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THREE-DIMENSIONAL DYNAMIC ANALYSIS OF 18 STOREY STEEL-MASONRY COMPOSITE HERITAGE BUILDING

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ABSTRACT

The three-dimensional dynamic analysis of a 18 storey steel-masonry composite downtown heritage building is described. This analysis is used as an example to discuss numerical modelling of masonry and to demonstrate how the analysis and design of complex structures can be automated. It is emphasized that the spatial distribution of the stiffness and mass of all masonry panels must be included in the dynamic analysis model, unless they are separated from the frame by adequate clearance.

INTRODUCTION

Many existing older buildings are unreinforced masonry structures designed with little or no consideration for seismic requirements. The seismic hazards posed by such structures have been extensively documented (Bruneau 1994). Structural engineers have sometimes adopted unduly conservative analytical models in their seismic evaluations of unreinfoced masonry structures which have translated into expensive rehabilitation cost.

Yolles was engaged recently to evaluate the seismic performance of the exterior masonry walls in a 18 storey building constructed in 1930 and located in downtown Toronto. The building structure will be briefly described below. Then the numerical modelling of masonry

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will be discussed, followed by the automated analysis and design processes. Finally, conclusions will be presented at the end of this paper.

BUILDING STRUCTURE

The total length of this building in the north-south (X) direction is about 90 m (300 ft.) and the widths in the east-west (Y) direction vary between 18m (60 ft.) and 30 m (100 ft.). The total building height is about 84 m (275 ft.) with major set-backs at the 9th and 12th floor levels. A tower starts at the 13th floor. Each floor of the tower gradually reduces in plan dimensions. The storey heights range from 2.9 m (9.5 ft.) to 7.4 m (24 ft.).

According to the "Guidelines for Seismic Evaluation of Existing Building" (IRC 1993), the structural system of this building in the Y-direction is a steel moment frame and in the X-direction a steel frame with infill shear walls. All of the infills are solid unreinforced masonry walls consisting of exterior limestone with a clay brick backup. The exterior flanges of the exterior columns are embedded in the masonry walls. Most of the masonry infill panels contain a window measuring 1.4 m x 2.8 m (4.5 ft. x 9 ft.). Masonry wall thickness ranges from 0.6 m (2 ft.) at the ground floor to 0.3 m (1 ft.) at the 18th floor.

The yield strength of the structural steel sections is 207 MPa (30 ksi). The limestone imported from Indiana, USA, has compressive and flexural strengths of 36.6 MPa (5.30 ksi) and 8.14 MPa (1.18 ksi), respectively. The stone density is 2275 kg/m^3 (142 $lb./ft.^3$).

NUMERICAL MODELLING

Model Dimension

Most dynamic analysis models for building structures have been one-dimensional, that is, a cantilever model with a concentrated mass at each floor level. Along with the development of computer technology, two-dimensional frame models and some simplified three-dimensional models have also been used in recent years to investigate the dynamic responses of complex structures.

A three-dimensional model consisting of 3641 frame elements and 1810 joints is used to model this steel-masonry composite building based on the following considerations:

- Since the masonry walls were built integrally with the exterior steel frames and the structure is irregular in both plane and elevation, most building codes (NRCC 1990a, ICBO 1994) require using a three-dimensional dynamic analysis to determine the seismic forces in the masonry walls.
- All static and dynamic loads and combinations of these loads can be applied to a three-dimensional structural model consisting of all the major structural members in one micro-computer run which takes only a few hours to complete.

- A three-dimensional structural model is relatively ease to develop with the help of AutoDesign© (Guo 1994), a computer program package developed to automate the whole design process from AutoCAD® models, finite element analyses, section designs to the structural drawings.

Linear Spectral Analysis v. Nonlinear Time History Analysis
It may be argued that nonlinear time history analysis should be conducted for this 18 storey composite building. It is true that nonlinear time history analysis is the most sophisticated tool for the purpose of predicting design forces and displacements under seismic loading. Based on the following considerations, however, a linear spectral analysis was used to predict the dynamic responses of this building:

- Assumptions made for the earthquake characteristics imply considerable uncertainty in the predicted responses.
- There is still no generally accepted rate dependent constitutive model for masonry infills under seismic loading.
- The computational effort involved in the total nonlinear time history analysis of this 18 storey composite building was considered to be too extensive because a separate analysis would need to be carried out for every load combination.

Structural Effects of Masonry Infills
Unreinforced masonry infills are often neglected in dynamic analysis models. However, it should be emphasized that unless separated by adequate clearances from the frame, masonry infills should always be modelled in a dynamic analysis. This is because masonry infills are very rigid and thus attract large earthquake forces which could drastically alter the dynamic responses of the structure. The National Building Code of Canada (NRCC 1990a) clearly requires that "all portions of the structure shall be designed to act as integral units in resisting horizontal forces, unless separated by adequate clearances." The Guidelines for Seismic Evaluation of Existing Buildings (IRC 1993) also recognizes that "infill walls meant simply as partitions or as part of exterior wall between columns may have substantial stiffness," and thus requires that "a mathematical model of the physical structure should represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate to calculate the significant features of its dynamic response."

It is true that integrally built masonry infills could be neglected in a dynamic analysis if all the infills were to be separated from the steel frames by adequate clearances. Providing these clearances to the integrally built masonry infills, however, will require the following:

- To cut the thick masonry panels at all the masonry-steel interfaces;
- To design and install an elastic and weather proof joint;
- To design and install a lateral restraint system to provide out-of-plane stability for all of these free standing masonry panels;
- To strengthen the existing steel frame because the above work significantly reduces the stiffness and the lateral load resisting capacity of the existing structure.

Model of Masonry Infilled Frame

Brief Background. The modified equivalent braced frame method (Paulay et. al. 1992) is widely used to analyze infilled frames. This method was originated from Polyakov's concept that the composite action of an infilled frame could be modelled by replacing the infill with an equivalent diagonal strut (Polyakov 1960). Stafford-Smith et. al. further developed this concept into a practical analysis method (Stafford-Smith et. al. 1969). Plane stress finite elements (Riddington et. al. 1977, King et. al. 1978, Chiostrini et. al. 1991) and plastic collapse theory (Liauw et. al. 1983) have also been used to analyze infilled frames.

Discussions. The equivalent braced frame method was not used in this study since it was proposed for frames with solid infills and not for infills with large openings which would significantly change the stiffness and behaviour of infilled frames (King et. al. 1978, Dawe et. al. 1989, Paulay et. al. 1992). Another reason for not using the equivalent braced frame method is that this method was based on the assumption that the infill and frame were not constructed integrally which is not true for this building. Plane stress finite elements were also not used because the required rate dependent constitutive laws for the masonry and interfaces are still to be developed. Due to the large window openings, the infilled frames were modelled in this study as steel-masonry composite moment-resisting wall frames.

Masonry Stiffness

Dynamic responses of unreinforced masonry structures are normally analyzed using uncracked masonry sectional properties because both the design earthquake forces (NRCC 1990a) and the member capacities (CSA 1984) are based on elastic responses.

For the dynamic responses of steel-masonry composite wall frames, however, using uncracked masonry sectional properties is questionable. This is because unreinforced masonry is much weaker than steel in tension and thus will normally be cracked far before the flexural strength of steel section is reached. If the flexural capacity of unreinforced masonry members is relied on to resist lateral forces, uncracked masonry sectional properties should be used. In many cases, however, the steel sections in a steel-masonry composite frame are much stronger than the unreinforced masonry in bending. The major structural function of the masonry infills is to resist vertical axial compressive load and lateral shear.

According to the above considerations, the flexural stiffnesses of the steel-masonry composite columns were calculated in this study using only the steel sections whereas the cross sectional areas and shear areas required in the analysis were computed using both the steel sections and the transformed masonry sections. The contribution of masonry to the cross sectional area of column sections was included because masonry is strong in compression, though weak in tension. The decision to include the contribution of masonry to the shear areas was based on the fact that columns were normally subjected to compression and under compression masonry could still resist shear forces even after cracks appeared (Guo 1991). Since beams are mainly subjected to bending and also tension, their sectional properties were calculated using the steel sections only.

AUTOMATED ANALYSIS AND DESIGN

This preliminary three-dimensional dynamic analysis was carried out with the help of the computer program AutoDesign© briefly described in the appendix.

AutoCAD® Model

Although the architect's drawing file is not available for this historic building, a 3D model was easy to setup for each typical floor. Columns and beams were grouped according to their sectional properties as required by AutoDesign©. Each group was drawn on a separate layer. The final model was simply built up using the typical floor models.

Finite Element Program Input File

Defining joint numbers, member numbers and structural topology has been the most time consuming and error prone task for preparing finite element input data. Fortunately, it is not the case anymore when AutoDesign© is used. Although our 3D model contains 3641 elements and 1810 joints, all of the joint numbers, member numbers and structural topology were automatically determined by AutoDesign©. AutoDesign© was also used to automatically detect and delete all of the closely spaced extra joints and members resulting from numerical error and inaccuracy in the graphical model. All of the member orientations, member end conditions and load specifications were also defined with the help of AutoDesign©. All of the cast-in-place concrete floors were modelled as rigid diaphragms using a master joint for each floor. The out-of-plane stiffness of the floors was modelled through the stiffness of the beam members. Floor mass was applied at the geometric centre of each floor diaphragm. The masses for all of the columns and beams were applied using distributed member mass densities to represent the actual spatial distribution. A critical damping of 5% was assumed for all modes.

Spectral Analysis

The spectral analyses were conducted using the normalized design distribution spectrum specified in the supplement of NBC90 (NRCC 1990b) scaled by the zonal velocity ratio of 0.05 for Toronto. The complete quadratic combination method was used to combine the 10 modal responses. The base shears from the spectral analyses were 4760 kN (1070 kip) and 5347 kN (1202 kip) for the X- and Y-direction, respectively. The accumulated mass participation factor was 95.8%. Shown in Fig. 1 is the deformed shape of the building frame under the seismic load in the X-direction.

The base shears from the spectral analyses were required to be scaled to the specified minimum lateral seismic force of the code (NRCC 1990a). Following the code equations, the calculated periods were 0.79 seconds and 1.8 seconds for the X- and Y-direction, respectively. These values were much smaller than the periods of 8.8 seconds and 6.2 seconds determined from the dynamic analysis for the X- and Y-directions. Using a foundation factor of 1.0 for rock foundation and the code permitted 20% increase of the periods (NRCC 1990a), the code specified minimum seismic base shears were 8066 kN

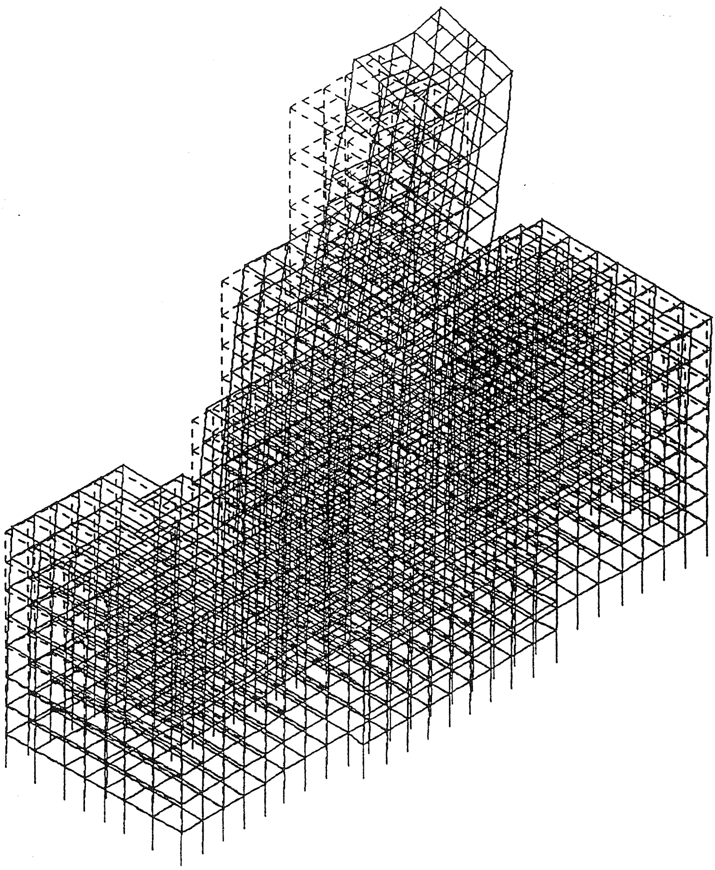

Fig. 1 Deformed Shape Under Seismic Load in the X-Direction

(1813 kip) and 2671 kN (600 kip) for the X- and Y-direction, respectively. The factors of 1.69 and 0.50 were therefore used to scale the spectral analysis results.

As reported above, the 3D dynamic analyses gave much larger fundamental periods than those calculated using the code specified method. This is simply because 3D dynamic analyses tend to overestimate the fundamental period and the code method tends to underestimate it. The reason for the code method to underestimate the fundamental period is that lower periods would usually result in larger base shears for the majority of the buildings. One of the major reason for 3D dynamic analysis to overestimate is that the fundamental period for 3D models has not been clearly defined. The concept of the fundamental period for multi-storey buildings is based on the simplified one-dimensional cantilever model with mass concentrated at each floor level. For such a model, the first modal response usually dominates the total dynamic responses and thus closely relates to the code specified fundamental period. For a 3D model, the contribution of the first modal response to the total dynamic response is usually much less. If some local modal responses have longer periods than the global modal response, the first period of the 3D model has little similarity with the code specified fundamental period. Therefore, the current NBC90 practice to scale spectral base shears to the code specified base shears according to the fundamental periods is not as good as the method adopted in UBC94 (ICBO 1994).

The relative magnitude of the base shears in the two directions from the dynamic analysis is also quite different from that calculated using the code method. According to the code method, the base shear in the X-direction is 3 times of that in the Y-direction, whereas the dynamic analysis gave a ratio of 0.89. The major reason for the code method to over-estimate the base shear in the X-direction is that the code specified period is based on the total length of the building without considering the reduced stiffness due to the large openings in the walls and the weak tensile strength of the masonry.

Demand/Capacity Ratio

After obtaining finite element program outputs for the dead load, live load, and earthquake loads in both the X- and Y-directions, AutoDesign© was used to analyse all the 27 load combinations and to check the demand/capacity ratios for all of the 3641 columns and beams. Although AutoDesign© can analyze and design 20 different steel section types currently used in North America, the steel sections in this building were manufactured 60 years ago and thus equivalent sections were used in the code checking (but the original sectional properties were used in the dynamic analyses).

For each of the 145 member groups, AutoDesign© summarized all of the compressive, tensile, bending and shear strengths in both the major and minor principal directions, the maximum demand/capacity ratio, the critical load combination, the number of the critical member and the critical section of that member. According to the maximum demand/capacity ratios, all the member groups that should be strengthened could be easily identified. To identify exactly how many members should be strengthened and where they were located in the structure, AutoDesign© was again used to generate AutoCAD® drawings with the maximum demand/capacity ratio for each member, the member and joint numbers, the section tags, and the six force components.

Shown in Fig. 2 is an example drawing in which only maximum demand/capacity ratios and member numbers were printed for clarity. Based on these maximum demand/ capacity ratios it is clear that the 7 lower floors need not be strengthened. The upper floors, however, have to be strengthened because of the large demand/capacity ratios.

The in-plane moment envelope shown in Fig. 3 clearly indicates one of the reasons for the large maximum demand/capacity ratios for the upper floors. That is, the dynamic responses of the tower were amplified by the dynamic responses of the transfer girders. This fact emphasized once again the importance of providing a direct load path to the foundation.

CONCLUSIONS

Based on our three-dimensional dynamic analyses of the 18 storey steel-masonry composite heritage building, the following major conclusions are drawn:

1. The spatial distribution of the stiffness and mass of all masonry infills must be included in the dynamic analysis model, unless they are separated from the frame by adequate clearance.
2. The dynamic analysis of three-dimensional building models consisting of thousands of structural members is both feasible and efficient to reliably analyze, design and evaluate complex structures.
3. AutoDesign© is a useful computer program package for structural analysis and design.

APPENDIX: MAIN FEATURES OF AUTODESIGN©

Automated Analysis and Design Processes

After building structural models directly from the architect's drawing files, AutoDesign© may be used to automatically define all the joint numbers, member numbers and structural topology. When finite element analysis results of basic load cases are available, AutoDesign© can automatically combine all the load cases, design all members, provide structural drawings with new section tags, and provide member or reaction force envelopes according to user's brief instructions.

Combined Powers from ACAD®, Finite Element Programs, Design Packages and Database.

AutoDesign© users can utilize all the power of AutoCAD® to quickly define sophisticated structural models without worrying about how to number the joints and members error free. Because AutoDesign© can be used to prepare the input data file and to read the output results of several commonly used finite element programs, the user can freely choose the best

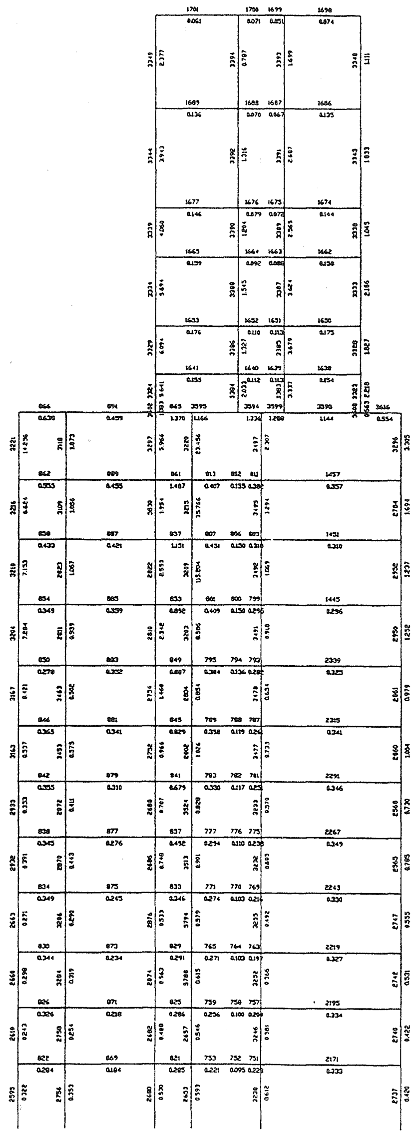

Fig. 2 Example Drawing of Maximum Demand/Capacity Ratios and Member Numbers

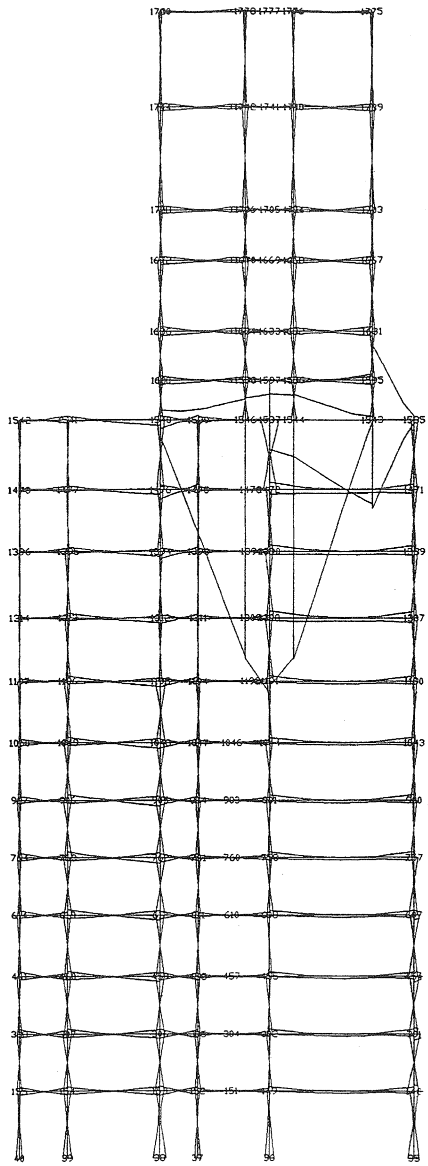

Fig. 3 Example Drawing of In-Plane Moment Envelope and Joint Numbers

program without worrying about the complicated input and output data. AutoDesign© users can also analyze and/or design all the hot-rolled steel, cold-formed metal, wood and concrete members in a structure simultaneously. To increase the speed and flexibility, AutoDesign© also automatically creates , uses, updates and maintains its database.

High Speed and Accuracy
Since the whole design process has been automated, AutoDesign© not only dramatically reduces design time but also eliminates many human errors. For example, AutoDesign© took only 8 hours on a micro-computer to change 4 input files for the new downtown Vancouver arena, to conduct 4 finite element runs including the dynamic analysis of 40 mode shapes for 15624 degrees of freedom, to combine all the load cases, to design all the 5885 members, to plot the reaction force envelopes at all column bases, and to plot all the required structural drawings with new member forces and section tags.

Freedom of Modelling, Analysis and Design
Because AutoDesign© can automatically transfer AutoCAD® models into input files for different finite element programs, to combine the results and to design the member sections of different materials, engineers can freely use very complicated structural models, advanced analytical methods and structural members of suitable materials.

Advanced Modelling Capacities
AutoDesign© has many unique modelling capabilities not generally available from other commercial computer program packages. The following are only some examples:

- Automatic elimination of extra joints and members
- Automatic calculation of rigid end offsets
- Automatic adjustment for link beam joint coordinates in eccentrically braced frames

Broad Applications.
In addition to the automated analysis and design capabilities for 20 different hot-rolled steel section types, 10 different cold-formed metal sections, all wood types and grades listed in CSA-O86.1-M89, and rectangular or circular concrete sections, AutoDesign© can also be used to solve other problems:

- Determine screw capacities according to CSA-S136-M89 or AISI new specifications
- Calculate torsional stresses in structural beams
- Quick analyze and design for struts
- Determine buckling strength with unique load positions, load types and end conditions
- Compute dynamic effect of vortex shedding
- Automatic finite element program input file update

Virtually Unlimited Capacities
AutoDesign© permits the use of up to 32767 members, 32767 joints, 32767 member groups, 32767 section types, 32767 load cases, and 32767 finite element program output files.

REFERENCES

Bruneau, M. (1994), State-of-the-Art Report on Seismic Performance of Unreinforced Masonry Buildings, J. Struct. Eng., ASCE, Vol. 120, No. 1, pp. 230-251.

Chiostrini, S. and Vignoli, A. (1991), Mechanical Modelling of Masonry Walls with Various Openings, Proc. of the Second Int. Conf. on Repairs and Maintenance of Historical Buildings, Seville, Spain, Computational Mechanics Publications, pp. 247-262.

Dawe, J.L. and Seah, C.K. (1989), Behaviour of Masonry Infilled Steel Frames, Canadian J. Civil Eng., Vol. 16, pp. 865-876.

Guo, P. (1991), Investigation and Modelling of the Mechanical Properties of Masonry, Ph. D. Thesis, McMaster University, Hamilton, Ontario, Canada, 388 pages.

Guo, P. (1994), AutoDesign©: Computer Programs to Automate Design Processes from ACAD® Model, Structural Analysis and Design to ACAD® Drawings, 22 pages.

Institute for Research in Construction (1993), Guidelines for Seismic Evaluation of Existing Buildings, National Research Council of Canada, Ottawa, Ontario, Canada, 150 pages.

International Conference of Building Officials (1994), Uniform Building Code, Vol. 2, Whittier, California, USA, 1339 pages.

King, G.J.W. and Pandley, P.C. (1978), The Analysis of Infilled Frames using Finite Elements, Proc. Inst. Civil Eng., London, UK, Part 2, Vol. 65, pp. 749-760.

Liauw, T.C. and Kwan, K.H. (1983), Plastic Theory of Infilled Frames with Finite Interface Shear Strength, Proc. Inst. Civil Eng., London, UK, Part 2, Vol. 75, 1983, pp. 707-723.

National Research Council of Canada (1990a), National Building Code of Canada 1990, Ottawa, Ontario, Canada, 423 pages.

National Research Council of Canada (1990b), Supplement to the National Building Code of Canada 1990, Ottawa, Ontario, Canada, 259 pages.

Paulay, T., and Priestley, M.J.N. (1992), Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley & Sons, Inc., Toronto, 744 pages.

Polyakov, S.V. (1960), On the Interaction between Masonry Filler Walls and Enclosing Frame when Loaded in the Plane of the Wall, Earthquake Engineering, Earthquake Engineering Research Institute, San Francisco, CA, pp. 36-42.

Riddington, J.R. and Stafford-Smith, B. (1977), Analysis of Infilled Frames Subjected to Racking with Design Recommendations, Structural Engineer, Vol. 55, No. 3, pp. 183-199.

Stafford-Smith, B., and Carter, C. (1969), A Method of Analysis for Infilled Frames, Proc. Inst. Civil Eng., London, UK, Vol. 44, pp. 31-48.