COLLISION SHEAR WALLS TO MITIGATE SEISMIC POUNDING OF ADJACENT BUILDINGS

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ABSTRACT :

The use of collision shear walls (bumper-type), acting transversely to the side subject to pounding as a measure to minimize damage of reinforced concrete buildings in contact, is investigated using 5-story building models. Due to story height differences potential pounding in case of an earthquake will occur between floor slabs, a case specifically chosen because this is when pounding can turn out to be catastrophic. The investigation is carried out using non linear dynamic analyses for real earthquake motions and also a solution for triangular dynamic force of short duration, comparable to the forces caused by pounding. The effects of pounding are expressed in terms of changes in rotational ductility factors of the building elements and in terms of stresses in the collision walls. Results indicate that pounding will cause instantaneous acceleration pulses in the colliding buildings and will increase somewhat ductility demands in the members of the top floor, but all within tolerable limits. At the same time the collision walls will suffer repairable local damage at the points of impact, but will effectively protect both buildings from collapse, which could occur if columns were in the place of the walls.

KEYWORDS: pounding, collision shear walls, inelastic response, multistory buildings, earthquake, impact

1. INTRODUCTION

The problem of earthquake induced pounding between adjacent buildings or bridge deck segments, has received substantial attention in the past two to three decades. In many major earthquakes around the world, there have always been cases of reported damage due to pounding, ranging from light local damage to more heavy damage that might even have initiated collapse. [Bertero (1986), Rosenblueth and Meli (1986), Anagnostopoulos (1995)]. However, such damage has sometimes been exaggerated [Anagnostopoulos (1996)]. The most important such case is the reported damage in Mexico City caused by the 1985 Mexico earthquake, where according to Rosenblueth and Meli (1986) "out of a total of 330 collapsed or severely damaged multistory buildings, pounding with adjacent structures occurred in over 40% of the cases, while in 15 percent of all cases it led to collapse". This is the most cited statement in the recent bibliography on earthquake induced pounding, used by many as a justification of the importance of the problem. In this respect, the Mexico City damage is unique, because in no other earthquake has pounding been determined as a leading cause of building collapse and thus it seemed to be an outlier point that would merit further investigation [Anagnostopoulos (1995, 1996)]. In a personal communication with the second author, the following clarifications were given [Meli (1994)]: "In 15% of buildings with major damage or collapse (not only collapse) evidence of pounding was found. Not necessarily pounding was the main cause of collapse. Probably only in 20-30% of these cases pounding could have been a significant factor in the structural damage". This is obviously a much weaker statement on the effects of pounding, making the original assessment in Rosenblueth and Meli (1986) quite an overstatement In fact, the revised estimate by Meli suggests that pounding could have been a significant factor for damage in only 3% to 4.5 % of the total number of buildings that suffered serious damage or collapse. Therefore, even in this case, this percentage is quite small and becomes insignificant if one considers the total population of buildings that were subject to pounding.

To avoid this problem, modern codes require a seismic separation between adjacent buildings. Of course, this cannot affect the great numbers of buildings constructed before such requirements were introduced. Moreover, even for new construction the seismic separation requirement may not be easy to apply, as there is strong

opposition by property owners, developers and engineers for a number of economic, technical and legal reasons [Anagnostopoulos (1988, 1992)]. In addition, two other arguments are often heard against the seismic separation requirement. The first is drawn from field observations in past earthquakes, which indicate that although great numbers of buildings have been subject to pounding, only a tiny fraction suffers damage from it, while the fraction with serious damage and perhaps failure as a result of pounding alone is often negligible. This is true also in the case of the 1985 Mexico earthquake. The second argument, based both on field observations and numerical studies, is that weak buildings in contact with stronger buildings at both sides may actually benefit from such contact, provided that pounding will not cause any serious local damage that could initiate failure.
In view of the above, alternative practical solutions to the seismic separation are highly desirable and a few measures have been proposed and examined in the past [e.g. Anagnostopoulos (1988), Westermo (1989), Anagnostopoulos (1986), Spiliopoulos and Anagnostopoulos (1996), Anagnostopoulos and Karamaneas (2008)]. These include filling of a smaller separation distance with some soft material, permanent connection of the adjacent buildings, introduction of dampers or use of “collision” (“bumper”) walls. With the exception of the collision walls, all other measures are impractical, difficult or impossible to apply in most typical cases and do not solve all aspects of the problem, including legal obstacles (in the case of permanent connectors). The only practical solution eliminating the seismic separation gap is the use of collision shear walls, properly designed not only as elements for seismic resistance but also as elements to minimize the risk of collapse due to pounding.

2. COLLISION” SHEAR WALLS AS A SOLUTION TO ELIMINATE THE SEISMIC SEPARATION

One solution that was proposed to prevent seismically induced pounding while eliminating or reducing the seismic separation, was the permanent connection between the adjacent buildings [Westermo (1989)]. This, however will always penalize one of the two buildings [Spiliopoulos and Anagnostopoulos (1996)] and hence it will not be acceptable by the owner of the building that will be penalized as a result of the permanent connection. In addition it may not be easy to apply if one of the buildings is already in place. Therefore, for all practical purposes this solution is not practical, at least when the two buildings belong to different owners.
The only solution that has been proposed and applied as a practical alternative to the seismic separation and which can protect buildings from suffering heavy damage due to pounding, is the use of “collision” shear walls as it has been suggested by Anagnostopoulos (1992, 1995, 1996). This solution is shown in Figure 1, indicating a plan view of two buildings on both sides of the property line. It requires at least two “collision” shear walls to be used with their axes perpendicular to and extending up to the property line (zero seismic separation), with the remaining structure (beams, columns, infills) built 3-4 cm away from the property line. The same can be applied to the adjacent building. The solution works protecting both buildings, even if one of them does not have “collision ” walls and is already built up to the property line. This is because any potential pounding will take place at the protruding “collision ” shear walls and thus the most dangerous type of pounding that can cause shearing of columns of either building by horizontal elements (beams or slabs) of the adjacent building is avoided. This solution has been examined briefly by Spiliopoulos and Anagnostopoulos (1996) and was found acceptable. We must note here that some local damage is unavoidable with this solution. However, the most catastrophic type of failure that can occur when columns of a building are crashed between floors, pounded by a heavy slab, especially the top slab, of the adjacent building, can certainly be avoided. Moreover, the short duration acceleration spikes caused by the collisions will make necessary the anchorage of any attachment, which in any way should be anchored, even for normal earthquake action.

3. BUILDINGS USED IN THE INVESTIGATION

Two reinforced concrete buildings, named A and B, were designed according to the Greek Earthquake Resistant design and Reinforced Concrete design codes, for a horizontal PGA = 0.24g. Building A is a nearly square, 5 story frame-shear wall building, whose layout is shown in Fig. 2. Its first story height is 4.00 m and all the others are 3.00 m. Its columns are 50 × 50 cm square in the lower two stories and 40 × 40 cm in the upper three, the beams are 25 × 60 cm and all slabs are 15 cm thick. At the right side, three shear walls in the x-direction serve also as “collision” walls against the adjacent building B. Building B is also a 5 story building, but with

slightly different story heights than building A – 3.80 m (first) and 2.80 m (all others) – so that pounding would occur at points between floor levels.
In both buildings A and B, the collision walls protrude slightly (3-5 cm) from the main body of the building and thus they will be the only elements subject to pounding. The story height difference between the two buildings brings the top of building B just at the mid-height of the top story of building A, creating one of the most dangerous conditions for pounding damage during an earthquake [Bertero (1986)]. The fundamental period of building A is T_o= 0.466s and of building B T_o= 0.528s.

4. NON LINEAR DYNAMIC RESPONSE OF THE TWO BUILDINGS: EFFECTS OF POUNDING

The effects of earthquake induced pounding on the two buildings were investigated by carrying out inelastic analyses for 10 sec of the El-Centro 1940(NS) record, 15 sec of the Taft 1952(S96E) record and 10 sec of the Eureka 1962 (N79E) record – all scaled to the same PGA =0.24g. Since in each building, the plane frames along their x-axes are identical and moreover the x-axes are axes of symmetry, it suffices to analyze only one plane frame per building using 1/3 of the masses, so that the building periods in the x direction remain the same. Figure 3 shows the two adjacent plane frames analyzed.
The analyses were carried out with the well known program DRAIN-2DX, using the plastic hinge model for beams and columns. To simulate pounding, special spring-dashpot elements have been used only in the upper half of the buildings, as indicated in Fig.3, given that in the lower stories the number of collisions and their effects are drastically reduced [Anagnostopoulos (1995)]. The stiffness of these elements, estimated from the local slab-beam stiffness, was set equal to K_{imp}= 1392096 kN/m and the dashpot constant was set equal to C_{imp} = 3192 kN/m/s, corresponding to a coefficient of restitution r = 0.65, according to Anagnostopoulos (2004). The severity of inelastic response is assessed in terms of the rotational ductility factor, defined as:

$$\mu_{\vartheta} = 1 + \left(\frac{\theta_p}{\theta_y}\right) \tag{4.1}$$

where θp is the maximum plastic hinge rotation at the ends of a member (beam or column) and θy is a normalizing "yield" rotation, typically set equal to θy = Myl/6EI.
Initially, the two buildings were analyzed as independent structures, i.e. without pounding and subsequently the analysis was carried out for the model of Fig. 3, considering pounding. A slight decrease in the peak displacement of both buildings was observed as a result of pounding. Figures 4 and 5 show the effects of pounding on ductility demands of beams and columns of the two building and their distribution with height. These are average values of the peaks from the three quakes. Before assessing these results, we must note that this idealization cannot model the local damage at the points of impact and thus the numbers given for the members with joints subject to impact are not reliable. Reliable results for such members are given in the next chapter, where a detailed finite element model was used for shear walls. However, for members not having joints subject to impact, these results are reliable. We observe that the effects of pounding on rotational ductility demands are practically limited in the beams, i.e. the members designed to be ductile. These increases are on the average below ~25%, while in the columns they are negligible. Given that the values of the increased ductility demands are not very high, it may be concluded that any damage away from the points of impact can be tolerated and therefore use of the "collision" shear walls could be an acceptable solution, provided of course that the local damage is also tolerable. This is examined in the next chapter.

5. LOCAL DAMAGE DUE TO POUNDING

With the exception, perhaps, of Karayannis et all (2005), local damage as a result of pounding has not been investigated before. If however "bumper" shear walls are to be recommended as an alternative to the seismic gap, their local damage due to pounding needs to be examined. This was done by means of the well known computer program ANSYS.

5.1 Simulation of pounding with an impact force

To carry out the analyses with the detailed ANSYS model, an impact force was estimated and applied at the midheight of the top story of the “collision” wall of frame A. The estimate of this force was made as follows. The impact forces during pounding are a series of nearly triangular spikes of very short duration [Anagnostopoulos and Karamaneas (2008)]. The duration is inversely proportional to the impact element stiffness while the ordinate of the spike increase proportionally, so that the total impulse remains practically constant. Figure 6 shows the sequence of collision forces at the top story of buildings A and B due to the El Centro record
Assuming that a mass m impacts on a shear wall with a velocity v_o and that after impact its velocity is zero, we can write for the impact force F_i:

$$\int_i^{\Delta t} F_i dt = mv_o(1+e) \qquad (5.1.1)$$

where e = coefficient of restitution. If we further assume that the impact force has a triangular shape with peak value $F_{i\text{-}max}$, it follows that $F_{i\text{-}max} = 2mv_o/2\Delta t$. Typical values of v_o can be estimated from the code design spectrum, using, e.g., respresentative periods T=0.4 s, T=0.6 s and T=0.8 s that correspond to the three spectral regions. Then for a PGA=0.24g we obtain S_v= 0.375 m/s, 0.56 m/s and 0.6 m/s, for the three periods, respectively. On the basis of this range of values, which represent potential impact velocities during pounding, we assumed a value of V_o = 0.60 m/s corresponding to the design acceleration 0.24g for a period T=0.8s. If we further assume a floor mass of ~ 120t, a coefficient of restitution e = 0.65 and an impact duration of 0.02 s, we obtain a peak impact force Fmax = 11880 kN. Considering that there must be at least two “collision” walls to receive the impact, the estimate of the impact force per wall is ~ 6000 kN, for an impact duration of 0.02s. This impact force, shown in Figure 7, is quite conservative compared to the force values determined from the time history analyses with the 3 earthquake motions. It is applied at midheight of the top story of the shear wall of the plane frame representing building A, as shown in Figure 8.

5.2. Model used in the analysis

The model used for this analysis is one of the x frames of building A, shown in Fig. 8. The shear wall subject to collisions was discretized using 2D quadrilateral elements, with a dense mesh around the point of impact and a gradually coarser mesh away from it. Concrete behavior was modeled as elastoplastic with different strengths in tension and compression and with application of the Drucker-Prager yield criterion. Concrete quality was assumed to be C20/25 having a compressive strength 20MPa and a tensile strength 2.2 MPa. The reinforcing steel was assumed to be S500, with a yield point of 500 MPa, and was modeled by one dimensional bilinear elements with a strain hardening ratio 3%. The longitudinal reinforcement was lumped at five locations in lines parallel to the wall axis, while the transverse shear reinforcement was lumped along the lines forming the quadrilateral concrete elements. Due to the heavy computational requirements and considering that the columns remain elastic, as it will be seen below, all structural members except the collision wall were modeled for the local damage analysis (ANSYS program) as prismatic, linearly elastic. This approximation is considered to have little effect on the computed stresses in the wall. In addition to this, a separate inelastic analysis was carried out for the triangular impact loading, using the DRAIN-2DX program that allowed ductility factor computation for beams and columns. The model for this analysis is the same model used for the seismic response analyses described in the previous chapter.
The triangular load of Figure 7, simulating now one of the strongest possible impacts due to pounding, was applied as a distributed force in a length of 0.50m, with its center at the midheight of the top story of the “collision” wall. With a force duration of 0.02 s , the analysis was carried out for 2.5 s, i.e for a little over five cycles of free vibration of the frame after the force was applied.
Results from the two analyses are given in Figures 9, 10 and 11. Figure 9 gives a comparison of the displacement of a joint at the top middle point of the shear wall, as computed with the ANSYS and the DRAIN-2DX analyses for the triangular loading. The agreement is quite good indicating that the inelasticity of the frame

members, ignored in the ANSYS analysis, is not important. Figure 10 shows the equivalent stress distribution in the part of the wall where the impact takes place, computed with the Drucker-Prager yield criterion at time 0.01 sec, i.e the instant the impact force is maximum. The corresponding ratios of maximum equivalent stress to the equivalent yield stress parameter range from 1.50 in the tension zone to 2.56 in the compression zone, indicating that there is failure of concrete in the areas where the maximum equivalent stresses developed [Karamaneas (2000)]. The concrete failure in the compression zone, which is of primary interest, extends essentially over an area of application of the impact force (0.5m) and in a depth of about 0.40m [Karamaneas (2000)]. In addition, yielding of reinforcement was found at both sides of the shear wall around the area of impact [Karamaneas (2000)]. Away from the area of impact, the results are given from the DRAIN-2DX analysis (also with a triangular load at the top story wall midheight) as member rotational ductility factors shown in Figure 11. These values are within the range expected from earthquake analyses and comparable to the results derived from the pounding analysis of the two buildings under the El Centro earthquake record (compare with Figure 5). These results suggest that permitting pounding to occur at “collision” walls, limits the damage locally at the points of impact, eliminates the danger of shearing vulnerable columns and does not create any significant penalty in the remaining structure away from the points of collisions.

6. CONCLUSIONS

Based on the results presented herein and subject to the limitations of the underlying assumptions, it may be concluded that using well designed shear walls as “collision “ walls is an attractive and viable alternative to the seismic separation requirement between adjacent buildings that modern codes require. The advantages of this solution are:

1. It can minimize and practically eliminate the seismic separation gap and all its disadvantages.
2. It can protect both buildings, even if one is already built up to the property line and does not have “collision” walls, from shearing of their columns by the impacting horizontal slabs of the other building. This is by far the greatest danger posed by earthquake induced pounding.
3. Being part of the earthquake resisting system, it appears that the “collision” shear walls could survive the pounding by suffering only local and repairable damage.
4. Away from the points of impact, the effects of pounding do not appear to pose any significant threat to the other structural members.
5. The impacts at the “collision” walls generate high, short duration, acceleration spikes that may cause non-structural damage, if no provisions are made for the building contents. Such provisions , however, will not be different from those required to protect building contents from earthquakes even without pounding.

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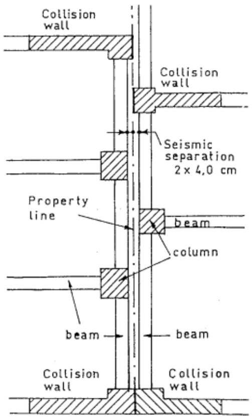

Figure 1. Collision walls configuration at the property line.

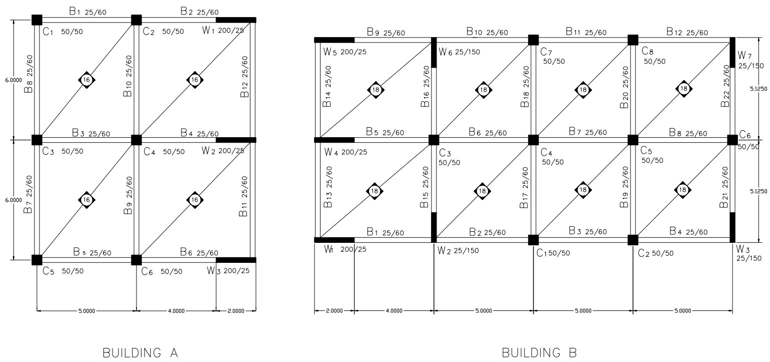

Figure 2. Plan View of buildings A and B.

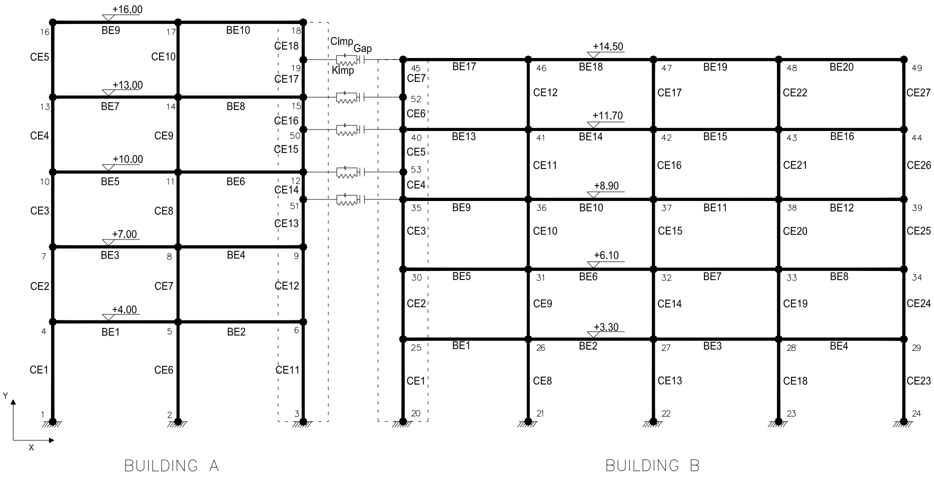

Figure 3. 2d model for the analysis of the collisions between buildings A and B with five points of contact.

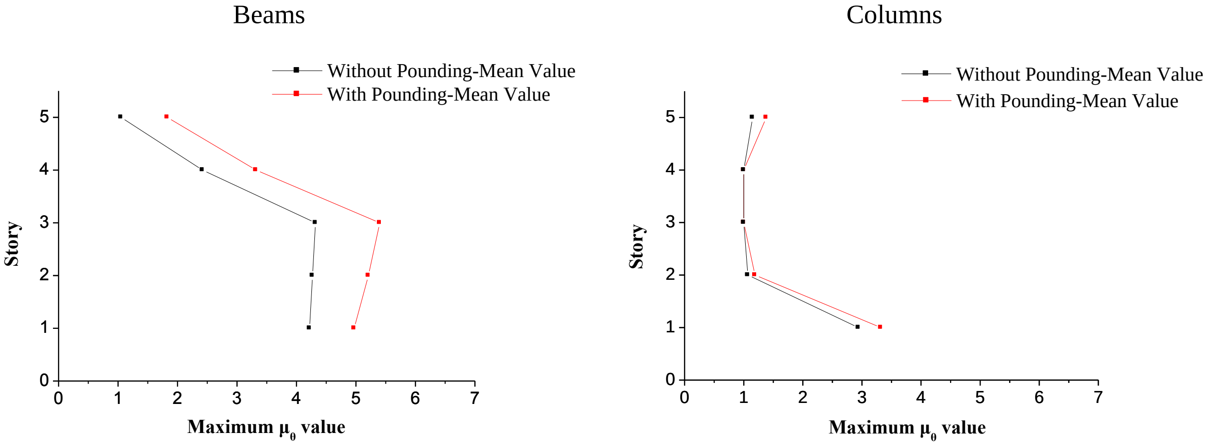

Figure 4. Mean values of maximum rotational ductility factors in building A with and without pounding: El Centro, Taft and Eureka records.

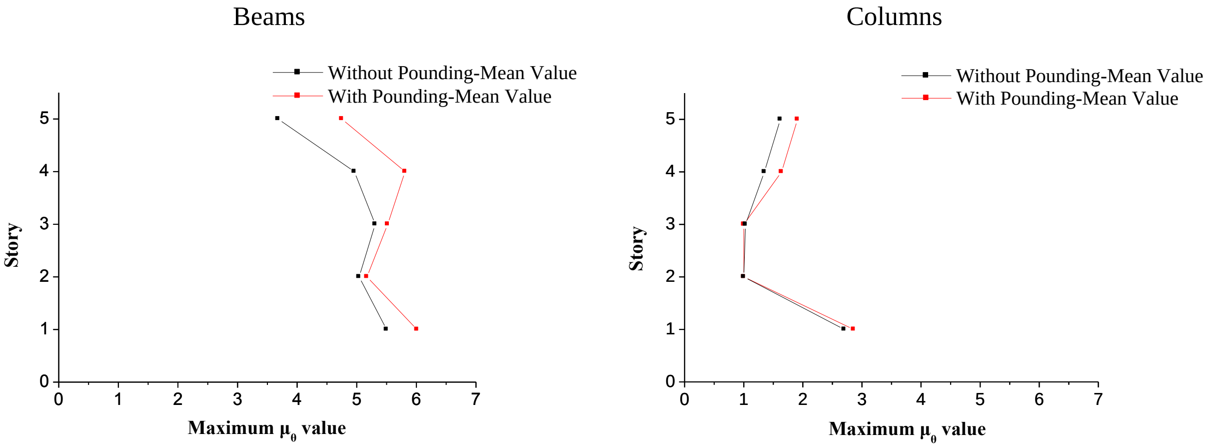

Figure 5. Mean values of maximum rotational ductility factors in building B with and without pounding: El Centro, Taft and Eureka records.

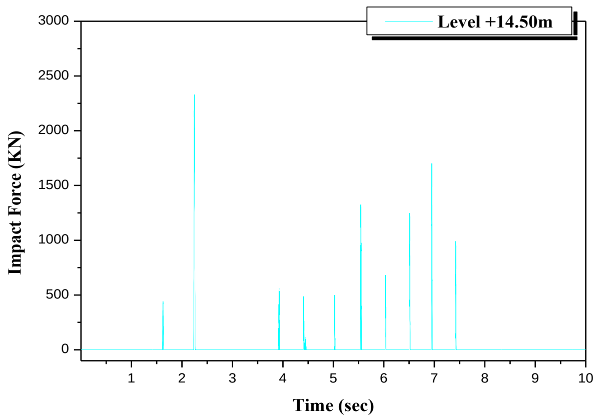

Figure 6. Impact forces at the top story of building A collision shear wall.

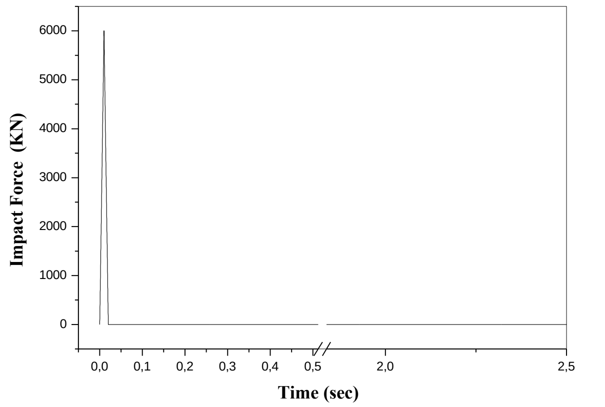

Figure 7. Triangular impact force diagram of 6000KN at 0.02secof a 2.5sec total analysis time.

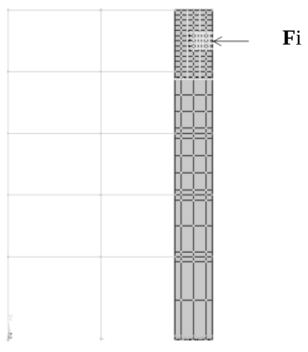

Figure 8. Building A 2d quadrilateral finite element model.

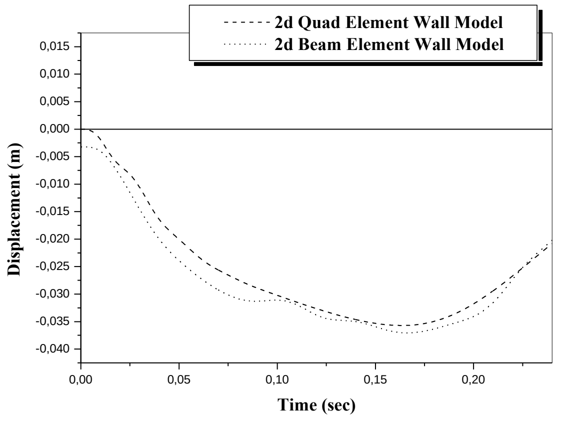

Figure 9. Horizontal displacement of building's A top floor calculated by the two different models.

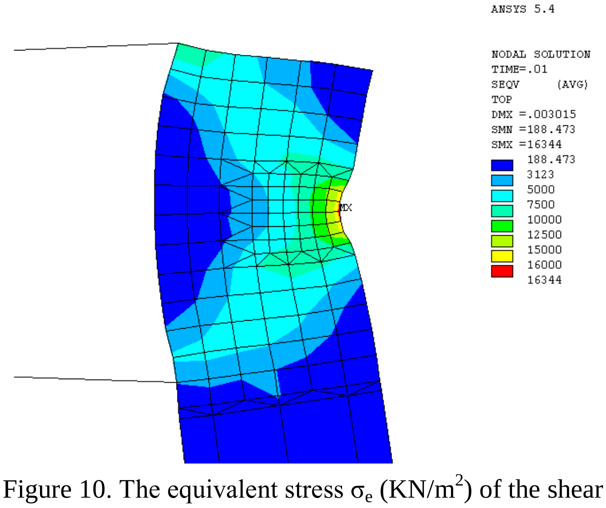

Figure 10. The equivalent stress σ_e (KN/m^2) of the shear wall at time 0.01sec.

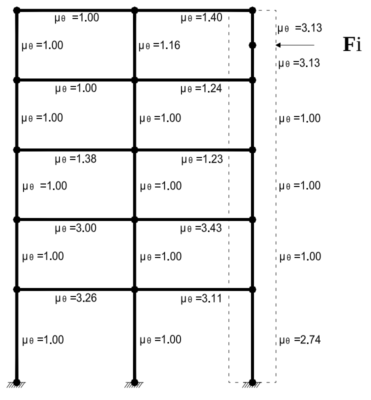

Figure 11. Rotational ductility factors for the members of building A.