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Seismic Performance of Reinforced Concrete Columns with Plain Longitudinal Reinforcing Bars

by Thanh Phuong Pham and Bing Li

This paper presents an experimental investigation carried out on nine reinforced concrete (RC) columns with plain longitudinal reinforcing bars subjected to simulated seismic loadings. The variables of interest are axial loads, aspect ratios, and cross-sectional shapes. The performance of specimens was discussed in terms of crack patterns, hysteretic response, initial stiffness, shear strength, drift capacity, and energy dissipation capacity. The test results were compared with available experimental data of five RC columns reinforced with deformed reinforcing bars. The comparisons highlighted that apart from lower shear capacities; columns with plain longitudinal bars have rather large initial stiffness and drift capacities, even higher than columns with deformed longitudinal reinforcing bars. Further comparisons with the existing model for evaluation of existing structures revealed that the ASCE/SEI 41-06 provision tends to overestimate initial stiffness and shear strength, but substantially underestimates drift capacities of columns with plain reinforcing bars. A correction factor is therefore proposed when employing ASCE/SEI 41-06 concrete provisions for shear strength evaluation of RC columns with plain longitudinal reinforcement.

Keywords: axial compression failure; plain reinforcement; reinforced concrete column; seismic loading; shear strength; splitting failure.

INTRODUCTION

There has been increasing emphasis in many countries on seismic assessment of existing structures designed to pre-1970s seismic codes, when plain reinforcing bars were commonly used to reinforce concrete members. Earthquake observations showed that numerous buildings suffered severe seismic damage attributable to the use of plain longitudinal reinforcement in columns. Studies¹⁻⁴ have been devoted to further understand the seismic behavior of such reinforced concrete (RC) columns, which is essential in the structural evaluation of existing structures. The current understanding, however, is generally limited to concrete members reinforced with deformed reinforcing bars.

There are fewer studies on the seismic response of columns reinforced with plain round reinforcing bars compared with those on columns built with deformed reinforcing bars. Verderame et al.^{5,6} conducted a test on 12 columns to assess the sources of deformation related to poor bond performances of smooth plain reinforcing bars. The test results showed that the main difference in the deformation mechanism between columns with plain reinforcing bars and deformed reinforcing bars was the extensive yielding of plain reinforcing bars in column-foundation interfaces. Yalcin et al.⁷ also reported experimental results of two columns reinforced with plain reinforcing bars in comparison with three columns retrofitted with carbon fiber reinforced polymer (CFRP). Ozcan et al.⁸ tested five column specimens with

plain reinforcing bars focusing on the effectiveness of CFRP wrapping retrofit technique. Arani et al.⁹ conducted a test on four columns reinforced with plain reinforcing bars and also found out that slip (fixed-end rotation) contribution was the major source of deformation. Effectiveness of strengthening methods on behavior of concrete columns with plain reinforcing bars was investigated by Bousias et al.¹⁰ through CFRP wrapping and RC jacketing methods; the results showed that the un-retrofitted specimens had low deformation capacity and energy absorption capacity. Pandey and Mutsuyoshi¹¹ tested 15 RC columns to examine the influence of bond between longitudinal reinforcement and concrete on the seismic behavior of the test specimens, the test results demonstrated that the failure mode of the test specimens at the ultimate state could be changed from shear to flexure by reducing the bond strength of the longitudinal reinforcing bars.

There are two shortcomings in the previous studies reviewed. First, none of the tests on columns with plain reinforcing bars was conducted under double curvature bending, so that several important aspects which seldom occur in cantilever tests tended to be ignored, particularly the splitting bond failure. Second, there is limited understanding about the difference between columns with plain reinforcing bars and columns with deformed reinforcing bars. Thus, the actual seismic performance of RC columns with plain longitudinal reinforcing bars is still far from being well understood.

This paper contributes to further understanding of the seismic behavior of existing RC columns reinforced with plain longitudinal reinforcing bars. Nine half-scale RC columns were tested under a combination of constant axial load and double curvature bending simulating seismic load. The test results were examined in terms of cracking patterns, hysteretic response, initial stiffness, shear strength, drift capacities, and energy dissipation capacity. The results were then compared with previous experimental data of five columns with similar details, but reinforced with deformed longitudinal reinforcing bars¹² and results predicted using ASCE/SEI 41-06.¹³

RESEARCH SIGNIFICANCE

Concrete columns reinforced with plain reinforcing bars were commonly built in historical structures worldwide and are currently required for seismic assessment. This paper

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Table 1—Summary of test specimens

Specimen	b , mm	h , mm	L , mm	a	A_s , mm ²	A_v , MPa	s , mm	f'_c , MPa	$P/f'_c A_g$	V_y , kN	V_u , kN	V_n , kN
SP1-1.7-0.2	350	350	1200	1.7	2512	56.52	125	29.8	0.2	286.7	328.3	304.1
SP2-1.7-0.35	350	350	1200	1.7	2512	56.52	125	29.2	0.35	353.3	363.3	359.5
SP3-2.4-0.2	350	350	1700	2.4	2512	56.52	125	30.6	0.2	202.4	231.8	233.6
SP4-2.4-0.35	350	350	1700	2.4	2512	56.52	125	28.7	0.35	249.4	256.5	272.7
SR1-1.7-0.35	250	490	1700	1.7	2512	56.52	125	23.3	0.35	251.6	302.4	357.5
SR2-1.7-0.5	250	490	1700	1.7	2512	56.52	125	22.5	0.5	289.5	312.6	321.1
SW1-1.4-0.2	200	610	1700	1.4	2412	56.52	125	21.7	0.2	272.6	327.6	382.3
SW2-1.4-0.35	200	610	1700	1.4	2412	56.52	125	20.2	0.35	315.3	351.8	423.1
SW3-1.4-0.5	200	610	1700	1.4	2412	56.52	125	20.5	0.5	332.5	365.8	501.7

Notes: 1 kN = 0.025 kip; 1 mm = 0.04 in.; 1 MPa = 0.145 ksi.

Fig. 1—Detail of specimens. (Note: Dimensions in mm; 1 mm = 0.04 in.)

contributes to further understanding of the seismic response of this type of RC columns. Comprehensive comparisons with test results of columns with deformed reinforcing bars and with the ASCE/SEI 41-06 provide valuable information for consulting structural engineers.

EXPERIMENTAL PROGRAM

Specimen details

Nine half-scale columns were designed for the tests with combined axial loads and cyclic shears as shown in Fig. 1 and Table 1. The columns' geometries, details, and materials were selected to be representative for old-type columns in existing buildings. The test columns were divided into three groups: square sections, rectangular sections, and wall-like column sections. All columns were designed to have a similar cross-sectional area and longitudinal reinforcement ratio. The clear height of specimens was set to be 1200 or 1700 mm (47.24 or 66.93 in.), resulting in a shear span-depth ratio (aspect ratio) of 1.7, 2.4 for square and rectan-

gular columns, or 1.4 for wall-like columns. All longitudinal reinforcements used were smooth reinforcing bars with a yield stress f_y of 320 MPa (46.4 ksi). Six square and rectangular columns were reinforced with eight R20 reinforcing bars (diameter of 20 mm [0.787 in.]), resulting in a longitudinal reinforcement ratio of 2.05%. The other three columns were reinforced with 12 R16 reinforcing bars (diameter of 16 mm [0.630 in.]) with a longitudinal reinforcement ratio of 1.98%. The transverse reinforcement of all test specimens comprised of R6 mild steel reinforcing bars with 135-degree bent, spaced at 125 mm (4.92 in.), were characterized by a yield strength f_{yt} of 500 MPa (72.5 ksi). Theoretical flexural strengths V_u of the test specimens were estimated using the material properties obtained through tests and in accordance with the recommendations provided by ACI-318.¹⁴ The nominal shear strengths V_n of the test specimens were calculated based on the suggestion of ASCE/SEI 41-06.¹³ The values of V_u and V_n of the test specimens are tabulated in Table 1.

Test setup

Test specimens were attached to a loading apparatus which enabled the columns to be displaced under double curvature bending, as shown in Fig. 2. Two vertical actuators were used to control the level of axial force as well as to maintain the boundary condition of zero rotation at the top of the specimen. A horizontal actuator was used to produce cyclic shear force to the top end of specimens. All actuators were pinned at both ends to allow rotation during the test. The base of the specimens was fixed to a strong floor held by four post-tensioned bolts. The axial load was applied to the specimens until the designated level. During each test, the column axial load was maintained by manually adjusting the vertical actuators after each loading step. The lateral load was applied cyclically through the horizontal actuator in a quasi-static fashion, as shown in Fig. 2.

Instrumentations

The test specimens were extensively installed with measuring devices both internally and externally. Lateral displacement was measured by two horizontal LVDTs parallel to the horizontal actuator. Shear and flexural deformation was recorded based on the reading of a number of

LVDT sets located throughout the height of the specimens. Arrangement of strain gauges that were mounted to capture strains in both longitudinal and transverse reinforcements at critical sections is as shown in Fig. 1.

EXPERIMENTAL RESULTS AND DISCUSSIONS

This section presents and discusses the test results of nine specimens reinforced with plain longitudinal reinforcing bars. Hysteretic responses of all test specimens are depicted in Fig. 3. The response of test specimens was highlighted by four performance levels: flexural yielding (PL1), maximum shear force (PL2), shear failure (PL3: shear strength dropped

by 20%), and axial failure (PL4; when columns were unable to sustain the applied axial force). Crack patterns of test specimens at PL2 and PL4 are shown in Fig. 4 and 5, respectively. Generally, the hysteretic responses illustrate the pinching effect, reduction of stiffness, and reduction of shear capacity during repeated cyclic loadings. Except for SR1, all specimens did not reach theoretical yield force, and eventually did not reach nominal shear strength and flexural strength. Strain gauge readings indicated that only in SR1 the yielding of longitudinal reinforcement occurred before the maximum shear force was recorded. Behavior of test specimens is described as follows.

Square section columns

In the first short column, SP1, with an aspect ratio of 1.7, vertical cracks were observed along the center reinforcing bars around the mid-area of the column height at a drift ratio of 0.33%. Surprisingly, at this stage, there were only two flexural cracks with the length of one-fourth the column section height at the bottom end. The appearance of the vertical cracks was the result of the forming and joining of a series of micro-inclined cracks crossing each other along the length of mid-bar. After reaching the maximum shear force at a drift ratio of 1.67%, the specimen tended to separate into two parts along the vertical cracks. There was a sudden loss of shear capacity right after the maximum horizontal load was attained, as seen in the hysteretic response in Fig. 3. This was due to the appearance of a diagonal crack forming

Fig. 2—Experimental setup.

Fig. 3—Hysteretic response of test specimens. (Note: V_u is flexural strength; V_n is nominal shear strength; PL1 is yielding of longitudinal reinforcement; PL2 is maximum shear force attained; PL3 is shear failure, shear strength dropped by 20%; PL4 is axial failure; 1 mm = 0.04 in.)

Fig. 4—Crack patterns at maximum shear force.

Fig. 5—Final failure modes of columns with plain reinforcing bars.

Fig. 6—Typical axial failure modes.

with an angle of approximately 45 degrees to the column axis at the top end of the specimen, as depicted in Fig. 4. Unlike diagonal failure observed in other column tests^{1,3,4,15} in which the diagonal cracks developed throughout column height and forming a diagonal sliding plane, the diagonal crack observed in SP1 only developed from the column-base interface to the middle region of the column section. After shear failure occurring at a drift ratio of 2.2%, the column was still able to sustain the axial force until a large drift ratio

of 4.0% because the diagonal crack did not develop further. The test ended with a vertical cleavage formed throughout the column section; this is defined as Failure Mode 1. Five transverse reinforcements at the center height were fractured at the end of the test, as depicted in Fig. 6, which was due to the sliding movement of the column along the cleavage.

Figure 7 depicts the variation of strains in longitudinal reinforcing bars and transverse reinforcements at different drift ratios for SP1. It is noted that the strain patterns of other specimens were found similar to strain patterns of SP1. Under an initial applied axial stress of $0.2f'_c$, the average compression strain in longitudinal bars was approximately 0.0004, which was approximately 25% of the yield strain. When loaded to a drift ratio of 1.2%, strains in longitudinal bars increased linearly, while strains at all transverse reinforcements remained relatively low at initial strain value. After a drift ratio of 1.2%, strains in longitudinal bars at different locations varied differently. This variation was due to the development of flexural cracks, which led to the redistribution of stress along the length of the main bars. At the point when the diagonal cracks first appeared, strains in transverse reinforcements at both column ends increased significantly. There was no yielding, however, of both longitudinal and transverse reinforcements observed until a

drift ratio of 3%, beyond the point which shear failure had occurred. Compared with the case of columns with deformed reinforcing bars in the Tran and Li tests,¹² strain at different locations of longitudinal bars in the column with plain reinforcing bars did not vary significantly as in columns with deformed reinforcing bars.¹

In another short column, SP2, with a higher applied axial force ($P/f'_c A_g = 0.35$), vertical cracks were also observed at a relatively early stage, at a drift ratio of 0.4%, along the center reinforcing bars. These vertical cracks fully propagated along the column height in the next cycle (at a drift ratio of 0.5%). Dissimilar to SP1, however, the vertical cracks in SP2 only developed until a drift ratio of 1%. At this stage, diagonal cracks, in the direction of 70 degrees to the column vertical axis, appeared and quickly spread from the top to the bottom end of the specimen. As expected, under a higher level of axial force, the column responded more brittle. Thus, the specimen reached its maximum shear force at this point, and suddenly lost its shear capacity due to sliding along the diagonal failure plane. This is defined as Failure Mode 2. It was observed that strain distribution in the reinforcements of SP2 was almost similar to that of SP1, except at a drift ratio of 1.25%, when strains in transverse reinforcement grew rapidly and exceeded the yield value. This also resulted from the sliding movement of the diagonal plane.

SP3 and SP4, with an aspect ratio of 2.4, behaved in a manner in which flexural response was dominant until the end of the tests. The calculated shear deformations for these two specimens were relatively small, at approximately 8% of the total displacement. Both specimens only developed flexural cracks at column ends before maximum was reached. SP3 attained maximum shear force at a drift ratio of 1.96%, while SP4 reached shear strength at a drift ratio of 1.25% due to a 75% higher applied axial force level. Without diagonal cracks, the response of SP3 and SP4 remained stable, with gradual reduction in shear capacity even after shear failure. Both specimens failed by crushing of concrete at both column ends and buckling of longitudinal reinforcement in these regions.

Rectangular section columns

The rectangular specimens with an aspect ratio of 1.7, SR1 and SR2, were tested under the applied axial force level $P/f'_c A_g$ of 0.35 and 0.5, respectively. SR1 attained the maximum shear force at a drift ratio of 1.2%, when some diagonal cracks and crushing of concrete at both ends were observed. Right after that point, vertical cracks were formed along the mid-longitudinal bars, starting from center area and developing to the ends of the specimen. There was also a diagonal crack at mid-height of the column that connected the vertical cracks along two mid-longitudinal reinforcements. The test on SR1 was ended by the splitting of the vertical crack, crushing of concrete, and buckling of longitudinal reinforcements at both ends of the specimen. This failure mode was similar to Failure Mode 1 observed in the test of SP1. Strain gauge readings indicated that no yielding of transverse reinforcement was observed until shear failure was attained. The shear deformation was relatively small, at approximately 10% of the total horizontal deformation.

Fig. 7—Typical strain gauges reading, Specimen SP1. (Note: Dimensions in mm; 1 mm = 0.04 in.)

SR2 reached the maximum shear force with the appearance of two vertical cracks along center longitudinal bars starting at a distance of one half column section height to the bottom base. Crushing of concrete at the bottom end resulted in significant loss of shearing capacity. Spalling of concrete and buckling of longitudinal reinforcing bars at the bottom end caused axial failure. The calculated shear deformation was relatively small, approximately 10% of total deformation; it was almost the same as in SR1.

Wall-like columns

Three wall-like columns, SW1, SW2, and SW3, were tested under three different axial force levels: $0.2f'_c A_g$, $0.35f'_c A_g$, and $0.5f'_c A_g$, respectively, but failed by the same failure mode as Failure Mode 1. Observations in all three specimens found that there was no obvious flexural crack at the column-base interface when loading to maximum shear force (Fig. 4). Thus, the contribution of slip deformation to the total deformation was not significant. It seemed that increasing the axial force from SW1 to SW3 reduced the diagonal cracks.

In SW1, with an applied axial force of $0.2f'_c A_g$, diagonal cracks with an angle of approximately 65 degrees first appeared at a drift ratio of 0.67%, and continued developing until a drift ratio of 1%, when the specimen reached maximum shear force. Right after this point, splitting cracks were observed in the mid-longitudinal bars. In subsequent cycles, the shear capacity gradually reduced, which was caused by the opening of the vertical cracks and crushing of concrete at both column ends. At the end of the test, concrete spalling formed a damage area along the mid-bars, as shown in Fig. 5.

In SW2 with an applied axial force of $0.35f'_c A_g$, only few diagonal cracks occurred at column ends with an angle of approximately 40 degrees when loaded to a drift ratio of

Fig. 8—Comparisons of backbone curves of columns with plain reinforcing bars. (Note: 1 mm = 0.04 in.; 1 kN = 0.225 kip.)

0.8%. These cracks did not propagate though the out-column section as in SW1, but only started from the mid-longitudinal bars. The vertical cracks were formed at a drift ratio of 1% by the crossing of diagonal cracks developed at two different directions that corresponded to pulling and pushing of the horizontal actuator. This occurred earlier than that in SW1. The test on SW2 was ended at a drift ratio of 2.3% by the development of the vertical cracks, the buckling of longitudinal reinforcement, and spalling of concrete at column corners. The crack pattern at the final stage of SW2 was similar to the axial failure mode of SW1.

In SW3, tested under a high axial force of $0.5f_c'A_g$, vertical cracks were observed at an early stage when loading to a drift ratio of 0.5%. At this stage, there were only very minor cracks at the column bottom end, while no obvious crack was observed at the column top end. The vertical cracks appeared and propagated at all three middle longitudinal reinforcements in subsequent cycles. The column remained at a stable response until a drift ratio of 1.7%, then reached axial failure by buckling of the main bars at the bottom end.

Comparisons and discussions of test results

Failure modes—Most of the column tests with plain reinforcing bars in this study appeared to be controlled by vertical cracks that led to splitting failure. It should be noted that the observed vertical cracks are different from the vertical cracks found in the grain silos which were presented in the article by Collins et al.¹⁵ In the case of silos, the concrete walls are expected to bear high circular tensile stress resulted from horizontal pressure exerted by the grain storage, while the axial force is not significant. Thus, when the horizontal (hood) reinforcement of the silos yielded, vertical cracks in concrete would be inevitable. In this study, however, the strain gauge reading indicated that no yielding of transverse reinforcement was observed before maximum shear strength was attained. Moreover, vertical cracks occurred in the test specimens before maximum shear force was attained. Therefore, horizontal tensile stress would not be the source of vertical cracks in test specimens in this program.

Splitting failure was also observed in tests on RC columns with deformed reinforcing bars done by Aoyama,¹⁶ Gupta and Collins,¹⁷ Tran and Li,¹² Pham and Li.¹⁸ In the most recent attempt to explain this, Aoyama¹⁶ assumed the columns to be in a fully developed tensile hinge zones before the formation of vertical cracks. Observations from earthquakes and experiments, however, have shown that

the vertical splitting cracks may develop at very small horizontal displacement, and do not require the formation of flexural cracks or a plastic hinge zone. Moreover, Aoyama¹⁶ assumed external force had resulted in compression in transverse reinforcement, which was not reasonable because the stirrups normally carry tensile stress. To answer this question, an analytical model has been developed in another study,¹⁹ in which the splitting failure was proved to occur when the sliding shear stress along vertical cracks reached its splitting shear capacity. This stage can be reached even when concrete in the compression zone (at two column ends) and transverse reinforcement were still in elastic range. The explanation is in good agreement with the observation of the failure pattern at the end of the tests, as shown in Fig. 6, in which transverse reinforcements at the central region of the columns were fractured in a direction that was parallel to the column's axis.

The columns that failed in Failure Mode 1 developed very limited horizontal cracks at column-base interfaces; therefore, the contribution of slip deformation to the total deformation was not significant. This is remarkably different from the cases of columns tested under single curvature, as discussed in the introduction,^{5,6,9} in which the main source of deformation in RC columns with plain reinforcing bars was found due to the slip deformation of longitudinal reinforcing bars at connection interfaces. It is noted that, when tested as a cantilever, the loading and boundary condition at the top of the columns do not allow formation of a splitting failure mechanism. Moreover, because there is no concentrated stress along the length of the plain reinforcing bars, the appearance of flexural cracks will be limited; slip at the connections is then inevitable. In the tests of RC columns under double curvature, however, as in this study, vertical sliding along the splitting cracks contributed significantly to hysteretic energy, and prevented slip deformation at column-base interfaces.

Shear strength—Comparisons of backbone curves representing the response of the three groups are shown in Fig. 8. The effects of various test parameters to the performance of specimens are presented in Fig. 9 through 11. The shear strength of test specimens was normalized as $V_{max} / (\sqrt{f_c'A_g})$, and is illustrated in Fig. 9(a). As it has long been recognized that the shear strength is beneficial from high axial force, Fig. 10(a) depicts that shear strength increased with increasing applied axial force. When the axial force ratio ($P/f_c'A_g$) increased from 0.2 to 0.35, the normalized shear

Fig. 9—Effects of test parameters on: (a) shear capacities of columns; and (b) initial stiffness. (Note: 1 kN/m = 5.71 kip/in.)

Fig. 10—Effects of test parameters on drift capacities of column specimens.

Fig. 11—Effects of test parameters on energy dissipation capacity. (Note: 1 kNmm = 8.85 lps.in.)

strength of columns with an aspect ratio of 1.7 and wall-like columns was increased by approximately 12%, while only a slight increment of approximately 5% was recorded in square columns with an aspect ratio of 2.4. It is understandable because the effect of axial force on shear strength is reflected by an arch mechanism. This effect is reduced with an increment of shear span (aspect ratio). It also explains, as shown in Fig. 9(a), that shear strength increased with a decreasing aspect ratio. At the axial force ratio of 0.35, the highest normalized shear strength of 0.48 was found in SW2, which was 9% higher than that of SR1. This value fell to 0.40 in SP2, and dropped to 0.30 in SP4.

The contribution of axial force to shear strength was not significant when the test specimens were subjected to high levels of axial compression. It was shown that when the applied axial ratio force was increased from 0.35 to 0.5, the normalized shear force increased by approximately 7%

in all specimens. This observation has been found in few studies^{18,20}; however, it has not been considered in modern design and assessment codes.^{13,14} Therefore, in some cases,^{17,18} the codes may provide unconservative predictions for the shear strength of RC columns. In summary, axial force has minor effect on the shear strength of columns with high aspect ratios. It also shows evidence that shear strength is beneficial from high axial force if the axial force level is not greater than a certain ratio $(P/f'_c A_g = 0.35)$. This observation agrees with experimental and numerical results of RC columns with deformed reinforcing bars in a previous study.¹⁸

Initial stiffness—Initial stiffness is normally determined based on a point obtained from the measured force-displacement envelope with a shear force that is equal to the theoretical yield force. This is defined as either the first yield that occurs within the longitudinal reinforcement, or when

the maximum compressive strain of the concrete attains a value of 0.002 at any critical section of the column. This definition, however, would not be applicable to columns in this investigation, in which the shear strength does not substantially exceed its theoretical yield force. In this case, therefore, the initial stiffness is defined based on a point on its measured force-displacement envelope with a shear force that equates to 80% of the obtained maximum shear force. Figure 9(b) demonstrates the effects of axial load, aspect ratio, and sectional shape on initial stiffness. It is generally shown that the initial stiffness increased when the applied axial force was increased. The largest change was recorded in short square columns, in which the initial stiffness increased 24% from 25.8 kN/mm (147,215 kip/in.) in SP1 to 31.9 kN/mm (182,021 kip/in.) in SP2. The incremental rate of initial stiffness was slowed down to 21% from SP3 to SP4. This value was 16% when axial force was increased from SR1 to SR2. The most remarkable result was found in wall-like columns in which initial stiffness increased very slightly, 3.6 and 6.3%, when the axial force ratio was increased from 0.2 to 0.35 and 0.5.

Drift capacities—The effects of test parameters on the drift capacities (drift ratio at maximum shear force, at shear failure, and at axial failure) are shown in Fig. 10. Overall, drift capacities were decreased when applied axial force was increased. The drift ratio at maximum shear force dropped sharply in the group of square columns when the axial force ratio increased from 0.2 to 0.35. The drift degradation was 48.2% from SP1 to SP2, and 57% from SP3 to SP4. Due to the difference in failure mode with the sudden loss of shear capacity in SP2, the gap in drift capacities between SP1 and SP2 remained at shear failure and axial failure. Apart from a slight slope between SP3 and SP4 at shear failure, the trends of drift capacities for other specimens remained consistent at different stages of the tests, where wall-like columns showed the lowest drift capacities.

Energy dissipation capacity—Energy dissipation capacity was defined as the area under the load-displacement hysteresis curves. Effects of test parameters on energy dissipation capacity at different performance levels are shown in Fig. 11. The observed trends were almost similar to that of drift capacities. The total energy dissipated (at axial failure) depends on not only aspect ratios and applied axial force, but also sectional shapes. Provided the same cross-sectional shape and decreasing the aspect ratio from 2.4 to 1.7, the total energy dissipated decreased by 68% when the axial force ratio was 0.2 (from SP3 to SP1), and by 275% when the axial force ratio was 0.35 (from SP4 to SP2). Having the same aspect ratio and axial load ratio, the rectangular specimen, SR1, produced a much higher energy dissipation capacity compared with that of SP2 with a square section.

COMPARISONS WITH COLUMNS WITH DEFORMED REINFORCING BARS

Five specimens in this study were designed to be similar to columns with deformed reinforcing bars in tests by Tran and Li¹² in terms of specimen geometries, reinforcement ratios, and concrete strength. All tests in both studies were conducted using the same test setup. The columns with

deformed reinforcing bars, including SC-1.7-0.2, SC-1.7-0.35, SC-2.4-0.2, RC-1.7-0.35, and RC-1.7-0.5, were compared with columns with plain reinforcing bars SP1, SP2, SP3, SR1, and SR2, respectively. Comprehensive comparisons were then possible to find out how the surface characteristic of longitudinal reinforcement influences the seismic response of RC columns.

Comparisons of overall response and failure modes

Comparisons of backbone curves between columns with plain reinforcing bars and deformed reinforcing bars are shown in Fig. 12. Crack patterns of columns with deformed reinforcing bars are presented in Fig. 13. Most columns with plain reinforcing bars performed in flexural dominant failure mode. Even those specimens designed with light transverse reinforcement exhibited sufficient flexural deformations; shear deformation was relatively small, at 10 to 15% of total deformation, when maximum shear force was attained. On the other hand, significant shear deformations were recorded in columns with deformed reinforcing bars; the ratio was between 20 and 40%. This was because all columns with deformed reinforcing bars experienced substantial diagonal cracks during the elastic range, while columns with plain reinforcing bars developed vertical cracks from early stage. When the vertical cracks occurred, the external energy was mainly dissipated by the splitting of concrete through sliding between two separated parts along the cracks. Thus, the longitudinal and transverse reinforcement did not yield, even when shear failure occurred in columns with plain reinforcing bars. On the contrary, the diagonal cracks developed on columns with deformed reinforcing bars, a result of combined shear and flexural stresses, were inclined at an angle to the axis of the columns. These cracks resulted in yielding of transverse reinforcement before the columns reached maximum shear force. The columns with deformed reinforcing bars also developed flexural yielding before the maximum shear force was attained. Because of the low bond condition between reinforcement and surrounding concrete due to plain reinforcing bars, the tensile force carried by longitudinal reinforcement, if any, was distributed more regularly along its length than columns with deformed reinforcing bars. Therefore, in columns with plain reinforcing bars, strain along longitudinal reinforcing bars did not vary significantly, and crack spacing was larger than in columns with deformed reinforcing bars.

From Fig. 13, it can be seen that only the rectangular section column under the applied axial force of $0.5f'_cA_g$ failed with the same failure mode. Specimens with plain reinforcing bars SP1, SP3, and SR1 failed by the formation of vertical cracks; the specimens with deformed reinforcing bars SC-1.7-0.2, SC-0.2-0.2, and RC-1.7-0.2 failed by the development of diagonal shear plan. The reversal change was found in SP2 and SC-1.7-0.35. Therefore, this shows evidence that the surface of longitudinal reinforcement changes the failure mechanism of RC columns.

Comparisons of initial stiffness

As mentioned in the preceding section, the initial stiffness of columns with deformed reinforcing bars might be calcu-

Fig. 12—Comparisons of backbone curves between columns with plain reinforcing bars and deformed reinforcing bars. (Note: 1 mm = 0.04 in.; 1 kN = 0.225 kip.)

Fig. 13—Axial failure of columns with deformed reinforcing bars.¹²

lated based on the yielding point of longitudinal reinforcement. Most columns with plain reinforcing bars, however, did not develop flexural yielding; therefore, to be consistent, initial stiffness of all columns was calculated based on the first point of 80% of shear strength obtained. The comparison of initial stiffness is tabulated in Table 2. Interestingly most columns with plain reinforcing bars possess higher initial stiffness compared with columns with deformed reinforcing bars. The biggest difference was found in rectangular section columns, where plain reinforcing bars resulted in a stiffness improvement of 71 and 108% on SR1 and SR2, respectively. The comparisons also showed a trend for short columns that deformed reinforcing bars do not produce higher initial stiffness. The reason was because the flexural cracks occurred in columns with deformed reinforcing bars, making its initial stiffness gradually reduced. It was observed that all specimens with an aspect ratio of 1.7 did not develop many flexural cracks; instead, those specimens tend to fail by splitting cracks.

Comparisons of shear strength

The comparisons of normalized shear strengths are summarized in Table 2. The results demonstrate that in all cases, the columns with plain reinforcing bars produced shear strength significantly less than the columns with deformed reinforcing bars. The reduction of shear strength was almost similar in the three square section columns, and was approximately 33%, while it was approximately 21% in the rectangular section columns. These reductions did not seem to be sensitive to the change of aspect ratio or applied axial force, but were relatively sensitive to the variable of the cross-sectional shape of the columns.

Under the combination of axial force and shear force, the columns may form vertical splitting cracks along the length of the mid-reinforcing bars. As observed in this study, the shear strength of the test specimens was likely to be controlled by its splitting strength. Naturally, plain reinforcing bars yield lower bond capacity than deformed reinforcing bars, leading to lower splitting strength. Therefore, the columns with plain reinforcing bars predictably produced lower shear strength than columns with deformed reinforcing bars.

A previous test conducted by Pandey and Mutsuyoshi¹¹ also found that the specimen with plain reinforcing bars (Specimen C6) produced 22% lower shear strength than the specimen with deformed reinforcing bars (Specimen C1). Faella et al.²¹ conducted a test series on RC columns with plain and deformed reinforcing bars, and with retrofitted and un-retrofitted columns. The results showed that, in all cases, the columns with deformed reinforcing bars possessed higher shear strength than the columns with plain reinforcing bars. The differences ranged from 20 to 28% in un-retrofitted RC columns, and 33 to 43% in retrofitted columns. All of these studies confirm that columns with deformed reinforcing bars possess higher shear capacity than columns with plain reinforcing bars.

Table 2—Comparisons between columns with plain reinforcing bars and columns with deformed reinforcing bars

Specimen	$P/f_c'A_g$	$V_{max}/A_g(f_c')^{0.5}$	I_{in} , kN/mm	δ_m	δ_s	δ_a	Failure mode	f_c' , MPa	Energy dissipation (kNmm) at		
									PL2	PL3	PL4
SP1-1.7-0.2	0.2	0.36	25.8	1.67	2.22	4.00	1	29.8	19.0	22.0	51.4
SC-1.7-0.2	0.2	0.47	22.4	1.30	1.43	1.82	2	27.5	5.0	7.5	13.6
SP2-1.7-0.35	0.35	0.40	31.9	1.13	1.43	1.52	2	29.2	10.2	14.1	18.6
SC-1.7-0.35	0.35	0.54	25.6	1.26	1.44	1.56	1	25.5	5.5	7.5	9.5
SP3-2.4-0.2	0.2	0.29	12.8	1.96	2.87	4.06	1	28.2	25.8	57.6	86.3
SC-2.4-0.2	0.2	0.38	14.2	1.44	1.98	2.82	2	22.6	10.5	22.0	34.0
SR1-1.7-0.35	0.35	0.45	31.6	1.02	1.81	2.80	1	23.3	13.2	33.6	59.0
RC-1.7-0.35	0.35	0.54	15.2	1.45	1.65	2.02	2	27.1	11.0	21.5	25.0
SR2-1.7-0.5	0.5	0.47	36.5	0.85	1.36	1.71	1	22.5	14.0	23.0	34.0
RC-1.7-0.5	0.5	0.57	21.4	1.44	1.67	1.80	1	26.8	11.5	15.0	22.5

Notes: δ_m , δ_s , and δ_a are drift ratio at maximum shear force, shear failure, and axial failure, respectively; 1 kN = 0.225 kip; 1 mm = 0.04 in.; 1 MPa = 0.145 ksi.

Comparisons of drift capacities

Generally, columns with deformed reinforcing bars developed flexural yielding before reaching maximum shear force, while columns with plain reinforcing bars reached shear strength before yielding of longitudinal reinforcement. Under lower applied axial force, plain longitudinal reinforcing bars resulted in more ductile response in RC columns. In particular, the specimen with plain reinforcing bars, SP1, reached maximum shear force at a drift ratio of 1.67%, while the specimen with deformed reinforcing bars, SC-1.7-0.2, obtained maximum shear force at a drift ratio of 1.3%, which was 30% earlier. Similarly, with an aspect ratio of 2.4, the column with plain reinforcing bars (SP3) attained maximum shear force at the drift ratio approximately 50% larger than that of the column with deformed reinforcing bars (SC-2.4-0.2). When the applied axial force was increased to $0.35f_c'A_g$, the recorded drift ratios at the maximum shear force were almost identical in both columns with plain reinforcing bars and columns with deformed reinforcing bars. The results were approximately 1.56% for SR1 and RC-1.7-0.35, and 0.8% for SP2 and SC-1.7-0.35. When applying high axial force, $0.5f_c'A_g$, the trend was reversed. The columns with deformed reinforcing bars reached its maximum shear force at a drift ratio equal to almost two times the corresponding drift ratio for columns with plain reinforcing bar.

The drift capacity at axial compression failure of columns with plain reinforcing bars was significantly higher than those of columns with deformed reinforcing bars when the applied axial force was lowest at $0.2f_c'A_g$. The results are tabulated in Table 2, which shows that for the short columns with an aspect ratio of 1.7, the column with plain reinforcing bars, SP1, reached axial failure at 4.0% drift, while the column with deformed reinforcing bars, SC-1.7-0.2, reached axial failure at 1.82% drift. The columns, with an aspect ratio of 2.4, failed at a drift ratio of 4.1 and 2.8% for the columns with plain reinforcing bars and deformed reinforcing bars, respectively. When the applied axial force level was higher than $0.35f_c'A_g$, the drift ratio at axial compression

failure tended to be similar on columns with both deformed and plain reinforcing bars.

Comparisons of energy dissipation capacity

Energy dissipation corresponding to each performance level of all specimens is tabulated in Table 2. Generally, energy dissipation in columns with plain reinforcing bars was significantly higher than that of columns with deformed reinforcing bars at all performance levels. At the point when the maximum shear capacity was attained, the energy dissipated by square columns with plain reinforcing bars was two times the energy dissipated by the square columns with deformed reinforcing bars, but only a slight difference was found for rectangular section columns. The discrepancy in energy dissipation by columns with deformed reinforcing bars and columns with plain reinforcing bars was even increased when loading to the shear failure stage. When axial compression failure occurred, although they failed with the same failure mode, SR2 still dissipated an amount of energy 50% higher than RC-1.7-0.5. In the other four pairs, the observed failure mode changed, and the difference was more remarkable. Energy dissipation was enhanced by 278, 154, 136, and 95% on columns with plain reinforcing bars SP1, SP3, SR1, and SP2, respectively.

COMPARISON WITH ASCE/SEI 41-06

In this part, the backbone curves obtained from the experimental results of all test specimens are compared with assessment results from the ASCE/SEI 41-06 guideline models.¹³ According to ASCE/SEI 41-06, the force-displacement relationship follows the general trend, as shown in Fig. A1 (Appendix A*) for RC columns with strengths limited by its flexural strength. The shear capacity is normalized with respect to its yield force. The deformation indexes (a , b) are defined as flexural plastic hinge rations that depend

*The Appendix is available at www.concrete.org/publications in PDF format, appended to the online version of the published paper. It is also available in hard copy from ACI headquarters for a fee equal to the cost of reproduction plus handling at the time of the request.

Table 3—Comparisons between test results and ASCE/SEI 41-06

Specimen	Initial stiffness, kN/mm			Drift at V_{max} %			Drift at axial failure, %			Shear strength, kN			
	K_{test}	K_{ASCE}	K_{test}/K_{ASCE}	δ_{test}	δ_{ASCE}	$\delta_{test}/\delta_{ASCE}$	δ_{test}	δ_{ASCE}	$\delta_{test}/\delta_{ASCE}$	V_{test}	V_{ASCE}	$V_{Proposed}$	V_{test}/V_{ASCE}
SP1-1.7-0.2	25.8	224.2	0.12	1.67	0.71	2.22	4.00	1.55	2.59	238.3	304.1	228.1	0.78
SP2-1.7-0.35	31.9	222.0	0.14	1.13	0.59	1.90	1.52	1.32	1.15	263.2	359.5	269.6	0.73
SP3-2.4-0.2	12.8	79.9	0.16	1.96	0.80	2.46	4.06	1.63	2.49	193.3	233.6	175.2	0.83
SP4-2.4-0.35	15.4	77.4	0.20	1.25	0.69	1.80	3.06	1.43	2.14	198.0	272.7	204.5	0.73
SR1-1.7-0.35	31.6	136.7	0.23	1.02	0.56	1.81	2.80	1.30	2.16	263.0	357.5	268.1	0.74
SR2-1.7-0.5	36.5	110.8	0.33	0.85	0.46	1.87	1.71	1.16	1.48	272.0	422.4	316.8	0.64
SW1-1.4-0.2	24.4	203.6	0.12	1.05	0.66	1.59	3.31	1.49	2.22	247.8	382.3	286.7	0.65
SW2-1.4-0.35	25.3	196.4	0.13	0.91	0.52	1.74	2.27	1.26	1.81	266.0	423.1	317.3	0.63
SW3-1.4-0.5	26.9	169.8	0.16	0.76	0.43	1.77	1.71	1.13	1.51	289.0	501.7	376.3	0.58
Average			0.18			1.91			1.95				0.70
Coefficient of variation			1.10			0.40			0.71				0.32

Note: 1 kN = 0.225 kip; 1 mm = 0.04 in.

on column axial load, nominal shear stress, and details of columns. The index c is equal to 0.2.

The shear strength as defined in ASCE/SEI 41-06 is given as

$$V_n = k \frac{A_v f_{yt} d}{s} + \lambda k \left(\frac{0.5 \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \quad (\text{MPa}) \quad (1)$$

where k is a parameter to account for the effect of displacement ductility demand; k is taken as 1 for displacement ductility less than 2, as 0.7 for displacement ductility more than 6, and varies linearly for intermediate displacement ductility; M/Vd is the largest ratio of moment to shear times effective depth, should not be taken greater than 4 or less than 2; λ is equal to 1 for normalweight concrete; f_{yt} is the yield strength of transverse reinforcement; d is the distance from the extreme compression fiber to centroid of tension reinforcement; s is the spacing of transverse reinforcement; A_v is the total transverse reinforcement area within spacing s ; and P is the applied column axial load. It should be noted that the shear strength equation of ASCE/SEI 41-06 comes from a regression model on a database of 51 lightly reinforced concrete columns,²² and does not employ the effect of plain reinforcing bars.

Figure A2 (Appendix A) and Table 3 compare the backbone curves of the test specimens with analytical predictions by the ASCE/SEI 41-06 provision. It is shown that ASCE/SEI 41-06 significantly overestimates the initial stiffness of columns with plain reinforcing bars. The stiffness from the guidelines is not consistent, varying from three times to 13 times higher than those from the test. In turn, however, the drift at maximum shear force and at axial failure is underestimated by ASCE/SEI 41-06. On the other hand, the shear strength recorded by the tests was consistently smaller than estimated results from ASCE/SEI 41-06 by approximately 30%. The comparison between tests on columns with deformed reinforcing bars and plain reinforcing bars also showed that shear strength could be improved by 20 to 30%

when replacing the plain reinforcing bars by deformed longitudinal reinforcement. Moreover, previous experimental studies¹⁸ have shown that ASCE/SEI 41-06 provides good estimations of shear strength for RC columns with deformed reinforcing bars. Therefore, it is recommended to multiply a modification factor k_m , $k_m = 1.0$ for columns with deformed longitudinal reinforcement, $k_m = 0.70$ for columns with plain longitudinal reinforcement, to shear strength formula (Eq. (1)) when evaluating shear strength for columns with plain reinforcing bars

$$V_n = k k_m \frac{A_v f_{yt} d}{s} + \lambda k k_m \left(\frac{0.5 \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{P}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \quad (\text{MPa}) \quad (2)$$

Verifications from the current test and previous tests on columns with plain reinforcing bars are shown in Table A1 (Appendix A). The results demonstrate that the mean difference between the modified shear strength formula and results from 19 tests of columns with plain reinforcing bars is almost 0%, with a 6.8% coefficient of variation. This is an improvement compared with shear strength predictions calculated by the ASCE/SEI 41-06 formula (Eq. (1)).

CONCLUSIONS

An experimental study was carried out on nine RC columns with plain longitudinal reinforcing bars under simulated gravity and seismic forces. The experimental results were analyzed and compared with the test results of columns with deformed reinforcing bars, also used to evaluate ASCE/SEI 41-06. The conclusions can be drawn as follows:

1. All columns with plain reinforcing bars failed with the development of vertical cracks forming along the longitudinal reinforcing bars located at mid-height of the column sections. Vertical cracks developed earlier on columns under a higher applied axial load. This type of crack did not produce significant shear deformation, and did not substantially affect the initial stiffness. Shear deformation of test specimens was then generally less than 10% of total defor-

mation before maximum shear force was attained. Unlike columns with plain reinforcing bars tested as cantilevers, the columns tested under double curvature bending did not experience remarkable slip deformation. Instead, the deformation resulting from sliding movements of the column body along splitting cracks was believed to contribute significantly to the energy dissipation capacity.

2. Effects of the aspect ratio, levels of axial force, and cross-sectional shapes on the performance of columns with plain reinforcing bars were similar to those with columns with deformed reinforcing bars. Most remarkably, the shear strength of columns with plain reinforcing bars was beneficial from increasing axial force up to $0.35f_c'A_g$.

3. Different from columns with deformed longitudinal reinforcing bars, most columns with plain reinforcing bars did not develop flexural yielding, and also did not experience the yielding of transverse reinforcement before the maximum shear force was reached. Columns with deformed reinforcing bars produce higher shear strength than columns with plain reinforcing bars, ranging from 22 to 33% depending on cross-sectional shapes, but regardless of applied axial force or aspect ratio. Columns with plain reinforcing bars, however, generally possess higher initial stiffness and significantly higher energy dissipation capacity. Under moderate levels of applied axial compressive force, the failure mechanism might change from diagonal shear failure to splitting failure when deformed reinforcing bars are replaced by plain longitudinal reinforcing bars.

4. The method to assess the performance of RC columns in existing buildings, ASCE/SEI 41-06, inconsistently and substantially overestimates the initial stiffness of RC columns with plain reinforcing bars. Consequently, drift capacities are not well predicted at any acceptable level. Furthermore, the guideline offers an overestimate of shear strength for columns with smooth reinforcing bars that can be proven unsafe in practice. Results from the tests highly recommend multiplying a modification factor of 0.70 for the values predicted by Eq. (6-4) in ASCE/SEI 41-06 for columns with plain reinforcing bars.

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NOTATION

A_g	=	cross-sectional area
A_v	=	total transverse reinforcement area within spacing
b	=	width of column section
d	=	distance from extreme compression fiber to centroid of tension reinforcement
I	=	initial stiffness
E	=	cumulative energy dissipation
L	=	clear height of column
f'_c	=	compressive strength of concrete
f_y	=	compressive strength of longitudinal reinforcement
f_{yt}	=	compressive strength of transverse reinforcement

h	=	height of column section
M/V	=	ratio of moment to shear at critical section
P	=	applied axial load
s	=	spacing of transverse reinforcement
V_{max}	=	maximum of applied horizontal force
V_n	=	nominal shear strength of columns
V_u	=	theoretical flexural strength of columns
ρ_v	=	transverse reinforcement ratio

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APPENDIX A

Fig. A1-Generalized force-displacement relationship in ASCE/SEI 41-06⁽¹³⁾

Fig. A2- Comparison between experimental backbone curves and ASCE/SEI 41-06 Models⁽¹³⁾
(Note: 1 kN = 0.225 kip; 1 mm = 0.04in)

1 **Table A1-**Modify shear strength equation of ASCE/SEI 41-06 for columns with plain rebars

<i>Specimen</i>	<i>L</i> (mm)	<i>b</i> (mm)	<i>h</i> (mm)	<i>f_c</i> (Mpa)	<i>f_{yt}</i> (Mpa)	<i>d</i> (mm)	<i>A_v</i> (mm ²)	<i>s</i> (mm)	<i>ρ_v</i> (%)	<i>P</i> (KN)	<i>V_{test}</i>	<i>V_n (kN)</i> [ASCE]	<i>V_n (kN)</i> [Modified]	$\frac{V_{test}}{V_{ASCE}}$	$\frac{V_{test}}{V_{Mod}}$
Current Test															
SP1-1.7-0.2	1200	350	350	29.8	475	300	56.5	125	0.129	730.1	238.3	303.1	212.1	0.79	1.12
SP2-1.7-0.35	1200	350	350	29.2	475	300	56.5	125	0.129	1252	263.2	354.0	247.8	0.74	1.06
SP3-2.4-0.2	1700	350	350	30.6	475	300	56.5	125	0.129	749.7	193.3	235.9	165.1	0.82	1.17
SP4-2.4-0.35	1700	350	350	28.7	475	300	56.5	125	0.129	1230	198.0	266.4	186.5	0.74	1.06
SR1-1.7-0.35	1700	250	490	23.3	509	440	56.5	125	0.092	999.0	263.0	357.5	250.2	0.74	1.05
SR2-1.7-0.5	1700	250	490	22.5	509	440	56.5	125	0.092	1378	278.0	389.6	272.7	0.71	1.02
SW1-1.4-0.2	1700	200	610	21.7	509	560	56.5	125	0.074	529.5	247.8	382.3	267.6	0.65	0.93
SW2-1.4-0.35	1700	200	610	20.2	509	560	56.5	125	0.074	862.5	266.0	423.1	296.2	0.63	0.90
SW3-1.4-0.5	1700	200	610	20.5	509	560	56.5	125	0.074	1250	289.0	471.1	329.8	0.61	0.88
Arani et al ⁽⁹⁾															
WOS-M	1500	250	250	23.9	370	200	100.5	200	0.201	224.1	49.8	88.4	61.9	0.56	0.80
WOS-C	1500	250	250	22.9	370	200	100.5	200	0.201	214.7	62.1	87.0	60.9	0.71	1.02
SOS-M	1500	250	250	24	370	200	100.5	200	0.201	225.0	65.0	88.5	62.0	0.73	1.05
HOS-C	1500	250	250	24.8	370	200	100.5	200	0.201	232.5	65.0	89.6	62.7	0.73	1.04
Yalcin et al ⁽⁷⁾															
L0C0	3200	200	400	16	360	350	100.5	300	0.084	352.0	77.0	92.3	64.6	0.83	1.19
L50C0	3200	200	400	17	360	350	100.5	300	0.084	374.0	75.0	94.4	66.1	0.79	1.14
Faella et al ⁽²¹⁾															
M5-S	3400	300	300	26.4	346	250	100.5	200	0.167	332.6	51.2	85.9	60.2	0.60	0.85
C3-S	3400	300	300	25.7	346	250	100.5	200	0.167	323.8	52.7	85.2	59.6	0.62	0.88
C16-S	3400	300	300	27.5	346	250	100.5	200	0.167	990.0	81.5	106.7	74.7	0.76	1.09
C18-S	3400	300	300	13.5	346	250	100.5	200	0.167	486.0	42.3	82.1	57.4	0.52	0.74
													Ave	0.70	1.00
													COV	0.097	0.068

(Note: 1kN = 0.225 kip; 1mm = 0.04in; 1 MPa = 0.145 ksi)