situ test.

2. DESCRIPTION OF THE IN-SITU TEST

2.1 Location of the in-situ test

It was decided that the in-situ test be carried out inside a typical,4-storeyed masonry building in the older part of Ljubljana's city centre (see Fig.1). Historical records indicate that the greater part of this building was built during the 17th and 18th Century, although parts of the ground-floor walls probably date from an earlier period.

Fig.1 : View of building in the old part of Ljubljana, where the in-situ test was carried out.

A thorough preliminary examination of the structure of the load-bearing, masonry walls of the building was first carried out, by chipping away the plaster and cutting out some of the material of the wall, at a total of 80 locations. Suitable samples of the wall material were removed for analysis and investigation in the laboratory. It was found that the majority of the original load-bearing walls of the building were built of a relatively compact mixture of stone, brick and mortar, the volumetric proportion of stone to brick being approximately 4:1. Most of the stone blocks, with a maximum dimension of 0.30 m, consisted of micaceous-siliceous sandstone, originating from the nearby castle hill, with a compressive strength in the range 40.0 - 70.0 MPa. The brick, which was handmade, had a typical compressive strength of 15.0 MPa. The mortar consisted of a mixture of lime and unwashed, somewhat clayey, sandy-gravelly aggregate, of particle size 0 - 5.0 mm (max. 8.0 mm). Due to carbonatization of the mortar into a kind of conglomerate structure, a process which took place over a period of many years, the mortar had achieved a quite high compressive strength, in the range 2.5 - - 4.0 MPa.

The in-situ test of the stone-and-brick masonry wall was carried out in a section of a transverse load-bearing wall on the 3rd floor of the building (see Figs. 2 and 3). This location was convenient for several reasons, as follows: 1) at this location it was easy to make an accurate enough estimate of the existing vertical loading in the wall, which was of importance for the analysis of the test results, (2) the existing vertical loading being relatively small, a smaller horizontal

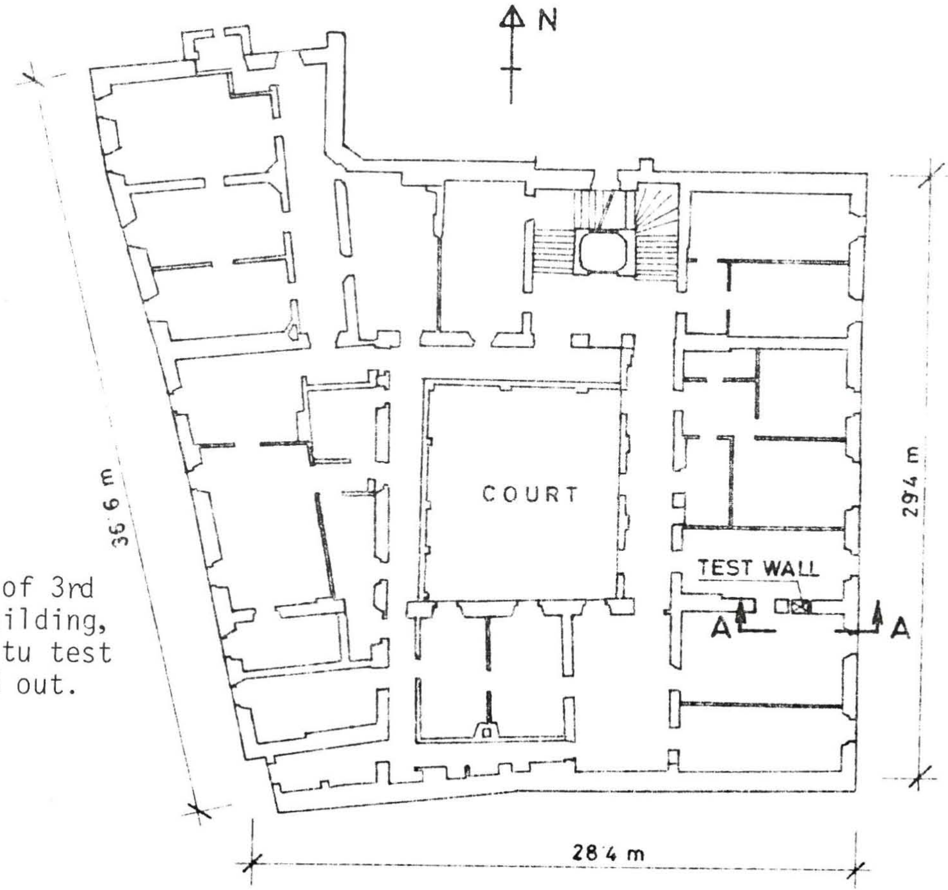


Fig.2 : Floor-plan of 3rd floor of building, where in-situ test was carried out.

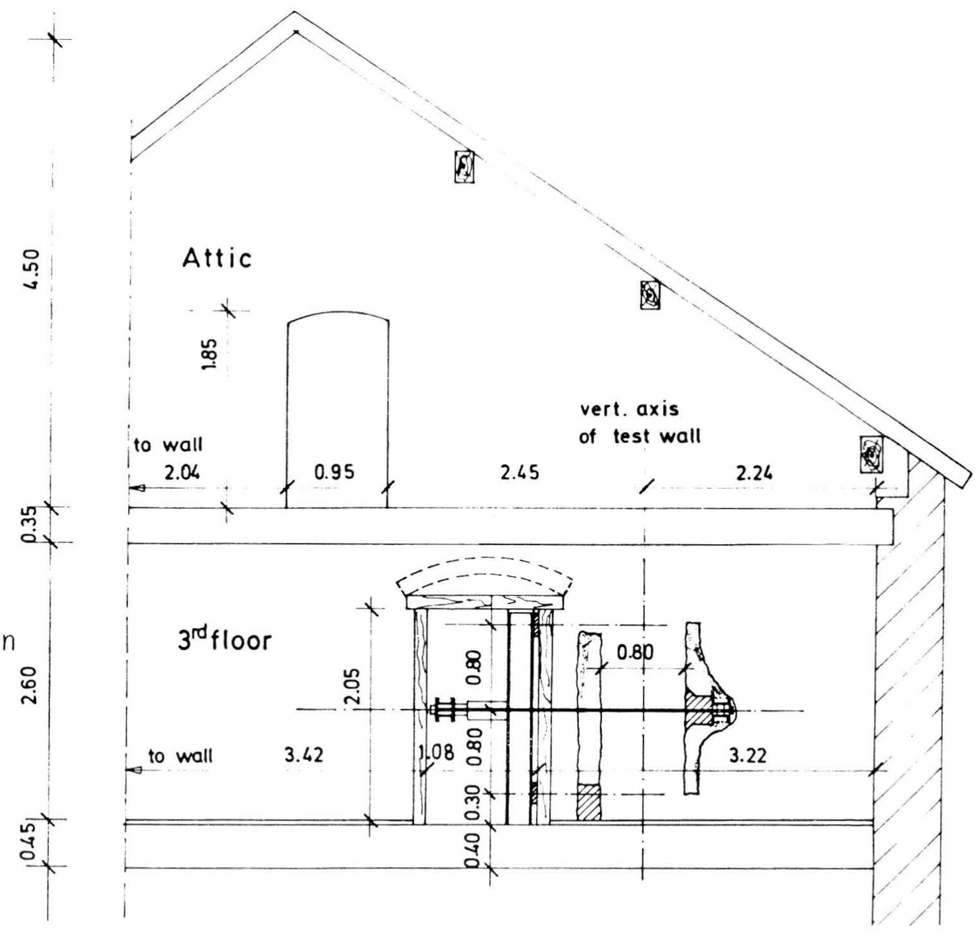


Fig.3 : Cross-section of building, (A - A).

load was needed to produce failure of the tested wall element, which meant that less powerful and more easily transportable testing equipment could be used and (3), being towards the top of the building, there was no danger of the test causing any significant structural damage. By comparing the structure of the tested wall element with the structure of the rest of the load-bearing walls of the building, it could be concluded that the tested, stone-and-brick masonry wall element was typical of the latter. From an examination of the walls at other nearby locations it was concluded that the section of wall tested actually belonged to the attic wall of the earlier three-storeyed 17th - 18th Century building, which was later raised by one storey to form the present building.

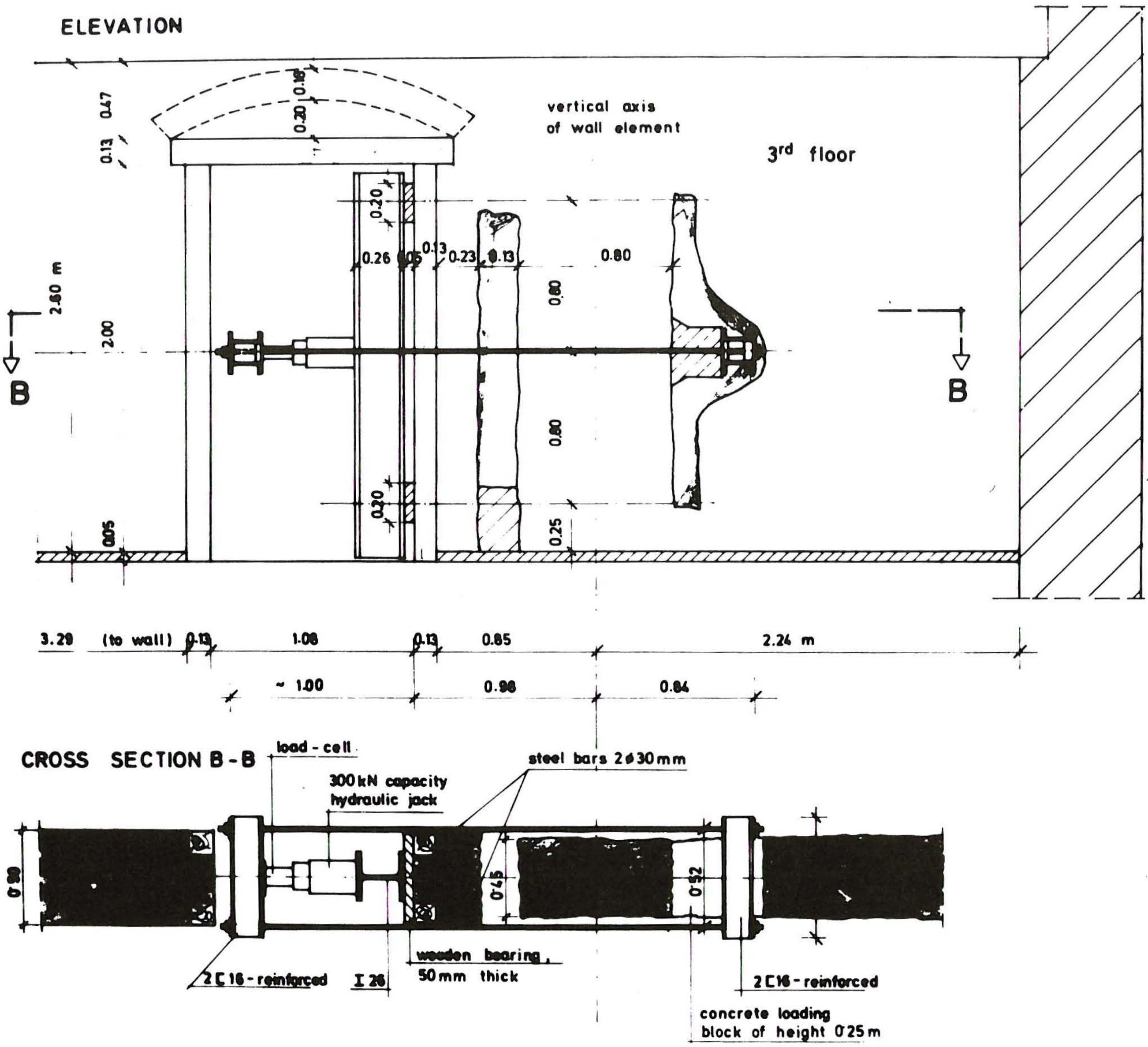


Fig.4 : Set-up for carrying out in-situ test of stone-and-brick masonry wall element.

2.2 Design of the test set-up

The test set-up (see Fig.4) was designed in such a way that the behaviour of the tested wall element would be easy to define and analyse. The method used was the following. About 0.30 m distant from the original door opening in the wall, two vertical slits, 1.6 m high and approximately one metre apart, were carefully cut out of the wall by hammer and chisel. In this way a completely intact wall element of height 1.6 m, effective width 0.8 m and thickness 0.45 m (without plaster) was obtained (see Fig. 5). At the mid-height of the wall element, a suitable concrete bearing-block, 0.30 m high and 0.45 m wide, was fixed to one side of the wall, so that a horizontal load could be applied. This was done by means of a hydraulic jack of capacity 300 kN, which acted on the wall via two horizontal steel tie-bars of diameter 30 mm, fixed to two transverse I-beams as shown in Fig.4. The jack's horizontal reaction was transferred to a vertically-erected I-beam, whose points of support, 1.6 m apart, coincided vertically with the two points of support of the tested wall element.

By choosing the above-described set-up for horizontal load application, in fact two symetrically-arranged wall-elements of dimensions 0.80 x 0.80 x 0.45 m were created, both having a height-to-width ratio of 1. It can be considered that practically full fixity of the two wall elements, at their top and bottom horizontal surfaces, was obtained, i.e. that these surfaces remained horizontal and parallel to one another throughout the test.

2.3 Carrying out of the test

The uni-directional horizontal load (2H) was gradually increased, in equal increments of approximately 15.0 kN, with intermediate unloading steps, until maximum horizontal load was achieved and diagonal cracks, indicating shear failure, occurred in both the top and the bottom half of the wall element (see Fig.6). Horizontal deformation of the wall element was measured by means of three LVDT's, located at the top, mid-height and bottom of the wall as shown in Fig.9 (c). The resulting horizontal load-deformation ("H-δ") diagrams for the top and the bottom half of the wall element, which were practically identical, were plotted on two "x-y" recorders. The results for the top half of the wall are shown in Fig.9 (a). Vertical elongation or shortening of the wall was measured by means of two dial-gauges.

At the end of the first test, strengthening of the tested wall element by means of full cement-grouting was carried out (see Fig.7). The mixture used for grouting consisted of 90% ordinary portland cement according to B.S.12 (or quality PC-45 according to the Yugoslav standard JUS B.C1.011), and of 10% pozzolana, to achieve plasticity and to prolong suspension of the mixture. According to volume, the ratio of the dry mixture to water was 0.75 to 1.0. A total of 75 kg of the dry mixture per cubic metre of the wall's volume was used, the maximum working pressure used for grouting being about 2.5 bar. The average compressive strength of three hardened cylinders of a sample of the grout mixture after a period of 28 days amounted to 24.5 MPa.

After 28 days had elapsed, the wall element was retested in the same manner. At the maximum achieved horizontal load, which was considerably higher than in the case of the original wall element, once again diagonal cracks occurred, indicating shear failure (see Fig.8). The horizontal load-deformation diagram for the top half of the wall, recorded during this test, is shown in Fig.9 (b).

3. RESULTS OF THE IN-SITU TEST

The results of the test are shown graphically in Fig.10, where the horizontal load-deformation envelopes for the basic and the strengthened wall have been compared. From this diagram it can be seen that the horizontal shear resistance, corresponding to one half of the wall element (H_{max}) increased by 45%, from 71.5 to 104 kN, whereas initial shear stiffness (K_o) increased by a factor of approxima-

Fig.5 : Cut-out wall element before test

Fig.6 : Basic wall element during test

Fig.7 : Cement-grouting of wall element

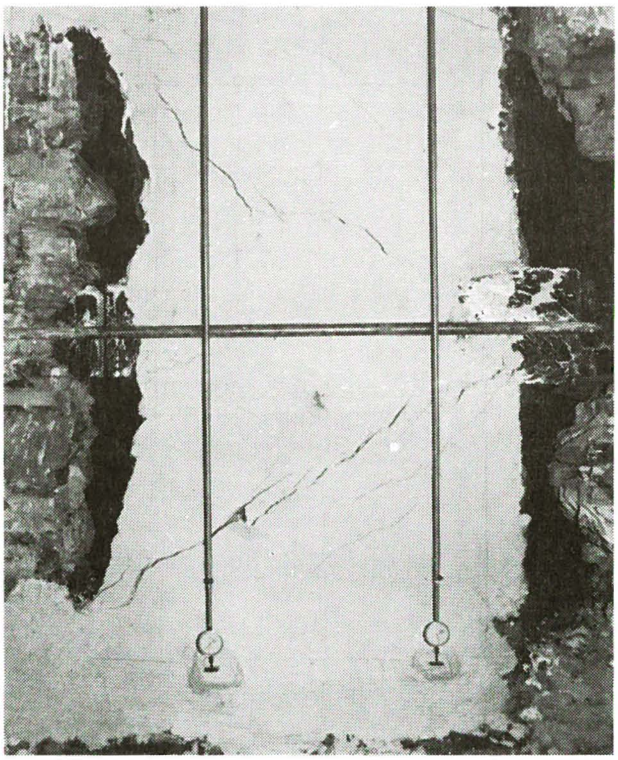
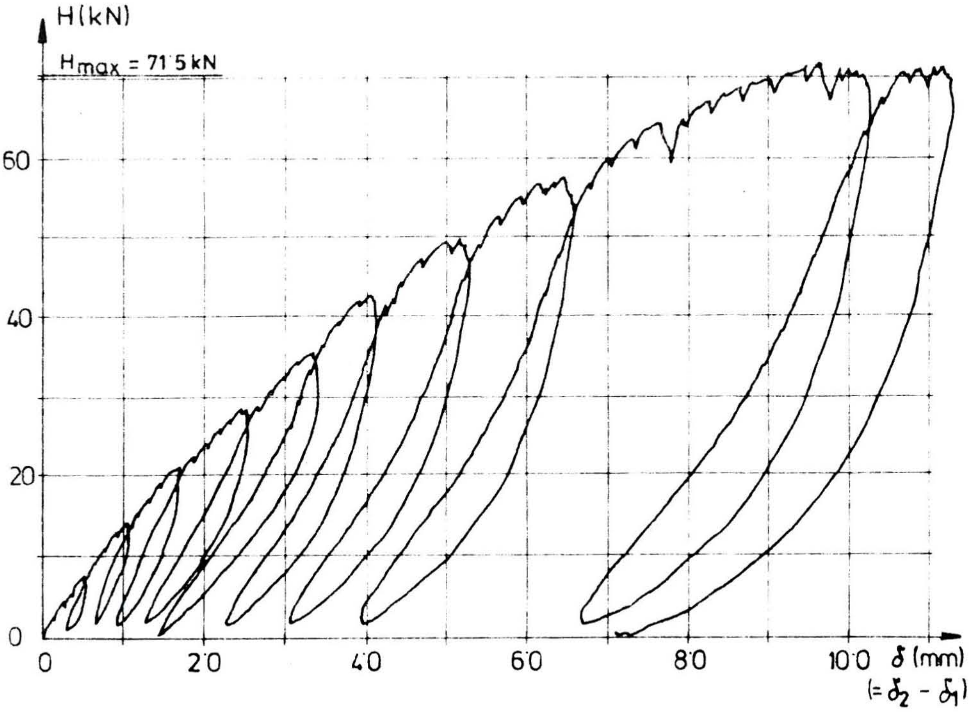
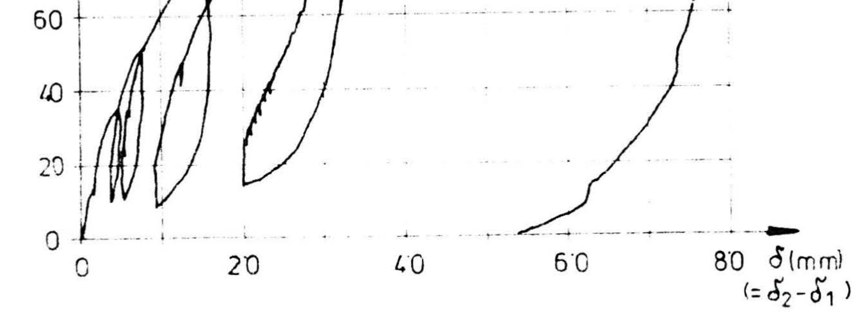


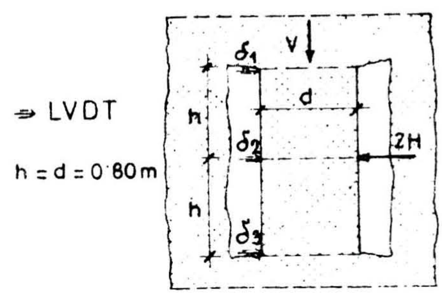
Fig.8 : Failure of strengthened wall element, with diagonal cracks



a) RECORDED H - δ LOOPS FOR BASIC WALL



b) RECORDED H - δ LOOPS FOR STRENGTHENED WALL



c) TEST SET UP

Fig.9 : Results of in-situ test of stone-and-brick masonry wall element

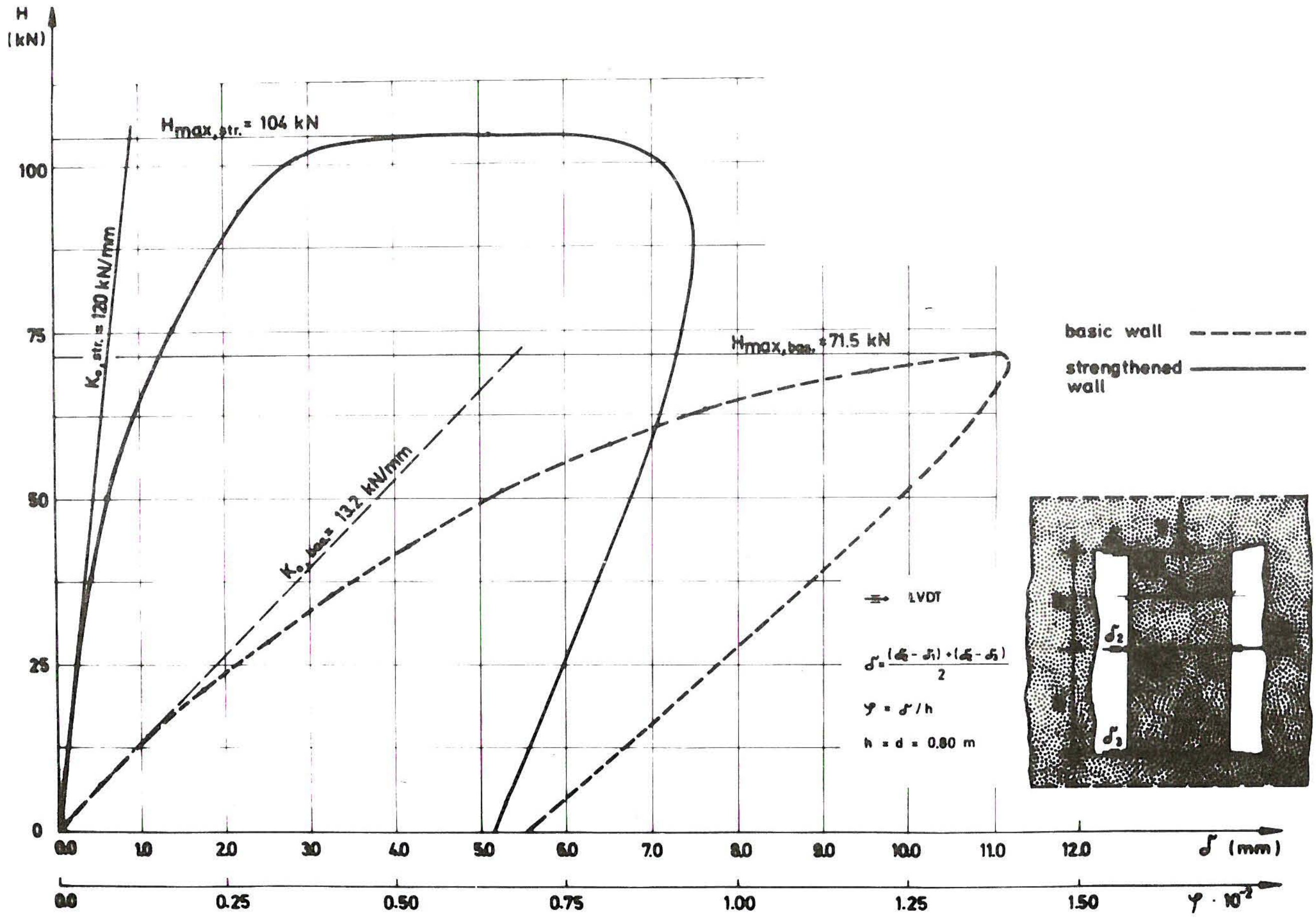
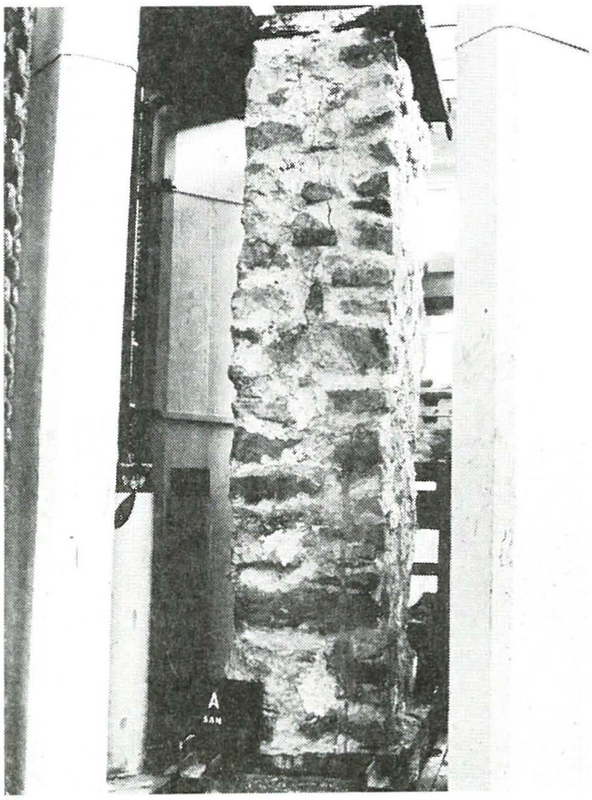
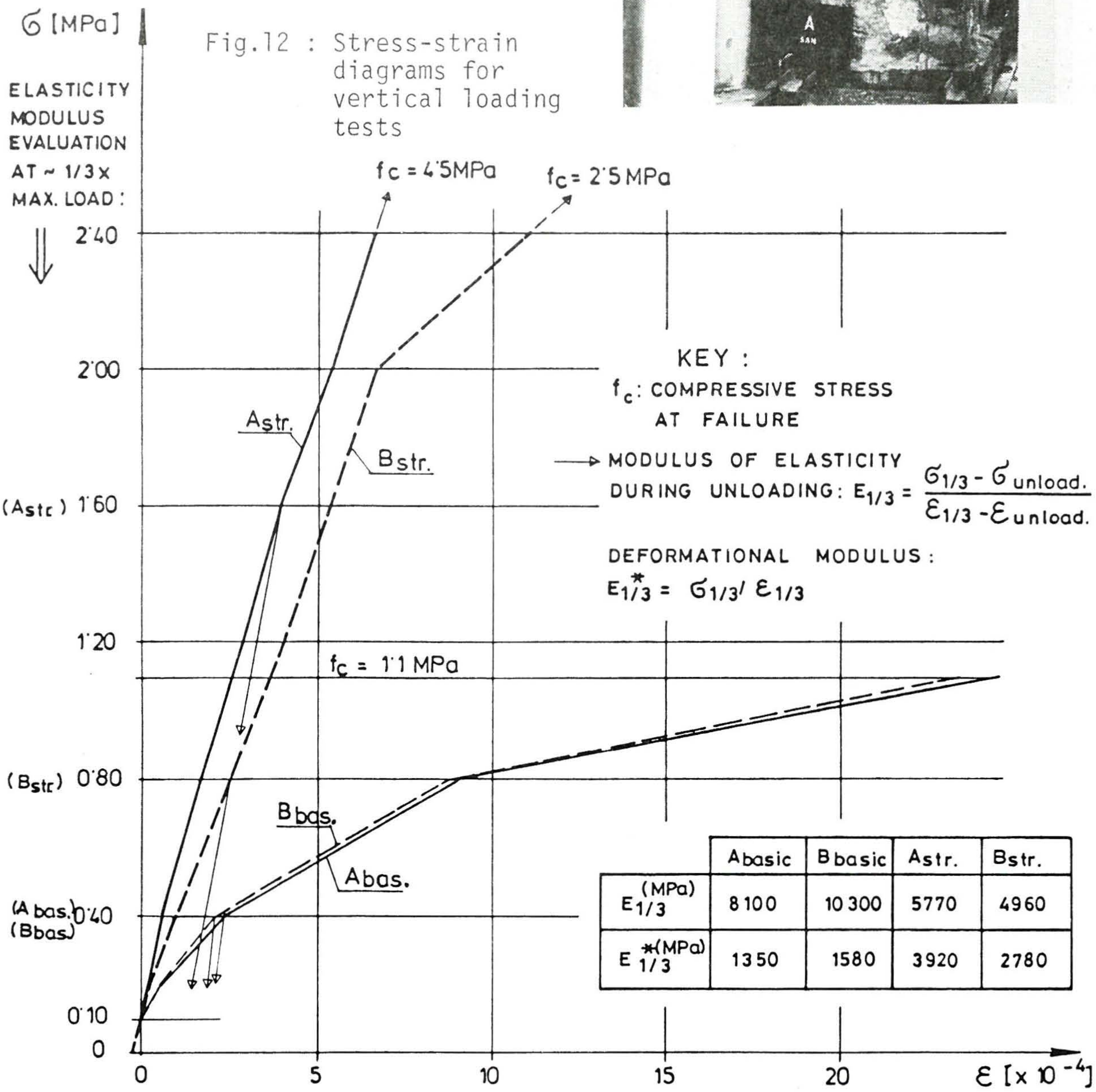


Fig.10 : Comparison of horizontal load-deformation diagrams for basic and strengthened wall element (upper half).

Fig.11 : Reproduced stone-and-brick masonry wall during axial vertical loading test





	Abasic	Bbasic	Astr.	Bstr.
$E_{1/3}$ (MPa)	8100	10300	5770	4960
$E^{*}_{1/3}$ (MPa)	1350	1580	3920	2780

Fig.12 : Stress-strain diagrams for vertical loading tests

tely 9.0, from 13.2 to 120.0 kN/mm. The theoretical horizontal deformation at the elastic limit ($\delta_o = H_{max}/K_o$) was reduced from 5.4 to 0.9 mm, but the horizontal deformation at maximum load (δ_{max}) decreased only from 11.0 to 7.0 mm.

From an analysis of the weight of the masonry and other material above the wall element, it was estimated that the vertical load acting on the latter amounted to V = 100 kN. Although it was not possible to make accurate measurements, it is estimated that vertical load increased during the test to a maximum value of approx. V = 150 kN. In the case of the strengthened wall element, the vertical load at failure was estimated to have had a minimum value of V = 120 kN.

The referential tensile strength (f_t) of a wall element with a height-to-width ratio of h/d = 1 : 1 can, in the case of shear failure with diagonal cracks, be defined as the maximum tensile stress at the centre of an equivalent, homogeneous wall panel, using the following equation (see reference [3]).

$$f_t = \sigma_o \left[-0.50 + \sqrt{\left(b \cdot \frac{\tau_o}{\sigma_o}\right)^2 + 0.25} \right] \qquad \ldots (1)$$

In this equation σ_o is the average compressive stress in the wall due to vertical load ($\sigma_o = V/F$, $F = 0.36\ m^2$) and τ_o is the average shear stress in the wall at failure ($\tau_o = H_{max}/F$). The parameter "b" is a function of the parameters σ_o and τ_o, and has, on the basis of the statistical analysis of a larger number of tests of walls with h/d = 1 (see reference [3]), been defined by the following equation:

$$b = 1.543 - 0.478\ (\tau_o/\sigma_o) \qquad \ldots (2)$$

The initial shear modulus of the wall (G_o) was determined from the expression for the flexibility of the wall when subjected to combined vertical and horizontal loading:

$$\frac{\delta}{H} = \frac{1}{K_o} = \frac{1.2\ h}{G_o \cdot F} \left[1 + \frac{G_o}{E^*_{1/3}} \cdot \frac{1}{1.2} \cdot \left(\frac{h}{d}\right)^2 \right] \qquad \ldots (3)$$

This equation assumes full fixity conditions at the top and bottom of the wall element. In calculating the values for G_o, a value of $E^*_{1/3}$ = 1000 MPa was assumed for the basic wall, and $E^*_{1/3}$ = 3000 MPa for the strengthened wall. $E^*_{1/3}$ is the deformational (secant) modulus of the wall when subjected to axial vertical loading, measured at 1/3rd of the load needed to cause compressive failure (see 4.).

The indicator of ductility (D_u) was defined as the ratio between the maximum horizontal deformation of the wall at H = 0.9 H_{max} and the theoretical horizontal deformation of the wall at the elastic limit:

$$D_u = \delta_{max}/\delta_o \qquad \ldots (4)$$

The values of f_t, G_o and D_u, calculated according to equations (1) - (4), are shown in Table 1. From this table it can be seen that, with respect to the only minor differences caused by assuming somewhat different values of the vertical load, a referential tensile strength of 0.13 MPa may be adopted for the basic wall, and of 0.20 MPa for the strengthened wall.

	Vertical load, V	Ref.tensile strength, f_t	Initial shear modulus, G_0	Ductility indicator, D_u
	kN	MPa	MPa	-
Basic wall	100	.137	40.0	~ 2.0
	150	.126		
Strengthened wall	120	.200	350.0	~ 7.0
	150	.199		

Table 1: Values of referential tensile strength, initial shear modulus and ductility indicator for basic and strengthened walls.

4. COMPARISON OF RESULTS OF IN-SITU TEST WITH RESULTS OF LABORATORY TESTS

During the same period that the in-situ test was going on, two wall elements were built in the laboratory out of the same original stone and brick, taken from a similar building in the old part of Ljubljana, using laboratory-prepared mortar of similar strength. These two wall elements, of height 2.50 m, width 1.0 m and thickness 0.50 m, designated wall "A" and wall "B" (see Fig.11), were subjected to an axial vertical loading test in a 5000 kN testing machine, then fully grouted using the same method as described in section 2.3, and then retested to failure. Vertical load was applied in equal increments, with intermediate unloading steps. Vertical deformation of the wall was measured by 4 dial gauges. The results of these tests are shown graphically in Fig.12.

Average compressive strength of the wall, f_c, increased from 1.1 MPa, for both basic walls, to 2.5 and 4.5 MPa (on average 3.5 MPa) for the two strengthened walls. This means that the ratio between the referential tensile strength of the wall, as determined in the in-situ test, and the average compressive strength of the wall, amounted to 0.13/1.1 = 12.0 % for the basic wall, and 0.20/3.50 = 6.0% for the wall strengthened by cement-grouting. These percentages are within the normal range for such walls. The ratio of elastic shear and deformational (secant) moduli amounted to $G/E \simeq 1/25$ for the basic wall, and $G/E \simeq 1/10$ for the wall strengthened by cement-grouting.

The results of the in-situ and laboratory tests of the stone-and-brick masonry walls fit in quite well with the results of previous laboratory tests of stone-masonry walls, carried out at ZRMK Ljubljana (see reference [2] - wall categories III and IV), although the load-carrying characteristics of the basic, unstrengthened wall were somewhat more favourable than might have been expected from previous experience. This can be explained by the fact that it is very difficult to reproduce accurately the older types of stone-masonry wall, where carbonatization of the mortar into a kind of conglomerate has taken place over a period of many years. In this case, in-situ tests, of the kind described here, can provide the most accurate information about the load-carrying and deformational characteristics of such walls.

5. CONCLUSION

On the basis of the results of the in-situ test, as well as of the vertical loading tests of the reproduced stone-and-brick masonry walls carried out in the laboratory, the following design values are proposed, which may be used for estimating the seismic resistance of older masonry buildings, built in a similar way out of a similar kind of material:

	Basic walls	Walls strengthened by full cement-grouting (80-120 kg of dry grout mixture per m^3 of wall)
Ref.tensile strength, f_t	0.08 MPa	0.16 MPa
Compressive strength, f_c	0.9 MPa	1.8 MPa
Initial shear modulus, G_o	50 MPa	300 MPa
Deform.modulus $E_{1/3}$ (vert.ld.)	1000 MPa	3000 MPa
Ductility indicator	1.5	2.0

The strength and deformability characteristics of older masonry walls can, however, vary widely. In the case of revitalization projects it is therefore recommended that suitable in-situ tests of a similar kind to the one described here be carried out in order to determine these characteristics as accurately as possible.

6. ACKNOWLEDGEMENT

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