

Post-tensioned bracing for seismic retrofit of RC frames

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ABSTRACT: The performance of post-tensioned bracing as a retrofit scheme for non-ductile moment frames of reinforced concrete is evaluated by examining the inelastic dynamic response of two buildings. The buildings represent typical low and medium rise construction of the 1960's in the U.S. and are analyzed for three major earthquake records measured on firm and soft soils. The results indicate the post-tensioned bracing system can be used for low rise buildings located on both firm and soft soils. However, for medium rise buildings the technique appears to be more appropriate for structures located on firm soils than on soft soils.

1 INTRODUCTION

Many buildings designed according to past standards are often found to have inadequate lateral strength and poor energy dissipation capacity to withstand major earthquakes. In the United States, non-ductile reinforced concrete (R/C) frames constructed twenty five years ago or more have been identified as one of the types of structures which appear to represent the greatest hazard to society. As a result, significant research effort has been devoted to developing a variety of techniques for the seismic retrofit of R/C frames during the last decade. The most common techniques for seismic retrofit include reinforced concrete and steel jacketing, addition of infill walls and the addition of steel bracing systems.

The present study focusses on a retrofit technique that involves addition of post-tensioned steel rods as bracing for reinforced concrete frames. The technique uses alloy steel rods which are initially prestressed to increase the initial lateral stiffness of the building. Due to the high strength of steel rods, the lateral strength of an existing frame can be effectively increased with relatively small amounts of material. A previous study (Miranda et. al 1990) on school buildings located in the Pacific Coast of Mexico, suggested that the technique was particularly beneficial for low-rise buildings on soft soil sites.

The main purpose of this study is to evaluate the performance of post-tensioned bracing systems to improve the seismic response of low and medium rise R/C frames. The behaviour of the post-tensioned bracing system is evaluated by examining analyt-

ically the inelastic dynamic response of two R/C frames. The buildings are analyzed for three ground motions representative of major earthquakes on firm and soft soils.

2 BUILDINGS SELECTED FOR STUDY

The buildings were selected to represent typical low and medium rise construction of the 1960's in the United States. Low-rise construction was represented by a three-storey building designed according to the 1964 Uniform Building Code, UBC, (1964) for lateral forces and the 1963 American Concrete Institute, ACI, (1963) for reinforced concrete members. A typical plan view and elevation in the longitudinal direction of the building are shown in Fig. 1. Because of the low level of lateral forces specified by the 1964 UBC, proportioning and design of members was governed by gravity loads. The weak links in the structure are lightly-confined, short lap splices at the base of columns (24 bar diameters) and short anchorage lengths of bottom beam reinforcement (150 mm) in all stories. Such reinforcement details are typical of buildings not designed to resist seismic forces, and cannot develop the yielding moment capacity of the sections. Premature failure of splices at the base of columns and pull-out of bottom beam reinforcement are likely to occur during the early stages of severe ground motion. This will limit the lateral strength and ductility of the structure.

A twelve-storey building was selected to represent medium rise construction. Typical dimensions in

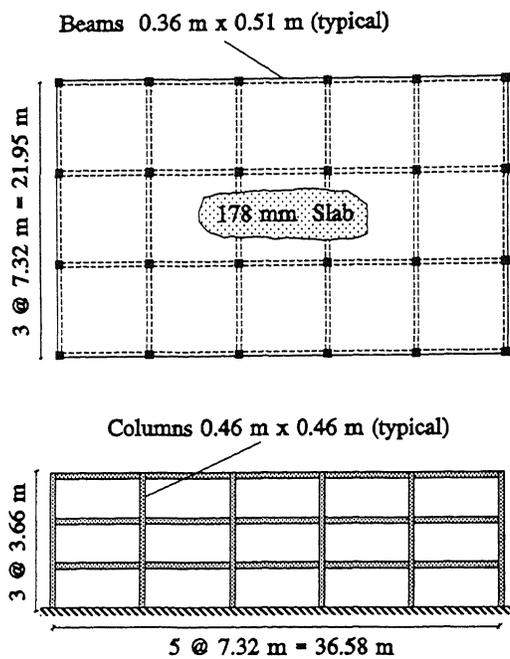


Figure 1 Plan view and elevation of the three-storey building.

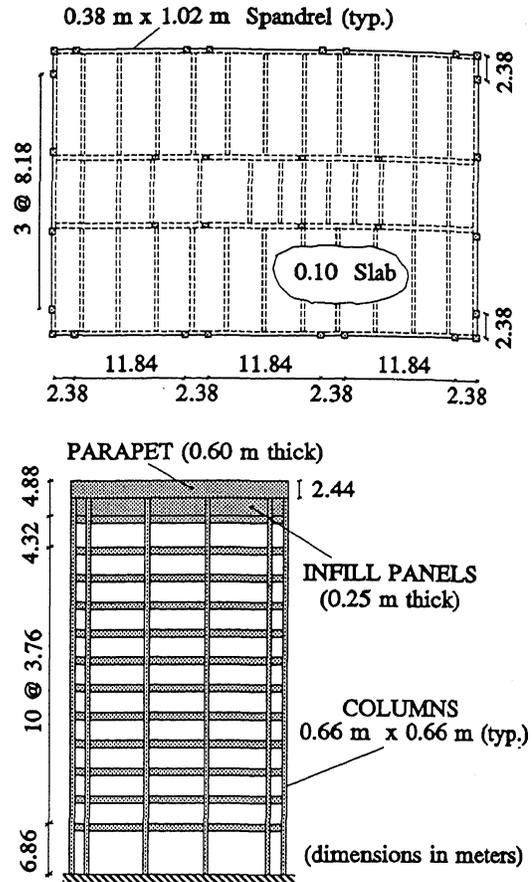


Figure 2 Plan view and elevation of the twelve-storey building.

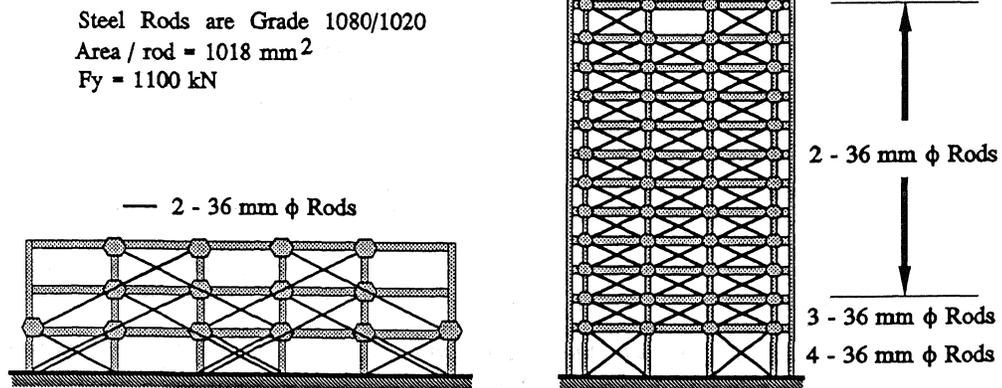


Figure 3 Bracing schemes for the three and twelve storey buildings.

plan and elevation in the short direction of the building are shown in Fig. 2. The lateral load resisting system of the building is provided primarily by the perimeter frames. In these perimeter frames a double column configuration was used in design, probably to increase the lateral stiffness and the redundancy in the structure. The building features a flexible "soft" first storey and a massive parapet at the roof. Overall, detailing of the flexural reinforcement in hinging regions of columns and beams is adequate to develop flexural yielding of the members. Longitudinal reinforcement in columns is typically spliced in the mid-length region with a 40 bar diameter lap, and in beams, bottom reinforcement is anchored 30 bar diameters into the joint. Hoops and crossies in the hinging regions in columns are closely spaced at 100 mm over a length that meets current provisions for detailing in seismic zones. In contrast, shear reinforcement in the central portion of columns is widely spaced at 450 mm (more than half of the depth of the member) which is insufficient to prevent column shear failure prior to the development of plastic hinges at the ends of the members. Spacing of transverse reinforcement in beams outside the hinging region is also 450 mm and is insufficient to develop the flexural capacity of the members. During strong ground motion, shear failure of columns and/or beams is anticipated, thus jeopardizing overall integrity and stability of the building.

3 RETROFIT SCHEMES

Bracing schemes adopted for the three and twelve storey buildings are shown in Fig. 3. To expedite the rehabilitation process and minimize disturbance to the occupants, the bracing system was added only to perimeter frames. Braces consisted of steel rods with a yield strength of 1080 MPa and were initially prestressed to 75% of the yield strength of the braces. The high level of prestress was selected to allow braces to yield in tension at relatively small drifts. Thus, the bracing system will begin to dissipate energy during the early stages of an extreme event. Such design criterion has shown to be more effective than the use of lower levels of initial brace prestress (Pincheira 1992). Also, a high level of initial brace prestress will reduce the likelihood of shortening of the braces to the point where they become slack. If braces remain in tension, the reductions in stiffness are minimized.

Initial brace prestressing will induce additional forces in the structure that modify the internal force distribution due to gravity loads alone. In the three-storey structure, maximum bending moments after prestressing reached 40% of the splice capacity of columns, compared to a maximum value of 25%

under the action of gravity loads alone. In beams, the effects of initial brace prestressing are minimal. Maximum bending moments in the twelve-storey building after brace prestressing reached at most 25% of the flexural yield strength of the members. The magnitude of the moments induced upon brace prestressing does not pose a threat to the integrity of the structure, but requires special consideration to satisfy serviceability and/or durability requirements.

4 LATERAL LOAD ANALYSES

Static inelastic frame analyses using a uniform lateral load distribution were conducted to evaluate the stiffness and strength of the existing and braced buildings. Idealization of the behaviour of splices in columns and pull-out of bottom beam reinforcement from columns followed a strength degrading model. Shear failure of R/C members was characterized by a sudden loss of lateral stiffness and strength of the members. In Fig. 4, the base shear coefficient and drift at the centroid of inertia forces for the existing and retrofitted three-storey structure is presented. The increase in lateral stiffness and strength of the three and twelve storey buildings with bracing is substantial. Ultimate strength of the original structure is governed by failure of splices in all first storey columns at a drift of about 0.75%. After bracing of the building, the lateral strength is increased by a factor of about 9. Due to the high level of initial brace prestressing, yielding of the braces begins at drift of about 0.35%, prior to failure of splices (see Fig. 4). As noted earlier, such a design criterion was imposed so that the bracing system begins to dissipate energy at small lateral drifts. Ultimate strength of the braced structure is reached when braces that shorten begin to sag at drift of about 1%.

The existing twelve storey building showed almost linear-elastic behaviour up to a drift of about 0.5% when sudden shear failure of second storey columns began, as shown in Fig. 5. The brittle nature of such a failure triggered the progressive failure of the first and third storey columns which led to the collapse of the building. After bracing of the structure, lateral strength is increased by a factor of almost 4. As for the three-storey building, yielding of the braces begins at a drift of about 0.35%, prior to the onset of column shear failure. Similar to the behaviour of the original frame, the mode of failure of the braced building is due to the shear failure of columns in the lower stories.

5 DYNAMIC RESPONSE

The dynamic response of the buildings was evaluated

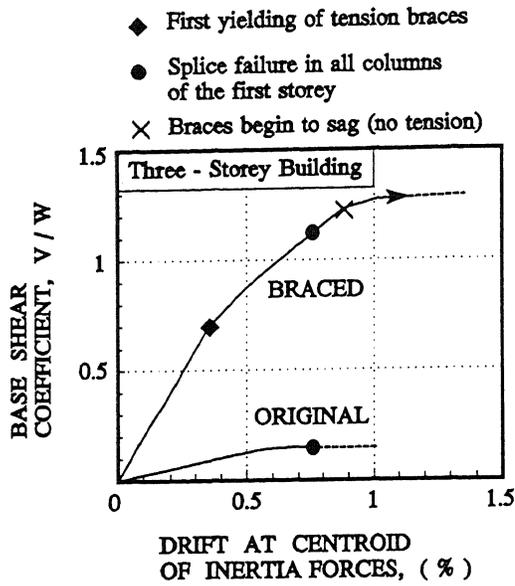


Figure 4 Base shear coefficient and drift relationships for the three-storey buildings.

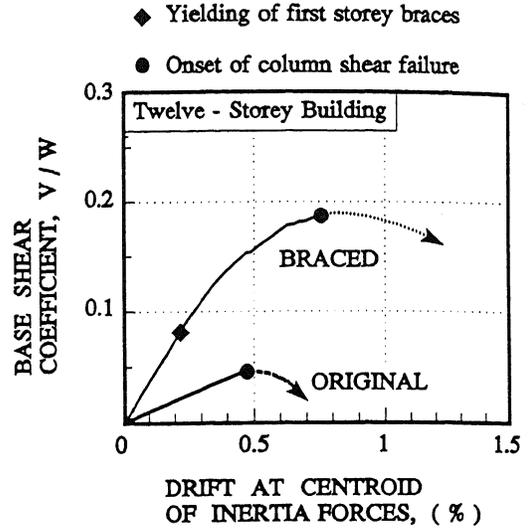


Figure 5 Base shear coefficient and drift relationships for the twelve-storey buildings.

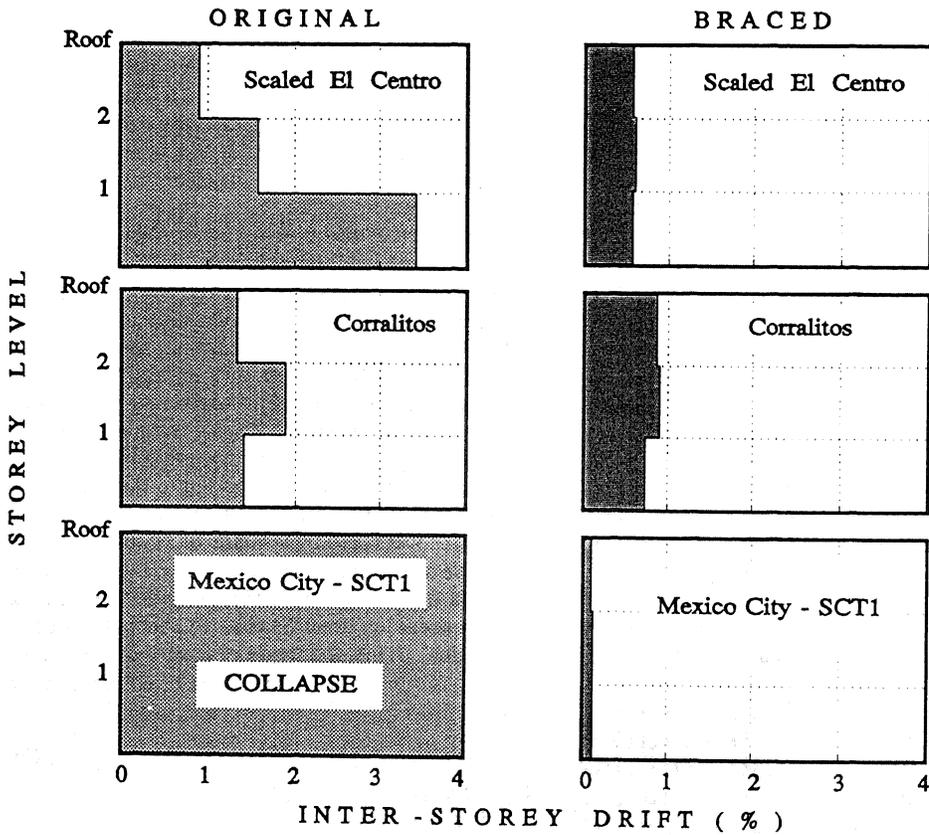


Figure 6 Maximum inter-storey drifts for the three-storey building.

for three earthquake records. The 1940 El Centro record (N00E component), scaled to a peak ground acceleration of 0.5 g, and the 1989 Corralitos record (N00E component) with a peak ground acceleration of 0.63 g, were used to represent major earthquakes on firm soils. The 1985 Mexico City - SCT1 record (N90E component) was used to represent an extreme event on soft soils. Viscous damping was assumed as 2% of critical in all cases. Analyses of the buildings on soft soils included the effects of soil-structure interaction using a simplified procedure.

The fundamental period of the three-storey building based on cracked sections was computed as 1.1 secs. For the twelve storey building the fundamental period was estimated as 3.5 secs. Such a long period is attributed to the unusual first storey height (almost twice that of the upper stories) and to the use of cracked sections in the calculations. The fundamental periods of the braced structures were 0.4 and 1.9 secs. for the three and twelve storey buildings respectively.

In Fig. 6, the maximum inter-storey drifts obtained for the original and braced three-storey buildings for all three earthquake records are compared. Inter-storey drifts for the scaled El Centro record exceeded a value of 3% in the first storey. For the Corralitos record, maximum inter-storey drifts occurred in the second floor and reached about 2%. In both cases, splice failure was observed in all first and second storey columns, with large inelastic rotations after peak resistance ($\mu_p \approx 10$). Such high inelastic rotations led to almost total loss of the flexural capacity of column sections at the base. Pull-out of bottom reinforcement anchored in the column was observed in almost every beam, which led to a complete loss of the flexural capacity for positive moment. The large inter-storey drifts and the failure of several elements in the structure under events on firm soil pose a serious threat to the overall integrity and stability of the building. For the Mexican earthquake, computed inter-storey drifts exceeded 4% in all stories, which would lead to the collapse of the building.

For the braced structure, maximum inter-storey drifts are below 1% in all cases (see Fig. 6). For the scaled El Centro record, splice failure was prevented in all columns. For the Corralitos record, failure of splices was still observed in a few columns of interior frames. In these columns, the residual flexural strength was estimated at 80% of the peak moment capacity and was not considered to jeopardize the integrity of the building. Pull-out of bottom beam reinforcement was not prevented in either of the two records on firm soil. Braces reached yielding and some of them became slack during the Corralitos record (imposes the largest demands on the braced structure). Despite yielding of the braces, the pre-

stress force was never lost completely and all braces became taut upon unloading of the frame. For the Mexican earthquake, the adequate performance of existing frame members and steel braces is apparent from the low values obtained for maximum inter-storey drifts. Column splice failure was prevented in all columns and braces remained elastic during the entire response to the earthquake. Pull-out of bottom beam reinforcement was observed in only a few beams and with only minor reductions in flexural capacity. The low inter-storey drift values and the satisfactory performance of the existing R/C members and that of the braces suggest that a smaller brace size could have been used for the building located on soft soils.

In Fig. 7, maximum inter-storey drifts for the original and braced twelve storey buildings are compared. The results indicated that the original structure would probably collapse during the three records considered for study. Collapse of the building is attributed to the sudden and progressive shear failure of all second storey columns which essentially created an unbraced storey at that level. Thus, lateral displacements in the second storey increased without restraint until instability of the upper structure resulted in collapse.

In the braced structure, maximum inter-storey drifts for the scaled El Centro and the Corralitos records remained all under 1%, as shown in Fig. 7. For the scaled El Centro record, column shear failure was still observed in a few columns in the second, third and fourth storey levels. Shear forces in the rest of the columns showed that they were close to reaching their estimated shear capacity. First and second storey braces reached yielding and became slack during the response to the scaled El Centro record, but never lost prestress. The residual prestress force in the most stressed brace was 15% of the brace yield strength. The performance of the braces for the Corralitos record was better. Due to the high shear forces obtained in columns of the braced structure, it is recommended that columns be encased with jackets (steel or R/C) to improve their deformation capacity and to prevent a generalized column shear failure in similar or larger events.

For the Mexican earthquake, the bracing system did not prevent column shear failure and was unable to control lateral drifts. As noted earlier, the period of vibration for the braced structure was estimated at 1.9 secs which corresponds approximately to maximum peak response of the Mexico City - SCT1 record at 2 secs. Displacement ductility demands in that period range are extremely high and it is unlikely that a retrofitted structure would be able to satisfy such high demands, irrespective of the retrofit scheme. The retrofit strategy for such cases must involve shortening the period of the building to a

point where displacement ductility demands are reduced.

6 CONCLUSIONS

The performance of a post-tensioned bracing system as a retrofit scheme for two non-ductile R/C frame buildings was evaluated for three earthquake records representative of major events on firm and soft soil sites. For the medium rise building on the soft soil of Mexico City, the bracing system did not perform well because the period of the braced building coincided with that corresponding to peak response of the Mexican earthquake. The performance of the bracing system for the medium rise building under events on firm soils was satisfactory, even though a few columns failed in shear. For the low rise building, the post-tensioned bracing system performed satisfactorily on firm and soil sites, and was particularly effective under the event on soft soil.

ACKNOWLEDGEMENTS

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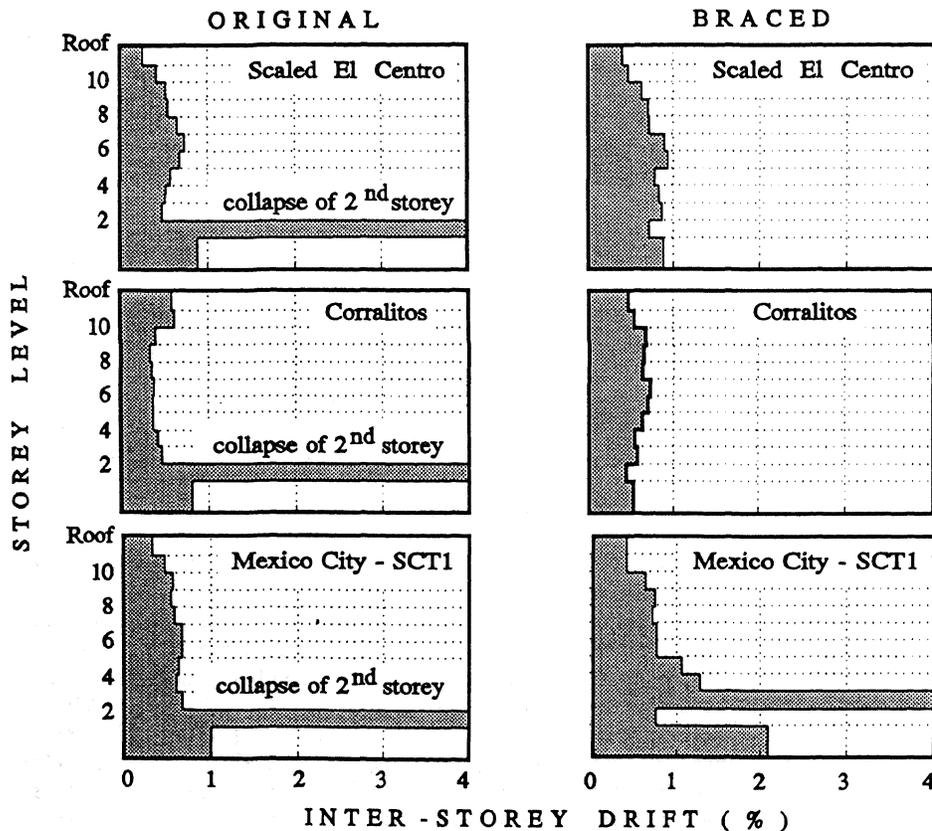


Figure 7 Maximum inter-storey drifts for the twelve-storey building.