The influence of tension strain of wall ends to their resistance against lateral instability for low-reinforced concrete walls

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ABSTRACT:
One important aspect of seismic design of buildings with a dual reinforced concrete structural system is the lateral stability of structural walls, when they face this danger basically due to flexural overstrain. The deep excursion in the yield region of the boundary parts of bearing walls increases dramatically their flexibility and since at the same time they are liable, because of the earthquake vibration, to a reversing axial loading (tension - compression), their lateral stability is at stake. The possibility of failure because of lateral instability is limited significantly with the proper choice of an adequate thickness, which is specified by (most) modern seismic codes as a percentage of the height of the bottom storey. The current work investigates one of the most basic parameters affecting the stability of structural walls, which is (apart from the wall thickness) the degree of tension strain of the longitudinal reinforcements of the boundary edges of load-bearing walls. The present work is experimental. It has to be noted that in order to examine experimentally the influence of tension strain, 5 test specimens of scale 1:3 simulating the boundary edges of structural walls were used. These specimens were reinforced with low longitudinal reinforcement ratio (2,68%) and they all had the same reinforcement ratio. The degree of tension strain which was applied was different for each specimen and it took values equal to 0‰, 10‰, 20‰, 30‰ and 50‰. The present article tries to investigate the influence of the degree of tension strain to the ultimate bearing capacity of test specimens.

Keywords: R/C walls, lateral instability, tension strain, reinforcement ratio

1. INTRODUCTION

A usual practice of seismic design of buildings from reinforced concrete is the utilization, at the conceptualization of spatial body of structure, of sufficient walls. Relative researches (Wallace and Moehle, 1992) have shown that buildings, of which the static system has been conceptualized with the use of a large number of walls, have demonstrated exceptional behaviour against seismic action, even if they had not been reinforced according to the modern perceptions. Moreover, it is known that the reinforced concrete walls constitute a sufficient system of resistance against lateral forces for multistorey buildings, because of their capability to limit drastically the lateral displacements, either under serviceability limit state loading conditions (winds), either under extreme seismic conditions, thanks to their innate capability of ductile behaviour. For seismic loads, the walls should be designed in such way in order to ensure that they have sufficient capacity resistance against shear and their bending reinforcement must be sufficient against local buckling under the effect of large intensity cyclic loading. At the same time, such geometric characteristics, and mainly thickness, should be selected, so that there is no sensitivity against lateral instability (see Fig. 1.1). With the proper design and the correct detailing of walls as well as with the application of principles of capacity design, increased displacement ductility factors can be achieved, that can exceed even the value of four (Paulay, 1986), as well as exceptional capability of energy damping through structural elements of utmost seismic importance.

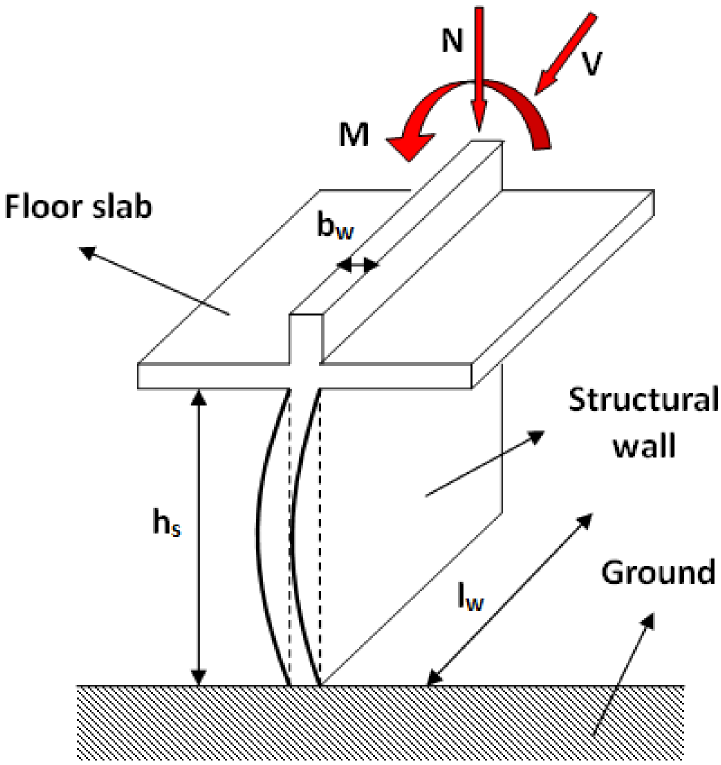

Figure 1.1. Out-of-plane buckling of structural wall

It is expected that walls which were designed either with increased ductility requirements according to Ε.Κ.Ω.Σ. 2000 (Greek Concrete Code 2000) or were designed to be in a high ductility category according to EC8: 2004, NZS 3101: 2006 and other modern international codes, present extensive tensile deformations, especially in the plastic hinge region of their base. Depending on the geometric characteristics and the level of ductility design of walls, tensile deformations until 30‰ are expected (Chai and Elayer, 1999). These tensile deformations can cause their lateral instability depending on their size. Large width cracks, which are created as result of deep entry in the plastic region, are required to close, so that the in-plane flexural mode of wall can be completely developed at the reversal of loading sign. It is obvious that there should be a sufficient wall thickness, so that it is ensured that the compressive force can be developed in the compression zone of the wall cross-section without the event of out-of-plane buckling. A critical situation arises when at the reversal of the sign of moment, the cracks that emanate from tension (at the previous semi-cycle of loading) cannot close and thus, traverse buckling takes place, as the one which is presented in the Fig. 1.1.

The phenomenon of transverse buckling in the compression edges of walls in the plastic hinge region (base of wall) is a no warning (and consequently very dangerous) phenomenon since it can lead the structures to total collapse and in particular without leaving any proofs that the total collapse and failure emanated from this specific phenomenon. Prominent researchers (Paulay, 1986) propose the use of flanges or enlarged boundary elements in the edges of walls, which provide protection to the bending compression regions against transverse instability and are easier to be confined. Also, international codes, such as New Zealand NZS 3101: 2006, propose the construction of such elements when the minimum wall thickness is determined from the criterion of evasion of transverse instability. However, these elements in order to be manufactured require more time, more reinforcement and more effort, thus resulting in bigger construction cost. Also, architectural restrictions and aesthetic reasons many times constitute, if they do not impose, the avoidance of configuration of such elements at the wall edges. Thus, the requirement for walls of continuous rectangular cross-section (without enlarged and distinguished edges), which will not fail because of the phenomenon of out-of-plane buckling, is henceforth established.

Although the phenomenon of transverse instability, as it was mentioned before, is a critical phenomenon for the structural stability and consequently the safety of constructions, too, however the number of investigations that has been carried out worldwide is limited. Relevant study (Paulay and Priestley, 1993) showed that this phenomenon depends and is influenced mainly by the size of tensile deformations which are imposed at the edges of walls at the first semi-cycle of loading and not so much by the size of flexural compression which is imposed at the reversal of loading. In the same study (Paulay and Priestley, 1993), it is reported that the properties of inelastic buckling depend not so much on the height of the wall but on its length. The present work constitutes a small part of an extensive research program that took place in the Laboratory of Reinforced Concrete and Masonry Structures of the Faculty of Engineering of A.U.Th., on the phenomenon of out-of-plane buckling and the factors that influence it.

2. RESEARCH SIGNIFICANCE

The influence of the degree of elongation of the longitudinal reinforcement of extreme regions of walls in the resistance of seismic walls against transverse instability is experimentally examined. The experimental results are based on experiments that were carried out in columns simulating the extreme zones of walls. These columns were strained to different degrees of elongation and then were submitted to compressive loading. With this way, the large sizes of tensile deformations which are applied in the plastic hinge regions of ductile walls and the compressive loading that is followed due to alternation of the sign of seismic loading are modelled. The experimental results are analyzed and important conclusions are formulated with regard to the influence of the mechanical parameter of degree of elongation in the problem of out-of-plane buckling of reinforced concrete walls.

3. EXPERIMENTAL INVESTIGATION

3.1. Aim of experimental investigation

The main objective of the experimental investigation was to ascertain the influence of the degree of tension of extreme regions of a wall in the reduction of their effective rigidity $(EI)_{eff}$ and hence in their weakening, against the risk of transverse instability, since this influence will have as a consequence the reduction of the critical buckling load, too, when the same flanges are submitted later in compression.

3.2. Test specimen characteristics

The test specimens were constructed using the scale 1:3 as a scale of construction. The dimensions of specimens are equal to 7.5x15x90 cm. The reinforcement of specimens is constituted by 6 bars with a diameter 8 mm each one of them. The total number of specimens is equal to 5. Each specimen was submitted first in uniaxial tensile loading up to a specific and preselected degree of elongation and then was strained under central compression loading. The differentiation of specimens lies in the different degree of elongation that was imposed in each one of them. The specimen characteristics are brought together in Tab. 3.1, while Fig. 3.1 and 3.2 present the cross-section and the end plan of the test specimens both for tensile and compressive loading.

Table 3.1. Specimen Characteristics

N/A	Description of specimens	Dimensions (cm)	Longitudinal reinforcement	Transverse reinforcement	Longitudinal reinforcement ratio (%)	Concrete cube resistance at 28 days (MPa)	Steel bar yield resistance (MPa)	Degree of elongation (‰)
1	Y-6Ø8-268-0-1	15x7.5x90	6Ø8	Ø4.2/3.3cm	2.68	24.89	603.77	0.00
2	Y-6Ø8-268-10-2	15x7.5x90	6Ø8	Ø4.2/3.3cm	2.68	24.89	603.77	10.00
3	Y-6Ø8-268-20-3	15x7.5x90	6Ø8	Ø4.2/3.3cm	2.68	24.89	603.77	20.00
4	Y-6Ø8-268-30-4	15x7.5x90	6Ø8	Ø4.2/3.3cm	2.68	24.89	603.77	30.00
5	Y-6Ø8-268-50-5	15x7.5x90	6Ø8	Ø4.2/3.3cm	2.68	24.89	603.77	50.00

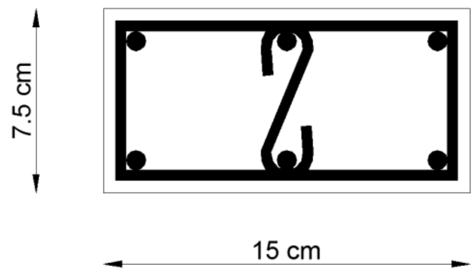

Figure 3.1. Cross-section of test specimens

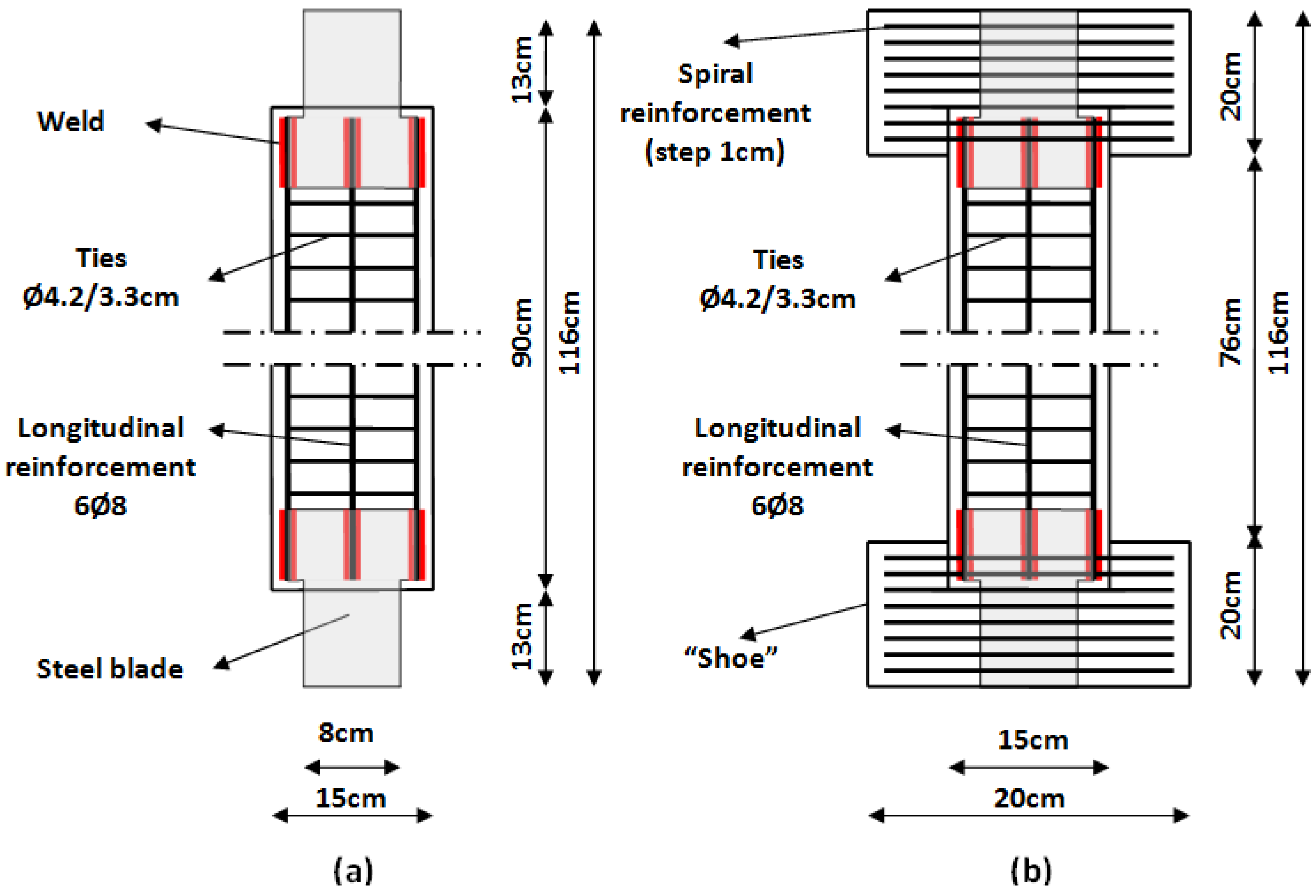

Figure 3.2. Sketch of end plan of specimens for: (a) tension, (b) compression

3.3. Loading of specimens

The experimental setups that were used to impose on the specimens the uniaxial tensile load in the first semi-cycle of loading and the concentric compression load in the second semi-cycle of loading are presented in the Fig. 3.3.

(a) (b)

Figure 3.3. Test setup for application of: (a) tensile loading, (b) compressive loading

4. EXPERIMENTAL RESULTS

Fig. 4.1 refers to the experiment of uniaxial tension and presents the change of elongation of specimens with respect to the imposed tensile load. It becomes obvious, from a simple observation of the diagram, that the actual degrees of elongation differ a little bit from the nominal degrees of elongation 10‰, 20‰, 30‰ and 50‰. However, in all cases, the differences are small and negligible. Fig. 4.2 refers to the experiment of concentric compression and presents the change of shortening with respect to the imposed compressive load this time. It becomes, easily, obvious the large drop that exists in the resistance of specimens for the cases of degrees of elongation 30‰ and 50‰. Finally, Fig. 4.3 presents the various modes of failure of specimens after the completion of compressive loading.

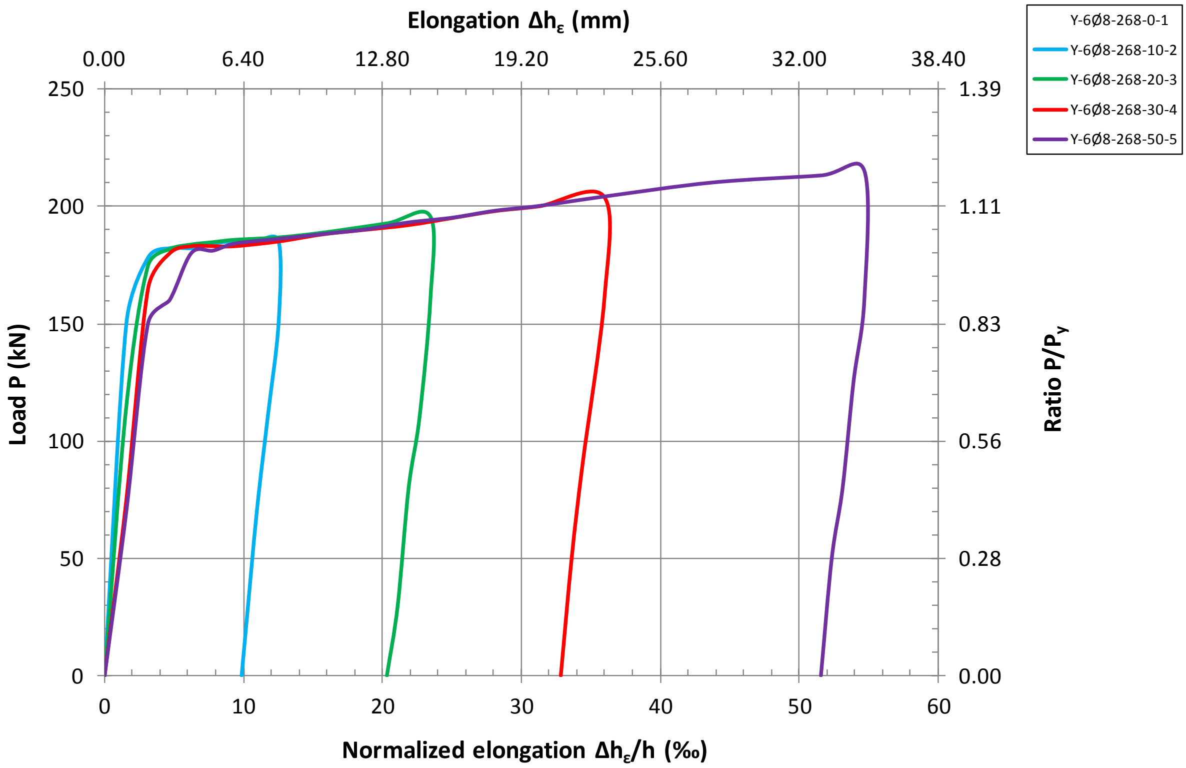

Figure 4.1. Diagram of tensile load [P(kN), P/P_y] - elongation [$\Delta h_\varepsilon/h$(‰), Δh_ε(mm)]

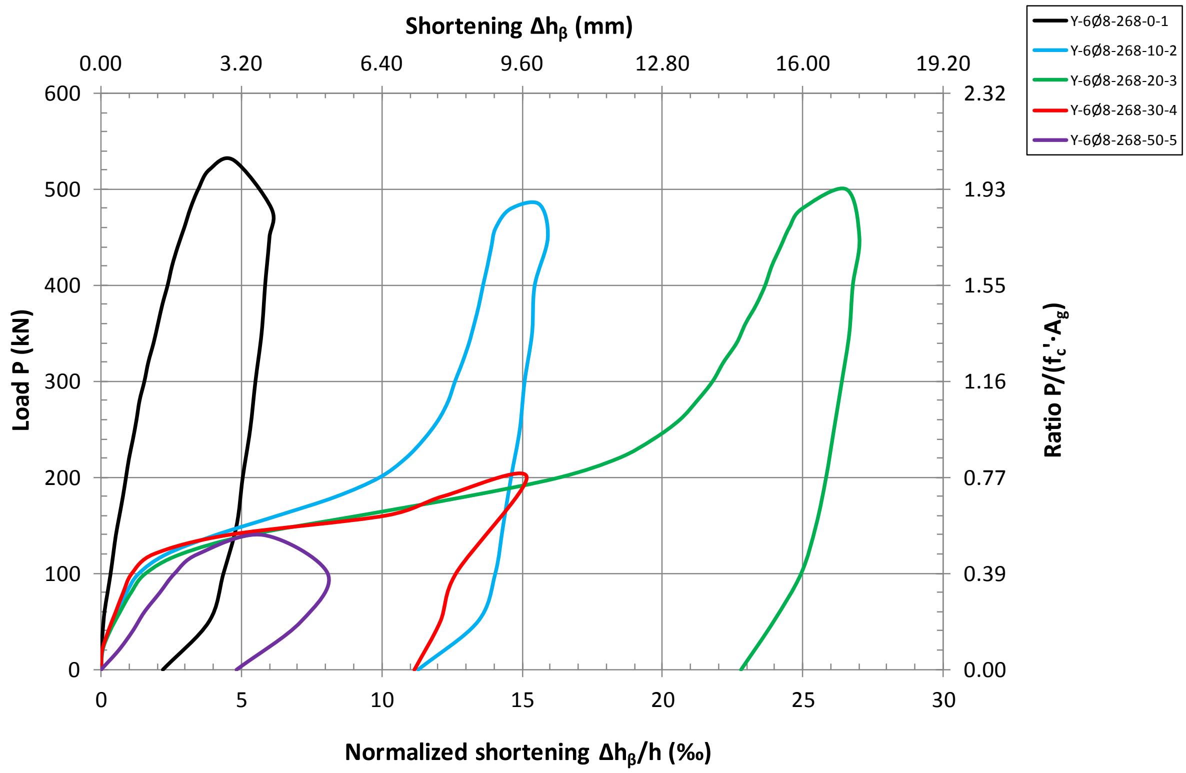

Figure 4.2. Diagram of compressive load [P(kN), $P/(f_c'\cdot A_g)$] - shortening [$\Delta h_\beta/h$(‰), Δh_β(mm)]

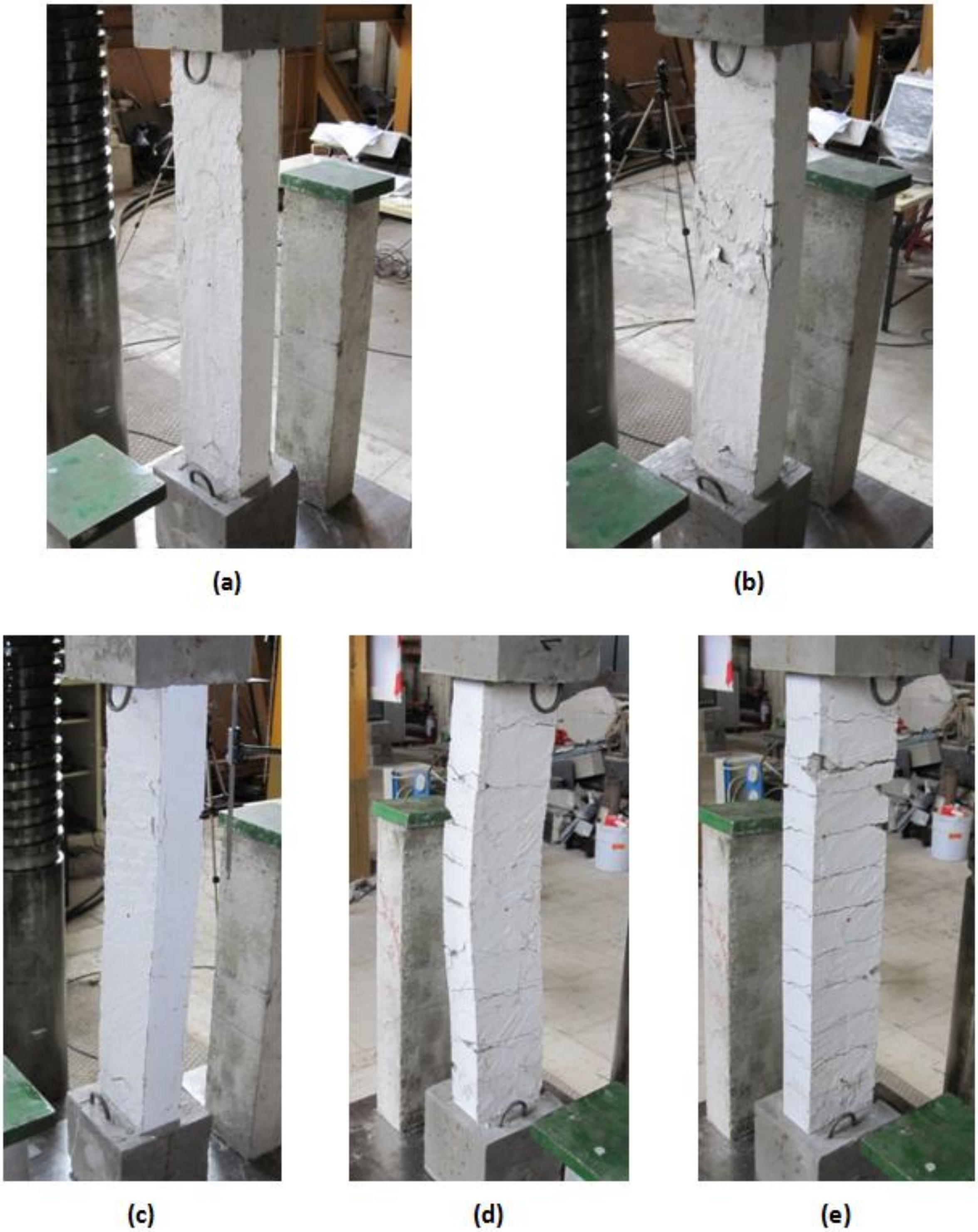

Figure 4.3. Modes of failure specimens after the experiment of compression: (a) Y-6Ø8-268-0-1, (b) Y-6Ø8-268-10-2, (c) Y-6Ø8-268-20-3, (d) Y-6Ø8-268-30-4, (e) Y-6Ø8-268-50-5

5. ANALYSIS OF RESULTS

From the conduct of experimental investigation and the evaluation and analysis of test results of specimens, observations arise with regard to the behaviour of test specimens and are the following:

1. The increase of the imposed degree of elongation in the specimens at the first semi-cycle of loading causes a change in the way of failure of test specimens at the second semi-cycle of loading, where compressive loading is applied. Specifically for degrees of elongation 0‰, 10‰ and 20‰, failure of test specimens emanates from excess of the compressive resistance of cross-section and crash of the compression zone of columns. For degrees of elongation 30‰ and 50‰, failure of specimens results from the buckling of specimens at their weak direction, that is to say the direction of their thickness. This observation becomes obvious from a simple study of Fig. 4.3.

2. Since for the degrees of elongation 10‰ and 20‰, closure of cracks is observed, this implies that for the degrees of elongation in question, large part of resistance of specimens comes from the contribution of concrete resistance. On the contrary, for the degrees of elongation 30‰ and 50‰, where closure of cracks is not observed (for degree of elongation 30‰ some closure of cracks is observed in the compression fiber of the specimen) and failure comes from buckling, the resistance of specimens comes only from the resistance of their reinforcements (or almost only from the resistance of these). Thus, this explains also the large drop in resistances of specimens that have suffered a degree of elongation equal with 30‰ and 50‰ (Fig. 4.2, Fig. 5.1 and Fig. 5.2).

3. The increase of the imposed degree of elongation in the specimens at the first semi-cycle of loading causes reduction of their maximum failure load (resistance of specimens) at the second semi-cycle of loading where the compressive loading is applied for the degrees of elongation 30‰ and 50‰. Specifically for the degree of elongation 30‰, the critical failure load of the specimen equals to the 38% of the failure load of the equivalent “virgin” specimen (Fig. 5.2). With regard to the degree of elongation 50‰, the critical load of specimen equals to the 26% of the failure load of the equivalent “virgin” specimen (Fig. 5.2). It appears, that is to say, that the increase of the degree of elongation influences considerably the resistance of specimens against transverse instability since a reduction in the resistance of specimens can take place of the order of 60-70% compared to the specimen that has not undergone any type of tensile loading (“virgin” specimen).

4. The increase of the imposed degree of elongation in the specimens at the first semi-cycle of loading does not cause an important differentiation in the value of maximum failure load at the second semi-cycle of loading, where the compressive loading is applied, for the degrees of elongation 10‰ and 20‰. Specifically for the degree of elongation 10‰, the critical failure load of the specimen equals to the 92% of the failure load of the equivalent “virgin” specimen (Fig. 5.2). With regard to the degree of elongation 20‰, the critical failure load of the specimen equals to the 94% of the failure load of the equivalent “virgin” specimen (Fig. 5.2). It is noted that for the degrees of elongation in question, closure of cracks is realized resulting in the participation of concrete to the resistance of the specimens, as it has been also said in observation No. 2. Consequently, when an increase of failure load is observed for the degree of elongation 20‰ compared to the degree of elongation 10‰, this is believed that it is owed to the instability of the material of concrete with regard to its resistance. It is stressed that conscientious efforts were made so that all specimens, as much as possible, have the same quality of concrete by concreting all specimens together using concrete from the same mixer. When a reduction of failure load is observed for the degrees of elongation 10‰ and 20‰ compared to the “virgin” specimen, it is then speculated that it is owed once again in the instability of concrete resistance or in some depreciation of the resistance of the longitudinal reinforcement bars.

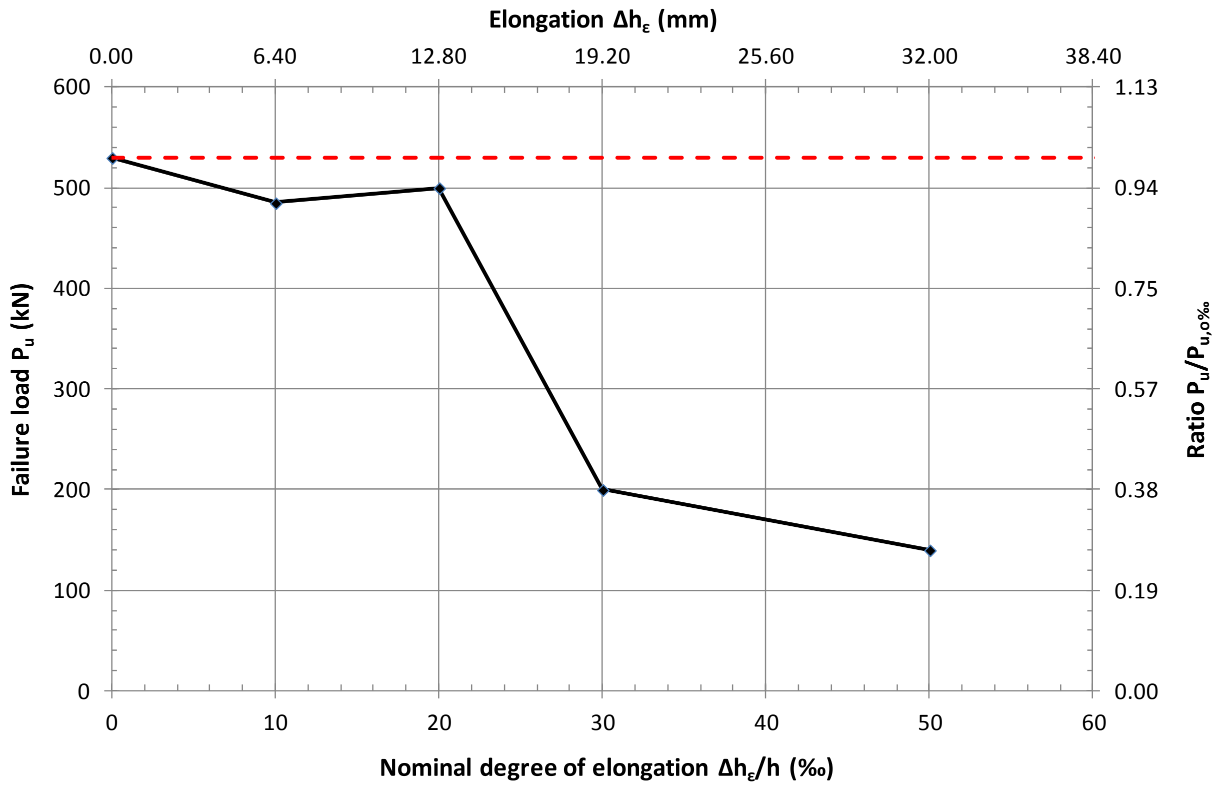

Figure 5.1. Diagram of maximum failure load [P(kN), $P_u/P_{u,0‰}$] - elongation [$\Delta h_\varepsilon/h$(‰), Δh_ε(mm)]

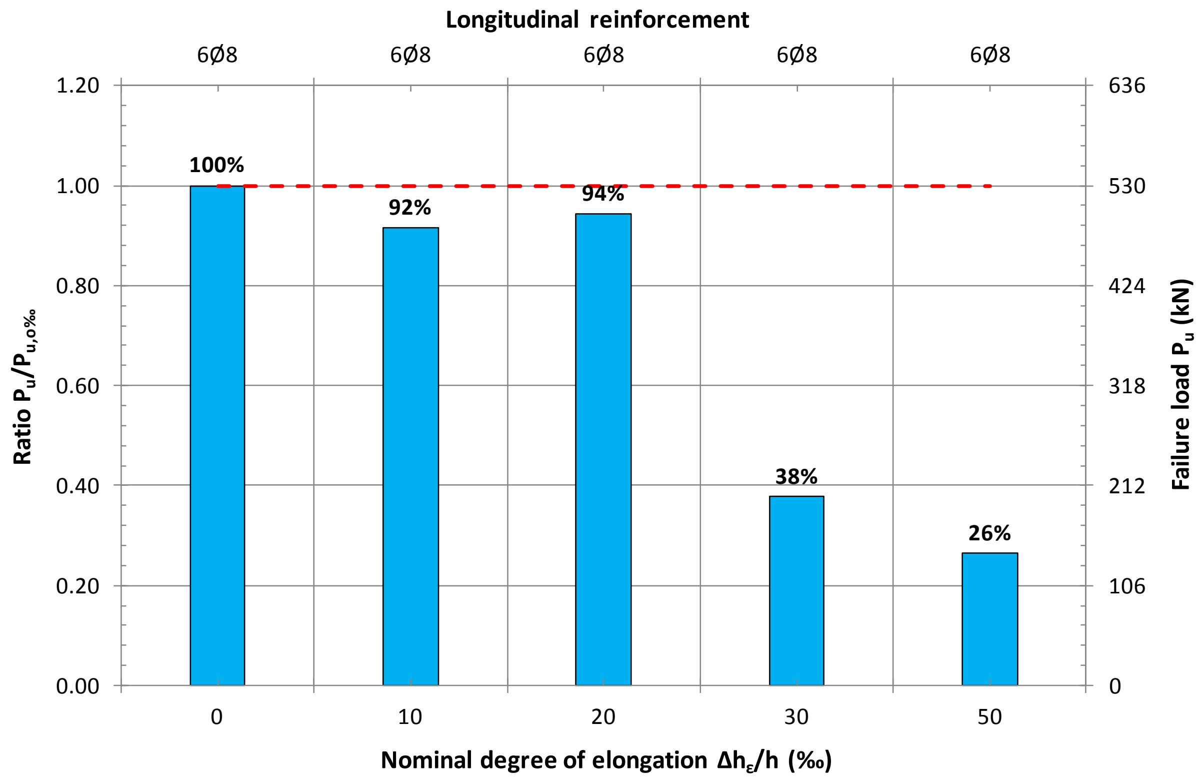

Figure 5.2. Column diagram of failure load [$P_u/P_{u,0‰}$, P(kN)] - elongation and type of longitudinal reinforcement [$\Delta h_\varepsilon/h$(‰)]

6. CONCLUSIONS

Based on the preceded test investigation and analysis and evaluation of its results, the following conclusions arise:

1. The main conclusion of the present study is that the transverse instability of walls is a complicated phenomenon, which does not depend only on the height of ground floor (as implied by the vast majority of modern international codes) but also on other mechanical parameters, as it is e.g. the degree of elongation.

2. The imposed degree of elongation at the first semi-cycle of tensile loading has catalytic influence, above a certain value, in the behaviour, in the way of failure and in the maximum failure load at the second semi-cycle of compressive loading and more generally in the phenomenon of transverse instability.

3. With regard to the minimum thickness of seismic walls, which varies depending on the applied code, important additional investigation is required and in any event, it does not constitute the unique parameter of influence in the problem of undesirable transverse instability of structural elements, which ensure the seismic shielding of multistorey constructions.

4. It is experimentally documented that there is a direct correlation between the imposed degree of elongation, the width of cracks that are created resulting from the imposed degree of elongation, and ultimately the way of response and failure of test specimens.

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