A lack of experimental data related to the cyclic behavior of chimney structures to severe earthquake ground shaking has resulted in design standards generally ignoring the effects of ductility and adopting conservative aseismic design provisions. This paper presents results from an experimental study that demonstrates that correctly detailed reinforced-concrete chimney sections are not brittle but possess some ductility. The experimental results have been used to develop a nonlinear dynamic analysis procedure for evaluating the inelastic response of tall reinforced-concrete chimney structures. The procedure is used to study the seismic response of ten chimneys, ranging in height from 115 to 301 m. Based on the nonlinear dynamic study, a series of code design recommendations has been developed and incorporated into the 2001 CICIND code to reduce the seismic loads by a factor of $R = 2$ by detailing for ductility and preventing the formation of brittle failure modes. The 2001 CICIND design recommendations result in both improved performance and cost savings of up to 20% compared with existing design practices.

**Keywords:** ductility; load; plastic hinge; test.

**INTRODUCTION**

The behavior of tall reinforced-concrete chimneys subject to earthquake excitation is not well understood, and, consequently, codes of practice around the world provide conservative aseismic design guidelines. Codes of practice for structures generally recognize that it is not economical to design structures to remain elastic at the ultimate earthquake event and generally allow some inelastic behavior. A commonly accepted measure of the energy-absorption capacity of a structure is the ductility factor, which is the ratio of the displacement of the structure at failure to the displacement at first yield. Most codes specify the response spectrum method for calculating the magnitude and distribution of earthquake-induced forces in reinforced concrete chimneys. The key parameters associated with the earthquake analysis and the design response spectrum (DRS) specified in codes of practice are in the form $\text{DRS} = aCS \times IF \times LFIR$. The factor $aCS$ defines the elastic response spectrum representative of the site while the importance factor $IF$ effectively modifies the return period of the design earthquake event from the standard 475-year return period (that is, exceeding 10% in 50 years). The effective ductility factor $RILF$ modifies the elastic spectrum for inelastic response $R = \text{structural response factor}, LF = \text{load factor}$). The ductility factor specified by codes for chimneys has historically been significantly less than those for normal building structures due to the belief that chimneys were brittle with no redundancy and the failure of one plastic hinge could cause collapse. The adoption of a ductility value in the order of unity (that is, elastic behavior) tends to make reinforced concrete chimneys uneconomical in areas of high seismicity.

This paper presents results from an experimental study undertaken to investigate the ductility of typical reinforced concrete chimney sections under cyclic loading. Based on these experimental results, an inelastic analysis procedure has been established for assessing the performance of reinforced chimney structures under extreme earthquake excitation. This procedure has been used to analyze a number of chimneys from which design recommendations and appropriate ductility factors have been established. These recommendations have been incorporated into the 2001 edition of the International Committee on Industrial Chimneys (CICIND) Code, and allow a reduction of $R = 2$ in earthquake forces for chimneys that have been designed for limited ductility. The resulting chimneys are cheaper and have improved performance characteristics compared with chimneys designed using other codes of practice, such as CICIND (1998), ACI Committee 307 (1998), UBC (1997), and EC8-1 (1996).

The paper focuses on the seismic behavior of the windshield and does not address the response of the flue liner in any detail. It is noted, however, that a top-hung steel flue system has significant structural advantages over other more brittle flue systems in seismic regions because a lighter, thinner flue in tension can be used. Such tension systems have some inherent ductility and can accommodate the installation of energy dissipation devices at the lateral support locations to further reduce the overall earthquake response of the chimney. The study also assumes that the chimney has been appropriately designed for thermal effects so that over the operational life, the reinforced concrete windshield is not subject to excessive temperatures or excessive temperature gradients.

**RESEARCH SIGNIFICANCE**

A lack of experimental data related to the cyclic behavior of chimney structures to severe earthquake ground shaking has resulted in the current design standards adopting conservative aseismic design provisions. This paper documents a comprehensive and innovative experimental study undertaken to investigate the inelastic cyclic behavior of reinforced concrete chimney sections that typically possess a large diameter to thickness ratio, low axial stress ratio, and small reinforcement ratio. The experimental results demonstrate that the inelastic cyclic response is moderately ductile, which is in contrast to the current codes of practice that assume that the response is brittle. The experimental testing program and subsequent analyses have resulted in design recommendations that have been incorporated into the 2001 CICIND code. The aseismic design recommendations result in a reduction in earthquake forces of $R = 2$ and cost savings of up to 20% for chimneys constructed in high seismic regions.

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MS No. 01-305 received September 30, 2001, and reviewed under Institute publication policies. Copyright © 2002, American Concrete Institute. All rights reserved, including the making of copies unless permission is obtained from the copyright proprietors. Pertinent discussion will be published in the July-August 2003 ACI Structural Journal.
PREVIOUS EXPERIMENTAL TESTS

The few studies that have attempted to examine the behavior of reinforced concrete hollow tubes under cyclic loading have used test configurations that do not accurately reflect typical chimney sections with features such as lower diameter/thickness ratios $D/t$, higher axial stress ratios, or higher longitudinal steel ratios. Other tests have included the effects of transverse confinement steel, which is a reinforcement configuration generally not suited and not used in reinforced concrete chimneys. The pipes were specially spun to achieve this exceptionally thin thickness, with a concrete strength in the order of $D/t = 40$, which is representative of real chimneys. The pipes were specially spun to achieve this exceptionally thin thickness, with a concrete strength in the order of $D/t = 40$, which is representative of real chimneys. The cyclic lateral load placed in the 30 mm-thick concrete, resulting in a typical cover of 12 mm (two bar diameter). The stress-strain relationships for the heat-treated longitudinal steel used in the four tests are plotted in Fig. 3. Although the ratio of the ultimate stress to yield stress for the 5.8 mm heat-treated reinforcement was greater than the commercially available full-scale reinforcement, the ultimate strain at fracture was representative of commercial reinforcement. Consequently, for ultimate design strength calculations, the reinforcement stress $f_y$ at 3% strain was considered more representative than the yield strength $f_y$. Each of the four test units had a different ratio of longitudinal steel $f_y$ and hoop steel $f_y$ as described.

Details of test units

Four circular, hollow reinforced concrete specimens were designed and constructed to investigate the inelastic behavior of chimney sections under severe cyclic loading. The reinforced concrete pipes were assembled and configured as horizontal cantilevers and tested by applying a cyclic transverse load at the free end. The cantilever fixed-end support consisted of a 320 mm-thick reinforced concrete block (which simulated a pilecap) rigidly connected to a steel anchor block that was securely fixed to the laboratory strong floor.

The reinforced concrete tubes were constructed to be representative of the typical section properties of a reinforced concrete chimney. The critical section of the pipe was located adjacent to the pilecap base. The experimental setup for the tests is shown in Fig. 1 and 2. The outside diameter of the pipes was 1194 mm with a thickness of 30 mm, resulting in a diameter to thickness ratio of 40, which is representative of real chimneys. The pipes were specially spun to achieve this exceptionally thin thickness, with a concrete strength in the order of $D/t = 40$. (Conventional reinforced concrete construction techniques would have resulted in a minimum thickness of at least 50 mm).

An axial load of 226 kN was applied using two 16 mm-diameter prestressing wires placed symmetrically top and bottom within the pipe void. The cables were anchored externally to the pilecap and steel pipe at either end. The resulting axial compressive stress of 2 MPa corresponded to an $f_y/F_t$ ratio of 0.05, typical of reinforced concrete chimneys. The maximum length of pipe that could be spun by the commercial pipe manufacturer was 2.44 m, which was effectively reduced to 2.20 m to allow a development length in excess of 200 mm for casting the pipe reinforcement into the foundation. An additional length of steel pipe was designed and fabricated, and the two pipes connected using 12 steel straps that were bolted and epoxied to the steel and concrete sections, respectively, as shown in Fig. 1 and 2. The 4.565 m length of hybrid pipe resulted in a shear span to member diameter of 3.8, which was considered representative of reinforced concrete chimneys. The cyclic lateral load was applied to the cantilever tube using a 250 kN capacity actuator with a ± 125 mm travel range.

The 5.8 mm-diameter deformed longitudinal and 4.8 mm-diameter transverse (spiral) reinforcement was centrally placed in the 30 mm-thick concrete, resulting in a typical cover of 12 mm (two bar diameter). The stress-strain relationships for the heat-treated longitudinal steel used in the four tests are plotted in Fig. 3. Although the ratio of the ultimate stress to yield stress for the 5.8 mm heat-treated reinforcement was greater than the commercially available full-scale reinforcement, the ultimate strain at fracture was representative of commercial reinforcement. Consequently, for ultimate design strength calculations, the reinforcement stress $f_y$ at 3% strain was considered more representative than the lower yield stress value $f_y$. Each of the four test units had a different ratio of longitudinal steel $f_y$ and hoop steel $f_y$ as described.
in Table 2. The spacing of the circumferential reinforcement was in excess of 10 bar diameters, and, hence, was not expected to provide buckling restraint to the longitudinal reinforcement. Two ratios have been listed for the longitudinal reinforcement, concrete, and overall model reflected the characteristics of a full-scale prototype, thus enabling the results to be directly scaled. In particular, deformed reinforcement and a typical full-scale concrete mixture was used for the model tests (with the restriction of a 10 mm maximum aggregate size) to avoid the unrealistic bond characteristics associated with micro-concrete and to ensure that the experimental results were representative of equivalent full-scale prototype tests. Full descriptions of the experimental investigation and test results are provided in Reference 17.

**RESULTS OF EXPERIMENTAL INVESTIGATION**

**Cyclic behavior**

Test Units 1 and 4 behaved in a ductile and tough manner under cyclic loading as demonstrated from the force deflection hysteresis loops for test Unit 1 shown in Fig. 4. A series of circumferential cracks developed along the length of the pipe in Units 1 and 4, which opened and closed and widened as the longitudinal strains increased on subsequent cycles. In contrast, the ductility associated with Units 2 and 3 was less than that of Units 1 and 4 due to the low reinforcement ratio. This low reinforcement ratio resulted in the undesirable feature of the cracking moment exceeding the ultimate section capacity and, consequently, the development of only a single circumferential crack. Under cyclic loading, this single crack opened and closed and prevented the formation of further cracks, thus concentrating the damage and inelastic behavior to one location.

The strength, deformation, and ductility properties, together with estimates of the plastic hinge length for test Units 1 through 4, are summarized in Table 3. The ultimate drift for Units 2 and 3 was limited to 0.8% due to the low reinforcement ratio and the formation of only one circumferential crack. In contrast, Units 1 and 4 achieved ultimate drift ratios of 1.5 and 1.9%, respectively, and demonstrated good ductile qualities with the formation of plastic hinges of moderate lengths. All units achieved good displacement ductilities, although the large \( \mu \Delta = 13 \) value achieved with Unit 4 was a little misleading because the yield stress and associated yield displacement were unrealistically low. This highlights the difficulties associated with ductility factors: Does a high ductility value reflect a large ultimate displacement/curvature value or a low yield displacement/curvature value?

The hysteresis shape was stable for all test units with increasing displacements associated with strain hardening of the reinforcement and increasing bending moments. The reduction in stiffness associated with an increase in ductility is characteristic of the closure of wide cracks, softening of the concrete matrix, and the softening of the reinforcement due to the Bauschinger effect. The pinched shape of the hysteresis loops on the unloading section is common for members with low axial loads and reflects the opening of cracks in the compression zone and an associated reduction in stiffness from the combined effects of concrete and reinforcement to that of the reinforcement only. (A detailed description of the pinching effect is provided in Reference 18.) The loading

<table>
<thead>
<tr>
<th>Steel ratio</th>
<th>Unit 1, %</th>
<th>Unit 2, %</th>
<th>Unit 3, %</th>
<th>Unit 4, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho_v )</td>
<td>0.93</td>
<td>0.36</td>
<td>0.36</td>
<td>1.10</td>
</tr>
<tr>
<td>( \rho_v ) (400)</td>
<td>0.90</td>
<td>0.25</td>
<td>0.25</td>
<td>0.85</td>
</tr>
<tr>
<td>( \rho_0 )</td>
<td>0.75</td>
<td>0.75</td>
<td>0.25</td>
<td>0.75</td>
</tr>
</tbody>
</table>
The maximum tensile strains measured were large and in the order of 5.0%, whereas the compressive strains were less than 0.3% and, hence, smaller than the levels of 0.6 to 0.8% typically associated with unconfined concrete compression failure. In addition, the presence of large tensile strains ensures the neutral axis is located near the tube wall at the ultimate moment, thereby limiting the compressive strains. A significant increase in the axial load or reinforcement ratio would tend to move the neutral axis towards the centroid of the pipe, increase the compressive strains, and, therefore, reduce the ultimate curvature capacity of the section.

The cover concrete around the hoop and longitudinal reinforcement in the vicinity of the cracks began to steadily spall as the concrete was cycled back and forth from extreme tension to compression. The cyclic strains tended to weaken the concrete matrix through the development of microcracks and the cyclic opening and closing caused a slight but increasing mismatch of the two cracked surfaces, resulting in local deterioration of the concrete. This resulting zone of damaged concrete adjacent to the fixed end behaved like a plastic hinge.

The ultimate curvatures achieved were large and higher than the theoretical predictions, with values ranging from 0.034 to 0.051 m⁻¹ in Units 3 and 4, respectively, at the critical section adjacent to the fixed end. The product \( \phi_p \cdot D \) is a useful measure of the total tensile and compressive strain at a particular section with peak values ranging from 0.041 to 0.061 in Units 3 and 4, respectively. The curvatures were calculated by dividing the sum of the average strains on the extreme compression and tension sides of the pipe by the pipe diameter. The underlying assumption that plane sections remain plane was investigated with Unit 2, where average strains were measured at the base of the pipe over a gage length of 200 mm at four locations across the pipe diameter. The proposition that strains be assumed linear across the width of the section appeared reasonable, particularly because the cracks were circumferential flexural and not inclined shear-flexural cracks.

**Failure mode**

The test units were all loaded to failure by increasing the cyclic displacements, and all failed in a similar manner, with the cover concrete spalling and the exposed longitudinal steel buckling in compression and then fracturing in tension on the reverse cycle. The less ductile longitudinal steel in Unit 1 (\( \varepsilon_p = 11\% \)) fractured after eight cycles of testing, compared with 13 cycles for Unit 4 (\( \varepsilon_p = 20\% \)), and demonstrated the benefits of increased material ductility. The test units behaved in a very ductile manner before the longitudinal steel buckled, suggesting that typical chimney sections have adequate inherent ductility and curvature capacity without the introduction of anti-buckling steel due to both their low axial-stress and low reinforcement ratios. The possibility of the concrete shell buckling under compressive loads was investigated using Unit 3 where the pipe had been locally modified to shift the critical section some 300 mm (10 ft) away from the pile cap. The pipe exhibited no tendency to buckle locally and the transfer of shear forces was excellent, even when subject to wide flexural cracks at large displacements. An analysis of the concrete thin-walled cylinder confirmed that the axial stresses needed to cause local buckling were significantly greater than the concrete crushing strength for all test units.

**Equivalent plastic hinge length**

Park and Paulay and Priestley, Seible, and Calvi presented a simple model for calculating the ultimate displacement \( \Delta_u \) at the tip of a cantilever associated with the ultimate shear force \( V_p \). The model assumes that \( \Delta_u \) consists of an elastic component equal to the yield displacement scaled to the ultimate shear force \( \Delta_{yy} \) and an inelastic component \( \Delta_p \), representing deformations associated with the plastic hinge as expressed in the following

\[
\Delta_u = \Delta_{yy} + \Delta_p
\]  
(1a)

where

\[
\Delta_p = (\phi_u - \phi_{yy}) \cdot I_p \cdot (L - 0.5I_p)
\]  
(1b)

Equation (1a) and (1b) may be modified to estimate the tip displacement associated with a nominal shear force \( V^* \), as follows

\[
\Delta^* = \Delta_{yy} \cdot V^*/V_p + (\phi - \phi_{yy} V^*/V_p) \cdot I_p \cdot (L - 0.5I_p)
\]  
(2)

**Table 3—Ultimate strength, deformation, ductility, and plastic hinge properties for four test units**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Units</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit 3</th>
<th>Unit 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi_{yy} )</td>
<td>m⁻¹</td>
<td>3.8E-3</td>
<td>2.7E-3</td>
<td>3.0E-3</td>
<td>1.8E-3</td>
</tr>
<tr>
<td>( \Delta_{yy} )</td>
<td>mm</td>
<td>11.9</td>
<td>5.7</td>
<td>6.1</td>
<td>6.6</td>
</tr>
<tr>
<td>( V_p )</td>
<td>kN</td>
<td>77</td>
<td>40</td>
<td>43</td>
<td>75</td>
</tr>
<tr>
<td>( M_{yy} )</td>
<td>kNm</td>
<td>353</td>
<td>188</td>
<td>184</td>
<td>369</td>
</tr>
<tr>
<td>( \phi_p )</td>
<td>m⁻¹</td>
<td>45E-3</td>
<td>37E-3</td>
<td>34E-3</td>
<td>51E-3</td>
</tr>
<tr>
<td>( \phi_p \cdot D )</td>
<td>mm</td>
<td>0.054</td>
<td>0.044</td>
<td>0.041</td>
<td>0.061</td>
</tr>
<tr>
<td>( \Delta_u )</td>
<td>mm</td>
<td>70</td>
<td>35</td>
<td>35</td>
<td>85</td>
</tr>
<tr>
<td>( \mu_p )</td>
<td></td>
<td>12</td>
<td>14</td>
<td>11</td>
<td>28</td>
</tr>
<tr>
<td>( \mu_p = \phi_p/\phi_{yy} )</td>
<td></td>
<td>5.9</td>
<td>6.1</td>
<td>5.7</td>
<td>13</td>
</tr>
<tr>
<td>( I_p )</td>
<td>mm</td>
<td>300</td>
<td>100</td>
<td>100</td>
<td>350</td>
</tr>
<tr>
<td>( I_p/D )</td>
<td></td>
<td>0.25</td>
<td>0.08</td>
<td>0.08</td>
<td>0.29</td>
</tr>
<tr>
<td>( L )</td>
<td>mm</td>
<td>4565</td>
<td>4565</td>
<td>4265</td>
<td>4565</td>
</tr>
<tr>
<td>Drift</td>
<td>%</td>
<td>1.5</td>
<td>0.8</td>
<td>0.8</td>
<td>1.9</td>
</tr>
</tbody>
</table>
The observed damage and curvature ductility distribution indicated that the effective plastic hinge length for Units 1, 2, 3, and 4 was in the order of 300, 100, 100, and 350 mm, respectively, as listed in Table 3. These observations were verified by applying Eq. (2) with the actual curvatures \( \phi^* \) and excellent correlation was obtained between the actual and the predicted tip displacements \( \Delta^* \). In addition, the ultimate plastic hinge rotations varied from 0.4 degrees for Units 2 and 3 to 0.8 and 1.0 degrees for Units 1 and 4, respectively.

The effective plastic hinge length observed from the damage and calculated using Eq. (2) indicates some variation between Units 1/4 and Units 2/3. The hinge lengths for Units 2 and 3 are associated with a single crack and, hence, are not representative of the distributed damage expected from a section more heavily reinforced with a moment capacity in excess of the cracking capacity. In contrast, the Unit 1 and 4 tests produced meaningful results and suggested that a conservative plastic hinge length \( l_p = 0.2D \) or \( l_p = 0.05L = 0.05M/V \) could be assumed for thin-walled reinforced concrete circular hollow sections typical of chimneys. These nominal plastic hinge lengths are consistent with the tests undertaken by Whitaker and are in the order of 60% of the values recommended by Priestley, Seible, and Calvi for solid circular bridge columns.

### Splice tests

Lapped splices are generally recognized as a source of potential weakness in reinforced concrete structures subject to earthquake excitation. The effectiveness of lapped splices is of fundamental importance for chimney structures that are typically slip-formed and reinforced with longitudinal and hoop reinforcement without the provision of confinement ligatures. A number of special cyclic tests were carried out on a longitudinal section from the chimney with 50% of the reinforcement ratio calculated using Coldes program. The Coldes, a spreadsheet program, was developed to evaluate the bending moment strength, curvatures, and neutral axis depths at both the yield and ultimate conditions for reinforced concrete circular hollow sections. The program requires the input of the concrete strength \( f'_c \), nominal yield of reinforcement \( f_y \), reinforcement ratio \( \rho \), inner and outer radii, and axial force \( N \) which is input indirectly by optimizing the neutral axis depth \( d_n \). The algorithm for calculating the strength and geometric characteristics of the section is based on the following assumptions: plane sections remain plane, tensile stresses develop in the reinforcement (and not the concrete), and no relative slip occurs between the concrete and reinforcement. The characteristics of the section at yield were calculated using elastic stress-strain properties and the transformed area concept, whereas the ultimate properties were estimated using an equivalent rectangular stress block for concrete with an ultimate concrete strain of 0.3%.

The experimental results were consistent with the theoretical ultimate moment predictions from the CICIND, the ACI 307 code (without the inclusion of a capacity reduction or material factor), and the Coldes program. The Coldes calculations assumed a nominal steel stress at 3% strain, whereas the CICIND and ACI calculations were calculated on the basis of \( f_y = 400 \) MPa, and, hence, the reinforcement ratios for each unit were scaled by the ratio \( f_y/400 \).

The ultimate curvature \( \phi_u \) was calculated for a range of axial stress and reinforcement ratios using the Coldes program and expressed in the nondimensional form \( \phi_u/D \) where \( D \) represents the mean diameter of the section. The results have been plotted in Fig. 6 and clearly indicate the reduction in curvature capacity as the axial stress ratio or the reinforcement ratio increases. The experimental ultimate curvature values exceeded the predicted theoretical \( \phi_u/D \) values, suggesting that the assumptions in the Coldes program were conservative. In particular, the experimental study confirmed that the notional maximum steel and concrete strains of 5 and 0.3% were conservative for ductile reinforcement and unconfined concrete and accounted for the effects of low cycle fatigue. Studies have indicated that spalling of concrete is likely when the concrete strains approach 0.5 to 0.8%.

### Comparison of experimental results with theory

The study presented in this section demonstrates that the experimentally measured strength and deformation properties of reinforced concrete circular hollow members (with a large diameter-to-thickness ratio) were consistent with theoretical predictions. The effects of low cycle fatigue were included indirectly in the analysis by limiting the curvature capacity and, hence, ultimate extreme fiber strains of the reinforced concrete sections.

### Section behavior

Coldes, a spreadsheet program, was developed to evaluate the bending moment strength, curvatures, and neutral axis depths at both the yield and ultimate conditions for reinforced concrete circular hollow sections. The program requires the input of the concrete strength \( f'_c \), nominal yield of reinforcement \( f_y \), reinforcement ratio \( \rho \), inner and outer radii, and axial force \( N \) which is input indirectly by optimizing the neutral axis depth \( d_n \). The algorithm for calculating the strength and geometric characteristics of the section is based on the following assumptions: plane sections remain plane, tensile stresses develop in the reinforcement (and not the concrete), and no relative slip occurs between the concrete and reinforcement. The characteristics of the section at yield were calculated using elastic stress-strain properties and the transformed area concept, whereas the ultimate properties were estimated using an equivalent rectangular stress block for concrete with an ultimate concrete strain of 0.3%.

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while values of 0.5% are often used for calculating the ultimate curvature in concrete specimens. A minimum $\phi_d D$ value of 0.03 (which corresponds to a compressive strain of 0.3% and a tensile strain of 2.7%) is recommended for the design of chimneys to ensure ductile behavior.

The neutral axis depth (measured from the extreme compression fiber of the section) associated with the ultimate moment was calculated using the Coldes program for a range of axial stress ratios ($\sigma = 0$ to 0.20) and reinforcement ratios ($\rho_r = 0.3$ to 2.0%) assuming $f_y = 400$ MPa. The ratio of the neutral axis depth to shell thickness $d_p/t$ varied from 0.1 to 7 for 0.3% reinforcement and 4 to 11 for 2.0% reinforcement as the axial stress ratio increased from 0 to 0.20 as shown in Fig. 7. The ratios calculated from the strains measured in the experimental program suggested that $d_p/t = 2$ for Units 1 and 4 and $d_p/t = 1$ for Units 2 and 3, which were consistent with the theoretical results shown in Fig. 7. Clearly, the curvature capacity of a section reduces with an increasing $d_p/t$ ratio. The minimum recommended $\phi_d D$ value of 0.03 corresponds to a $d_p/t$ ratio of approximately 4.

**Member behavior**

A simple finite element model of the cantilever pipe system for test Units 1 and 4 was established using the inelastic frame analysis program Raumoko. The model consisted of a beam element with a discrete plastic hinge at the fixed end to model the concrete pipe and an elastic beam element to replicate the steel pipe. The hysteretic behavior of the plastic hinge was represented by the modified Takeda rule. An average stiffness of $0.5E_1 g$ and a yield moment equal to 70% of the ultimate moment was assumed for the pipe section, and plastic hinge lengths of 0.30 and 0.35 m were assumed for Units 1 and 4, respectively (Table 3). A comparison between the analytical and experimental behavior is presented in Fig. 8 for the final cycle of test Unit 1. A good correlation was displayed between the analytical prediction and the experimental results for test Units 1 and 4, confirming that a plastic hinge with the modified Takeda rule provided a reasonable fit to the data and accounted for the effects of stiffness degradation.

**Inelastic earthquake analyses of chimneys**

The earthquake design and analysis of chimneys to earthquake excitation has typically been undertaken using linear dynamic procedures such as the response spectrum or time history modal analysis technique. The modal analysis method accurately predicts the response of tall reinforced concrete chimneys in the elastic range as confirmed from a number of experimental studies carried out on real chimneys using ambient wind vibrations. These studies have been limited to the elastic range; however, the response of tall chimneys to severe earthquake ground motions may require the stack to respond in the inelastic range. A literature review indicated that few inelastic studies have been undertaken. In most of the studies, the inelastic behavior of the chimney was represented by a stick model using continuum finite elements (fiber elements) to explicitly represent the degrading hysteresis loops of the concrete and the reinforcement. Although these methods provide a useful insight into the inelastic response of chimneys, they assume that all sections are fully cracked and therefore ignore the significant effects of both tension stiffening and the tensile strength of concrete.

In this study, the elastic and inelastic seismic response of ten reinforced concrete chimneys ranging in height from 115 to 301 m were examined using a lumped mass stick model with a series of discrete plastic hinges. The discrete plastic hinge approach allows the analyst to directly model the effective elastic stiffness of the chimney to account for tension stiffening effects and provides some choice in the shape of the hysteresis selected for modeling the nonlinear and inelastic behavior. The inelastic procedure developed by the author involves a number of assumptions consistent with the experimental results and includes:

- plastic hinge length equal to 20% of the chimney diameter;
- yield moment equal to 70% of the ultimate moment;
- hysteretic behavior of the plastic hinges represented by the modified Takeda hysteretic model; and
- effective stiffness of 0.50 $E_1 g$ to account for cracking and the increased stiffness caused by tension stiffening effects.

The chimneys were designed to resist earthquake actions calculated from the 1994 UBC soft-soil response spectrum. An acceleration coefficient of 0.15 g corresponding to a 1 in 475 year event was selected to represent a region of moderate seismicity. The application of the 1.4 load factor in accordance with the 1998 CICIND recommendations increased the nominal elastic design earthquake to $a_e = 0.21$ g. The chimneys were detailed for ductility by providing overstrength around the openings so that the base remained essentially elastic, thereby encouraging inelastic flexural behavior to occur at higher levels in the windshield. The wind actions were representative of a temperate wind environment and were found to be significantly less than the lateral loads resulting from earthquake excitation. Based on the critical bending moments at each of the nodes, the required quantity of longitudinal reinforcement was calculated in accordance with the CICIND and ACI 307 recommendations for the ultimate limit state strength design of reinforced concrete chimneys.

Elastic time history analyses of the chimneys were then undertaken in accordance with the 1998 CICIND recommendations assuming 5% damping and uncracked properties. The
resulting elastic bending moments were evaluated and compared with the ultimate moment capacities at each node, and the accelerogram was scaled so that the ratio of the moment demand to moment capacity equaled, but did not exceed, unity at the critical node. The resulting scaled peak ground acceleration for that accelerogram was deemed the elastic acceleration $a_e$.

The chimney was then analyzed inelastically and each of the six accelerograms was scaled until the curvature demand exceeded the curvature capacity at one of the plastic hinges, at which point the chimney was deemed to have failed. The resulting scaled peak ground acceleration was deemed the failure acceleration $a_f$. The acceleration ratio $b = a_f/a_e$ provides a valuable insight into the ductility of the chimney with a ratio close to unity, suggesting a brittle structure while a ratio in excess of 3 to 4, implying some ductility. The results demonstrated that the $a_f/a_e$ ratio exceeded 4 for all chimneys, provided that a minimum curvature capacity of $\psi_D = 0.03$ was available at all potential plastic hinge locations up the height of the structure. The critical section of the chimney was in the region between 30 and 75% of the chimney height and indicated that the inelastic behavior was widespread with the formation of multiple plastic hinges rather than being confined to one plastic hinge location.

The study, which is described in detail in References 17, 40, and 42, highlights the complex dynamic response of a typical reinforced concrete chimney under earthquake excitation. The structure can be thought of as a highly tuned profiled cantilever that is whippy in nature and dominated by higher mode effects. The behavior of such a structure cannot be readily predicted using a simple static pushover analysis nor by a simple single degree of freedom substitute structure. Overall dynamic stability of the chimney is maintained as a result of the chimney sections possessing adequate curvature capacity through the inelastic yielding of reinforcement combined with the nature of earthquake ground motions that are characterized by a number of short-duration, high-frequency pulses that continually change direction.

A cost-effective solution for the design of chimneys to resist earthquakes is to limit the maximum moments that can be developed by encouraging the formation of multiple plastic hinges rather than a single plastic hinge. Multiple plastic hinges have the advantage that the inelastic curvature demand will be spread over a wide region of the chimney to dissipate the seismic energy. Chimneys that have been designed and detailed for ductility will inherently possess a reasonable curvature capacity at all plastic hinge locations and will consequently develop some global ductility. Design recommendations for encouraging ductile behavior and consequently reducing the earthquake forces by a factor of $R = 2$ to account for the hysteretic energy absorption are presented in Appendix 1. These recommendations result in cost savings for the wind shield and foundation in the order of 10 to 20% for the chimneys sited in high seismic regions and importantly produce chimneys that possess some ductility.

CONCLUSIONS

Based on the findings of this study, the following conclusions can be drawn:

1. The test units that were representative of chimney structures exhibited ductile and tough behavior with stable hysteresis loops under cyclic loading. The inelastic action was dominated by the development of large tensile strains in the ductile reinforcement and only small compressive strains in the unconfined concrete with the neutral axis located near the tube wall. The neutral axis position is characteristic of the relatively low axial loads and low reinforcement ratios associated with reinforced concrete chimneys. The theoretical and experimental studies suggested that the $D/h$ and axial stress ratios of typical chimney sections were too small to cause instability through local buckling of the concrete shell;

2. The higher reinforcement ratios of Units 1 and 4 ensured that the ultimate moment capacity significantly exceeded the cracking moment strength and resulted in the formation of multiple cracks along the length of the pipe. Consequently, a reasonable plastic hinge developed near the pipe base characterized by moderate concrete damage, large tensile strains, and large curvatures. The 1.5 and 1.8% drift ratio and plastic hinge rotations of 0.8 and 1.0 degrees for Units 1 and 4 exceeded the 0.8% drift ratio and 0.4 degree rotation developed in Units 2 and 3. The cracking strength of Units 2 and 3 exceeded the ultimate moment capacity of the pipe section and that prevented the development of more than one circumferential crack. This is a highly undesirable feature because a plastic hinge of reasonable length cannot form and all the inelastic action and damage is concentrated in the reinforcement and concrete adjacent to the solitary crack;

3. The maximum ultimate curvatures developed in the four test units were generally large and higher than the theoretical predictions. Strain measurements recorded across the pipe diameter near the base confirmed the assumption that plane sections remain plane was a reasonable approximation. A minimum curvature capacity of $\psi_D = 0.03$ is recommended for chimney structures so that all cross sections possess reasonable ductility. This minimum value corresponds to a ratio of neutral axis depth to thickness of $d_e/t = 4$, a compressive strain of 0.3%, and a tensile strain of 2.7%. The limiting compressive strain of 0.3% is considered modest and significantly less than the 0.5 to 0.8% range of strain needed to cause concrete spalling;

4. An effective plastic hinge length $l_e = 0.2D$ or $l_e = 0.05 M/V$ is considered a representative value for typical chimney sections provided that the ultimate moment capacity exceeds the cracking strength of the section;

5. The modeling of the cyclic behavior of test Units 1 and 4 using discrete plastic hinges with the modified Takeda hysteresis rule, and assuming an effective stiffness of 0.5 EI, provided a reasonable fit to the data and accounted for the effects of stiffness degradation. Importantly, this discrete plastic hinge representation is also computationally efficient and ideally suited for incorporation into an inelastic global finite element model for reinforced concrete chimneys;

6. The experimental tests suggested that splices with sufficient development length and with 50% of the bars continuous would behave satisfactorily when subjected to a moderate number of inelastic reverse cycles. Well-detailed reinforced concrete chimneys typically possess low reinforcement ratios and low axial stress ratios, and, consequently, inelastic action is developed through large tensile strains of the reinforcement with compressive strains typically less than the spalling strain of concrete. These moderate compressive strains result in the concrete responding essentially in the elastic range and further assist the performance of lap splices under cyclic loading. In addition, tall reinforced concrete chimneys possess a long natural period of vibration (often in excess of 3 to 4 s), and, consequently, lap splices would be subject to only a few cycles of reverse cyclic strains under a severe earthquake event;
7. Well-detailed reinforced concrete chimneys are not brittle and possess some ductility developed through yielding of the reinforcement in tension. Tall chimneys being highly tuned, profiled cantilevers respond in a complex manner to earthquake excitation with the response dominated by higher mode effects in both the elastic and inelastic range; and

8. The limited ductile design (LDD) approach described in this paper and incorporated into the 2001 CICIND code recommends that the seismic forces be reduced by a factor of $R = 2$ by detailing for ductility. The LDD approach encourages limited ductile rather than brittle behavior through the formation of multiple plastic hinges in the windshield away from openings to dissipate the seismic energy and minimize the induced seismic forces. LDD results in cost savings in the order of 10 to 20% for the windshield and foundation of chimneys sited in high seismic regions.

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NOTATION

$A$ = cross-sectional area

$\rho_{CS}$ = elastic response spectrum

$a_{0}$ = elastic acceleration coefficient

$a_{f}$ = failure acceleration coefficient

$b$ = acceleration ratio $= a_{f}/a_{0}$

$c$ = cement content

$D$ = mean diameter

$d_{n}$ = neutral axis depth

$E$ = elastic modulus $= 30,000$ MPa

$F'_{c}$ = concrete compressive strength

$f_{c}$ = axial compressive stress

$f_{y}$ = yield stress

$f_{p}$ = importance factor

$I_{4}$ = gross second moment of area

$L$ = cantilever length

$L_{F}$ = load factor

$L_{h}$ = plastic hinge length

$M$ = base bending moment

$M_{y}$ = yield moment

$M_{u}$ = ultimate moment capacity calculated from Eq. (4)

$n$ = normalized ultimate moment

$N$ = axial compressive force

$n_{a}$ = normalized axial force

$R$ = structural response factor

$r$ = wall thickness

$V$ = shear force

$V_{y}$ = yield strain of reinforcement

$\phi_{u}$ = ultimate curvature at $V_{y}$

$\phi_{y}$ = yield curvature scaled to $V_{y}$

$\phi$ = curvature associated with $V_{y}$

$\rho$ = longitudinal steel ratio

$\rho_{h}(400)$ = equivalent longitudinal steel ratio assuming $f_{y}$ = 400 MPa

$\rho_{h}$ = hoop steel ratio

$\rho_{h}$ = yield displacement factored to $M_{u}$

$\Delta_{w}$ = displacement associated with $V_{y}$

$\Delta_{y}$ = elastic displacement

$\Delta_{pl}$ = inelastic/plastic displacement

$\Delta_{u}$ = ultimate displacement

$\Delta_{p}$ = plasticity, ductility

$\phi_{d}$ = curvature ductility

$\Omega_{m}$ = moment overstrength factor

$\Omega_{y}$ = shear overstrength factor

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APPENDIX—CODE RECOMMENDATIONS

The seismic design approach described in this section that has been incorporated into the 2001 CICIND code\(^1\) is based on dual performance criteria:

* designing the chimney elastically to resist earthquake-induced loads considered reasonable for a serviceability limit state earthquake event (SLS); and
* designing the chimney with sufficient ductility so that the chimney will survive an extreme earthquake event without premature failure and collapse at the structural stability limit state (SSLS).

It is recommended that the seismic actions be calculated using the response spectrum method assuming uncracked properties with a structural response factor \(R\) factor dependent on the level of seismic detailing specified: \(R = 1.0\) (no specific seismic detailing) and \(R = 2.0\) (capacity design and seismic detailing). The design of the chimney should be consistent with the principles of capacity design. The foundation system and the shell in the vicinity of openings should be designed for overstrength (flexure and shear) so that inelastic flexural behavior will develop in the ductile regions of the shell away from significant openings. A moment overstrength factor of \(\Omega_m = 1.5\) is recommended at the base to prevent premature flexural failure. Similarly, a shear overstrength factor of \(\Omega_s = 2.5\) to account for both moment overstrength and the increased shear forces associated with inelastic action is recommended at the base reducing to \(\Omega_s = 2.2\) between 10 and 80% of the chimney height.\(^{17,43}\) This additional shear demand is typically not critical as the circumferential reinforcement required for ovaling wind moments usually provides sufficient capacity for seismic design.

Specific seismic detailing requirements include: use of ductile reinforcement with an ultimate tensile strain in excess of 10%, introduction of staggered splices, and specification of sufficient longitudinal reinforcement to ensure that the ultimate moment capacity of the chimney at any cross section is greater than the nominal cracking strength. In addition, to ensure adequate ultimate curvature capacity \(\theta_n D > 0.03\) in regions where plastic hinges could develop, the longitudinal reinforcement percentage should be limited to \(\rho = 0.240 - 14 n\), where \(n\) is the axial stress ratio \((n = f_p / f_y')\) and \(f_y' = 400\) MPa.

In regions of high seismicity, it is strongly recommended that chimneys be designed for limited ductility with \(R = 2.0\). This design strategy will result in chimneys that are both economical and sufficiently tough and ductile to survive an extreme earthquake event.\(^{43,44}\)