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# REHABILITATION OF INFILLED NON-DUCTILE FRAMES WITH POST-TENSIONED STEEL BRACES

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## ABSTRACT

In recent years, it has been shown that the seismic performance of existing buildings can be enhanced considerably by bracing them with post-tensioned rods or cables. The use of this upgrading technique yields several advantages, such as architectural versatility, low cost, fast and clean construction, and does not add any significant reactive mass to the existing facility. This paper has the following objectives: to investigate the use of post-tensioned steel braces for the seismic rehabilitation of non-ductile frame buildings with unreinforced masonry infills; to discuss some of the issues that need to be considered for the design of the rehabilitated building; to assess the advantages of the use of this technique by studying the seismic performance of an infilled non-ductile frame building before and after it has been upgraded with post-tensioned braces; and finally, to offer conclusions and research recommendations.

## KEYWORDS

Unreinforced masonry, seismic performance, displacement control, interstory drift, seismic rehabilitation

## INTRODUCTION

During the rehabilitation of an existing building, the design options available to the designer are limited by the existence of a building that usually has some structural deficiencies. To make possible the successful rehabilitation of an existing building, the designer needs to identify its structural weaknesses through the understanding of its local and global mechanic characteristics. In the case of non-ductile frame buildings infilled with unreinforced masonry (URM) walls, this understanding should not only include the characteristics of the frame members, but also those of the infills. Given the poor performance of URM buildings and elements during past and recent earthquakes, the notion that this material should not be used to resist lateral loads is widely extended. Nevertheless, recent research has challenged this notion while demonstrating that as with any other structural material, the use of URM to resist seismic loads has advantages and disadvantages, depending on how the URM is used.

The efficient rehabilitation of an existing building is possible when all its existing structural materials and elements can be used to help resist the seismic loads once the building is rehabilitated. This may be achieved in infilled frames if the large natural sources of strength, stiffness and viscous and hysteretic energy dissipation provided by the infills are taken advantage of. That is, rather than eliminating the infills' contribution to resist the seismic loads, it is necessary to consider that the presence of URM infills may improve the overall seismic performance of a frame building. Under these circumstances, the main challenge confronted by the designer can be summarized with the following question: What changes can and should be introduced to the mechanic characteristics of the building, through the use of post-tensioned (PT) steel braces, in such a way that the infills are put to work according to their strengths rather than their weaknesses?

## SEISMIC PERFORMANCE OF URM ELEMENTS

The overall earthquake-resistant capacity of URM elements is considerably higher than was previously thought. URM walls and infills have a considerably larger strength than that at first cracking, a significant inelastic deformation capacity and if their in-plane deformation is limited within certain values, a fairly stable hysteretic behavior. Even after several loading cycles, URM elements are able to carry a large percentage of their peak strength for relatively large drift. Thus, it is not surprising that URM infills that are properly introduced into frame buildings enhance considerably the ultimate strength and stiffness of these buildings, and even their energy dissipation capacity (Klingner and Bertero, 1976; Schuller *et al.*, 1994).

Depending on the loading condition, the relative strength and stiffness of the frame and infills, the bond between them, and the mechanic characteristics of the masonry, a number of failure mechanisms are possible in an infilled frame. The designer should be aware of these mechanisms, and accordingly, avoid undesirable modes of failure. Other issue that deserves consideration is the multi-directional nature of ground motion. Drift and acceleration are the most relevant in-plane and out-of-plane seismic demands, respectively, in URM infills. It has been observed that in-plane damage reduces the out-of-plane strength of an infill, especially for those having large slenderness ratios (Sakamoto, 1978; Angel and Abrams, 1994). Usually, out-of-plane failure should not be a design concern because URM infills possess high out-of-plane strength (Angel and Abrams, 1994). Nevertheless, some experimental results and the out-of-plane failure of URM infills during past earthquakes suggest that large in-plane demands in combination with significant out-of-plane demands may result in out-of-plane failure (Teran *et al.*, 1995).

## BASIC BEHAVIOR OF A PT STEEL BRACE

Figure 1a shows the deformation vs. force curve of an axially loaded cable (or rod). As shown, the cable buckles elastically when subjected to compressive strains; while it can develop its yielding strength under tension. Figure 1b illustrates the behavior of the same cable subjected to cyclic loading. As shown, all inelastic tensile elongation accumulates every time the cable yields in tension. Figure 2a shows a counterpart of Fig.1a for a prestressed cable. As shown, both figures are similar, except that there is an initial state of stress and strain in the prestressed cable, which is accounted for in Fig.2a by shifting the origin of the force vs. displacement axes. Because of this shift, the cable can now "resist" compressive forces (a reduction of its initial tension can be interpreted as the cable developing a compressive force). Note that a reduction in the initial prestress of the cable implies a reduction in its capacity to "resist" compression, as can be concluded by following the path OABC in Fig.2b (Teran *et al.*, 1995). Figures 1 and 2 provide help in understanding the consequences that excessive yielding or buckling of a PT brace can have in its seismic performance: first, yielding (loss of prestress) reduces its ability to "resist" compressive forces; second, yielding (axial elongation) results in the reduction of its lateral stiffness; and third, net compressive strains in the brace (buckling) can lead to sudden and undesirable changes in the strength and deformation demands in the existing frame members. The axial strength and the amount of prestress provided to the braces should be designed to prevent their buckling and/or excessive yielding.This is only possible if a reasonable estimate of the maximum drift demand in the braces is available.

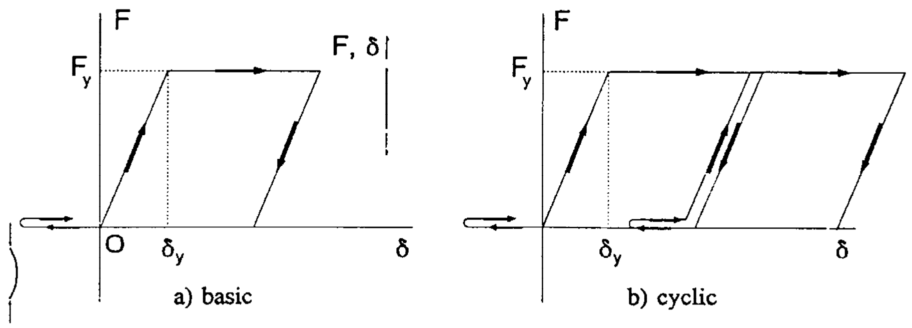



Fig. 1 Axial displacement vs. axial load behavior of rod or cable

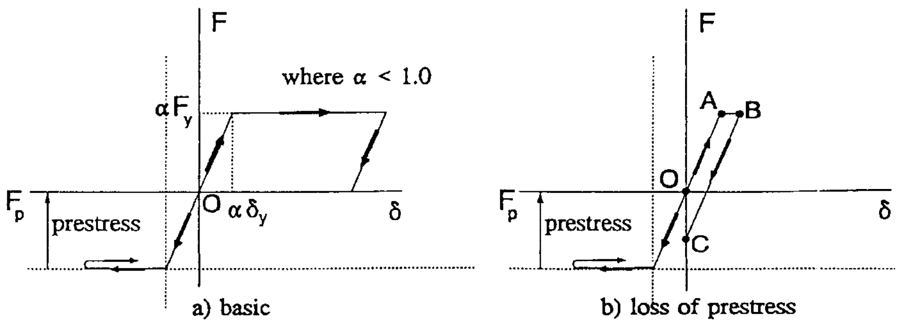



Fig.2 Axial displacement vs. axial load behavior of prestressed rod or cable

The use of high levels of prestress, so that the PT braces may moderately yield in tension at relatively small drifts, has been recently discussed (Pincheira and Jirsa, 1992). Although this is an attractive option, it should be considered carefully, because braces with high levels of prestress are likely to yield, and thus of partially loosing their high level of prestress during the small and moderate seismic events that usually occur prior to the safety level ground motion.

## USE OF PT BRACES FOR REHABILITATION OF INFILLED NON-DUCTILE FRAMES

The use of PT steel braces in seismic rehabilitation is a relatively new technique that can be applied efficiently to rehabilitate existing low and middle-rise frame buildings by providing significant increases in the lateral stiffness and strength of these buildings (Rioboo, 1989; Pincheira and Jirsa, 1992).

Previous to the rehabilitation of an existing building, it is desirable to carefully define the design goals of this rehabilitation. One way of defining the design goals consists of first defining qualitatively what constitutes an acceptable behavior (performance criteria) of the upgraded building, an then to quantify this qualitative definition by setting limits on the global and local seismic demands of the upgraded building. For instance, damage in frame members and in-plane damage in URM infills can be controlled by limiting their deformation and energy dissipation demands, while out-of-plane damage control in the infills can be achieved by limiting their in-plane damage and out-of-plane acceleration. The design of an adequate PT bracing system can be based on different performance criteria. Once these criteria have been established and quantified, different philosophies of design can be used to satisfy them. Essential to the good performance of the upgraded infilled non-ductile frame is to keep the frame members from developing non-ductile failures, preventing the collapse of the URM infills and preventing the PT braces from yielding in excess or buckling. To achieve this performance criteria, the following philosophy of design is suggested: design the PT braces to remain elastic and to control the story drift in the building to avoid the failure or collapse of the non-ductile frame members and the URM infills. Controlling the drift demand on a building through its rigidization while keeping the non-ductile frame members and PT braces essentially elastic is very likely to result in large increases in the base shear and story acceleration demands in the upgraded building. Nevertheless, provided the deformation demands on the URM infills are controlled within certain limits, they can supply a large and fairly stable source of viscous and hysteretic energy dissipation that will reduce these increases.

It should be emphasized that the good performance of the frame members, infills and PT braces in the upgraded building can only be achieved through drift control. This implies that the quantification of the performance criteria can be attempted by setting limits to the maximum drift or interstory drift index, IDI, demand in the building. An attractive aspect of the use of PT braces in rehabilitation is the wide range of lateral stiffness that can be considered during their design. Once the stiffness of the existing structure is evaluated, a bracing system with adequate stiffness can be easily developed. In the particular case described here, the braces should allow the structure to deform enough so as to allow the infills to contribute to resist the seismic loads, while controlling this deformation so as to let the infills provide this contribution in a stable manner and to keep the non-ductile frame members from failing. Although PT braces are usually designed to remain elastic, they can be fabricated from different types of steel, in such a way that they can be designed for a wide range of elastic deformation.

Due to the prestress in the PT braces, an initial state of stress is induced into the existing elements. Because of the large plan area of the infills, the presence of the braces induces a moderate state of compression that is likely to enhance the behavior of the infills. If so, it is desirable to brace the infilled frames of the building. Some advantages of using PT braces are: usually it is possible to distribute and design them in such a way as to avoid modifying the existing foundation; they add small reactive mass; there is no concern for inelastic buckling of the braces; clean and fast construction process; multiple architectonic possibilities; and they provide an efficient and economic solution. To achieve an adequate rehabilitation of an infilled frame using PT braces, it is necessary to check several aspects of the global and local behavior of the upgraded building: the change in behavior and failure mode of the existing frame members; the effects of the change in the dynamic characteristics of the building; the connection of the braces to the existing structure; and the use of the bracing system to correct significant strength and stiffness irregularities in plan and height (Teran *et al.*, 1995).

## EXAMPLE OF REHABILITATION OF AN INFILLED NON-DUCTILE FRAME BUILDING

A six-story reinforced concrete (RC) infilled non-ductile frame building (CSMIP Station No. 23544) was selected to illustrate the use of PT braces. This building has insufficient lateral strength and stiffness, as well as large mass, stiffness and strength irregularities in plan and height. CSMIP Station No. 23544 was constructed in 1923, and has a penthouse, a mezzanine and a basement level. The floor system consists of a 3" thick one-way slab system. Some of the frames of the building, including the four perimetral frames, are infilled with URM. Figure 3 shows plan views of the building while Fig.4 shows the elevation view of 5 frames parallel to the E-W direction. Due to the inappropriate distribution of the infills in plan and the presence of a mezzanine and a penthouse, large mass and stiffness eccentricities in plan exist in the E-W and N-S directions; while the irregular distribution of infills through height produce the existence of a flexible and weak first story (soft story). The sizes of the beams are fairly constant over height. Columns are square and their size decrease considerably in higher stories. Transverse reinforcement in the columns is provided by closely spaced spirals. The exact detailing of the longitudinal steel in beams and columns is not known, but the fact that this building was designed for gravitational loads in 1923 strongly suggests that the frame members may not reach their ultimate or even their yield flexural strength.

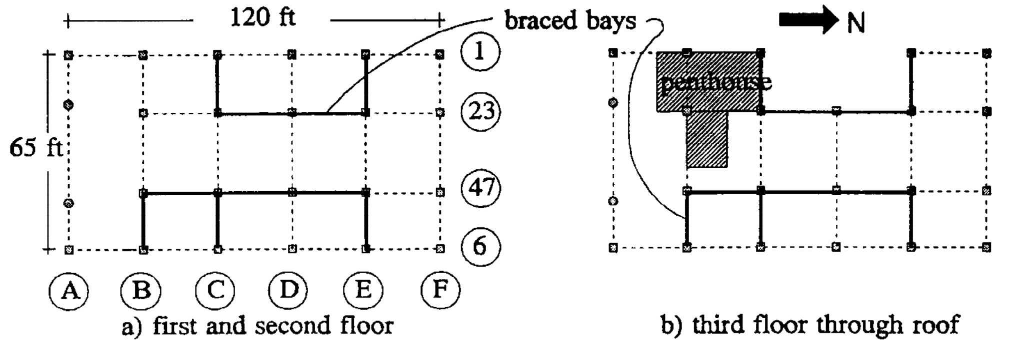



Fig.3 Plan view of CSMIP Station No. 23544

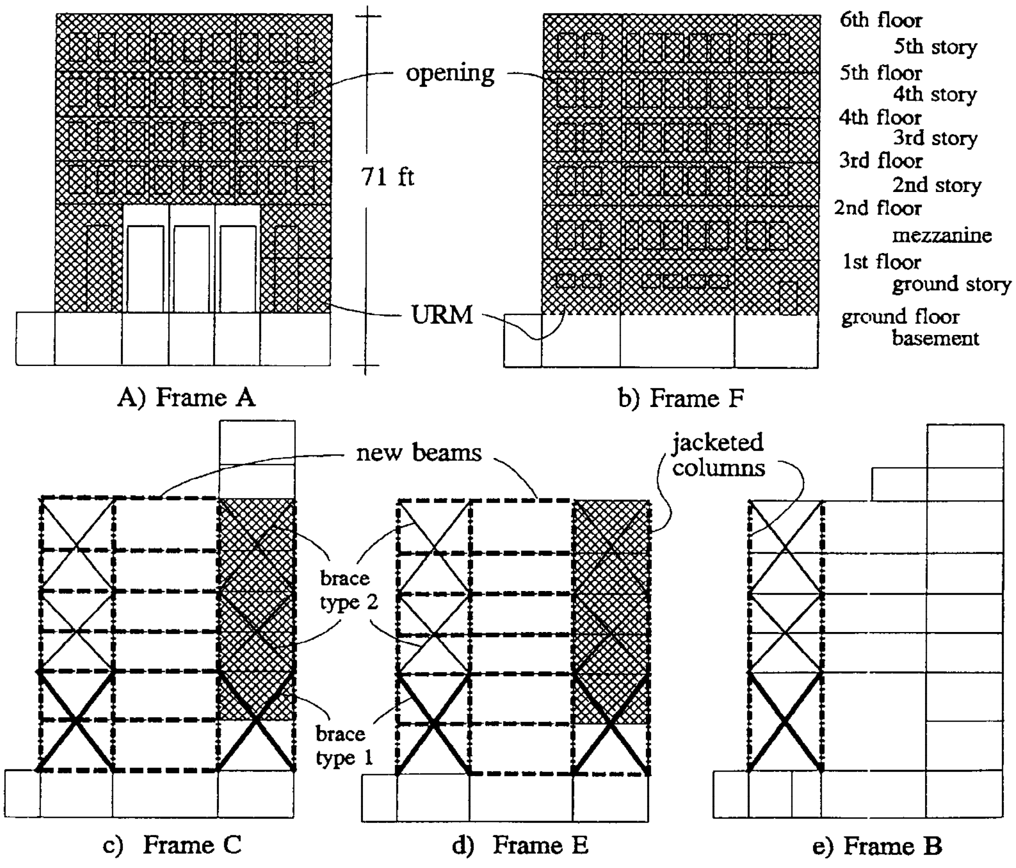



Fig.4 Elevation view of frames in E-W direction

Modelling Considerations

Under cyclic loading beyond cracking, URM infills suffer large degradation of stiffness and strength, and their viscous damping coefficient usually increases considerably with respect to that of the virgin infill (Klingner and Bertero, 1981). Because of the above, the large variability of the mechanic characteristics and geometry of the infills, and the fact that their lateral stiffness and strength are very sensitive to the quality control of the material and to workmanship, the analysis of an infilled frame building usually involves a large uncertainty.

At some stage of their lateral behavior, infills usually suffer extensive diagonal cracking which foments the formation of a lateral load-resisting mechanism based on one or more compression struts. Thus, infills can be modelled as equivalent struts whose properties may be determined from their lateral force vs. lateral deformation curves determined experimentally or analytically. Considerations done to model the URM infills with equivalent struts for the linear and nonlinear analyses reported herein are discussed in Teran *et al.* (1995). Cracking of the concrete was accounted for in the estimation of the stiffness of the RC members for the above mentioned analyses. For planar (2D) nonlinear analyses, a lumped plasticity model was used for the RC members, while for three-dimensional (3D) nonlinear analyses, a fiber model was used (Teran *et al.* 1995).

Seismic Performance of the Building Before and After its Rehabilitation

As mentioned before, the quantification of the performance criteria can be made through setting limits to the maximum IDI demand in the upgraded building. Table 1 shows this quantification, which is discussed in detail in Teran *et al.* (1995). Usually, energy dissipation should also be considered in the quantification of the performance criteria, but in the current case, this was done indirectly through setting drift limits in such a way that the URM infills are likely to exhibit stable hysteretic behavior. The critical IDI is 0.005 and is associated to the performance criteria for the URM infills. Because an adequate performance of the upgraded building depends on controlling its maximum IDI demand through controlling its lateral displacement, a displacement ($\delta$) design spectra was established simultaneously with a strength ($S_a$) design spectra for the safety limit state. The site spectra, illustrated in Fig.5, were obtained for a viscous damping coefficient ($\xi$) of 0.05 and according to the guidelines of SEAOC (1993).

Table 1. Quantification of performance criteria for safety

| Response Condition: SAFETY | Condition to satisfy performance criteria | Quantification of IDI to satisfy performance criteria |
|---|---|---|
| State of RC members | Elastic | Can at least undergo IDI demands of 0.005 |
| State of URM infills | Stable hysteretic behavior | Less or equal to 0.005 |
| State of PT braces | Elastic | To be designed |
| State of nonstructural elements | Pre-collapse | Less or equal to 0.0125 |

Although in the original research work the rehabilitation was carried out in both the N-S and E-W directions (Teran *et al.*, 1995), in this paper only the behavior in the E-W direction of the building is discussed. The fundamental mode of translation (T) and the seismic coefficient ($S_a/g$) of the original building in this direction were estimated as 1.0 sec and 0.14, respectively. Fig.5a shows that a single-degree-of-freedom system (SDOFS) having a $S_a/g$ of 0.14 and a T of 1.0 sec would suffer a displacement ductility demand, $\mu$, of about 4 when subjected to the design ground motion; while Fig.5b shows that the same SDOFS would have a displacement demand around 8".

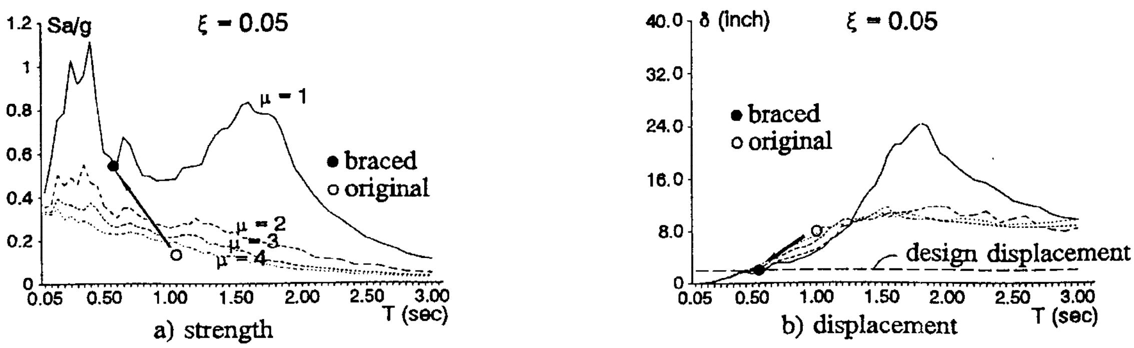



Fig.5 Design Spectra

Figure 6 shows the roof displacement ($\delta_{roof}$) vs. base shear ($V_b$) curve obtained from a 2D nonlinear pushover analysis of the original building, while Figs.7 and 8 show floor displacement and IDI envelopes obtained from a 2D nonlinear time-history analysis of the original building subjected to a ground motion having a seismic input consistent with that of the design spectra. Note that in Figs.7 and 8, both the negative and positive envelopes are plotted in the same side of the displacement or IDI axis. As shown in Fig.7, the maximum $\delta_{roof}$ demand obtained from the nonlinear analysis (around 10") is consistent with the $\delta$ of 8" obtained in the SDOFS on Fig.5b; while the IDI demands in the lower stories reach values of around 0.02, which as suggested in Fig.8, are too large for the non-ductile frame members and URM infills to accommodate. Fig.9, which shows how the six floor diaphragms move relative to each other and to the ground (discontinuous rectangle) according to a 3D nonlinear pushover analysis, suggests that the building does not exhibit a significant torsional response when loaded in the E-W direction, and thus that the 2D results may be used to assess the response of the building.

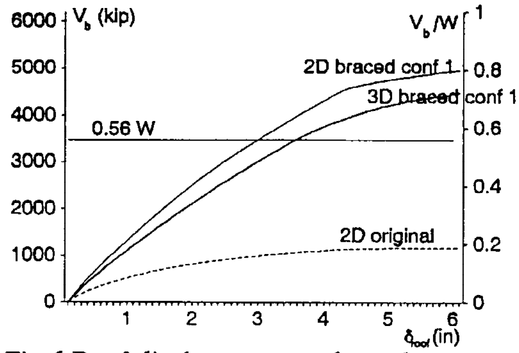



Fig.6 Roof displacement vs. base shear curves obtained from nonlinear pushover analyses

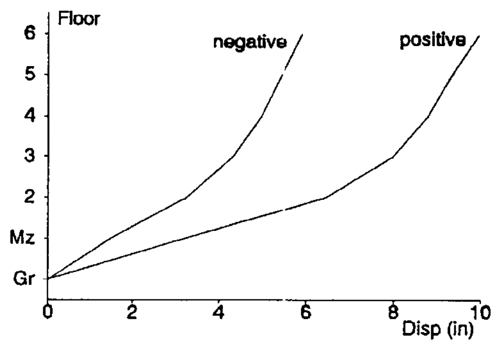



Fig.7 Floor displacement envelopes from 2D non-linear time-history analysis of original building

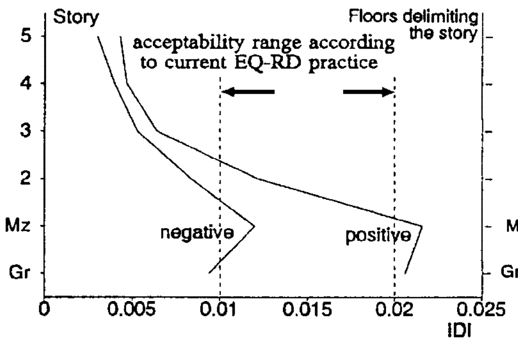



Fig.8 IDI envelopes from 2D nonlinear time-history analysis of original building

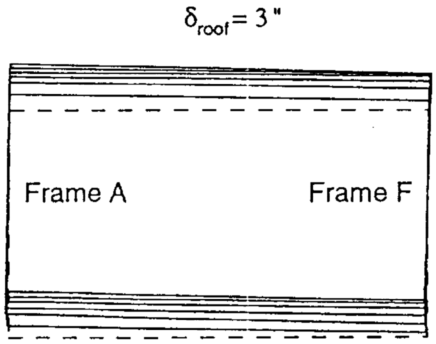



Fig.9 Floor displacement envelope from 2D non-linear time-history analysis of original building

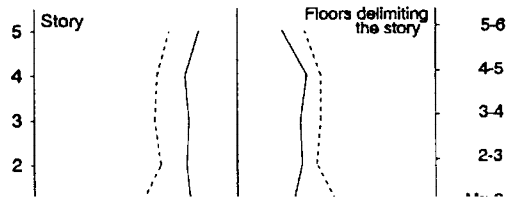
It was estimated that the maximum IDI demand in the rehabilitated building reaches a value of 0.005 approximately when its $\delta_{roof}$ reaches a value of 2.5", which corresponds to a $\delta$ of about 2" in its equivalent SDOFS model (Teran *et al.*, 1995). Thus, to limit the maximum IDI demand in the building to a value of 0.005, it is necessary to first reduce the T of the building from 1.0 to about 0.55 sec, as schematically shown in Fig.5b. Once the maximum T has been set as 0.55 sec, it is possible to determine the design strength by considering that the braces are designed to remain elastic, and thus that $\mu$ is equal to 1, as shown in Fig.5a. This yields a $S_a/g$ of about 0.56 (design $V_b$ of 0.56W, where W is the reactive weight of the building). Note that using design spectra corresponding to a $\xi$ of 0.05 is usually conservative, because it neglects the energy dissipation provided by the infills. The PT bracing system was designed to meet the requirements illustrated in Fig.5. As shown in Figs.3 and 4, braces were located in internal frames to avoid disturbing the architecture of the facades. Each diagonal in Fig.4 represents two braces, which are provided one at each side of the frame members to avoid creating undesirable loading conditions. Every brace spans two floors and, as shown in Figs.4c to 4e and Table 2, two different type of braces were used. Braces with bigger cross section were provided in the lower two stories to diminish the existing strength and stiffness irregularities in height. Initially two brace configurations were considered: configuration 2, which includes the 5 braced bays in the E-W direction shown in Figs.3 and 4; and configuration 1, which includes only 4 braced bays because the one shown in Fig.4e is eliminated. The braces are made of cable wire with a minimum yield stress ranging from 140 to 160 ksi, half of which is used for post-tensioning. As shown in Fig.4, the majority of columns located in Frames B, C and E need to be jacketed to resist the large axial forces induced to them by the braces, while some beams need to be added to avoid the buckling of the one-way slab floor system. Also, given the large axial force induced at the base of the columns that support the braces, the existing basement and foundation need to be modified and upgraded (Teran *et al.* 1995).

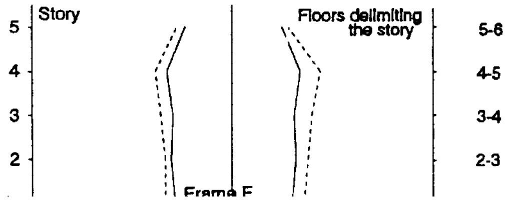
The T corresponding to the upgrade configurations 1 and 2 are 0.70 and 0.62 sec, respectively. Fig.6 shows the

Table 2. Sizes of the PT braces

| Brace type | Required area ($in^2$) | Nominal diameter (in) | Provided area ($in^2$) |
|---|---|---|---|
| 1 | 5.2 | 3 1/8 | 5.86 |
| 2 | 2.6 | 2 1/8 | 2.71 |

$\delta_{roof}$ vs. $V_b$ curves obtained from 2D and 3D nonlinear pushover analysis of upgrade configuration 1. As shown, the target $V_b$ of 0.56W is reached for a $\delta_{roof}$ larger than the target value of 2.5" set for $\delta_{roof}$, because the actual T of 0.70 sec is larger than its design counterpart of 0.55 sec. Figures 10 and 11, which were obtained from 3D time-history elastic analyses, show IDI envelopes corresponding to the end frames (A and F) of the building when upgrade configurations 1 and 2, respectively, are subjected to a ground motion with seismic input consistent with that of the design spectra. As shown, both configurations are able to control successfully the maximum IDI demand to values less than 0.005 while eliminating the formation of the original soft story. Nevertheless, the difference between the IDI demands of Frame A and F in Figs.10 and 11 suggest that while configuration 2 is able to control the torsional response of the upgraded building, configuration 1 can not, specially in the ground story. Figures 12 and 13, which show floor displacement and IDI envelopes, respectively, obtained from the 2D nonlinear time-history analysis of configuration 1 subjected to the same seismic input mentioned above, shows that the $\delta_{roof}$ and IDI demands have been controlled within their target design values of 2.5" and 0.005, respectively, while the deformation demands through height are fairly constant. Figures 12 and 13 strongly suggest that if the torsional response of the building is not significant, 4 braced bays (configuration 1) should be enough to achieve the design goals of the rehabilitation. Nevertheless, as suggested in Fig.10 and shown in Fig.14, which was obtained from the 3D nonlinear pushover analysis of configuration 1, there is a large torsional response associated to configuration 1. One last configuration was considered for the bracing system: configuration 3, which has only 4 braced bays which include all the braced bays shown in Figs.4c to 4e except for the first braced bay of Frame C. Figure 15, obtained from the 3D nonlinear pushover analysis of configuration 3, shows that for this case, the torsional effects are practically eliminated. A nonlinear 2D time-history analysis was not carried out on confi- guration 3, because its response should be very similar to that shown in Figs.12 and 13. Nonlinear 3D time-history analyses of the upgraded version of the building were not carried out because of the extremely large computational demands associated to such analysis, and thus, it was not possible to assess in a reasonable way the simultaneous in-plane and out-of-plane demands (IDI and out-of-plane acceleration, respectively) in the URM infills.



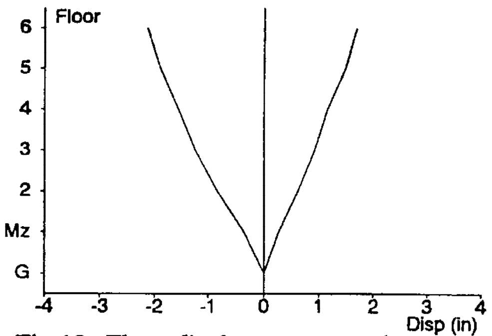



Fig.12 Floor displacement envelopes from 2D nonlinear analysis of configuration 1

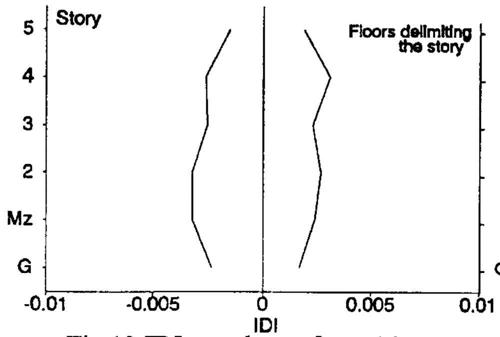



Fig.13 IDI envelopes from 2D non-linear analysis of configuration 1

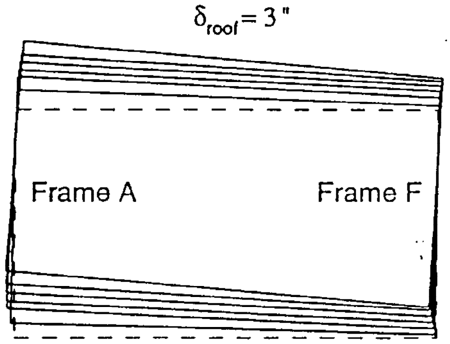



Fig.14 Displaced floor diaphragms according to 3D pushover analysis of configuration 1

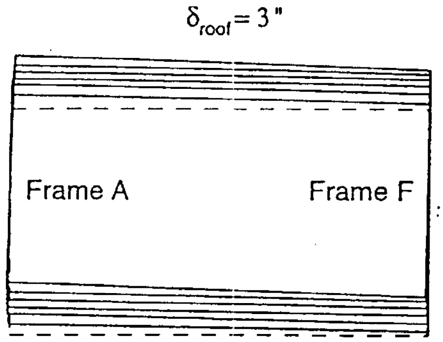



Fig.15 Displaced floor diaphragms according to 3D pushover analysis of configuration 3

Considerable research needs to be carried out to clarify several issues of the in-plane and out-of-plane behavior of URM infills, including the effects of multi-directional loading in their seismic performance and the development of reliable analytical methods and tools to model their behavior. The comparison of the results obtained from the elastic and nonlinear analyses of the upgraded building suggests that elastic analyses may be used reasonably well to estimate the response of infilled frame buildings rehabilitated with PT braces; nevertheless, it is necessary to develop methods to account for the energy dissipation provided by the URM infills (such as the use of an equivalent viscous damping coefficient).

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