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Research Paper

Consequences and Influences of Active Deflection in the Design of Concrete Frames with Brick Walls and Diaphanous Ground Floor

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ABSTRACT*: After a century, using concrete in construction, nowadays it is still the most complex structural material, in its congruent formulation of the analysis and its calculation. Concrete (fragile) and Steel (ductile), lay out a controversy of difficult solution that regulation try to approach to reality. The analysis of fissures and the concepts of active deflection are not enough to explain frequent phenomena of fissures in ordinary construction. This article develops and demonstrates this evidence in concrete frame structures and extracts specific conclusions on how to correct the design in frames with brick walls and diaphanous ground floor. This analysis proves an increase in active deflection not consider in current regulation.*

Keywords: *Active deflection, concrete, design, fissure, crack.*

I. INTRODUCTION

Reinforced concrete is the most complex structural material, not only in the definition of its behavior, but also in its normative setting and its employment. Nowadays is, and cannot be expected that will not be, the most employed material in building structures. Moreover, it has been from ancient times (lime-based concretes) to present times, the essential material in many kinds of constructions..Many years of experiences and mistakes have passed since the architect Le Corbusier defined concrete as the material of the future. Enough to understand how right was his guess, but also how limited. Today all his works have been or are being deeply revised.

Up to the sixties of the 20th century, concrete was calculated by permissible stress design, like any other purely elastic material, as Steel or timber. To understand the true behavior of reinforced Steel, a composite section with two very different materials has to be considered, even if its thermical dilatation coefficient is at the same time cause and benefit of its mechanical association. The concrete mass, with fragile behavior (silicon-based chemistry) and the rebar, with elastic behavior (carbon-based chemistry) present a combined action very difficult to represent according to a coherent process from a point of view of both mechanics and rigidity. From this premises, since those times, it was obvious that this reality could not be negated, and that the norms should be based on agreement, looking for applied, practical solutions.

Spanish codes EH-61 (20) y later EH-68 displayed this agreement in the form of a compressed concrete section with a rectangular plastificated block under a stress of 0,85•fck, that allowed an approach to the difficult, if not impossible to know real stress diagram in the section. The limit bending moment defined by the engineer Eduardo Torroja allowed and still allows (it is still present in the actual norm EHE-08, because of its likeness to the cracking moment) to solve the problems of designing an ordinary section with flexion. EH-68 norm and its successors EH-73, EH-77, EH-82, EH-91 followed this pattern for the design of sections in flexion without solving the disagreement between the elastic analysis of the whole structure (matrix method, Cross, Kany, …) and the section design (limit moment, cracking, …). Even today, analysis is performed following elastics models while sections design follows models more in accordance with actual knowledge and contemporary norms. Present Spanish norm EHE-08 (11), born from an update of EHE-00, has been a development in several directions.

First, existing concrete norms are merged in one (reinforced concrete, prestressed concrete, and slabs). Second, the different checks have become more complex, while at the same time, results become more accurate. At present, concrete must fulfill several needs: production and pouring conditions, quality control, fire resistance, and specially, durability. In the last years, it has been developed a growing concern about this issues, resulting in EH norms more restrictive, according to the increased knowledge of the material. Only 20 years ago, according to the norm then in use, was possible to build a cantilever in concrete with a ratio length/depth L/d=10. Today, with EHE-08, this slenderness has been reduced to 8. What has happened to the material? Is it less resistant? Is it different? Besides that, the shear resisted has also changed. Until a few years ago, an H-175 concrete could resist a shear stress of 0,5x√fcd = 5,4 Kp/cm2. Today an H-25 concrete (42,8% more resistant), according to EHE-08 only resists a Little more of 3 Kp/cm2. Loss of confidence? Acknowledgment of former mistakes? New knowledge? Taking into account of cracking conditions? Under these circumstances, building professionals must be aware of legal suits following opening of cracks in structures built with slenderness conditions corrects from the point of view of former norms, but which have produced major damages in facades and internal divisions, even if no structural failures have been recorded.

Moreover, to this picture the following norm-related events should be added:

1st. During the sixties, improvement of living conditions lead to design bigger rooms, increasing structural bays from 4 m. to 6-7 m., and the architects designed concrete frames with flat beams (with the same depth as the slab), a mechanical nonsense, as a beam cannot be flat, but that changed the slenderness of the usual design form L/d=10 to 1/22-1/28, driven by commercial issues (no beams interfering with the architectural layout) and quietly increasing beams slenderness. Steel ratios skyrocketed from 5-7 kg/m2 to 20-30 kg/ m2, with a cost that only developed countries could afford.

To this situation should be added the events below:

1st: During the 60s living standards improved, pushing housing designs to increase bays from 4 m. to 6-7 m., and architects started to design reinforced concrete frames with shallow beams, something hard to justify from a mechanical point of view, as a beam should not have the same depth as the slab it is supporting. Common designs increased slenderness from an L/h ratio of 10 to 22-28, based in commercial and economic reasons. Reinforcement ratios skyrocketed from 5-7 kg/m2 to 20-30 kg/m2, which only was affordable in developed countries.

2nd. indiscriminate use of flat slabs, without caring for deformations control.

3st. Constructions systems including partitions that are more rigid. The traditional plaster used to fix bricks was changed to cement. Nowadays, the situation has become more complicated because of new partitions materials (drywalls), and the resultant incompatibility of deformations due to the difference in rigidity of partitions and cladding, and the slab, much smaller. Crack are hence a sure think.

4th. Employment of high resistance steels, with great elastic limit, but keeping its modulus of elasticity, that assures important deformations with service loads.

5th. Because of architectural needs, suppression of partitions in the ground floor (the aim of this paper).

These causes have led to redefine acceptable deflection limits, but keeping some miscalculations because:

•Incorrect value of the concrete tangent modulus of elasticity, not considering its variation with time.

•Incorrect value of the beam’s section moment of inertia, leading to a constant beam modulus (EI), when both parameters are variable. Elasticity modulus is a function of time, and moment of inertia is a function of the cracking it each section of the beam, which depends of the bending moment and reinforcement. These conditions may vary with time, and hence the problem cannot be rigorously solved.

•Variable stiffness of partitions have complicated the incompatibility of deformations, leading to bigger difficulties in solving the calculations above.

Building experience has led to define deflection limits in building codes as variable, to represent cracking and non-elastic behavior, and to consider the different chemical constitution of concrete (silicon chemistry) and steel (carbon chemistry) and there different mechanical behaviors (1, 4, 5 and 6).

Moreover, creep of concrete and reinforcement, beyond the elastic phase should be also considered, that not only increase deflection, but also change the stiffness of the structure as time passes. Deformations linked to moisture, temperature and refraction should be taken into account, especially as cements have increased their resistance in the last years.

According to these principles, we will focus on the current situation regarding analysis and related codes, applying them to a real case that will allow us to draw conclusions to properly design reinforced concrete structures .

II. EXPERIMENTAL PROCEDURE .

2.1 Study of model

In order to show the influence of brittle partitions and their building process in building with open ground floors, we have chosen a conventional housing building with a rectangular plan, with reinforced concrete shallow beams and columns. Its structure has been analyzed with code EHE-08, considering different hypotheses to take into account different quantities of storeys (5, 7 or 9), and the bay of the studied beam (5, 6 and 7 m.) that will allow to draw comparable conclusions.

The essay will be carried on the extreme frame, which receives façade load made of brick (12 cm. width), chamber, and inner brick wall. The slab is unidirectional, with prefabricated prestressed girders, with 23 cm. of forms depth and a total depth of 27 cm. (23+4). The building has been analyzed without wind or any horizontal loads (fig. 1 and 2).

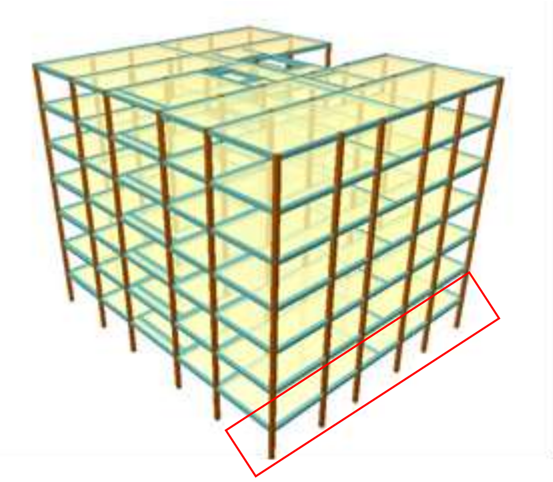

Fig.1

Fig.2

2.2 Load cases.

Load cases are shown in tables 1 y 2.

Table 1. Partitions weight (kN/m2).			
STRUCTURAL SYSTEM	*Weight kN/m2*	*Partition. Rendering on two sides*	*Partition. Rendering on one sides*
Cored brick, 45 mm depth (24,3x10,8x7,9 cm)	0,60	0,90	
Cored brick, 90 mm depth	0,90	1,20	
Rendering	0,15		
Sold brick. 12 cm.	2,16	2,36	2,36
Cement rendering	0,20		

Table 2.Loads characteristic values according to CTE DB SE-AE	
Load types	*Valor*
Live load	2,00 KN/m2
Dead load	2,00 KN/m2
Façade load	8,66 KN.ml
Partition loads	2,025 KN.ml
Slab self-weight	2.35 KN/m^2

According to results seen in built buildings, when five-storey buildings (ground floor plus four), in most of them cracking has appeared in rigid cladding at the two lowest floors (first and second). This occurrence can be seen in seven-storey (6+1) in the first three floors (first, second and third), and in the nine-storey at the first four floors (from first to fourth).

This results lead to present a more realistic hypothesis about the way the loads reach these partitions and facades where cracking appears, as they are receiving loads from upper floors and they have not been designed to withstand them, but just as partition or cladding, which leads to the fracture of the masonry, cracks opening and accumulation of loads from upper floors to lower ones.

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This hypothesis takes into account the accumulation of loads and the weight of the cracked wall that does not go directly to columns. The additional load from the cracked wall supported by the beam has been calculated for three different lengths: 5 m., 6 m. and 7 m., for a floor height of 2,9 m. and an inclination of the cracking line of 45 degrees from the intersection of the slab and the column.

For a five story building (4+1), with two floors with cracks, the analysis has been made considering that in the second floor, the beam will receive the slab loads plus the upper cracked wall, and in the first floor the slab loads plus the accumulated from the cracked walls above.

This analysis has been made for seven-storey buildings (6+1), accumulating loads from the three lower storeys, and from the four lower ones for nine-storeys, which represent the most common cases in this kind of buildings. The load model proposed is a triangular and trapezoidal load, depending on the case studied. Thus, for a 5 m. bay a triangular load shall be taken into account as shown in fig. 4(a,b), for a 6 m. bay a triangular load as shown in fig 5(a,b)., and for a 7 m. bay a trapezoidal load as shown in fig. 6(a,b). All these loads are deducted from the cracking hypothesis shown in fig 3.

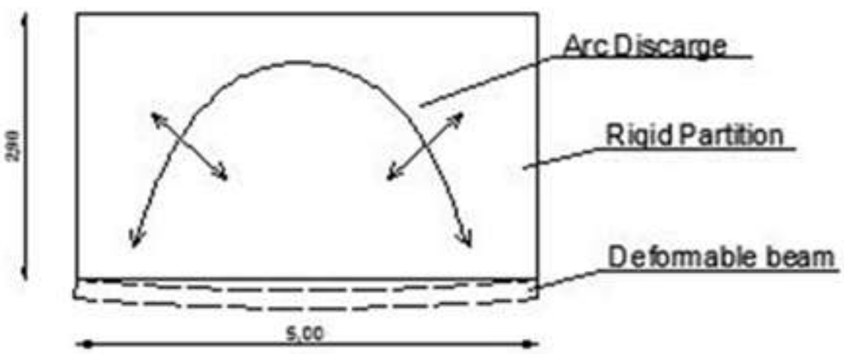

Fig.3 Arc of discharges when deforming support beam

Loads calculation
For a 5 m. bay,(fig. 4)
G1=(L x H)/2= (5m x 2,50 m.)/2= 6,25m2 x 480kg/m2= 3000 kg= 30kN

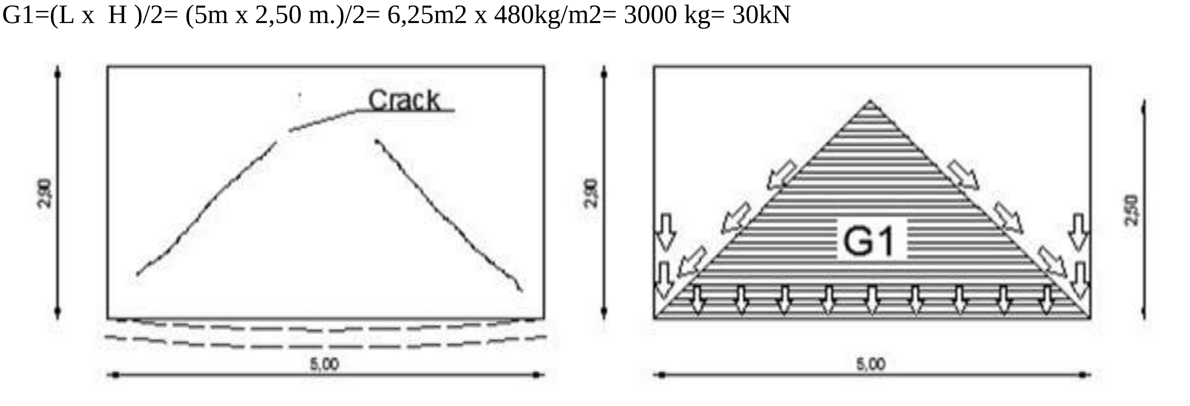

Fig.4 Cracks when deforming support beam of 5m. And creep loads.Loads calculation.
For a 6 m.(fig. 5)
G2=(L x H)/2= (6m x 2,90 m.)/2= 8,70 m2 x 480kg/m2= 4.176 kg= 41,76Kn

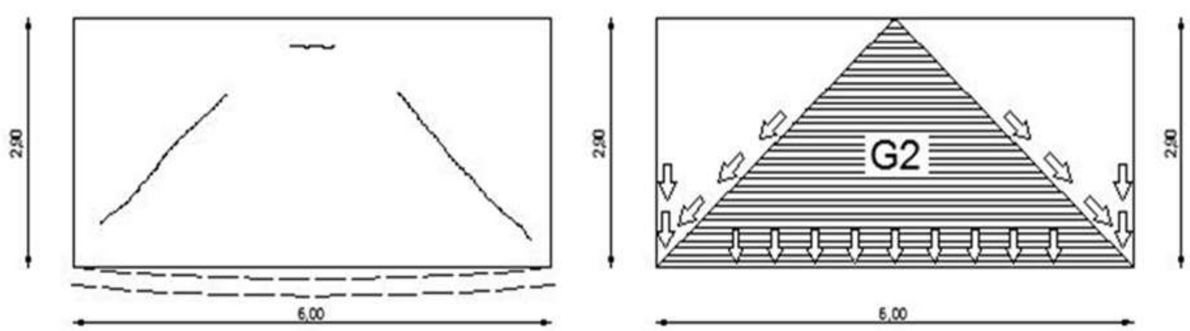

Fig.5 Cracks when deforming support beam of 6m. And creep loads.Loads calculation.

Loads calculation
For a 7m. (fig. 6)
G3=(L x H)/2= ((7m + 1,20 m.)/2)x2,90= 11,31m2 x 480kg/m2= 5.428 kg= 54 kn

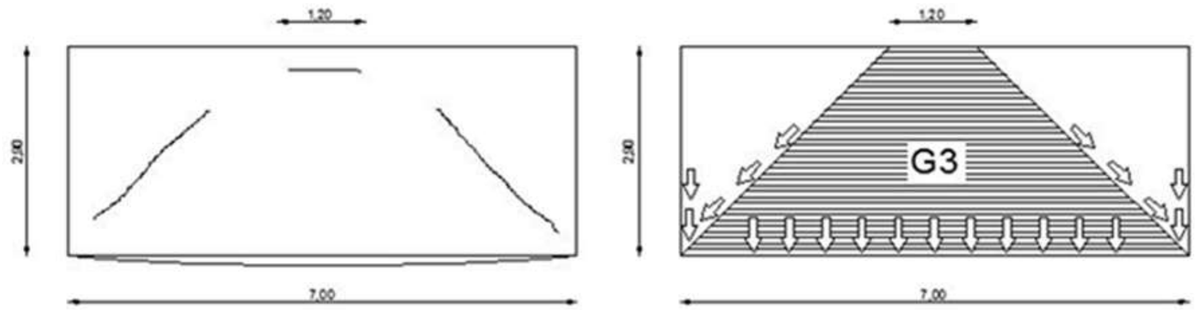

Fig.6 Cracks when deforming support beam of 7m. And creep loads.Loads calculation.

III. RESULTS AND DISCUSSION

Table 3. Deformation increases per floor.

7 Floors Residential Building			
Numbers Floors	Floors no Cracking	Two Floors Cracking	Value standard deformation
1	0.75	1.44	1.00
2	0.75	1.22	1.00
3	0.75	0.97	1.00
4	0.75	0.75	1.00
5	0.75	0.75	1.00
6	0.75	0.75	1.00
7	0.75	0.75	1.00

Table 2. Deformation increases per floor.

9 Floors Residential Building			
Numbers Floors	Floors no Cracking	Four Floors Cracking	Value standard deformation
1	0.70	1.26	1.00
2	0.70	1.25	1.00
3	0.70	1.12	1.00
4	0.70	0.97	1.00
5	0.70	0.70	1.00
6	0.70	0.70	1.00
7	0.70	0.70	1.00
8	0.70	0.70	
9	0.70	0.70	

IV. CONCLUSION

The increase in the value of the active deformation is evident when the loads arising from the accumulation of the plant-to-plant loads are taken into account, as a result of the discharge of the partition walls when cracking. This part of the loads is notorious that the greater number of fissured plants is greater the accumulation and the greater the value of the active arrow that occurs in descending direction, and of course the plant that suffers the most is the lowest or last which It has to bear not only its burdens of services and its own weight, but that which comes from the higher plants.

This observed problem of the analysis of the 5-storey building (Figure 9) is again amplified in the 7-storey model building (Figure 8), and repeatedly in the 9-storey model building, whose anomalous results Can be observed in the (Figure 7).

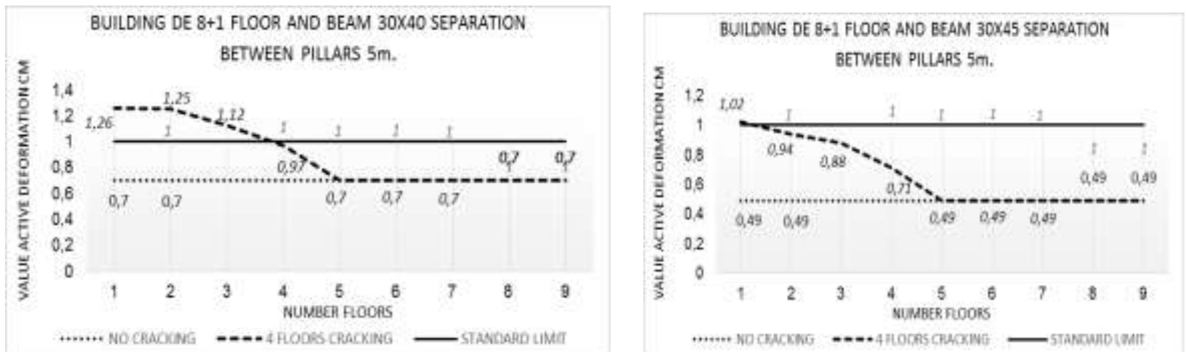

Fig. 7. Deformation for beam de 30x40, 30x45,30x50, comparation between values active deformation.(1)

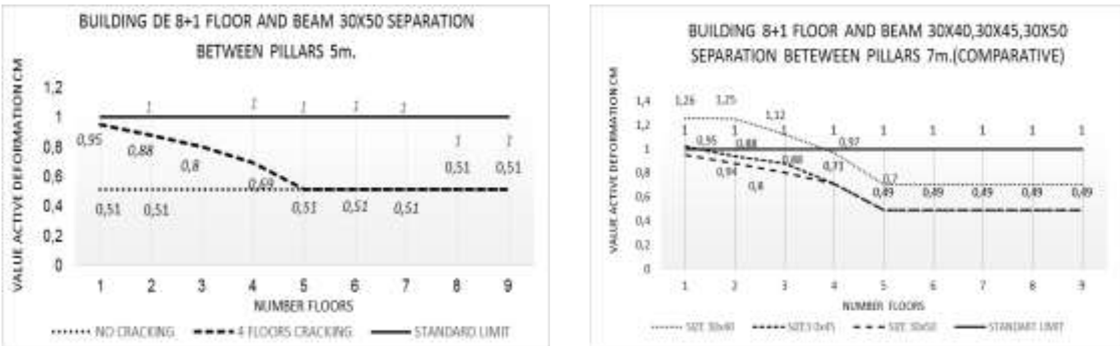

Fig. 7. Deformation for beam de 30x40, 30x45,30x50, comparation between values active deformation.(1´)

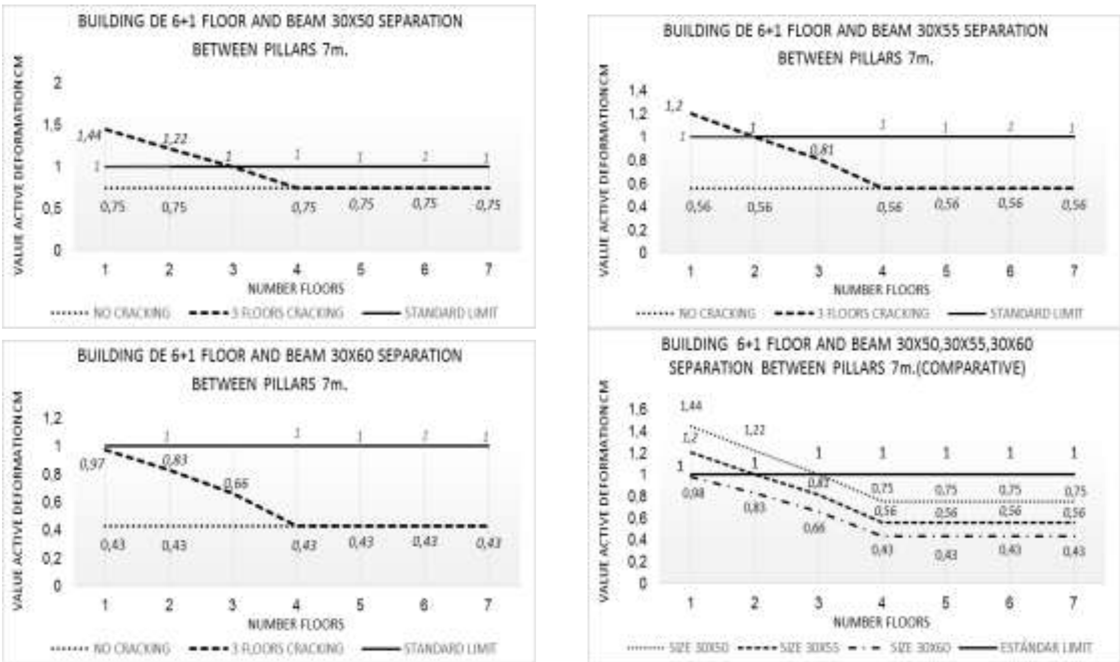

Fig.8 30x50, 30x55 and 30x60 beams deflections. Comparison between active deflection values.

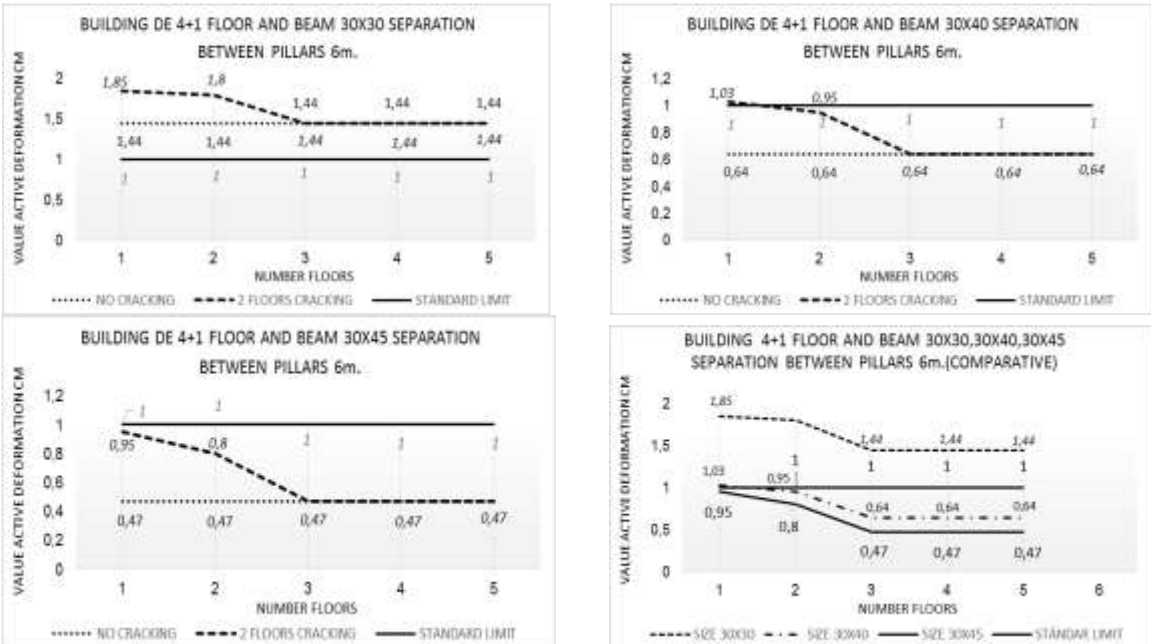

Fig.9. 30x30, 30x40 and 30x45 beams deflections. Comparison between active deflection values

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