McMULLAN & ASSOCIATES, INC.

SEISMIC EVALUATION REPORT

for the

The Castle Building

Prepared For

Smithsonian Institution
Washington, D.C.

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SEISMIC EVALUATION REPORT

I. INTRODUCTION

A. Purpose

In accordance with Executive Order 12941 dated December 1, 1994, an assessment of the seismic safety of all federal buildings is required per the provisions of ICSSC RP 5. These assessments are performed using FEMA 310 *"Handbook for the Seismic Evaluation of Buildings" – January 1998*. The work effort was broken into three phases; an evaluation phase, a screening phase, and a report phase.

The purpose of the evaluation phase was to provide a preliminary evaluation of the building and to determine the evaluation and analysis methodology needed for the subsequent phases. Our work in the evaluation phase served to identify potential structural and non-structural seismic deficiencies and to provide an initial indication of the significance of each deficiency. The scope of screening and report phases was to fully evaluate and document the seismic risks identified.

B. Structural Approach: FEMA 310

The FEMA 310 Handbook is intended to serve as a guide to structural engineers in identifying the earthquake-related life-safety risks posed by the structural and non-structural components of a building. The handbook details suggested procedures to be used by the evaluating engineer to determine the presence and severity of seismic deficiencies and to locate key weak links in building systems. The FEMA 310 evaluation approach is based primarily on experience gained from the evaluation of buildings damaged in areas of high seismicity, thus, some engineering judgment is required in implementing the evaluation results in areas of low-seismicity like Washington, DC.

The FEMA 310 approach is applicable to buildings of normal or high importance. The Castle Building is considered to be of normal importance. Buildings that are considered essential facilities require a greater level of earthquake resistance beyond "substantial life safety" and are required to undergo a more extensive review than buildings of normal importance.

The FEMA 310 evaluation is intended to evaluate existing buildings. An evaluation of proposed changes to an existing building is essentially a different task. The impact of the proposed modifications on the seismic resistance of the building is discussed in Section VIII.

We have performed the FEMA 310 evaluation based on the existing layout of the building.

During the next phase of the project it may be necessary to decide if seismic retrofit work for the existing building in its existing condition should be developed further or that only the seismic work appropriate to the modified building will be designed.

C. Non-Structural Element Approach

The approach taken for the evaluation of non-structural elements consists of the completion of the appropriate evaluation checklist for the non-structural sections contained in the FEMA 310 (Chapter 3).

D. Historic Considerations

Although the procedures for evaluating historic buildings are essentially the same as for any other buildings, it is recognized by FEMA that historic buildings require a more sensitive approach to seismic remediation than do typical buildings. In historic buildings, architectural and structural components are more linked. The interaction between architectural and structural elements may play an important role in the overall performance of the building. Disturbance of historic architectural components in order to perform structural evaluation and subsequent seismic rehabilitations of the structural system is often unacceptable.

Some feel that historic buildings should meet the safety levels of other buildings since they are a subset of the general seismic safety needs. Others feel that historic structures, because of their value to society, should meet a higher level of performance. In some cases a reduced level of performance has been allowed to avoid damaging historic fabric.

Our approach for this report has been to treat the Castle Building in the same way as we would any other building. However as the seismic retrofit work becomes more developed in later phases, we anticipate that consideration of the historic nature of the Castle Building to play a significant factor in the adoption of structural solutions.

II. EXECUTIVE SUMMARY

The Castle Building has several potential seismic risks related to the structural and non-structural components of the building. Each of these deficiencies can be mitigated. Specific retrofit recommendations are presented for each of the seismic deficiencies.

The items identified in the evaluation that require seismic mitigation and present the greatest potential risk in the event of a strong earthquake are as follows:

The connections between the floors and the brick walls do not appear to be adequate to transfer the lateral seismic load path. Typical connections consist of steel beam pockets at varying intervals. This connection is only sufficient to transfer limited direct shear to the walls but is not sufficient to transfer concentrated (localized) shear as a result of seismic load.

The connections between the brick walls and the roof are not adequate to maintain a viable load path for seismic forces. Thus the walls would break away from the roof and become laterally unstable.

It is likely that unbraced masonry piers in the Great Hall will deform during seismic events.

The existing floor infills, added to the building, have increased the seismic demand without adding any lateral force resistance system (such as, additional shear walls or diagonal bracing.)

III. BUILDING DESCRIPTIONS

A. Configuration Description

The Smithsonian Institution Building, the "Castle", is the original Smithsonian Museum which contained exhibition rooms, lecture halls, offices, living quarters, and a library. Today it serves primarily as an administrative office building, library, and research center. The building consists of a central section and a partially independent North and South towers. There are also smaller towers (The NW, the SE, the West, the Octagonal and the Campanile). In addition to the central section, there are the east and the west wings and ranges. See Appendix A7.

The original structural system was masonry walls, piers and columns, with wood beams and floors. However, as a result of a collapse during construction, the 1865 post-fire reconstruction, and several alternations, the predominant floor construction has been changed to brick arches on iron beams. (See Appendix A7)

B. Gravity Structural System

The existing exterior walls of the building appear to be stone walls below the 3rd floor and be a mixture of brick and stone above the 3rd floor. These walls are approximately 2'-0" thick. Typically, the exterior walls are perforated by arched openings and windows that vary in height and width. The interior walls are brick with varying thickness (typically 18"). The original floors and infills in the Great Hall (the Central Section) are supported on masonry piers and columns. The interior walls foundation system consists typically of tapered red and blue cut stones. The exterior wall foundation consists typically of rubble stone.

C. Lateral Force-Resisting System:

The existing lateral force resisting system for the building consists of a series of un-reinforced masonry shear walls, which historically are one of the most vulnerable force resisting systems to damage or collapse during an earthquake.

The Castle Building contains multiple towers that have separate lateral resisting system from the central building. Where the towers are joined to the main building, the walls will act together (more rigid). If the towers were to separate during a seismic event, something that appears likely, then they will be much more slender and less rigid. This could lead to their collapse.

IV. SEISMIC EVALUATION METHODOLOGY

A. Structural

Once the Region of Seismicity and the desired Level of Performance for the building was determined, the checklists required were compiled for review in accordance with Table 3-2 of FEMA 310.

After reviewing the available documentation for the Castle Building, site visits were performed to observe existing structural conditions and to look for evidence of seismic deficiencies.

The FEMA 310 checklist 3.6 was used as the basis for the structural review of the building. A Tier 1 analysis was performed in order to facilitate the building evaluation. The non-compliant (NC) evaluation statements serve as a 'checklist' for a potential seismic weakness, specific to a particular class of building.

Statements for a structural element or system that are answered as (C) generally indicate an acceptable condition not requiring further evaluation. Conversely, statements for which the response is (NC) lead to further engineering analysis. General methodology for this analysis is specified in FEMA 310 and is used in this study. In some cases, certain evaluation statements are 'not applicable' (N/A). In addition, a number of evaluation statements had been described as 'indeterminate' pending more detailed analysis or document review is required.

B. Non-Structural

Checklist 3.6 was used for the initial phase of review for the non-structural components for the building. The general methodology used to complete the non-structural review consisted of a review of the available architectural drawings and a building survey where typical conditions were examined for each component category. Individual assessments were made of each of the components listed in the Evaluation Checklist.

V. FINDINGS – STRUCTURAL

A. FEMA 310 Evaluation Checklists:
The following checklists were evaluated:

- 3.6 Region of Low Seismicity Checklist.

B. Summary of Earthquake Risks:

The following structural items were determined as non-compliant in evaluations statements contained in the checklist.

1. Load Path - In a building such as the Castle Building where relatively tall, heavy walls are used to provide both lateral-force resistance and support gravity loads, the seismic force resistance required at the connection between the walls and floor/roof diaphragms is very high. The seismic risk in this instance is that walls that are not positively anchored (which seems

to be the case in the Castle Building) to the floor/roof diaphragms may separate from the structure of the building and become laterally unstable. This could lead to a partial collapse of the floor and/or roof and possible overturning of the wall itself. This hazard is most critical at the roof and upper floors since building response typically amplifies the ground motion with increasing height above the base. The calculations show that the existing diaphragm (original floor brick arches and fill) is not adequate to transfer the seismic load path at the shear wall. In addition, the diaphragm's connection to the walls (via beam pockets every 4'-0" on center) is not adequate to transfer the seismic shear force load to walls.

The non-original floor infills, especially in the central section, have added to the overall seismic demand on the walls without providing any additional lateral force resistance. In some instances, the variations in the mass of the floors between concrete and brick causes eccentricity to develop between the center of mass of the building and the center of rigidity for the building which adds to the overall seismic demand.

2. Stone/Brick Walls Shear Resistance – The calculations show that the reinforced brick/stone walls in the east west direction are not sufficient to resist the seismic load. The stress caused in a seismic event will be greater than 30 psi which is allowable for new brick construction. The allowable stress in the Castle Building (once tested), we believe will be in range of 20 psi, similar or less than the Arts and Industries Building tested results.

3. Towers – A preliminary analysis on the towers was performed. It is difficult to determine the stress magnitude at the main building to tower connection. It is obvious that the main building is much stiffer than the tall slender towers. Therefore, the period developed by the tower is significantly different than what is developed by the main stiffer part of the building. This could lead to stressing the tower / main building connection, and possibly cracking and then total separation. The towers will be considered slender when height/depth ratios are more than 2.5. The following towers are of significant concern:

    a. The Campanile Tower: The stone portion of this tower extends 58 ft. from the 4th floor of the east range. The roof of this tower extends an additional 24 ft., making the unbraced length equal to 82 ft. above a braced diaphragm. This tower is 16 ft. sq. The ratio of height/depth = 82/16 = 5.1 > 2.5.

    b. The Flag Tower: This tower separates from the north wing from the fifth floor up. The unbraced length to the top of the stone tower is approximately 76 ft. This tower is 22 ft. sq. The ratio of height/depth = 76/22 = 3.45 > 2.5. In addition the cross section and mass of this tower changes drastically with each floor.

    c. The West Tower: If it can be assumed that this tower is braced at the roof eve level (not a conservative assumption). This means that the stone portion of the tower cantilevers 38 ft. and the tower roof extends an additional 19 ft.. The total unbraced length equals 57 ft. This tower is 12'-6" sq. The ratio of height/depth = 57/12.5 = 4.56 > 2.5.

d. All of the other small towers are slender with an unbraced length/depth ratio > 2.5 (the Octagonal, the Northwest and the Southeast tower).

VI. FINDINGS – NON-STRUCTURAL

A. FEMA 310 Non-Structural Evaluation Checklists:

- 3.6 Region of Low Seismicity Checklist

B. Summary of Earthquake Risks

1. Glazing – There are several very large panes of glass in the building. Some of them are above some of the entrances. They do not appear to be safety glass. The seismic hazard is that glazing may shatter due to building drift or racking during an earthquake. This becomes more critical where large panes of glass are located over egress areas.

2. Parapets – The parapets at the top of the Central Section, the East Wing and South Towers are very tall with ornamentation mounted to it at the top of them. An investigation of the stability of these slender parapets is required. The un-reinforced masonry parapets are typically a major falling hazard during an earthquake.

3. Roof Ornamentations – All of the towers (except the flat roofed ones) have ornamentations on the top of their roofs. The anchorage of the ornamentations is not verified at this stage. However, we believe that they could be at risk of falling during a seismic event.

4. Chimneys – Although the existing chimneys are built integral with the walls of the building, the overall slenderness of the chimneys make them a potential seismic hazard. Unreinforced masonry chimneys have historically been highly vulnerable to damage in earthquakes. Typically, chimneys, which extend above the roof more than twice the least horizontal dimension of the chimney (this is the case in the west and east wings), the chimneys could crack above the roof line and then become dislodged and fall through the roof creating a life safety hazard.

5. Equipment – Mechanical equipment such as fans and pumps in the attic space and the mechanical rooms on the second floor, are essentially unrestrained for seismic forces and are likely to become inoperable during an earthquake. The interconnecting piping is also likely to suffer damage.

VII. RECOMMENDATIONS

A. FEMA 310 Evaluation Checklists Retrofit Options

1. Load Path - Continuous and adequate connections between the diaphragms and shear walls are required. This could be achieved by strengthening the existing beam pocket connection and/or by providing a new connection utilizing steel angles and adhesive bolts. However prior to strengthening these connections, the existing brick arch floors should be strengthened and modified in order to act as an adequate diaphragm. Any strengthening of this system may affect its historic value. Grouting or the replacement of the existing lime sand and stone fill (see Appendix 7) may be required. The existing flagstone tile will likely need to be removed in places. Another option may be to provide a flat steel reinforcing plate reinforcing continuously connected to the bottom flanges of the beams.

2. Shear Walls – Additional concrete or masonry shear walls should be utilized. The East Range and Wing are rigid due to the fact that they contain large numbers of interior brick walls. The Central Section and the West Range and Wing do not have any interior walls and thus are less rigid than the East Range and Wing. Therefore the new shear walls should be added to the Central Section. These walls should extend from the basement level to below the 4th floor. New shear walls should be added below the first floor of the West Range and Wing. The brick columns above the first floor of the West Range require strengthening. This could be accomplished with a steel plate encasement system. The Commons Hall walls requires strengthening or bracing between the first floor and the roof.

3. Towers – We recommend that at the next level of analysis that a complete computer modeling of the entire building, including all the towers be performed. Only accurate finite element modeling will predict the behavior of each element of the building. This preliminary analysis indicates that some of the towers may separate from the main building during a seismic event and fall. For these towers, we recommend that they be reinforced with a steel plate box system. This system should be attached to the entire inside face of the tower. The plate box system should extend, at a minimum, 10 feet below the point at which the tower is connected to the main building. The plate system will need to be stiff enough in order to relieve stresses from the existing walls.

4. Glazing – Consideration should be given to securing the historic glazing glass above the entrance areas with safety glass mounted in a frame, which can accommodate the appropriate level of lateral drift.

5. Parapets – A mechanical connection may be required to brace the parapet wall. This should apply to the wall with height to depth ratio that is more than 2:5. Since there is no existing documentation of the building parapet detail, an investigation, which includes some selective demolition, is

needed to confirm the existing anchorage conditions prior to the design of the new bracing connections.

6. Chimneys – Typically, seismic bracing of the chimneys is more expensive to perform than it would be to remove the chimneys in their entirety. If the chimneys cannot be removed or filled, then a steel plate box system should be used to line the chimney from inside. Non-shrink grout should be poured between the chimney brick and the steel plate box. This box system should span between the upper portion of the chimney and 5' below the existing roof structure.

7. Equipment – Generally, the loss of service of several pumps and fans for non-life safety building equipment is not considered a significant seismic hazard to merit seismic mitigation.

B. Priorities

Prior to any seismic mitigation, a complete and more sophisticated investigation , testing and analysis should be performed. The seismic mitigation priorities are ranked as follows:

1. Strengthen of the existing original floor system is required in order to provide an adequate diaphragm. In, addition strengthening of the wall anchorage (load path) between the existing masonry walls and the roof and floor diaphragms is required. This also applies to all stairs in the towers and ranges.

2. Addition of shears walls in the Central Section and West Range and Wing. Lateral bracing of the West Range and Wing roofs is needed.

3. Deficient towers need to be reinforced with possibly a steel plate box system on the entire inside face.

4. Retrofit, reinforce or brace the existing chimneys.

5. Brace the existing parapet and ornamentation above the parapet.

6. Strengthen or replace the existing glass above the entrances with safety glass mounted in a frame, which will accommodate some racking/lateral drift.

7. Brace the ornamentation over the existing tower roofs.

VIII. SEISMIC EFFECT OF PROPOSED BUILDING MODIFICATIONS

The major structural modifications being considered in the scope of the renovation work that impact the seismic demand/capacity of the existing building; is lowering the basement slab and possibly the addition of an under-slab basement space. The extensive underpinning work proposed to lower the basement could negatively impact the lateral force resistance of the building by increasing the effective length and slenderness of the masonry piers. In addition, it will be necessary to carefully detail the connection between the bottom of the brick piers and the new concrete underpinning such that to ensure that shear can be transferred.

Additional study shall be performed to determine the impact of the proposed building modifications on the seismic force resistance of the existing building. To mediate the slenderness of the basement walls and columns, additional shear walls may be required. The location and amount of shear walls will depend on the area to be lowered.

Due to gaps in the documentation of the existing structure, a significant amount of site survey and possibly destructive structural testing will be required in subsequent phases of the project in order to make an accurate assessment of the existing structure.

IX. SEISMIC MITIGATION COST ESTIMATE:

At this basic level of analysis and given the fact that this particular building is an extremely complicated structure, we believe that to provide a cost estimate at this time may be highly problematic. However, from past experience, we believe that retrofitting the Castle Building will be more intrusive on the historic nature of the building, and more extensive and expensive than the AIB seismic retrofitting. Our best estimate at this time is in the range of 5-8 million dollars.

X. APPENDICES

A1: Table 3-2 – Check List for Tier Evaluation
A2: Table 3-3 – Further Evaluation Requirements
A3: Figure 1 – Spectral Acceleration Map
A4: Region of Low Seismicity Check List
A5: Calculations
A6: Summary Data Sheet
A7: Sketches

A1: Table 3-2. Checklists Required for a Tier 1 Evaluation

Region of Seismicity	Level of Performance[2]	Required Checklists [1]					
		Region of Low Seismicity (Sec. 3.6)	Basic Structural (Sec. 3.7)	Supplemental Structural (Sec. 3.7)	Geologic Site Hazard and Foundation (Sec. 3.8)	Basic Nonstructural (Sec. 3.9.1)	Supplemental Nonstructural (Sec. 3.9.2)
Low	LS	✔					
	IO		✔		✔	✔	
Moderate	LS		✔		✔	✔	
	IO		✔	✔	✔	✔	✔
High	LS		✔	✔	✔	✔	✔
	IO		✔	✔	✔	✔	✔

[1] A checkmark (✔) designates that the checklist that must be completed for a Tier 1 evaluation as a function of the region of seismicity and level of performance.

[2] LS = Life-Safety; IO = Immediate Occupancy; defined in Section 2.3.

A2: Table 3-3. Further Evaluation Requirements[1]

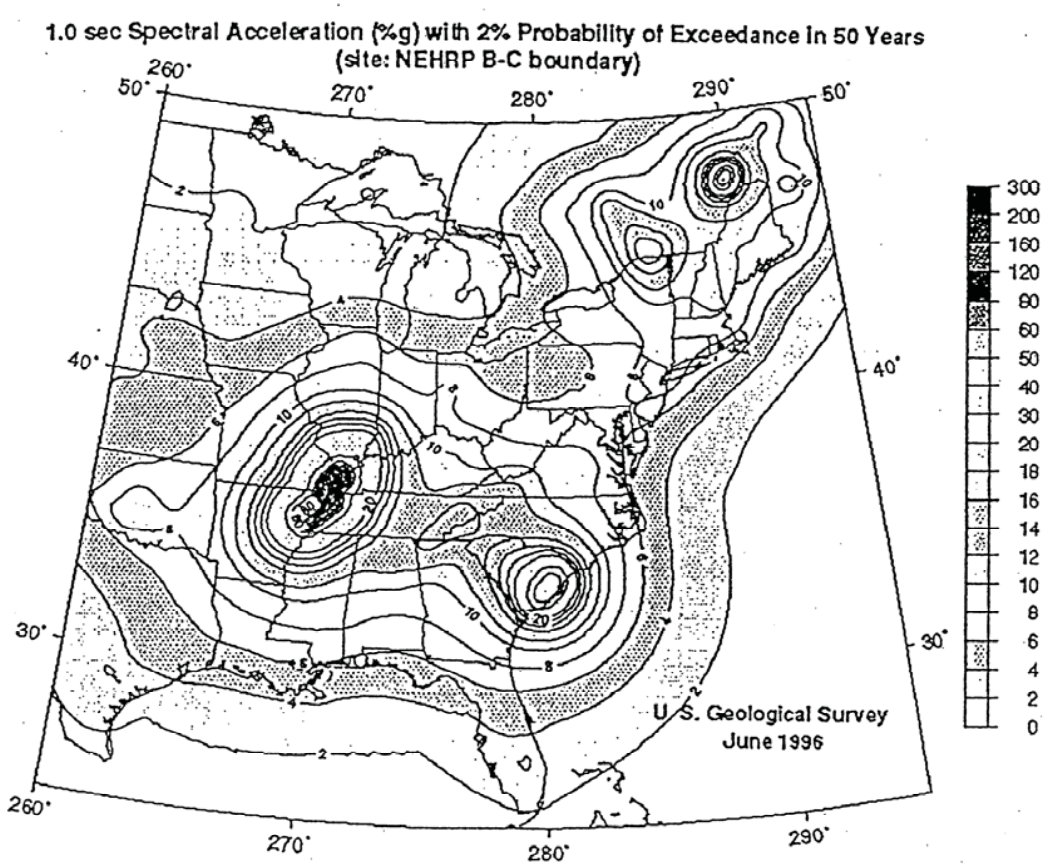
	Number of Stories[2] beyond which a Full-Building Tier 2 Evaluation is Required					
	Regions of Seismicity					
	Low		Moderate		High	
Model Building Type	LS	IO	LS	IO	LS	IO
Wood Frames						
Light (W1)	NL	2	4	2	2	2
Multistory, Multi-Unit Residential (W1A)	NL	3	4	2	2	2
Commercial and Industrial (W2)	NL	2	4	2	2	2
Steel Moment Frames						
Rigid Diaphragm (S1)	NL	3	6	T2	3	T2
Flexible Diaphragm (S1A)	NL	3	6	T2	3	T2
Steel Braced Frames						
Rigid Diaphragm (S2)	NL	3	6	2	6	2
Flexible Diaphragm (S2A)	NL	3	6	2	6	2
Steel Light Frames (S3)	NL	1	2	1	2	1
Steel Frame with Concrete Shear Walls (S4)	NL	4	6	4	6	3
Steel Frame with Infill Masonry Shear Walls						
Rigid Diaphragm (S5)	NL	2	6	T2	2	T2
Flexible Diaphragm (S5A)	NL	2	6	T2	2	T2
Concrete Moment Frames (C1)	NL	2	6	T2	6	T2
Concrete Shear Walls						
Rigid Diaphragm (C2)	NL	4	6	4	6	3
Flexible Diaphragm (C2A)	NL	4	6	4	6	3
Concrete Frame with Infill Masonry Shear Walls						
Rigid Diaphragm (C3)	NL	2	6	T2	2	T2
Flexible Diaphragm (C3A)	NL	2	6	T2	2	T2
Precast/Tilt-up Concrete Shear Walls						
Flexible Diaphragm (PC1)	NL	1	2	T2	2	T2
Rigid Diaphragm (PC1A)	NL	1	2	T2	2	T2
Precast Concrete Frames						

	Number of Stories[2] beyond which a Full-Building Tier 2 Evaluation is Required					
	Regions of Seismicity					
	Low		Moderate		High	
With Shear Walls (PC2)	NL	4	6	4	4	T2
Without Shear Walls (PC2A)	NL	T2	6	T2	3	T2
Reinforced Masonry Bearing Walls						
Flexible Diaphragm (RM1)	NL	3	6	T2	3	T2
Rigid Diaphragm (RM2)	NL	3	6	3	3	2
Unreinforced Masonry Bearing Walls						
Flexible Diaphragm (URM)	NL	1	NL	T3	NL	T3
Rigid Diaphragm (URMA)	NL	1	6	T3	3	T3
Mixed Systems	NL	2	6	T2	6	T2

[1]A full-Building Tier 2 or Tier 3 Evaluation shall be completed for buildings with more than the number of stories herein.
[2]Number of stories shall be considered as the number of stories above lowest adjacent grade.
NL – No Limit (No limit on the number of stories).
T2 – Tier 2 (A Full-Building Tier 2 Evaluation is required; proceed to Chapter 4).
T3 – Tier 3 (A Tier 3 Evaluation is required; proceed to Chapter 5).

A3: Spectral Acceleration Map

8.2 Figure 1: Spectral Acceleration Map



Appendix A4

3.6 Region of Low Seismicity Checklist

This Region of Low Seismicity Checklist shall be completed when required by Table 3.2.

Each of the evaluation statements on this checklist shall be marked compliant (C), non-compliant (NC), or not applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 evaluation procedure; the section numbers in parentheses following each evaluation statement correspond to Tier 2 evaluation procedures.

Structural Components

C **NC** N/A LOAD PATH: The structure shall contain load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1.1)

Comment: The strength and connectivity of the diaphragms does not appear adequate to transfer seismic shear force from the masonry walls.

C **NC** N/A WALL ANCHORAGE: Exterior masonry walls, that are dependent on the diaphragm for lateral support, shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check Procedure of Section 3.5.3.7. (Tier 2: Sec. 4.6.1.1)

Comment: Diaphragm connections appear to be not adequate for out-of-plane forces. Floor beams are pocketed @ 4' – 0" on center.

Geological Site and Foundation Components

C NC N/A FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.1)

Nonstructural Components

C NC N/A EMERGENCY LIGHTING: Emergency lighting equipment shall be anchored to prevent falling or swaying during an earthquake. (Tier 2: Sec. 4.8.3.2)

C **NC** N/A GLAZING: Glazing in curtain walls and individual panes over 16 square feet in area, located up to a height of 10 feet above an exterior walking surface, shall be safety glazing. Such glazing located over 10 feet above an exterior walking surface shall be laminated annealed or laminated heat strengthened safety glass that will remain in the frame when cracked. (Tier 2: Sec. 4.8.4.9)

C **NC** N/A PARAPETS: There shall be no laterally unsupported unreinforced masonry parapets or cornices with height-to-thickness ratios greater than 1.5 in regions of high seismicity and 2.5 in regions of low or moderate seismicity above the highest point of anchorage to the structure. (Tier 2: Sec. 4.8.8.1)

Comment: All of the parapets with ornamentation need more investigation.

C NC N/A CHIMNEYS: Masonry chimneys shall be anchored to the floor and roof. (Tier 2: Sec. 4.8.9.2)

Comment: Chimneys are slender and require investigation.

C **NC** N/A STAIRS: Walls around stair enclosures consisting of unbraced hollow clay tile or unreinforced masonry shall be braced to the structure for seismic forces. (Tier 2: Sec. 4.8.10.1)

Comment: No adequate shear transfer. Need more investigation.

C NC N/A EMERGENCY POWER: Equipment used as part of an emergency power system shall be anchored. (Tier 2: Sec. 4.8.12.1)

C **NC** N/A EQUIPMENT: Equipment mounted on vibration isolators shall be equipped with restraints or snubbers. (Tier 2: Sec. 4.8.12.4)

Appendix A5 **Calculations**

SI := 0.14 (Map – 11) Soil Classification = E

Ss := 0.28 (Map – 9)

Fv := 3.4 Table 3-5

Fa := 2.4 Table 3-6

SD1 := 2/3 · Fv · SI SD1 = 0.317

SDs := 2/3 · Fa · Ss SDs = 0.448

$T = C_t h_n^{\ 0.75}$ h_n= 76' 0" for the central section (the largest section)

$T := 0.020 \cdot 76^{0.75}$ T = 0.515 seconds

Sa := SD1/T Sa = 0.615 > 0.448 therefore Sa = SDs

C := 1.0(Table 3-4)

Estimated Central Section Dead Load

Roof

Trusses + Purlins	=	12 psf. (assumed)
Metal Deck	=	3 psf.
Wood Decking	=	5 psf.
Vapor Barrier	=	1 psf.
Metal Roof	=	1 psf.
MEP	=	6 psf.
Ceiling	=	2 psf.
		30 psf.

1st Floor

Typical Brick Arches and fill	=	155 psf.
2" Flagstone	=	27 psf.
MEP	=	8 psf.
1" plaster concrete	=	10 psf.
		200 psf.

3rd Floor Slab

Typical Brick Arches and fill	=	155 psf.
Ceiling	=	12 psf.
Mechanical	=	8 psf.
		175 psf.

4th Floor

Concrete Slab Infill	=	35 psf.
Beams/Joists	=	5 psf.
Mechanical	=	8 psf.
Ceiling	=	7 psf.
		55 psf.

Attic Floor	=	32 psf.
Ceiling	=	5 psf.
Mechanical	=	8 psf.
		45 psf.

Central Section Area ≈ 215 x 58 = 12470 SF.

Weight of Slabs and Roof in kips:
(30 + 200 + 175 + 55 + 45) (12470) / 1000 = 6297 kips

Estimated Weight of Exterior Walls
28” avg. of stone/brick and mortar (20% open) = approximately 268 psf.
(268 psf) + (215+215 + 50 + 50) (76’ high) / 1000 = 10,795 kips

Masonry Piers
28 x 580 x 64 / 1000 = 1039 kips

Mezzanine Levels (estimated)	=	500 kips
Partitions (estimated)	=	300 kips
		800 kips

Total Dead Load in the Central Section = 18931 kips

<u>East Range:</u>

1st, 2nd, 3rd floors =

Typical Brick Arches and fill	=	155 psf.
MEP	=	8 psf.
1” plaster	=	10 psf.
Floor Finish	=	5 psf.
		178 psf.

Roof:

Structure	=	30 psf.
Mechanical	=	8 psf.
Ceiling	=	7 psf.
		45 psf.

Area of East Range = 64 x 45 = 2880

Weight of Floors [(178 x 3) + 45] 2880 / 1000 = 1668 kips

Weight of Exterior Walls:
24" avg. wide stone/brick 40% open = 173 psf
(66 x2) x 47" high x 173 / 1000 = 1073 kips

Partitions / Interior Walls: 18" brick/stone ≈ 220 psf
(170 long x 47" high x 220 psf) / 1000 = 1758 kips

Total Dead Load of East Range = 4499 kips

<u>East Wing</u>
1^{st}, 2^{nd}, 3^{rd}, 4^{th} Floors

Typical Brick Arches and Fill	=	155 psf.
MEP	=	8 psf.
1" plaster	=	10 psf.
Floor Finish	=	5 psf.
		178 psf.

Roof = 45 psf.

Area of East Range = 48 x 76 = 3648

Weight of Floors = [(178 x 4) + 45] (3648) / 1000 = 2762 kip

Exterior Walls = 24" x 20% open = 230 psf

Weight of Exterior Walls = [(48 x 2) + 98] x 60' high x 230 / 1000 = 2677 kips

Average Weight of Partitions = 198 length x 176 (average weight) x 60' high / 1000 = 2090 kips

Total Dead Load of East Wing = 7529 kip

West Range

1^{st} Floor

Typical Brick Arches and Fill	=	155 psf.
MEP	=	8 psf.
1" plaster	=	10 psf.
Floor Finish	=	5 psf.
		178 psf.

Area of 1^{st} Floor = 64 x 48 = 3072 sf.

Weight of 1^{st} Floor – 178 x 3072 / 1000 = 547 kips

2nd Floor mechanical room

Assumed Weight = 178 psf

Area of Mechanical Room = 61 x 10.5 = 640 sf

Weight of 2nd Floor = 178 x 640 / 1000 = 114 kips

Attic = 45 psf

Area of attic = 61 x 29.5 = 2410 sf.

Weight of attic = 45 x 2410 / 1000 = 108 kips

Weight of roof = 45 psf x 60 x 48 / 1000 = 130 kips

Exterior Walls = 24” with 30% open = 202 psf
(69 x 2) x 39’ high x 202 / 1000 = 1087 kips

Interior Columns from 1st floor to attic = 540 PLF x 12 columns x 15’high / 1000 = 96 kip

Interior Walls below 1st floor = 206’ length x 9’ high x 216 psf / 1000 = 400 kip

Interior Wall above columns = 17’ high x 216 psf (60x2) x 50% open / 1000 = 220 kip

Total Dead Load for West Range = 2702 kips

The West Wing (Commons Hall) will not be used in this analysis since it does not share the same diaphragms.

THE DEAD LOAD OF THE CENTRAL SECTION AND EAST AND WEST RANGES AND EAST WING
= 18931 + 4499 + 7529 + 2702 = 33661 kips

V = C Sa W
V = 1 x .448 x 33661 = 15080 kips demand

Capacity of shear walls in the east west direction

Exterior walls:

A_m at central section = [215 x 2 x 28” x 12” x .8 (openings)] = 115584 in^2

A_m at east range = (64 x 2 x 24” x 12” x .6) = 22118 in^2

A_m at east wing = (50.5 x 2 x 24” x 12” x .8) = 23270 in^2

A_m at west range = (68 x 2 x 24” x 12 x .7) = 27418 in^2

Total for exterior walls = 188390 in^2

Interior Walls

A_m for east range and wing = (175' length x 14" x 12") = 29400 in^2

Total A_m = 188390 + 29400 = 217790 in^2

m = 1.5 Vj = 15080

V= 1/m x (Vj/Am)

V = 1/1.5 (15080/217790) = 46 psi
This value is an average. The walls in the east range and wing will have different stresses than the walls in the central section since the east range and wing are much stiffer than the rest of the sections.

Allowable for new brick is 30 psi

46 > 30 psi not ok

In addition, we believe that the allowable shear stress in the Castle Building brick and stone walls, once tested, will be less than 30 psi. We estimate it to be around 20 psi.

Additional shear walls should be provided. The area that shear wall needs to be provided is the central section, and the west range and wing.

By inspection, the west wing (common hall) walls are slender and are not sufficient to resist the seismic load.

Diaphragm analysis:

Assume (simplified analysis) that stress at east range interior walls connection to slab ≈ (46 psi x 14 x 12) / 1000 = 7.728 kip/ft.

Strength of one foot section of slab =
Assume consistent depth of lime sand, broken stone or gravel fill + brick = 13"

From past experience, lime sand mortar allowable stress is < 15 psi. Assume 10 psi.

(7.728 x 1000) / (13 x 12) = 49 psi > 10 psi

Slab needs to be strengthened to act as a diaphragm.

Assuming that the shear load is transferred to walls through the existing steel iron beam pocket, every 4'-0" on center.
Load = 7.728 x 4 = 30.9 kips

Bearing area between brick and steel iron beam = 8" (assumed bearing length) x 15" (deep) = 120 in^2

(30.9 x 1000) / 120 = 257.5 psi > .2 x 700 psi = 140 Not good

The 700 psi was determined to be the average for the Arts and Industries Building (using flat jack testing). The AIB was built in the 1890s.

Strengthening of the diaphragms connection to brick wall is required, in addition to strengthening the diaphragms themselves.

BUILDING DATA

Building Name: Castle Building — Date: February 28, 2002
Building Address: Washington, D.C.
Latitude: — Longitude: — By:
Year Built: 1855 — Year(s) Remodeled: various — Original Design Code: Unknown
Area (sf): 60,000 + — Length: 460' ± — Width (ft): varies
No. Stores: 4 typical — Story Height: varies — Total Height varies

USE ☐ Industrial ☒ Office ☐ Warehouse ☐ Hospital ☐ Residential ☒ Educational ☐ Other

CONSTRUCTION DATA

Gravity Load Structural System: Unreinforced masonry pier and masonry arches; some steel beams and columns.
Exterior Transverse Walls: Unreinforced brick/stone — Openings?: Yes
Exterior Longitudinal Walls: Unreinforced brick/stone — Openings?: Yes
Roof Materials/Framing: Metal roof decking or wood decking supported by steel trusses.
Intermediate Floors/Framing: Mostly brick arches. Some concrete fill over joists.
Ground Floor: Slab-on-grade.
Columns: Mostly masonry piers. Some steel columns — Foundation: Stone footings.
General Condition of Structure: Good.
Evidence of Setting?: Some at South Tower and Commons (see structural replanning study dated December 9, 1988).

Special Features and Comments: The building is made up by many separate sections and towers.

LATERAL FORCE RESISTING SYSTEM

	Longitudinal	Transverse
System:	Unreinforced masonry/stone.	Unreinforced masonry/stone
Vertical Elements:	Unreinforced masonry piers and stone/brick walls.	Unreinforced masonry piers and stone/brick walls
Diaphragms:	Brick arches and metal deck.	Brick arches and metal deck.
Connections:	Riveted and embedded.	Riveted and embedded.

EVALUATION DATA

Spectral Response Accelerations: S_S= 0.28 — S_1= 0.14
Soil Factors: Class= E — F_A= 2.4
Design Spectral Response Accelerations: S_{DS}= 0.448 — S_{D1}= 0.317
Region of Seismicity: Low — Performance Level = Life safety.
Building Period: T= 0.515 s
Spectral Acceleration: S_A= $0.615 > 0.448 \therefore$ use 0.448
Modification Factor: C= 1.0 — Building Weight: W= $33,661^K$
Pseudo Lateral Force: $V=CS_AW=$ 15080^K

BUILDING CLASSIFICATION: Unreinforced masonry/stone shear walls with flexible diaphragm – Type 15.

REQUIRED TIER 1 CHECKLISTS

	Yes	No
Basic Structural checklist	☒	☐
Supplemental Structural checklist	☐	☐
Geologic Site Hazards and Foundations checklist	☐	☐
Basic Nonstructural checklist	☐	☐
Supplemental Nonstructural checklist	☐	☐

FURTHER EVALUATION REQUIREMENT: Towers, diaphragm and diaphragm connection to walls.