Influence of external steel plates on the characteristics of existing RC columns under near field ground motion

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ABSTRACT
Near field ground motions are different from ordinary ground motions in that they often contain strong coherent dynamic long period pulses and permanent ground displacements. The dynamic motions are dominated by a large long period pulse of motion that occurs on the horizontal component perpendicular to the strike of the fault, caused by rupture directivity effects. Near fault recordings from recent earthquakes indicate that this pulse is a narrow band pulse. These characteristics make near field earthquakes unique compared to far field ground motions, which nearly all seismic design criteria are based on. Older buildings built before 1987 in Iran are known to demonstrate poor seismic performance in terms of ductility and energy dissipation capacity during severe seismic events. Various methods are developed for strengthening of reinforced concrete columns against shear. Strengthening of reinforced concrete columns using external bounding of steel plates was one of the popular research areas of recent years. This paper presents the effect of near field motions on reinforced concrete columns strengthen by external bonding of steel plates. Five reinforced concrete columns were analyzed under seismic loading. Three main types of steel members with different arrangements were bonded to the column. The purpose was to obtain ductile behavior for shear deficient reinforced concrete columns. The analysis result confirmed that all steel plate arrangements improved the strength and stiffness of the specimens significantly. The tension reinforcement of all strengthened reinforced concrete specimens was yielded. The failure modes and ductility of specimens were proved to differ according to the type of the steel member and arrangement along the column.
KEYWORDS: near field, directivity effects, reinforced concrete column, strengthening, steel plate.

1. INTRODUCTION

In the proximity of an active fault system, ground motions are significantly affected by the faulting mechanism, direction of rupture propagation relative to the site (e.g., forward directivity), as well as the possible static deformation of the ground surface associated with fling-step effects. These near field outcomes cause most of the seismic energy from the rupture to arrive in a single coherent long-period pulse of motion (note that backward directivity records typically do not exhibit pulse-type motions). Ground motions having such a distinct pulse-like character arise in general at the beginning of the seismogram, and their effects tend to increase the long-period portion of the acceleration response spectrum [1]. These types of ground motions may generate high demands that force the structures to dissipate this input energy with few large displacement excursions. Consequently, the risk of brittle failure for poorly detailed systems is considerably enhanced [2]. The detrimental effects of such phenomena have been recognized during many worldwide earthquakes, including the 1992 Erzincan, 1992 Landers, 1994 Northridge, 1995 Kobe, 1999 Kocaeli, Duzce, Chi-Chi and recently the Bam earthquakes.
While considerable advances have been made in the use of numerical methods to evaluate seismic performance of civil structures, recently there is a clear trend that more RC analytical are being conducted or planned worldwide to gain more knowledge on failure mechanism in view that the fundamental characteristics of structural collapse are not easily amenable to an experimental treatment at the present stage. Older buildings built before 1987 in Iran are known to demonstrate poor seismic performance in terms of ductility and energy dissipation capacity during severe seismic events. During the December 26 2003 Bam Iran earthquake, a large

number of older buildings sustained severe damage and many others suffered from complete failure. Observed damages during this earthquake have shown that these columns exhibited severe shortcomings with respect to their flexural and shear strength, as well as to their ductility capacity. Bam earthquake, however, was a reminder that central Iran can be subjected to strong near fault earthquakes [3]. Furthermore, the structural vulnerability of existing RC structures in highly populated urban areas of central Iran is sufficiently high that the associated seismic risk cannot be neglected. In central Iran, the majority of reinforced concrete structures incorporates columns with similar reinforcement details as those that suffered severe damage, or collapse, during recent near fault earthquakes.
These observations have triggered reevaluations of the seismic design of RC structures in regions of high seismic activity under near fault ground motion. These studies have led to the development of new seismic evaluation procedures and retrofit techniques for existing RC structure, as well as recommendations for new structures under near field earthquake. The seismic risk associated with RC structures is equal to the seismic hazard of the region, which is the probability of occurrence of an earthquake causing a certain level of ground shaking.

2. RESEARCH SIGNIFICANCE

This paper presents results of an analytical study performed on the strengthening of shear deficient columns by using external bonded steel plates. Nonlinear dynamic analysis, therefore, is conducted to investigate global low-ductility collapse of old RC columns due to poor detailing. Five models, one of which is a control specimen and the remaining which have deficient shear reinforcement are analyzed under near field earthquake. RC columns with deficient shear reinforcements are strengthened with different arrangements of external steel plates. The aim is to obtain ductile flexural failure for all strengthened specimens. The results of the analysis on these columns are compared with that for the control column. The effects of the type and arrangement of the steel plates that are used for strengthening on the behavior, strength, stiffness, failure mode and ductility of the models will be investigated.

3. OBSERVED FAILURE MODES OF R.C. COLUMNS DURING EARTHQUAKES

Observed damage to reinforced concrete columns during major near field earthquakes in California (1994 Northridge), Japan (1995 Kobe), Turkey (1999 Kocaeli and Duzce) and in Iran (Bam 2003) can be divided into two categories. The first category is related to the flexural failure of columns caused by inadequate flexural strength and (or) flexural ductility capacity. The second category is related to the shear failure of columns caused by inadequate shear capacity. The potential occurrence of these two failure modes depends on the height of the column, the geometry of the cross section, the longitudinal and transverse reinforcement distribution, and the presence of stiffening elements.

3.1. Flexural Failure and Lack of Flexural Ductility

Flexural failures in plastic hinge regions occur mainly in columns with continuous longitudinal reinforcement. Such failures occur because the concrete core is not sufficiently confined by the transverse reinforcement to allow the column to reach the inelastic displacement demand imposed by the near field ground motion. Failure of a plastic hinge is preceded by the occurrence of horizontal cracking, spalling of the concrete core in compression, failure of transverse reinforcement, and buckling of longitudinal reinforcement, as shown in Figure 1.
Flexural failures generally occur with large inelastic displacements and are not as catastrophic as shear failures. They usually occur in predictable plastic hinge regions, except when architectural features, such as level difference, cause a migration of plastic hinges. For columns incorporating lap-splices at the footing–column connection, flexural failure usually occurs through a sliding mechanism between the longitudinal reinforcement of the column and the dowel bars of the footing. Because of a lack of adequate clamping pressure across the fracture surfaces of the lap-splice region, sliding takes place before the ultimate flexural capacity of the cross

section can be reached. This sliding mechanism is activated at the onset of vertical microcracking in the concrete core. Sliding increases as vertical cracking amplifies and the integrity of the concrete cover deteriorates over the lap-splice region. The degradation of the flexural strength usually occurs for low displacement ductility demands and can even occur before yielding of the longitudinal bars in the column.

Fig. 1. Failure of a reinforced concrete column caused by inadequate flexural ductility

Fig. 2. Shear failure and migration of plastic hinge region

3.2. Shear Failure

A large number of shear failures of reinforced concrete columns have been observed following recent major near field earthquakes. Shear failures are brittle in nature and lead to a quick degradation of the lateral strength of a column. Short columns with conventional transverse reinforcement details are particularly vulnerable to shear failure, since, for a given lateral load; the available flexural strength is usually much larger than the available shear strength.
Shear failures can take various forms depending on the cross section of a column. A pure shear failure is associated with major diagonal cracking along the complete height of the column before any yielding of the reinforcement. A combined shear–flexure failure can also occur and can be accompanied by a migration of the plastic hinge region. A plastic hinge can migrate to a section where the ratio of the confining pressure to the applied bending moment is a minimum. A shear failure can also occur away from plastic hinge regions if the spacing of the transverse reinforcement is not uniform over the height of the column, as shown in Figure 2. Shear failure is more dangerous than flexural failure. For that reason RC columns must be designed to develop their full flexural capacity and assure a ductile flexural failure mode under extreme loading. However, many RC structures encounter shear problems for various reasons, such as mistakes in design calculations, improper detailing of the shear reinforcement, construction errors or poor construction practices, changing the function of a structure from a lower service load to a higher service load, and reduction of the shear reinforcement steel area due to corrosion in service environments.

4. SEISMIC RETROFIT WITH STELL JACKETS
4.1. Retrofit Strategies

If the seismic evaluation of an existing RC column has highlighted deficiencies towards flexural or shear resistance, a retrofit strategy must be considered. The known strengthening techniques of columns are as follows: CFRP, strengthening with externally applied clamps, jacketing with concrete layers and external bonding of steel plates. Typical retrofit strategies involve placing a jacket of a given material (steel, concrete, or composite) around the critical section of the column. Capacity design is used to evaluate the critical mode of failure and to determine the required characteristics of the jacket. To ensure a ductile behavior of the RC structure for the largest possible horizontal displacement demand, the formation of a plastic hinge at the base of the column must be favored as the weakest link mechanism of the structure. Therefore, the shear strength of any section of the column must resist the maximum shear force induced by this plastic hinge mechanism. For strengthening shear deficient columns, although numerous tests have been carried out, and shown that composite materials are an excellent option for use as external reinforcing, the steel plate bonding technique is becoming preferable for strengthening due to several advantages such as easy construction work, minimum change in the overall size of the structure after plate bonding and being an economical technique.

A jacket is typically composed of two steel sections that are welded vertically along the height of the column. A cement-based fill material (grout or concrete) is injected between the original cross section and the steel jacket to insure a composite behavior. A gap of about 50 mm is left between the base of the jacket and the top of the footing. This gap is introduced to allow the formation of a plastic hinge at the base of the column without increasing substantially its lateral stiffness and strength. Without this gap, the footing would be highly solicited because of the increased shear and flexural strength at the footing–column connection. Therefore, the gap prevents the migration of the plastic hinge region from the bottom of the column towards the footing.

Steel jackets improve the seismic behavior of RC columns by offering passive confinement of the original concrete cross section. A confinement pressure is mobilized when the lateral expansion of the concrete takes place. In the compressed zone, concrete tends to dilate. The confinement pressure is carried by hoop stress in the jacket. In the tensile zone, crack dilatation caused by a sliding mechanism (splitting action) along the lap-splice region is prevented mechanically in the same manner. In fact, a steel jacket can be thought of as a continuous assembly of transverse hoops that also provides an important contribution to the shear strength of the composite cross section.

4.2. Design Criteria for Steel Jackets

Design criteria for steel jackets depend on the primary performance requirement of the retrofit strategy that has been identified from a detailed evaluation procedure, namely, improvement of the flexural ductility, improvement of the integrity of the lap-splice region, or improvement of the shear strength.

4.2.1. Improvement of flexural ductility

The capacity at the base of a column is directly related to the confinement level of the concrete core in the plastic hinge region. A poor confinement does not allow the concrete to reach significant deformation in compression. In that case, only limited curvature deformation can be achieved without collapse. Steel jackets improve the compressive deformation capacity of the concrete core and, therefore, the flexural ductility capacity of the column. Furthermore, buckling of the longitudinal reinforcement is limited within the gap region and, therefore, the integrity of the concrete core can be maintained for large ductility levels.

4.2.2. Improvement of integrity of lap-splice region

When microcracking is initiated at the steel–concrete interface, sliding of lap bars can be prevented if a proper pressure is applied through passive confinement by the steel jacket. Experimental investigations [4] have shown that an effective coefficient of friction μ_f = 1.4 at the cracked interface is sufficient to prevent excessive dilatation of the cracks. To achieve this coefficient of friction and to prevent yielding of the steel, the strain in the jacket should not exceed 0.0015.

5. NONLINEAR DYNAMIC ANALYSIS
5.1. Specimens and Material Properties

The model was designed to represent a real 5-story commercial-resident complex, which is quite popular in the central part of Iran. The column design is aimed to reproduce genuine local engineering practice in Iran before 1987, in contrast to the new design code documents introduced after 2003. Plan and elevation view of mentioned RC structure are shown in figure 3. Also, the detail and cross section of a typical column used in first story is shown in Fig. 4.

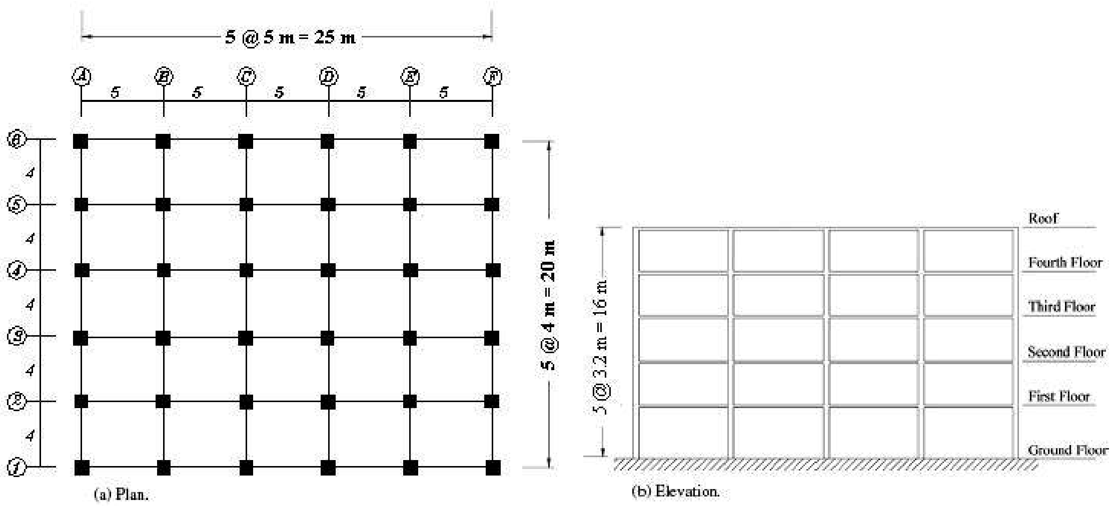


Fig. 3. Case study building.

The longitudinal reinforcement consists of eight 16 mm diameter steel deformed bars. The shear reinforcement consisted of 8 mm diameter closed stirrups, spaced at 200 mm center to center throughout the column except for Specimen of Column-1. The closed stirrup spacing for Beam-1 was 40 mm. The yield strengths of the longitudinal steel bars and stirrups were f_{sy} = 400 MPa and f_{sy} = 300 MPa respectively. The average compressive strength of the concrete was 25 MPa.

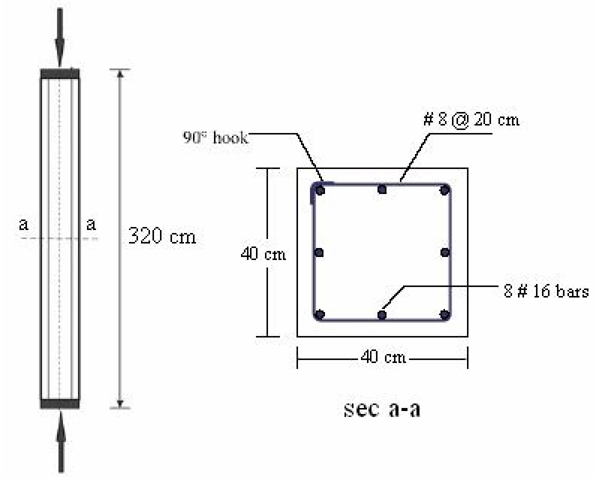


Fig. 4. Cross section of a typical column used in first story

5.2. Input Near Field Ground Motions

When an earthquake occurs, the velocity at which the fault ruptures is approximately the same as the velocity at which shear waves emulate [5]. The accumulated energy is concentrated into the form of a short duration, high amplitude pulse perpendicular to the fault because shear waves are of transverse wave type. The same effect is not observed in the rear of fault rupture direction. As a result the energy is spread over a long duration, and the earthquake record is similar to those of far field earthquakes. Recorded during the 2003 Bam Iran earthquake, the Bam station was approximately 8 km from the fault and considered as a near field earthquake. The fault normal component of the Bam ground motion Acceleration and velocity history is shown in Figure 5. As can be seen in the figure, the Bam ground motion showed a clear and distinguished forward directivity pulse. Therefore, Bam record data is used as input near field ground motions for dynamic analysis.

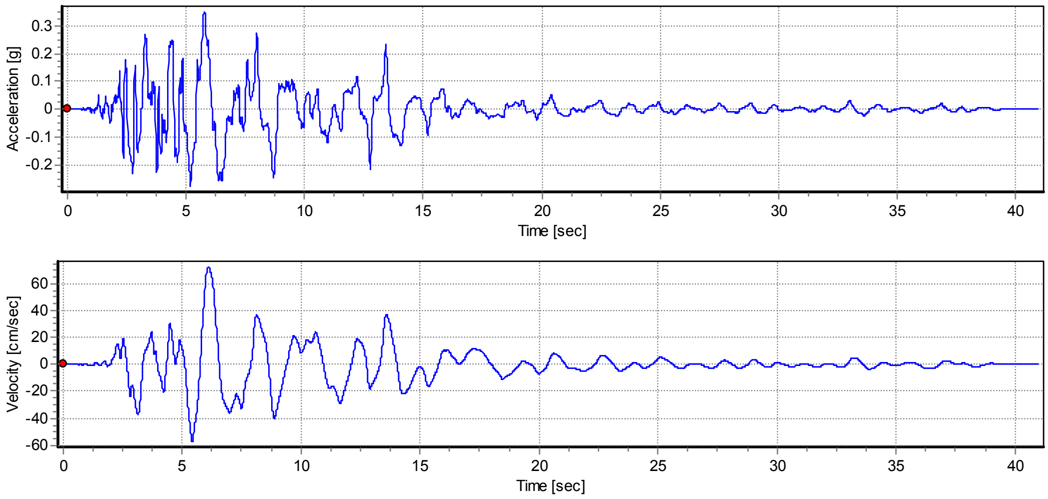


Fig. 5. Acceleration and velocity history of Bam (Iran) earthquake

6. ANALYTICAL RESULTS AND EVALUATION

6.1. Specimen Behavior and Failure Modes

Column-1 was the control specimen that was designed such that it had greater shear strength than flexural strength. Thus, ductile flexural failure was the dominant mode of failure. The other RC columns were designed to be deficient in shear capacity; thus, shear failure was their dominant mode of failure. The ratio of the shear deficient columns' stirrup reinforcement ratio to the control member's stirrup reinforcement ratio was 0.25. Shear deficient columns were strengthened by bonding steel straps and plates to all sides of the column. Steel straps or plates were designed such that they could increase the shear force up to the columns' ultimate flexural capacities without yielding. The steel strap and plate geometric dimensions are shown in Figure 6. The thickness of all steel plates was 4 mm.

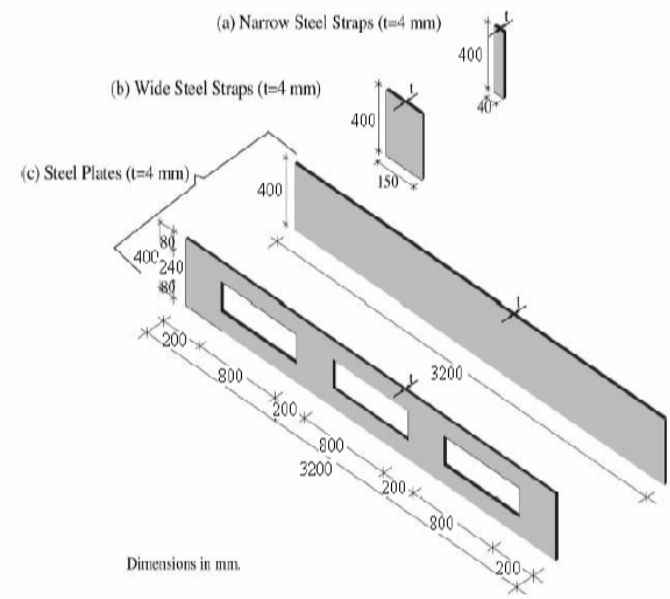


Fig. 6. Steel straps and plates used for strengthening

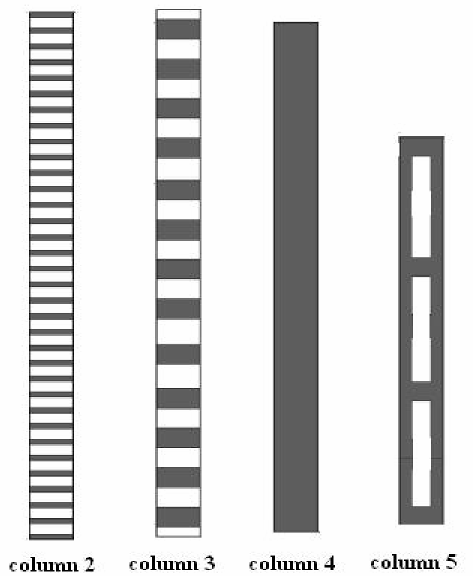


Fig. 7. Steel strap and plate arrangements of strengthened specimens

As can be seen from Fig. 6: (a) A narrow steel straps with dimensions 40 × 400 ×4 mm were applied; (b) A wide steel straps with dimensions 150 × 400×4 mm were applied and (c) Two types of steel plates with dimensions 3200×400× 4 mm and the same dimensional plates with openings were applied. The steel plates with openings were from 3200 × 400 × 4 mm plates by cutting three symmetrical openings with dimensions 800 × 240 mm. Steel strap and plate arrangements of strengthened specimens are given in Figure 7.

Analysis results are summarized in Table 1. In all specimens, the first crack always appeared as a flexural crack in the maximum bending moment region of the column. In general, first flexural cracks developed at 17% of the ultimate strengths of the specimens. Shear cracks developed at load levels between 40% and 50% of the ultimate load. Control specimen Column-1 showed ductile flexural behavior as a result of longitudinal tension reinforcement yielding. After yielding, Column-1 developed large displacements, and reached an ultimate load value that was 11% greater than the yield load. Column-1 failed because of crushing of the concrete in the extreme compression fiber. Flexural cracks of specimens strengthened with steel straps propagated as oblique cracks between the steel straps with increasing load. Shear cracks reached to the steel straps at a load level of 55% of the ultimate strength of the specimens. At these loads levels propagation of the shear cracks was restricted by the steel straps and cracks did not pass under the steel straps. Some shear cracks followed the edge of the steel straps and propagated to the column's top. The longitudinal tension reinforcement of all specimens yielded. After yielding, the arrangement of the steel members determined the behavior of the specimens.

Table 1. Results of nonlinear dynamic analysis

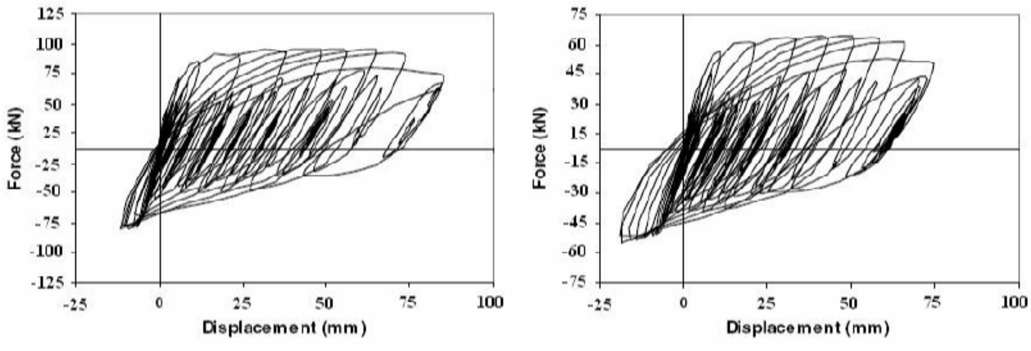
Model	**Cracking load (kN)**		**Yield load (kN)**	**Ultimate load (kN)**	**Yield disp. (mm)**	**Ultimate disp. (mm)**	**Stiffness at yield (kN/mm)**	**Ductility ratio**	**Failure mode at ultimate**
	Flexure	shear							
Column1	13.4	36	81	91	23.5	85	3.45	3.61	flexure
Column2	14	36.6	79.2	81	25.2	60.4	3.14	2	shear
Column3	13.6	35.7	79	80	22.8	40.7	3.47	1.79	shear
Column4	12.8	38.2	81.3	88.6	22	93.7	3.69	4.26	flexure
Column5	13.5	37.8	81	84.7	23.5	67.9	3.44	2.89	shear

Column 4 strengthened by bonding a steel plate along the whole height showed ductile flexural behavior. Shear cracks of Column 5, strengthened with steel plate with openings, propagated to the column's top and the specimen collapsed in shear. In general, there were fewer shear cracks for strengthened specimens than for the control specimen, and the strengthened specimens collapsed due to propagation of one main shear crack.

Load-displacement relationships for the specimens of Column-1 and Column-2 are shown in Figsure 8. All specimens showed the same stiffness at yielding. Strengthened specimens all showed approximately the same displacement and load values at yield. The calculated secant stiffnesses were close to each other for all specimens. The average secant stiffness of the specimens was 3.44 kN/mm at yield. As can be seen from Fig. 9, the spacing of the steel straps was a significant parameter affecting the ultimate load carrying capacity, displacement capacity and failure mode. The specimens had 41% and 61% less displacement capacity than the control specimen, respectively. Of the specimens strengthened with narrow steel straps, only column-4 showed ductile flexural behavior. Column-4 behaved similarly to the control member, when the ultimate load carrying capacity, failure mode and displacement capacity are taken into account. The ultimate load and displacement capacities for column-3 that was strengthened with wide steel straps were significantly lower than the corresponding quantities for the control member. Specimens that were strengthened with steel plates along the whole height showed very similar load-displacement behavior to the control specimen. Column-4 had slightly more ductility than the control specimen. Column-5 had the least ultimate ductility of all specimens strengthened with steel plates along the whole height. Column-5 had 20% less ultimate displacement than the control specimen.

6.2. Ductility

Displacement ductility ratios were calculated as the displacement at the maximum load divided by that at the yield load. Column-4 that was strengthened with steel plates along the whole height had more ductility than Column-1. The ductility ratio of Column-5 was 32% less than Column-4's ductility ratio. Column-2 that was strengthened with narrow steel straps showed 44% less ductility than Column-1. Column-3 that was strengthened with wide steel straps had approximately 50% less ductility than Column-1. The behavior of the specimens that were strengthened with steel straps showed that the spacing of the steel straps was closely related to the ductility ratio.



(a) (b)

Fig. 8. Load–displacement curves of specimens strengthened with steel plates (a) Column-1 and (b) Column-2.

6.3. Strength

Ratios of the ultimate strength of strengthened specimens to the control member's ultimate strength were between 0.88 and 0.98. The largest ultimate strengths were for the specimens strengthened with steel plates along the whole height. The ratios of the ultimate strengths of Column-4 and Column-5 to the ultimate strength of the control specimen were 0.98 and 0.94, respectively. The largest increase in strength at the ultimate stage was for Column-3 for the specimens strengthened with steel straps. The ratio of the ultimate strength of Column-3 to the control member's ultimate strength was 0.93.

7. CONCLUSION

In this study, strengthening of RC columns against shear by using external steel plates with different arrangements under the near field ground motion was investigated. General results obtained from the nonlinear dynamic analysis are as follows:

All steel member types bonded externally had improved column strength, stiffness and ductility. Strengthened specimens showed similar behavior to a control specimen up to flexural yield. Specimens reached the flexural yield strength with the same stiffness for nearly the same time history loading and displacement. The type of steel member and its arrangement on the column were among the effective parameters directing the ductility behavior and determining the failure mode. The displacement ductility ratio was increased when the spacing of the steel straps was decreased. Increase in the bonding area reduced the propagation of shear cracks significantly. Specimens that were strengthened with steel plates showed strength and ductility close to those of the control member. Steel plates prevented propagation of shear cracks, clearly. Instead of using one large steel plate along the whole of the height, segmenting the steel plates and then bonding them adjacent to each other showed successful results.

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