Chapter 6

WIND AND SEISMIC ANALYSIS

The structural system of the Smithsonian Castle, when taken as a whole, lacks significant symmetry and repetition. As a result, there was an early recognition that evaluation of the Building's lateral force resisting system would involve several distinct stages. First, the main force resisting systems in each major segment of the building needed to be analyzed. Second, individual elements in those systems were judged potentially vulnerable to high stresses when subjected to seismic accelerations perpendicular to their primary strength and stiffness. Third, certain portions of the building, most notably the towers, were thought likely to behave independently of the main building. As a result, these "sub-structures" had to be analyzed separately. Finally, the ability of the floor and roof structures to transfer loads to the vertical elements of the lateral system had to be investigated to as great a degree as possible.

Figure 1 - Smithsonian Castle

Nature of Lateral Loads

Lateral load is usually derived from one of two sources. The first, to which virtually every building is exposed, is loading due to wind. The magnitude of wind forces is based on local terrain and climatic information and, most importantly, on the height of the building. The second source of lateral load is earthquake, or seismic, activity. A given building may never be exposed to such loading during its "lifetime", but it is wise to provide it with some capacity to survive such exposure, should it occur. Aside from the concern for human safety which drives such strategy, a building such as the Smithsonian Castle merits special consideration due both to its valuable contents, and its own status as an "object" of great historical and artistic value.

Due to the limited human understanding of seismic events, design loads are based on regional seismic history and potential sources of seismic activity (known fault lines, even if they have no recorded history of activity). From this information, the probabilities of seismic events of various magnitudes are established for different regions (seismic "Zones"). From this statistical study, in turn, recommended lateral design load formulae are developed for different types of buildings within the different Zones. The one uniform factor, however, is the weight of the building under consideration. As the weight of a building increases, it is subject to higher and higher earthquake loads.

In the particular case of the Smithsonian Castle; the structure is fairly tall, especially by the standards of it's day. The building is also quite heavy, due to the thick masonry walls, piers, and columns which constitute its vertical structural system, as well as the heavy masonry vault and arch floor construction. As a result, even with the low seismic risk associated with the Washington, DC. area, one would expect seismic loads to govern over those caused by wind.

Lateral Load History

The Smithsonian Castle was designed and built before the development of reliable means to evaluate lateral loads and before the evolution of the modern quantitative structural analysis techniques now used to study a structure subjected to those loads. As a result, the building's design is a product of the knowledge and experience gained through generations of the builder's art, gleaned from both successful buildings that have survived for centuries, and from structural failures that provided invaluable bits of knowledge, often at the cost of human lives.

This empirical design process produced a building that has survived quite well for more than a century through multiple alterations in its use, including drastic increases and decreases in the vertical (gravity) loading applied to it. In addition, it has weathered the several tropical storms, remnants of hurricanes, and even occasional full scale hurricanes that have passed through the District of Columbia during this time, with no apparent structural damage. On the other hand, there is no record of significant seismic activity in the area, and, as discussed above, it is unlikely that the building's designers would have had any knowledge of techniques to mitigate the damage from such an event. It seems prudent, therefore, to examine the lateral strength and stability of the building in light of modern load evaluation and structural design techniques.

Relevant Documents

There are several documents in use in various parts of the United States which define the lateral design loads on geographical, climatic, and historical bases. The one in most common use in the Washington, DC. area is the "National Building Code" published by Building Officials & Code Administrators International, Inc. (BOCA). In addition, the International Conference of Building Officials (ICBO) publishes the "Uniform Building Code" (UBC) and the American Society of Civil Engineers (ASCE) publishes "Minimum Design Loads for Buildings and Other Structures" (designated ASCE 7). Although there are some editorial and format variations, the lateral load provisions of ASCE 7-93 (1993 edition of the document) are essentially identical to those of the 1993 edition of BOCA (BOCA 93).

By way of background, BOCA and UBC are "model building codes", commonly incorporated into local building code legislation. BOCA is primarily used in the Eastern half of the United States, and has been adopted by the District of Columbia and most jurisdictions in Maryland and Northern Virginia. Likewise, UBC is the model building code most commonly adopted as part of local ordinances West of the Mississippi River. As a result, there is a general belief that the UBC seismic provisions are somewhat more refined than those included in BOCA, since the UBC has evolved in the more earthquake prone regions of California (the most critical area), the Rocky Mountains, the Pacific Northwest, and, to a lesser degree, areas near the confluence of the Mississippi and Ohio rivers.

While BOCA and UBC, as model building codes, address all aspects of building-related public safety issues, the ASCE document is concerned only with the evaluation of structural loads. It is a recognized national standard used widely throughout the United States. While not normally adopted as part of a local building code per se, it is commonly recognized as accepted practice and used for specific direction in specialized or unusual loading conditions not directly addressed by the model building codes.

Seismic Loads

The initial task was to select conservative, yet reasonable, criteria for the evaluation of the potential seismic loads on the Castle Building. To this end; the earthquake provisions of ASCE 7-93, the 1993 edition of BOCA, and the 1991 edition of UBC were applied to the specific building and location in question. By way of explanation, BOCA 1993 was examined in lieu of BOCA 1990 (the legally binding code in Washington, D.C.) because it was considered appropriate to use the most advanced code provisions available for the seismic analysis.

ASCE 7-93 and BOCA 1993 assign Washington, DC the lowest seismic risk possible under their (virtually identical) provisions. Under the criteria associated with this level of risk, there is no requirement that the structure, as a whole, be evaluated for any specific earthquake induced loading. The only requirements are that there be a complete "path" for the transfer of seismic loads from the elements in which they originate to the building's lateral system, and, then, to the supporting ground. Further, the connections between elements in this path are required to satisfy certain strength requirements.

Evaluation of lateral strength using the loads generated from ASCE and BOCA code provisions (in spite of the lack of a requirement to do so) would have imposed an analytical lateral load equivalent to 10.0% of the building's weight. This value is somewhat higher than the value associated with a (hypothetical) identical building in UBC seismic zone 2B, which covers parts of the Rocky Mountain region, the Pacific Northwest, and Alaska. The elevated load resulted from the application of a "minimum" value for one of the factors in the lateral load equation. Based on past discussions with individuals involved in the development of the BOCA standards, a realistic value for this factor in the Washington area is probably far less than that "minimum" employed, hence the codes' requirement for nothing beyond the load path and connection strength study.

UBC 1991 places the District of Columbia in a seismic zone 1 (on a scale from 0 to 4). UBC does require an analysis of the structure and its elements (not just identification of a load path), and quantifies the lateral loads for the Castle Building at 3.4% of the weight of the building. Other seismic loads are applicable to other portions of the structure or to other types of analyses. These are summarized in Table 1. As a result of these studies, the ASCE 7-93 and BOCA 1993 provisions were dismissed as inapplicable, and the UBC 1991 provisions were selected for use in this study.

Type of Structure	Magnitude of Seismic Load (Expressed as a Portion of Dead Load)
Main Lateral System Components	3.400%
Appendages (Towers, Decorations, etc.)	15.000%
Components (Individual Walls, Columns, etc.)	5.625%

Table 1 - Summary of UBC Seismic Loads

Wind Loads

Due to the consistency associated with using a single set of loading criteria, there was a predisposition toward using the UBC 1991 provisions for wind loading as well. To verify the conservatism of such a decision, UBC wind load values were compared with those derived from the BOCA and ASCE provisions. Since the BOCA/ASCE provisions for wind load are somewhat more refined than those in UBC, the loads derived from the UBC provisions were greater (more conservative) than those produced by ASCE and BOCA, and were confirmed for use in this study. Total design wind loads on the primary building structure under both UBC and BOCA/ASCE are presented in Table 2.

True to earlier anticipation, the great mass (weight) of the building resulted in total seismic loads that far exceeded the wind loads. Because of this, the proposed "lateral load study" was; to a large degree, but with some specific exceptions; transformed into a study of seismic loads only.

Height Above Grade (feet)	Total Windward and Leeward Pressure (pounds per square foot)	
	UBC-1991	BOCA-1993/ASCE 7-93
100	19.93	16.47
90	19.14	15.81
80	18.35	14.97
70	17.56	14.29
60	16.76	13.46
50	15.79	12.62
40	14.82	11.61
30	13.41	10.43
25	12.70	9.77
20	11.82	9.09
15 and below	10.94	8.25

For elevations above grade of more than 15 feet, pressures at elevations between those included on the table may be derived by linear interpolation between pressures provided at adjacent tabulated elevations.

Table 2 - Summary of Wind Load

Main Lateral System

The vertical structure of the building is constructed of clay masonry (brick), red sandstone, cementitious mortar and, possibly, other types of masonry. As is true of most masonry, these materials are quite strong under compressive loads, but have very little tensile strength. Lateral loads almost invariably lead to tensile stresses in building elements. In most older masonry buildings, these tensile stresses are not high enough to overcome the "natural" compression in the masonry caused by its own sheer weight. Although there was some concern about some of the towers projecting above the main roof of the building, this general tendency and the overall proportions of the Castle building encouraged an initial evaluation based on a series of extremely conservative assumptions. In many cases, somewhat more precise assumptions were employed when these preliminary assumptions yielded mildly unfavorable results (the hope being that a more precise, yet still acceptably conservative, analysis would confirm the adequacy of the structural element in question).

It was first assumed that each significant segment of the building is supported only by those walls to which it is directly connected. This procedure was predicated on concerns for the capacity of the horizontal (floor and roof) framing to act as effective diaphragm elements. In reality, the horizontal structure may be capable of transmitting excess lateral load in one segment to an "under loaded" structural element in an adjacent segment. Second, the typical building wall pierced by arched openings was treated as an infinitely rigid horizontal beam or beams (representing the deep upper portions of the wall) supported by a series of relatively slender vertical columns connected rigidly to the horizontal element(s) and to the foundation.

In addition to the modeling techniques described above, the horizontal seismic shear derived both from the horizontal floor and roof structure and from the weight of the wall below the level in question was assumed to act at the level of the rigid horizontal element(s). This had the effect of "moving" components of the lateral load resulting from wall weight above their actual center of action and, thereby, maximizing the moments (and resultant flexural-tension stresses) imposed on the walls and columns resisting those loads.

Finally, the seismic study assumed that the building is supported laterally only at the bottom of the basement level. In fact, virtually all of the structure is supported, to some degree, by earth enveloping the basement walls. While this support was ignored for the purposes of this investigation, it is, in all probability, significantly reducing both the total horizontal force (by supporting the basement walls and first floor slab) and the moments resulting from the forces at higher levels (by reducing the effective elevation of those levels).

In addition to the conservative physical and load assumptions outlined above, a 1.5 safety factor normally associated with stability (overturning) analysis was employed in stress calculations. This safety factor is achieved by analyzing the critical section under full lateral (seismic) load, but using only two-thirds of the known dead load of the structure to counteract that lateral load. An allowable tensile stress for the masonry was selected based on the provisions of the American Concrete Institute (ACI) publication "Building Code Requirements for Masonry Structures" (ACI 530-88). Because of the lack of information with regard to the actual masonry and mortar types in place; the lowest allowable tension value prescribed for solid clay masonry units, 13 pounds per square inch (psi), was selected. The ACI document, as well as the BOCA and UBC model building codes, recognizes a 33% increase in the specified allowable stresses when the load includes wind or seismic components. As a result, the final analytical allowable tensile stress became 17.33 psi.

The first step in the actual analysis of the structure was to determine the distribution of the total horizontal load (equivalent to 3.4% of the building weight) through the height of the building. In order to model the increases in acceleration at greater elevations above the structure's base (foundation), UBC requires larger percentages of the total horizontal load to be applied at higher elevations and smaller percentages to be applied at lower elevations. The details of this distribution vary with the height and weight distribution of the structure but, per the requirements of UBC, the distribution was determined based on the height of the main roof of the building, ignoring the vertically projecting towers. Those towers were addressed either as separate structures, or as projections from the primary structure.

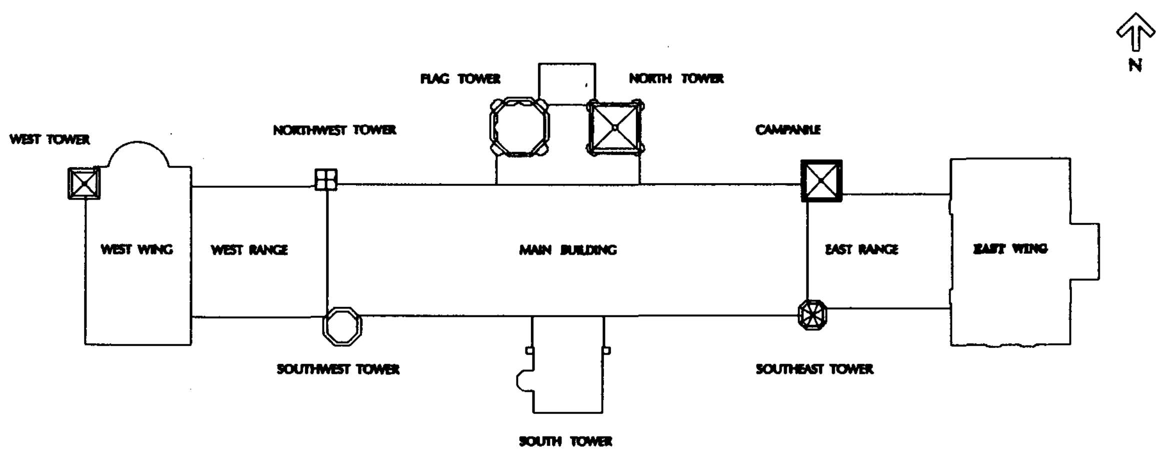

Figure 2 - Key Plan of Smithsonian Castle

The first part of the building to be studied was the central portion which houses the "Great Hall". This segment contains the first through fourth floors (the fourth floor being new with the 1969 renovation) and an attic and roof. The lateral system was assumed to consist of the exterior walls at the North and South faces of the building (not counting the vestibule/tower structures at each face) in the "long" direction (defined for all parts of the building as East to West), and, in the "short direction" (North to South), the walls separating this segment from the East and West "Ranges". Initially, the analysis was limited to the long direction, since the short direction system is shared with the adjacent Ranges. As one further conservative assumption, the load at the various levels of the structure was assumed to be resisted solely by the "columns" between the large windows on the North and South facades. This assumption ignored the strength provided by portions of these walls at the North and South entrances. At each window level, the seismic shear was assumed to be transmitted to the "columns" by a rigid wall mass above the windows. The analysis described indicated some net tension in both the upper and lower window columns, but the values were less than 10 psi, well below the allowable value of 17.33 psi adopted for the analysis.

Since the two Ranges display similar lateral systems, but the East Range is one floor taller than the West Range, a long direction analysis was performed only for the taller of the two Ranges. This study was very similar to that described above for the central area, except that it was performed at each of the three shorter windows in the North and South Range walls. No net tension was observed at any of the locations studied.

Figure 3 - West Range, 1849

Continuing the procession outward from the center of the building, the long direction strength of the East and West wings was next to be studied. At the East wing, the seismic shear was assumed to be divided evenly between the North and South exterior walls. The North wall was then investigated, as its structure is somewhat lighter than that of the South wall. The study was quite similar to that performed for the Ranges and central area, but the load was proportionally divided among the two wall segments separating the three exterior windows and the two remaining wall segments extending from the window to the corners of the wing. Performing the analysis at both the upper and lower windows, some net tension was observed, but, once again, the values were less than 10 psi, and were deemed acceptable.

Unfortunately, the West wing analysis yielded somewhat less encouraging results. The West wing houses a large open volume from the first floor up to the roof level, serving as the dining room for the Castle. The critical part of the West wing's main lateral system is the North wall, since it is effectively reduced to two ten-foot long segments by the apse at the North end of the room. It was recognized, however, that the wall segment to the West of the apse engaged the small campanile, increasing its total length to approximately twenty feet. The investigation proceeded by imposing the estimated seismic load associated with the roof and ceiling vault structure on the two wall segments described above in proportion to their lengths. Two different structural mechanisms were explored. The first treated the walls segments as vertical cantilevers, resisting their respective portions of the seismic load independently. The second mechanism assumed that the wall segments would act together as a couple to resist the overturning moment as axial members. The shear was assumed to be transferred to them by a rigid element representing the solid portion of the exterior wall above the apse. While this second mechanism reduced the analytical bending stress in the wall segments by allowing them to act in reverse curvature, the analysis of both mechanisms yielded tensile stresses in the masonry wall significantly in excess of the adopted allowable value.

Having concluded the analysis of the main lateral force resisting system in the long direction, the forces determined during that analysis were used to study the short direction system. It was anticipated that some structural problems might be discovered during this analysis since, unlike wind loads which are dependent on the exposed vertical surface area, seismic loads are derived from building mass which is, of course, independent of "direction" of an earthquake. Two major structural elements were studied during this investigation, the masonry walls shared by the central portion of the building and the East Range, and by the East Range and the East wing. These two walls were chosen since, due to minor asymmetries in the building, they were somewhat more heavily loaded and, as a result of corridor and door penetrations, slightly less continuous.

Even with the penetrations noted, the central/Range wall is reasonably modeled as a single, continuous wall for this type of investigation. It was decided, on the other hand, that the Range/East wing wall should be split into two equal segments for analytical purposes. The pair of walls at the Range/wing border was found to be subject to net tensile stresses at all levels, but those stresses only approached the allowable maximum at the foundation level, and were typically well below that limiting value. The wall at the border between the central area and the East Range, however, displayed less satisfactory results. Due, in large part, to the absence of significant dead load aside from the wall's own weight, the applied seismic load led to net tensile stresses varying from 6 psi at the fourth floor level, to over 75 psi at the foundation. The values exceeded the 17.33 psi allowable at every level below the fourth floor.

Detailed investigation of the short direction walls at the extreme ends of the East and West wings was considered unnecessary. Since the loads would be effectively identical to those imposed on the long direction walls in these wings, yet the resisting elements are nearly twice as long and significantly less perforated (especially in the West wing), it was clear from the results of the long direction analysis that net tensile stresses in these elements would be quite acceptable, if present at all. It should be understood at this point that this discussion applies to the adequacy of the short direction wing structures as part of the main lateral system. While they appear quite sufficient in that role, some problems were noted with the behavior of the West wall of the dining room. These problems were associated not with its ability to resist external loads applied parallel to the wall, but to resist loads generated by the wall's own weight when applied perpendicular to its surface. This is discussed in greater detail below.

"Secondary" Structures

As mentioned above, the investigation of the "Main" lateral force resisting system did not include either the loads or the strength contributed by the various towers and other projections that provide so much of the Castle's Romanesque flavor. Largely because of the questionable nature of the horizontal diaphragms in the building (a topic which will be discussed in detail later), the larger towers were examined as separate structures. The procedure for this analysis at each tower was essentially the same as that for the main building, including evaluation of a total base shear, and division of that total load through the height of the tower. For the Flag Tower and North Tower (the two towers in the North entrance structure), the resulting forces were imposed on the full square "tube" section of the tower at the base and at the elevation of the main roof. In both of these towers, the flexural tension stresses never overcame the compressive stresses due to the tower weight.

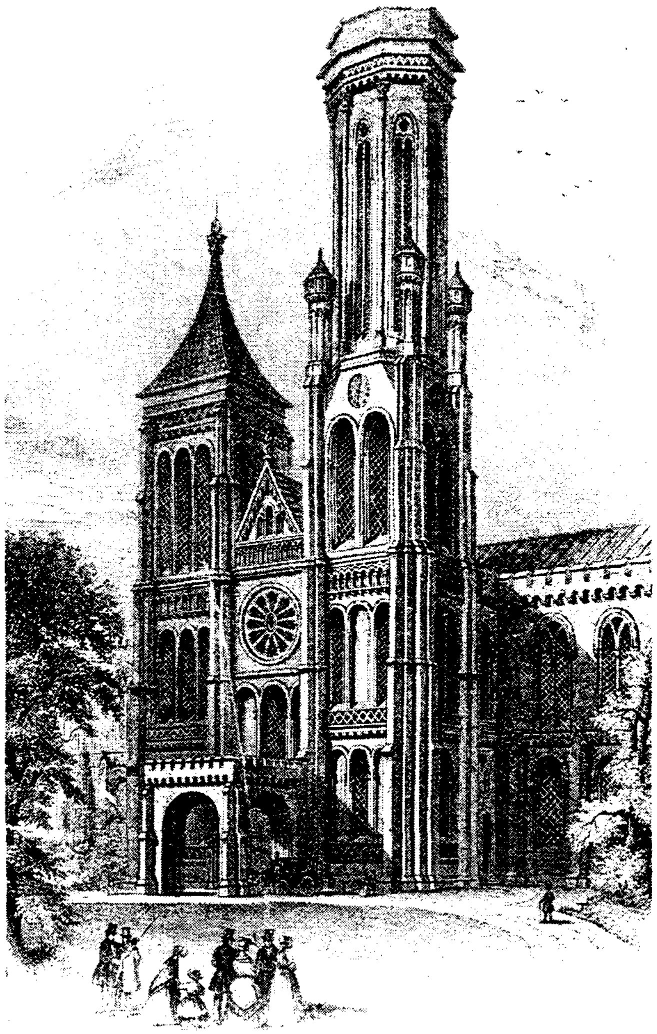

Figure 4 - Flag and North Tower, 1829

At the Campanile (at the Northeast corner of the central portion of the main building), however, the more open tower structure indicated that the full section of the tower could not be expected to behave as a single structural member. In response to this, the tensile stresses induced by imposing the horizontal shear on the eight "column" sections between the windows were compared to the compressive stresses induced on those eight sections by the weight of the tower. As was done for the two towers discussed above, this analysis was performed at the Campanile base and again at the main building roof level. At the base, the net tensile stress was approximately 30 psi, while at the higher elevation it approached 170 psi.

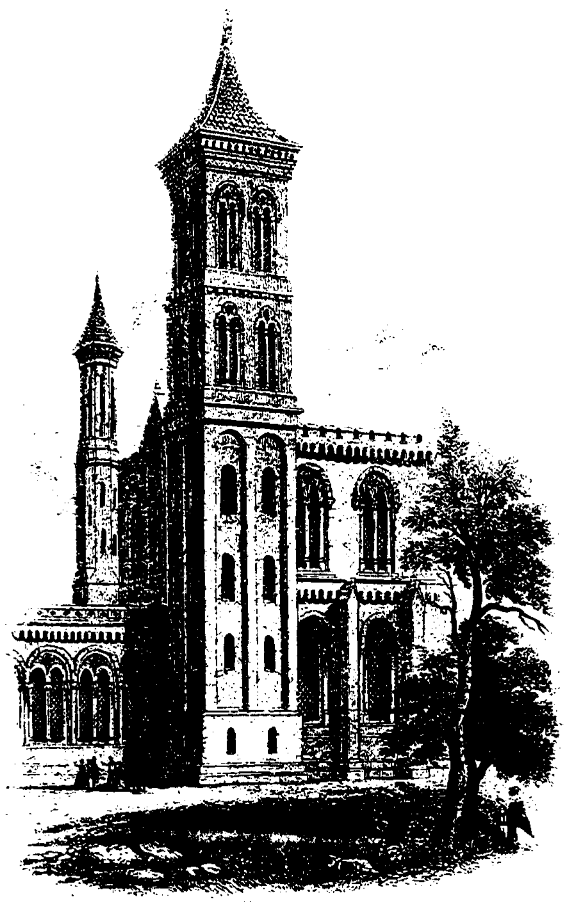

Figure 5 - Campanile

"Appendages" to the Main Structure

In addition to the large towers addressed in the preceding section, the Castle has several smaller towers attached to the main building structure. While the larger towers could potentially respond to seismic motion independently of the main building structure (with some potential cracking of the floor and roof structures that do, in fact, link them), the relatively small scale of the other towers makes such behavior unlikely. As a result, it is more reasonable to treat these elements as "appendages" to the main structure. The provisions of UBC 1991 require that such an element be loaded with a lateral force equivalent to 15% of its weight. As a representative of this type of element, an analysis was performed on the so-called "Octagonal Tower" at the Southwest corner of the central portion of the building. The analysis consisted of an examination of the portion of the tower above the main roof level acting as a cantilever off the fifth floor main building structure. This analysis revealed net tensile stresses approaching 700 psi, implying the potential for significant structural damage or failure during a seismic event.

A similar study was performed on the smaller octagonal tower at the Southeast corner of the central portion of the Smithsonian Castle (often referred to as the "Southeast Tower"). This investigation also indicated net tensile stresses in the tower masonry at the fifth floor level of between 100 and 200 psi. While these values are still unacceptably high, they do reflect the stress reductions that are expected with shorter tower structures.

Because of the extremely high net tensile stresses at the Octagonal and Southeast Towers, an analysis of the wind loads on these two elements was performed. This analysis indicated that tensile stresses induced by the wind loads prescribed by UBC-1991 would be approximately 40% of those caused by seismic loading. If BOCA/ASCE wind loads were to be employed, those stresses would be reduced by an additional 20% (to approximately 32% of stresses generated by seismic loading). While either of these wind stress values represents a great improvement over the seismic stresses, net tensile stresses well in excess of the allowable value (and approaching 200 psi in the Octagonal Tower under UBC loads) are anticipated even under the lower ASCE/BOCA wind loading. Despite these high, wind induced, tensile stresses; the structure appears to have performed quite well during close to one hundred and fifty years of exposure to Washington area wind loading. This suggests one of several possibilities. First, the actual strength of the masonry may be far higher than has been assumed. While this is possible, it is very unlikely that it is sufficient to resist the extremely high tensile stresses suggested by the wind load analysis. Second, it is possible that the wind loads prescribed by UBC-1991 have not yet been imposed on the structure, either because they are unrealistically high values for the region in question, because the design wind storm has not occurred since the towers were completed, or through a combination of these two. Finally, it is possible that the elements in question have, in fact, cracked during exposure to high winds, but that, upon cessation of the maximum gust, the cracks have closed. In such a situation, the cracking causes a temporary stress redistribution within the structural section, and/or to adjacent portions of the structure, which can serve as an effective "safety net" It is also possible that more sophisticated structural modeling, as discussed below, may prove the structures in question to be stressed less significantly than the current study has indicated. On the basis of these observations, it would be highly advisable to develop and implement an inspection program to identify and evaluate any evidence of wind induced tensile cracking or other damage in the tower structures throughout the building.

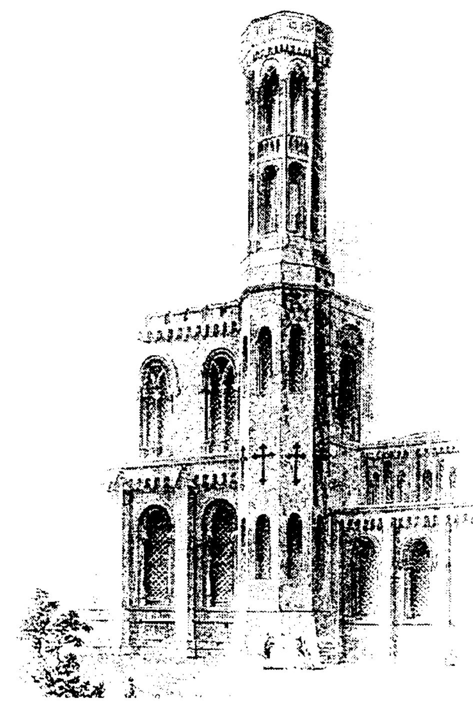

Figure 6 - Octagonal Tower

"Vulnerable" Components

In addition to the investigations, discussed above, of the main lateral load resisting system; several individual structural elements were identified as potentially vulnerable to local failure during a seismic event. These elements were addressed as structural components under the provisions of UBC 1991, requiring application of a lateral load equivalent to 5.625% of the weight of the element, in combination with gravity loads resisted by that element.

First, it was observed that, since there was no effective diaphragm at the attic/roof eve level of the central portion of the building, the seismic loads generated by roof and attic structures would have a tendency to be carried directly into the North and South exterior walls. These loads would be carried by those walls, acting as cantilevers up from the next lower floor. The analysis of this condition indicated that the net tension resulting from this behavior would slightly exceed the allowable value of 17.33 psi if those walls were assumed to be supported at the fourth floor. The fourth floor, however, is a recent addition and certain portions of it are targeted for potential removal. Without the fourth floor structure, the exterior walls would be required to cantilever up from the third floor structure. Under such conditions, net tension stresses of approximately 50 psi are anticipated, severely exceeding the value allowed. As a result, removal of the "new" fourth floor structure may reintroduce a seismic strength problem which the construction of that level had previously helped mitigate.

The two story columns in the "Great Hall" were also seen as potentially problematic under seismic load. Fortunately, however, there is more than sufficient compressive stress in those columns (derived from the dead loads they carry) to compensate for the tensile stresses induced by the design seismic loads. This would be the case even without the dead loads contributed by the "new" fourth floor structure, so these elements do not appear to be of concern.

Finally, as mentioned above, some problems were anticipated with the West wall of the West (Commons) wing. This wall spans vertically from the first floor to the fourth floor (roof) level, a distance of approximately forty feet. Under the design seismic loads generated by the wall's own weight, net tensile stresses of just over 23 psi are anticipated. Once again, this value exceeds the allowable stress adopted for this investigation, indicating some cause for concern.

Horizontal Load Transfer

While most of the seismic load carried by the elements of the Castle's lateral system is generated by that structure itself, a significant portion of the horizontal load is derived from the weight of the horizontal (floor and roof) structures that are supported (vertically and horizontally) by that system. As a result, the preceding discussions rely on the adequacy of the horizontal elements to act as diaphragms to transmit horizontal loads to the building's walls, and on the connections between those horizontal elements and the building's walls to actually transfer the load.

Unfortunately, the only clear documentation of diaphragm construction and connections is associated with the new fourth floor structure that was added in 1969. The drawings for that work indicate a $2^1/_2$" thick lightweight structural concrete slab supported on steel joists and beams. The slab itself is quite adequate to perform the necessary diaphragm function at this level. In addition, the structural steel beams added at that time are embedded in bearing pockets introduced into existing masonry walls. These connections appear adequate for transmission of horizontal (seismic) loads to the walls.

What remains to be addressed, however, is the typical, older, horizontal structure in the balance of the building and its connection to the lateral load resisting system. There is some reason for concern on this front, as some documentation of existing conditions done in conjunction with the 1969 renovation indicates that the typical horizontal construction includes sand and un-mortared masonry fill between the bottom brick arch structure and a brick or timber top layer. The top timber layer was replaced with concrete in certain parts on the second and third floors during the 1969 renovation, and that concrete should provide an adequate diaphragm. Several areas retain the brick or timber topping, however, and will likely not provide adequate diaphragm strength.

A final issue related to the efficacy of the original masonry floor diaphragms regards the connection of those elements to the building's walls. The nature of that connection mechanism is critical for two reasons. First, the connection must have sufficient strength to transfer shear forces (parallel to the resisting wall) from the diaphragm to the wall. Second, the exterior walls will all face problems similar to those faced by the West wing wall, as discussed above, if they are not supported (perpendicularly) by the floor slabs. Such support is not a problem when the transverse loads on the walls are pushing them into the slab, as the necessary support is provided through compression. Those loads are just as likely, however, to be applied in such a way as to pull the wall away from the floor structure. The vertical wall spans are large enough to result in wall to slab tensile loads exceeding the allowable value of 17.33 psi. Further, that allowable value may be irrelevant if the horizontal and vertical structures were not constructed in such a manner as to create a continuous masonry structure. Unfortunately, it is not possible to arrive at any definite conclusions on this matter without resorting to invasive exploration and materials evaluation in the undocumented portions of the structure. This, unfortunately, is true for both seismic and wind loads. While the seismic diaphragm loads are much higher than those caused by wind, without the ability to document the diaphragm strength, there is nothing to compare to loads from either source.

Remedial Action

Despite the small probability of significant seismic activity, the special significance of this particular structure may warrant a more detailed structural survey, and a more sophisticated structural analysis, than is included in the scope of the current study. Given the magnitude of some the problems identified in the initial study, however, it is unlikely that <u>all</u> of the present concerns would be laid to rest by such a program. The structural deficiencies remaining after a more detailed analysis could be addressed by structural reinforcement, though such a construction program is likely to involve significant financial expense and potential disruption (both temporary and permanent) to the function and/or historical integrity of the Building. The decision to proceed with either advanced structural analysis or seismic reinforcement must be based on the cost of such work relative to the actual risk of a seismic event and the ensuing damage.

Summary

The primary lateral systems of the Smithsonian Castle appear quite adequate for the gravity and wind loads they must resist. There are, however, some apparent problems regarding the performance of certain tower structures when exposed to code wind loads. While there is some comfort to be derived from the absence of any apparent failure of these structures during the past one hundred-fifty years, the analytical findings suggest a strong recommendation for more detailed inspection to locate any areas of tensile cracking or other distress. In addition, a more comprehensive and sophisticated analysis may relieve some of the concerns raised by the current investigation.

The investigation has generated some very real concerns for the structure's ability to resist seismic loads derived from the 1991 edition of the Uniform Building Code. The potential problems identified during this investigation include difficulties in both directions of the main lateral force resisting system, tensile overstresses in the Campanile structure, a high likelihood of damage or failure in the Octagonal Tower (and other, similar, parts of the building), and potential element failures in the upper levels of the exterior main building walls and the West wall of the Commons. In addition, the areas of floor construction which retain the brick or timber "topping" layer cannot be expected to provide adequate diaphragm strength to resist either seismic or wind loads. Finally, even where the diaphragm requirements are met by new concrete topping, there is no documentation of the connection of any of the horizontal structure (except the "new" fourth floor) to the vertical walls which ultimately resist lateral loads. Without that documentation, the ability of the diaphragms to transfer load to the walls, or to provide necessary perpendicular bracing for the walls, is uncertain and cannot be relied upon. In recognition of the complexity of these findings, a somewhat qualitative summary of the risk to various elements of the Building from lateral loads is presented in Table 3. The tensile stresses from which these assessments are derived are presented in Table 4.

The initial reaction to these finding may, understandably, be one of grave concern. It should be remembered, however, that these potentialities will only be realized if a significant seismic event does, in fact, occur. As discussed previously, Washington D.C. is extremely unlikely to experience a seismic event of any real significance. The area has no known history of anything but minor seismic events. Seismology is not, however, an exact science, and it would be irresponsible to state that no significant event will happen within any given time frame. The financial, functional, and historical costs of more detailed investigation, and/or structural reinforcement against damage, must be weighed against the historical value of the Building and its contents. Unfortunately, there is only one entity truly qualified to make such a qualitative evaluation- the Smithsonian Institution itself.

In Table 3, a "Low" risk categorization generally reflects analytical net tensile stresses near or below the selected allowable value of 17.33 psi, and indicates very little likelihood of significant damage. "Medium" reflects analytical stresses above the allowable value, but below 50 psi. This upper bound stress value makes some allowance for the conservatism of the material properties assumptions and analytical techniques, while noting areas where cracking and localized failure are likely, with some possibility of major damage or collapse. A "High" rating reflects net tensile stresses in excess of 50 psi, and indicates a high probability of major damage, including a very real possibility of total collapse.

ELEMENT	Seismic Damage Risk			Wind Damage Risk		
	Low	Medium	High	Low	Medium	High
MAIN LATERAL SYSTEM ANALYSIS						
North & South Central Walls	✓			✓		
North & South Range Walls	✓			✓		
North & Southeast Wing Walls	✓			✓		
North & Southwest Wing Walls		✓		✓		
Central / East Range Border			✓	✓		
Central/West Range Border			✓	✓		
East Range/Wing Border	✓			✓		
West Range/Wing Border	✓			✓		
Extreme East Wall	✓			✓		
Extreme West (Commons West) Wall	✓			✓		
Flag Tower	✓			✓		
North Tower	✓			✓		
Campanile			✓		✓	
Octagonal Tower			✓			✓
Southeast Tower			✓		✓	
LOCAL ELEMENT ANALYSES						
Top of North & South Central Walls		✓		✓		
Great Hall Commons	✓				N/A	
Commons West Wall		✓		✓		
Concrete Floors	✓			✓		
Brick/Timber Floors		?			?	
Sand/Rubble Floors		?			?	

Table 3 - Qualitative Summary of Risk of Damage Due to Lateral Loads

It is important to recognize that collapse of an element is likely to be followed by collapse of other portions of the Building supported by that element. For example, collapse of the Commons West Wall is likely to cause collapse of most or all of the Commons roof structure and portions of the North and South Commons Walls.

ELEMENT	Seismic Tensile Stress	Wind Tensile Stress
	(pounds per sq. in.)	(pounds per sq. in.)
MAIN LATERAL SYSTEM ANALYSIS		
North & South Central Walls	15	O.K. By Inspection
North & South Range Walls	Compression Only	O.K. By Inspection
North & Southeast Wing Walls	Compression Only	O.K. By Inspection
North & Southwest Wing Walls	44	O.K. By Inspection
Central / Range Border Walls	75	19
Range/Wing Border Walls	17	O.K. By Inspection
Extreme East Wall	O.K. By Inspection	O.K. By Inspection
Extreme West (Commons West) Wall	O.K. By Inspection	O.K. By Inspection
Flag Tower	Compression Only	O.K. By Inspection
North Tower	Compression Only	O.K. By Inspection
Campanile	168	50
Octagonal Tower	691	256
Southeast Tower	169	57
LOCAL ELEMENT ANALYSES		
Top of North & South Central Walls	20	O.K. By Inspection
Great Hall Commons	Compression Only	O.K. By Inspection
Commons West Wall	23	O.K. By Inspection

Table 4 - Summary of Analytical Stresses Due to Lateral Loads