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Seismic behaviour of tall industrial masonry chimneys

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ABSTRACT: Two distinct types of failure have been documented in tall industrial masonry chimneys. Failure at the base of the chimney was found to be caused by the strong participation of the fundamental mode in the chimney's response. The strong participation of the second mode was found to be the cause of chimney failures which occurred at the top third of the height. Normally, this response mode is not a critical consideration under wind loading design. The application of a post-tensioning force was found to be an adequate retrofit technique. The technique was most successful for chimneys with potential for the excitation of the second mode of vibration.

1 INTRODUCTION

Many of the existing industrial chimneys in many parts of the world are constructed of unreinforced masonry. These structures were found to be vulnerable to damage during earthquakes. It has been documented that these types of industrial masonry chimneys may fail at either of two different locations. These structures have experienced extensive cracking or complete failure either at or near its base or at the top third of its height (China Academy of Building Research, 1986). Several cases of failure at the top third of the chimney's height were observed in chimneys located in the city of Tangshan, China, during the earthquake event of July 28, 1976 (Shu-quan, 1981). Chimneys that were located in the city of Beijing, China, experienced failure at their bases. The city of Tangshan was located very near the event's epicentre whereas Beijing was located over 150 kilometres from the epicentre.

The purpose of this study is to investigate the behaviour of tall industrial masonry chimneys when subjected to earthquake ground motion. An attempt is made to explain the cause of these two types of failure. The chimneys are modelled using a simple one-dimensional lumped mass approach. The chimneys are subjected to ground motion events that are typical of both near and far field events. A range of chimney heights and bottom outer diameters were examined in order to obtain an understanding of the behaviour of various possible configurations. The practice of post-tensioning the chimney is also examined as a possible retrofit technique.

2 ANALYTICAL PROCEDURE

A simple one-dimensional lumped mass system is used to model the chimney structure. This type of model is adequate to determine the vibrational characteristics as well as the respective mode shapes. Of interest in the study is to determine the location and magnitude of the maximum induced stresses in the chimney.

2.1 Selected chimney configurations

The chimneys analyzed in this study were designed according to the 1985 National Building Code of Canada (NBCC 1985) for wind loads and its self weight. Four different chimney heights were selected in this study. These heights are 20.0, 30.0, 40.0, and 50.0 metres. These values cover the practical range for existing masonry chimneys. The top diameter and the top wall thickness were fixed at 2.0 and 0.2 metres, respectively. The chimney heights were then non-dimensionalized with respect to the top outer diameter of the chimney (referred to as the height ratio). The bottom outer diameter and the bottom wall thickness were determined from the wind and self-weight design requirements of the NBCC 1985. The bottom wall thickness was determined to be 0.6 metres for all selected configurations. Design requirements for wind loads resulted in the diameter ratios of the chimneys to vary from 1.5 to 4.0, 2.0 to 4.0, 2.5 to 4.0, and 3.0 to 4.0 for height ratios of 10, 15, 20, and 25, respectively. The diameter ratio is the bottom outer diameter divided by the top outer

diameter. A total of 18 chimney configurations were considered in this study. A schematic of the chimney configurations analyzed is presented in figure 1A.

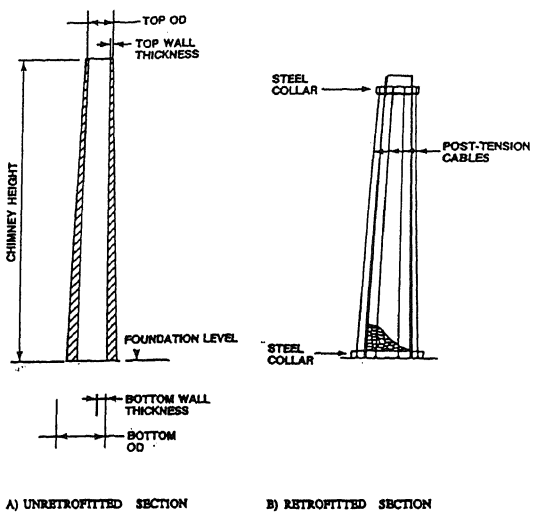

Figure 1. Schematics of chimney configurations.

2.2 Dynamic analysis of non-retrofitted structure

The system is idealized as a one-dimensional lumped mass system. The flexural deformations are considered to dominate the response of the structure. The effects of shear and rotary inertia were evaluated and then assumed to be negligible. The mass matrix of the system is generated assuming that the mass segments are annular elements. The outside diameter decreases linearly from the bottom to the top of the mass segment. The wall thickness is assumed to also have a linear variation along the chimney's height. The inside diameter is assumed to be either constant or decreasing with increasing height. The stiffness matrix for the system is generated using the standard principles of structural analysis (Liable, 1985). The variation in the moment of inertia of the mass segment is accounted for in the generation of the stiffness matrix. The rotational degrees of freedom of the mass segment are removed using a static condensation procedure. The rock foundation underneath the chimney is assumed to be rigid with no structure-foundation interaction effects.

A standard dynamic analysis was performed on the system's equations of motion. This yields the vibrational characteristics and the response of the chimney.

2.2.1 Number of mass segments used

The number of mass segments used in the study was dependent upon two criteria. First, the number of masses establishes the number of modes of vibration that could be determined by the eigen value analysis. If ten mass segments were used, then ten modes of vibration can be calculated. From these ten determined modes, only the first five were considered to be clearly defined. The remaining modes were neglected for two reasons. First, the fifth through tenth modes were considered to be influenced by the higher, undetermined modes. Secondly, modes higher than the fifth mode of vibration were shown to contribute very little to the chimney's overall dynamic response (Baumber, 1989).

The second criteria for determining the number of masses used was the degree of definition required in the flexural stress diagram. Better definition was obtained in this diagram when a greater number of mass segments were used in the analysis. This criteria proved to be the governing factor. Fifteen mass segments were therefore used in this study. This provided sufficient definition in the flexural stress diagram as well as being able to have sufficient accuracy in determining the first five modes of vibration.

2.2.2 Time history analysis

A time history analysis was used to calculate displacements and forces which are induced into the structure by the earthquake ground motion. The structure's equation of motion was solved using the Newmark-Beta numerical integration technique. The acceleration record's time step was interpolated linearly to match that of the integration scheme. The magnitude and time of occurrence of the maximum forces for each mode of vibration was determined in the analysis. From these forces, an envelope of maximum shear and flexural stresses can be obtained.

2.3 Earthquake ground motion

The chimneys analyzed in this study were subjected to two actual earthquake ground motion records. First, the Imperial Valley event was used as a typical intermediate a/v ratio ground motion record. The a/v ratio of an earthquake record is the ratio of the maximum ground acceleration, in g's, to the maximum ground velocity, in m/s. The second selected earthquake event was the Tangshan earthquake of July 28, 1976. No records for this event were recorded in the city of Tangshan itself.

However, the motions recorded in the city of Beijing were therefore attenuated back to Tangshan. The average of five attenuation relationships was used to arrive at the ground motion at the city of Tangshan. The form of the acceleration record for this attenuated ground motion event was assumed to be similar to that of the aftershock of August 9, 1976 recorded at Tangshan (Baumber, 1989).

2.4 Material properties

Limited research has been reported on the dynamic behaviour of clay brick masonry, in particular, the type used in existing masonry structures. The majority of the research has concentrated on the static properties (Van der Keyl, 1979; Omote, 1977). For the purposes of this study, the static properties were used. The clay brick units were assumed to have a compressive strength of 70 MPa. Type S mortar was assumed to have been used in the construction of the chimney. The assemblage's ultimate tensile and compressive strengths were taken as 1.0 MPa and 20 MPa, respectively. The elastic modulus of the masonry assemblage was 20,000 MPa. The shear strength of the material when no precompression is applied was assumed to be 0.67 MPa. The shear strength was assumed to increase linearly with the level of precompression. The coefficient of friction for the masonry was taken as 0.9.

The masonry was assumed to be completely elastic. No inelastic behaviour was considered. Failure of the masonry assemblage is defined as when the level of stress in the material exceeds the ultimate values that are stated above. In the absence of research into the energy dissipation properties of masonry, the damping was assumed to be five percent of critical. All modes of vibration have the same level of damping.

3 RESPONSE OF CHIMNEYS

In this section, the response of the existing chimneys is evaluated to determine the cause of the two observed failure types. The response of the chimney configurations is examined as each mode is introduced into the analysis separately. This will enable the participation of each mode in the overall response to be observed. The configurations were first subjected to the ground motions of the Imperial Valley event and then to the Tangshan event. The results are presented in the form of plots of the non-dimensional chimney height versus the normalized flexural stress. The non-dimensional chimney height is simply the location in question divided by the total height. The normalized flexural stress is the maximum tensile flexural stress at a section divided by the maximum tensile flexural stress occurring at the base of the structure.

3.1 Response to intermediate a/v ratio earthquake ground motion

When subjected to the Imperial Valley strong motion record, the fundamental mode of vibration was found to dominate the response of the chimneys. The higher modes do not significantly contribute to the structure's response. This is the case for all the chimney configurations investigated in this study.

The dominance of the response by the fundamental mode of vibration can be observed by examining the normalized flexural stress diagram as presented in figure 2. This figure shows the normalized flexural stress on the tension side for a chimney that has a height ratio of 20.0 and a diameter ratio of 2.5. The maximum normalized flexural stress occurs at a normalized chimney height of 0.2 from the chimney's base. This mode's response is shown in figure 2 by the curve marked by the plus signs.

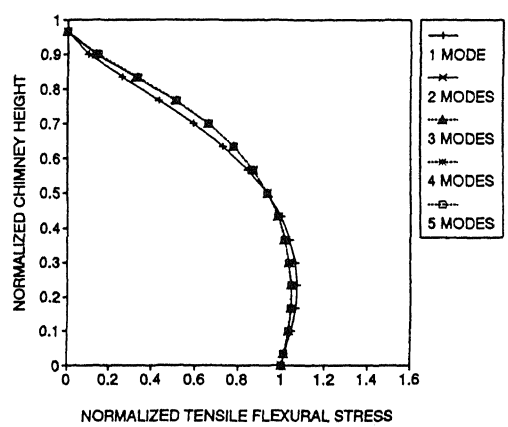

Figure 2. Response to Imperial Valley.

The response of the chimney does not significantly change when the effect of the second mode of vibration is included in the analysis. The normalized flexural stress changes in magnitude slightly but the shape of the diagram remains basically the same. This is shown in figure 2 as the curve marked by the crosses.

The second mode does not have a significant participation due to the nature of the earthquake ground motion. This event has high spectral accelerations in the period range of approximately

0.5 to 1.0 seconds. The chimney's fundamental period is 0.42 seconds. The spectral accelerations for this event decrease as the periods decrease. The structure's second mode typically has a lower spectral acceleration than the fundamental mode. The second mode has a period of vibration of 0.11 seconds. For other intermediate a/v ratio events, the fundamental mode also has a higher participation in the chimney's response than the higher modes (Baumber, 1989).

The addition of the third and higher modes of vibration does not significantly affect the response of the chimney to this type of ground motion. This can also been seen in figure 2. The shape of the normalized flexural stress diagram does not change noticeably when the higher modes are introduced into the analysis.

When subjected to the Imperial Valley strong motion record, the location of failure for all the chimney configurations studied was at or near the base of the structure.

3.2 Response to high a/v earthquake ground motion

The selected chimney configurations were analyzed when subjected to the Tangshan record with high a/v characteristics. In this case, the fundamental mode may no longer be the strongest participant in the response when the chimney configuration is tall and is more flexible. The higher modes of vibration significantly alter the response of the structure. This can be seen by examining the normalized flexural stress diagram (figure 3) of a chimney having a height ratio of 20.0 and a diameter ratio of 2.5. It can be seen in this figure that when the second mode of vibration is added to the response, the location of the maximum flexural stress is shifted to the top third of the chimney's height.

When the fundamental mode of vibration is the only mode considered in the analysis, the flexural stress diagram is of a similar shape to that for the intermediate a/v ratio cases. This is shown as the curve with the plus signs in figure 3. The shape of this diagram is significantly altered when the participation of the second mode is added. This is shown as the curve with the crosses in figure 3. Instead of one region of high flexural stress occurring, there are now two distinct regions of high stress. When the spectral accelerations of the fundamental and second modes are examined, it is found that the second mode has a large value. This therefore causes this mode to have a stronger participation than that of the fundamental mode.

Inclusion of the third mode only increases the location of the maximum flexural stress that occurs in the top third of the chimneys height. The fourth and fifth modes of vibration do not significantly alter the flexural stress diagram.

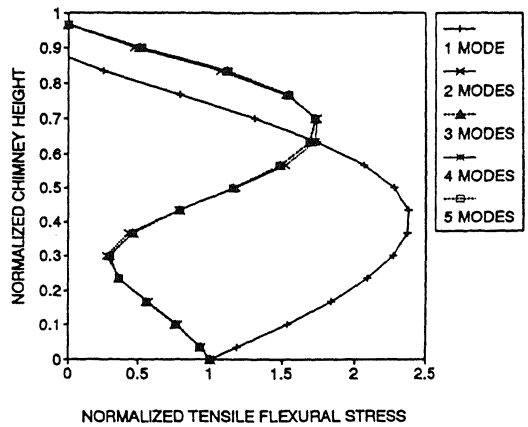

Figure 3. Response to Tangshan.

The location of failure for chimneys can occur at either of two points along the height when it is subjected to the Tangshan strong motion record. The chimney can fail at or near its base or in the top third of its height. The location of failure is dependent upon which mode of vibration is the strongest participant in the response. For chimneys that failed at or near its base, the fundamental mode of vibration was the strongest participant. These chimneys were typically short and had a low taper. This is the case when using the intermediate a/v ratio records.

Strong participation from the higher modes of vibration is likely for relatively flexible chimneys on rock when subjected to near field ground motion. Failure in these instances may occur at the top third of the chimney's height. This higher mode participation is not normally accounted for in simple code formulae or design requirements for wind loads.

4 POST-TENSIONING RETROFIT SYSTEM

There are several retrofit techniques used to strengthen industrial masonry chimneys. One practical method is to apply post-tensioning tendons to the exterior of the chimney. This will induce a compressive stress into the masonry which will effectively increase its tensile capacity. This technique will be examined in more detail.

4.1 Post-tensioning retrofit system

The post-tensioning retrofit system involves the application of a compressive force onto the chimney's cross sections. This force is applied to the chimney by placing vertical post-tensioning tendons along the height of the chimney (figure 1B). The compressive force created by the post-tensioning results in compressive stresses being created in the masonry. This additional compressive stress effectively increases the tensile capacity of the section. The tensile stresses induced by the earthquake ground motion must first overcome this initial stress before the masonry will experience tension. The post-tensioning tendons are assumed to have negligible shear strength. These tendons are applied to induce a compressive stress only and not to actively resist the lateral loads. The unloading of the tendons during flexure has been neglected.

4.2 Level of post-tensioning force

The amount of post-tensioning force that can be applied to the masonry chimney is governed by both the area of the chimney's cross sections and the compressive strength of the masonry assemblage. The permissible post tensioning stress is the difference between the allowable compressive stress of the masonry and the sum of the compressive loads experienced by the structure. These compressive loads are created by the axial load due to self weight and the compression induced by the actions of the earthquake ground motion. The critical section for the majority of the chimney configurations analyzed was the section at the very top of the chimney. Even though this section has no applied loads, the cross sectional area is small enough that it is the first section to crush when the post-tensioning force becomes too large. The post-tensioning force used the analysis is approximately 8000 kN.

4.3 Effect on vibrational characteristics

The post-tensioning force will alter the periods of vibration from the non-retrofitted case. The presence of the axial force will reduce the stiffness of the structure. This will in turn increase the periods of vibration. The post-tensioning force increased the periods of vibration by a maximum of two percent for all the configurations examined. This occurred for the fundamental mode. The higher modes were affected to an even lesser extent. The effect of the axial load on the overall stiffness of the chimney was therefore neglected in the dynamic analysis.

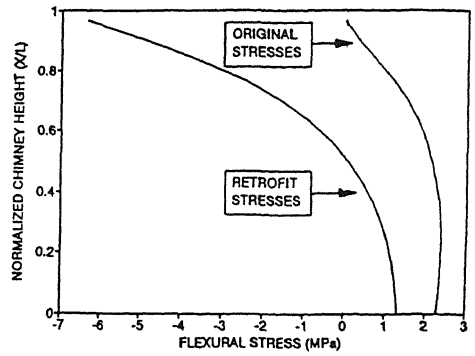

Figure 4. Retrofit stresses - Imperial Valley

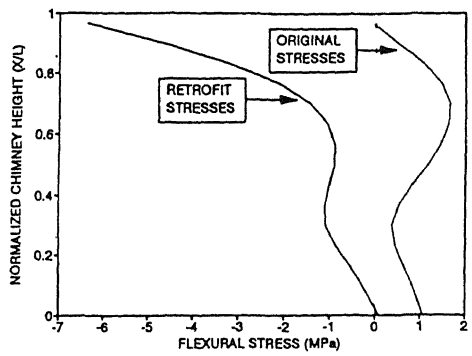

Figure 5. Retrofit stresses - Tangshan.

4.4 Response of retrofitted chimney

The stress distribution that occurs on the tension side of the cross section of a chimney which is subjected to the Imperial Valley ground motion is presented in figure 4. This chimney has a height ratio of 20.0 and a diameter ratio of 2.5. In this figure, tensile stresses are plotted as positive values. The retrofit technique is incapable of reducing the tensile strength experienced by the masonry to a point below the allowable limit. The allowable limit for tension normal to the bed joint for plain brick masonry is given as 0.25 MPa by the Canadian Masonry Code (CAN3-S304-M84, 1984). The tensile stress in the masonry is not even reduced to below that of the ultimate tensile strength of the masonry that was assumed in the previous section (1.0 MPa).

This technique was not found to be an adequate means of retrofitting existing chimneys whose response is dominated by the fundamental mode of vibration. Post-tensioning was found to be more effective for chimneys with a slight taper. This technique relies on the added compression that results from the application of a post tensioning

force. As the outer diameter increases, the cross sectional area also increases. This results in the compressive stress caused by the post-tensioning force to decrease towards the base of the structure. This severely limits the retrofit's ability to aid in resisting the earthquake induced lateral loads.

The stresses on the tension side of the cross section of the above chimney, when subjected to the Tangshan ground motion, are presented in figure 5. Tensile stresses are again plotted as positive values in this figure. Using this retrofit technique, it was possible to reduce the induced tension to a level which is below that prescribed by the Canadian Masonry Code. The large tensile stresses that occur at the top third of the height are successfully reduced. This is because there is sufficient compression created by the post tensioning force in this region. The base of the chimney still experiences a slight amount of tension but it is below the allowable value. The use of this technique enables the successful strengthening of chimneys that experience ground motion that results in the second mode to have the strongest participation.

As evident from the above discussion, this technique is adequate for strengthening some cases of industrial masonry chimneys. The intensity of the ground motion influences the effectiveness of this technique. The lower the induced moments that the chimney experiences, the lower the tensile stresses the post-tensioning force must overcome. In the above examples, the Tangshan event causes a bending moment at the base of 9.08 MN-m where the Imperial Valley event induced a moment of 23.93 MN-m. The chimney subjected to the Tangshan event obviously will have less tension to overcome and therefore this retrofit technique has a greater chance of being successful. In the case of retrofitted chimneys, tension cracking may not cause collapse due to the ductility provided by the steel. In this case, higher intensity ground motion can be tolerated if masonry crushing failure can be avoided.

5 CONCLUSIONS

The principle cause of failure of masonry chimneys in the top third of their height is the strong participation of the second mode of vibration in the dynamic response. On the other hand, the strong participation of the fundamental mode in the response is responsible for the failure of the chimney at or near its base. Chimneys which are subjected to high a/v ratio ground motion are especially vulnerable to the effects of second mode participation. These types of ground motion are typical of records on rock and stiff soils and near field events. There is a clear need for considering higher mode effects when designing masonry chimneys for seismic loads. These modes are not automatically considered by designers using simple code approaches and under wind load conditions.

The post-tensioning technique can be successful in retrofitting tall and flexible industrial masonry chimneys. In deciding whether or not to choose this technique, the geometry of the chimney must be considered. This retrofit system is most successful in strengthening chimneys that have a slight taper when failure at the base is anticipated. The post-tensioning retrofit technique is shown to be adequate for strengthening tapered chimneys that have a strong participation from the higher modes.

REFERENCES

Baumber, T.A. 1989. Seismic behaviour of tall industrial masonry chimneys. Master of Engineering Thesis. McMaster University, Hamilton, Ontario, Canada.

Canadian Masonry Code, CAN3-S304-M84, 1984. Canadian Standards Association, Rexdale, Ontario, Canada.

China Academy of Building Research. 1986. The mammoth Tangshan earthquake of 1976 building damage. China Academic Publishers. Beijing, PRC.

Liable, J.P. 1985. Structural analysis. CBS College Publishing, New York.

National Building Code of Canada, 1985. National Research Council, Ottawa, Ontario, Canada.

Omote, Y., Mayes, R.L., and R.W. Clough 1977. A literature survey - transverse strength of masonry walls. Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley. EERC 77-07.

Shu-quan, Z. 1981. Repair of damaged structures in tangshan city. Final Proceedings: FRC-US Joint Workshop on Earthquake Disaster Mitigation through Architecture, Urban Planning and Engineering; Beijing, November 2-6: 396-409.

Van der Keyl, R. 1979. Behaviourial characteristics of brick masonry. Master of Engineering Thesis. McMaster University, Hamilton, Ontario, Canada.