Discussion of "Lateral Stiffness of Brick Masonry Infilled Plane Frames" by P. G. Asteris

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The author investigates the lateral stiffness of infilled frames, particularly focusing on the reduction of stiffness due to openings. The finite-element method is used to analyze various frame configurations. Infill behavior in itself is quite complex, and openings add to the complexity of the problem. Thus, the author is to be commended for examining this problem. The discussers, however, have several questions with respect to the analysis, and also offer experimental data on a structural clay tile infilled frame that had a square opening in the upper corner, similar to those systems analyzed in the given paper.

The author used a four-node isoparametric rectangular finite element. This element is known to be quite stiff in bending and to exhibit shear locking (Cook et al. 2002). Thus, the element is probably not appropriate for the frame members of the infilled frame model, and perhaps even the infill itself. A much better element would be either an eight-node or nine-node quadrilateral element. The discussers are interested as to why the four-node element was used, and how the model was validated.

The author used what amounts to a gap element to model the frame/infill interface. Although the author claims that this is a new finite-element technique, gap elements have been used for many years, with two examples of the application of gap elements to infilled frame analysis being Liauw and Kwan (1982) and Jamal et al. (1992). Gap elements determine contact lengths and contact stress as part of the analysis without any ad hoc assumptions. The author's method of analysis does not allow for any sliding of the interface, or implicitly assumes infinite shear strength. Typically, gap elements allow for sliding. The lack of sliding can lead to distorted elements, such as the third and fourth element up from the bottom on the right side of the infill in Fig. 2(c). The discussers are curious as to why a more traditional gap element was not used, and if the author investigated the effect of preventing sliding at the interface.

The discussers tested a 2.84-m-long by 2.24-m-high structural clay tile infill in a steel frame with a 0.61-m-square opening in the loaded corner (Flanagan 1994). This amounts to an opening percentage of 5.8%. When the frame was pulled, corresponding to case C of the opening outside and up right of the diagonal, the stiffness was essentially the same as a corresponding solid infill frame. This agrees with the author's Fig. 5. When the frame was pushed, corresponding to case B of the opening upon the diagonal, the stiffness was only approximately 40% of the corresponding solid infill stiffness. The author's Fig. 5 indicates a stiffness of just over 70% of the solid infill stiffness. This is a wide discrepancy between analytical and experimental results.

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The author is thankful to the discussers for their interest in our work and also for bringing to attention some interesting points, the further elaboration of which will constitute part of our future research. Given that the real overall behavior of an infilled frame is a complex indeterminate problem (Smith 1966), the following should be pointed out:

1. In the discussed paper, a new finite-element technique for the analysis of brickwork infilled plane frames under lateral loads has been presented. The basic characteristic of this analysis is that the infill/frame contact lengths and the contact stresses are estimated as an integral part of the solution, and are not assumed in an ad hoc way, as it is commonly used. For the analysis, a well-known four-node isoparametric rectangular finite-element model with eight degrees of freedom (DOF) has been used, making the application and the control of the proposed method much easier. It is worth noting that the validation of the method does not depend of the type of finite element, e.g., an eight-node or nine-node quadrilateral element. The use of a four-node isoparametric rectangular finite element with a finer mesh has been shown to be a suitable model for the modeling of masonry (Samarasinghe 1980; Asteris 2000; Asteris and Tzamtzis 2003) as well as for the modeling of infilled frames (Syrmakezis and Asteris 2001; Asteris 2003).

2. In spite of the assertion of the discussers that our method is equivalent to a gap element to model the frame/infill interface, we do not actually use an interface (gap) element; thus, our method better represents the real cases (which do not include interface elements between the masonry infill and the surrounding frame). We agree that gap elements have been used for many years to model the infilled frames behavior but are limited to simple cases, mostly to one-story one-bay infilled structures. In our work the investigation has been extended to the case of multistory, fully, or partially infilled frames. It is shown that the redistribution of shear force is critically influenced by the presence and continuity of infill panels. The presence of infills leads, in general, to decreased shear forces on the frame columns. However, in the case of infilled frame with a soft ground story, the shear forces acting on columns are considerably higher than those obtained from the analysis of the bare frame.
3. Using our approach, the influence of the masonry infill panel opening in the reduction of the infilled frames stiffness has been investigated. A parametric study is carried out using as parameters the position and opening percentage of the masonry infill panel opening for the case of one-story one-bay *reinforced concrete* infilled frame. The discussers compare the analytical results with their experimental results, which refer to (**a**) infill in a *steel* frame (and not concrete!), and (**b**) different boundary conditions (supports). It should be pointed out that, as is well known, the response of infilled frame is critically influenced by the above two parameters. It is worth noticing that the contribution of the infill wall to the frame lateral stiffness is much reduced when the structure is subjected under reversed cyclic loading, as in real structures under earthquake conditions. The relevant experimental findings (Vintzeleou and Tassios 1989) showed a considerable reduction in the stiffness of infilled frames under reversed cyclic loading.

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Discussion of "Contributions of C. A. P. Turner to Development of Reinforced Concrete Flat Slabs 1905–1909" by D. A. Gasparini

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The author has admirably presented significant contributions of C. A. P. Turner. In spite of technical confusions and patent controversies surrounding the development of flat slabs, the fact that numerous flat slabs of his invention had been built and that his extant 1906 Marshall building is still in service demonstrates his unique contribution. He was one of those trailblazers in the concrete field at the turn of the twentieth century who boldly attempted to transform conceptual innovations into profitable realities. The author's paper is a welcome tribute to this quintessential American structural engineer.

Perhaps to contemporary structural engineers, like this discusser, the most striking aspect of Turner's flat slab design is its very low steel requirement. As Sozen and Seiss noted, a comparison of steel weight made by McMillan in 1910 was dramatic and illustrated unacceptable significant variations among the flat slab design methods during that period. Fig. 1 shows the amounts of steel required by seven different design methods in a 20×20 ft interior panel of an 8 in. thick flat slab for a design live load of 200 lb per sq ft. by McMillan. Amazingly, Turner's design method uses the lowest amount, with just over 500 lbs per panel. For further comparison, Fig. 1 shows two additional steel weights calculated by the discusser according to the current ACI 318-02: one for the 60-grade steel and 3,000-psi concrete was approximately 1,240 lbs per panel, while another for the 40-grade steel

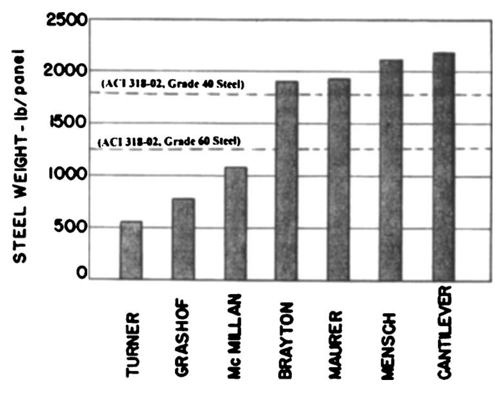

Fig. 1. Comparison of weight of steel required in the interior panel of a flat slab by various design methods in 1910 and by the current ACI 318

and 3,000-psi concrete was approximately 1,780 lbs per panel.

In spite of Turner's very low steel amounts, it is abundantly clear that the building officials and his competitors at the turn of the twentieth century must have grudgingly accepted the structures because of his load test results (Table 1). With the satisfactory results verified by load test after load test as listed in Table 1, can any structural engineer, not only in the 1910s but even in the present, suppress his unshakable confidence in the design method? Undoubtedly, Turner's tenacity and relentlessness of defending his design were founded on this repetition of ample physical evidence, no matter how elaborate and convincing analytical arguments might have been advanced to prove his design unacceptable.

Subsequently, the apparent satisfactory behavior of Turner's flat slabs had been explained by many studies, such as one by Westergaard and Slater. The discusser specifically considers the following five factors key to Turner's success in the load tests: (1) concrete tensile strength contribution to the flexural strength as proven by Westergaard and Slater; (2) Turner's column capitals, which were formed with special cast-iron forms, increased effective punching shear sections and also reduced clear panel span lengths; (3) Turner's mushroom shearhead, which provided additional shear strength; (4) the relatively thicker slab thickness; and (5) the effect of adjacent unloaded panels. As the dimensions noted in Table 1 indicate, his slab thicknesses were in general much thicker than the current minimum slab thicknesses, perhaps due to heavier design live loads such as warehouse loading. Factors 2, 3, and 4 noted above virtually eliminated any possibility of punching shear failures.

However, it may be worthwhile to further examine the flexural strengths of Turner's flat slabs. As Westergaard and Slater demonstrated that during load tests a substantial part of the load resistance was from the uncracked concrete tensile strength, the test loads were made of two resisting sources: one from the concrete tensile strength (uncracked) and another from steel stress. However, ultimate flexural strengths are governed by the amount of steel with well-developed yield-line patterns, as the comprehensive two-way slab studies at the University of Illinois indicated. Therefore, it may be plausible that under further loading to ultimate flexural failures of Turner's flat slab load tests, the apparent maximum test loads might have been reduced, and also possibly the final failure modes may have been less ductile compared to the flat slabs designed according to the current code. This aspect has not been clearly expounded in the discussion of Turner's flat slab design. The discusser welcomes any further information or discussion from the author as to Turner's low steel amounts and his successful load tests.

Closure to "Contributions of C.A.P. Turner to Development of Reinforced Concrete Flat Slabs 1905–1909" by D. A. Gasparini

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Thank you very much for your discussion. As noted in the paper, McMillan's weight of flexural reinforcement was obtained by simply doubling Turner's moment coefficient, hardly an independent design method.

I do not know whether any Turner flat slabs have been tested to failure. Observations on strength, ductility, and failure modes of his designs would indeed be interesting, but perhaps only for historical reasons given today's two-way flexural reinforcing systems and different approaches for providing shear strength.