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HORIZONTAL FLEXURAL TESTS ON BED JOINT REINFORCED DUPLEX CAVITY WALLS (DCW)

Josep Mª Adell[1], Carlos González Bravo[2], Felix Laheras-Merino[3]

[1]Professor, Dr. Architect. UPM. Universidad Politécnica Madrid. DCTA. Architectural Building and Technology Department. ETSAM. Escuela Técnica Superior de Arquitectura.
[2]Architect, ETSAM-DCTA-UPM. [3]Dr. Architect, ETSAM-DCTA-UPM

ABSTRACT

One of the most widely employed construction systems is the two leaf wall with inner cavity known as the Cavity Wall. This type of wall is formed by an outer leaf which makes up the facing skin of the building and protect this from wind, sun and water, while the inner leaf encloses the habitable interior spaces and the two work together to form a complete building system.

Given the form and technical nature of masonry walls in Spain it is necessary to find ways of solving the ever more common symptoms that are found in these types of construction.

In July 1996, the Department of Architectural Technology and Construction (DCTA) at Madrid Polytechnic (UPM) began tests to establish the horizontal flexural strength of masonry walls. The tests were carried out on different specimens (walls 3m long x 1m or 1.20m high).

The test findings evaluate the more relevant aspects of using horizontal bed joint reinforcement in clay cavity walls.

Key words: *ACW-Auto-supporting Cavity Wall, DCW-Duplex Cavity Wall; Bed joint reinforcement*

OBJECTIVES

The tests aimed to establish the improved bearing capacity of cavity or double walls when using Murfor truss type bed joint reinforcement.

With regards to this type of walls it should be underlined that the cavity walls consisted of two leaves which were tied by triangular reinforcement which assured their flexural resistance[4] (See Fig. 1) and which allow the transfer of loads from one leaf to another. The walls have a considerable inertia due to the spacing between the two leaves which, in turn houses the wires of the compression and tension reinforcement. The flexural resistance of the walls is approximately two times that of reinforced walls with one leaf of Perforated Clay Brick[5].

The loadings which can provoke horizontal bending in two leaf walls are that of wind action[6] in exposed walls and soil pressure in retaining walls.

In order to evaluate the horizontal bending of masonry walls a series of tests were duly carried out so that the results of the same could then be applied to brick technology and manufacture in Spain. The results of this analysis are summarised in the conclusions taken from these horizontal flexural tests.

From the tests it is possible to make a comparison between the numerical calculus and the test results which will shed some light on the ever increasing problem facing architects and builders regarding the problems of masonry enclosure walls. However, it is true that one thing leads to another as the enclosure walls are commonly employed as an interconnection within the structure of the building (housed between the floor slabs) which means that any deformation of the slab (particularly

Figure 1. DCW Wall tied with Murfor bed joint reinforcement every 36cm high (6 courses).

in the case of incorrectly dimensioned overhangs) is reflected in the walls which become load bearing and have to support the weight from the top floor down to the ground floor with the ensuing increase on each consecutive floor as a result of its own floor slab. This is then reflected in the masonry on the first floor which duly cracks as it is not capable of withstanding a load that it was never designed for.

This is a relatively common problem which may be solved by placing watertight joints between the slabs and the enclosure walls which are capable of absorbing the different deflections between one floor and another without suffering an ensuing transfer of load. If these joints could also be capable of absorbing the movement and overturning caused in seismic areas this would then allow a new form of construction in Spain.

This then makes it necessary to include the calculation of horizontal bending in masonry walls. As we have already mentioned, walls with joints between slabs may well withstand wind action by means of columns, piers or any other vertical element serving as intermediate strut.

Here the calculation process was modelled (in the case of two leaves) in the form of ties and struts. While in reinforced concrete it is necessary to "imagine" these elements, this is not necessary with the steel truss that ties the two leaves[7]. The ties and struts correspond to the simple arrangement of the truss where the most compressed bar is obtained from the shear at the supports[8].

Therefore, and in the light of the introduction of the strut and tie model in the new Spanish reinforced concrete code EHE[9], it seems logical to establish the same considerations here.

In spite of the enormous differences between reinforced concrete and reinforced masonry, in the conclusion to these tests we shall consider their possible similarities in terms of calculation.

In all walls it is necessary to seek the maximum admitted load without cracking and to verify whether this cracking is above or below that indicated in the Code NBE-AE-88.

The conclusions of this work also give a comparison of the cracking load values in the walls and the dynamic pressure values.

TEST PLAN

The walls tested were as follows:

1. Wall of PCB + Ca + PCB
2. Wall of PCB + Ca + DVB
3. Wall of PCB + Ca + LCB
4. Wall of DVB + Ca + LCB

Where:

PCB	Perforated Clay Brick
DVB	Double Void Brick
LCB	Lightweight Clay Block
Ca	Cavity or separation between two leaves

1. Wall of PCB + Ca + PCB

The wall (Fig. 2) is formed by two half brick thick leaves 300 x 108 x 11.5cm (PCB) tied by 3 Murfor NRD 5/Z-250 (diagonal 3.75mm) bed joint reinforcement set every 36 cm, and with a cavity of 6 cm (Ca = 6cm).

Two specimens of this wall were tested, denominated $11A_1$ and $11A_2$

2. Wall of PCB + Ca + DVB

The wall (Fig. 3) is foemed by a one half brick thick leaf 300 x 113 x 11.5cm (PCB) and another half brick thick leaf 300 x 113 x 7cm (DVB) tied by 3 Murfor NRD 5/Z-250 (diagonal 3.75mm) bed joint reinforcement set every 36 cm and with a cavity of 6 cm (Ca = 6cm).

Two specimens of this wall were tested, denominated $12A_1$ and $12A_2$

Figure 2. Wall 11A showing arrangement of loading and reaction points.

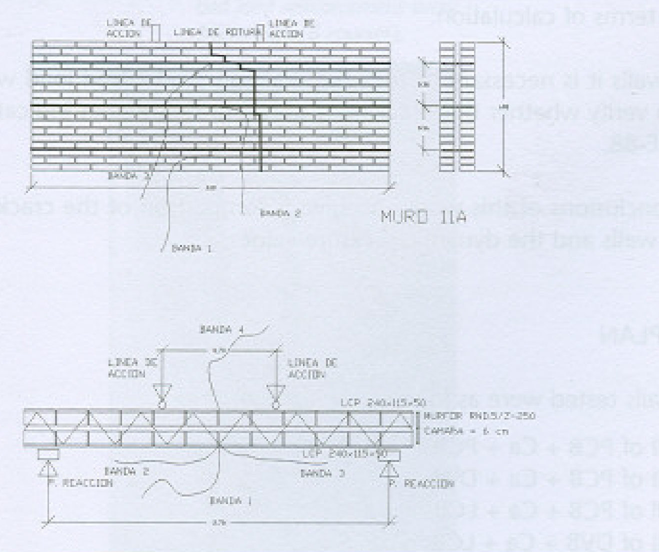


Figure 3. Wall 12A showing arrangement of loading and reaction points.

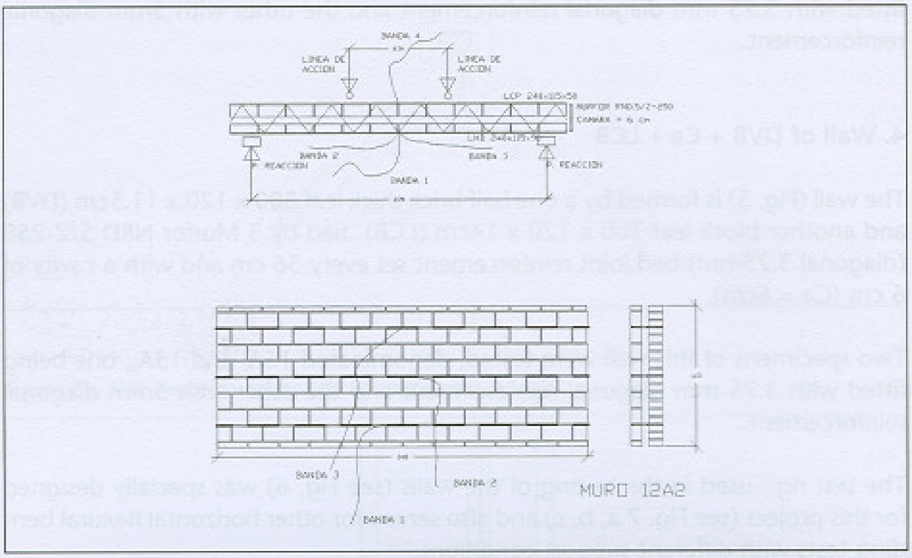
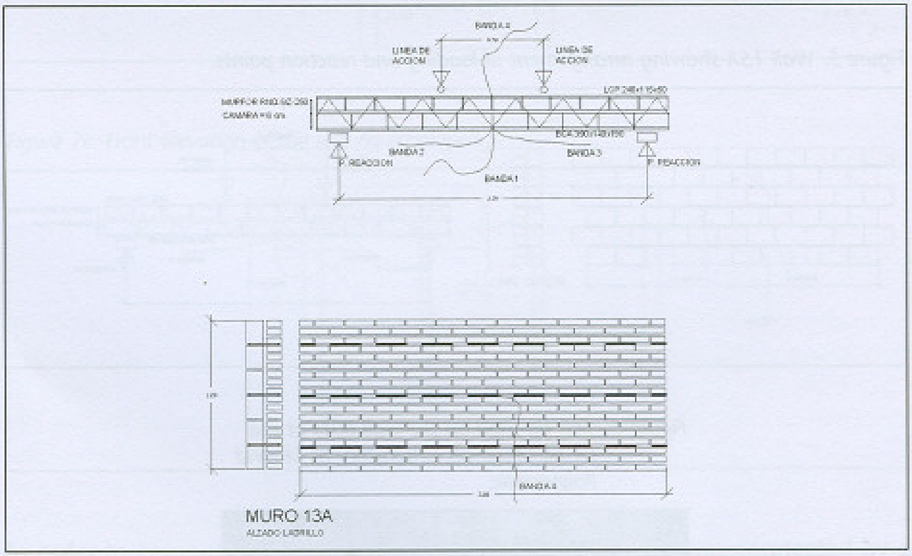


Figure 4. Wall 13A showing arrangement of loading and reaction points.



3. Wall of PCB + Ca + LCB

The wall (Fig. 4) is formed by a one half brick thick leaf 300 x 120 x 11.5cm (PCB) and another block leaf 300 x 120 x 14cm (LCB) tied by 3 Murfor NRD 5/Z-250 (diagonal 3.75mm) bed joint reinforcement set every 36 cm and with a cavity of 6 cm (Ca = 6cm).

Two specimens of this wall were tested, denominated $13A_1$ and $13A_2$, one being fitted with 3.75 mm diagonal reinforcement and the other with 5mm diagonal reinforcement.

4. Wall of DVB + Ca + LCB

The wall (Fig. 5) is formed by a one half brick thick leaf 300 x 120 x 11.5cm (DVB) and another block leaf 300 x 120 x 14cm (LCB) tied by 3 Murfor NRD 5/Z-250 (diagonal 3.75mm) bed joint reinforcement set every 36 cm and with a cavity of 6 cm (Ca = 6cm).

Two specimens of this wall were tested, denominated $15A_1$ and $15A_2$, one being fitted with 3.75 mm diagonal reinforcement and the other with 5mm diagonal reinforcement..

The test rig[10] used in the testing of the walls (see Fig. 6) was specially designed for this project (see Fig. 7.a, b, c) and also served for other horizontal flexural bending tests with different support conditions.

One of the fundamental aspects when evaluating the wall tests was the computerised instruments involved.

Figure 5. Wall 15A showing arrangement of loading and reaction points.

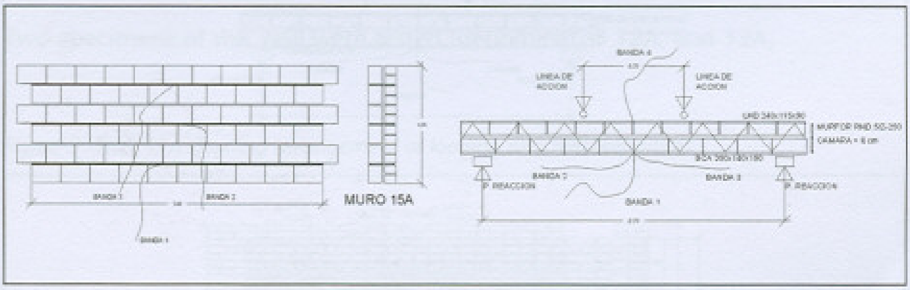


Figure 6. Test rig used on the wall tests at the DCTA Materials Laboratory at Madrid Polytechnic.

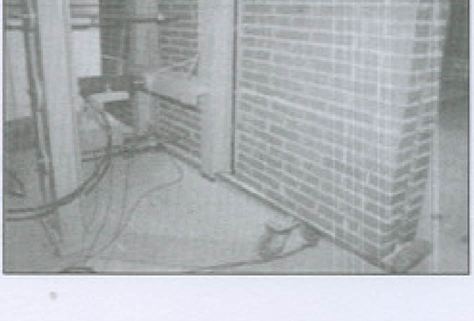


Figure 7a. Plan of the test rig indicating the distance between loading and reactions points.

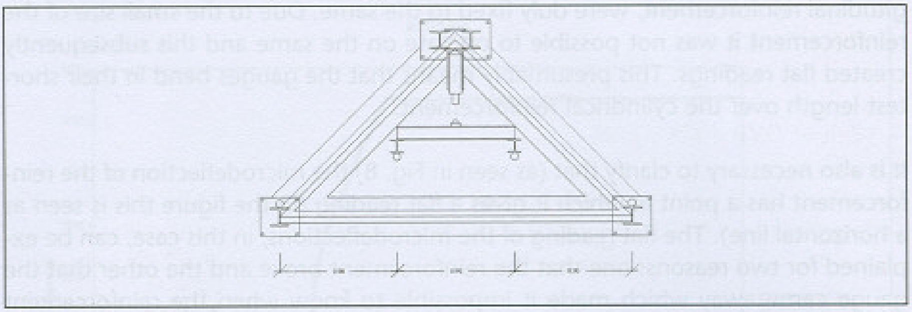


Figure 7b. Side elevation of the test rig employed.

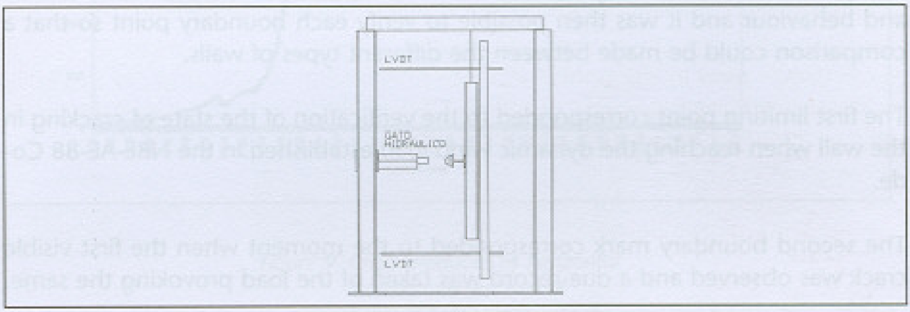
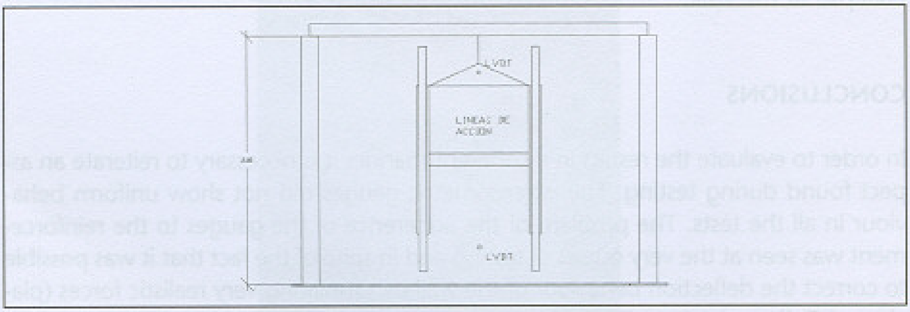


Figure 7c. Front elevation of the test rig employed.



In order to computerise the information a laptop computer was connected by a PCMCIA card to one of the transducers entrusted with converting the electrical impulses from the extensometer gauges (fixed to the reinforcement) in microdeflections which could be read in graph and table form. In this way it was possible to analyse the type of microdeflection under a specific load and, subsequently, the tension borne by the reinforcement.

The testing was optimized in this manner by trial and error in order to simulate as closely as possible the real state of forces within constructed walls in buildings.

Extensometric gauges, of a size which could be adjusted to the 4mm section longitudinal reinforcement, were duly fixed to the same. Due to the small size of the reinforcement it was not possible to operate on the same and this subsequently created flat readings. This presumably means that the gauges bend in their shortest length over the cylindrical reinforcement.

It is also necessary to clarify that (as seen in Fig. 8) the microdeflection of the reinforcement has a point in which it gives a flat reading (in the figure this is seen as a horizontal line). The flat reading of the microdeflections, in this case, can be explained for two reasons: one that the reinforcement broke and the other that the gauge came away which made it impossible to know when the reinforcement yielded and more importantly the tension under which it failed.

This problem was solved by the meticulous control of the tests in terms of time and behaviour and it was then possible to verify each boundary point so that a comparison could be made between the different types of walls.

The first limiting point corresponded to the verification of the state of cracking in the wall when reaching the dynamic wind load established in the NBE-AE-88 Code.

The second boundary mark corresponded to the moment when the first visible crack was observed and a due record was taken of the load provoking the same.

Finally, a record was made of the loads which provoked flat readings on the extensometric gauges and the time of their duration until the first indications of the collapse of the wall.

CONCLUSIONS

In order to evaluate the results in a coherent manner it is necessary to reiterate an aspect found during testing. The extensometric gauges did not show uniform behaviour in all the tests. The problem of the adherence of the gauges to the reinforcement was seen at the very outset of testing and in spite of the fact that it was possible to correct the deflection behaviour of the wall by simulating very realistic forces (placing of Teflon plates, uniformity of reaction, etc) the dimensions of the elements (reinforcement and gauge) made it impossible to ensure a more adequate performance.

In spite of this, 80% of the strain gauges worked below their breaking strain within a sufficient period of time to record the necessary data.

The test results are shown in diagrams such as that indicated in Fig. 8 where if is possible to see the response of the strain gauge and subsequently that of the reinforcement to which it was attached. There is a band at the beginning of the test in which the gauge does not record strain or does so very slowly. After this mo-

Figure 8. Diagram of the loading on a DCW wall showing the readings of the strain gauge.

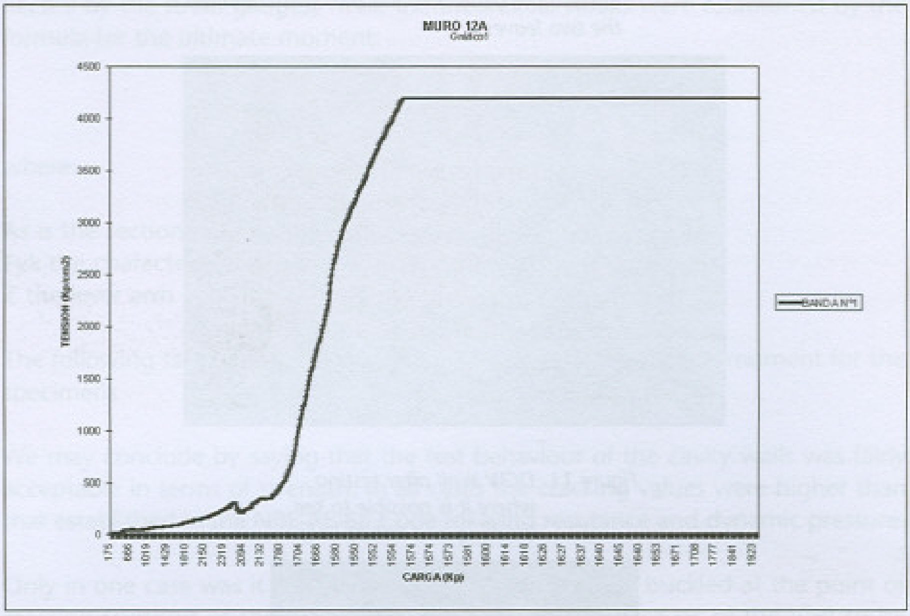
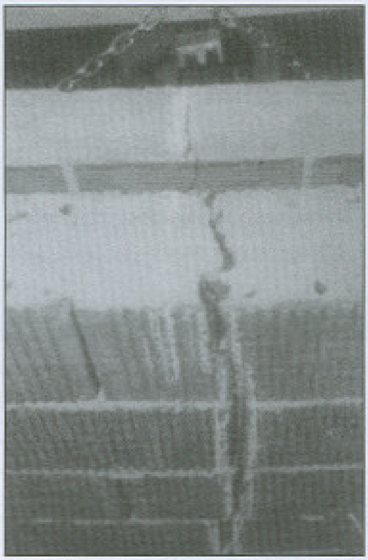


Figure 9. Crack produced by horizontal loading in a test wall of lightened clay block + cavity + Perforated clay brick.



ment a line is drawn with a certain deviation which may be compared to the stress-strain diagram of the steel. There is finally a sudden point of levelling out which reflects either the breaking strain or the moving of the gauge from the rein-

Figure 10. DCW Wall during testing, it being possible to observe the deflection and the cavity between the two leaves.

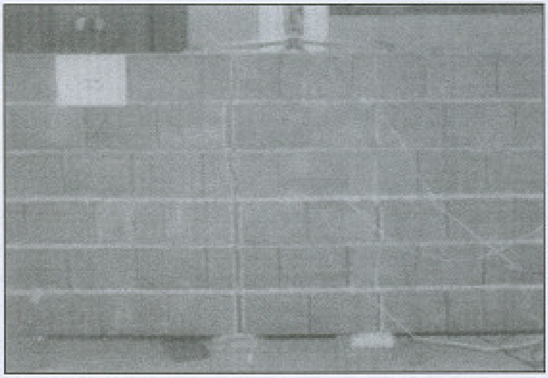


Figure 11. DCW Wall after testing where it is possible to see cracking and failure lines.

Table 1

	11A	12A	13A	15A
Test Moment (KNm)	16.75	10.03	15.51	-
Theoretical Moment (KNm)	9.97	9.97	9.97	9.97

forcement. Logically, at the point where there is no further reading, the cracking in the masonry is patently obvious (See Fig. 9, 10, 11).

After testing a comparison was made of the bending moment resisted by the test walls with that of its theoretical value.

The test moment was established on the basis of the breaking load of the wall reflected by the strain gauges, while the theoretical values were established by the formula for the ultimate moment:

$$M_u = A_s \, fyk \, z$$

where:

As is the section of reinforcement
Fyk the characteristic strength of steel
Z the lever arm

The following table shows the test moments and the theoretical moment for the specimens

We may conclude by saying that the test behaviour of the cavity walls was fairly acceptable in terms of strength. In all cases the cracking values were higher than that established in the NBE-AE-88 Code for wind resistance and dynamic pressure.

Only in one case was it seen that the reinforcement had buckled at the point of reaction (support of the wall), this having led the two leaves of the wall to be drawn together[11]

In terms of the strut-and-tie calculation method established in the new EHE code, the similarity of reinforced masonry with reinforced concrete, while not being immediate apparent, may be considered to be possible.

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NOTES

[4] In clay two leaf cavity walls, the reinforcement tying the two leaves together ensures that these work together and transfers the stresses from one leaf to another. This then assures that the structure has sufficient flexural strength by means of these tying braces and/or connecting rods.

[5] Statistic taken from the test results (walls of one and two leaves) found in the sub-project "New technical contributions of bed joint reinforced masonry in walls subject to lateral loading", within the general project "Investigation into the physical and mechanical behaviour of bed joint reinforced clay masonry walls to prevent cracking and fissures in masonry and to improve technical and architectural possibilities".

[6] In tall buildings it is necessary to verify the leeward and windward dynamic pressure in order to ensure that the masonry is capable of resisting the same. It is very common to ignore these elements in calculation as there are no current standards regarding the same in the code NBE-FL.90.

[7] We are referring to the fact that in reinforcing concrete the ties and struts are not physically seen as they normally embedded in the members and on occasions there are two or three possible means of operating (see Calavera Ruiz, J.A. A new Development in the Spanish Code EHE: The Strut and Tie Method. Intemac Quarterly No. 34-2 Quarter 1999)

[8] C=V/sin (, where V is the shear at the support of the truss for an isostatic model, C the compression of the bar and sine (the sine of the angle formed between the angled bar and the horizontal.

[9] The strut and tie method is based on the theories of MORSCH and RITTER developed at the beginning of the 20th century

[10] Adell, J.M. La fabrica armada. Ed. Munilla-lería. Madrid 2000 (pp 323-326).

[11] See Gonzalez Bravo. C/ Adell.Argilles J.M/ Del Rio, C. "The Slenderness contribution of Murfor diagonals in reinforced DCW walls tested in Spain, DCTA-UPM, 12th IBMAC. Madrid 2000.