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FINITE ELEMENT ANALYSIS OF THE SOUTH WEST TOWER OF THE WEST BLOCK PARLIAMENT BUILDINGS, OTTAWA, CANADA

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ABSTRACT

Built during the 1859 to 1865 construction period of the West Block, the square 39m high massive stone masonry South West Tower of the West Block Parliament Buildings is a part of the Parliamentary Precinct of great historic value to Canada. By the early 1990's, the upper half of the tower above the fifth floor exhibited serious distress in the form of movements. cracking, bulging and mortar deterioration. The nature of distress and the extent of deterioration called for immediate stabilization measures to ensure adequate life safety and serviceability. Because of the complex geometry of the tower that involved eccentric wall offsets and a plinth region at about the sixth floor level, a three-dimensional finite element was carried out for the part of the tower above the fifth floor to help understand the causes of distress and define appropriate stabilization measures. This paper presents the ALGOR 3-D analysis results for both the as-built tower and key stabilization measures. Results indicate firstly, that the eccentric wall offset must have caused structural cracking soon after completion of the tower construction a century ago and secondly, that recommended stabilization measures consisting of interior tie rods, wall ties, wall stitching and a limited interior steel supporting system would significantly reduce critical tensile and splitting stresses. The stabilization measures were implemented in 1994 without damage to the tower's historic fabric. Also, a minor monitoring program has been implemented to ensure future safety and serviceability.

INTRODUCTION

The square 39 m high, massive stone masonry South West tower of the West Block Parliament Buildings is a part of the Parliamentary Precinct of great historic value to Canada. Built during the 1859 to 1865 construction period of the West Block, especially the upper half of the tower above the fifth floor exhibited distress such as cracking, movements, and mortar deterioration prior to undertaking stabilization measures. The nature and extent of serious distress were such that stabilization measures were quickly initiated during the summer of 1994 and were completed by year's end.

TOWER DETAILS AND DISTRESS

As indicated in Figs. 1 and 2, the square loadbearing stone masonry tower exhibits an unusual wall offset at the 6.floor plinth region. The offset is large for all four tower elevations and is largest for the South wall where it amounts to 18 in. (about 450 mm) as shown on Fig. 2. The walls are composed of inner and outer wythes of roughly squared Nepean sandstone filled solidly with rubble stone and mortar. The plinth as well as the quoin chain stones, pointed arch voissoirs, moldings and trim consist of dressed Ohio sandstone. The two floors of relevance to the tower analysis dealt with here act structurally very differently; while the fifth floor consists of shallow brick masonry arches spanning between the lower flanges of the four substantial steel beams indicated in Fig. 2 (and the steel beams extend to the centre of the East and West walls thus providing adequate support for potential stabilization measures), the sixth floor prior to the 1994 repairs had ten minor steel beams encased in deteriorated concrete. These steel beams extended little into the East and West walls and were additionally supported at their third points by two deteriorated brick masonry arches.

While condition surveys of the tower in 1993 and 1994 established serious distress (such as the typical vertical cracking shown in Fig. 1), displacements between external stones of up to 50 mm, other wall movements and bulges, as well as mortar and rubble core deterioration, a key mandate was to determine if the unusual wall offsets could have been the initiator of cracking distress a long time ago and other distress would be largely due to the ingress of moisture, freeze-thaw action and other time dependent weathering/aging/corrosion effects. Cracking distress was found to be worst in the plinth region and in the storey below.

FINITE ELEMENT ANALYSIS

To help understand the cause of the cracking and define stabilization measures, a 3-D finite element analysis of the upper part of the tower above the 5.floor was carried out for gravity loading and wind effects. A 3-D analysis was required partly because of the complex tower geometry involving different wall offsets in the 6.floor plinth region and partly because a 2-D analysis could not capture corner continuity effects.

In employing the ALGOR finite element system, three different finite elements were used: an eight noded brick element to model the stone masonry walls, a space truss element to model a potential tying system, and a 3-D beam element to model a potential interior frame as part of stabilization measures. The analysis was carried out in two phases. Phase 1 established firstly, if gravity loading alone could have caused the cracking and secondly, to what extent various stabilization schemes could reduce damaging tensile stresses. The Phase 2 work determined the effect of wind loading on critical tensile stresses caused by gravity loading.

Throughout the analysis, materials were assumed to be linearly elastic, the tower was assumed to be uncracked as would have been the case after completion of construction, and the boundary conditions at the bottom of the model (5.floor level) were modelled to provide freedom for the walls to move in the lateral direction.

In total, 22 different cases were run to ascertain the cause of cracking and determine an economical/effective/minimally intrusive stabilization scheme. Parameters considered in the analysis included the following: effect of varying stiffness of the tying system; effect of stiffness of the 6.floor; effect of different interior framing systems including various stiffnesses and configurations; and effect of various wind directions.

ANALYSIS RESULTS AND STABILIZATION SCHEME

The analysis of the tower under its own weight indicated that the wall offsets cause significant spreading forces concentrated mainly at the bottom of the plinth. These forces in turn cause transverse tensile stresses in the horizontal direction in both the plinth region and the storey below. Critical horizontal tensile stresses on both the outer and inner wall surfaces are shown for the wall having the largest offset and therefore the largest stresses, i.e. the South wall, in Fig. 3(a). A critical vertical tensile stress also occurs in the plinth region as illustrated in Fig. 3(b) for the South wall. Based on the premise that massive loadbearing stone masonry walls must be kept in compression to prevent cracking and the direct tensile bond strength of stone masonry built with low strength lime mortar is very weak at all times, the 3-D analysis results indicate that the eccentric wall offsets likely initiated structural cracking soon after completion of tower construction over a century ago. The effect of wind loading was found to increase critical tensile stresses by only about 20 percent; this is as expected due to the massiveness of the walls and overall tower geometry.

In arriving at an appropriate stabilization scheme, many configurations were tried until final measures consisting of two sets of tie rods (in the North-South and East-West directions) and an interior steel support system were adopted. As shown in the plinth detail of Fig. 4, the tie rods were found to be most effective when located at the bottom of the plinth to resist spreading forces caused by the wall offsets; the interior "safety support system" consists of a newly cast triangular reinforced concrete beam underneath wall offsets, as well as load spreading steel beams and steel columns resting on the substantial steel beams at the 5.floor level. In evaluating the effectiveness of various stabilization measures by means of 3-D analyses, it was assumed that the measures would be acting on the uncracked tower as originally built. While this is known not to truly represent the much more complex situation of the tower in its cracked and deteriorated state prior to the 1994 repairs, such analyses do reflect altered load paths and stresses of stabilization measures and also provide design guidance. For the final stabilization measures adopted, Fig. 3 shows that the maximum tensile stresses for the South wall below the plinth would be reduced substantially (by about 60 percent) and that other critical tensile stresses are eliminated completely. While it would have been theoretically desirable to arrive at stabilization measures which would have prevented the occurrence of tensile stresses altogether, this proved to be impossible to achieve for the major wall offsets and reasonable compatibility between the stiffnesses of the masonry tower and stabilization measures. From a practical viewpoint all of this means the tower will be subjected to limited cracking in the future and as such cracking takes place, the tie rods and interior support system of the stabilization scheme will start to participate in carrying loads.

CONCLUDING REMARKS

Following the 3-D analyses, a detailed design of the stabilization measures was carried out and the installation of these measures was completed by the end of 1994. Note that the measures could all be installed from the tower's interior thus not affecting the exterior aesthetics of this classified federal heritage structure. As part of the tower's stabilization, deep repointing of all exterior and interior mortar joints above the 5.floor was performed. Demec extensometer points have been installed in a belt course fashion around the plinth to monitor future horizontal movements.

ACKNOWLEDGEMENTS

The analysis work was the responsibility of the first two authors, while the detailed design was carried out by the third author who served as the prime consultant on the project. The work was performed for Public Works and Government Services Canada under the fourth author's responsibility as project manager. The subject of this paper was also addressed in a paper published in the proceedings of the STREMA, Greece, 1995.

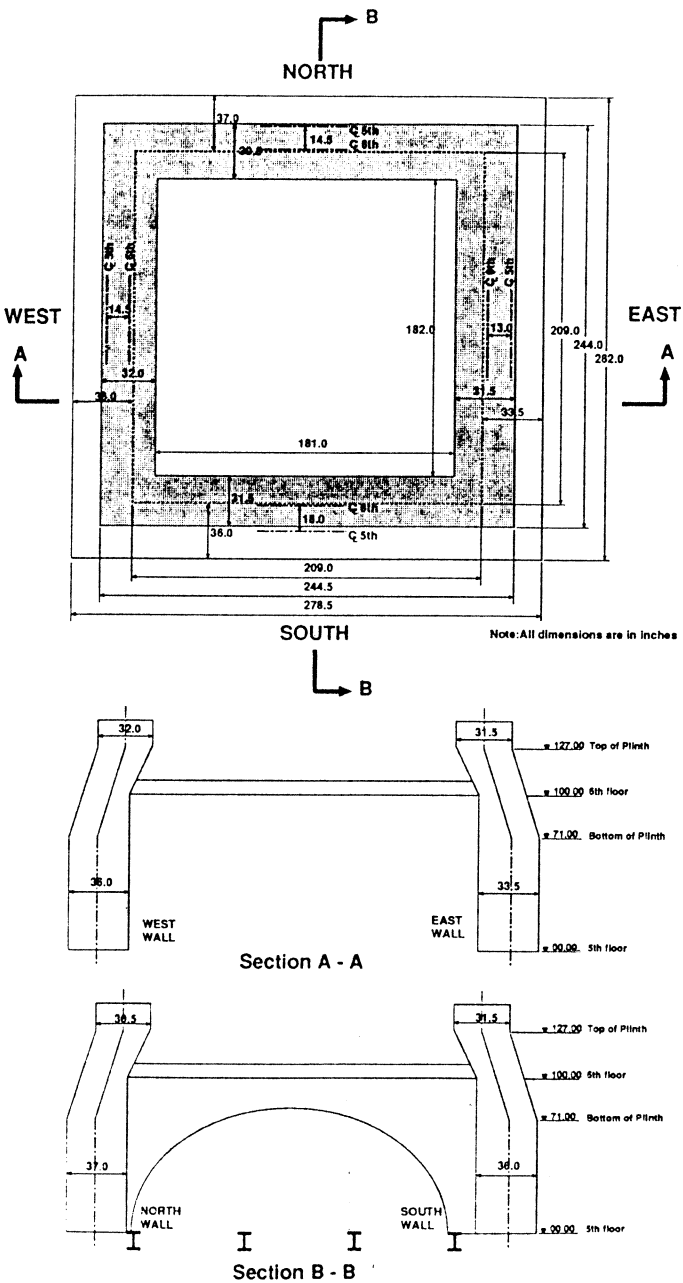
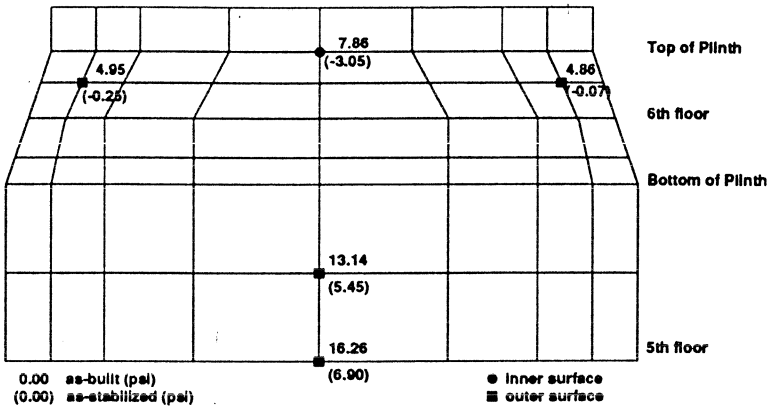
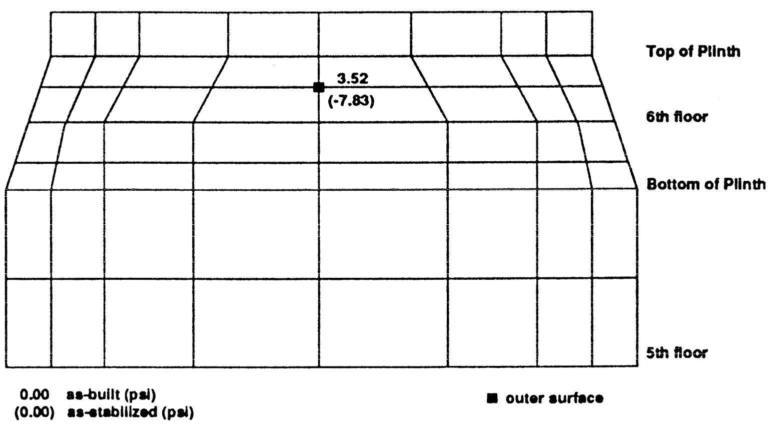


Fig. 2 Sectional Plan and Cross-Sectional Elevations for the Tower between the 5th and the 6th floor



(a) Horizontal Stresses at Key Points on Surface of the South Wall
(positive stress is tensile)



(b) Vertical Stresses at Key Point on Surface of the South Wall
(positive stress is tensile)

Fig. 3 Comparison of Key Stresses Before and After Applying the Stabilization Scheme

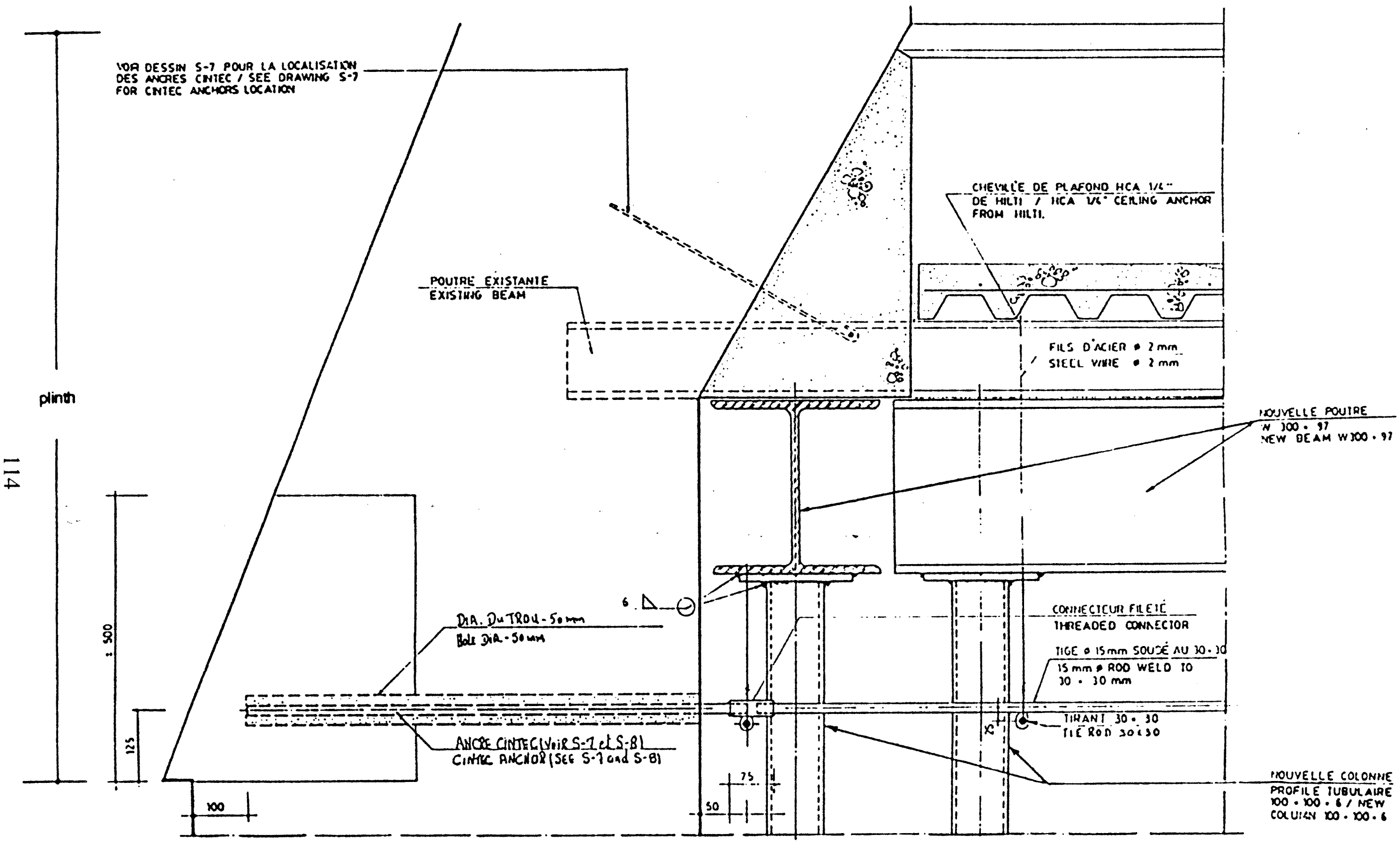


Fig. 4 Enlarged Plinth Detail Showing Proposed Stabilization Scheme