STRUCUTRAL SYSTEMS RESEARCH PROJECT

SEISMIC BEHAVIOR OF BRIDGE COLUMNS BUILT INCORPORATING MMFX STEEL

By

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<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{bi}$</td>
<td>diameter of the column longitudinal bars anchored in footing</td>
</tr>
<tr>
<td>$f'c$</td>
<td>compressive strength of unconfined concrete</td>
</tr>
<tr>
<td>$f_{su}$</td>
<td>ultimate tensile strength of steel</td>
</tr>
<tr>
<td>$f_y$</td>
<td>yield strength of steel</td>
</tr>
<tr>
<td>$f_{ye}$</td>
<td>yield strength of the longitudinal reinforcement</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Young’s modulus of concrete</td>
</tr>
<tr>
<td>$H$</td>
<td>distance from base of column to point of application of lateral force</td>
</tr>
<tr>
<td>$I_e$</td>
<td>effective stiffness of cracked concrete section</td>
</tr>
<tr>
<td>$L_p$</td>
<td>equivalent plastic hinge length</td>
</tr>
<tr>
<td>$M_n$</td>
<td>nominal flexural strength</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>yield spread coefficient</td>
</tr>
<tr>
<td>$\beta$</td>
<td>strain penetration coefficient</td>
</tr>
<tr>
<td>$\varepsilon_{su}$</td>
<td>strain at the ultimate tensile force (uniform strain)</td>
</tr>
<tr>
<td>$\phi$</td>
<td>curvature</td>
</tr>
<tr>
<td>$\phi_y$</td>
<td>reference yield curvature in idealized moment-curvature relationship</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>lateral displacement</td>
</tr>
<tr>
<td>$\Delta_y$</td>
<td>reference yield displacement of the idealized bi-linear lateral force displacement response</td>
</tr>
<tr>
<td>$\mu_\Delta$</td>
<td>displacement ductility $= \Delta / \Delta_y$</td>
</tr>
<tr>
<td>$\mu_{\Delta\mu}$</td>
<td>displacement ductility capacity</td>
</tr>
<tr>
<td>$\mu_\phi$</td>
<td>curvature ductility $= \phi / \phi_y$</td>
</tr>
<tr>
<td>$\rho_l$</td>
<td>area ratio of longitudinal reinforcement</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>volumetric ratio of transverse reinforcement</td>
</tr>
</tbody>
</table>
1. GENERAL
Two 35% scale bridge column units, representing typical construction of the approach structure for the Oakland Touchdown Substructure of the new San Francisco Oakland Bay Bridge, were constructed and tested as part of this research program. The Oakland touchdown or landing consists of parallel cast-in-place, multi-cell box girder superstructures, supported by twin column bents with pinned top and fixed bottom detailing. The reinforced concrete columns are expected to form plastic hinges with limited ductility demand at the bottom during the design earthquake (Seible et al., 2003).

The first unit, Unit 1, was built using conventional Grade 60 reinforcement conforming to ASTM 706, whereas Unit 2 was entirely built using high strength MMFX 2 reinforcing steel. Unit 1 was designed according to the Caltrans Bridge Design Specifications (July 2002). The column reinforcement in Unit 2 was proportioned to give approximately the same flexural strength of the benchmark unit, Unit 1.

The aim of the test program was to assess the seismic performance of columns built using high strength reinforcing steel and to compare the performance with that obtained from columns constructed incorporating ASTM 706 reinforcement. MMFX 2 steel is a high strength reinforcing steel that has a nominal yield strength of approximately 100 ksi and a ultimate tensile strength of 150 ksi.

The tests conclusively showed that both units can be designed to form ductile flexural plastic hinges and can sustain drift levels of approximately 4% without failure and that the MMFX 2 reinforced column unit could respond in a limited ductile manner.
2. DESCRIPTION OF THE TEST UNITS

Figure 2.1 shows the dimensions of the geometrically identical units tested. The units incorporated a 3 ft. (914 mm) diameter and a shear span of 9.5 ft. (2.9m). The columns were founded on a 9 ft. long by 6.9 ft. (2.1m) wide by 2.6 ft. (762mm) thick footing and had a 5.4 ft. (1.65m) wide by 5.4 ft. (1.65m) long by 2.45 ft. (747mm) tall loading stub at the top of the column. The test units were cast in three different stages: the footing, the column and the loading stub. Self-consolidating concrete with $f'_c = 8$ ksi (55MPa) was specified throughout.

Figure 2.2 shows the reinforcing details of Unit 1 (ASTM A706 reinforcement). The column reinforcement for this unit consisted of two cages, each containing 42 #5 bars and with #3 hoops spaced at 1.56 in. (40mm) on centers. The longitudinal reinforcement ratio for the column of this unit was $\rho_l = 2.54\%$ and the equivalent volumetric steel ratio was $\rho_s = 1.74\%$. Design checks enabled the column bars of this unit to be anchored as straight bars into the footing. It should be noted that the incorporation of two cages is typical of the columns in the approach structure of the new San Francisco Oakland Bay Bridge.

The column of Unit 2 (MMFX 2 reinforcement), see Figure 2.3, incorporated only a single cage with 42 #5 MMFX 2 longitudinal bars and with #3 MMFX 2 hoops spaced at 1.56 in. (40mm) on centers. Therefore, the longitudinal reinforcement ratio for the column of this unit was $\rho_l = 1.27\%$ and the volumetric steel ratio was $\rho_s = 0.85\%$. Compared to Unit 1, the presence of higher strength MMFX 2 steel resulted in the elimination of the inner column cage, thus, resulting in significant savings in labor and construction time while matching the capacity.

Units 1 and 2 were designed to develop flexural plastic hinges at the base of the columns while the remaining regions in the specimens would remain elastic. A capacity design procedure was used to ensure this objective would be achieved. The design of the column of Unit 2 was aimed at developing the same nominal moment capacity of Unit 1.
The theoretical and idealized moment-curvature relationships for the columns of these units is depicted in Figure 2.4. The moment-curvature relationships show that the initial response, up to 500 kip-ft. (675 kN-m), the units have nearly identical moment-curvature relationships. This is because the concrete remains uncracked and contributes significantly towards the flexural rigidity of the section whereas the influence of the longitudinal reinforcement is negligible. Upon cracking and up to the first yield point, see point \((M_y, \phi_y')\) in Figure 2.4, the moment-curvature response becomes significantly influenced by the longitudinal steel ratio while the influence of the concrete gradually becomes less significant. The theoretical effective flexural rigidities, defined as \(E_c I_e = M_y / \phi_y'\) where \(E_c\) is the concrete elastic modulus and \(I_e\) is the effective moment of inertia, were \(E_c I_e = 1.53 \times 10^6\) kip-ft\(^2\) (0.63 \times 10^6 kN-m\(^2\)) and \(E_c I_e = 0.74 \times 10^6\) kip-ft\(^2\) (0.31 \times 10^6 kN-m\(^2\)), for Units 1 and 2, respectively. Thus, the effective flexural rigidity for the section of the column of Unit 1 was about twice that of Unit 2. The analysis also shows that the nominal moment capacity for the sections, \(M_n\), was nearly identical, being \(M_n = 2788\) kip-ft. (3764 kN-m) for the column section of Unit 1 and \(M_n = 2798\) kip-ft. (3777 kN-m) for that of Unit 2. Figure 2.4 indicates that the ultimate curvatures predicted for the column sections of Units 1 and 2 were similar. The ultimate curvature predicted for the Unit 1 was limited by fracture of the transverse reinforcement while the ultimate curvature of Unit 2 was the result of longitudinal bar fracture. For Unit 2, the theoretical curvature ductility capacity, given as the ratio of the ultimate curvature and the reference yield curvature, was \(\mu \phi = 0.00208 / 0.000332 = 6.3\).

The lateral displacement response of the units was computed by double integration of the curvature distribution along the column height. The fixed-end rotation due to strain penetration of the column longitudinal bars anchored in the footing was included in the calculations [Priestley et al., 1996]. All other sources of deformation in the column and the small flexibility of the footing were ignored. The idealized bi-linear moment-curvature diagrams shown in Figure 2.4 were used for this purpose. The equivalent plastic hinge length \(L_p\) proposed by Priestley et al. (1996) was employed to determine the post-elastic lateral force–drift response of the units:
Figure 2.5 depicts the theoretical monotonic lateral force-drift response of the two units. As expected for these units, the lateral force-drift response is significantly influenced by the sectional moment-curvature response. The reference yield drift computed for Unit 2 of $\Delta y / H = 1.64\%$, where $\Delta y$ is the reference yield displacement of the idealized bi-linear response, was about twice the value computed for Unit 1 of $\Delta y / H = 0.79\%$. A bi-linear response, with a small post-elastic stiffness, was predicted for Unit 1 whereas an elasto-plastic response was predicted for Unit 2. For Unit 1 the theoretical ultimate drift was $\Delta u / H = 3.7\%$ and for Unit 2 the ultimate drift was $\Delta u / H = 4.6\%$.  

$$L_p = 0.08H + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \quad (f_{ye} \text{ in ksi})$$

where $H$ is the distance from the base of the column to the point of application of the lateral force, $f_{ye}$ is the yield strength of the longitudinal reinforcement and $d_{bl}$ is the diameter of the column longitudinal bars anchored in the footing.
3. MATERIAL PROPERTIES

Self-consolidating concrete with a compressive 28-day strength of 8 ksi (55 MPa) was specified for the test units. The measured spread of the fresh concrete, which gives an indication of its flowability, and the compressive strengths are shown on Table 3.1. Note that the measured compressive strengths of the concrete cast in columns of 9.3 ksi (64 MPa) and 8.19 ksi (56.5 MPa) are within typical variations expected from different batches. The difference between these strengths has a very small influence in the flexural strength and ductility of columns loaded with small axial compression as was the case of the columns tested.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Spread [in.]</th>
<th>Location</th>
<th>Age at testing [day]</th>
<th>$f'_c$ [ksi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>25</td>
<td>footing</td>
<td>34</td>
<td>9.54</td>
</tr>
<tr>
<td>1</td>
<td>27</td>
<td>column</td>
<td>21</td>
<td>9.30</td>
</tr>
<tr>
<td>2</td>
<td>28.5</td>
<td>footing</td>
<td>27</td>
<td>10.50</td>
</tr>
<tr>
<td>2</td>
<td>26</td>
<td>column</td>
<td>22</td>
<td>8.19</td>
</tr>
</tbody>
</table>

(1) Average of three tests

Figures 3.2 to 3.6 show the tensile stress-strain relationships obtained from three coupons of each bar types used in the columns on Units 1 and 2. Table 3.2 summarizes the main properties of these bars. The #3 and #5 bars used to reinforce the column of Unit 1 were conformed to ASTM 706 requirements. The #3 bars did not show a typical yield plateau as these bars were cut from spare hoops and straightened before testing.

The #3 and #5 MMFX 2 bars showed a non-linear response which is characteristic from high strength reinforcing and stainless steels. As these bars did not show a yield plateau, the yield strength was obtained with the 0.2% strain offset definition. The #5 bars had a yield strength of 94 ksi (648 MPa) and a tensile strain of $\varepsilon_{su} = 5.1\%$ at the peak tensile force. The #3 bars were tested as straight coupons. Of the six coupons tested, three
included a welded splice. One of the welded bars failed prematurely at the weld. The remaining welded bars failed next to the weld in the heat-affected zone. As shown in Figures 3.5 and 3.6 and in Table 3.2, the presence of the weld splice had a detrimental effect on the tensile strain at the peak tensile force.

Table 3.2 – Measured reinforcing material properties.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Bar #</th>
<th>( f_y )</th>
<th>( f_{su} )</th>
<th>( \varepsilon_{su} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>#3(^{(1)})</td>
<td>60.0 ksi</td>
<td>93.3 ksi</td>
<td>12.8 %</td>
</tr>
<tr>
<td>1</td>
<td>#5</td>
<td>61.8 ksi</td>
<td>103.2 ksi</td>
<td>10.5 %</td>
</tr>
<tr>
<td>2</td>
<td>#3(^{(2)})</td>
<td>120.0 ksi</td>
<td>170.6 ksi(^{(4)})</td>
<td>5.3 %</td>
</tr>
<tr>
<td>2</td>
<td>#3(^{(3)})</td>
<td>135.0 ksi</td>
<td>160.0 ksi(^{(4)})</td>
<td>2.5 %</td>
</tr>
<tr>
<td>2</td>
<td>#5</td>
<td>94.0 ksi</td>
<td>154.8 ksi(^{(4)})</td>
<td>5.2 %</td>
</tr>
</tbody>
</table>

(1) Straightened coupons
(2) Straight bar
(3) Straight bar with fuse weld, weld fracture at one out of three coupons
(4) Yield strength defined with 0.2% strain offset criterion
4. TEST RESULTS AND OBSERVATIONS

4.1 General

The test units were subjected to unidirectional quasi-static reversed cyclic loading. A 500 kip (2224 kN) capacity actuator was placed at 9.5 ft. (2.9m) above the base of the column to apply push-pull cycles, see Figure 1.1. Equivalent gravity load was simulated via four externally post-tensioning threaded high-strength bars. A constant axial load of 600 kip (2669 kN) was applied through by four hollow-core jacks. A pin connection attaching the post-tensioned bars to the strong floor as well as slots in the footing allowed the external bars to sway with the units as they were subjected to lateral displacements. Through the use of computer controlled hydraulic valves the axial load was kept constant despite the elongation and shortening of the threaded bars under cyclic loading.

To prevent slip and foundation uplift, the footing was post-tensioned to the strong floor using threaded rods. The rods were fed through the vertical ducting in the footing and in the strong floor. The force provided by every rod was 200 kips (890 kN), except for the rods in the corners of the footing were the force was 140 kips (668 kN).

The loading protocol for Unit 1 was divided in force-controlled and displacement-controlled phases. The initial cycles, performed while the unit remained elastic, were force-controlled. Cycles to 25, 50 and 75% of the theoretical yield strength were applied in this phase. In the displacement phase three cycles to a nominal displacement ductility of $\mu_\Delta = +2$, followed by three cycles to a nominal displacement ductility of $\mu_\Delta = +3$, followed by three cycles to a nominal displacement ductility of $\mu_\Delta = +4$, followed by three cycles to a nominal displacement ductility of $\mu_\Delta = +6$ and so on, were applied to the test unit. The target lateral displacement in this phase was obtained as the target displacement ductility times the reference yield displacement. The reference yield displacement was obtained by integration of the theoretical curvatures along the column and accounting for the theoretical fixed-end rotation. Unit 2 was tested under displacement-controlled cycles, following the identical displacement test pattern applied.
to Unit 1, except that three cycles to a drift of +1% were applied to the unit to fully characterize the response at the onset of yielding.

4.2 Summary of Observations Recorded During Testing

In Unit 1 the first cracks were observed to develop at the column base from the very first cycle to 25% of the theoretical lateral force capacity of the unit. These cracks were only 1/250 in. wide and closed completely once the unit was unloaded.

The first cycles beyond the elastic limit enable the determination of the actual reference yield drift at 1.0%. This value was 26.5% greater than that predicted theoretically. Cover-concrete crushing was observed to occur at the base of the column during the cycles to $\mu_\Delta = +2$, which corresponded to a drift ratio of 2%. Residual crack widths reached 1/100 in.

As the test continued to $\mu_\Delta = +3$, which corresponded to a drift of 3%, more cover concrete crushed. The formation of a plastic hinge at the base of the column was evident during these cycles. Residual cracks of up to 1/50 in. were measured at the culmination of the cycles to this level of ductility.

Visible yielding of hoops was observed during the cycles to $\mu_\Delta = +6$, which corresponded to a nominal drift ratio of 6%. This led to incipient buckling of the longitudinal reinforcing bars that fractured during the first cycle towards $\mu_\Delta = +8$ when being straightened after significantly buckling had occurred in the last cycle to $\mu_\Delta = +6$, see Figures 4.3 and 4.4. At this point a significant visible structural damage had been observed and a decrease in the lateral force capacity of more than 20% of the maximum measured lateral force had been measured.

Unit 2 developed a crack pattern very similar to that recorded for Unit 1. The measured yield drift ratio for this unit matched exactly the value predicted theoretically of 1.64%. Crushing of the concrete cover at the base of the column was first noticed during the
cycles to a drift of 2.9%, which corresponded to $\mu_\Delta = +1.8$. At this level of loading very fine vertical bond-splitting cracks were also noticeable in the column.

Three complete cycles to a nominal drift of 3.9%, which corresponded to $\mu_\Delta = +2.4$, were completed with only cosmetic damage in the column resulting from crushing the concrete cover, see Figure 4.3. Up to this drift level the hoops had very effectively restrained the longitudinal bars from buckling.

Degradation in the response began with the fracture of a hoop at 125 mm above the base of the column. This hoop fractured at 3.1% drift on the first cycle to a target drift of 6%. The hoop fractured in the heat affected region adjacent to the fuse weld, see Figure 4.4 (a). This lead to the buckling of the column longitudinal bars in compression and to crushing of the concrete core. Further lateral displacement led to successive ductile fracturing of the longitudinal bars on the tension side of the column, see Figure 4.4 (b). The first longitudinal bar fractured at 4.5% drift and led to the gradual loss of lateral force capacity.

4.3 Hysteretic Response

Figures 4.5 and 4.6 plot the hysteretic lateral force-drift response for Units 1 and 2, respectively. The theoretical bi-linear response is also plotted in these figures. Both units showed a stable hysteretic response, this is clearly evidenced by the little differences that were observed in the response during between the first, second and third cycles to a target drift level. It is also evident that the theoretical bi-linear responses provide and excellent envelope to the measured response, except that the ultimate drift predicted for Unit 1 was significantly under predicted. The excellent match between the predicted and observed response for Unit 2 means that the response of components built incorporating MMFX 2 steel is predictable with current tools.

Unit 1 was, as expected, stiffer than Unit 2. A consequence of the increased flexibility for components designed with MMFX 2 reinforcement is that the displacement ductility capacity is smaller than that obtained for conventionally reinforced components if the
ultimate lateral deformation capacity are the same. For example, the displacement ductility
capacity of Unit 1 was $\mu = 5.7\% / 1.0\% = 5.7$. At 4% drift the displacement ductility for
Unit 1 was $\mu = 4\% / 1.0\% = 4$ whereas the displacement ductility capacity for Unit 2 was
$\mu = 3.9\% / 1.64\% = 2.3$. Unit 2 attained a limited ductility capacity which would be
acceptable under the design framework of the new San Francisco Oakland Bay Bridge
(Seible et al., 2003).

A comparison of the hysteretic response of Units 1 and 2 clearly indicates that the residual
drifts after a peak drift are much smaller for Unit 2 than for Unit 1, see Figure 4.7.

4.4 Components of Lateral Displacement

Figures 4.8 and 4.9 show the main sources of deformation at peak positive cycles of Units
1 and 2, respectively. The sources of deformation for both units are very similar. The
response was dominated by flexure, through bending in the column and fixed-end rotation
due to strain penetration of the longitudinal bars anchored in the footing. Column bending
accounted for about 60% of the deformations and fixed-end rotation accounted for about
30%. Other sources of deformation, like shear deformations in the column, did not exceed
10%.

4.5 Curvature Distribution

Figures 4.10 and 4.11 depict the curvature distribution in the columns of Units 1 and 2,
respectively. As it was expected, there was a concentration of curvatures at the base of the
columns as a plastic hinge developed there. Yielding in both columns extended from the
base up to about 1.5 times the column diameter.

The equivalent plastic hinge length was obtained from the data measured during the tests
for both units. This length can be expressed similar to Eq. 2.1, but with the generic
multipliers $\alpha$ and $\beta$:
\[ L_p = \alpha H + \beta d_{bl} \]  \hfill (4.1)

The values of \( \alpha \) and \( \beta \) computed for the two units at different displacement ductility levels is given in Figures 4.12 and 4.13, respectively. In both units the value obtained for factor \( \alpha \) is lower than that given by Eq. 2.1 of \( \alpha = 0.06 \), although for Unit 1 this factor approximates 0.06 at very high displacement ductilities. The smaller value of factor \( \alpha \) obtained for Unit 2 indicates that the plastic hinge is more constrained for components designed with reinforcements that have the stress-strain characteristics of MMFX 2 steel.

It is interesting to note that the values of factor \( \beta \) computed for both units are not greatly different, which indicates that factor \( \beta \) might not be strongly correlated to the yield strength of the reinforcement, \( f_y \) as Eq. 2.1 indicates, see Figure 4.13. The average values of factor \( \beta \) obtained for the positive and negative cycles resulted in \( \beta = 8.5 \) for Unit 1 and in \( \beta = 9.2 \) for Unit 2. The value of \( \beta = 9 \) predicted by Eq. 2.1 for the ASTM 706 reinforced Unit 1 was somewhat above the average value obtained in the test of this unit. Nevertheless, for the MMFX 2 reinforced Unit 2 Eq. 2.1 predicts \( \beta = 14.1 \) which is 53\% greater than the value derived from the test. The lower values of factors \( \alpha \) and \( \beta \) obtained for Unit 2 lead to the conclusion that plastic hinges in MMFX 2 reinforced components tend to spread less that for those reinforced with ASTM 706 reinforced components. It is therefore recommended that the prediction of the equivalent plastic hinge length when using MMFX 2 reinforcement be evaluated from Eq. 4.1 with \( \alpha = 0.025 \) and \( \beta = 9.2 \). In some particular cases the evaluation of the lateral displacement capacity for columns reinforced with MMFX 2 steel might result in drifts ratios below 4\%, a value that is desirable at the collapse prevention limit state. Debonding of the longitudinal reinforcement at the base of the column and in the footing immediately below the column base or increasing the tensile strain capacity in the reinforcement will result in an increase of the lateral displacement capacity to achieve this performance limit state.
5. DISCUSSION

The testing of the MMFX 2 Unit 2, conclusively showed that components designed with high strength reinforcing steel with the stress-strain relationship of MMFX 2 steel, can be designed for earthquake resistance for limited ductility response. The response of such components can be accurately predicted with current analytical tools. This is clearly evidenced by comparing the envelope of the lateral force-drift response with the theoretical prediction as shown in Figure 5.1.

In comparison with the ASTM 706 reinforced Unit 1, the MMFX 2 reinforced Unit 2 showed a softer initial response, smaller ultimate deformation capacity, smaller displacement ductility capacity and smaller residual drifts.

The main implication of the limited ductility capacity observed for the MMFX 2 unit is that components designed for earthquake resistance will have to be designed for lateral forces that are greater than those used currently in conventional designs. The larger design lateral forces and the residual smaller drifts that are expected from components designed with higher strength steels have inherent advantages. The current design philosophy of ductile design gives significant weight to seismic performance to the displacement ductility capacity and this is reflected in the lateral force reduction factors used in design. Little, if any, emphasis is currently given to the loss of performance due to residual drifts. However, newer trends in seismic design are moving more towards giving more emphasis to the impact that residual drifts can have, mainly to issues related to loss of function and reparability after moderate, but more common, seismic events. From this point of view the use of higher strength reinforcements could be encouraged in seismic design. An additional advantage of higher strength reinforcements used as transverse reinforcement is the effectiveness to restrain the longitudinal reinforcement and avoid buckling. This is due to the fact that due to the stress-strain characteristics of this type of steel, the hoops do not show unrestricted yielding that is characteristic of hoops made of conventional steel.
Failure of Unit 2 was initiated by the sudden fracture of a hoop in the heat affected region of a weld. Further work is required to ensure that uniform strain of welded MMFX 2 bars is not significantly compromised by the presence of the weld. In addition, it is recommended that a minimum uniform strain, that is, the tensile strain associated with peak tensile force in a tensile, be certified as part of the quality assurance process.
6. CONCLUSIONS AND RECOMMENDATIONS

The testing performed to an ASTM 706 reinforced unit, Unit 1, and on a MMFX 2 reinforced unit, Unit 2 clearly indicates that ultimate drifts of 4% can be attained with current reinforcement detailing and independent from the reinforcement type and that the response of components designed for earthquake resistance with this type of reinforcing steel can be predicted accurately.

If designed for the same lateral force capacity, the response of components designed with higher strength reinforcing bars is expected to result in a softer initial response. A corollary of this is that such components will show a limited ductility response. Therefore, such components should be designed with lateral force reduction factors that are smaller than those used in the design of conventional components.

In general, the response of components designed using high strength reinforcing bars will most likely result in a reduction of residual drifts after moderate and larger earthquakes. This is because, when compared with the response of components designed with conventional steel reinforcement, the residual drifts are significantly smaller.

An advantage of using higher strength reinforcements as transverse reinforcement is the effectiveness to restrain the longitudinal reinforcement and avoid buckling. This is due the fact that due to the stress-strain characteristics of this type of steel, the hoops do not show unrestricted yielding that is characteristic of hoops made of conventional steel.

Further metallurgic work is required to ensure that the tensile strain at peak tensile force in welded connections is not reduced due to the presence of the weld. It is also recommended that a minimum tensile strain at peak tensile force be certified by the manufacturers. A minimum value of $\varepsilon_{u}$ = 0.06 is recommended in this report.
REFERENCES


APPENDIX A

Figure 2.1 – General dimensions of test units and test set-up
Figure 2.2 – Reinforcing details of the ASTM 706 steel reinforced Unit 1.
Figure 2.3 – Reinforcing details of the MMFX 2 steel reinforced Unit 2.
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(a) At peak of cycle to +XX.X

(b) During the cycle to –X.XX

Figure 4.2 – Longitudinal bar fracturing process resulting in failure of Unit 1.
Figure 4.3 – View of Unit 2 at the end of the test.

(a) Hoop fracture on North face  
(b) Ductile longitudinal bar fracture on South face

Figure 4.4 – Transverse and longitudinal bar fracturing resulting in failure of Unit 2.
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Figure 4.11 – Curvature distribution on column of Unit 2
Displacement ductility, $\mu_\Delta$

Figure 4.12 – Plasticity spread coefficient $\alpha$.

Figure 4.13 – Strain penetration coefficient $\beta$. 

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Figure 5.1 – Lateral force – drift response envelopes for Units 1 and 2.