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Floor and Wall Interaction in Unreinforced Masonry Buildings

Interaction plancher-paroi dans les bâtiments en maçonnerie

Wechselwirkung von Wand und Decke in unbewehrten Mauerwerksbauten

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SUMMARY

This paper focuses on the application of a finite element technique that has been developed to model the friction and impact characteristics of a wood joist bearing on a brick wall. A traditional yield model was used for friction, in conjunction with an innovative impact formulation which uses a stiff spring and damper to approximate the energy lost in impact between a floor or roof diaphragm and a wall. The nonlinear, dynamic interaction between the wood floor diaphragm and the wall at the interface is shown to be significant and the finite element procedure developed can represent both the retrofitted and unretrofitted conditions.

RÉSUMÉ

Cette communication traite d'une technique par éléments finis qui a été développée pour reproduire les caractéristiques de friction et d'impact d'une solive appuyée sur un mur en brique. Un modèle traditionnel a été employé pour la friction appuyée avec une formulation innovatrice de l'impact utilisant un ressort rigide et un amortisseur pour représenter la quantité d'énergie perdue dans l'impact d'un plancher ou d'un toit avec un mur. L'interaction dynamique et non linéaire entre le plancher et le mur est importante et la technique par éléments finis qui a été développée peut représenter également les conditions avec ou sans restauration.

ZUSAMMENFASSUNG

Es werden Anwendungen eines Finite-Element-Modells der Reibungs- und Aufprallcharakteristiken eines Holzbalkenauflagers auf einer Mauerwand vorgestellt. Dabei ist die Reibung herkömmlich als Fliesskriterium modelliert, während für den Energieverlust beim Aufprall zwischen einer Decken- oder Dachscheibe auf die Wand eine neuartige Feder-Dämpfer-Formulierung entwickelt wurde. Wie gezeigt wird, ist die dynamische Wechselwirkung im Auflager bedeutend. Das Verfahren ist für Zustände mit und ohne Verstärkungsmassnahmen geeignet.

1. INTRODUCTION

Typically, joists bearing on masonry walls are modeled as frictionless rollers in an equivalent static analysis. Since this model rarely satisfies conditions for structural stability, the engineer designing a structural retrofit is required to assume a pinned condition in the structural model, for which an approximate pin is constructed at the joist-wall interface when the actual building is retrofitted. All such details are currently designed based on the assumption that during an earthquake, the unretrofitted joist-to-wall connection has no lateral load capacity. In reality, however, the joist bearing has some lateral load capacity due to friction and contact between the elements. In order to assess the dynamic affects of motion between the joist end and the wall, and more accurately analyze retrofit alternatives, an analytical model that accounts for this motion has been developed.

2. THE ANALYTICAL MODEL

The analytical model must be capable of capturing the dynamic friction behavior of a joist bearing connection. It must also take into account impacts that might occur when the joist end comes into contact with the end of the joist pocket.

2.1 Friction Model

The fact that, in general, there is frictional resistance to sliding of one body on another occurs due to relative motion has been known for centuries, and classical models considering static and dynamic friction components have been developed (e.g., Coulomb). A finite element friction model which accounts for the dynamic friction behavior of a joist bearing on a brick wall has been developed by the author and explained in detail elsewhere (Cross 1992, Jones and Cross 1991).

2.2 Impact Model

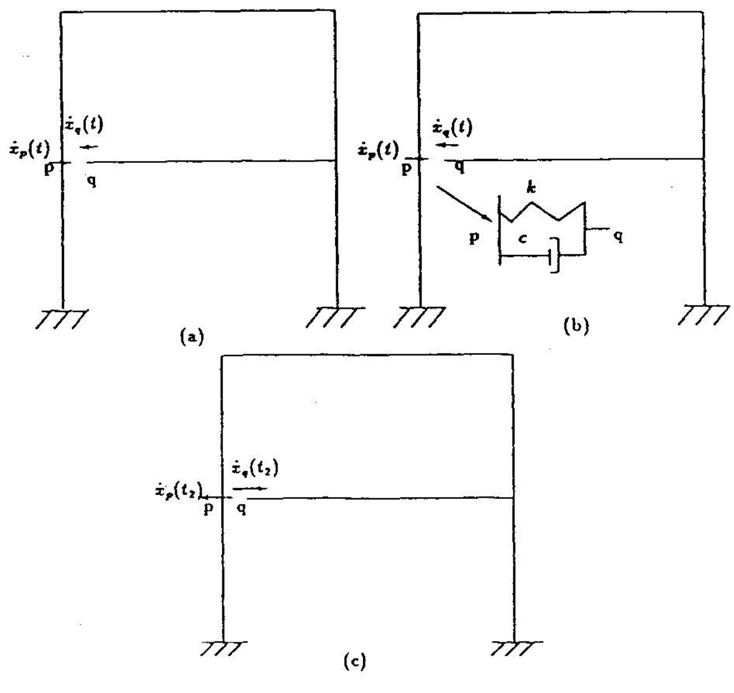

Figure 1: (a) Before Impact, (b) During Impact, (c) After Impact

The impact formulation developed for this research is a coefficient-of-restitution method, using an equivalent spring-damper placed at the point of impact for the duration of impact. The structure immediately prior to interior impact is modeled as shown in Figure 1(a), during impact as shown in Figure 1(b), and after impact as shown in Figure 1(c). The stiffness and damping properties of the model change as the structure goes through the three states. Full details of the derivation of the impact procedure briefly described above are given in Cross and Jones (accepted for publication 1993).

3. APPLICATION OF THE METHOD

The finite element procedure described above has been applied to an actual unreinforced masonry structure. Since the building was instrumented before the Loma Prieta earthquake, data from the structure were gathered during that earthquake. The connection analysis procedure derived in this research will be applied to this structure, and comparisons made to recorded data. In addition, the role of the retrofit strategy used in this building will be examined.

3.1 The Gilroy Historic URM Building

The structure to be investigated in this study is an historic unreinforced masonry building in Gilroy, California. It is currently used for commercial space. The original part of the building was constructed in 1890 and survived the 1906 earthquake. The building was surveyed in 1990 and as-built drawings with details are available. The building is a two-story, brick masonry structure with story heights of about 3.73 meters (the second floor unbraced height varies due to differences in roof elevations). The 30.5 cm, 3-wythe brick walls act as shear walls, and the wood floors are horizontal diaphragms to resist lateral load. These floors consist of 25.4 mm by 101.6 mm diagonal wood sheathing nailed to timber joists that are supported by wood beams at the second floor framing and wood trusses at the roof. Floor joists measure 50.8 mm by 355.6 mm, and the roof rafters are 50.8 mm by 152.4 mm.

The diaphragms and the walls are tied by 19.05 mm diameter steel rods anchored in the outside wythe of the walls by a hook, and with or without a hook in the diaphragms. These ties are placed every 1.55 m nominally at the east and west walls, and every 1.83 m at the south, center, and north walls. The building is founded on spread footings whose dimensions and depth were not determined by a field survey. (CSMIP 1990, Tena-Colunga 1992). The structure was instrumented by the California Strong Motion Instrumentation Program (1990).

3.2 Finite Element Model of the Gilroy URM Building

A two-dimensional, linear-elastic, finite element model was chosen for the wall and floor elements of the Gilroy building. This was done for two reasons: First, that the actual response of the structure was in the linear range during the Loma Prieta event (Tena-Colunga 1992), and second, the influence of connection nonlinearities on global structural response was to be examined in this research. The two-dimensional model was based on an east-west cross section of the building, the dimensions of which are shown schematically in Figure 2.

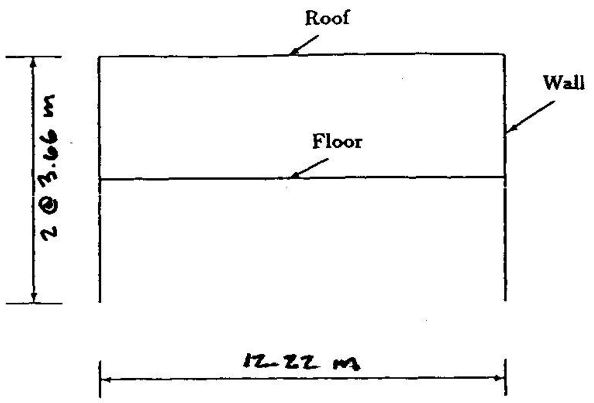

Fig 2: East-West Cross Section of the Gilroy Historic Commercial Building

The cross section at the center of the original portion of the structure was modeled, because it was at this location that horizontal diaphragm deflections would be the largest, and hence the relative motion between the floor and the wall was potentially greatest at this location. A 1.83 metre section of the wall was chosen as a tributary width to correspond to the nominal spacing between the 19.05 mm wall anchors. This particular width was also found to correspond well to the width required to obtain the appropriate generalized mass for a simple beam spanning horizontally between shear walls. The 1.83 m section was also used to obtain floor and roof properties.

Shear walls were modeled by placing stiff vertical members at the center of the model, which were in turn attached to horizontal springs connected to the floor and roof members. The horizontal springs were used to model diaphragm flexibility. Additionally, the transverse stiffness of the bearing walls at each level was modeled by horizontal springs attached to the shear walls. This permitted the outside walls to behave as beams spanning horizontally between shear walls. The stiffness of this spring could be adjusted to account for a shorter span between floor-to-wall anchors. The foundations were modeled as per the recommendations of the NEHRP provisions (FEMA 1989).

4. ANALYSIS OF THE GILROY HISTORIC 2-STORY COMMERCIAL BUILDING

4.1 Retrofitted Structure

As a calibration of the model, the retrofitted structure with wall ties was analyzed, and the results were compared to actual recorded values from the Loma Prieta Earthquake of 1989. The input parameters for the model were those discussed in the previous section. Base acceleration was input as the east-west component of the Loma Prieta Earthquake. The linear finite element approximation gave results that are close to the recorded results. The maximum predicted roof acceleration relative to the base is .78g, while the maximum recorded acceleration relative to the base was .75g.

Field observations of the structure (Tena-Colunga 1992) suggest that the structure responded in its elastic range to the Loma Prieta earthquake. This is verified by the stresses computed by the finite element model; in no case did the stresses in the retrofitted structure exceed their ultimate limits. Maximum tensile stress in the outside walls approached 689.48 kPa, which is close to the ultimate tensile strength of brick (258.4 kPa as measured by Ali and Page (1988)). Ten percent damping was used for the masonry material; additional damping would lower this stress value.

4.2 Unretrofitted Structure

In the study presented in this section, the possible response of the unretrofitted Gilroy structure (i.e., the structure without wall ties) to the Loma Prieta Earthquake of 1989 is examined. Different input parameters for the pocket element were considered, including coefficients of friction of .2 and .4, coefficients of restitution of .25 and .5, and distance to the end of pocket of 2.54 mm and 25.4 mm. These quantities will be varied while using the default values discussed above for all other parameters. For these analyses, the system limit states are considered to be either forces high enough to initiate cracking of the brick masonry, or displacement response great enough to make the joist fall off of the wall. The effects of coefficient of friction and distance to the wall will be examined in reference to the system limit states.

4.2.1 Nonlinear Gilroy Model

The complete Gilroy nonlinear finite element model described above, with the joist pocket elements incorporated, was analyzed using the finite element method. The initial parameters for the pocket element were $\mu = .2$, distance to wall = 25.4 mm, and $e = .5$. The graphs of absolute displacement indicated that the floor and roof joists begin to slide at about 3.2 seconds. For this coefficient of friction (0.2), sliding continues throughout the record for both roof and floor pockets. The relative displacement between the joists and the walls is high (Figure 3). If only 50.8 mm of bearing exist for each joist, the right roof would fall at 4.5 seconds, and the left roof would fall at 4.4 seconds. A full 69.9 mm of bearing is required to assure that the left roof joist would stay on its bearing for the ten-second time history. The relative displacement at the floor level is lower. About 38.1 mm of relative motion occurs between the joist and the wall, with the maximum occurring at 4.5 seconds.

The joist-falling-off-bearing condition does not assure collapse, but it indicates a possibly high-risk condition. Joist bearing lengths can vary in older structures from as much as 152.4 mm (or higher) to merely 25.4 mm. The actual bearing state depends greatly on the individual structure and joist in question, and can vary from joist to joist in even a well-constructed building.

The forces developed at the wall interface are relatively low until an impact occurs, varying between μN and $-\mu N$. When an impact occurs, the horizontal force at the joist bearing increases. The largest magnitude impact force occurs at the left roof pocket, with a magnitude of 2363 N. For the 1.83 m section of roof used in the analysis, the cross-sectional area is 144 in^2. If it is assumed that this is the contact area during impact, the contact stress between the joist and the wall is 406.79 kPa. This stress is well below the ultimate stress required to crush the brick or the wood in bearing. The punching stresses would be even lower, and would not be large enough to cause the joist to punch through the brick wall.

The maximum moment occurs at 4.4 seconds in the lower section (below the first floor) of the right wall, with a magnitude of 37.97 KN-m. This would cause a bending stress in the brick wall of 1344.5 kPa (after accounting for axial compression). This stress is higher than the ultimate tensile stress for typical brick masonry (689.5 - 758.4 kPa), and indicates that the force of the impact between the diaphragms and the walls could cause bending failure in an unretrofitted wall (note that the maximum moment occurs at the same time as the impact of the floor with the wall). The maximum shear force in the wall was 12.4 kips, with a corresponding average shear stress of 99.3 kPa for a 1.83 m section of wall, which is within reasonable limits, even for conservative analysis. Maximum axial loads were low.

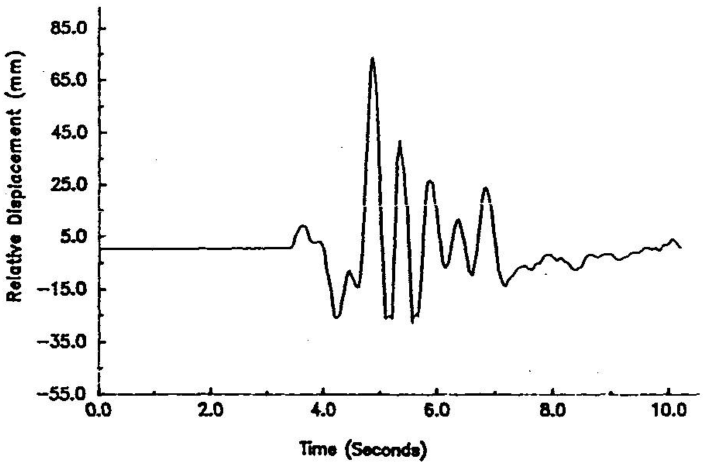

Fig 3: Relative Displacement (μ =.2 , Left Roof)

4.2.2 Friction Coefficient of .4

A second run was made, with all parameters the same as the first nonlinear run, except for a higher coefficient of friction at the bearing. The coefficient of friction used in the first run (.2) is lower than the static coefficient of friction specified in Machinery's Handbook; it is possible that the actual coefficient of dynamic friction between wood and brick could be closer to the coefficient of static friction. Also, since in this study the effects of vertical accelerations have not been taken into account, an effective coefficient of .4 between the joist and its bearing is not unreasonable.

The maximum relative displacement for the μ =.4 case is 44.45 mm, occurring at 5 seconds at the left roof pocket. This is 25.4 mm less than the 69.85 mm relative displacement occurring in the μ =.2 run. The implications for this for retrofit design are clear: a 50.8 mm bearing condition would prevent the joist from falling off the wall for the .4 coefficient of friction. Clearly, further experimental tests of wood bearing on brick would allow more certainty in the choice of the coefficient-of-friction value.

The wall is impacted only twice at the right roof pocket, with a maximum moment in the right wall of 293 in-k. This would cause a net tensile stress of 999.7 kPa. This stress is still higher than the ultimate tensile strength of brick in bending, but with only two high intensity impacts occurring, the wall may not suffer catastrophic failure. Further of modeling of the full nonlinear response of the brick masonry may be warranted in this case.

4.2.3 Coefficient of Restitution of .25

A run was made with the coefficient of restitution between brick and wood set at 0.25. All other parameters were at their default values; in particular, μ was .2, and the distance to the end of the pocket was 25.4 mm. The response was very similar to the e=.5 case, which indicates that the system response is not very sensitive to the choice of *e*.

4.2.4 Distance to End of Pocket of 2.54 mm

The final run for the Gilroy structure was made using the default parameters described in the previous section, with a distance to the end of the joist pocket of 2.54 mm. In actual field conditions, the joist may be grouted at the end of the pocket, or loose bricks or other debris can limit travel to the end of the pocket. Also, the proximity of the floor sheathing to the wall can be very close in some structures, which would effectively limit the distance that the joist may travel before impact. Therefore, it is not unreasonable to assume that the end of the pocket is 2.54 mm.

The results from this analysis are very interesting. It is immediately apparent from the output that the relative displacements are lower than those for the first example (with a 25.4 mm distance to end of pocket). The maximum relative displacement occurs at the right roof, where the relative distance is 53.34 mm. This is less than the 69.85 mm relative displacement computed for the 25.4 mm pocket. This seems to indicate that if the distance from the end of the joist to the end of the pocket is minimized, a smaller bearing length is required.

The force of impact, however, is apparently higher with the shorter pocket distance. The maximum force can be seen to occur at the floor pockets, with a magnitude of just over 2780 N. This is still relatively low, however, compared to the bearing strength of wood and brick. The maximum moment occurs in Element 17, which is in the second level of the right wall. It has a magnitude of 25.42 kN-m, which would cause a corresponding maximum tensile bending stress of 813.6 kPa. This stress is just over the ultimate tensile stress for brick masonry (689.5 - 758.4 kPa), and is 40% lower than the stress in the 25.4 mm pocket case. It is possible that the shorter end distance is producing a partial bracing effect, at least between the end of the pocket and the joist. This would explain the higher impact forces: since wall horizontal motion is more fully restricted, the joists are acting more fully as stiff supports throughout the time history and taking more load. Another possible explanation for this effect is that the lack of motion in the direction of the wall produces less energy loss due to frictional work, and this could raise the momentum associated with the impact.

Thus, it can be seen that in terms of the bearing distance required for the joist and the moment response of the exterior walls, a reduction in pocket depth is desirable. To produce this result in actual structures, the joist pockets could be grouted (effectively reducing any travel distance). This procedure would be relatively easy and inexpensive with existing equipment. It is not currently a recommended retrofit procedure. It should be noted that this could increase the force between the wall and the joist. If a wythe separation condition is of concern, then the attachment of the brick wythes at the floor and roof levels should be examined carefully before grouting.

5. CONCLUSIONS

Failure of this 2D model of the structure was observed in two distinct cases. The first failure mode occurs when the joists move sufficiently far relative to the wall to cause the joists to fall of the wall. Although in some cases it may be possible for the joists to move back on to the wall (this would require a 3D analysis), it is a highly undesirable event which could lead to catastrophic collapse. This failure mode is dependent on the length of the bearing, however, and in the worst case for this structure, a 114.3 mm minimum bearing length for the joists would have been sufficient to avoid this type of failure.

The second failure mode would be collapse of the wall in out-of-plane bending. Interestingly, the movement of the joist relative to the wall causes impact of the floor or roof diaphragm with the wall in all cases examined. These impact forces can be high enough to cause tensile bending failure of the brick wall. Such a failure is very dangerous, because bearing wall structures usually exhibit little redundancy. The collapse of a bearing wall could lead to total collapse of the structure. Although the forces do not appear to be high enough to cause bearing or punching failure of the wall at the floor and roof levels, it is possible that the impact forces could cause wythe peeling, a common failure in brick buildings during earthquakes.

If properly accounted for in analysis and design, the mobilization of some joints between the floor or roof and the wall in a URM building may allow for a minimization of retrofit construction (allowing some of the joists to remain unattached to the wall, for example), which could reduce repair costs. Additionally, for historic buildings, the minimization of retrofit interventions might allow more of the existing structure to remain intact, which is a primary goal of preservation architects.

In conclusion, it is clear that the dynamic interaction between the floor and roof diaphragms and the bearing wall would have been significant in the Gilroy structure if it had not been retrofitted. The retrofitted structure responded elastically to the Loma Prieta earthquake.

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