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Retrofit techniques for masonry chimneys in seismic zones

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ABSTRACT

Unreinforced masonry chimneys sustained severe damage during recent earthquakes. The failure or collapse of such structures during a seismic event may result in the loss of industrial production as well as the loss of human life. It is now necessary to retrofit existing masonry chimneys as many were built either according to older codes or before current seismic codes were in place. It was found that the use of a reinforcing steel cage is the most effective retrofit system. This system can be designed and adapted to resist high levels of lateral inertia forces. The post-tensioning retrofit system was found not to be a reliable technique. This system worked adequately for short, slightly tapered chimneys but not for tall, highly tapered chimneys.

INTRODUCTION

Unreinforced masonry chimneys sustained severe damage during recent earthquakes. The susceptibility of masonry chimneys to severe damage was dramatically observed during the 1976 Tangshan (China) earthquake event (China Academy of Building Research, 1986). The failure or collapse of such structures during earthquakes may result in the loss of industrial production as well as the loss of human life. Many of the existing masonry chimneys were built according to older codes and long before the current seismic code provisions were available. Damage to chimneys was observed both at the base and at the top third of the height. The causes of these two types of failures were investigated in a recent study by Baumber and Ghobarah (1990). Now it is necessary to retrofit existing masonry chimneys to improve their seismic performance. The objective of this study is to evaluate retrofitting systems and investigate their effect on the seismic response of masonry chimneys.

A retrofit technique that was used in China involved the application of steel angles to the exterior of the chimney. Niu (1981) developed analytical expressions for the strength of the retrofitted structure. In his analysis, Niu smeared the reinforcing steel into a shell around the exterior of the chimney. Shu-quan (1981) also used this technique but did not present any details concerning the theoretical capacity.

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The two most common retrofit methods were investigated in this study. The first method involves the addition of a reinforcing steel cage to the chimney's exterior. The reinforcing steel will resist the tensile loads once the applied stress exceeds the tensile strength of the unreinforced masonry. The second retrofit technique involves the addition of a post-tensioning stress into the structure. The post-tensioning will induce initial compression into the masonry. This will allow the chimney to resist higher moments and thus the lateral load carrying capacity of the chimney will be effectively increased. Sketches of the two retrofit systems are presented in Fig. 1.

REINFORCING STEEL CAGE RETROFIT SYSTEM

The retrofit system involves the attachment of equally spaced bundles of reinforcing steel (e.g. 32 bundles). The large number of bar bundles is intended to spread the reinforcement as uniformly as possible around the circumference of the chimney. These bundles are fastened to the chimney by welding them to steel hoops which are attached to the chimney. The area of each bundle is dependent on the bending moment that must be resisted by the structure. In this system, the steel cage and chimney work together to resist the earthquake induced loads.

A dynamic analysis was performed utilizing a lumped mass approach. The masses of the masonry and steel were lumped at the mid-point of the mass segment. The analysis is carried out as outlined by Baumber and Ghobarah (1990).

Stress Distribution

Unreinforced masonry behaves in a brittle manner when subjected to earthquake loads. To avoid failure, unreinforced masonry must remain elastic. Any excursion beyond the elastic range will result in cracking of the masonry assemblage and thus causes failure. The reinforcing steel cage resists the tensile loads. Once the applied loads reach levels that exceed the tensile strength of the masonry, the masonry cracks and the steel then begins to resist all the tensile loads.

The stress distribution upon cracking of the masonry is assumed to be linear across the section. The maximum allowable design stress on the compression side is 0.32 f'm, where f'm is the ultimate compressive stress of the masonry assemblage. This design level is given by the Canadian Masonry Code, CAN3-S304-M84, 1984. The masonry is assumed to carry no tensile load. The stress in the steel is converted to an equivalent stress in the masonry by dividing by the ratio of the elastic moduli of steel to masonry.

Theoretical Strength

The theoretical strength can be determined from the known section and material properties. The steel is assumed to yield in tension just as the masonry reaches its' compressive capacity. The location of the neutral axis and the forces in both the steel and the masonry can then be determined for this balanced condition. For equilibrium, the sum of these forces must equal the applied axial load. In the case of chimneys, the applied axial load arises from both the self weight of the chimney and from forces induced by the vertical component of the earthquake ground motions. Equilibrium conditions are determined by a trial and error procedure. The force balance is adjusted by altering the area of steel required.

Periods of Vibration

Several chimneys of various practical dimensions were analyzed. The chimney configurations are listed in Table 1. The vibrational characteristics of the structure are altered when the mass and stiffness of the reinforcing steel cage is included in the dynamic analysis. Typically the retrofit increases the moment of inertia at the base of the structure by approximately ten percent. The mass of the base element is typically increased by approximately five percent. When the section is heavily reinforced, the increase in the moment of inertia of the base and in the mass of the base element can be as large as twenty five percent.

For small amounts of reinforcing steel, the effect of the reinforcing cage can be neglected in the analysis of the structure. As the size of the reinforcing cage increases, the effect on the vibrational periods increases. The periods of three chimney configurations with and without a reinforcing cage are presented in Table 2. The large change in the magnitude of the periods of some of the taller chimneys warrants an analysis which takes into account the effect of the stiffness and mass of the reinforcing cage.

Effectiveness of Technique

The effectiveness of this technique is best illustrated through the use of an example. A chimney with a height ratio of 20 and a diameter ratio of 3.0 was selected (configuration 13). This chimney's fundamental mode was the strongest participant in the response when subjected to intermediate a/v ratio ground motion records. When the chimney was subjected to high a/v ratio ground motion records, the second mode was predominant.

The critical section in the design of the reinforcing steel is that at the base of the chimney. This is the location of the maximum induced bending moment. This section is critical when the structure is subjected to both intermediate and high a/v ratio events. Previous work (Baumber and Ghobarah, 1990) showed that for high a/v ratio events the maximum bending stress occurred at a section at the top third of the chimney's height. The bending stress at this section is at a maximum even though the bending moment is not. This is a result of the taper of the chimney.

The reinforcing steel designed to resist the base moment is continued along the entire height of the chimney for convenience in the analysis. Ideally, the reinforcement should be decreased with height as the earthquake loads imposed on the structure decrease with height. This will provide a design that is economical as well as capable of resisting the forces induced by the earthquake ground motions.

When the resistance offered by the retrofit system is compared to the bending moments experienced by the original geometry, the system was found to perform adequately. The induced bending moments are always less than the resistance offered by the retrofitted section. This comparison is presented in Fig. 2.

This retrofit system is an adequate means of strengthening masonry chimneys. The system is effective for chimneys of small height ratios. These chimneys experience relatively low earthquake induced bending moments. A small amount of steel is required for the chimney to be able to resist the applied loads. The moment capacity and maximum applied bending moments for four chimney configurations are presented in Table 3.

POST TENSIONING RETROFIT SYSTEM

This retrofit technique involves the application of a compressive force onto the existing structure. This force is applied to the chimney by post-tensioning tendons placed parallel to the height. These tendons are anchored at both the foundation and the top of the chimney to a steel hoop. The tendons are not bonded to the exterior of the chimney and thus only provide a means to apply a compressive force.

Post-Tension Force

The magnitude of the post-tensioning force is limited by the compressive capacity of the smallest cross section at the top of the chimney. This section has no earthquake applied moment and therefore all the compression on the section is due to the post-tensioning force. Other cross sections will have earthquake induced bending stresses. The post-tensioning stress at these locations will be lower than at the top of the structure. The post-tensioning force is constant, however, for tapered chimneys, the cross-sectional area increases thus decreasing the applied stress.

In the design of the post tension force, the compressive stress at any section should not exceed the allowable given by the Canadian Masonry Standard. The limiting allowable post tension force for the majority of the chimney configurations used was approximately 8000 kN. For some of the configurations, this allowable force is slightly smaller.

Periods of Vibration

When the chimney is post tensioned, there is an axial force acting on the structure. This axial force will result in a reduction in the structure's stiffness. When the chimneys were analyzed taking the effect of the axial force into consideration, an increase in the magnitude of the periods of vibration did occur. The periods for three cases are presented in Table 4. The largest increase was for the fundamental period of the chimney configuration 16. The increase when compared to the period of the same configuration without the effects of the axial force considered was 1.68 percent. The higher modes were effected to an even lesser degree. This increase was deemed to be insignificant.

Effectiveness of the Technique

This retrofit technique was found to be adequate for structures that have a fundamental mode which is the strongest participant in the response. Short chimneys with a slight taper benefit the most from this retrofit system. The area of the base is small enough that the limiting allowable post tension force can still create a large compressive stress relative to the magnitude of the induced bending stress. This compressive stress will be able to reduce the created tension to a level which is below the masonry's allowable limit. The stress distribution for chimney configuration 13 subjected to the ground motions of the Imperial Valley record is presented in Fig. 3. The bending stress shown is the combination of the compressive stress which arises from the post-tensioning force and the stresses induced by the earthquake ground motions. As the height ratio increases, the effectiveness of this technique diminishes.

For chimneys with a second mode which participates strongly in the response, this retrofit system was also found to be effective. The stress distribution of chimney configuration 13 when subjected to the Tangshan ground motion is also presented in Fig. 3. The post-tensioning force is large enough that it was able to create significant compression stresses in the cross section. This compression again allowed the tension stress experienced by the cross section to be below the allowable limit.

SUMMARY

Observed failures of unreinforced masonry chimneys during earthquakes clearly show the need for retrofitting existing chimneys. The attachment of a reinforcing steel cage to the exterior of the chimney is an effective retrofit technique. This technique is able to resist any induced forces from any type of earthquake ground motion when an adequate amount of steel is utilized. Dynamic analysis of the structure including the steel cage is usually not required. The stiffness increase due to the presence of the cage does not significantly effect the periods of vibration. Also, this technique can be implemented both readily and at a reasonable cost.

The post tensioning system is not as an effective retrofit technique. The application of the technique is limited by the fact that the force induced by the post tensioning is dependant on the area of the smallest cross section of the chimney and the ultimate compressive strength of the material. This technique is adequate for short, slightly tapered chimneys. As the taper increases, the post tensioning stress at the base is small enough to significantly reduce the tensile stress.

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Table 1. Chimney configurations

CONFIGURATION DESIGNATION	DIAMETER RATIO BOTTOM OD* / TOP OD	HEIGHT RATIO HEIGHT / TOP OD
1	1.5	10
13	3.0	20
16	3.0	25
17	3.5	25

* O.D. refers to outside diameter.

Table 2. Periods of vibration

CASE	MODE	RETROFIT (SEC)	NO RETROFIT (SEC)	% CHANGE
01	FIRST SECOND THIRD	0.18713 0.04014 0.01562	0.19413 0.04284 0.01685	3.61 6.30 7.27
16	FIRST SECOND THIRD	0.51351 0.13365 0.05530	0.52201 0.14928 0.06401	1.63 10.47 13.61
17	FIRST SECOND THIRD	0.43440 0.12421 0.05313	0.43593 0.13056 0.05697	0.35 4.86 6.72

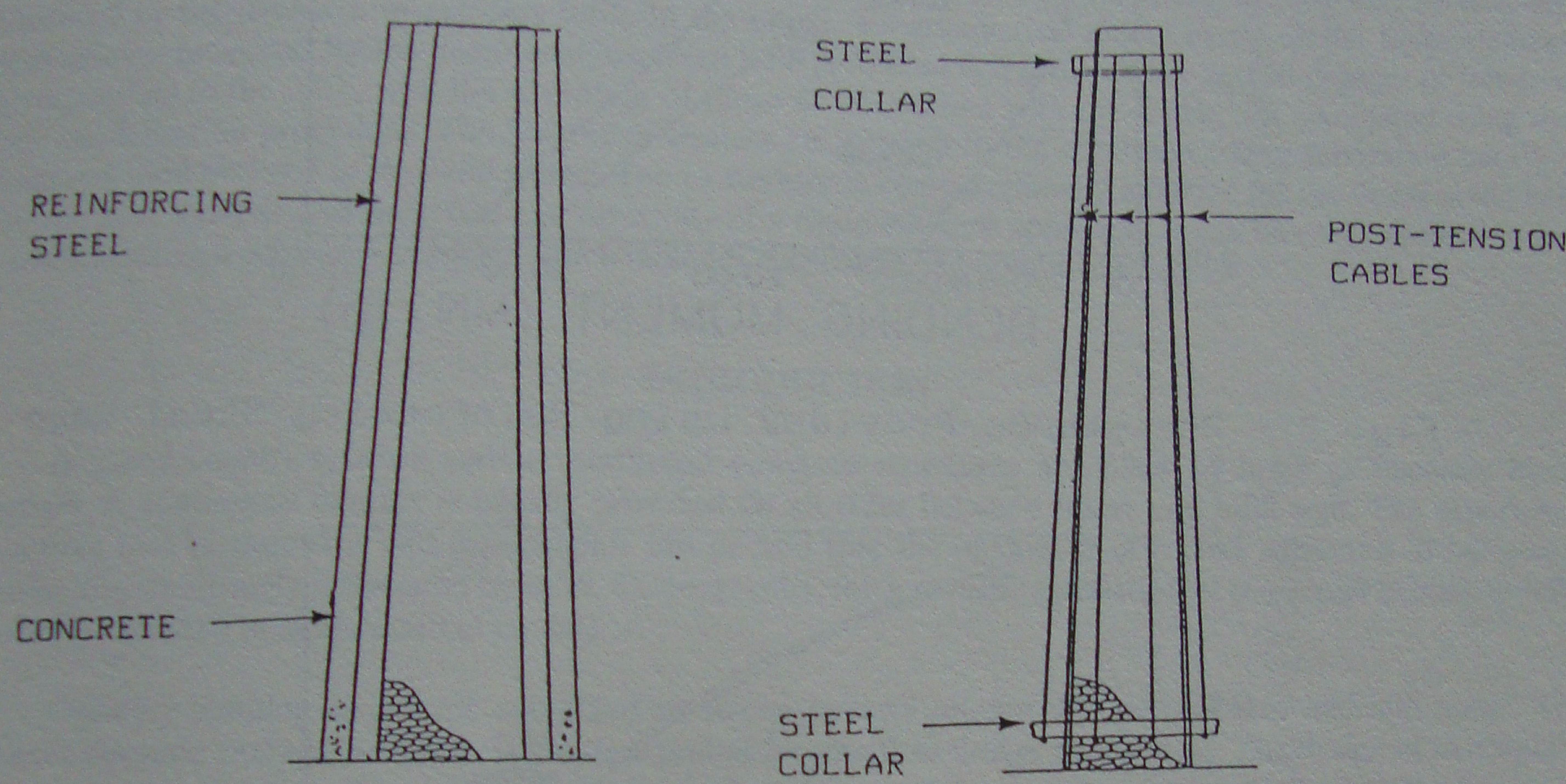
Table 3. Resistance provided by reinforcing cage

CASE	STEEL AREA * (mm sq.)	RESISTANCE (kN m)	INDUCED MOMENT (kN m)
01	1400	8555	7544
13	1500	32638	29426
16	6000	63167	60627
17	2500	55395	52903

* Total steel area of one bundle, chimney reinforced by 32 such bundles

Table 4. Post tension vibrational period

CASE	MODE	RETROFIT (SEC)	NO RETROFIT (SEC)	% CHANGE
01	FIRST SECOND THIRD	0.19679 0.04300 0.01687	0.19413 0.04284 0.01685	1.37 0.39 0.14
16	FIRST SECOND THIRD	0.53080 0.15046 0.06421	0.52201 0.14928 0.06401	1.68 0.80 0.32
17	FIRST SECOND THIRD	0.44096 0.13136 0.05712	0.43593 0.13056 0.05697	1.15 0.62 0.27



(a) Reinforcing Steel Cage (b) Post-Tensioning

Fig. 1 - Sketches of Retrofit Techniques

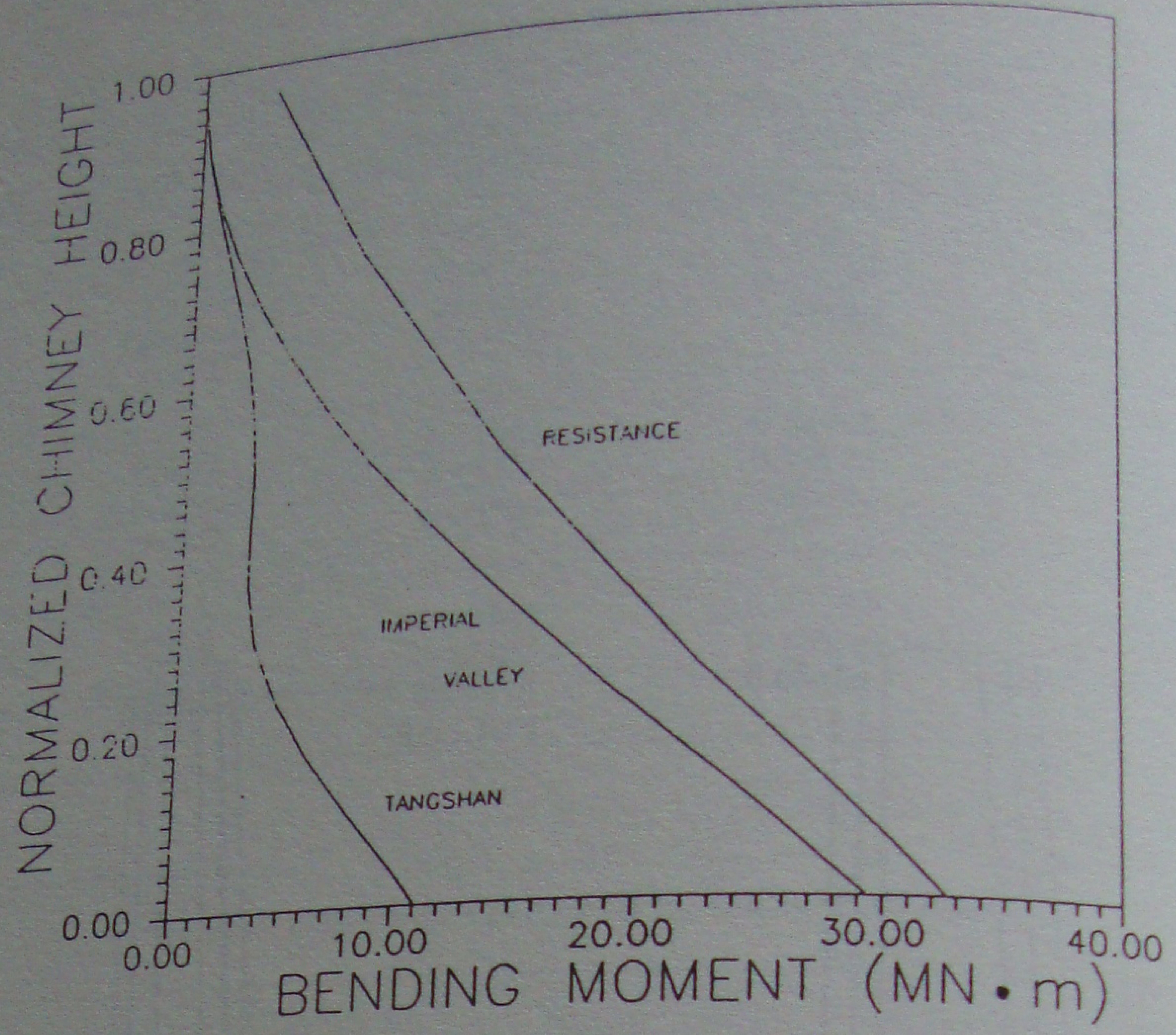


Fig. 2 - Resistance Provided using Reinforcing Steel Cage

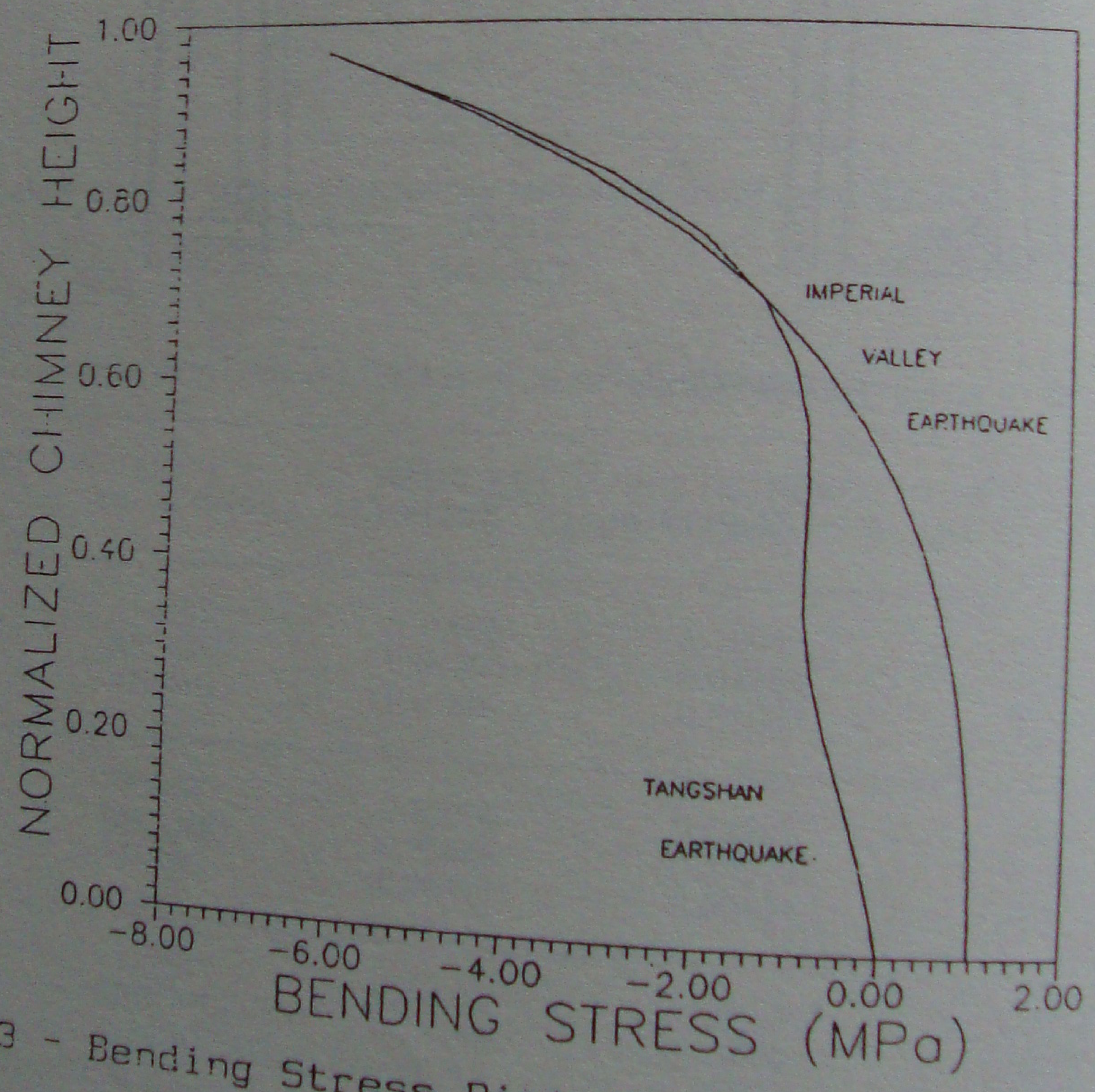


Fig. 3 - Bending Stress Distribution on Tension Side,

Post-Tension Technique