READER COMMENTS

Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads*

by PCI Committee on Precast Concrete Bearing Wall Buildings

Comments by Emil C. Hach, William Hanuschak, Alan H. Mattock and Committee

EMIL C. HACH†

The Committee and Mr. Speyer are to be commended for a very valuable report. It would appear that much if not most of the report deals with good construction irrespective of the type of loads, i.e., normal or abnormal.

From the writer's vantage as a consulting structural engineer for a systems precast concrete bearing wall type building a few comments seem in order.

1. The spacing of expansion joints in general at 180 ft apart in Section 5.5 does not seem reasonable. We have seen problems in buildings under 150 ft and not seen them in buildings over 200 ft depending on the climate and layout of the bearing walls. While it would be nice if one could set a certain number down, it would appear that is not possible in a recommended practice.

Finally, the spacing of expansion joints is not dictated by abnormal loads. Rather, it is determined by the normal forces due to shrinkage, temperature and creep. In our view, this does not belong in this report.

2. In Section 7.8.3 the minimum amount of steel is set at $0.001bL$. Neither b nor L are defined here. Elsewhere (Sections 7.2.1 and 7.2.2) they are defined as the height and length, respectively, of a wall. This of course, is not what was meant. If it is the usual thickness times length (or width), then it is substantially less than that in ACI 318-71 either Chapter 14, 10 or 11. We question whether less reinforcing steel than that required in ACI 318 should be sanctioned for reinforced concrete precast bearing walls.

3. The requirement of Section 6.2.2 of 2½ in. and 3½ in., respectively, for bearing of solid/hollow-core slabs and ribbed slabs, respectively, is of some interest. About 50 buildings of the type we have been involved with have been built since "Operation Break Through." All have been built with solid slabs bearing on precast walls. Tolerances are as per the *PCI Design Handbook* for hollow-core slabs (p. 8-34). The bearing length of these solid slabs has been 2 in.

In questioning personnel involved with these buildings and from our own experience, no distress due to insufficient bearing has been noted. A check of bearing stresses shows that considerably less bearing than 2 in. is required before problems from insufficient bearing would be expected to occur.

We would expect some hollow-core slabs would have problems, but then this is covered in the beginning of the paragraph, "Precast floor or roof elements should have a sufficient bearing length to safely transfer applied loads by direct bearing." We would suggest that 2 in. for solid slabs is sufficient and the requirement should be changed to this.

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WILLIAM HANUSCHAK*

The Committee (with Mr. Speyer's assistance) have presented practical minimum recommendations for the design of precast elements and particularly for connections to ensure a degree of continuity and stability to withstand abnormal loadings.

The writer believes that this report is welcomed by designers particularly for low to medium rise precast buildings in non-seismic areas where simple gravity type connections are used and these minimum requirements will govern.

The application of the "minimal tie requirements" between walls and floors as outlined in Chapter 6 should answer the concern many designers have had with such buildings and particularly their connections.

This PCI committee is to be commended for offering these guidelines to designers thereby ensuring a consistent and reasonable minimum standard and safety factor for precast bearing wall buildings.

ALAN H. MATTOCK†

Mr. Speyer is to be congratulated on assembling a most useful set of design guidelines for precast bearing wall buildings. The references given should also be very useful to any designer involved with this type of construction.

In Section 9.5.3, it is proposed that the shear-friction theory be used for the design of connections between the wall panels. For smooth joints, it is proposed that the shear-friction coefficient μ be taken to be 0.70 when designing the reinforcement. This value for μ was apparently taken from Table 6.1.3 of the *PCI Design Handbook*,[33] since Section 11.15—Shear Friction of ACI 318-71[1] only permits the shear-friction method of shear transfer design to be used when a roughened interface exists.

The origin of the value of 0.70 for μ given in Table 6.1.3 for a smooth concrete interface is uncertain, since no shear transfer test data existed for this condition at the time the *PCI Design Handbook* was written. It is probable that it was based on judgement, having in mind the value of μ established experimentally for a concrete-structural steel interface.

Tests recently made at the University of Washington of shear transfer across a concrete to concrete smooth interface indicate that the value of $\mu = 0.7$ is unconservative for this situation. The actual variation of shear transfer strength with amount of shear transfer reinforcement provided is shown in Fig. A. The shear strength is expressed as a nominal shear stress, $v_u = V_u/(bd)$; and the amount of reinforcement is expressed in terms of the reinforcement parameter:

$$\rho f_y = \frac{A_{vf} f_y}{bd}$$

In plotting the data and correlating it with the shear-friction theory, the value of the capacity reduction factor ϕ was taken as 1.0, since all material strengths and specimen dimensions were known exactly. It can be seen that the shear strength of a smooth concrete-to-concrete interface can be predicted closely using the shear-friction equation provided the value of μ is taken as 0.60.

In Series D, every precaution was taken to obtain a good bond between the precast concrete and the concrete cast

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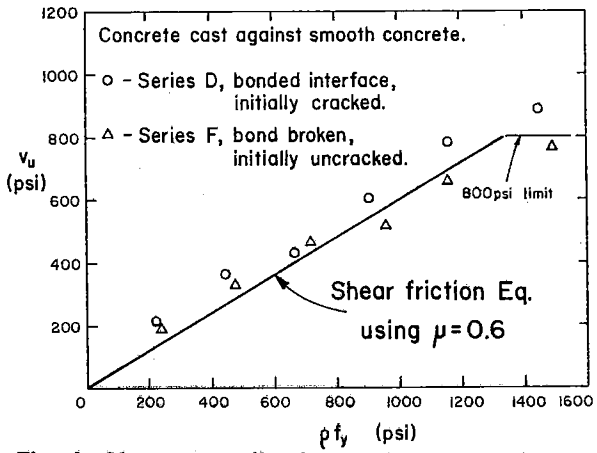

Fig. A. Shear strength of smooth concrete joints.

against it. A crack was then induced at the interface before making the shear test, by applying line loads to the front and rear faces of the specimens. This is so as to be consistent with the underlying philosophy of the shear-friction theory, which is that a crack exists in the shear plane before the shear acts.

It was also thought that the effect on shear strength of loss of bond at the interface should also be studied, so in Series F, bond at the interface was deliberately prevented by applying a bond breaker to the surface of the precast concrete before the other concrete was cast against it.

It can be seen that the shear strengths obtained for the two interface conditions were very similar and that a value of $\mu = 0.60$ is appropriate when using the shear-friction theory to calculate the shear strength of a smooth concrete to concrete joint.

The higher value of $\mu = 0.7$ obtained in tests of structural steel-concrete interfaces relates to the use of headed stud shear connectors as shear transfer reinforcement in that case. The higher value of μ obtained for the smooth steel-concrete interface than for the smooth concrete-concrete interface is due to the difference in the stress-strain curves between the headed stud shear connectors and the reinforcing bars used, and also due to the local enlargement of the stud cross section at the shear plane by the weld metal deposited around the circumference of the stud.

COMMITTEE CLOSURE

The Committee thanks Messrs. Alan H. Mattock, Emil C. Hach and William Hanuschak for their comments on these recommendations for precast concrete bearing wall buildings. The Committee appreciates Professor Mattock sharing with us research done on shear friction at the University of Washington. The report will be changed to reflect this recommendation of $\mu = 0.6$ for smooth concrete interface.

In reply to comments by Mr. Hach:

1. The Committee felt that spacing of expansion joints should be mentioned in the report so that engineers inexperienced in working with precast concrete would not think that, because the structure is composed of precast pieces, joints for thermal, creep and shrinkage control are not necessary. Creep of slabs due to prestressing also has an effect on the length between expansion joints. The consensus of the Committee was that in a straight line building setting a practical limit of 180 ft was consistent with previous experience in this type of structure.

2. The Committee felt that a lower minimum reinforcement is justified for precast panels than for cast-in-place walls which governed ACI 318-71 code requirements. Precast panels have practically no restraint at the edges during the curing and storing stages and, therefore, will not build up tensile shrinkage stresses as high as those in cast-in-place walls. For lightly loaded interior walls the reinforcement may be placed along the periphery only (Reference 3, p. 12).

3. To insure the strength requirements of the grout column between slabs where analysis requires a grout column, and at the same time, minimum support dimensions for slabs, the Committee is now recommending minimum plan dimension of 2 in. for 6-in. thick and 2.5 in. for 8-in. thick interior walls. The Committee recognizes that in many structures considerably less bearing than this would not cause distress at this connection.

The Portland Cement Association, under a contract to the U.S. Department of Housing and Urban Development (H-2131R), is conducting an analytical study and large scale testing of elements of large-panel concrete structures. Results to date from this research show no serious deficiencies in this Committee report.[35-38]

The PCA test series on multistory wall panel assemblies (not yet published) have shown that assemblies composed of precast wall elements and untensioned strand as reinforcement placed within the horizontal joint at every story can be detailed to perform as a cantilever (when a wall panel is ineffective) in a monolithic and ductile manner.

A second PCA test series on slab systems of large panel structures (not yet published) has demonstrated the need for large slab distortions (sag) without collapse. This is achieved by placing short lengths of untensioned strand as reinforcing in the key joints over the support, and designing for bond movement prior to tension failure of the strand.

A third PCA test series on vertical load-carrying capacity of horizontal connections of large panel structures is still in progress.

The Committee will continue its liaison with PCA on this important research and provide industry review and input to PCA.

References

35. Schultz, D. M., and Fintel, M., "Report #1, Loading Conditions," *Design and Construction of Large-Panel Concrete Structures,* Portland Cement Association, Skokie, Illinois, April 1975.
36. Fintel M., Schultz, D. M., and Iqbal, M., "Report 2: Philosophy of Structural Response to Normal and Abnormal Loads," *Design and Construction of Large Panel Concrete Structures,* Portland Cement Association, Skokie, Illinois, March 1976.
37. Kripanarayanan, K. M., and Fintel, M., "Report 3, Wall Panels: Analysis and Design Criteria," *Design and Construction of Large-Panel Concrete Structures,* Portland Cement Association, Skokie, Illinois, August 1976.
38. Schultz, D. M., and Fintel, M., "A Philosophy for Structural Integrity of Large Panel Buildings," PCI JOURNAL, V. 21, No. 3, May-June, 1976, pp. 46-49.

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