THE CONTRIBUTION OF FLANGES TO WALLS UNDER THE ACTION OF THE ARCHING EFFECT

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SUMMARY

This paper analyzes the arching effect in a masonry structure considering the interaction between walls. It discusses the choice of an experimental model and reports on an experiment with reduced scale models of walls supported on concrete beams for the analysis of whether or not the flanges contribute to walls under the arching effect.

INTRODUCTION

Studies on the composite wall-beam action, the arching effect, began with R. H. Wood in 1952 and, since then, many researchers have dedicated themselves to studying this problem. Some books including those by Hendry (1981), Hendry *et al*. (1981) and Drysdale *et al*. (1994) consolidated the results of the main studies carried out and presented design methodologies for obtaining the loads as a function of the redistribution of stresses due to the arching effect, particularly the methods proposed by Riddinghton and Stafford Smith (1978) and Davies and Ahmed (1978).

The studies of these researchers showed that structural masonry walls supported by beams act together, when the beam, on being loaded by the wall, deforms, causing a displacement of the wall in the deformed region. This means that the action of the wall on the beam, initially a uniformly distributed load, becomes concentrated close to the supports, thus causing a substantial reduction in the bending moments in the beam in relation to those obtained with the uniformly distributed load.

The study and quantification of the effects of this composite action, besides being necessary for understanding it, aims at the rational sizing of the wall and of the beam, in order to resist the actions resulting from the behavior of the arching in a more economical way, that is, to allow a reduction in the material costs while maintaining the structural integrity.

However, the most notable fact is that, until now, the studies carried out have concentrated on isolated elements, separately from the structure, that is, they have not analyzed the interaction between the walls, or between the walls and columns (of masonry), so common in the structures comprising structural masonry.

EXPERIMENTAL METHODOLOGY

In structural masonry the walls can be considered without flanges (buttresses), as isolated panels, however, they are commonly associated with flanges in a “C”, “L”, “T”, “I” or “Z”

form. For the adoption of the model, the aim was to maintain, as far as possible, the structural characteristics of the wall, as part of the structure. However, the discretization of the structure leads to, or highlights, effects and loads which are non existent or of little significance in the real structure. A numerical pre-simulation of these walls supported on a concrete base for the visualization of the deformed structure shows, as predicted, that the displacement of a wall in the form of a “C”, or the rotation of a wall in the form of a “Z” or “L” along the main axes of inertia, make the load on them eccentric. Consequently, torsion stresses occur altering and distorting the loads in the panel faces and the buttresses.

Of the possible sections of panels with flanges, the “I” and “T” sections remain. A numerical pre-analysis showed that in an “I” wall the action of the buttresses on both sides, absorbing the stresses displaced to the sides of the walls, practically decharacterize the action of the arching effect and, the “T” wall presented a mixed behavior: on the return wall, very small stresses, similar to those of the I panel and, on the free wall, much greater stresses, similar to those of the isolated wall. Thus, the panel in the “T” form was chosen since it also enables the analysis of sides with and without buttresses (Figure 1).

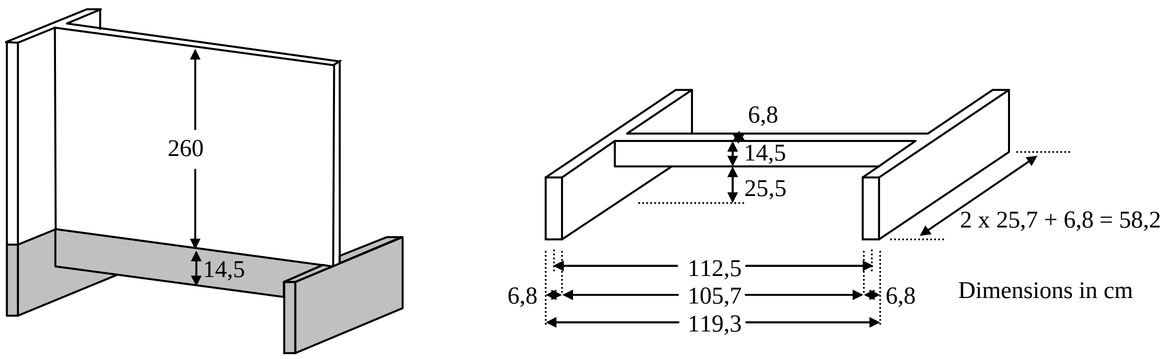

Figure 1. Diagram of the model with flanges adopted

The dimensions were adopted based on a real structure. A building on stilts, in structural masonry, was considered, with ten stories, the first story being a reinforced concrete structure, comprised of slabs, beams and columns. The wall loading was obtained according to the Brazilian standard NBR-6120 (1980) and the beam span and wall height were adopted according to values commonly used in buildings.

Initially, the sizing of the concrete beam was carried out and adopting p = 130.99 kN/m, f_{ck} = 20.0 MPa, CA50A - f_{yk} = 500 MPa, L = 3.76 m, b_{wv} = 0.17 m the following values were obtained: h = 0.70 m, d = 0.65 m and As = 15.28 cm^2.

On determining the area of contact of the applied load with the wall (= 0.5264 m^2) and the stress applied to this area of contact (= 935.65 kN/m^2) the transposition for the calculation in reduced scale was carried out, adopting a scale of 1:3.333, and as the correlation between the real structure and the model, the same stress applied to the area of contact was adopted as the parameter.

For the model, the following values were adopted: stress applied to the area of contact (maintained) = 935.65 kN/m^2, b_{wp} = 0.051 m, and span = 1.128 m. On determining that the load/wall area of contact of the model is 0.04738 m^2, the total applied load in the model was obtained as 44.33 kN, implying 39.30 kN/m, enabling the sizing.

The model was pre-sized conventionally (without considering the arching effect), considering the arching effect (concentration of stresses – triangular – close to the supports) and according to the design methodology proposed by Davies and Ahmed (1978). On analyzing the results obtained in the pre-sizing, a support base section was adopted, observing the increase in beam width to 0.068 m due to shearing.

	Mf (kN.m)	b_w (cm)	h (cm)	As (cm^2)
Conventional calculation	6.251	5.1	21.0	1.382
Arching effect – triangular loads	2.084	6.8	14.5	0.604
Arching effect – Davies and Ahmed	1.086	6.8	14.5	0.289

Finally, the adopted section was verified for the design and test conditions (with/without safety coefficients), and a rupture prediction for the test conditions was obtained, for a loading between 80 and 100 kN.

The masonry was comprised of ceramic blocks.

For the experimental analysis four experiments were carried out: two prototype tests with buttresses and two prototypes without buttresses. For each experiment tests were carried out in relation to concrete and to masonry.

For the concrete of the support base, the cement CPII-F-32 was used, and coarse aggregate and medium sand commercially available in Florianópolis. The aggregates were measured by weight and water volume, the mix adopted being (cement:sand:gravel:w/c) 1:1.192:2.679:0.56 l/kg, and the following values were obtained in the tests $f_{c,average}$ = 29.50 MPa and $E_{c,average}$ = 27143 MPa.

The mortar for the masonry (walls, small walls and prisms) was made with CPII-F-32 cement, hydrated lime and a very fine sand. The mix adopted was 1:1:6 by volume corresponding to 1:0.435:5.694 by mass, with a water/cement factor of 1.69 (water/agglomerant = 1.18), the aggregates being measured by weight and the water by volume. The average resistance to compression was $f_{a,average}$ = 5.97 MPa

The masonry was comprised of ceramic blocks in reduced scale (1:3.33) as shown in Figure 2. The characteristics of the masonry were determined through testing of the blocks, prisms and small walls.

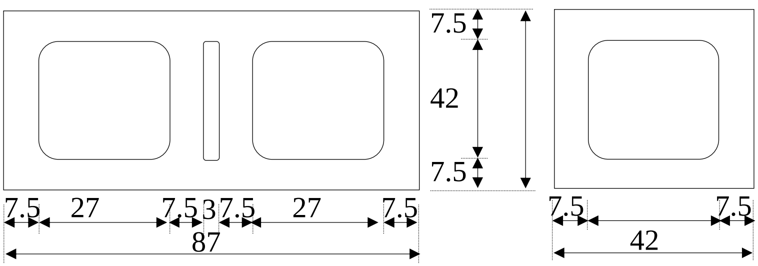

Figure 2. Shape and dimensions (mm) of the blocks and half blocks, in reduced scale.

The blocks showed an average resistance to compression of f_b = 14.18 MPa and a longitudinal elasticity modulus of E_b = 5325.3 MPa. The prisms of the three blocks showed an average

resistance to compression of $f_{pr.}$ = 9.09 MPa. The small walls (RILEM – six rows of four blocks) were used in order to determine the longitudinal and transversal elasticity moduli and the Poisson coefficient. For the small walls loaded longitudinally the values obtained were: f_{pa} = 8.90 MPa, $E_{pa.}$ = 2876 MPa and Poisson coefficient = 0.237 and, for those loaded laterally: f_{pa} = 2.97 MPa, $E_{pa.}$ = 1740 MPa and Poisson coefficient = 0.312

EXPERIMENTS

For the experiments a test frame was used with a 20 ton-force capacity, a load cell for a 20 ton-force and an HBM data acquisition system, Spider 8, with the use of the application CATMAN from the HBM software itself. For the measurement of wall deformations four inductive displacement transducers and six strain gauges, fixed to the blocks of the first row on both sides of the wall, were used. In the panels with flanges, an inductive displacement transducer was used on each flange. In order to determine the beam deflection an inductive displacement transducer was used. Figure 3 shows the positioning of the instrumentation, the strain gauges and the transducers used in the experiments.

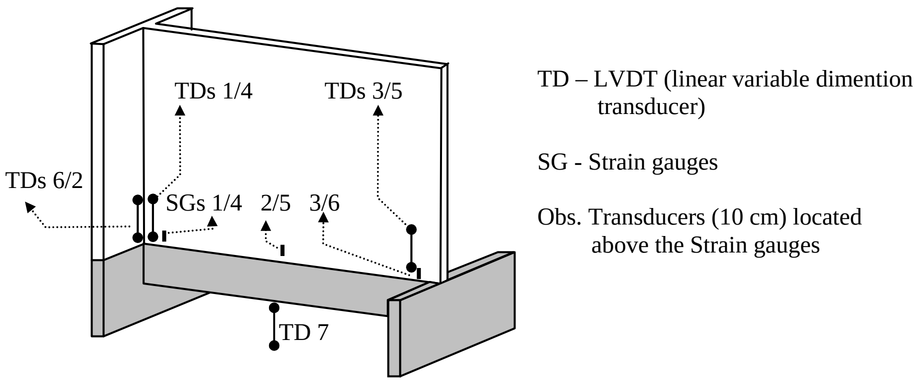

Figure 3. Positioning of the instrumentation

The tests were carried out according to the recommendations of the Brazilian standard NBR 8949 (1985), two pre-loadings being carried out to accommodate the masonry and the instrumentation, one of 8 kN and the other of 14 kN, maintaining a load application level for five minutes, after which the structure was unloaded. The loading velocity used was approximately 0.003 $N/cm^2/s$ (1.5 N/s) and the instrumentation was removed when the opening of cracks indicated a near collapse. As established prior to the study (service situation), the analysis of the results was made within the range of 40 to 50 kN. Figure 4 shows the construction of the prototypes, details of the instrumentation and the rupture of prototype 04.

Figures 5 and 6 show, for experiment 2 (without flanges), the deformations obtained through the strain gauges and the transducers, for the loads of 10, 20, 30 and 40 kN and Figures 7 and 8 show, for experiment 3 (without flanges), the deformations obtained through the strain gauges and the transducers, for the loads of 10, 20, 30 and 40 kN

Since the structures are symmetrical, Figures 5 to 8 should show the same values for the SGs 1/4 and 3/6 and for the TDs 1/4 and 3/5, respectively, but show a greater deformation on the left side of the walls. The diagrams of the experiments indicate, for prototypes 2 and 3 (without flanges), strong indications that the left side of the wall was more loaded than the right.

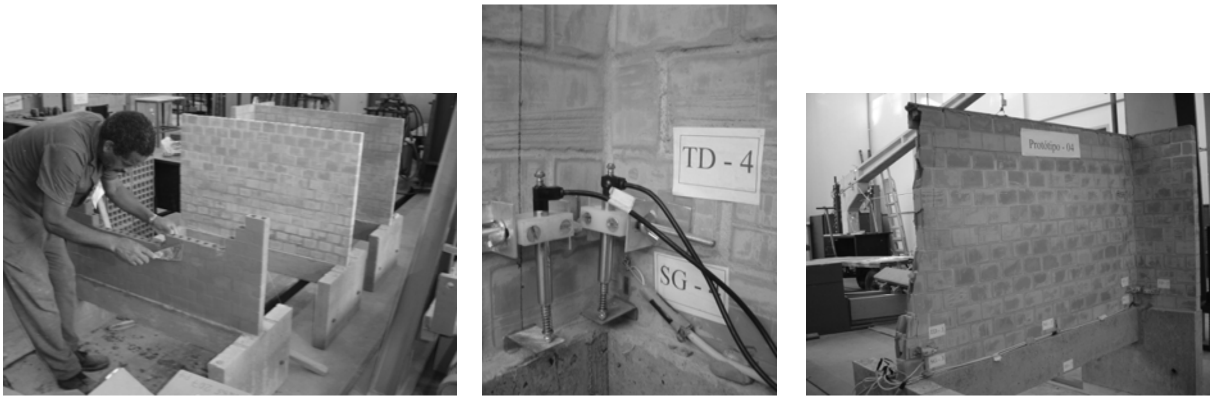

Figure 4. Construction of the prototypes, details of the instrumentation and the rupture of prototype 04.

Fixing the load at 10, 20, 30 and 40 kN, the deformation of the strain gauges and transducers were traced, in order to visualize the behavior of the deformation distribution (stresses) of the four prototypes.

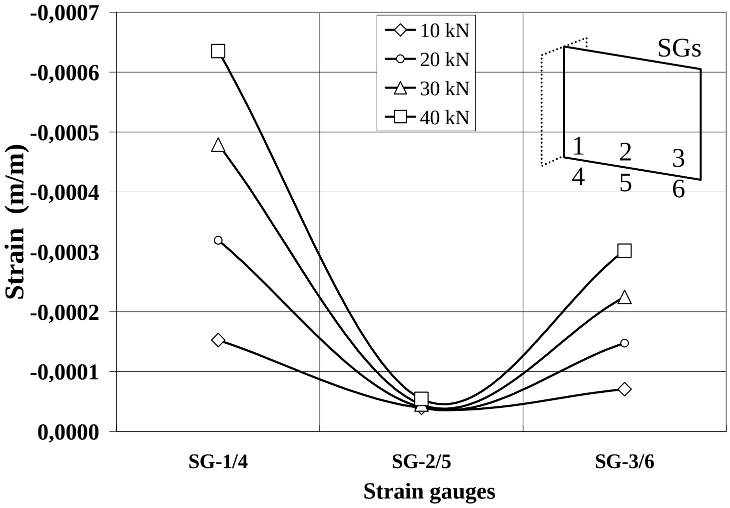

Figure 5. Prototype 2 – Average deformations of opposite strain gauges.

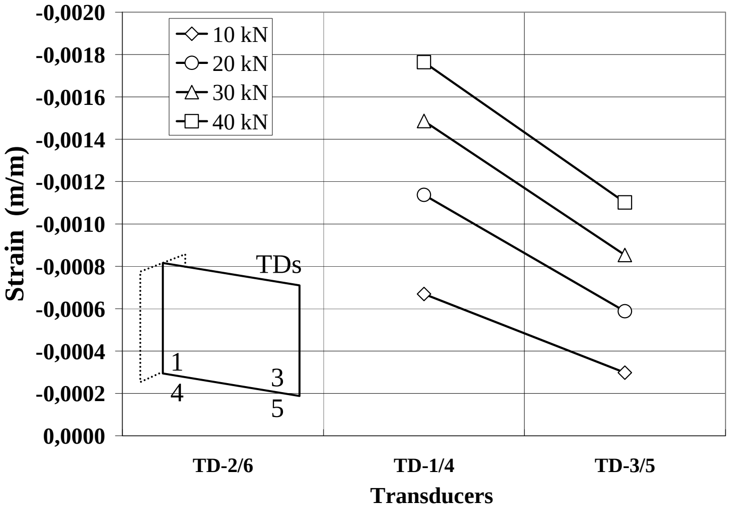

Figure 6. Prototype 2 – Average deformations of opposite transducers.

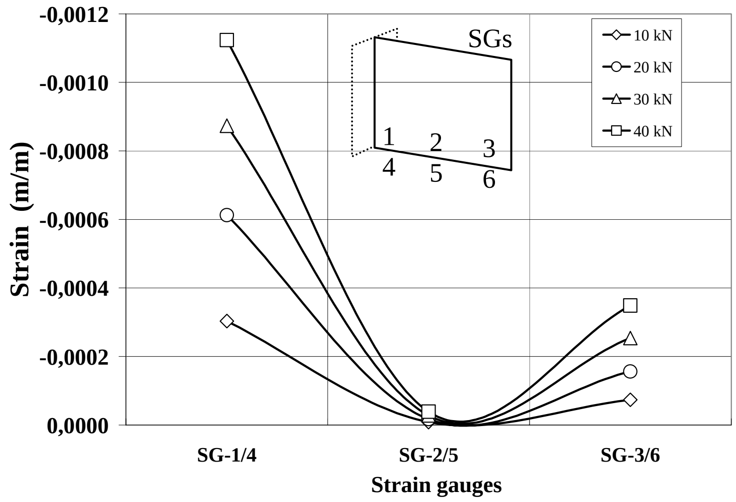

Figure 7. Prototype 3 – Average deformations of opposite strain gauges.

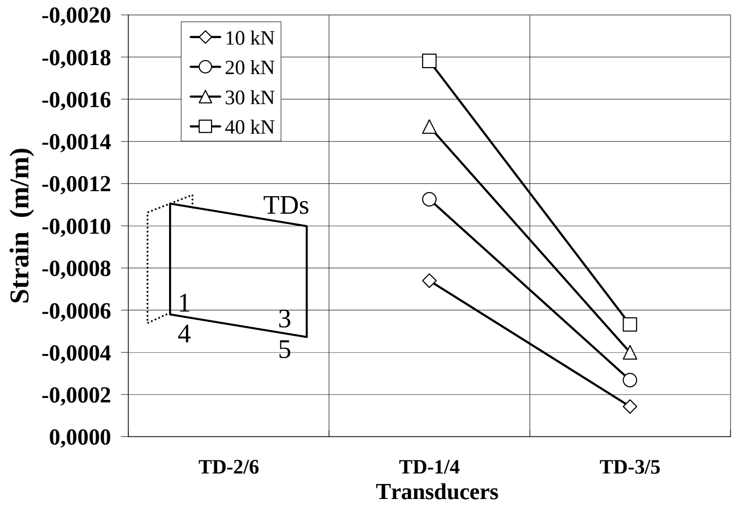

Figure 8. Prototype 3 – Average deformations of opposite transducers.

Figures 9 to 12 show, for experiments 1 and 4 (with flanges), the deformations obtained through the strain gauges and the transducers for loads of 10, 20, 30 and 40 kN.

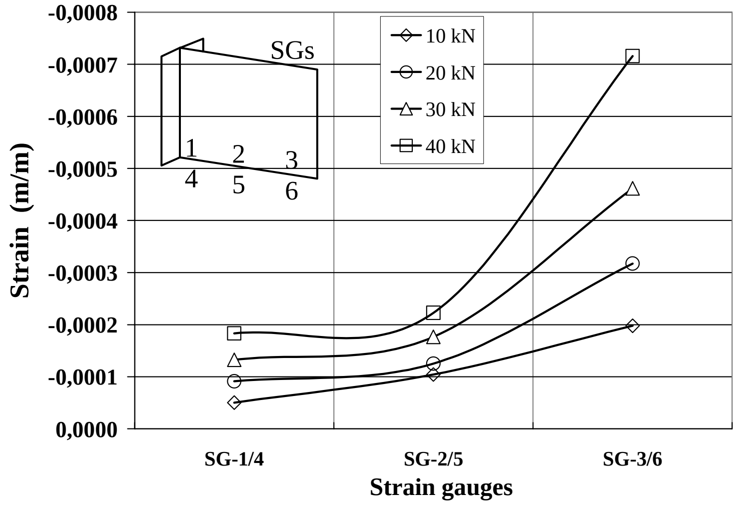

Figure 9. Prototype 1 - Average deformations of opposite strain gauges.

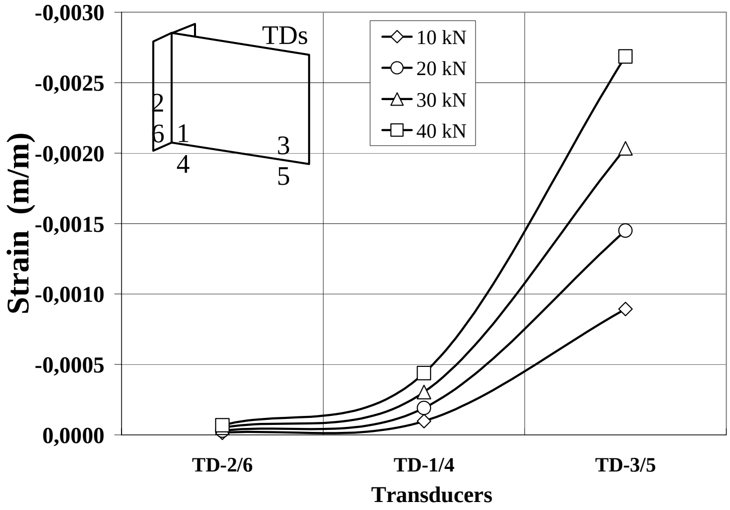

Figure 10. Prototype 1 - Average deformations of opposite transducers.

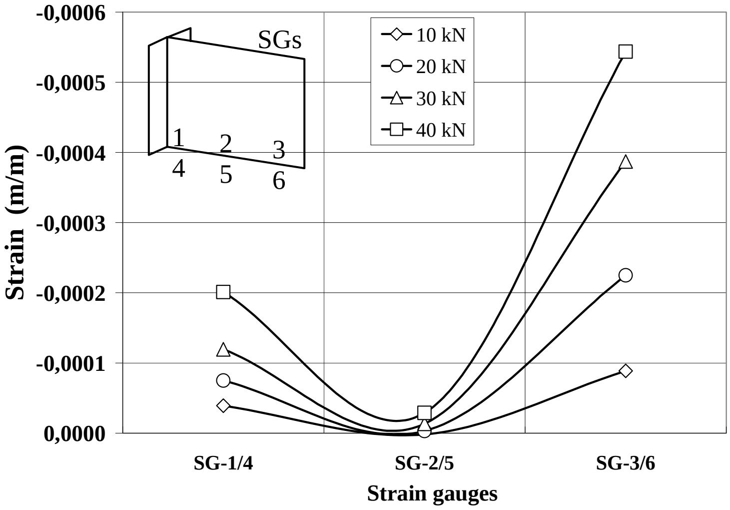

Figure 11. Prototype 4 - Average deformations of opposite strain gauges.

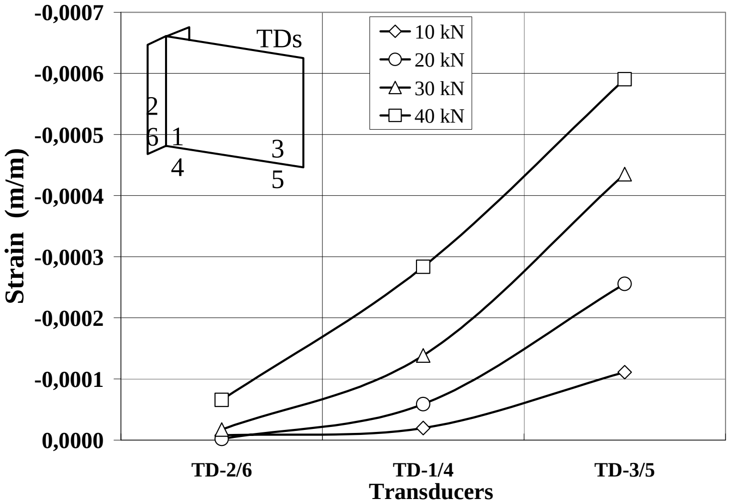

Figure 12. Prototype 4 - Average deformations of opposite transducers

The contribution of the flanges is very clear in these diagrams. The deformations of the side with flanges are much smaller than those on the side without flanges (SGs 1/4 << SGs 3/6 and

TDs 1/4 << TDs 3/5) and at the extremity with flange (Figures 9 and 10) the deformations are similar and slightly greater (of the same order of magnitude) than those obtained at the center and much smaller than those obtained at the extremity without flange. Figures 10 and 12, which show the deformations obtained by the transducers, besides highlighting this behavior, show the contribution of the flanges through the deformations of transducers 2 and 6.

In Figures 13 and 14 a comparison is made between the average values of the deformations obtained for a load of 40 kN, for the prototypes with and without flange.

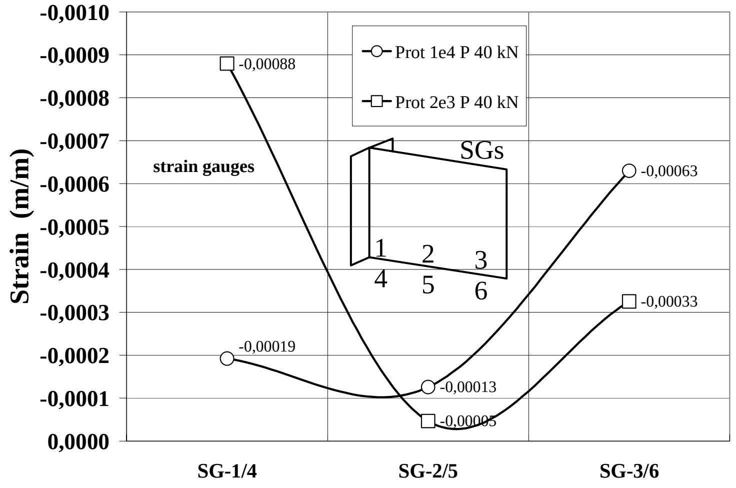

Figure 13. Average deformations of opposite strain gauges – averages of prototypes 1-4 and 2-3 (P = 40 kN).

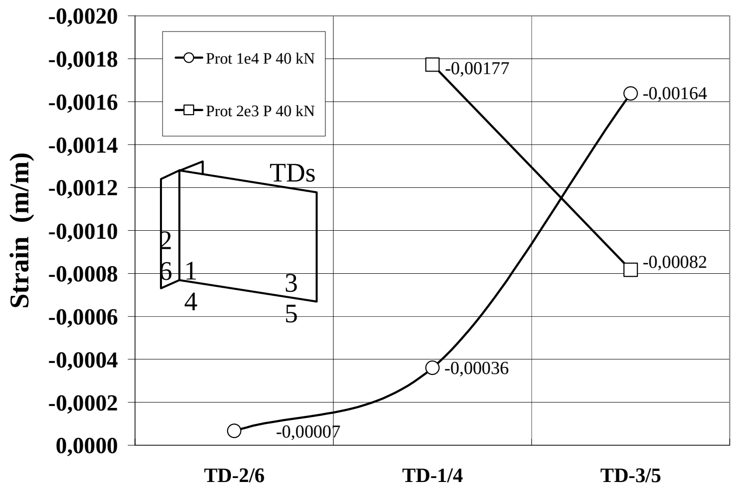

Figure 14. Average deformations of opposite transducers – averages of prototypes 1-4 and 2-3 (P = 40 kN).

Figures 12 and 13 show, as mentioned above, that in the panels without flange, (prototypes 2 and 3) the left side has greater deformations that the right side, whereas, given their symmetry, they should be the same. It can thus be concluded that the left extremity was slightly more loaded than the right one; however, even so, the panels with flange had

considerably smaller deformations on the sheared extremity, showing the partial absorption of the stresses by the flanges.

ANALYSIS

Signor and Roman (2002) carried out an analysis of the transfer of vertical loads between orthogonal walls uniformly supported on concrete bases. On comparing with research carried out by Sinha and Hendry in 1979, which indicated a transfer of vertical loads of between 5 and 6 %, it can be concluded that this transfer was much greater, achieving a complete distribution of stresses which reached the wall/flange intersection.

The experiments showed that the flanges absorbed a significant part of the stresses. In the panels with flanges, the stresses corresponding to the transducers on the extremity with flanges are approximately 22% of the stresses in the isolated extremity. Analogously, the stresses corresponding to the strain gauges on the extremity with flanges are approximately 30% of the stresses in the isolated extremity.

It was observed that the stresses acting on the side with flange are of the same order of magnitude as the stresses originating from the loading (12 to 15 % greater), whereas on the isolated side these stresses are four to five times greater than the stress applied by loading.

The experimental values were compared to those obtained by the methodology of Davies and Ahmed (1978), verifying that the maximum stresses obtained through the proposal of the researchers were 40% greater than those obtained in the experiments without flanges, that is, they can be considered to be in agreement.

From an analysis of the experiments with and without flanges, it can be observed that:

- on the sides of the T panels without flanges, similar stresses to those verified on the isolated sides of the panels were observed;
- on the sides of the T panels with flanges, there was a reduction in the compression stresses of the masonry of the order of 2/3, when compared with the stresses of the free sides (isolated);
- the experimental results for the sides without flanges were consistent with those obtained using the calculation method proposed by Davies and Ahmed (1978) for masonry panels over simply supported concrete beams.

CONCLUSIONS

On the side with flanges the concentration of stresses close to the support was absorbed by the flanges (there was a significant transfer of stresses – of load – to the flanges), however, in contrast, on the isolated side there was a concentration of stresses and the stresses in the masonry obtained for a load of 40 kN, a load possible in buildings with structural masonry, would have reached unacceptable levels in terms of design – of the order of four times the applied stress.

The studies carried out in this research indicate the relevance of the analysis of the wall surroundings subject to the action of the arching effect for designs in structural masonry under this action. This is because studies relating to the interaction of walls in which the action of the arching effect is present are still incipient, and design techniques which impose the action of the arching effect for the reduction of the support beam section are unreliable since they do not consider the load transfer between the walls.

However, in situations where the walls are isolated, the consideration of the arching effect allows significant reductions in the beam height, even with the limitations imposed by the verification of the shearing according to the prescriptions of the NBR 6118 (2003). In this case, the concentration of stresses in the masonry must be considered, in the lower corners of the walls.

For future research, a deepening of the studies related to the interaction of walls is suggested, when they are subjected to the action of the arching effect, emphasizing both the other sections for the wall/flange, and the relation between the stiffness of the support beams and the transfer of stresses between the walls. Regarding the model loading, it is also proposed that the interactions with the loaded walls and flanges are studied.

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