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# Confinement of Reinforced Concrete Columns with Welded Reinforcement Grids

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*Experimental research was conducted to investigate structural performance of reinforced concrete columns confined with welded grids. Ten full-scale columns with different volumetric ratios, spacing, and arrangement of welded reinforcement grids were tested under simulated seismic loading. The columns were subjected to constant axial compression accompanied by incrementally increasing lateral deformation reversals. Results indicate that welded reinforcement grids can be used effectively as confinement reinforcement, provided the steel used has sufficient ductility and the welding process employed does not alter the strength and elongation characteristics of steel. The grids improved the structural performance of columns, which developed lateral drift ratios in excess of three percent with the spacing and volumetric ratio of transverse reinforcement similar to those required by the ACI 318-95 Building Code. Drift capacity further increased when grids with larger number of cells were used. Furthermore, the use of grids reduced congestion of reinforcement while the dimensional accuracy provided effective support to longitudinal reinforcement.*

**Keywords**: columns; confined concrete; ductility; earthquake-resistant structures; structural design; welded wire fabric.

## INTRODUCTION

Reinforced concrete columns in seismically active regions are required to be confined with properly designed and detailed transverse reinforcement. Confinement of concrete improves strength and deformability of columns. This, in turn, ensures the strength and stability of the entire structure. Conventional confinement reinforcement used for square and rectangular columns consist of perimeter hoops, overlapping hoops, and crossties. The requirements of confinement reinforcement for such columns often result in high volumetric ratios, close spacings, overlapping of hoops, and bends and bend extensions. Although these requirements are necessary for improved behavior of earthquake resistant columns, they may lead to the congestion of the column cage and create constructibility problems.

An alternative to using conventional tie reinforcement is to use a welded reinforcement grid, prefabricated to the required size, arrangement, and volumetric ratio of transverse reinforcement. Welded reinforcement grids offer easy cage assembly, dimensional accuracy for proper support of longitudinal reinforcement, and savings in materials because of the elimination of laps at tie ends and bend extensions. Furthermore, closely spaced grid legs in the cross-sectional plane result in improved distribution of confinement pressure around the core, improving the behavior of core concrete.

The use of welded reinforcement grids as column confinement steel was investigated in the current research program. A total of ten first story columns was tested under different levels of constant axial compression and incrementally increasing lateral deformation reversals. The results are presented and discussed in this paper.

## RESEARCH SIGNIFICANCE

Confinement reinforcement requirements specified in current building codes often result in congestion of column cages when implemented using conventional overlapping hoops and crossties with 135 deg bends and bend extensions. Furthermore, the required transverse confinement reinforcement detailing for earthquake resistant construction may become prohibitively complex. Alternative reinforcement, in the form of welded grids, was considered in this research program. The performance and feasibility of reinforced concrete columns confined with welded reinforcement grids were investigated. The results are intended to improve the construction process for seismic resistant columns.

## EXPERIMENTAL PROGRAM

Ten column specimens, representing part of a first story column between the footing and point of inflection, were designed, constructed, and tested. The columns had a square section with 350 mm (14 in.) cross-sectional dimension. Four different arrangements of reinforcement were used, consisting of 4-cell and 9-cell grids, manufactured from either 9.53 mm (3/8 in.) or 6.60 mm (1/4 in.) reinforcement. Two different grid spacings and five different volumetric ratios of transverse reinforcement were employed as test variables. Figures 1 and 2 illustrate the geometry and reinforcement arrangements. Table 1 provides a summary of material and geometric properties for all columns.

### Material properties

Ready-mixed concrete, with a 28-day strength of 34 MPa (4900 psi), slump of 80 mm (3 in.), and maximum aggregate size of 20 mm (3/4 in.) was used in all columns. Fig. 3(a) illustrates the stress-strain relationship of the concrete, determined through standard cylinder tests. The column tests were conducted a month after casting the columns. Therefore, the 28-day strength was representative of the concrete strength in the columns at the time of testing.

Three different sizes of Grade 400 MPa (58 ksi) steel bars were used as longitudinal reinforcement, consisting of no. 15 (16 mm diameter), no. 20 (19.5 mm diameter) and no. 30 (29.9 mm diameter) bars. Coupons were taken from each batch and tested to establish stress-strain relationships. Fig. 3(b) illustrates the stress-strain relationships for longitudinal reinforcement.

Welded steel grids, used as transverse confinement reinforcement, were manufactured to have a square shape with an

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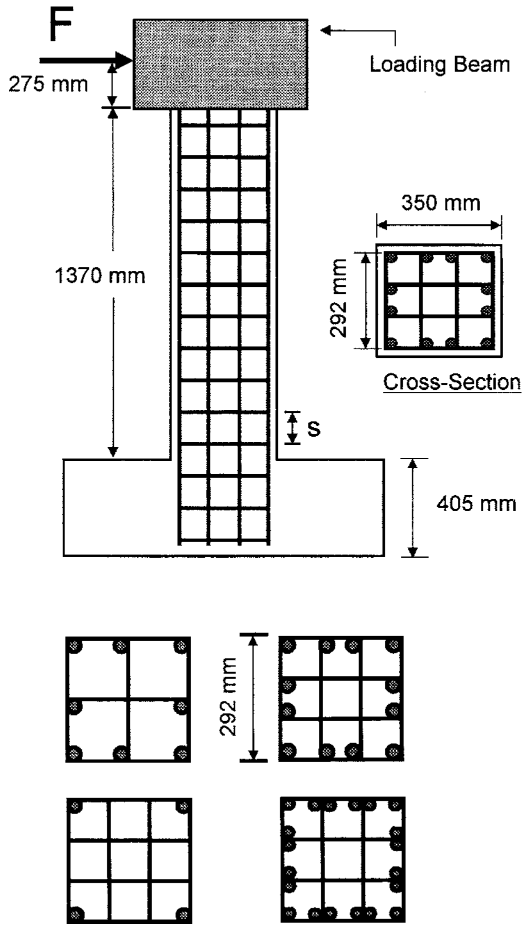



*Fig. 1—Geometry of column specimens.*

exterior dimension of 292 mm (11.5 in.) out-to-out. Three different grid types were used, consisting of: (1) 9.53-mm (3/8 in.) diameter reinforcement welded to form 4 equal-size square grids; (2) 9.53-mm (3/8 in.) diameter reinforcement welded to form 9 equal size square grids; and (3) 6.60-mm (1/4-in.) diameter reinforcement welded to form 9 equal-size square grids. The reinforcement was positioned to form the required grid dimensions and pressed together and spot welded at each intersection. Details of the grid geometry are shown in Fig. 4.

Coupons were taken from each type of grid such that a welded intersection and part of a crossbar were included in the middle of the coupon. Strain gages were placed on each coupon: one immediately behind the weld and the other between the welds. An extensometer with a 51-mm (2-in.) gage length was also placed over the weld location. Extensometer readings were verified against the corresponding strain gage readings, and were found to give good correlations until the strain gage ceased to function beyond its limit. The gage readings between the welds also agreed closely with those recorded near the weld. Yielding of coupons consistently occurred to the side of the weld where "necking"

*Fig. 2(a)—8-bar reinforcement arrangement with 4-cell grids.*

*Fig. 2 (b)—12-bar reinforcement arrangement with 9-cell grids.*

*Fig. 2(c)—4-bar reinforcement arrangement with 9-cell grids.*

*Fig. 2 (d)—20-bar reinforcing arrangement with 9-cell grids.*

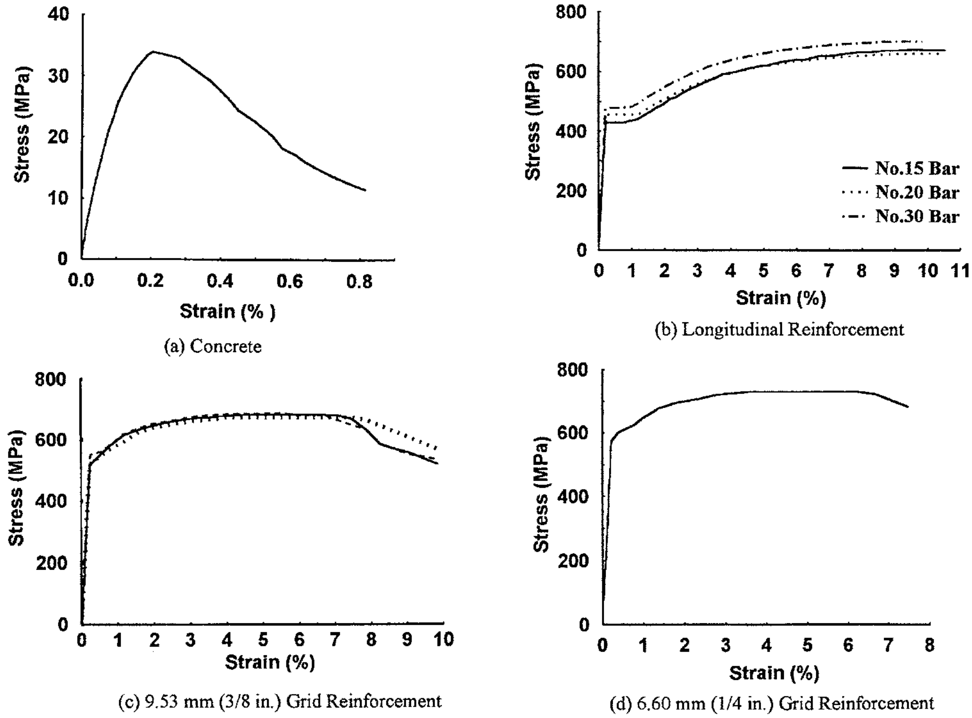



*Fig. 3—Material stress-strain relationships: (a) concrete; (b) longitudinal reinforcement; (c) 9.53 mm (3/8 in.) grid reinforcement; and (d) 6.60 mm (1/4 in.) grid reinforcement.*

**Table 1—Properties of test specimens**

| Column label | Reinforcement arrangement | $\rho$, percent | $b_d$, mm | $s$, mm | $f_{yt}$, MPa | $\rho_s$, percent | $(\rho_s)_{ACI}$, percent | $\rho_s/(\rho_s)_{ACI}$ | $P$, kN | $P/P_o$ |
|---|---|---|---|---|---|---|---|---|---|---|
| BG-1 | 8 - no. 20 | 1.96 | 9.53 | 152 | 570 | 1.00 | 1.55 | 0.65 | 1782 | 0.39 |
| BG-2 | 8 - no. 20 | 1.96 | 9.53 | 76 | 570 | 2.00 | 1.55 | 1.29 | 1782 | 0.39 |
| BG-3 | 8 - no. 20 | 1.96 | 9.53 | 76 | 570 | 2.00 | 1.55 | 1.29 | 831 | 0.18 |
| BG-4 | 12 - no. 20 | 2.94 | 9.53 | 152 | 570 | 1.33 | 1.55 | 0.86 | 1923 | 0.38 |
| BG-5 | 12 - no. 20 | 2.94 | 9.53 | 76 | 570 | 2.66 | 1.55 | 1.72 | 1923 | 0.38 |
| BG-6 | 4 - no. 30 | 2.29 | 9.53 | 76 | 570 | 2.66 | 1.55 | 1.72 | 1900 | 0.40 |
| BG-7 | 12 - no. 20 | 2.94 | 6.60 | 76 | 580 | 1.26 | 1.52 | 0.83 | 1923 | 0.38 |
| BG-8 | 12 - no. 20 | 2.94 | 6.60 | 76 | 580 | 1.26 | 1.52 | 0.83 | 961 | 0.19 |
| BG-9 | 20 - no. 15 | 3.26 | 6.60 | 76 | 580 | 1.26 | 1.52 | 0.83 | 1923 | 0.38 |
| BG-10 | 20 - no. 15 | 3.26 | 9.53 | 76 | 570 | 2.66 | 1.55 | 1.72 | 1923 | 0.38 |

Note: $P_0 = 0.85 f'_c (A_g - A_s) + A_s f_y$.
$(\rho_s)_{ACI}$ is based on actual $f_{yt}$ reported in the table.
1 in. = 25.5 mm; 1MPa = 0.145 ksi; 1 kN = 0.225 kip.

of the coupons was visible. The coupons consistently ruptured immediately to one side of the weld at approximately seven percent strain for 6.60 mm (1/4 in.) reinforcement and ten percent strain for 9.35 mm (3/8 in.) reinforcement. Fig. 5 shows the instrumentation and failure location of coupons. The same type of failure was also observed during column tests when two of the ten columns developed grid failures at the end of the test following buckling of longitudinal reinforcement. The coupon test results are plotted in Fig. 3(c) and 3(d) for both sizes of grid reinforcement.

## Instrumentation, test setup, and loading program

The columns were instrumented with Linear Variable Displacement Transducers (LVDT) and strain gages to measure horizontal displacements, rotations of the hinging region, and steel strains. The lowest two grids above the column-footing interface were instrumented with strain gages. Two longitudinal bars in each column were also instrumented with strain gages, immediately below the column critical section. All instrumentation was connected to a data acquisition system and a microcomputer for data recording.

Three 1000 kN (225 kip) servo-computer-controlled MTS hydraulic actuators were used to apply the loads. Two of the actuators were positioned vertically, one on each side of the column, to apply constant axial compression. Eight columns were subjected to approximately 40 percent of their nominal axial load strength ($P_0$). Two columns were tested under approximately 20 percent of their nominal axial load strength to investigate the effect of axial compression on column deformability. The third actuator was placed horizontally for the application of lateral deformation reversals. This actuator was connected to a steel loading beam at one end and a reaction frame at the other end (Fig. 6). Lateral deformation reversals consisted of three full cycles at each deformation level, starting with 0.5 percent drift and

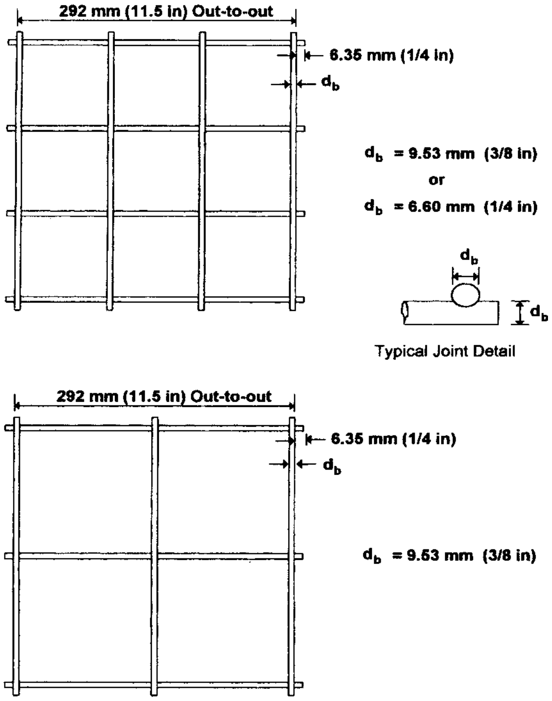



*Fig. 4—Geometric details of welded reinforcement grids.*

continuing with 1 percent, 2 percent, 3 percent, etc., until a significant drop was observed in the load resistance.

## OBSERVED BEHAVIOR AND TEST RESULTS

All columns showed similar behavior initially. The observed damage at 0.5 percent drift was limited to minor flexural cracking. The flexural cracks increased and diagonal shear cracks began forming at one percent drift. All columns developed 1 percent drift without any concrete crushing or loss in strength. Maximum load resistance was attained at 2 percent lateral drift. The flexural strength recorded at this load stage was higher than the nominal moment capacity computed based on the ACI 318-95 Building Code,[1] reflecting strength enhancement due to concrete confinement. Table 2 summarizes the strength values. Some spalling of cover concrete near the base was observed at 2 percent drift along with increased flexural and shear cracking. Column behavior beyond this load stage depended on the magnitude of axial compression and the amount and arrangement of transverse reinforcement.

Column BG-1 was reinforced with 4-cell grids and eight longitudinal bars with a volumetric ratio of transverse reinforcement equal to 65 percent of that required by ACI 318-95. The grid spacing was 152 mm (6 in.), which was twice the maximum spacing permitted by the code. The hysteretic moment-displacement relationship shown in Fig. 7(a)

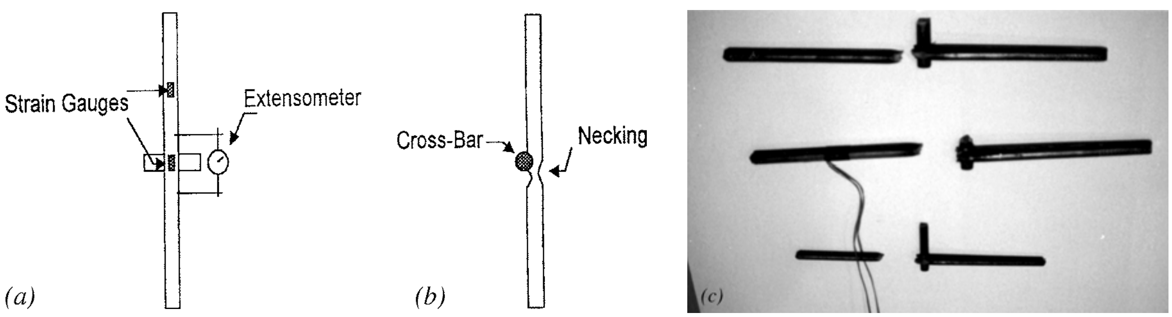



*Fig. 5—Grid coupons and rupturing of welded reinforcement: (a) instrumentation of typical coupon; (b) location of "necking;" and (c) typical coupons failures.*

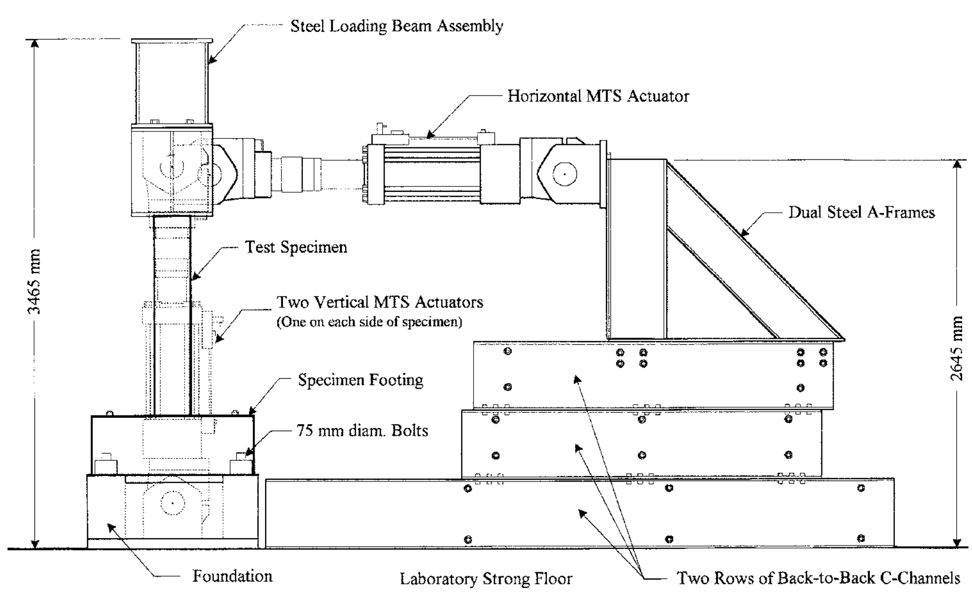



*Fig. 6—Side elevation of test setup.*

**Table 2—Summary of strength and drift capacities**

| | Column moment resistance | | Column force resistance | | ACI 318-95 |
|---|---|---|---|---|---|
| Column | $(M_{test})_{Max}$, kNm | Minimum drift at 80 percent $(M_{test})_{Max}$, percent | $(F_{test})_{Max}$, kN | Minimum drift at 80 percent $(F_{test})_{Max}$, percent | $M_n$, kNm |
| BG-1 | 320 | 1 | 169 | 1 | 236 |
| BG-2 | 311 | 3 | 164 | 2 | 236 |
| BG-3 | 260 | 7 | 143 | 4 | 214 |
| BG-4 | 338 | 2 | 177 | 2 | 262 |
| BG-5 | 363 | 5 | 190 | 3 | 262 |
| BG-6 | 351 | 5 | 180 | 3 | 266 |
| BG-7 | 348 | 4 | 184 | 3 | 271 |
| BG-8 | 326 | 6 | 179 | 4 | 279 |
| BG-9 | 359 | 3 | 189 | 3 | 270 |
| BG-10 | 366 | 5 | 192 | 3 | 268 |

Notes: $(M_{test})_{Max}$ and $(F_{test})_{Max}$ quantities represent average of maximum moments and lateral forces recorded in each direction of loading;Minimum drift capacity is obtained from respective hysteretic relationship as lateral drift at which column has completed a minimum of three deformation cycles with no more than 20 percent drop in strength. Readings are rounded off to smaller whole number; 1kN.m = 0.74 ft-kip; 1 kN = 0.225 kip.

suggests that the column experienced rapid strength degradation at 2 percent drift because of lack of confinement. However, the grids were able to maintain their integrity and fulfill their functions until after failure at 3 percent lateral drift, as shown in Fig. 8(a).

Column BG-2 was companion to BG-1, with a reduced grid spacing of 76 mm (3 in.) and an increased volumetric ratio of 2.0 percent that was equal to 129 percent of that required by ACI 318-95. The behavior improved significantly as shown in Fig. 7(b) and the column could sustain deformation cycles at 3 percent drift without a significant drop in moment strength, and with some strength decay during 4 percent and 5 percent drift cycles. The failure occurred during the second cycle at 5 percent drift due to longitudinal bar buckling. Fig. 8(b) illustrates that the grids remained intact even after the buckling of longitudinal reinforcement. The reduction in lateral load resistance was higher due to the P-Δ effect, and the column developed a minimum of 2 percent drift at approximately 20 percent decay in lateral load resistance. This test suggests that columns confined with 4-cell welded grids, conforming the spacing and volumetric ratio requirements of ACI 318-95 possess sufficient ductility even under a high level of axial compression.

The previous two columns were tested under a constant axial compression of approximately 40 percent of their nominal axial load strength. The effect of axial compression was investigated by testing Column BG-3 under approximately 20 percent of its concentric capacity. This column was identical to Column BG-2. The moment-displacement hysteretic relationship shown in Fig. 7(c) indicates extremely ductile behavior, implying that the confinement steel requirements can be relaxed for columns under low axial compression. The column developed 7 percent lateral drift at 20 percent decay in moment strength, and 4 percent drift at 20 percent decay in lateral force resistance.

Column BG-4 was confined with an improved reinforcement arrangement, consisting of 9-cell grids and 12 longitudinal bars, resulting in 86 percent of the volumetric ratio required by ACI 318-95. However, the spacing of grids was 152 mm (6 in.), which was twice the spacing permitted by the code. The column sustained 2 percent lateral drift with little strength decay, and failed by buckling of the longitudinal reinforcement during the second cycle at 4 percent drift. This is illustrated in the hysteretic relationship shown in Fig. 7(d). The superior performance of this column over the companion column BG-1 clearly shows the importance of the reinforcement arrangement and the confinement efficiency of 9-cell grids as compared with 4-cell grids. Fig. 8(c) shows the column hinging region at failure and the effective support provided by the grid reinforcement without any sign of failure. Further improvement in column deformability was achieved when the grid spacing was reduced to 76 mm (3 in.) to comply with ACI 318-95. Column BG-5, with the identical reinforcement arrangement, except with reduced grid spacing of 76 mm (3 in.) and a resulting increase in the volumetric ratio of transverse steel, showed stable hysteresis loops in the moment-displacement relationship, up to 5 percent lateral drift. Fig. 8(d) shows the column hinging region at 6 percent drift, illustrating continued confinement and bar support provided by the grids at this high drift level. The column failed during the first cycle at 7 percent drift by buckling of longitudinal reinforcement. The lateral force resistance showed a drop of 20 percent at 3 percent lateral drift, essentially due to the P-Δ effect. Fig. 7(e) illustrates the hysteretic relationship recorded for column BG-5.

The significance of transverse reinforcement arrangement was further investigated by testing Column BG-6 which was reinforced with four longitudinal corner bars tied with 9-cell grids. This arrangement was unique in a sense that the perimeter grid joints between the corners did not engage in longitudinal reinforcement. The behavior was similar to that of companion Column BG-5 with 9-cell grids and 12 longitudinal bars. The moment-displacement hysteretic relationship shown in Fig. 7(f) illustrates that the column developed 5 percent drift prior to 20 percent strength decay in moment resistance, and 3 percent drift at approximately the same level of decay in lateral force resistance. This result has a significant implication on application to practice, suggesting that the grids can provide the required lateral restraint at welded grid joints as if there are crossties, even if there is no longitudinal reinforcement to engage in. The close spacing of transverse reinforcement within the cross-sectional plane improved the distribution of lateral confinement pressure without the presence of longitudinal reinforcement. However, additional tests may be necessary to confirm this point further, as this specimen was the only column confined with this type of arrangement. Column BG-6 was one of the

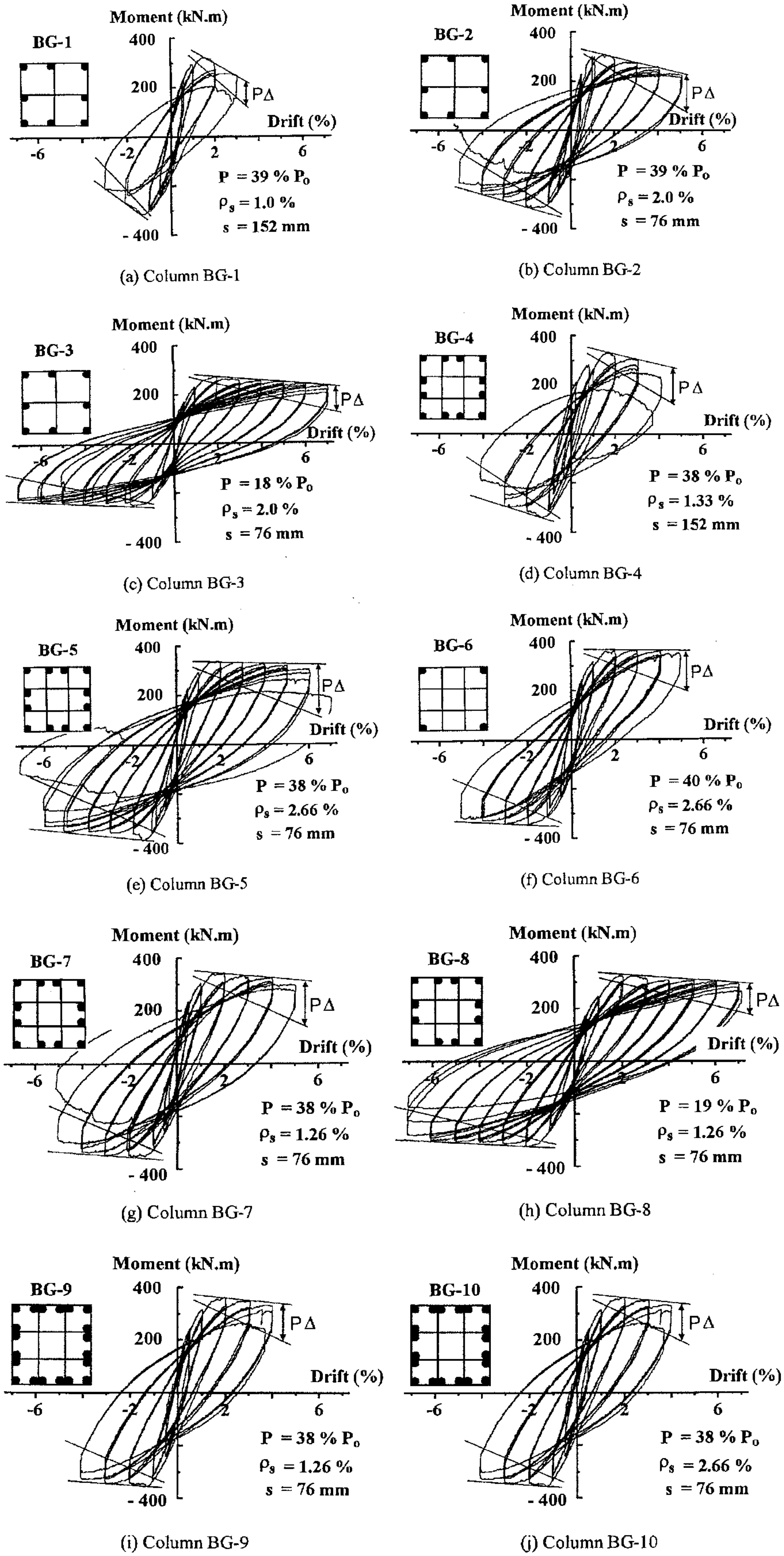
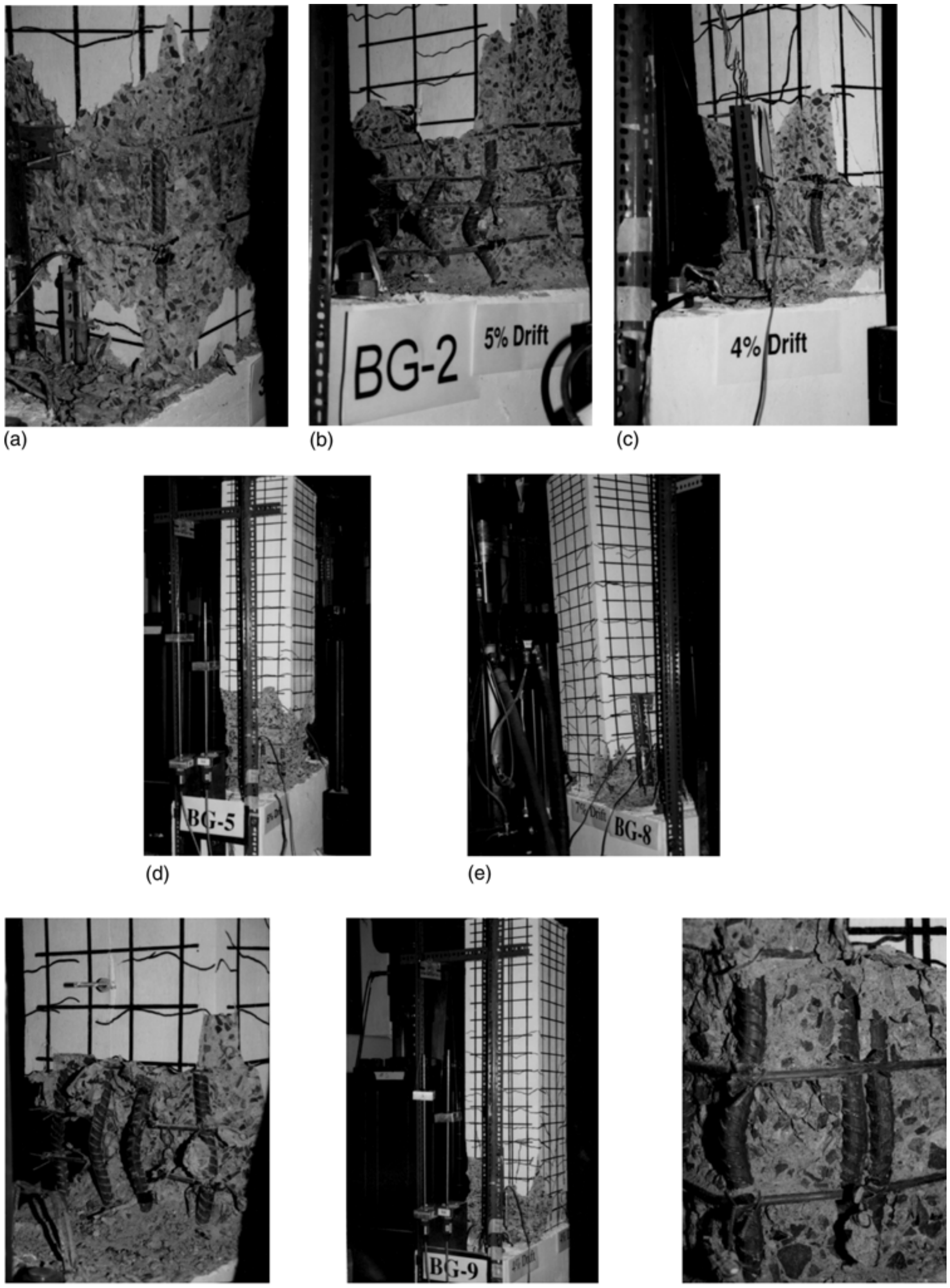



*Fig. 7—Experimentally recorded hysteretic moment-displacement relationships: (a) Column BG-1; (b) Column BG-2; (c) Column BG-3; (d) Column BG-4; (e) Column BG-5; (f) Column BG-6; (g) Column BG-7; (h) Column BG-8; (i) Column BG-9; and (j) Column BG-10.*




(a) (b) (c) (d) (e) (f) (g) (h)

*Fig. 8—Observed damage in selected columns: (a) BG-1; failure zone at 3 percent drift; (b) BG-2; failure zone at 5 percent drift; (c) BG-4; failure zone at 4 percent drift; (d) BG-5; hinging region at 6 percent; (e) BG-8 at 7 percent drift; (f) failure of BG-8 grids at end of test; (g) BG-9 at 4 percent drift; and (h) BG-9 grids remaining in tact at bar buckling.*

earlier columns tested in the experimental program, and the test was discontinued prematurely due to a safety concern at a high drift level of 5 percent before developing a substantial loss in strength.

It becomes conceivable that the volumetric steel-to-concrete ratio requirement may be lowered for columns with an improved grid arrangement without adversely affecting the column behavior. This point was investigated by testing columns containing grids with a higher number of cells but a reduced volumetric steel ratio. Columns BG-7 and BG-8 were reinforced with 9-cell grids and 12 longitudinal bars where the grids were made out of smaller size reinforcement with a diameter of 6.60 mm (1/4 in.). This resulted in a reduced volumetric ratio of 1.26 percent, which was equal to 83 percent of that required by ACI 318-95. Column BG-7 was tested under a constant axial compressive load of approximately 40 percent of its nominal axial load strength, whereas Column BG-8 was subjected to approximately 20 percent of its nominal axial load strength. The hysteretic relationships in Fig. 7(g) and 7(h) show that both columns behaved in a ductile manner. The moment resistance of Column BG-7 was maintained until 4 percent lateral drift without significant strength decay, while the lateral force resistance showed 2 percent drift at 20 percent strength decay. Column BG-8, under reduced axial load, showed even better performance with no strength decay in moment resistance up to 6 percent drift, although the lateral load resistance dropped to 80 percent of its peak resistance at 4 percent drift due to the P-$\Delta$ effect. Fig. 8(e) illustrates the column hinging region at 7 percent lateral drift. The longitudinal bars buckled during the second cycle at 7 percent drift. When the column was forced to deform beyond this limiting deformation level, two of the grids in the hinging region fractured near the perimeter welds. Fig. 8(f) shows a close-up view of the ruptured grids. The latter two tests demonstrated that columns confined with 9-cell grids, conforming the spacing requirements of ACI 318-95 showed ductile behavior even with 84 percent of the volumetric steel ratio required by the code.

Columns BG-9 and BG-10 were tested to investigate the performance of columns with two longitudinal bars at each perimeter grid joint, one on each side. This resulted in 20 longitudinal bars for 9-cell grids. These tests were also expected to provide information on the performance of grid welds in supporting longitudinal reinforcement when two bars were restrained by a single joint. The grids used in BG-9 were produced using 6.60 mm (1/4 in.) diameter reinforcements, resulting in 83 percent of the volumetric ratio required by ACI 318-95, while the grids used in BG-10 had 9.53 mm (3/8 in.) reinforcements, resulting in an increased volumetric ratio of 129 percent of that required by ACI 318-95. The results indicate that the columns behaved in a ductile manner, as indicated in Fig. 7(i) and 7(j), without any grid failure. Column BG-9 could develop 4 percent drift without a significant loss in strength and failed during cycles at 4 percent drift. This column was a companion to Column BG-7 and developed a similar ductility at 20 percent strength decay. However, BG-9 showed rapid strength degradation during subsequent cycles due to the instability of the longitudinal reinforcement. This may be attributed to the smaller size longitudinal bars used in this column, which were more susceptible to buckling. Fig. 8(g) and 8(h) show the column hinging region at 4 percent lateral drift. The close up view shown in Fig. 8(h) illustrates that the grids maintained their integrity and fulfilled their functions until after the buckling of the longitudinal reinforcement. Therefore, the use of double longitudinal bars in perimeter grid corners was not believed to be the reason for the rapid strength decay observed beyond 4 percent lateral drift. Column BG-10 showed better performance than Column BG-9, developing 5 percent drift without a significant loss in moment strength. The lateral load capacity was reduced due to the P-$\Delta$ effect, reaching approximately 80 percent of peak resistance at approximately 3 percent drift. The superior performance of this column can be explained by the increased volumetric ratio of transverse reinforcement. Table 2 summarizes lateral drift capacities recorded during each test.

### Moment-rotation relationships

Columns were instrumented for rotation measurements of the hinging region. The hinging region was assumed to extend from the column-footing interface to a section 350 mm (14 in.) above, having a length equal to the cross-sectional dimension of the column. The rotation of the hinging region relative to the column footing was measured as total rotation. Column rotation resulting from the slippage and/or extension of longitudinal reinforcement within the footing (anchorage slip) was also measured and formed a component of total rotation. This component reflected the rigid body rotation associated with the penetration of yielding of longitudinal reinforcement into the footing and resulting crack at the column-footing interface. The difference between the total and anchorage slip rotations gave the rotation due to flexure. Flexural rotation represented the rotation of the column segment between the column critical section near the base and the top of the column hinging region 350 mm (14 in.) above.

The anchorage slip becomes significant only after the strain hardening of the longitudinal reinforcement is attained at the critical section.[2] Therefore, this deformation component gains importance at high inelastic deformation levels, especially in beams where the axial force is negligible. Columns under low axial compression may also show significant column rotation due to an anchorage slip.[3] Most of the columns tested in this investigation were subjected to high axial compression. Therefore, the anchorage slip component formed only a small portion of total rotation. Fig. 9 illustrates the hysteretic moment-rotation relationships recorded for two columns with two different levels of axial compression. Detailed descriptions of rotation measurements, along with complete data recorded, are presented in Reference 4.

### Steel strain measurements

The columns were instrumented with electric resistance strain gages, placed on both the longitudinal and transverse reinforcement. Variation of strain in a typical longitudinal reinforcement near the critical section is illustrated in Fig. 10. The data showed that the extreme layer of column reinforcement yielded at 1 percent lateral drift. This observation was consistent among all the columns. The strain measurements taken on reinforcement grids indicated strains of approximately 0.1 percent at 1 percent drift. Yielding of reinforcement grids was generally recorded at or beyond 3 percent lateral drift. The variation of strain in one of the reinforcement grids is illustrated in Fig. 11. More detailed discussion of strain readings is provided in Reference 4.

## COMPARISONS WITH CONVENTIONAL TRANSVERSE REINFORCEMENT

Tests of companion columns with conventional hoops and crossties were not included in the experimental program. Therefore, force-displacement relationships of columns with conventional tie reinforcement were generated analytically. The analytical relationships were computed for lateral force-lateral displacement relationships under monotonic loading, including the P-$\Delta$ effect, and compared with the envelopes of experimentally obtained hysteretic relationships for columns confined with welded grids. An important aspect of a such comparison is the validity and reliability of the analytical tools and procedures used in establishing inelastic force-displacement relationships. The procedures employed included flexural analysis with confined concrete model developed by Saatcioglu and Razvi[5] and steel stress-strain relationships with strain hardening, recorded experimentally. The progression and gradual plastification of hinging regions were considered following the algorithm put forward by Razvi and Saatcioglu.[6] Deformations due to an anchorage slip were computed using the analytical model suggested by Alsiwat and Saatcioglu.[2] The analytical procedure had been verified extensively against a large number of column tests conducted under concentric, eccentric and reversed cyclic loadings, and had been reported to produce excellent agreement with test data.[5-7] Analytical force-displacement relationships were developed for columns confined with conventional hoops

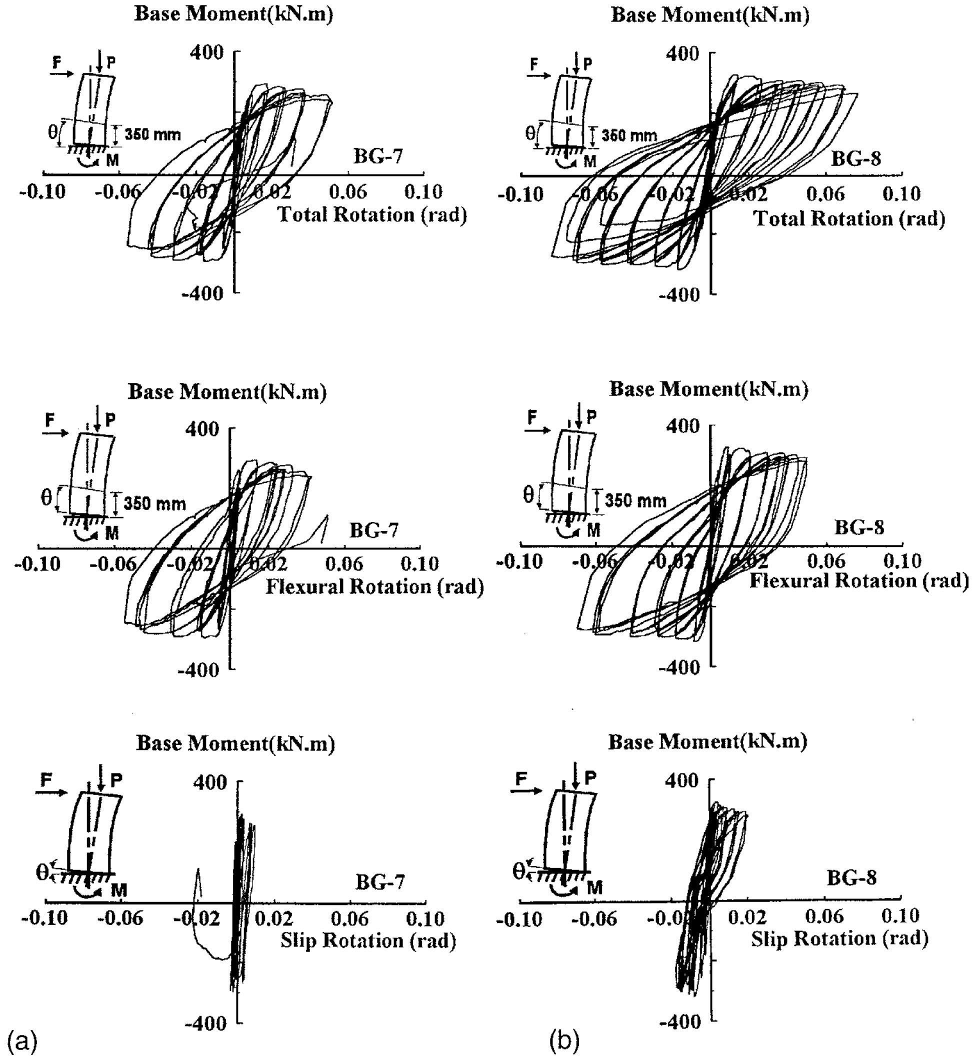



*Fig. 9—Moment-rotation relationships for two columns: (a) Column BG-7 (*P *= 38 percent* $P_o$*); and (b) Column BG-8 (*P *= 18 percent* $P_o$*).*

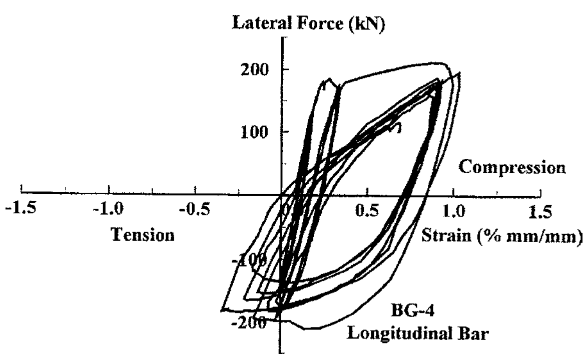



*Fig. 10—Strain readings recorded on longitudinal bar near critical section of BG-4.*

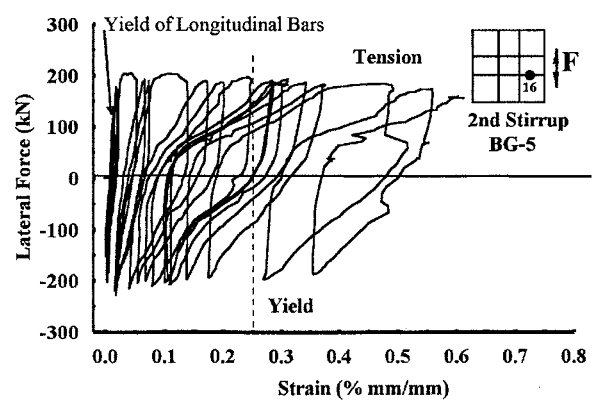



*Fig. 11—Strain readings recorded on reinforcement grid of Column BG-5.*

and crossties having equal volumetric ratio, spacing, and arrangement as those of the welded grids. The only exception was the column compared with BG-6 confined with perimeter hoops of an equal spacing and volumetric ratio as the 9-cell grids used in BG-6, since conventional crossties could not be provided with only four longitudinal bars. Fig. 12 shows the comparisons. The results indicate that while the overall behavior of columns with both types of transverse reinforcement were similar, the columns with welded grids showed slightly improved behavior, and developed between 5 percent to 10 percent higher strengths. The slight improvement in behavior may be attributed to the improved confinement characteristics of grids associated with increased rigidity resulting from the welds that produce a rigid joint at every cross bar location. This point is a subject for further research by the authors.

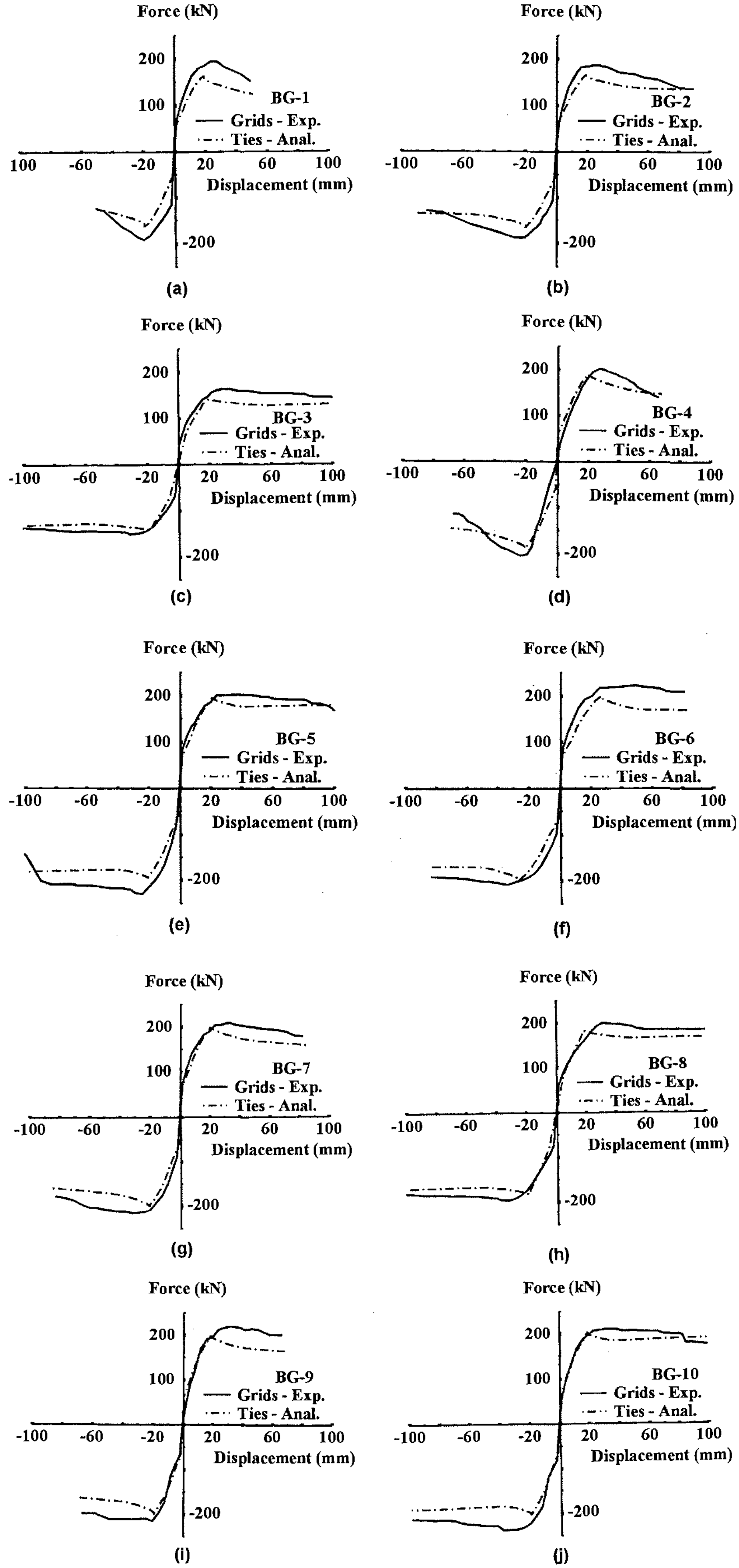



*Fig. 12—Comparisons of force-displacement relationships.*

## CONCLUSIONS

The following conclusions may be drawn from the investigation reported in this paper.

1. Welded reinforcement grids can be used as transverse confinement reinforcement if the welding process does not adversely affect strength and ductility of reinforcement and the steel has sufficient elongation. The reinforcement grids used in this research program met these requirements. While the coupon tests showed approximately 7 to 10 percent tensile strains at rupture, maximum tensile strains recorded during column tests were below 1 percent at 4 percent lateral drift when the steel grade ranged between 570 and 580 MPa, implying that the maximum elongation requirement may be much lower than the rupturing strain recorded during coupon tests. No grid failure was observed during column tests before either a longitudinal bar buckling in compression or a longitudinal bar fracturing in tension. Only two columns exhibited grid fracture beyond these limiting load conditions.

2. Columns confined with reinforcement grids showed ductile response, developing lateral drifts equal to or greater than those expected in columns confined with conventional hoops and crossties of equal volumetric ratio, grade, arrangement, and spacing. Column BG-1 was the only column that had a limited drift capacity of approximately 1 percent, mainly because this column had a grid spacing equal to one half of its cross-sectional dimension and the volumetric ratio equal to 65 percent of that required by ACI 318-95. All other columns showed minimum drift capacities of 2 percent and higher, even when the $P$-$\Delta$ effect was considered. Columns conforming to the spacing requirement of ACI 318-95 with volumetric ratios ranging between 83 and 129 percent of that required by the code showed ductile response even under a high axial compression of approximately 40 percent of nominal axial strength, and developing drifts of 3 to 6 percent prior to a 20 percent degradation in moment strength. The same columns developed 2 to 4 percent drifts prior to a similar degradation in lateral force resistance. Companion columns tested under 20 percent of nominal axial strength showed much higher deformabilities, developing 6 to 7 percent drifts prior to 20 percent degradation in moment strength and 4 percent drift at 20 percent decay in lateral force resistance.

3. For the same volumetric ratio and spacing, 9-cell grids produced higher deformability than 4-cell grids.

4. Welded grids with closely spaced reinforcement in the cross-sectional plane can be used to get the beneficial effects of well distributed and laterally supported longitudinal reinforcement with four corner bars only. Column BG-6 with four corner longitudinal reinforcement and 9-cell grids (having four welded nodal points per side) showed ductile response similar to that expected from a 12-bar arrangement with conventional ties. This observation, however, is based on only one column test, and further verification may be necessary.

5. Columns with longitudinal bars placed on both sides of a perimeter grid joint performed as good as those with a single longitudinal bar placed at each perimeter grid joint. This suggests that in cases where the use of bundled bars is considered, or many smaller size bars are preferred, welded grids may be used with longitudinal bars on both sides of grid joints.

6. Welded grids offer an economic alternative to conventional ties with reduced construction time, especially for earthquake-resistant construction where the tie details may be prohibitively complex. The experience with cage assembly in a laboratory environment has been most favorable in terms of dimensional tolerances and speed of construction. This may become a more significant asset in the field. Therefore, construction advantages, combined with superior performance observed in column tests make welded reinforcement grids a viable alternative to conventional ties. However, the conclusions drawn in this investigation may be limited to the materials considered in this research project. The extension of the results to other columns, especially with welded reinforcement grids having significantly different material properties, should be done with caution. Deformability of grid reinforcement and the strength of welded joints remain to be two important parameters to be specified before such a material can be authorized for use in reinforced concrete columns. The observations made in this research program indicate that welded grids must have: (a) welded joints stronger that the steel itself; and (b) a minimum elongation of 4 percent, when determined by a standard coupon test where the coupon contains a welded joint in the center.

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## NOTATION

| | | |
|---|---|---|
| $A_s$ | = | total area of longitudinal steel |
| $A_g$ | = | gross cross-sectional area of concrete |
| $b_d$ | = | diameter of transverse reinforcement |
| $f'_c$ | = | concrete cylinder strength |
| $f_y$ | = | yield strength of longitudinal reinforcement |
| $f_{yt}$ | = | yield strength of transverse reinforcement |
| $F$ | = | lateral force |
| $P$ | = | axial compressive force |
| $P_o$ | = | concentric capacity of column |
| $M$ | = | bending moment |
| $s$ | = | spacing of transverse reenforcement |
| $\varepsilon$ | = | strain |
| $\rho$ | = | longitudinal reinforcement ratio |
| $\rho_s$ | = | volumetric ratio of transverse reinforcement |
| $(\rho_s)_{ACI}$ | = | volumetric ratio of transverse reinforcement as required by ACI 318-95 building code based on actual $f_{yt}$ |

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