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ANALYTICAL AND EXPERIMENTAL RESEARCH ON IMPROVEMENT OF THE DUCTILITY OF BUILDINGS' R/C SEISMIC WALLS THROUGH SLIP PREVENTION WITH THE USE OF STOPPERS

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Abstract. *Several researchers propose placing diagonal reinforcing bars at the base of the wall to treat the shear slip, while others have suggested various ways to address this problem associated with the halting of the effects incurred by the through-crack in the base of the wall during recycling of loading. An indicative proposal of the bibliography is using large diameter reinforcement bars in the web of the wall as vertical reinforcements, so that through the dowel action of these bars to be able to better control the shear action. The two aforementioned proposals, while adequately address the phenomenon of shear slip, present significant disadvantages: The use of diagonal reinforcement is very difficult to be constructed, because of the density of existing reinforcement in the base of the walls, which involves compromising good concrete condensation. Also, the use of large diameter vertical reinforcement along the length of the whole wall section, including its web, leads to a strongly uneconomical solution. This works examines a solution without the aforementioned side effects. Innovation of the present work is the fact that it positions stoppers in combination with the use of conventional reinforcing bars at positions of the critical zones of the walls, in order to further prevent the expected slip along the through-crack in the base of the rigidly supported wall. The work is mainly experimental and includes investigation of the seismic mechanical properties of a wall specimen with conventional reinforcement according to EC8. This study presents the investigation on the effect of the shear span as resistance parameter, on the design of concrete interfaces. In the first part of the study the shear transfer between concrete interfaces, in which the value of the shear span is equal to zero, is investigated. The experimental investigation is extended to include values of the shear span greater than zero. These values are usually observed at bridge seismic stoppers. The experimental results presented in this study are used to derive an analytical expression of the resistance of bridge seismic stoppers.*

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1 INTRODUCTION

The structural components of low slenderness with longitudinal reinforcement and common ties exhibit great weaknesses, especially under cyclic loading of large deformations. It becomes evident that the thickening of the ties removes the risk of failure by fracturing or shearing. However, experimental investigations in the US and elsewhere [1], have demonstrated that when the axial load is tensile or has zero or low compressive value, in structural components which are subjected to shear stress of alternating sign, it is not possible to prevent the failure of in-plane shear along sections, which do not intersect with ties independent of how dense they are placed. This fact has as a consequence the rapid reduction of strength, of rigidity and of energy absorption capacity when the loading is cyclic of high intensity.

Fig. 1 gives successive states of a structural component, which is ensured against the risk of rupture using dense-placed ties. Fig. 1a shows the characteristic image of a wide flexural-shear crack. After the reversal of the sign of the shear, a moment will be required in order to close the flexural crack which has opened in the state (a). This moment is sufficient to open bending crack on the opposite side of the column, so we have the situation (b), in which the crack is wide. In this state of the structural component with the wide crack, the shear is obtained by the interlocking of the aggregates at flexural crack and the bolt actions in reinforcement. Certainly, this situation can be created for small axial loads and strong tie reinforcements even at columns.

It is known [2] that a column during an earthquake has not constant axial load. Particularly, this happens to the case of high-rise buildings, in which strong overturning moments are generated. As a result, there is a strong fluctuation of the axial load, especially for the perimeter columns.

As a consequence, it is possible in various phases of the earthquake to coexist large shear loads with small axial loads. In the same phase of the earthquake that was occurred the wide crack (b), the moment increases, the wide crack closes and a compressive zone in the end section is created, which was under tensile loading (c) in phase (a).

After several rounds of large movements for the structural element, the longitudinal reinforcement acquire large permanent extensions which broaden the wide crack and the contribution of bolt actions in sustaining shear gradually increases, as shown in case (d).

It is obvious that in this state, the structural element has lost a big part of its initial shear capacity, because the enlargement of the wide crack has removed the interlocking of the aggregates. Furthermore, shear is resisted only by the bolt action, because with the absence of compressive zone, the net mechanism cannot work and the ties are left inactive.

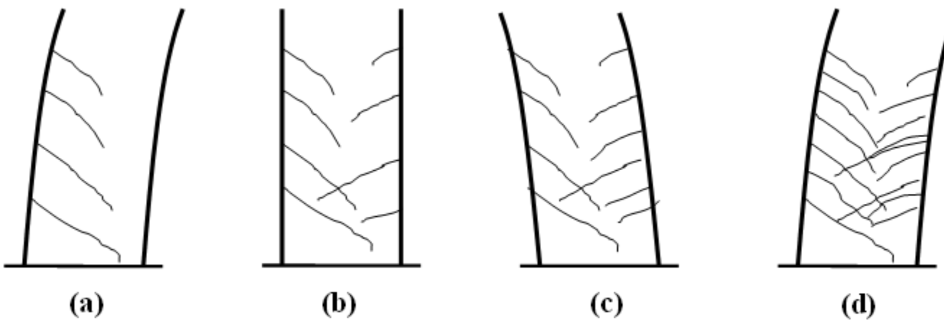

Figure 1: Destruction of plastic hinges due to cyclic loading that causes wide cracking at end sections.

Walls and frames usually used as protection systems against earthquakes in concrete buildings with usual height. The wall systems offer advantages in seismic design and their effectiveness has been tested in several occasions.

Due to their high bending resistance, a few walls in a building are enough to resist all horizontal seismic actions, simplifying and reducing section sizes of the other structural components, which can be quasi designed only for gravity loads. Surely, along with these benefits, problems concerning foundation and proper allocation of the walls in the design of spatial systems may arise.

Concerning the foundation problems, the axial load at the wall base has usually large eccentricity, due to the high overturning moment. Due to the inefficiency of individual footings, special foundations are often required, e.g. slab foundations.

Great emphasis should be given to their ductility during wall design. The walls are characterized by large inelastic deformation and by dissipation of energy of a plastic hinge formed at their base. This privilege requires high local ductility, which the usual design does not always ensure. The estimation of the real ductility requires the estimation of the plastic hinge height. The choice of this parameter is important not only for wall shaping but also because it has a direct impact on the estimation of the wall response. The estimation of the plastic hinge height is still an open problem, because the results of experiments are currently very few.

Since the early 1960s, a number of experimental tests were performed in a reduced scale shear wall of low slenderness with shear span ranging from 0.5 to 2.5. Review of these experimental tests until 1990 can be found in the references [3]. More recent experimental programs have been developed with similar elements and failure mechanisms have been studied. Experimental results on the behavior of full-scale walls are even rarer.

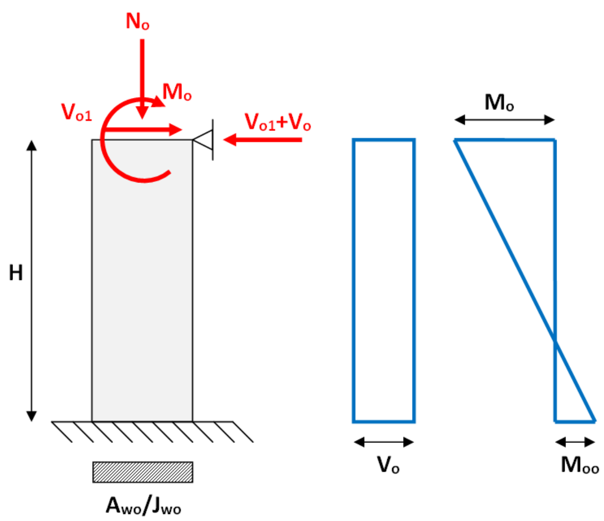

Figure 2: Stress state of internal basement shear wall under seismic loading.

All available results have shown that the inelastic response of normal walls, with values of relative shear span greater than 2, are characterized by deflections in the base of the wall. As a result, a plastic hinge is formed in the base of the wall and dominates the structural resistance and ductility.

Recent full-scale experimental tests showed that the ductility capacity depends on the capacity of the base plastic hinge to resist to the shear force for avoiding a brittle shear failure. This failure prevents the development of the full flexural deformation capacity. Moreover, Bachman stressed the need for adequate ductility of steel reinforcement to allow the inelastic deformation to extend beyond the concrete cracks and so (to allow) the development of an adequate rotation of plastic hinge.

The risk of shear slip in the base of the walls was investigated and was shown the beneficial effect of the diagonal reinforcement in the critical region. The diagonal reinforcement showed that it can be very effective in ensuring shear resistance and in eliminating or reducing the shear slip, limiting the “sting” (pinching) of hysteretic loops and strengthening the performance of walls against stresses of alternating sign in terms of seismic energy consumption.

Additionally, the influence of longitudinal and transverse reinforcement was investigated. Particularly, it is recognized that the increased axial load reduces the "pinching" and unwanted slips, while the amount of transverse reinforcement is not a determining factor in the behavior of the walls and is not sufficient to prevent premature shear failure which is derived from slipping.

The longitudinal reinforcement located primarily in the end portions of the wall section was shown that it is insufficient to control the slip, and until a level of displacement ductility around $\mu\Delta=4$, a sufficient capacity of shear resistance against the slip appears to grow from the combination of bolt action of longitudinal reinforcement and of friction to the compressive zone of concrete.

Paulay et al showed that the grid vertical reinforcement and the geometrical ratio of the sides of wall sections are critical parameters of shear slip during the inelastic seismic response, and gave a detailed description of the phenomenon under loading of alternating sign.

A research program concerning the ductility of full scale walls having large values of relative shear span took place a few years ago at the University of Brescia, in Italy. The obvious goal of the program was to achieve an experimental validation of the walls’ high ductility capacity which is a result from theoretical analyses.

In the framework of the present study, the problem of slip prevention at the base of the wall is dealt, as the title implies, experimentally and analytically, by placing the appropriate stoppers in contact on either side of the effective dimension. In particular, the following are investigated: (a) The problem of designing the stoppers and (b) the crucial question if the wide crack occurs at the base of the wall or at the faces of the stoppers. The criticality of the second problem examined is obvious, because if the wide crack takes place at the faces of these stoppers, their presence is unnecessary. The answer can only be given from the “regina probationum”, namely the experiment.

2 EXPERIMENTAL INVESTIGATION

2.1 Experimental investigation on the problem of the position of the through crack

2.1.1. Research significance

This phase of experiments seeks to draw conclusions regarding the position of the through crack. This examination takes place through the experimental investigation of mechanical behavior of a wall under alternating loading sign.

2.1.2. Test specimens (description, construction)

The present experimental program comprises a test specimen. The specimen, the scale of which is 1:2.5, has dimensions 200 x 600 x 4000 mm. It should be noted that the specimen, as a substitute of a reinforced concrete wall, corresponds to a double wall section 100 x 600 mm, since the width of 200 mm is not associated with the scale 1:2.5. The particular choice for the width of the test specimen was intended to prevent possible distortion phenomena outside the deformation plane during loading procedure. This sample represents a conventionally reinforced wall.

2.1.3. Test specimens (characteristics)

The shape and reinforcement of conventionally reinforced specimen is shown in Fig. 3.

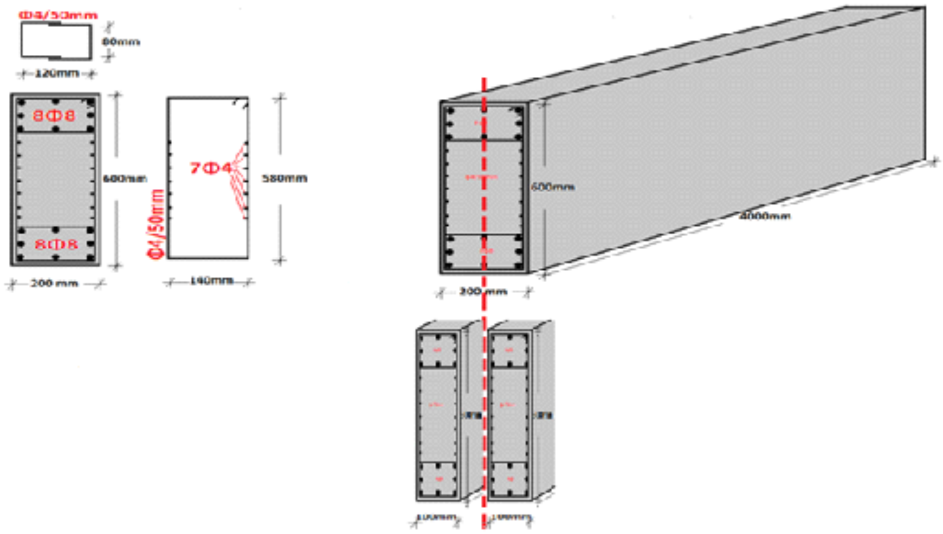

Figure 3: Shape and reinforcement of a conventionally reinforced wall.

2.1.4. Test setup

The test of mechanical behavior of this study took place using the loading machine of 6000 kN, which is located at the Laboratory of Experimental Strength of Materials and Structures at Aristotle University of Thessaloniki (Fig. 4). The application of the vertical load at the midspan of the specimens took place via a metal plate of 3cm thickness and dimensions 20 x 20 cm (Fig. 5). The measurement of deflections in the middle of the specimen took place using a digital meter having the ability to measure deflections up to 100 mm.

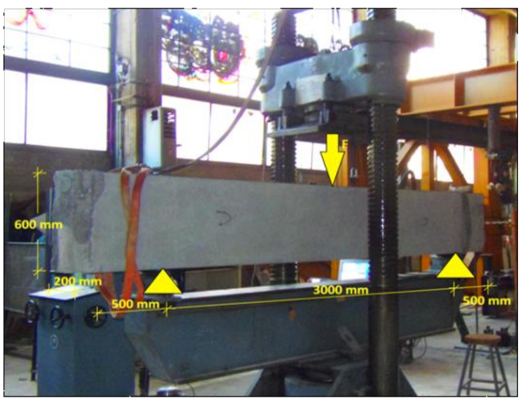

Figure 4: Loading test setup.

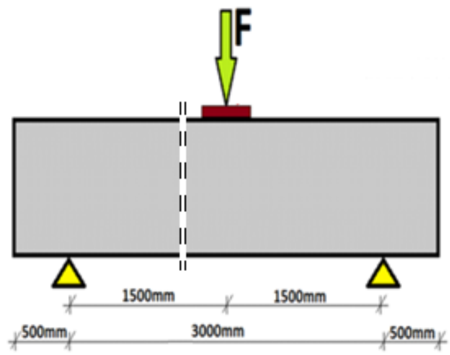

Figure 5: Sketch of loading test setup

2.1.5. Analysis of experimental results

Fig. 6 shows failure mode of specimen.

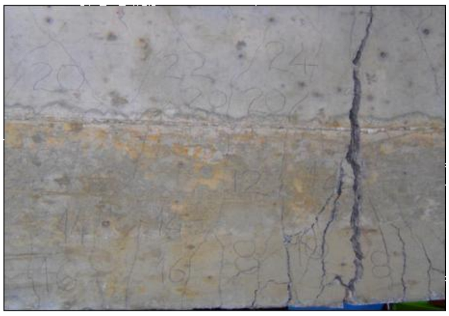

Figure 6: Cracking after failure.

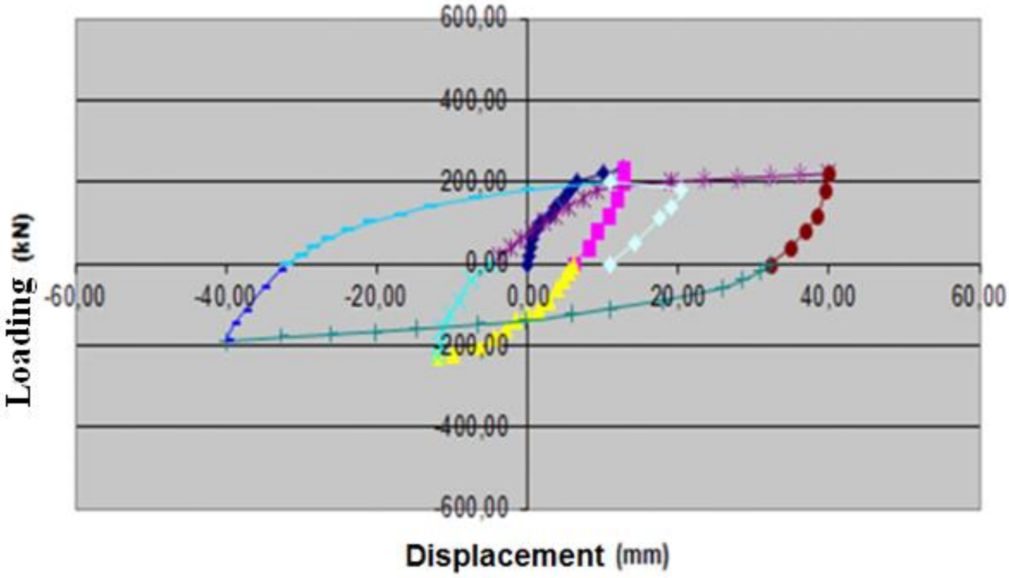

Figure 7: Load-Deflection diagram.

2.1.6. Conclusions

The main conclusions of this research phase can be formulated as follows:

1. There were very encouraging results in terms of available ductility when these stoppers are used. This is illustrated through the absence of “pinching” to the experimental hysteresis loops obtained by loading of alternating sign.
2. The wide crack at the base of this substitute wall din not occur as expected at the faces of the rigid plate interposed between the piston head and the upper flange of loadable beam, despite the presence of stoppers, which in this experiment are simulated by interposed metal plate. However, despite its strong presence, the wall ductility remained intact without "pinching".
3. Despite the non alteration of the loops and the absence of "pinch", the cause of failure was the breakage of reinforcement. This fact that justified Bachman’s concerns caused impression and shook currently existing assurances to the high ductility of steel used nowadays in Greece.

2.2 Resistance of stoppers

2.2.1. Introduction

The problem of stoppers appears mainly at bridges. Shear failure is one of the failure modes of R/C structures which mechanism is much different from the flexural one, as this type follows a formation of diagonal cracks. In general, it is a brittle failure compared with the flexural tension one. Shear failure is related to the shear span, which constitutes an important design parameter of concrete members. As it is known if the shear span-effective depth ratio (a/d) is large, diagonal tension failure occur, but when it is small, shear compression failure occurs. For the case of so-called deep beam the aforementioned ratio a/d is very small (a/d<1) the tied-arched shear resisting mechanism is formed as a compression strut jointing the loading and the support points.

The behavior of structural members of low slenderness subjected to antisymmetrical flexural distress and shear with or without axial force is characterized by two distinguished limits. The upper limit consists the case of the bending with axial force while the lower limit is the case of the interface shear transfer.

Concrete corbels are also a typical example of structural members of low slenderness. In this case the ratio of shear to span depths is often less than 1.0 [1]. The analysis and design of these corbels and other non-flexural members, like deep beams and pile caps becomes difficult with ordinary flexure theory. Different empirical approaches have been used for the design of such non-flexural members [2].

Bridge seismic stoppers [3] constitute a typical example of short corbels. These structural members are typically reinforced concrete blocks provided at the head of the piers and are used to protect bearing vulnerability mainly in the transverse direction of the bridge. However, current Codes [3] allow the use of the aforementioned seismic stoppers also in the longitudinal direction in order to reduce the seismic displacements of the deck and to prevent the unseating of the deck's beams. Bridge stoppers have usually small dimensions and usually receive the impact seismic forces of the adjacent beams.

Longitudinal seismic stoppers usually remain inactive during design earthquake [3, 4]. However in cases that they are used in combination with elastomeric bearings that have been designed according to the capacity design procedure, they should be designed by using the capacity design strengths of the pier. The aforementioned design procedure leads to large strength demands and disproportional stoppers' dimensions. The proper reinforcement of the stopper is the indicated method to maximize the strength of the stopper as the increase of the stoppers' dimensions is restricted. The dimensioning of these members usually follows the codes' provisions about short corbels in which the shear distance is between 1 and 0.5. However seismic stoppers fall in the category of ultra short corbels in which the value of the previous ratio is lower than 0.5 and for this reason the improvement of the reinforcement of these members is necessary. The aim of this phase of study is to propose proper reinforcement arrangements which maximize the resistance of the stoppers and simultaneously characterized by construction simplicity.

2.2.2. Shear span equal to zero

In the first part of this phase of study, the shear transfer between concrete interfaces in which the value of the shear span is equal to zero is investigated. The effect of the following parameters on the shear resistance of the interface is investigated: (1) Reinforcement ratio (7.5%, 2.5%, 0.5%, 0%), (2) roughness of the interface (smooth, rough and monolithic connection). Three groups of specimens had been constructed according to the roughness of the interface. The specimens are cubic (200mm x 200mm x 200mm) (Fig. 8).

The construction of the specimens includes two stages in order to achieve the creation of the construction joint. The first stage includes the construction of the middle 1/3 part of each specimen. Punched metallic plates of thickness less than 1mm were used for the hold of the reinforcement bars at the predefined position and the division of the form into three parts. A part of expanded polystyrene of thickness of 5mm was placed in the middle 1/3 of the specimen before the construction, in order to achieve the construction of the “legs” of the specimen. After the hardening of the middle part of the specimen, the metallic plates were removed and the construction of the other two parts of the specimens was followed. Between the two construction stages the careful treatment of the interfaces (e.g. use of hammer and chisel in the case of the rough interfaces) have been interposed.

The machine of uni-axial loading of the Laboratory of Experimental Strength of Materials and Structures of Aristotle University of Thessaloniki was used for the loading of the specimens (Fig. 9). A metallic plate of thickness 1cm was placed at the middle part of each specimen for the accomplishment of the uniform distribution. Each interface receives the half loading of the machine. All specimens were subjected to cycling loading. The value of the step of the loading was variable.

During the loading, measurements of the crack width as well as the relevant displacement of the interfaces were recorded (Fig. 8). These measurements were taken at the middle of the construction joint. Figs. 10, 11, 12 present the experimental results for the specimens whose reinforcement ratio was 2.5%. These specimens are considered the most representative of the investigation. The analytical strengths of the specimens calculated according to the codes provisions [5-8] are also marked at the same diagrams.

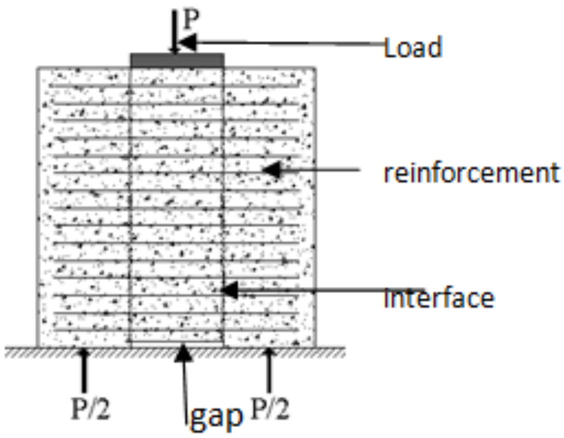

Figure 8: Loading setup.

Figure 9: Loading setup.

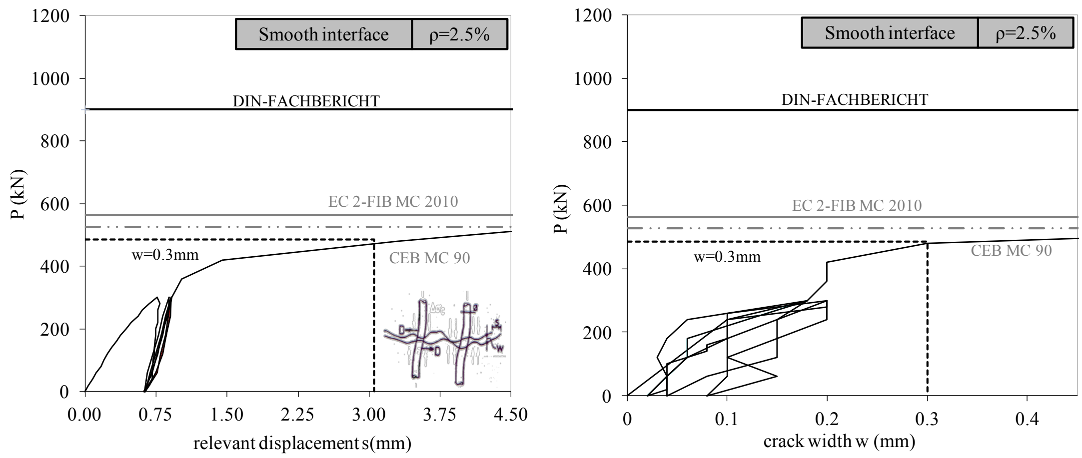

Figure 10: (a) Loading-Relevant displacement diagram, (b) Loading-crack width diagram (smooth interface, ρ=2.5%).

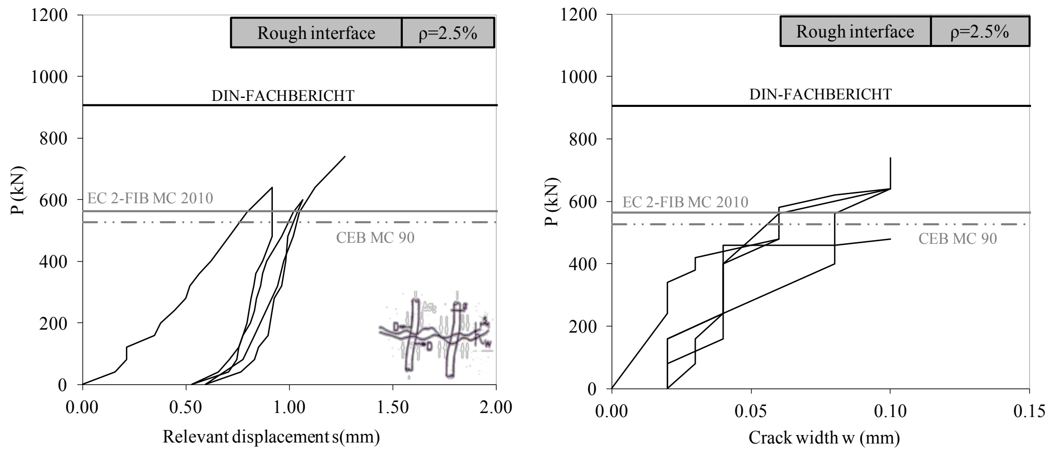

Figure 11: (a) Loading-Relevant displacement diagram, (b) Loading-crack width diagram (rough interface, ρ=2.5%).

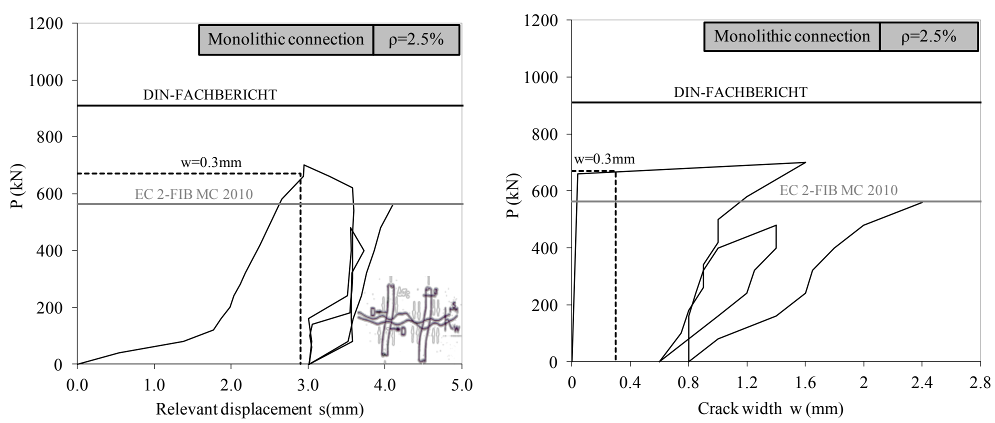

Figure 12: (a) Loading-Relevant displacement diagram, (b) Loading-crack width diagram (monolithic connection, ρ=2.5%).

2.2.3. Shear span < 0.5

The experimental investigation was extended to include values of the shear span greater than one. In this part of the study reinforcement layouts for seismic stoppers is proposed. The proposed reinforcement layout consists of opened stirrups uniformly arranged in plan. This reinforcement has the advantage of the full activation in contrary to the conventional single bars.

In order to maximize the specimens' resistance, the use of thick reinforcement bars is necessary. The bend radius of the bars in these cases should be about 15Ø. The need of increase of the number of the legs of the stirrups leads to the second reinforcement arrangement (Fig. 13b). The third complex reinforcement arrangement consists of inclined bars (Fig. 13c), and is most proper for the case of cyclic loading. The aforementioned reinforcement arrangements are analytically and experimentally investigated.

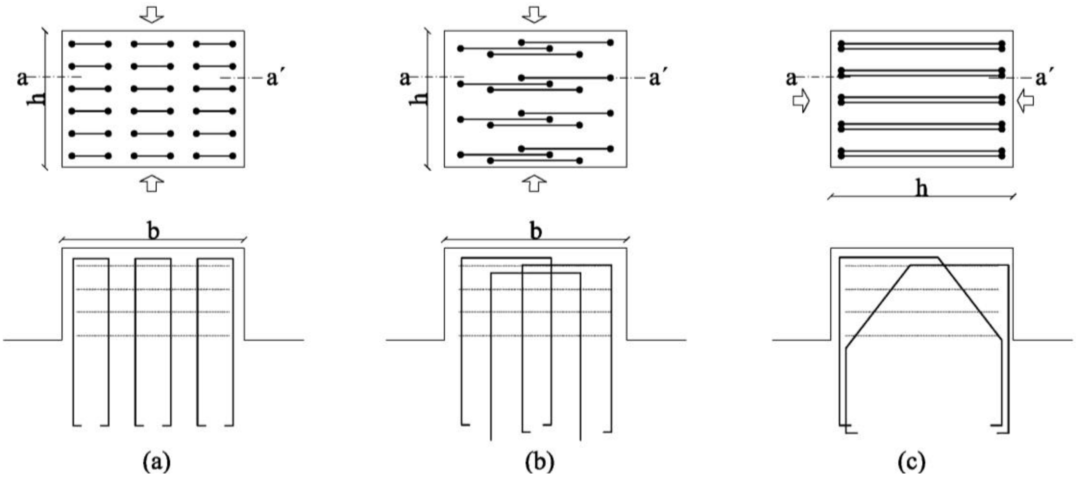

Figure 13: Proposed reinforcement layouts.

2.2.4. Experimental investigation of the proposed reinforcement layouts

The aim of the experimental investigation is the determination of the strengths of the specimens which have been dimensioned according to the proposal of this phase of study. The geometry of the specimens and the concrete class (C25/30) is the same for all of the specimens. The main parameters examined in this study are: (a) the reinforcement ratio ρ, i.e. the total area of the bars to the area of the cross-section of the specimen, (b) The diameter of the bars (Ø4mm and Ø8mm), (c) the morphology of the bars (inclined or vertical legs). Fig. 14 illustrates a typical section of the specimens as well as the loading setup. Table 1 presents the main characteristics of the constructed specimens. Figure 15 presents indicative results of the investigation.

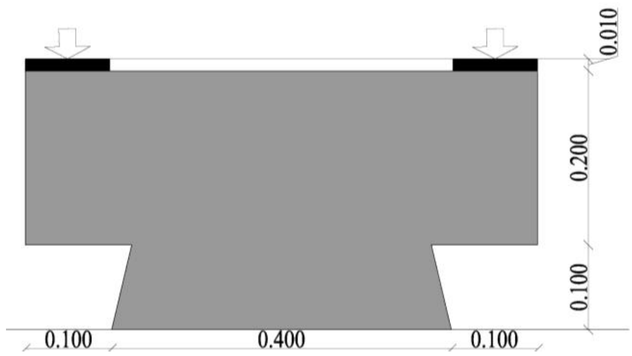

Figure 14: Loading setup.

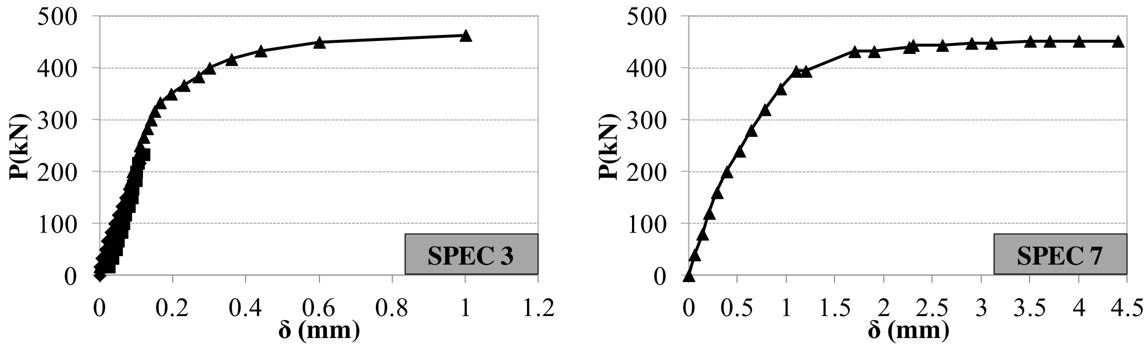

Figure 15: Load – displacement diagram for two specimens of the investigation (SPEC 3 and SPEC 7).

Specimen	Reinforcement	Layout (according to Fig. 13)
SPEC 1	Unreinforced	-
SPEC 2	12Ø4	a
SPEC 3	24Ø4	a
SPEC 4	12Ø4	b
SPEC 5	18Ø8	b
SPEC 6	12Ø8	a
SPEC 7	12Ø4+2Ø8(INCLINED)	a, c
SPEC 8	12Ø8+4Ø8(INCLINED)	a, c
SPEC 9	6Ø8	c
SPEC 10	10Ø8	c
SPEC 11	12Ø4	a

Table 1: Characteristics of the specimens.

2.3 Effective section of stoppers

2.3.1. Modes of failure

- Specimen 1: Continuous loading

Specimen 1 failed at 64 tn and presented few cracks at points where cantilever and web are jointed.

Figure 16: Failure mode (Specimen 1).

Figure 17: Failure mode (Specimen 1).

- Specimen 2: Partial loading

Specimen 2 failed at 34 tn and presented explosive failure in loading enforcement point (1/3 of length) and slight spalling at the cantilever ends.

Figure 18: Failure mode (Specimen 2).

Figure 19: Failure mode (Specimen 2).

- Specimen 3: Continuous loading

Specimen 3 failed at 44 tn and displayed a similar failure mode to specimen 1 showing slight cracking at the joint web-cantilever.

Figure 20: Failure mode (Specimen 3).

Figure 21: Failure mode (Specimen 3).

- <u>Specimen 4: Partial loading</u>

Specimen 4 failed at 26 tn presenting total destruction of the concrete at points where the web joints to the cantilever.

Figure 22: Failure mode (Specimen 4).

Figure 23: Failure mode (Specimen 4).

2.3.2. Analysis of results

The results for each specimen are given to the following Fig. 24.

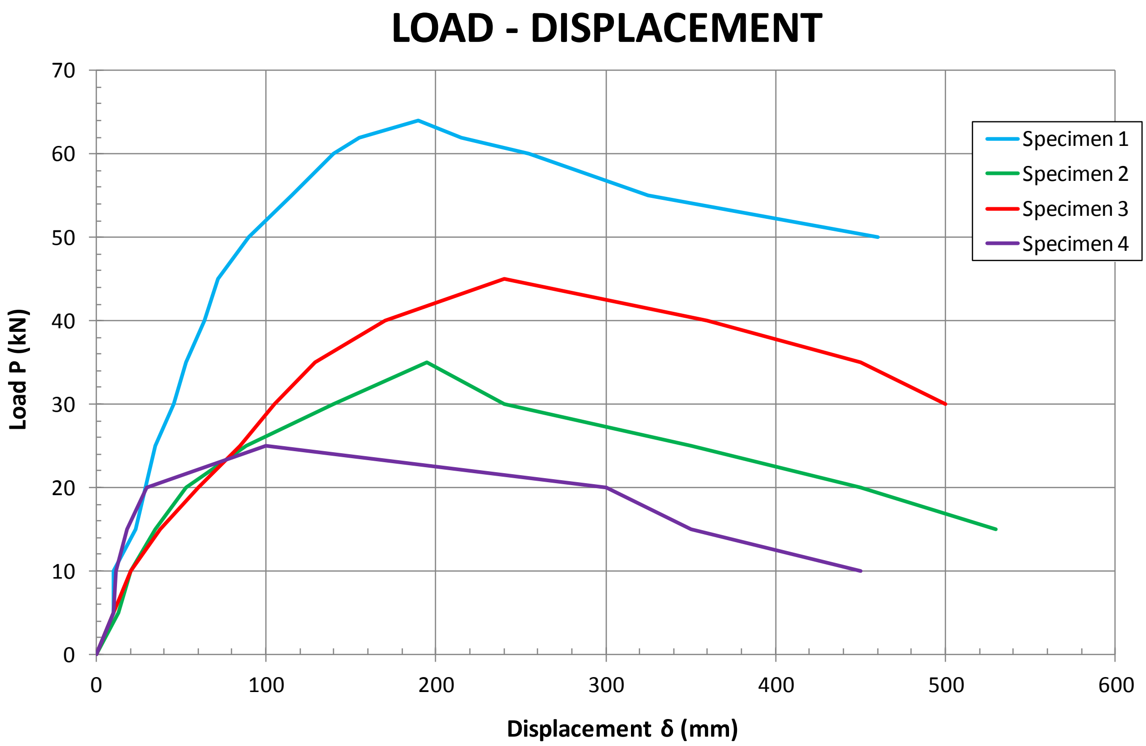

Figure 24: Load-Displacement diagram for all test specimens.

3 ANALYTICAL INVESTIGATION

3.1 Analytical investigation on the resistance of stoppers

The seismic stopper illustrated at Fig. 25 is used as the reference case of the investigation. The loading is applied across the short direction of the specimen. In this direction, the reinforcements are divided in two groups. The first group is referred to the distance h1 and the second one to the distance h_2. The reinforcements of the first group are activated when the angle θ of the compression strut takes the maximum value. The angle θ corresponding to the second group of reinforcements is smaller than the previous one. According to Eurocode 2 the cotangent of the angle θ of the compression strut must be greater than 1 but smaller than 2.5 ($22^o < \theta < 45^o$). A conservative value of the angle $\theta = 45^o$ is considered in this study.

As a result, the shear resistance of the reinforcements consists of two parts, according to Fig. 25.

$$V_1 = \frac{b}{x} \cdot \frac{h_1}{y} \cdot a_s \cdot f_{yd} \cdot \tan\theta \tag{1}$$

$$V_1 = 0.5 \cdot \frac{b}{x} \cdot \frac{h_2}{y} \cdot a_s \cdot f_{yd} \cdot \tan\theta \tag{2}$$

The sum of V1 and V2 gives the total shear resistance of the stoppers' reinforcements.

$$V_{tot} = V_1 + V_2 = A_s \cdot f_{yd} \cdot (1 - 0.5 \cdot \frac{a_v}{h} \cdot \tan\theta) \tag{3}$$

In the previous equation, A_s is the total reinforcement area of the stopper.

The shear resistance of an ultra short member with inclined reinforcement bars is calculated according to Eq. 4 (Fig. 26). In this equation θ_1 is the angle between the inclined bar and the longitudinal axis of the member. Furthermore $\tan\theta_2=0.5h/\alpha_v$.

$$V_{tot} = f_{yd} \cdot (\tan\theta_1 \cdot \sum a_{s1} + 2 \cdot \sin\theta_2 \cdot \sum a_{s2}) \tag{4}$$

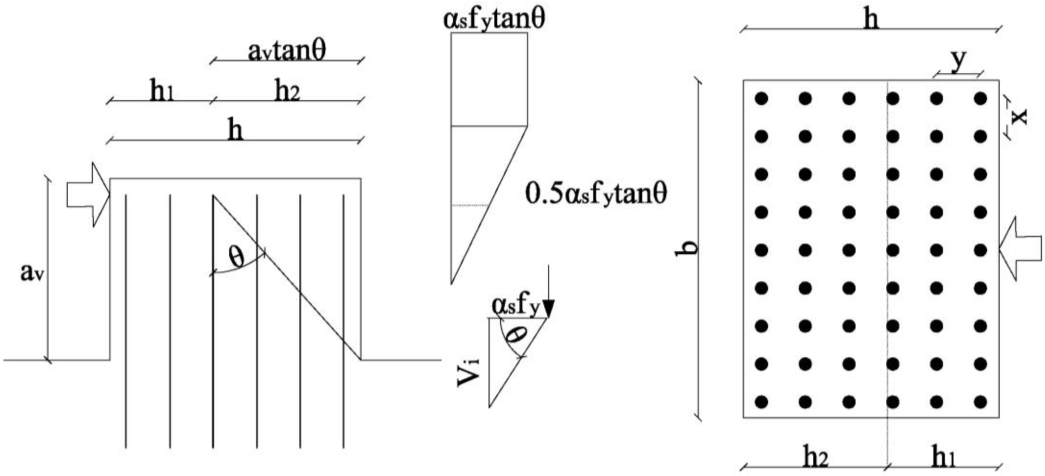

Figure 25: Corbel’s characteristics and arrangement of reinforcement in plan.

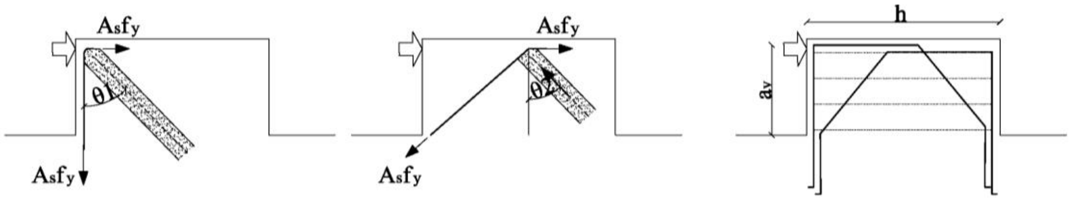

Figure 26: Features of short corbels with an arrangement of inclined reinforcement bars.

The previous equations are used for the dimensioning of the active seismic stoppers. The capacity design effects used as design action effects and an overstrength factor equal to 1.35 is used in order to determine the design shear action. The capacity design shear action is equal to 401.7 kN. Supposing that the reinforcement ratio ρ of the stopper is equal to 1.0% and by applying Eq. (3), it is derived that the required cross-section of the stopper is 0.185 m^2.

Fig. 27 presents the comparison between theoretical and experimental results.

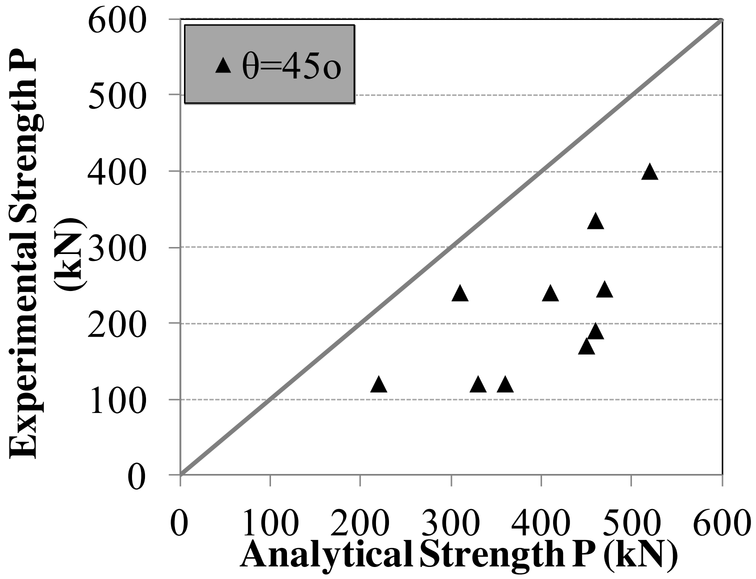

Figure 27: Comparison of analytical and experimental strengths of specimens.

3.2 Analytical investigation on the effective section of stoppers

The value of the associated effective width is function of the value of the shear span. Based on the experimental results, the following expressions for the desired effective width were obtained:

$$beff =(1 + \lambda \cdot \alpha) \cdot bo \tag{5}$$

where:

b_o : Loading width

α: Shear span

λ: Factor experimentally determined for different reinforcement arrangements (λ=1.2 for short cantilevers with 0.5 < α < 1, λ=1.6 for ultra short cantilevers with α < 0.5)

4 CONCLUSIONS

In the framework of the present study, the problem of slip prevention at the base of the wall is dealt, experimentally and analytically, by placing the appropriate stoppers in contact on either side of the effective dimension. In particular, the following were investigated: (a) The problem of designing the stoppers and (b) the crucial question if the wide crack occurs at the base of the wall or at the faces of the stoppers.

The main conclusions of the analytical and the experimental investigation regarding the position of the wide slip crack are:

1. The need for this research paper comes from the unsatisfactory until today way of addressing the likelihood of occurrence of a through-crack in the base of the construction walls. In the present context, stoppers were used to prevent slipping, which dramatically threatens the ductility of the structural member. The results were very encouraging as to the position of the wide crack, which does not take place at the faces of the stoppers used, as it was initially expected.

The experimental investigation on the effect of the shear span as resistance parameter was presented also in this study. The main conclusions of this phase of study are summarized in the following:

<u>Shear span equal to 0:</u>

1. The investigation showed that the experimental shear resistance is, in general, greater than the one calculated according to Eurocode 2 provisions, due to the upper limit of the compression of the concrete. Din Fachbericht overestimates the shear resistance due to the absence of the previous upper limit.
2. The experimental investigation proved the necessity of the presence of the upper limit of the resistance, as the strength of specimens with multiple reinforcement ratio was similar to the strength of specimens with lower reinforcement ratio.

Shear span <0.5:

1. The reinforcement layout consisting of vertical bars uniformly arranged in the cross-section of the specimen was proved to be very effective.
2. The reinforcement layout consisting of inclined bars has the advantage of the construction simplicity and is more effective for members subjected to cyclic loading.
3. The analytical strengths calculated according to the expressions proposed in this study are quite close to the experimentally calculated strengths.
4. The angle θ of the compression strut is suggested to be considered equal to 45o.

The analytical and experimental investigation on the effective width of stoppers led to the following results:

1. The price of the effective width of stoppers depends on the value of shear span.
2. A certain formula for calculating the effective width is derived.

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