

Concrete Columns Confined with Welded Reinforcement Grids

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ABSTRACT

Reinforced concrete columns subjected to strong earthquakes may experience inelastic deformations. Inelastic deformability of these columns is of utmost importance for overall strength and stability of structures. Column deformability may be increased through the confinement of core concrete. Conventional confinement reinforcement for square and rectangular columns consist of closely spaced perimeter hoops, overlapping hoops, and crossties. The confinement steel requirements of current building codes often result in high volumetric ratios of transverse reinforcement which may lead to the congestion of column cages, which may result in concrete placement problems. Bends with 135-- degree hooks and bend extensions may add to the problem of congestion, jeopardizing sound construction practice. Furthermore, the production and assembly of these individual ties within acceptable dimensional tolerances may be labor intensive and may require excessive time, resulting in significant increase in construction cost. One of the potential alternatives to conventional reinforcement is a welded reinforcement grid, prefabricated to required size and volumetric ratio of transverse reinforcement.

An experimental investigation was conducted to study the structural performance of concrete columns reinforced with welded grids. Ten large scale columns with different volumetric ratio, spacing and arrangement of welded reinforcement grids were tested under simulated seismic loading. The columns were subjected to concentric compression of approximately 20% or 40% of their capacities while also subjected to-incrementally increasing lateral deformation reversals. The results indicate, that the welded reinforcement grid can be used effectively as confinement reinforcement provided that the steel used has sufficient ductility and the welding process employed does not alter the strength and elongation characteristics of steel. The transverse reinforcement used in the experimental program met these requirements and showed 7% to 10% strains prior to failure. The grids improved the structural performance of columns which developed lateral drift ratios in excess of 3% with transverse reinforcement less than or approximately equal to that required by the AC13 18-95 Building Code (1). The drift capacity further increased when grids with smaller cells (larger number of cross bars) were used. Furthermore, the cage assembly became easier and faster as compared to columns with conventional tie reinforcement.

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INTRODUCTION

Performances of reinforced concrete buildings during recent earthquakes have demonstrated that unconfined columns. especially at the first story level. suffer significant damage during strong earthquakes Although the current design practice calls for strong columns and weak beams to dissipate seismic induced energy by yielding of the beams, it is difficult to prevent inelasticity in lower story columns during a strong earthquake Therefore, earthquake resistant columns are designed to develop a large number of inelastic deformation reversals without a significant loss of strength This is referred to as inelastic deformability of columns and can be attained through the confinement of core concrete by closely spaced transverse and longitudinal reinforcement Column confinement is a requirement of building codes (1.2) for seismic regions

Parameters of confinement include the amount, spacing, grade and arrangement of transverse reinforcement. Proper combination of these parameters results in strength and ductility enhancement in core concrete, which in turn improves the behavior of the column Rectilinear confinement reinforcement used in square and rectangular columns consist of perimeter hoops, overlapping hoops and crossties with bends properly anchored into the core concrete The close spacing of tie steel as required by the code. and the use of overlapping hoops with bends and bend extensions often result in congestion of the reinforcement cage, occasionally creating construction problems An alternative to conventional transverse reinforcement is a welded reinforcement grid, prefabricated to the required size and volumetric ratio of transverse steel These grids. when used as column transverse reinforcement, could potentially lead to savings associated with easy and fast cage assembly, and reduction in steel consumption since the overlapping reinforcement, bends and bend extensions are eliminated The welds prevent opening of the ties under lateral concrete pressure improving column behavior in the inelastic range. Furthermore, the precision of welded grid comers, as compared to bent conventional hoop comers, provides better and consistent support to longitudinal reinforcement, also improving column behavior The grid comers may also provide additional confinement pressure peaks without any longitudinal reinforcement, improving the uniformity of pressure.

In spite of the potential advantages of welded reinforcement grids as column reinforcement, its use In practice has been hindered by lack of research and experimental evidence. as well as the brittle nature of conventional wires used in the welded wire industry An experimental investigation was carried out at the Structures Laboratory of the University of Ottawa to investigate the strength and deformability of reinforced concrete columns confuted with welded reinforcement grids The results are presented and discussed in this report

OBJECTIVE AND SCOPE

The objective of the research project was to investigate strength and deformability of reinforced concrete columns confined with welded reinforcement grids to explore the potential use of this type of prefabricated grids as column confinement reinforcement

The scope consisted of the following tasks.

- Design of 10 large scale columns to be tested under constant axial compression and incrementally increasing lateral deformation reversals.
- Construction and instrumentation of column specimens.
- Testing columns and recording data
- Evaluation of test data and investigation of the effects of test variables
- Comparisons with columns reinforced by conventional ties.
- Presentation of results

PREVIOUS RESEARCH AND APPLICATION TO PRACTICE

Research on the characteristics of welded reinforcement grids and its application to the construction industry in North America have been fairly recent The use of welded wire fabric as column reinforcement was investigated by Razvi and Saatcioglu (3) on small scale columns. The early research involved the use of conventional welded wire fabric, and hence was limited to low volumetric ratios of transverse reinforcement The test program indicated potential benefits of

confining column concrete with transverse reinforcement that consisted of conventional ties used in combination with welded wire fabric More recently. however, grids with larger diameter reinforcement have become available The material was tested in boundary elements of concrete shear walls at the University of California. Berkeley (4) Welded reinforcement grids were also used as transverse reinforcement in beams and columns of specimens used for beam-column connection tests at the National Institute of Standards and Technology (5). Welded grids were recently used in a 17-story residence apartment building at San Francisco State University and in a 3-story prototype frame in Kauai, Hawaii by Baumann Engineering of Newport Beach, California, U.S.A. (6)

EXPERIMENTAL PROGRAM

Column Specimens

Ten near-full-size column specimens were designed and constructed for testing The specimens represented part of a first-story column between the footing and the point of inflection. with a 1645 mm shear span Figure 1 illustrates the geometric details of a typical specimen A 350 mm (14 in) square cross-section was used with four different configurations of reinforcement Three columns were built with 8-#20 (19.5 mm diameter) bars as longitudinal reinforcement. confined with 4-cell grids having 9.53 mm (3/8 in) diameter reinforcement Four columns were built with 12-#20(195 mm diameter) longitudinal bars. confined with 9-cell grids having either 9.53 mm (3/8 in) diameter or 6.60 mm (1/4 in) diameter reinforcement. One column was reinforced with 4-#30 (29.9 mm diameter) longitudinal bars and 9-cell grids having 9 53 mm (318 in) diameter transverse reinforcement. This column had four comer bars only without any longitudinal reinforcement in the remaining perimeter corners of grids Two additional columns were prepared with 20-#15 (I 6 0 mm diameter) bars as longitudinal reinforcement, one at each of the perimeter comer of 9-cell grids, having either 9.53 mm or 6.60 mm diameter transverse reinforcement Different spacings of grids were used resulting in different volumetric ratios of transverse reinforcement The volumetric ratio varied between 65% and 172% of that required by AC13 18-95. Figures 2 through 4 depict the details of cross-sectional configurations used The columns were subjected to approximately 40% of their concentric capacities computed based on Eq 1. except when the effect of axial compression was investigated as a parameter, in which case approximately 20% of concentric capacity was applied.

Table 1 provides a summary of column properties.

$$P_{o} = 0.85 f_{c}' (A_{g} - A_{s}) + A_{s} f_{v}$$
(1)

Concrete

Ready mix concrete, supplied by a local ready mix company, was used to cast the columns The concrete was specified to have a minimum 28-day compressive strength of 30 MPa and slump of **80** mm, with 20 mm maximum size crushed lime stone aggregate No air entrainment was requested in the mix The mix proponions of a cubic meter of concrete consisted of 204 kg of Normal Portland Cement (CS.4 Type 10, ASTM Type I), 83 kg of slag. 1130 kg of crushed lime stone 873 kg of natural sand, and 83 kg of water A large number of control cylinders were cast using 152 mm by 305 mm (6 in x 12 in) standard cylinders to perform cylinder tests The concrete strength was monitored by testing cylinders at 7, 14, 21. and 28 days The column tests were performed one month after casting. A 2200 kN-capacity Fourney hydraulic testing machine was used to test the cylinders in accordance with ASTM method C39. The cylinder test results are summarized in Table 2 The average 28-day strength was found to be 33.7 MPa Figure 5 illustrates the average stress-strain relationship obtained for concrete from cylinder tests

Longitudinal Reinforcement

Three different sizes of Grade 400 MPa bars were used as longitudinal reinforcement These were #15 (16 mm diameter), #20 (19.5 mm diameter) and #30 (29.9 mm diameter) bars manufactured to conform the CSA Standard G30.12 Sample coupons were tested from each batch of steel using a 1500 kN-Tenius Olsen universal testing machine to establish the stress-strain relationship of reinforcement in accordance with ASTM A370-72. An Alsmer Extensometer with a 5 lmm (2 in) gauge-length was used to record the strains Figure 6 illustrates experimentally obtained stress-strain relationships, indicating yield strengths of 420 MPa, 455 MPa and 470 MPa for #15, #20 and #30 bars, respectively.

Col	Reinf	Р	b _d	S	f_{vt}	ρs	$\left(\rho_{s}\right)_{ACI}$	$\rho_s/(\rho_s)_{ACI}$	Р	P/P_0
Label	Arrng.	(%) (m m)	(m m) (M	IPa) (%) (%)		(kN)	
BG-1	8-#20	1.96	9.53	152	570	1.00	1.55	0. 65	1782	0. 39
BG-2	8-#20	1.96	9. 53	76	570	200	1.55	1.29	1782	0. 39
BG-3	8-#20	1.96	9.53	76	570	2.00	1.55	1.29	831	0. 18
BG-4	12-#20	2 94	9. 53	152	570	133	1.55	086	1923	0. 38
BG-5	12-#20	7. 94	9 53	76	570	266	1.55	1.72	1923	03s
BG-6	4-#30	2. 29	953	76	570	2.66	1.55	1.72	1900	040
BG-7	12-#20	2. 94	6. 60	76	5 80	1.26	1. 52	0 83	1923	038
BG-8	12-#20	2 94	660	76	580	1.26	1 52	0 83	961	019
BG-9	20-#15	3. 26	6. 60	76	580	1.26	1. 52	0. 83	1923	038
BG-IO	20-#15	3. 26	953	76	570	2.66	1.55	1.72	1923	038

Table 1. Properties of Test Specimens

Table 2. Variation of Concrete Compressive Strength with Age

Compressive Strength in MPa

Age:	7 days	14 days	21 days	28 days
Cylinder 1	21.0	27.2	31. 8	34. 5
Cylinder 2	21.5	27 6	31.2	33. 3
Cylinder 3	19.3	25.1	29. 2	33. 3
Average	20. 6	26. 6	30. 7	33. 7

Welded Reinforcement Grids

Welded reinforcement grids, used as transverse confinement reinforcement. had a square configuration with a 292 mm (1 1.5 in) out-to-out dimension Three different types of g_{rids} were employed consisting of, i) 9.53 mm (3/8 in) diameter reinforcement welded to form 4 equal-size square grids, ii) 9.53 mm (3/8 in) diameter reinforcement welded to form 9 equal-size square grids. and iii) 6 60 mm (1/4 in) diameter reinforcement welded to form 9 equal size square grids The reinforcement in each direction protruded by 6.35 mm (1/4 in) beyond the perimeter welds. Figure 7 shows the geometric details of welded reinforcement grids used in the experimental program

Coupons were taken from each type of grid such that a cross reinforcement with a welded joint was included in the test region Two strain gauges were placed on each coupon, one at the back of a welded joint, and the other between the joints An Alsmer Extensometer with a 5 1 mm (2 in) gaugelength was used, spanning the cross bar location, to record strains at large extensions where strain gauges usually seize to function Figure S(a) illustrates the instrumentation of a typical coupon It was found that the extensioneter readings agreed closely with strain gauge readings at the joint. The first yielding was recorded by the strain gauge behind the weld and the extension over the joint The welds survived the tests without any failure. and none of the coupons tested failed at the welds The "necking" of steel started at the joint. behind the weld This led to the fracture of stee! on one side of the welded joint, while the weld itself remained intact Fig 8(b) shows a schematic view of "necking" and rupturing of steel observed during coupon tests Steel fracture occurred at approximately 7% to 10% tensile strain. consistently at the same location near the weld. Fig 8(c) shows a photograph of coupons after the test. illustrating the locations of steel rupture The same pattern and location of grid failure were also observed in three columns that experienced grid failure at very high inelastic deformations, following the buckling of longitudinal reinforcement. The failure of the grids was in the form of rupturing of steel near the welds, in the same manner as observed in coupon tests. Fig. 9 illustrates the stress-strain relationships for 9.53 mm and 6 60 mm grid reinforcements obtained through coupon tests. The coupons showed yield strengths of approximately 580 MPa and 570 MPa for 6 60 mm and 9 53 mm reinforcement. respectively

Preparation of Columns

Each specimen consisted of a heavily reinforced rigid footing and a cantilever column. representing the lower portion of a typical first-story column up to the point of inflection Column cages were assembled by first aligning the longitudinal bars horizontally on supports and then inserting and positioning the grids The use of grids as transverse reinforcement enabled easy assembly and perfect support for the longitudinal reinforcement. Figure 10 illustrates the column cages

Plywood formwork was used for casting the columns. All ten columns were cast vertically in two stages to simulate the actual construction practice. First the footings were cast The footing-column joint was intentionally left rough. The columns were cast two days later. A large number of control cylinders were also cast and cured together with the columns.

Instrumentation

The specimens were instrumented with Linear Variable Displacement Transducers (LVDT) and strain gauges to measure displacements, rotations and steel strains. Two LVDTs were placed horizontally to measure horizontal displacements. one at the point of application of the horizontal load (1645 mm from the footing), and the other immediately below the loading beam (1365 mm from the footing) Four additional LVDTs were placed vertically near the critical section to measure rotations of the hinging region and rotations caused by anchorage slip (extension of longitudinal reinforcement within the footing) Figure 11 illustrates the locations of LVDTs One of the two vertical LVDTs placed on sides perpendicular to the direction of loading measured the vertical displacement of the top of the hinging region relative to the footing, with a gauge length of 350 mm The other vertical LVDT measured the displacement of the bottom column section relative to the footing with a gauge length of approximately 25 mm While the difference between the readings of the column hinging region on either side divided by the horizontal distance, separating the two LVDTs would give the total rotation of the hinging region, the readings of the column base would be used in a similar manner to compute the rotation due to anchorage slip. The difference between the total hinge rotation and the anchorage slip rotation gives the rotation of the hinging region due to flexure alone.

Electrical resistance strain gauges were placed on transverse and longitudinal reinforcement to measure the steel strains Two of the longitudinal reinforcement in each column were instrumented with strain gauges. These gauges were placed on bars near the extreme compression and tension fibers at 50 mm below the footing surface The first two grids in each column immediately above the footing surface, were also instrumented with strain gauges. These grids were labelled as the first and the second, the first grid being the closest to the footing. The strain gauge readings are presented under "Strain Readings," with reference to their respective locations All the instrumentation. including the load cells and stroke LVDTs of the MTS actuators were connected to a Scimetric data acquisition system and an MTS controller for data collection. Two micro computers were used to control the data acquisition systems and the controller

Test Setup and Loading Program

Three 1000 kN capacity servo controlled MTS actuators were used to apply the load Two of the actuators were positioned vertically to apply constant axial compression during testing, They were connected to a rigid base on the laboratory strong floor at one end, and to a steel loading beam at the other end Figures 12 through 14 illustrate the details of the test setup. The third actuator was postttoned horizontally between the steel loading beam and the lateral support system. The lateral support system consisted of steel A-frames and channels bolted together and secured to the laboratory strong floor The steel loading beam was attached to the column at the top by means of high-strength bolts that had been cast in column core concrete Figure 15 illustrates different views of the test setup

The axial load was applied first, and was maintained at a constant level throughout the test. The horizontal load was applied in the deformation control mode. Lateral deformation reversals were applied starting with three elastic cycles at 0.5% lateral drift. followed by three cycles at 1% drift, which approximately corresponded to the yield displacement. The subsequent stages of loading included three cycles at each drift level, which increased incrementally following the loading history shown in Fig. 16.

OBSERVED BEHAVIOUR AND TEST RESULTS

All columns showed similar behaviour during the initial elastic cycles of deformation Some flexural cracking was observed on the tension side near the critical section. No sign of concrete damage was detected at this load stage Deformation reversals at 1% drift resulted additional flexural cracking It was noticed that all the flexural cracks occurred near the grid locations. Slight crushing of concrete cover was observed at the comers of the column-footing interface Diagonal tension cracks were also observed on side faces Very little additional flexural cracks formed during subsequent levels of inelastic deformation Instead, the existing cracks opened wider Unlike the flexural cracks, new Xshaped diagonal tension cracks appeared on side faces every time an increased level of inelastic deformation was imposed. Spalling of concrete cover usually took place beyond 2% drift Subsequent inelastic reversals of deformations resulted in further cover spalling and concrete crushing, followed by longitudinal bar buckling at large deformation levels The amount of damage in each column varied depending on the level of axial compression. reinforcement arrangement, spacing, and volumetric ratio of grid reinforcement A brief review of observed behaviour is presented for each column in the following sections. Experimentally recorded force-displacement hysteretic relationships are discussed to explain the effects of test parameters The hysteretic behavior is presented both in terms of moment-displacement and force-displacement relationships The moments plotted in these relationships are computed from the recorded test data as the net lateral force times the shear span plus the vertical axial force times the horizontal displacement (P-A moment). These momentdisplacement relationships correspond to lateral force-lateral displacement relationships when the axial load passes through the critical column section at the base. Hence, these relationships do not show the degradation of lateral load resistance caused by the P-A effect Lateral force-lateral displacement hysteretic relationships, on the other hand. are plotted using the net horizontal resistance of columns as recorded during testing and the corresponding horizontal displacements while the vertical axial load component is acting on the column Therefore, these relationships reflect the decay in lateral load resistance caused by the P-A effect. A summary of observed strength and drift capacities are tabulated in Table 3, along with nominal moment capacities computed following ACE 1 S-95.

	Column Mo	ment Resistance	<u>Column Fo</u>	orce Resistance	ACI31x-95
<u>Column</u>	(M _{test}) _{Max} (kN.m)	Min. Drift @ $80\%(M_{test})_{Max}$	(F _{test}) _{Max} (kN)	Min. Drift (a) 80%(F_{test}) _{Max}	Mn (kN.m)
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

Table 3. Summa? of Strength and Drift Capacities

<u>Notes:</u> 1 $(M_{test})_{Max}$ and $(F_{test})_{Max}$ quantities represent the average of maximum moments and lateral forces recorded in each direction of loading

2 Minimum drift capacity is obtained from respective hysteretic relationship as lateral drift at which the column has completed a minimum of three deformation cycles with no more that 20% drop in strength The readings are rounded off to the smaller whole number

Column BG-1

Column BG-1 was reinforced with 8 longitudinal bars resulting in 1.96% reinforcement ratio, confined with 4-cell grids The volumetric ratio of transverse reinforcement was 1.00%, which formed 65% of the amount required by ACI3 18-95 The spacing of grids was 152 mm (6 in), which was approximately twice the maximum spacing required by ACI3 1 S-95. The column was tested under a constant compressive force of 39% of its concentric capacity The hysteretic relationships shown in Figs. 17 and 18 indicate that the column showed stable hysteresis loops up to 1% drift but developed a significant strength decay during the second cycle at 2% drift. This was expected because of the low volumetric ratio of transverse reinforcement provided for the high level of axial compression imposed

The column failed during the second cycle at 3% drift due to the buckling of longitudinal reinforcement

Observations during testing indicated that the column experienced initial flexural cracking at 0 5% and 1% drift levels The flexural cracks became visible within the lower 625 mm portion of the column The cover concrete started crushing when the column was loaded to 2% lateral drift The crushing of concrete became severe especially on the east side of the column At the end of 2% drift cycles the cover concrete below the second layer of grids near the base spalled off completely. exposing the reinforcement cage The plastic hinging of the column near the base was visibly noticeable during the first cycle at 3% drift During the second cycle of 3% drift level, the longitudinal reinforcement in the north-west comer buckled between the first and second grids from the base, and subsequently the middle bars of the north side buckled between the second and third layers of grids The second grid from the base deformed excessively and bulged out under lateral expansion while maintaining its integrity There was no sign of weld failure or rupturing of transverse steel even after the buckling of longitudinal reinforcement The drop in load resistance was excessive during the second cycle of 3% drift and the test was discontinued Figure 19 illustrates the performance of Column BG-1 at selected load stages.

Column BG-2

Column BG-2 was companion to BG-1, reinforced with 8 longitudinal bars and confined with 4-cell grids Although the same size grids were also used in this column the grid spacing was reduced by a factor of 2 resulting in an increase in the volumetric ratio of transverse reinforcement The grid spacing was 76 mm (3 in) and the volumetric ratio was 2 00%. exceeding the confinement steel requirements of ACI318-95 by 29%. The column was subjected to 39% of its concentric capacity as constant axial compression It behaved much better than the companion column BG-1 The reduction in tie spacing increased the effectiveness of confinement and stability of longitudinal reinforcement The moment-displacement hysteretic relationship shown in Fig 20 indicates that the column showed stable hysteresis loops up to 3% drift with little strength decay and failed at 5% drift due to bar buckling and concrete crushing The force-displacement hysteretic relationship of Fig 21 shows

reduced deformability due to the P- Δ effect and indicates approximately 20% strength loss in lateral load resistance during 2% drift cycles

Observations during testing indicated that the first flexural crack occurred during the first cycle at 0.5% drift near the second grid from the base Additional flexural cracks occurred at other grid levels during the subsequent cycles of 0.5% drift Diagonal tension cracks started forming on side faces during 1% drift cycles Additional flexural cracking was also observed at this deformation level. although no cover crushing was evident The cover concrete started crushing near the base at 2%lateral drift Widening of diagonal cracks was also observed Gradual spalling of cover concrete occurred during 3% drift cycles. which eventually lead to the complete spalling of hinging region at 4% drift Cover spalling extended up to the fourth grid level, exposing reinforcement Additional damage to concrete was noticed during the cycles at 5% drift The hinging region became clearly visible and measured to be 350 mm from the base The longitudinal reinforcement buckled during the second cycle at 5% drift and the capacity dropped suddenly. When the column was pushed further to complete the second cycle at 5% drift. the fracturing of grid reinforcement was observed The longitudinal reinforcement developed high tensile strains showing 11% at 3% drift 134% at 4% drift. and 1 7% at 5% drift entering into the strain hardening region. also implying significant yield penetration into the footing Figure 22 illustrates the performance of Column BG-2 at selected load stages

Column BG3

Column BG-3 was identical to BG-2 in every respect. with 129% of the confinement steel required in ACI3 18-95 However, this column was tested under a lower level of axial compression. corresponding to 18% of its concentric capacity The hysteretic relationships illustrated in Figs 23 and 24 indicate that the column behaved in an extremely ductile manner showing virtually no decay in moment resistance up to 7% drift, and 20% decay in lateral force resistance at 4% drift, due to the P-4 effect This confirms the expected trend of improvement in column deformability with reduction in axial compression Observations during testing indicated that only hairline cracks formed at 0 5% drift cycles These cracks became wider at 1% drift No shear cracks were observed at this load stage Slight crushing of cover concrete on the compression side was observed during the cycles at 2% drift A wide flexural crack appeared at the column-footing interface. signifying yield penetration into the footing. Cover spalling started during the deformation cycles at 3% drift and continued during 4% and 5% drift cycles The cover concrete completely spalled off within the hinging region, which was measured to be approximately equal to the cross-sectional dimension of the column The column continued resisting loads during cycles of 6% drift and was able to sustain the first cycle at 7% drift, during which the longitudinal bars started to buckle. When the column was pushed to the second cycle at 7% drift the longitudinal and transverse reinforcement fractured, marking the end of the test. Figure 25 illustrates the performance of Column BG-3 at selected load stages

Column BG4

Column BG-4 was reinforced with 12 longitudinal bars resulting in 2.94% reinforcement. confined with 9-cell grids The volumetric ratio of transverse reinforcement was 1.33% which formed 86% of the amount required by ACI3 18-95 The spacing of grids was 152 mm (6 in). which was approximately twice the maximum spacing required by ACI31S-95 The column was subjected to 38% of its concentric capacity during lateral deformation reversals The hysteretic relationship shown in Fig 26 and 27 indicate that the column showed stable hysteresis loops up to 2% drift and failed during the second cycle of 4% drift when the longitudinal bars buckled and the core concrete crushed The column behaved much better than Column BG-I which had slightly lower volumetric ratio but was otherwise similar to BG-2, mainly because of the difference in the number of grid cells BG-4 was reinforced with 9-cell grids producing closer spacing of transverse reinforcement in the cross-sectional plane resulting in a more uniform distribution of lateral confinement pressure

Observations during testing indicated that initial flexural cracking started at 0.5% and 1% drift cycles Some shear cracking and cover crushing occurred during the cycles at 2% drift. The cover concrete started to spall off during the cycles at 3% drift Severe damage to the hinging region concrete was observed The column failed due to the buckling of longitudinal reinforcement and crushing of core concrete during the cycles at 4% drift No failure was observed in the grids Figure 28 illustrates the performance of Column BG-4 at selected load stages

Column BG-5

Column BG-5 was companion to BG-4 and was reinforced with 12 longitudinal bars. confined with 9-cell grids Although the grid size was kept constant, the spacing of grids was reduced in BG-5 by a factor of 2 resulting in an increase in the volumetric ratio The grid spacing was 76 mm (3 in) and the volumetric ratio oftransverse reinforcement was 2.66% which was 72% higher than that required by ACI318-95 The column was subjected to 38% of its concentric capacity The hysteretic relationships shown in Figs. 29 and 30 indicate that the column showed no loss of moment resistance up to 5% drift, while the lateral load resistance dropped to approximately 80% of its peak at 3% lateral drift because of the P-A effect The failure was initiated by buckling of longitudinal reinforcement at 7% drift The improved behavior relative to BG-4 was attributed to reduced grid spacing. and the improvement relative to BG-2 was attributed to the increased number of grid cells. providing better distribution of transverse reinforcement within the cross-sectional plane improving the uniformity of confinement pressure

Obsenations during testing revealed that the column developed initial flexural cracking at 05% and 1% drift Some diagonal tension cracks also formed on the side faces No visible damage was observed until the cycles at 2% drift were applied. Spalling of cover concrete started at 2% drift and continued during the cycles of 3% lateral drift Cycles at 4% and 5% drift resulted in the complete spalling of cover concrete within the hinging region, which was measured to be approximately **350** mm from the footing The longitudinal bars showed signs of buckling at 6% drift when the strain recorded was 1.4% One longitudinal bar fractured in tension during the first cycle at 7% drift resulting in a sudden drop of loading Figure 31 illustrates the performance of Column BG-5 at selected load stages

Column BG-6

Column BG-6 was reinforced with 4 - # 30 (29 9 mm diameter) comer bars The reinforcement grids

had 9 cells. without any longitudinal reinforcement in the remaining perimeter grid corners The volumetric ratio and spacing of transverse reinforcement were identical to those for Column BG-5 It was loaded to 40% of its concentric capacity as constant axial compression The hysteretic relationships shown in Figs 32 and 33 indicate that the column behaved as good as the companion column BG-5. developing 5% drift without any sign of strength decay in moment resistance, and 3% drift in lateral force resistance at approximately 20% strength decay This result has a significant implication on application to practice, indicating that the grids are able to provide the required lateral restraint at grid joints without any longitudinal reinforcement. 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 further confirm this point, as this specimen was the only column confined with this type of arrangement. Column BG-6 was one of the first columns tested and the test was discontinued prematurely due to a safety concern at a high drift level of 5%. prior to developing a significant loss in strength

Observations during testing indicated that only one flexural crack was observed during the cycles at 0 5% drift Additional four cracks were noticed at the bottom four grid locations during the cycles at 1% drift. in addition to a crack that formed at the column-footing interface. The concrete cover near the base started crushing at 2% lateral drift. while shear cracks were observed on the side faces Some crushing of cover concrete was observed near the base at 3% lateral drift. No new flexural cracks were observed at this drift level although additional shear cracks became noticeable Increased spalling of cover concrete was observed at 4% drift The test was stopped at 5% drift due to a safety concern related to the lateral bracing system Figure 34 shows the performance of Column BG-6 during selected stages of testing

Column BG7

Column BG-7 was companion to BG-5 with 12 - #20 (19 5 mm diameter) longitudinal reinforcement and 9-cell grids spaced at 76 mm (3 in) However. the diameter of grid reinforcement was reduced to 6 60 mm (1/4 in). resulting in a reduced volumetric ratio of 1 26% This amount of transverse steel corresponded to 83% of that required by ACI3 18-95 The axial load level was kept constant at 38% of the concentric capacity The hysteretic relationships shown in Figs 35 and 36 indicate that the column showed ductile response even with lower volumetric ratio than that required by ACI3 18-95 because of the improved distribution of confinement pressure associated with 9-cell grids The moment resistance was maintained till 4% lateral drift without any strength decay while the lateral force resistance showed 3% drift at 20% strength loss.

The observed behavior of column during testing indicated that some flexural cracks occurred during the cycles at 1% drift, including a crack at the column-footing interface No shear crack was observed at this load stage More flexure and shear cracks were observed during the cycles at 2% drift level. The deformation cycles at 3% drift resulted in spalling of the cover concrete Additional shear cracks were observed on the side faces The cycles at 4% drift resulted in spalling of the cover, exposing reinforcement. The plastic hinge length was measured to be approximately equal to the cross-sectional dimension The column failed during the first cycle at 5% drift due to the buckling of longitudinal reinforcement and subsequent crushing of the core concrete Figure 37 illustrates the performance of Column BG-7 at selected stages of loading

Column BG-8

Column BG-8 was companion to BG-7 with identical properties except the axial compression The level of constant axial compression was reduced to 19% of the concentric capacity to investigate the effect of axial load The hysteretic relationships shown in Figs 38 and 39 indicate that the column showed ductile response with no strength decay in moment resistance up to 6% drift, although the lateral load resistance dropped to 80% of its peak resistance at 4% drift, due to the P- Δ effect

Observations during testing indicated little flexural cracking at 0.5% drift Additional cracks were observed at 1% drift, including a continuous crack at the column-footing interface. Shear cracks were observed to form at 2% drift. while additional flexural cracking was also observed. Crushing of cover concrete started at 3% drift level and continued during the cycles of subsequent deformation levels at 4% and 5% drift, exposing reinforcement within the hinging region The longitudinal reinforcement

showed signs of buckling during the cycles at 640 drift One of the longitudinal bars buckled at 7% drift which lead to a sudden strength drop Figure 40 illustrates the observed behavior at selected stages of loading

Column BG9

Column BG-9 was reinforced with 20 - # 15 (16 mm diameter) longitudinal reinforcement, producing 3.26% reinforcement ratio The column was confined with 9-cell 6.60 mm diameter (1/4 in) grids There was one longitudinal bar in each of the perimeter grid comers. including one on each side of the interior perimeter joints This resulted in 20 longitudinal reinforcement. The grid spacing was 76 mm (3 in), and the volumetric ratio oftransverse reinforcement was 126% (83% of that required by ACI3 1 S-95) The hysteretic relationships shown in Figs 4 1 and 42 indicate that the column was able to develop 3% drift without a significant loss in strength, and was not able to sustain the deformation cycles at 4% drift This column was companion to Column BG-7, and although showed ductile response. was not able to develop the high ductility level exhibited by BG-7. This may be explained by the smaller size of longitudinal bars used in BG-9, which are more susceptible to buckling The failure was observed to occur due to the buckling of one of the comer bars Hence, the use double longitudinal bars in perimeter grid corners is not likely to be the cause of the slight reduction in deformability

Observations during testing indicated initiation of flexural cracks at 0 5% drift cycles Additional flexural cracks formed at 1% drift. one at each grid location within the lower 600 mm segment. Shear cracks were observed during the cycles at 2% drift. Some crushing of cover concrete was evident near the base. Spalling of the cover concrete was observed at 3% drift. The column failed by buckling of one of the comer bars during the third cycle of deformation reversals at 4% drift Figure 43 illustrates the performance of column at selected load stages

Column BG-10

Column BG-10 was companion to BG-9 with the same reinforcement arrangement. consisting of 20 - X15 (16 mm diameter) longitudinal bars and 9-cell grids This time, however. the grid bar size was

larger. increasing the volumetric ratio of transverse reinforcement to 2 66% This amount of confinement steel represented 172% of that required by ACI31S-95 The grid spacing was maintained at 76 mm (3 in) The hysteretic relationships shown in Figs. 44 and 45 indicate better performance for the column than that for Column BG-9, developing 5% drift without a significant loss in moment capacity The lateral load capacity was reduced due to the P-A effect, reaching approximately 80% of peak resistance at approximately 3% drift The superior performance of this column can be explained by the increased volumetric ratio of transverse reinforcement

Observations during testing indicated little flexural cracking at 0.5% and 1% drift Additional flexural cracks. as well as shear cracks occurred at 2% drift Some crushing of cover concrete was evident at this displacement level. near the base Cycles at 3% and 4% drift resulted in gradual spalling of the cover concrete. exposing reinforcement Failure was initiated by buckling of a longitudinal reinforcement during the second cycle at 6% drift Figure 46 illustrates the observed performance of column at selected stages of loading

Moment-Rotation Relationships

The columns were instrumented to measure two sets of rotation readings within the potential hinging region The hinging region was defined as a column segment between the column-footing interface and a section 350 mm above the footing (equal to the cross-sectional dimension) The first set of readings gave total rotation of the assumed hinging region relative to the footing This set of readings consisted of rotations mostly due to flexure and also due to anchorage slip. The latter resulted from slippage and/or extension of reinforcement within the footing. caused by the penetration of yielding into the footing Rotations due to anchorage slip were measured and formed the second set of readings These readings were taken as the rotation of a column section near the column-footing interface relative to the footing. Ideally, these readings should have reflected the rotation of the critical column section at the interface. However, the LVDT's used required some gauge length to be positioned on the column Hence, they were mounted on a section approximately 25 mm above the interface Therefore, these readings also include flexural rotations of the lowest 25 mm segment of the column and hence are not accurate representation of anchorage slip Furthermore, these

readings were taken within the most disturbed region of columns because of concrete crushing Hence. additional errors may have occurred especially near the end oftesting when significant damage was observed in column hinging regions The relatively small magnitude of anchorage slip rotations further increase the sensitivity of these readings to such errors Nevertheless. anchorage slip readings provide a good feel for the significance of this deformation component in columns The difference between total and anchorage slip rotations give rotations due to flexure

Anchorage slip is closely related to the level of yield penetration that occurs into the footing Therefore, it becomes important for columns subjected to low level of axial compression Extension of reinforcement in footing becomes sizeable only if the longitudinal column reinforcement experiences strain hardening in tension (7). Slippage of steel becomes noticeable in reinforcement with short embedment lengths Therefore, while anchorage slip is a major contributor to inelastic deformations of beams (without axial compression) it may become negligible in columns subjected to high axial compression Sample moment-rotation relationships, recorded for two columns with two different levels of axial compression, are plotted in Figs. 47 and 48. Complete data, reflecting moment-rotation hysteretic relationships for all columns, are presented in Appendix A

<u>Reinforcement Strain</u> Readings

Both longitudinal and grid reinforcement were instrumented with electric resistance strain gauges to measure the variation of strain in reinforcement Strain **gauges** were placed on two longitudinal bars in each column at 50 mm below the column-footing interface The gauges indicated that the extreme most layer of longitudinal column reinforcement yielded both in tension and compression at approximately 1% lateral drift In most cases the readings could be recorded up to approximately 1% strain A hysteretic relationship and an envelope curve, depicting the variation of one of the strain gauges placed on a longitudinal reinforcement is illustrated in Figs 49. Appendix B includes the envelope curves for longitudinal strain readings for all columns

Strain gauges were also placed on the first two grids above the footing The grid closest to the footing was labelled as the first grid and the other as the second The gauges were placed mostly at

perimeter weld locations. on bars immediately behind the welds Figure 50(a) illustrates the gauge location During response. however, the protruding part of bars beyond the perimeter welds curved towards the welds placing gauges in compression This is shown in Figs 50(b) and (c) Consequently. these readings did not provide any information regarding the amount oftension in grid reinforcement Fig. 5 1 includes two sample plots of strain gauge data recorded at these locations Additional strain gauges were also placed on grid bars between the welds Most of these gauges functioned well beyond yielding, although some seized to function earlier Figures 52 through 67 show the recorded data The figures indicate yielding of transverse reinforcement either during 3% or 4% drift cycles

COMPARISONS WITH COLUMNS CONFINED BY CONVENTIONAL REINFORCEMENT

Tests of companion columns with conventional tie steel were not included in the experimental program Therefore, force-displacement relationships of columns with conventional tie steel were generated analytically Columns with conventional tie steel were analysed for force-displacement relationships under constant axial compression and monotonically increasing lateral loads The monotonic curves obtained in this manner are generally regarded as good approximations of the envelopes of hysteretic relationships These curves were then compared with those obtained experimentally for columns reinforced with welded grids This type of comparison provides valuable information on performance of welded grids relative to conventional ties An important aspect of such comparison is the validity and reliability of analytical tools and procedures utilized in establishing the inelastic force-displacement relationships The procedure employed is summarized in the following sections. including verifications

Tests of reinforced concrete columns under simulated seismic loading indicate that total inelastic displacement consists of components due to flexure, shear and anchorage slip (8). Figure 68 illustrates that shear deformations in a typical column is very small because of its long shear span Therefore. unless a column is under-designed in shear, or is classifies as a short column, shear deformations may be ignored without significantly affecting the results However. deformations due to flexure usually

forms a large proportion of total inelastic displacement, with some contributions from anchorage slip, whose significance depends on the level of axial compression and the inelasticity imposed

Flexural Deformations

Flexural behavior of a column can be computed by first constructing a sectional moment-curvature relationship This is done by using a standard procedure outlined in most reinforced concrete text books, which includes a plane section analysis. However, the accuracy of material models becomes important in the inelastic range The use of a confined concrete model incorporating all the relevant parameters of confinement. and steel stress-strain relationship with strain hardening region are of utmost importance for the reliability of results

The confined concrete model developed by Saatcioglu and Razvi (9) was adopted in this research program since it incorporates all the significant parameters of confinement, including the amount. spacing, grade and arrangement of transverse reinforcement The model is applicable to square. circular and rectangular columns under concentric and eccentric loads. It has been verified extensively against a large volume of test data obtained from large scale column tests conducted under concentric and eccentric loads (9,10) The model is illustrated in Fig. 69 An important feature of the model. relevant to current research, is the distribution of lateral confinement pressure. According to the model the efficiency of lateral pressure in square and rectangular columns improve with distribution oftransverse steel legs within the cross-sectional plane In conventional tie reinforcement this implies the use of overlapping hoops and crossties supporting closely spaced longitudinal reinforcement. providing nodal points along the perimeter of column core, producing peak pressure regions In welded reinforcement grids the welds provide rigid nodal points to resist lateral expansion of concrete, providing lateral points of restraint at which the confinement pressure peaks As the number of cells in a grid increases, the pressure peaks approach each other generating near uniform confinement pressure which improves the confinement efficiency. Figure 70 illustrates non-uniform confinement pressure in columns with rectilinear reinforcement and the improvement in pressure with closely spaced nodal points

Once the moment-curvature relationship is constructed for a given column section with specific confinement characteristics. the distribution of curvatures along the height of the column can be established The distribution of curvatures within the hinging region requires consideration of gradual plastification of the region in the post-yield range An algorithm was developed and used by Razvi and Saatcioglu (11) to simulate the progression of hinging in column critical regions so that the curvature diagrams can be established for plastic hinge regions. Figure 71 illustrates the gradual formation of a plastic hinge in a column The area under the curvature diagram (integration of curvature diagram gives displacement due to flexure

Anchorage Slip Deformations

The penetration of yielding of flexural reinforcement into the adjoining member causes anchorage slip which results in a member end rotation This is often the case in reinforced concrete frame structures subjected to earthquake loads Beams and columns develop their maximum moment sections near adjacent members when subjected to lateral seismic loading Yielding of flexural reinforcement in these sections penetrates into the adjacent member, causing the extension of reinforcement within the adjoining member This extension may become significant if the reinforcement strains into strain hardening The cumulative extension of reinforcement within the adjoining member results in a wide crack at the interface. leading to a rigid-body rotation of the member This rotation may translate into a significant lateral drift if the level of axial compression is low.

An additional component of anchorage slip may be caused by bar slip. Bar slip occurs when the embedded steel is strained to its far end In this case the bond stress around the bar must increase to resist the applied tension in the bar Increased bond results in the slippage of reinforcement, amount of which can be established from a local bond-slip relationship Reinforcement slip is not common in well designed and detailed members since the embedment length is usually sufficient However, the extension of reinforcement within the adjoining member can not be avoided if the critical section in flexure is located at the end of the member. Reinforcement extension, with or without bar slip may be referred to as "Anchorage Slip" Deformations due to anchorage slip was computed in the analysis

of **columns** discussed in the subsequent section. following the analytical procedure developed by Alsiwat and Saatcioglu (7)

Force-Disolacement Relationships

Lateral force-total displacement relationships of columns. caused by flexure and anchorage slip, can be computed following the procedure described in preceding sections An extensive verification of the analytical procedure was made by comparing against test data (9,10,11). Figures 72 and 73 show selected sample comparisons of experimentally recorded force-displacement relationships with those obtained analytically (1 I) The results show that the analytical technique employed can be used to reproduce inelastic behavior of columns confined with conventional tie reinforcement fairly accurately.

The same analysis procedure was applied to columns companion to those tested in this investigation, except this time confined with conventional ties. The analytical model for confined concrete was developed for conventional ties. consisting of perimeter hoops. overlapping hoops and/or crossties Figure 74 shows the comparisons of the envelopes of experimental force-displacement hysteretic relationships for columns confined by welded grids with analytically generated force-displacement curves for columns confined with conventional ties. The comparisons indicate that columns confined with welded grids consistently showed approximately 5 to 10% higher strengths. In general, inelastic deformability beyond the peak, as indicated by the slope of the curves, was similar to those indicated by columns with conventional ties. The superior performance of columns with welded grids may be attributed to the improved confinement characteristics of grids associated with increased rigidity of welded ties. This point is a subject for further research by the authors.

CONCLUSIONS

The following conclusions may be drawn from the investigation conducted on column confinement with welded reinforcement grids.

- Welded reinforcement grids can be used as transverse confinement reinforcement provided that 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 indicated approximately 7% to 10% tensile strains at rupture, maximum tensile strains recorded during column tests were below 1% at 4% lateral drift when the steel grade ranged between 570 MPa and and MPa, implying that the maximum elongation requirement may be significantly lower than the rupturing strain recorded during coupon tests No grid failure was observed dunng column tests prior to either a longitudinal bar buckling in compression or a longitudinal bar fracture in tension Only two columns exhibited grid fracture beyond these limiting load conditions
- Columns confined with reinforcement grids showed ductile response, developing lateral drifts equal to or greater than those expected in columns confined with conventional ties of equal volumetric ratio. grade. arrangement and spacing With the exception of Column BG-1. which had a grid spacing equal to one half the cross-sectional dimension and the volumetric ratio equal to 65% of that required by ACI3 18-95 and hence developed 1% lateral drift. all others showed drift capacities of 2% and higher. even when the P-∆ effect was considered Columns conforming the spacing requirements of ACI318-95 with volumetric ratios ranging between 83% and 129% of that required by the code showed ductile response even under high axial compression of approximately 40% of nominal concentric capacity, developing drifts of 3% to 6% prior to 20% degradation in moment resistance The same columns developed 2% to 4% drift prior to similar degradation in lateral force resistance Companion columns tested under 20% of nominal concentric capacity higher deformabilities. developing 6% to 7% drift prior 20% degradation in moment resistance and 4% drift at 20% decay in lateral force resistance
- grids
- . 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 4 corner bars only Column BG-6 with 4 corner longitudinal reinforcement and 9-cell grids (having 4 welded nodal points per side) showed ductile response similar to that expected from

a 12-bar arrangement with conventional ties This observation. however. 15 based on only one column test, and further verification may be necessary

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

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- 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 aspect may be 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 is authorized for use in reinforced concrete columns The observations made in this research program indicate that welded grids must.
 - i) have welded joints stronger that the steel itself. and
 - ii) a minimum elongation of 4%, when determined by a standard coupon test where the coupon contains a welded joint in the center

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NOTATIONS

b _d	Diameter of transverse reinforcement
\mathbf{f}_{c}	Concrete stress
f' _c	Concrete cylinder strength
f' _{cc}	Strength of confined core concrete.
feo	Strength of unconfined concrete in column
$\mathbf{f}_{\mathbf{y}}$	Yield strength of longitudinal reinforcement
f_{vt}	Yield strength of transverse reinforcement
F	Lateral force
L_{pl}	Plastic hinge length
Р	Axial compressive force
P _o	Concentric capacity of column, calculated as per Eq 1
М	Bending moment
$M_1,\ldots M_4$	Bending moment at different stages of loading
S	Spacing of transverse reinforcement
E	Strain
$\epsilon_{_{01}}$	Strain corresponding to unconfined peak stress of concrete in column
ϵ_1	Strain corresponding to confined peak stress of concrete.
€ ₀₈₅	Strain corresponding to 85% of unconfined peak stress of concrete in column on the
	descending branch
ϵ_{20}	Strain corresponding to 20% of confined peak stress of concrete on the descending
	branch
e ₈₅	Strain corresponding to 85% of confined peak stress of concrete on the descending
	branch
φ	: Curvature
$\varphi_1 \ \ \varphi_4$	Curvatures at different load stages
ρ	: Longitudinal reinforcement ratio.
ρ_s	: Volumetric ratio of transverse reinforcement
$(\rho_s)_{ACI}$	Volumetric ratio of transverse reinforcement as required by ACI3 IS-95 building code