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Seismic Adequacy of Old Stone Masonry Buildings

Résistance aux séismes d'anciens bâtiments en pierres de taille

Seismische Widerstandsfähigkeit alter Mauerwerksbauten

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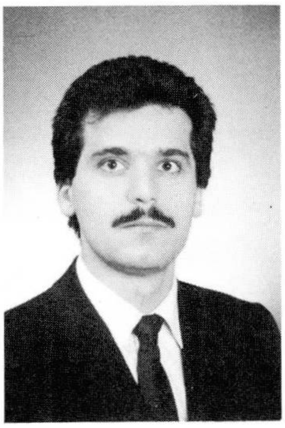

S.E. Chidiac, born 1961, obtrained his PhD degree in civil engineering and engineering mechanics from McMaster Univ. Hamilton, Ontario. For the last two years, he has been involved in investigations of seismic adequacy of stone unreinforced masonry subjected to moderate earthquakes.

SUMMARY

This paper examines different approaches available to determine the seismic resistance of unreinforced stone masonry buildings. First, the requirements of the National Building Code of Canada for unreinforced masonry walls are examined, then a method developed in California is briefly explored with respect to this type of construction. Both simple and sophisticated analytical models that have been proposed in the literature for the evaluation of in-plane and out-of-plane dynamic behaviour of stone masonry are reviewed.

RÉSUMÉ

L'article examine différentes méthodes pour la détermination de la résistance aux tremblements de terre de bâtiments en pierres de taille. Les exigences du code de construction canadien pour les murs en maçonnerie sont examinés, de même qu'une méthode développée en Californie est analysée pour ce type de construction. Les modèles analytiques simples et compliqués proposés dans la littérature pour l'estimation du comportement dynamique de maçonneries en pierres de taille, plan et dans l'espace, sont passés en revue.

ZUSAMMENFASSUNG

Der Bericht prüft verschiedene verfügbare Methoden zum Bestimmen der seismischen Widerstandsfähigkeit von unbewehrten Mauerwerksbauten. Als erstes werden die für unbewehrte Mauerwerkswände bestehenden Bauvorschriften Kanadas geprüft, dann wird eine in Kalifornien entwickelte Methode hinsichtlich derartiger Konstruktionen kurz vorgestellt. Sowohl einfache als auch anspruchsvolle analytische Modelle, wie in der Literatur für die Ermittlung des dynamischen Verhaltens von Steinmauerwerk innerhalb der Scheibenebene und quer zu ihr vorgeschlagen, werden begutachtet.

1. BACKGROUND

Many of today's historic monuments were built to express national or local pride and as a result, good workmanship and high material quality were used during their construction. Some old buildings, not intended as monuments, have also withstood the challenge of time. These structures have therefore, by their survival, demonstrated that they had the needed capacity to support all the loads, which also sometimes included earthquakes. However, natural deterioration coupled with limited maintenance may not guarantee continuing future stability, and thus a proper assessment is warranted.

In North America, a large number of historic buildings were constructed with three basic components: lime mortar, natural stone units and clay bricks. These old structures are labeled unreinforced masonry (URM) and were constructed in the absence of mandatory seismic requirements. Their poor performance under seismic activities, however, in the last decades, is a testimony to their vulnerability [1-3]. To address this problem, a multiyear project was sponsored by the National Science Foundation in the USA to review the unreinforced masonry construction and develop a methodology to mitigate the seismic hazards inherent to these buildings. This project was carried out by a joint venture of specialists and is known as the ABK [4].

Although the ABK study was comprehensive, the scope of the project did not extend to cover thick masonry and walls constructed with natural stone units. This is understandable given the limited number of buildings that fall in this category. In eastern Canada, however we have some historic monuments that were built with natural stone and lime mortar and in view of the projected seismic risk, an assessment of their present safety is felt necessary. This gave life to a study of the seismic vulnerability of these structures, is expected to take three years to complete, and is made up of various steps. For a specific project, a tower was selected and is shown in Fig. 1. In this paper, only that part of the investigation is given which briefly examines the relevant mathematical models reported in the literature. Also the applicability of both the ABK methodology and the national building code of Canada to stone masonry buildings are discussed.

2. STANDARDS & GUIDELINES

2.1 Design Code Requirements

Modern codes are intended as requirements for new construction and should not be applied to historical monuments [5]. This statement can easily be substantiated by examining the National Building Code of Canada, NBCC 1990, which requires that the building be designed according to a minimum seismic base shear force, V given by

$$V=\left(V_e/R\right)U \qquad \text{and} \qquad V_e=\upsilon S I F W \tag{1}$$

in which R is the force reduction factor, equal to 1.0 for unreinforced masonry and U the calibration factor, equal to 0.6. V_e is the elastic base shear, υ is the zonal velocity ratio, S the seismic response factor, I the importance factor, F the foundation factor, and W the weight of all the reactive masses that induce inertia forces in the structure. The code also requires a minimum reinforcement for all members that are either load bearing or that resist lateral loads in moderate seismic zones, although for small buildings the NBC Part 9 allows some walls in seismic zones without reinforcement. It is however clear that many stone masonry walls do not comply with the current seismic standards. Further, the Canadian design standard for masonry, CSA 1984 [6], requires that all structural members be designed according to either the elementary principles of elastic mechanics of materials coupled with a semi-empirical relationship to account for load eccentricity effects, or to empirical rules relying on the compressive stresses coupled with a limit on the wall slenderness ratio.

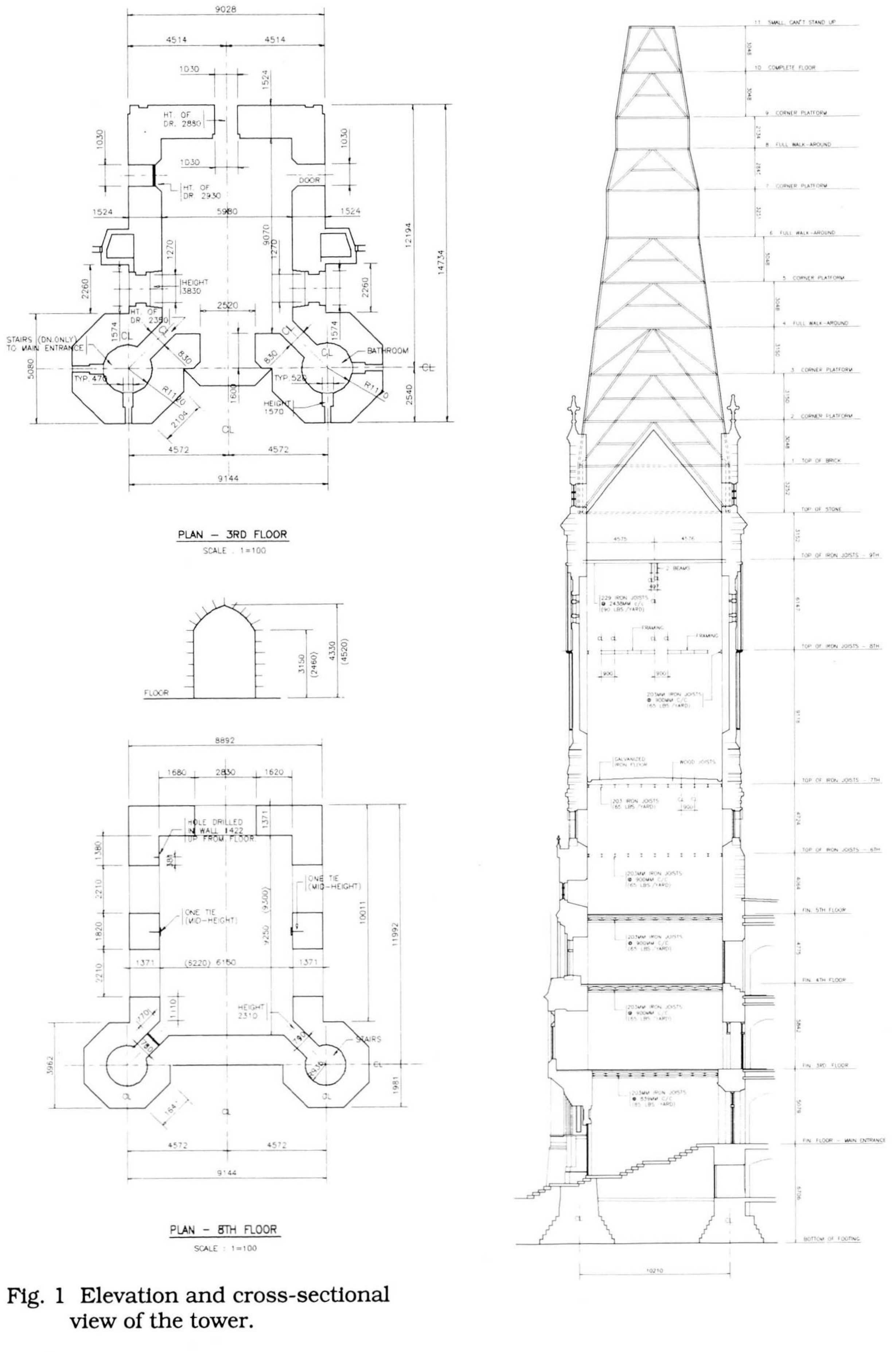

Fig. 1 Elevation and cross-sectional view of the tower.

2.2 The ABK Methodology

The objective of the ABK-Joint venture of three Southern California engineering consultant firms was to develop a methodology to mitigate the seismic hazards associated with URM buildings [7]. Their task included a survey of URM buildings, an experimental testing program and development of a procedure that structural engineers can follow to investigate the adequacy of an existing URM building. The detailed description and review of all aspects and steps of the proposed methodology falls beyond the scope of this paper.

The methodology was primarily focused on the common features noticed during the survey of URM buildings. Wooden floors and roofs were found in low rise commercial buildings, and cast-in-place concrete floors in important and large multi-storey structures. URM buildings through out the USA were found to be consistently not designed to resist seismic loads, but at the same time URM buildings have survived earthquakes of varying intensity and sometimes in spite of major structural deficiencies. Accordingly, static and dynamic testing of floor diaphragms and URM walls for out-of-plane motion were conducted with emphasis on the non-linear hysteretic behaviour. The walls examined are primarily 3 wythe common brick, clay block and concrete blocks of height/thickness ratio ranging from 14 to 25. After a comprehensive investigation, ABK recommended that limits are required for out-of-plane dynamic stability and for the demand/capacity ratios for diaphragms. However these limits were derived for regions of high seismic risks or for places where the soil condition will lead to ground amplification. In their study, ABK made two major assumptions; i) the in-plane response of a URM is infinitely rigid and, ii) the ground motion is directly transmitted unmodified to each floor by the end walls parallel to the direction of earthquake.

The ABK methodology made a major contribution for assessing seismic adequacy of URM. Most of their recommendations have been incorporated in new guidelines to assist engineers in the assessment of seismic adequacy [8,9]. However the method is not suitable when investigating stone masonry walls with the height/thickness ratio ranging between 2 to 8 are involved (as is the case shown in Fig. 1) and for regions where structures are subjected to a seismic activity of moderate intensity, as is the case in the eastern part of North America.

3. ANALYTICAL METHODS

3.1 General Considerations

The use of any numerical simulation of a historic structure must be founded on detailed preliminary studies designed to provide realistic data on the construction and the behaviour of the building. The historical knowledge of how it was constructed and why certain features are present are often missing. Once such a preliminary survey has been completed, the information can be used in analytical models and thus identify the weak points in the overall structure.

A sophisticated mathematical model can give very detailed information about the structural behaviour, while a coarser model can give information which may suffice in given circumstances. A fundamental difference between a coarse and a detailed model lies not only in the amount and quality of data needed to describe the structure but also in the interpretation of the results. Both input to, and output from a coarse model is usually simple and the interpretation of the results can be integrated into a design procedure in a clear and complete manner. Detailed models, on the other hand, require specific experimental studies but are usually more representative of the structure at hand.

3.2 Simple Models

Out-of-plane and in-plane simple models have been developed in the past to assist engineers in their assessment of structural safety. The out-of plane models are mostly developed on the basis of static stability conditions. Priestley [10] has modified the ABK recommendations and proposed a model for clay brick masonry based on energy considerations. For in-plane modelling, there are two approaches: either it is assumed that the pier is solid and that the spandrel beam will crack as given in Reference [11] or that the spandrel beam is solid and that the pier will crack under lateral load as given in References [12]. These models are conservative and examine only one part of the structure.

3.3 Material Constitutive Models

The behaviour of masonry while subjected to dynamic excitation has proven to be very complex due the heterogeneous nature of the mortar and brick, and the layering patterns. To overcome this difficulty, researchers and engineers have in the past assumed and still treat masonry as a linear, elastic, homogeneous and isotropic material. However, others have decided to take on the challenge of treating this complex material in a realistic manner and a brief summary of their findings follows.

The first attempt to model the behaviour of masonry consisted of developing a mathematical model that can simulate both the brick and mortar by a continuum mechanics approach and by the mathematical theory of mixture [13]. The model was shown to properly depict the linear elastic dynamic behaviour of URM walls. However, it was recognized that it is costly and a substructuring method was then proposed by Ali and Page [14]. Though substructuring did reduce the scale of the problem it was still not feasible to analyze a complete structure using such a technique.

The attempt to model the three-dimensional nonlinear behaviour of unreinforced masonry was first proposed in Reference [15] and is based on the idea that the nonlinear effects can be represented using the linear equivalent method, LEM, which was previously used in nonlinear soil-structure interaction. Mengi et al. [16] have assumed that the behaviour depends on two parameters, namely the shear modulus G, and its viscous counterpart G'. Based on experimental studies, the variations of G and G' with the shear strain can be described by bilinear and trilinear functions, respectively. In a more recent study, Tanrikulu et al. [17] have recognized that the bilinear and trilinear functions are only applicable to burned-clay bricks and that the suggested parameter $c=\tau_c/\bar{\tau}$, where τ is the shear stress, should be part of the constitutive model. This has led to a more general relation between G and the shear strain γ:

$$G=\begin{cases}\bar{G} & \text{for}\,|\gamma|\leq\bar{\gamma}\\ f\left(\bar{G},|\gamma|,\bar{\gamma},\gamma_c,c\right) & \text{for}\,|\gamma|>\bar{\gamma}\end{cases} \tag{2}$$

where the bar over a symbol denotes effective. A damage criterion similar to Reference [15] was also proposed for monitoring the amount of damage in the wall. Although the model is not hysteretic, during an earthquake excitation Tanrikulu et al.'s model predicted hysteresis loops similar to those observed experimentally. This model was shown to capture the dynamic behaviour of both clay brick and stone masonry.

The criterion by Benedetti and Benzoni [18] appears to be one of the few that have proposed a material model that is hysteretic in nature. The material model is based on the static shear tests carried out on single piers, requires seven parameters, and is constructed by superimposing three bilinear hysteretic shear sub-elements failing in a brittle manner at a prescribed strain intensity. Each sub-element is defined by its own mechanical properties, G, τ, γ and is limited by γ_c. The resulting shear panel hysteretic curve is obtained by direct summation of all individual sub-elements; the ones that have exceeded the limits, γ_c are excluded. This model was derived from tests on stone masonry and agreement with experimental results is very promising.

3.4 Modelling of Stone Masonry

A large number of proposed analytical models are found in the literature of which only a representative few are selected here. Modena [5] stated that the sophisticated analytical and numerical models, which require the knowledge of relevant material and structural properties, can be inadequate for interpreting the actual behaviour of such complex and special masonry structures. He went on to say that the process of properly analyzing these monuments is iterative and is accomplished by upgrading a simple model to a level that can simulate structural damage that has been historically recorded. This view is echoed by some other researchers who are attempting to model the behaviour of stone masonry.

In an attempt to check the safety of an ancient church located in a seismic area, Vestroni et al [19] acknowledge that a model that correctly takes into account the detailed aspects of nonlinear dynamic behaviour of the monument is practically out of the question, especially since the monument has a spatially distributed structure which, even if discretized, requires a large number of variables. Furthermore, the material is quite complicated and strongly anisotropic, and the incomplete knowledge of the structural integrity makes the analytical description more uncertain. In view of the foregoing, two models of different complexity were employed. The first was a linear 3D finite element model used to develop a detailed stress analysis of the structure as a tool to identify the weak components. The second model was a nonlinear single-degree-of-freedom oscillator that was used to study the nonlinear response of a substructure.

Turnsek et al. [20] provided some insight on the seismic resistance of stone masonry by investigating the damage caused by the medium-strength Slovenia earthquake of 1974 and the two-successive strong earthquakes which hit the Friuli region in Italy and the Soca valley region in Slovenia in May and September 1976. A model that computes the shear strength of the masonry building was developed based on the assumptions that i) for each individual wall a shear strength ductility diagram can be constructed on the basis of characteristics obtained from laboratory tests; ii) displacement due to torsional rotation about the enter of rigidity can be superimposed; and iii) failure of individual walls occurs when a limiting ductility is reached. The model was verified by comparing the computed results with an actual two-storey building that was subjected to two different earthquakes. The results obtained from this model are in the form of base shear coefficients. The computed values appear to be in agreement with the ground acceleration values from the MSK-64 scale.

3.5 Comparative Study of Analytical Models

Karantoni et al. [21] have used the 1986 Kalamata earthquake in Greece to compare a finite element modeling, a three-dimensional space-frame model and an approximate hand-calculation method for performing seismic response analysis of unreinforced stone masonry structures. The objective of these analyses was to assess the ability to predict both the location and severity of seismic damage for a given seismic excitation. Damage and failure are taken in the form of and originate from macro-cracking, which for stone masonry is taken to occur when the magnitude of the principal stress exceeds the strength of masonry in uniaxial tension.

For the three buildings investigated, Karantoni et al. found from eigenvalue/eigenmode analysis that the first significant mode of vibration consists solely of bending deformations with a natural frequency of 4 Hz. The in-plane displacement was almost nil. The second mode was found to have a natural frequency of 7.5 Hz and the deformation of two opposite exterior walls was mainly in-plane, with some bending of the other two walls. The authors stated that the bending and the in-plane action of the walls are generated by different and widely separated modes of vibration, and for the frequency content of their earthquake, bending was the dominant form of deformation.

The analysis performed was linear elastic and the equivalent static load was uniformly distributed along the height according to the distribution of mass. This approach was chosen in order to include the hand calculation method as one of the alternative techniques. It was concluded that the finite element analysis, which accounts for both in-plane and out-plane behaviour, was the only one to capture the key behaviour of the walls. The space frame idealization with rigid joints yielded a behaviour not observed in reality and gave an overestimate of principal tensile stresses. The hand-calculation approach, a shear beam model, accounts only for in-plane action and produced results that are lower and have an incorrect distribution of damage. In their final conclusion, Karantoni et al. noted that the finite element analysis revealed that the out-of-plane bending in walls is responsible for most of the observed damage and that this dominant behaviour is often overlooked in design.

4. SUMMARY

As has been briefly presented in this paper, different approaches are available to assess the seismic adequacy of old stone URM structures. It needs be recognized that such monuments are "unique" structures and that modern codes do not apply; therefore special requirements need to be developed.

Simple models have traditionally been the tools used by engineers to study behaviour and to check the safety of URM structures. However, it was found that these simple methods do not always apply and are not necessarily conservative. It should be recognized that the dynamic behaviour of stone URM is complex and nonlinear, and therefore an investigation of the safety of these monuments requires more sophisticated approaches. Consequently, more sophisticated models that employ the finite element method are increasingly used for the analysis. However, the usefulness of this approach is so far limited by the practical difficulty of adequately characterizing the material behaviour and of properly assessing the structural integrity. Further work toward realizing the potential of this method needs to be undertaken.

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