Code procedures, history, and shortcomings

Column-Slab Connections

by Jacob S. Grossman

The development of code provisions for transfer of unbalanced moments is traced. Flexible provisions of earlier codes now are found to be rigid in assigning the portions of the unbalanced moments to be transferred by flexure and eccentricity of shear. Flexibility in application, which is absolutely necessary in the design of three-dimensional structures with multiple column-to-slab connections, is not available to the designer. A research review indicates that a simplified design approach that allows the designer flexibility to develop connections without the need to revise sizes is possible and safe. Thus, alternate code provisions are urgently needed.

At the ACI convention in New York City in October 1984, an ad hoc committee chaired by Prof. James MacGregor met to discuss a special problem brought forth by practitioners attempting to use ACI Code provisions for transfer of unbalanced moments between flat slabs and supports.

The ad hoc committee was composed of members from the ACI 318 Subcommittees E, Shear and Torsion, and F, Two-Way Slabs, and Joint ACI-ASCE Committees 352, Joints in Monolithic Structures; 421, Two-Way Slabs; and 445, Shear and Torsion.

The purpose of this meeting was to coordinate the work of these groups and to arrive at a concensus of opinion as to how best to resolve the problems (specifically with ACI 318-83 Code Sections 11.11 and 11.12) that the industry was, and is still, having.

Results of some of the work performed since the October 1984 meeting was presented at the ACI Fall Convention in Seattle in November 1987 by Joint ACI-ASCE Committee 421. Some of the issues also were discussed at the ACI Spring Convention in March 1988 in a session by Joint ACI-ASCE Committee 352.

The problem, briefly stated, is as follows: ACI Code provisions for transfer of unbalanced moments presently are rigid in assigning the portions to be distributed by flexure M_f and by eccentricity of shear M_t (see Fig. 1, and consult ACI 318-83 for notation not described herein). While the portion of the unbalanced moment to be transferred by flexure can be accommodated by concentration of reinforcement, the portion assigned by the code to be transferred by eccentricity of shear often can indicate excessive shear caused by reversal of shear at Points C and D (Fig. 2).

Shear reinforcement is not commonly provided in slabs, nor do code provisions (ACI 318-83 Section 11.12.2.4) as yet permit the use of shear reinforcement (other than shear heads) to develop shear stresses resulting from moment transfer by eccentricity of shear. The options to comply generally rest with changing the size of the support or the thickness of the slab.

Often these options are not practical. For example, take the case of a thin but elongated column along the outer periphery of a building (Fig. 3). Such columns sometimes are demanded by architectural considerations to remove unsightly projections from the interior of a room. A reduction in shear capacity is required [ACI 318-83, Section 11.12.2.4.1, Eq. (11-41)] for rectangular supports with size-aspect ratio of the reaction area larger than 2. To enlarge the support to accommodate shear due to the rigid Code requirements often also increases the rectangularity aspect as well.

Another analysis also will indicate larger unbalanced moments at the larger supports. This, coupled with a greater penalty due to increase in rectangularity of support, could prevent shear capacity from converging on shear demand at the exterior edges (Points C and D in Fig. 3) of the slab. (The accuracy of this statement was verified by several members of the ad hoc committee.)

Historical overview

To better understand the problem and how it came about, a short review of recent ACI Code history is in order. Where possible and appropriate, notation has been adjusted to conform to ACI 318-83.

1956 ACI Code requirements

Chapter 10 of the 1956 ACI Code was simple and straightforward in its requirements for transfer of shear and unbalanced moment (Fig. 4). The shear capacity was limited to $0.03\ f'_c \leq 100$ psi. The critical periphery b_o was set at a distance d beyond the edges of the column or capital. "Column Head" reinforcing A_s (CH), equal to 50 percent of the negative reinforcement required for flexure in the column strip (c.s.) was to pass through the critical flexure zone ($C_2 + 2d$). If, however, only 25 percent of the column strip reinforcing was to pass through the critical flexure zone, the shear capacity was to be reduced to $0.025\ f'_c \leq 85$ psi (i. e., about a 15 percent reduction.)

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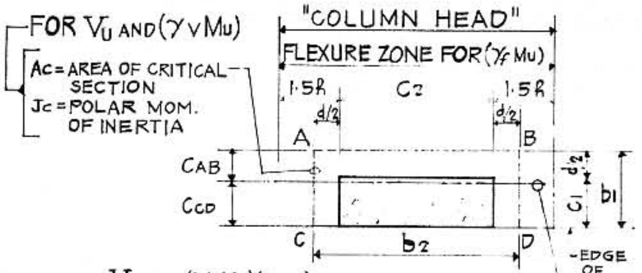
$$M_t = \gamma_v M_u \qquad M_f = \gamma_f M_u$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{b_1/b_2}} \quad \text{TRANSFER BY TORSION}$$

$$\gamma_f = 1 - \gamma_v \quad \text{TRANSFER BY FLEXURE}$$

Fig. 1—ACI Code (1986 revision): Fractions of unbalanced moment *Mu* (rigid requirements).



$$V_{u(AB)} = \frac{V_u}{A_c} + \frac{(\gamma_v M_u)(C_{AB})}{J_c} \leq V_c$$

$$V_{u(CD)} = \frac{V_u}{A_c} - \frac{(\gamma_v M_u)(C_{CD})}{J_c} \leq V_c$$

Fig. 2—1983 ACI Code (1986 revision).

This simple approach, which only indirectly considered unbalanced moment transfer, produced safe and servicable structures. The 1951 and 1949 Codes were similar to the 1956 Code. According to the report of Joint ACI-ASCE Committee 326,[1] this was the practice dating back to 1924 at which time the "Joint Committee of 1924"[2] tied shear capacity in slabs and footings to reinforcing concentration over supports.

It is important to emphasize how simply the unbalanced moment was taken care of in the 1956 and earlier codes. Assign a large portion of the required column strip reinforcement to the vicinity of the support to transfer all or most of the unbalanced moment by flexure. Maximum shear capacity is reduced (to allocate some reserve capacity) if less steel is available to transfer unbalanced moments by flexure and, therefore, larger torsional moments (which increase shear demands) are present.

The critical section for flexure (see Fig. 4), which I prefer to call the column head ($C_2 + 2d$) defined also one side of the critical periphery for direct (out of plane, excluding shear caused by unbalanced moments) diagonal shear requirements.

1963 ACI Code Requirements

The commentary of the 1963 Code, stated "that because of satisfactory service record of flat slab construction through the years, only a few changes have been made from the 1956 Code." This was a gross understatement of the revised provisions. If, as stated, all was well, why change?

But revise we apparently must! Among the revisions, shear requirements were made to conform to a report of Joint ACI-ASCE Committee 426 (formerly 326[1]) and the transfer of moment between column and slab was covered explicitly for the first time, thus opening the door to our present problem.

The 1963 Commentary to the Code discussed the issue of unbalanced moments (Section 2102(g) of the 1963 Code) in the following manner. It stated that in structures subjected to wind, earthquake, or unbalanced loading, the influence of column moments on the slab stresses was an important consideration. One such effect, resulting from the interaction of slabs and columns, was an increase in shear on one side of the column and a reduction in shear on the other.

The reference cited[3] was a paper by Distasio and van Buren, "Transfer of Bending Moment Between Flat Plate Floor and Column." It was gratifying to observe in that paper the thorough review of the various design considerations that were obviously based on observation of actual construction. Fig. 5 indicates Distasio's and van Buren's recommendations, somewhat simplified and in today's terms.

Distasio and van Buren suggested that the moment to be transferred by torsion M_t be the net leftover moment after capacities of flexural moments (M'_{AB} and M'_{CD}) on each side of the support within the critical flexural zone (the column head) were accounted for.

This was a practical solution to a tough problem. It fit well with the code provisions at that time, which allocated more direct shear capacity when larger concentrations of reinforcement are present to reduce the need to transfer a larger portion of the unbalanced moment by torsion.

This solution provided the flexibility needed by the designer to accommodate his or her design requirements. In Distasio and van Buren's paper, both the flexure and shear critical sections were identical and set at distance h − 1.5 in. (38 mm) from all faces of support.

The 1963 Code Commentary, in part on recommendations by Joint ACE-ASCE Committee 326, established the direct shear critical zone at distance $d/2$ (Fig. 6). The Code Commentary also extended (in the transverse to moment direction only) the critical zone, for the unbalanced moment transfer by both flexure and torsion, to $1.5h$ each side of the column width.

The 1963 Code was the first to recognize that the unbalanced moment transferred by torsion increased the stresses on the critical periphery of the column. The 1963 Code did not provide directions to compute the torsional moment. The Code did indicate (in agreement with van Buren's recommendations), that added flexure reinforcing could reduce the torsional requirements.

The commentary to the 1963 Code attempted to explain the process of transfering the unbalanced moment to indicate that, as the width of the critical section was increased, a larger portion of the moment was taken by bending at the end of the section and the torsion on the sides was correspondingly reduced until all the torsion was absorbed and transformed into the

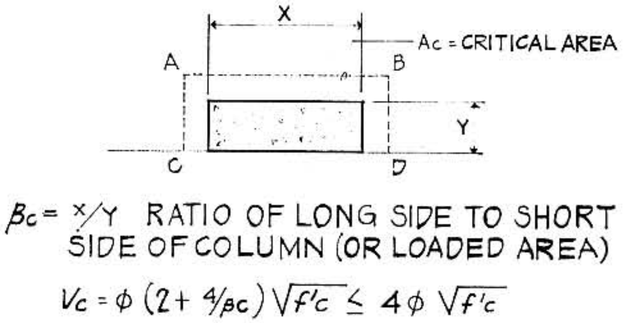


Fig. 3—1983 ACI Code (1986 revision).

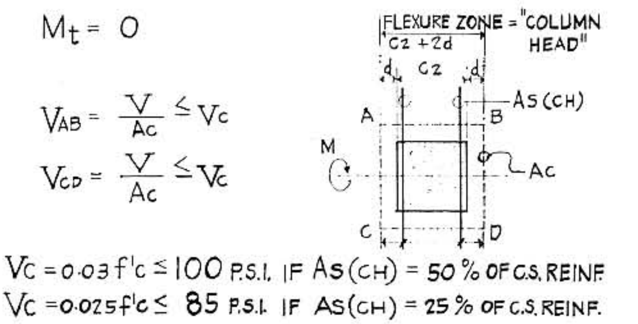


Fig. 4—1956 and earlier codes (no torsion requirements).

bending of the panel as a whole.

Shear reinforcing was considered ineffective by the 1963 Code in slabs less than 10 in. (254 mm) thick and only 50 percent effective in thicker slabs. The Joint ACI-ASCE Committee 326 report also indicated that although concentration of negative steel could prove beneficial when transfer of unbalanced moment between slabs and columns occurs, much larger concentration (as for the 1956 Code) will not increase the shear strength. The lower minimum requirements of 25 percent of the negative reinforcement of the column strip within $C_2 + 2d$ was, however, retained to insure that these bars cross the potential top surface cracks of the failure pyramid.

It is my opinion that the joint committee may have missed the underlying point here. While larger concentration of reinforcement may only marginally increase the (out-of-plane) shear capacity, the additional reinforcing would reduce the demands for larger torsional moments. Larger torsional moments do cause larger shear stresses. This was probably a contributing factor as to why earlier codes (such as the 1956 Code) had allowed an increase to out-of-plane shear capacity with larger concentration of reinforcing ratios that helped reduce any accidental or induced torsional moments at the column-slab connection.

The Joint ACI-ASCE Committee 326 reviewed the flexure-torsion division of the unbalanced moment as a function of the location of the critical section and the allowable shear capacity. For a critical section at $d/2$ and allowable shear capacity of $4\sqrt{f_c'}$ the committee suggested, based on limited data, a division of the total unbalanced moment of 20 percent by torsion and 80 percent by flexure.

It is probable that because the critical section selected by the 1963 Code differed from Committee 321 recommendations and was set to equal C_2 + 3h, that these ratios were not reported and, as stated already, the freedom of the designer to select the proportions was (thankfully) maintained. Research, however, to determine how much of the unbalanced moment is transferred by torsion was already set in motion.

1971 ACI Code Requirements

In 1968, Hanson and Hanson[4] recommended the use of the Distasio and van Buren type of analysis with 40 percent of the unbalanced moment transferred by the eccentricity of shear. This ratio is kept even today for square supports.

The 1971 Code redefined the critical zone for unbalanced moment by flexure and/or torsion to $d/2$ and empirically expressed the portion by torsion as a function of the shape of the column (Fig. 7).

The 1971 Code's extreme reduction of the transfer zone for both flexure and eccentricity of shear, from C_2 + 3h of the 1963 Code to $C_2 + d$ proved to be an "overkill" situation that was quickly (by the next Code Supplement) partially readjusted (Fig. 8). The adjustment re-established the critical zone for placement of the flexural reinforcing back to the $C_2 + 3h$. The critical zone for transfer by eccentricity of shear remained, however, confined to $C_2 + d$. These critical zones have been maintained to this day.

The 1971 Code also introduced the equivalent frame method of analysis for design, recognizing that development of moments between slabs (which are wide) and columns (which occupy only a portion of this width) must consider the flexibility of the connection. A mathematical gimmick to match limited testing of square panels introduced the concept of transverse torsional links that connect columns to slab beam strips.

This type of analysis reduces the unbalanced gravity moments, especially at edge columns—a step in the right direction. However, this method of design caused complications from the point of view that the average office had difficulty in both understanding and implementing these code requirements.

The requirement for concentration of 25 percent of column strip reinforcement over supports was, unfortunately, dropped from the 1971 Code. There is no record or discussion as to why this essential provision was removed. A recently proposed ACI Code revision to reinstate a similar requirement to assure that some additional flexural reinforcing is available in the vicinity of support to help transfer unbalanced moments did not receive the required committee support. It is the opinion of this author that concentration of reinforcing within the column head is essential for better slab behavior.

1977 ACI Code Requirements

Another critical adjustment, which put the "last nail in the coffin,"

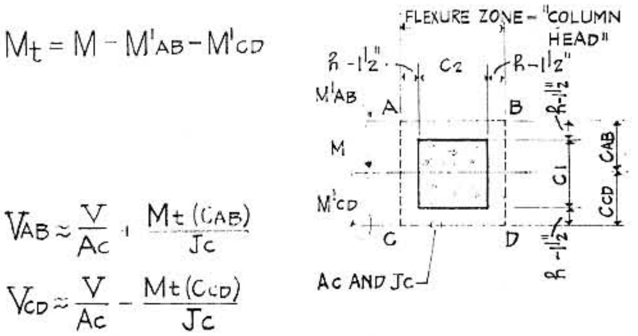


Fig. 5—Distasio and van Buren (flexible requirements).

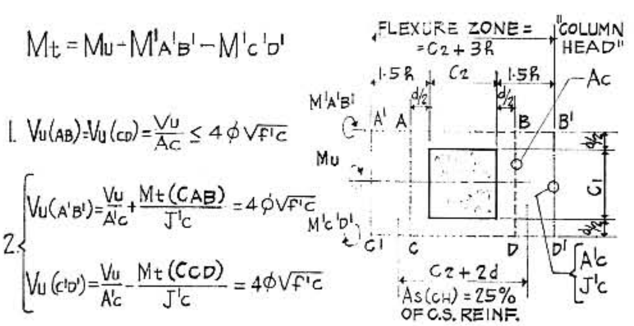


Fig. 6—1963 ACI Code (flexible requirements).

occurred when the 1977 Code reduced the punching shear strength of rectangular columns having aspect ratio $\beta_c > 2$. This was done by including the term β_c in the equation for V_c (Fig. 3). Due to lack of information concerning the moment-shear transfer of rectangular columns, this correction factor was extended also to the unbalanced moments induced shears.

1986 ACI Code Supplement and Future Codes

Present code provisions for flexure-torsion fractions of the unbalanced moment are essentially those of the 1971 Code, except that the intermediate 1986 Code supplement adjusted the fraction equations to better describe the critical zones at edge and corner columns (Fig. 1 & 2). Recently, ACI Committee 318 has approved another limit to shear capacity based on b_o/d ratios and, on a more positive note, allowed shear reinforcing even in thin slabs. Flexible code requirements for the transfer of unbalanced moments are on the agenda for future code revisions.

Consequences of rigid code provisions

As can be seen, we have progressed from a simple code (Fig. 4) that required, to transfer unbalanced moment, low shear stress due to direct shear and larger concentration of reinforcing within a column head equal to $C_2 + 2d$, to a very complex and rigid Code (Fig. 8) that now requires the use of computers to help analyze requirements.

Still, some may ask what is the problem? Many provisions in the code are not necessarily simple to use. I must emphasize that this particular problem has major implications and was caused unintentionally by the separate actions of many committees. It has not come about as the result of field observations of actual construction. The outcome is therefore a not very practical code.

Consider the following scenario, which is fairly common in consulting firms around the country: An average structure, say 40 stories high with 70 to 80 columns supporting each level, is being designed. This three-dimensional structure has over 3000 joints that have to be analyzed to ascertain that each is able to transfer shears and unbalanced moments due to gravity and lateral loads.

The code recognizes that each floor of this structure can, for the purpose of gravity load design, be analyzed separately with the ends of the columns assumed fixed. However, this is not the case for lateral load analysis, in which interaction of shear walls and frame elements and the effects of torsion must be reviewed generally for the whole three-dimensional structure.

Consider also that the determination of the capacity of the joints to transfer moment and shear will occur, in practice, at the tail end of the design process, after column sizes have been established, after slab thickness has been determined, and after the lateral load three-dimensional analysis has been performed.

Now the flat slab analysis for the combined gravity and lateral loads determines that a few (or many) of the joints are overstressed in shear due to the rigid code provisions for transferring of unbalanced moments. What to do? Thicken the slabs? Enlarge the columns?

Each of these solutions may dictate another round of analysis that could result in increased demands for these members that were made larger and, therefore, stiffer. As was already stated, it can be shown numerically that with the rectangularity penalty to shear capacity for long rectangular exterior columns (with the long side transverse to direction of the moment) there is no convergance between demand and capacity.

It is also observed that when analyzing transfer of unbalanced moments, the designer does not have the same flexibility he has in the design of, say, a concrete column or a beam, where additional reinforcement and stirrups can supplement larger requirements without the need for a change in size.

To thicken a slab from 7 to 8 in. (180 to 203 mm) could increase slab stiffness by about 50 percent and will require adjustments to distribution of lateral loads between slab members and other structural members, such as beams or shearwalls. Gravity loads also are increased.

Since slabs are not easily reinforced for shear, there is no recourse available except to redo the analysis. This obviously is not practical in the real world of construction. It is, therefore, paramount that the code be changed to allow a measure of flexibility to the designer in the design for transfer of unbalanced moments.

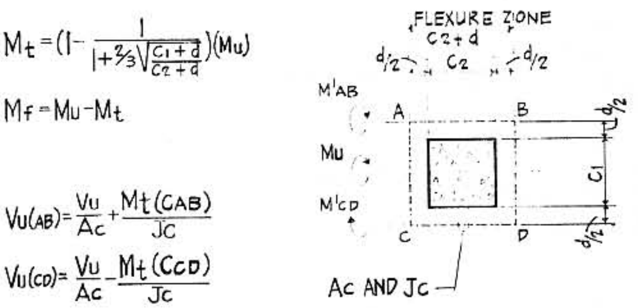


Fig. 7—1971 ACI Code (rigid requirements).

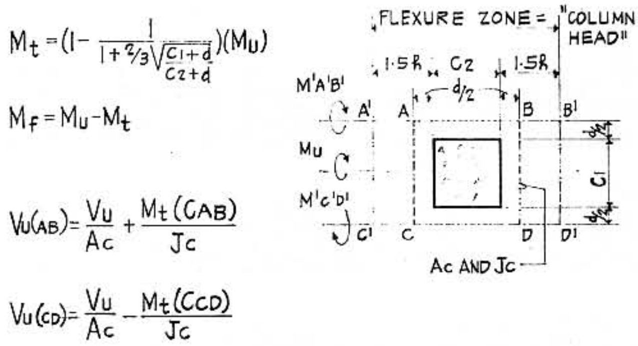


Fig. 8—1971 ACI Code Supplement (rigid requirements).

Recommendations

Common sense dictates that flexible code provisions for the transfer of unbalanced moments at the column/slab joint shoud be accompanied with details for ductility. Detailing the joint for ductile behavior will allow redistribution of moments using redundancy available in other parts of the structure or within the connection itself.

Proper detailing for ductility must provide for excess out-of-plane shear capacity, have both top and bottom reinforcement anchored in the support, and limit reinforcing in the column head to $(\rho - \rho') \leqslant \frac{3}{8}\,\rho_b$.[5] The latter limit is seldom, if ever, a problem in two way slabs. The first two requirements for ductility are good sound engineering practices that will enhance all aspects of flat slab behavior and also will discourage progressive collapse.

My firm has designed several hundred flat slab structures over the last 33 years using the varied provisions from each of the codes previously described. Regardless of the code used, no evidence of any problems was ever observed, reinforcing the contention that flexible code provisions are proper for the design of the column/slab joint.

On the basis of these observations I suggested to the ad hoc committee assembled at the 1984 Convention in New York City to review the problem the rigid code created, that a stop-gap temporary provision be adopted to allow the introduction of the $\leqslant$ sign to the equation for γ_v (Fig. 1). No decision was reached at that time except to prompt the various ACI Committees to refocus their attention on previous research and to generate new research to arrive at a solution.

The results of these investigations are gratifying and are discussed in detail in Reference 6 and 7. In a nutshell, this new look does agree with flexible code requirements. Reference 7 also studied the proposal of the introduction of the $\leqslant$ sign into the equation for γ_v in Fig. 1 and found it to be safe and to improve accuracy for edge columns when direct shear (excluding torsional moment influence) is limited not to exceed 75 percent of ϕV_c. Obviously when demands for $\gamma_v M$ are reduced, flexural demands are proportionatly increased.

The flexibility in design of the joints is an inherent property of a well detailed connection. The code should recognize this, encourage proper detailing, and allow a measure of flexibility in the design of the column-slab connection. This step is necessary to keep our code practical and help encourage the use of the most economical structural system—that of the flat plate construction.

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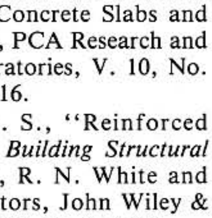

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