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7672 DISCUSSION

Analysis of infilled shear walls

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As a result of an investigation that I have been making on infilled frames, I have reservations about the validity of the Author's basic assumption that the bending stiffness of the frame members is the dominant frame parameter influencing its horizontal stiffness.

42. I have analysed single and three storey infilled frames using the finite element technique. The program included provision for the automatic generation of separation cracks on the frame–infill interface, the possibility of a friction or a non-friction sliding contact on this interface and the choice of rigid or pin-ended beam connexions.

43. Table 4 shows the results from seven of the three storey infilled frame analyses. Each infill was 8 units square and had a thickness of 1 unit and an *E* value of 200 000 units. In analyses 1, 2, 3, 6 and 7 the *E* value for the frames was taken as 800 000 units. In analyses 4 and 5 the *E* values and depths of the frame members were adjusted to give the *EA* and *EI* values shown. Horizontal loading of 50 units was applied at the top beam level, 100 units at the second beam level and 100 units at the third beam level.

44. It will be seen from the deflexion results in Table 4 that the overall deflexion of the frame depends to a far greater extent on the axial stiffness of the frame members than on their flexural stiffness. The Author's suggested method of analysis is based on the flexural and not the axial stiffness of the frame members, and thus its accuracy must be questioned. For example, it would give almost equal deflexions for problems 3 and 5 and also for problems 4 and 6.

45. I suggest that infilled frame deflexions can be estimated more simply and with more accuracy by considering the structure as a pin-jointed frame where the infill is represented by a single diagonal strut. For a conservative, i.e. upper bound, estimate of deflexions, I would suggest that the width of the strut should be taken as one tenth of the diagonal length of the infill. Deflexions calculated by this method are also included in Table 4.

Dr D. V. Mallick, Professor in Civil Engineering, University of Tripoli

The Author has presented an approximate method of finding the lateral response of infilled shear walls based on several simplifying assumptions. He has also stated various factors like the quality of material and workmanship, the magnitude of interface gaps present along the boundary junction, the contact length and the contact pressure on which the effectiveness of the composite action between the frame and the infill depends. However, no suggestions have been made for how to predict or estimate the number of gaps expected in the actual construction of such framed structures.

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Table 4

Analysis	*EA* frame members	*EI* frame members	Boundary friction?	Beam connexion	Deflexion by finite element analyses $\times 10^3$	Deflexion by proposed approximate method $\times 10^3$
1	496 000	15 900	Yes	Rigid	50·9	87·6
2	496 000	15 900	No	Pin-jointed	73·3	87·6
3	496 000	15 900	No	Rigid	65·1	87·6
4	496 000	534	No	Rigid	64·7	85·1
5	160 000	15 900	No	Rigid	162·8	184·1
6	160 000	534	No	Rigid	148·2	170·7
7	160 000	534	Yes	Rigid	125·4	170·7

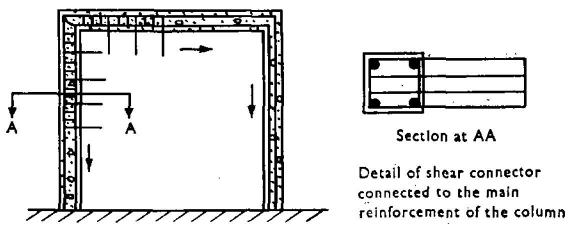

Fig. 17. Provision of shear connectors in reinforced concrete infilled frame

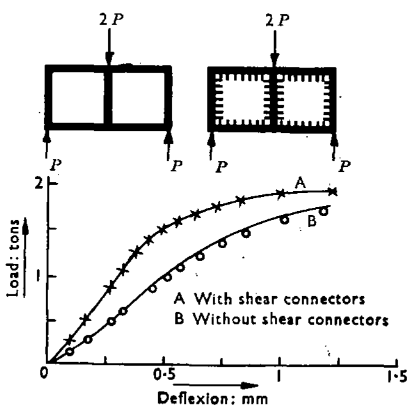

Fig. 18. Load–deflexion curve of infilled shear wall

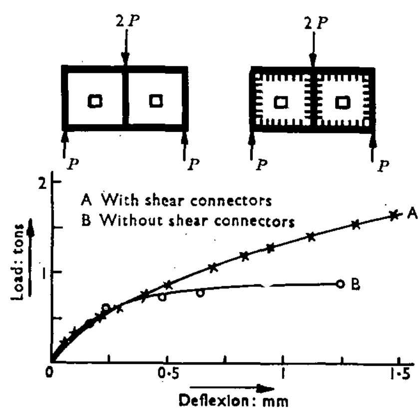

Fig. 19. Effect of the central opening on the lateral stiffness of the infilled shear wall

In my opinion it is not easy to do so. The presence of these boundary clearances considerably reduces the effectiveness of the infilled shear walls, because when spaces exist the frame members have to be designed to resist the lateral forces alone until these spaces are closed and composite action of the frame and the infill comes into operation. Hence, the advantage of composite action in which frame members are to be designed to carry fewer moments and forces is lost.

47. Thus, from the aspect of practical use of these infilled shear walls in multistorey construction, it becomes obligatory to find means by which the composite action of the frame and the infill becomes operative from the start of loading even if small interface openings exist along the boundary junction. The concept of providing shear connectors at a certain spacing along the internal boundary of the frame which penetrates into the infill to a certain length has been given.[4,5] The presence of these shear connectors proved very effective as was evident from the experimental results. For steel frames it is easy to weld such shear connectors, but for reinforced concrete frames they can be provided by means of reinforcing bars cantilevering from the main reinforcement at right angles to the members as shown in Fig. 17. Fig. 18 shows the load deflexion curve of two infilled frames with and without shear connectors. With the provision of these shear connectors it is possible to achieve the composite action of infilled shear walls more effectively and as such this type of structure will prove effective and economical in resisting earthquake and wind forces.

48. The Author has suggested the force–displacement method for analysing infilled shear walls. He derived equation (1) by applying compatibility and equilibrium conditions. In order to satisfy the horizontal equilibrium of forces at any level, the fourth equilibrium condition should yield the equation

$$\frac{2(m_a+m_b)}{h}+P\cos\alpha = W$$

49. In equation (2) the Author has calculated the value of Δ_f from the deformed geometry of the infill panel. The manner of calculating the value of A_w is not indicated. The value of A_w plays an important role in finding the response of infilled shear walls, because once the value of A_w is established and an equivalent model of the infilled frame conceived then any suitable method of analysis will determine the lateral stiffness of frame.

50. Research[1,6] has been published on the estimation of the effective width of the infill panel. I have found experimentally that if shear connectors are used, the formula suggested by Holmes[1] for finding the effective width of the infill panel that is equal to $d/3$ yields theoretical results which agree reasonably well with the experimental ones. In the absence of shear connectors A_w can be obtained using the curves given by Stafford Smith and Carter.[6]

51. Having estimated the value of A_w, the infill panel can be replaced by an equivalent diagonal strut hinged at the two opposite corners. The infilled shear wall can now be conveniently analysed by the stiffness method using the anti-symmetry with two unknowns, θ and Δ. As such the final size of the complete matrix for a multistorey frame will be greatly reduced, thus saving the computer time for inversion. The two equations using the stiffness method are derived in the form

$$-\frac{12E_cI_c}{h^2}\theta+(A_wE_w\cos^2\alpha/d-24E_cI_c/h^3)\Delta = W$$

$$(4E_cI_c/h+2E_cI_c/L)\theta+(-6E_cI_c/h^2)\Delta = 0$$

Knowing the values of θ and Δ, the values of m_a, m_b and P can be obtained from the expressions

$$m_a = 2E_cI_c/h(2\theta-3\Delta/h)$$

$$m_b = 2E_cI_c/h(\theta-3\Delta/h)$$

$$P = E_wA_w\cos\alpha\Delta/d.$$

52. The derivation of P'' in equation (10) is not obvious from the equilibrium condition of the infill panel.

53. The Author has mentioned that the appearance of diagonal cracks indicates that a double arching action rather than a direct strut action takes place. This thinking seems to be different from the observations made by other research workers. Fig. 19 shows the load–deflexion curves of an infilled frame with a central opening. It can be seen that the lateral deflexion is considerably decreased. More experimental evidence is needed to support this concept of infilled frame behaviour.

Mr M. R. Kadir, Department of Civil Engineering and Building Science, University of Edinburgh

The Paper illustrates the application of the force–displacement method for the analysis of infilled shear walls. In the analysis the infill is replaced by an equivalent diagonal compressive strut. Once this assumption is made, the solution for the forces and deflexions is a matter of solving a plane frame structure for which established analysis methods by computer are widely available, including shear and axial deformations in which their consideration is essential in the design of multistorey structures, and may cause large deflexion. Results from any analysis based on the principle of diagonal strut mainly depend on the value of its effective width. Stafford Smith[3] has shown that the effective width depends on the relative stiffness of the frame and panel. The Author seems to have taken an approximate value of $0{\cdot}3d$ for all his structures (one third of the diagonal length has been suggested by Holmes[1]), in spite of the variation in their relative stiffnesses.

55. In equation (2) A/D the term $d/E_w A_w$ represents the diagonal deflexion of the infill, which in fact must be converted to lateral deflexion as stated in equation (1) A/D.

56. In the analysis of the effects of spaces between frame and panel the Author assumes that all spaces are close together. However, it is possible that gaps at some of the storeys may come into full contact with the frame before others have reached this condition. This possibility may alter the analysis procedure.

57. In an infilled frame the infill may not rotate to position A; it is possible that contact will be at the column interface only which exerts normal and shear forces on the infill, and may cause cracking failure in the infill (especially in masonry panels) before fully bearing on both column and beam interfaces.

58. If the Author has carried out an example analysis for the effect of interface forces in multistorey infilled frame, it would be interesting to know the difference between the results compared with those neglecting interaction forces.

59. I should be grateful for more information about the experimental results and their related curves shown in the Appendix, because both the theoretical and the experimental results for the six storey structure are rather unrealistic. Analysis of the structure has been carried out by stiffness analysis and various approximate methods. The results obtained are more than 20 times higher than the analytical and experimental results from the Paper. If the value of the non-dimensional coefficient for deflexion ($K_0 = (\Delta_0 EI/Wh_3) \times 10^{-3}$) were converted to an actual value of deflexion, the result would be very small for the model structure described. Comparing the Author's results for the bare frame with the infilled frame gives

$$\frac{\Delta_{\text{bare frame}}}{\Delta_{\text{infilled frame}}} = 143$$

I wonder whether this is possible or not.

Dr Smolira

The proposed analysis does not presume that the bending stresses in the frame are the only dominant parameters influencing horizontal stiffness of the shear wall.

In the primary analysis suggested, the effect of axial forces has not been included so as not to complicate the presentation of the matrix. However, the effect of axial forces can be included using the force–displacement method with little additional effort; it would be necessary to take into account the effect of stiffness (or flexibility) of members by including the instability functions, the effect of change of lengths of columns and even the effect of the changed geometry of the deflected system.

61. **Mr Riddington**'s suggestion that the infilled shear walls can be analysed as trusses is generally accepted, and although it cannot be regarded as exact it may be satisfactory for practical design within certain limits. However, these limits have not yet been fully determined.

62. The effects of interface spaces can be taken into account in the analysis provided that they are known. However, the experimental verification has been found to be extremely difficult because of the uncertainty of the width and disposition of gaps, effect of shrinkage of concrete, temperature and moisture variations. Numerous tests have been carried out at the City University but they were inconclusive. Nevertheless they are being continued.

63. It appears that the solution of equations (3) should be

$$m_{01} = 0{\cdot}027W$$
$$m_{10} = 0{\cdot}0497W$$
$$P = 1{\cdot}0163W$$
$$\Delta = 0{\cdot}043Wh/EI$$

64. The ratio of 143 of the deflexion of the bare frame to the deflexion of the infill is difficult to interpret. Based on the ratio of the second moments of area of columns to the infill, this ratio is 865; while including the effect of the respective E values this ratio could be 109. The difficulties of estimating the actual stiffness of members arise from the age-old difficulties in ascertaining the I values for reinforced concrete members as well as their E values. The respective values for the brickwork are equally uncertain.

65. **Dr Mallick** is right in saying that the Paper does not give any suggestion on how to predict or to estimate the number of gaps expected in the actual construction of infilled shear walls. These gaps depend not only on the comparatively predictable effects such as shrinkage of concrete and cement mortar but also on the early effect of moisture expansion of bricks, ambient variations of moisture content, on temperature variations and on workmanship. These parameters are difficult to predict analytically but it may be possible to treat the problem of spaces statistically and to make allowances in the design based on this approach. However, I agree that the effectiveness of the shear wall can be seriously impaired by the presence of large spaces. Moreover, there must be some critical values of spaces beyond which the stiffening effect of the infill vanishes and the frame carries loads alone, at least within the working range of loads. At the ultimate, presumably even large gaps in the infill would be closed at least at some transitional period of collapse.

66. Shear connectors would no doubt be beneficial and would bypass the difficulties associated with workmanship. To the best of my knowledge they have not yet been used except for two cases: first when the brickwork is built before casting concrete and the projecting bricks are left as shear connectors (this is common practice in the USA), and second when bonding irons are used. The latter are always provided but for a different purpose and although they are very flexible they may to some extent act as shear connectors because of the small size of the spaces between the frame and the brick wall. It would be interesting to check the effectiveness of the bonding irons acting as shear connectors but I cannot give much information on this.

67. The value of the effective width of the strut is an important parameter in the analysis of infilled shear walls. Fortunately, it appears that the solution is not sensitive to the considerable variations in the assumed width of the strut. There

appears to be a wide variation of opinion with regard to the effective width of the strut, which ranges from $\frac{1}{3}$ to $\frac{1}{10}$ of the length of the diagonal.

68. The third row in matrix (2) should contain the term of cos α in the denominator, i.e.

$$\frac{dE_c I_c}{E_w A_w h \cos \alpha}$$

69. I am fully aware that the concept of the double arching effect is unexpected. However, experiments as well as the finite elements analysis prove this beyond doubt, i.e. it can be shown that the stresses within the central part of the wall can be practically nil provided that the wall is monolithic and is made of individual bricks. However, this concept does not appreciably affect the overall response of the infilled shear wall, and the analysis based on the strut action may still be adequate for practical purposes within certain limits.

70. With reference to the contribution from **Mr Kadir**, the effective width of the strut in the examples has been taken as $d/3$. The problem of the effective width is complex, but fortunately it appears that the resulting distribution of forces and displacements in the infilled shear wall is not very sensitive even to large variations of this parameter.

71. The value of δw should strictly be taken as the component of the diagonal deformation of the strut. The term cos α has been inadvertently omitted but is used in the examples.

72. I agree with Mr Kadir that in practice not all gaps will necessarily close at the same time. The example used is idealized to illustrate the effect of gaps in such frames.

73. It is difficult to visualize how the contact of the infill can exist only with the column when the panel's own weight is considered. The suggested analysis is linearly elastic and therefore does not apply when large cracks appear or at the ultimate.

References

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6. STAFFORD SMITH B. and CARTER C. A method of analysis of infilled frames. *Proc. Instn Civ. Engrs*, 1969, **44**, Sept., 31–48.