NISTIR 6315

Supporting Document for Rehabilitation Cost Estimates of FEMA Existing Buildings

H. S. Lew

Building and Fire Research Laboratory
Gaithersburg, Maryland 20899

United States Department of Commerce
Technology Administration
National Institute of Standards and Technology

NISTIR 6315

Supporting Document for Rehabilitation Cost Estimates of FEMA Buildings

H. S. Lew

Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, MD 20899

Prepared for

Federal Emergency Management Agency
Washington, D.C. 20472

March 1999

U.S. Department of Commerce
William M. Daley, *Secretary*
Technology Administration
Gary R. Bachula, *Acting Under Secretary for Technology*
National Institute of Standards and Technology
Raymond G. Kammer, *Director*

PREFACE

Pursuant to Executive Order 12941, *Seismic Safety of Existing Federally Owned or Leased Buildings*, all Federal agencies are required to inventory their owned and leased buildings, and to estimate the costs of mitigating unacceptable seismic risks in that inventory. The National Institute of Standards and Technology (NIST) performed these requirements for the Federal Emergency Management Agency (FEMA) under contract EMW-96-IA-0184.

The building data were collected and tabulated by Ann Bieniawski. Field evaluation of the selected buildings were performed by Drs. H. S. Lew and Michael Riley of NIST and Professor Bijan Mohraz of the Southern Methodist University who was on an "Intergovernmental-Personnel-Act" appointment at NIST. The buildings were evaluated jointly by Dr. Lew and Prof. Mohraz.

ABSTRACT

This report presents the results of seismic evaluation and cost estimates carried out by the National Institute of Standards and Technology (NIST) for rehabilitation of existing buildings owned by the Federal Emergency Management Agency (FEMA). The seismic evaluation and rehabilitation cost estimates were carried out in response to Executive Order 12941, *Seismic Safety of Federally Owned or Leased Buildings*. The seismic evaluation was performed based on ICSSC RP4, *Standards of Seismic Safety for Existing Federally Owned or Leased Buildings and Commentary,* and FEMA 178, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*. Rehabilitation costs were estimated using FEMA 156 and 157, *Typical Costs for Seismic Rehabilitation of Existing Buildings, Second Edition, Volumes 1 and 2*.

FEMA owns 137 buildings. Of these, 125 buildings are located in Maryland and Virginia (low seismic regions). Ten buildings were selected for evaluation, of which seven are located in Maryland and Virginia, two in Massachusetts (moderate seismic region), and one in Washington (high seismic region).

All sites where the 10 buildings are located were visited by the NIST team. None of these buildings has a complete set of architectural and structural drawings, particularly old buildings such as those at the Emmittsburg, Maryland site. For those buildings which are judged to have deficiencies according to the checklist in FEMA 178, additional analyses were carried out to determine whether in-situ structures are adequate for "life safety." If passed for life safety evaluation, the structure is judged to have no deficiencies.

Rehabilitation costs for the non-evaluated buildings were derived from the rehabilitation costs of the evaluated buildings. The location of building is considered in the estimation of the rehabilitation cost. The cost estimates are also adjusted to 1998. The rehabilitation costs include structural, non-structural, finishing and administration costs. The total estimated rehabilitation cost for the FEMA buildings is $13 910 000.

Keywords: buildings; costs; evaluation; existing; rehabilitation; seismic damage; structural performance; survey.

TABLE OF CONTENTS

1. Introduction

Executive Order 12941, *Seismic Safety of Federally Owned or Leased Buildings*, requires that Federal agencies develop a complete inventory of their owned and leased buildings, evaluate owned buildings for seismic performance, and develop cost estimates to rehabilitate those buildings found to be seismically deficient. The inventory, evaluations and cost estimates are to be completed following guidance published by the Interagency Committee on Seismic Safety in Construction (ICSSC) as RP4, *Standards of Seismic Safety for Existing Federally Owned or Leased Buildings and Commentary*; RP5, *ICSSC Guidance on Implementing Executive Order 12941 on Seismic Safety of Existing Federally Owned or Leased Buildings*; and TR-*17, How-to Suggestions for Implementing Executive Order 12941 on Seismic Safety of Existing Federal Buildings, A Handbook.*

This report presents the inventory of the FEMA buildings, the selection of buildings for evaluation, the descriptions and evaluation results of the evaluated buildings, and the rationale and process used to estimated the cost of rehabilitation of non-evaluated buildings. In addition to this written document, the inventory and cost data are prepared in electronic form which could be used in the Federal government-wide inventory and seismic rehabilitation cost development.

2. Inventory of FEMA Buildings

FEMA provided a list of sites where they owned buildings. The NIST personnel collected the building inventory data from the site representatives either by visits or by telephone. Because the Berryville, Virginia and Emmittsburg, Maryland sites had more than 90 percent of the FEMA buildings, these locations were visited.

A total of 137 buildings that FEMA owns are distributed as follows.

- Berryville, Virginia - 87
- Bothell, Washington - 1
- Denton, Texas - 6
- Emmittsburg, Maryland - 38
- Maynard, Massachusetts - 2
- Olney, Maryland - 3

A database of the building inventory was created and is attached with this report (Attachment A). This inventory includes all buildings listed in descending order of the "State Code." Other pertinent information about the buildings as specified in Section 2.3 of RP5 are also given according to the format described in Section 5.0 of TR-17.

This database identifies that forty-five (33 %) of the buildings are exempt from seismic evaluation per RP4, Section 1.3. The reasons for exemption are given in the database according to Table 5-2 of TR-17. The most common reason for exemption is that a building has only occasional human occupancy.

It should be pointed out that there are approximately 33 buildings at the Berryville, Virginia site, which are classified. These buildings are not part of the 137 buildings mentioned above, and are not included in the database.

3. Buildings for Essential Designation

Section 2.3 of RP5 defines essential buildings as those buildings which require a level of seismic resistance that is higher than life safety. These buildings have been designated in the database with an essential building code of Z1. Buildings which are recommended for this designation are listed in Table 1. Buildings which are on an historical registry are not included in this table. These buildings may need to be evaluated to a standard which is higher than life safety depending on the historical preservation requirements.

Table 1 - Buildings Recommended for Essential Designation

BUILDING NAME	LOCATION	FUNCTION
Building 311	Berryville, Virginia	Fire pumping station
Building 331	Berryville, Virginia	Houses emergency power
Building 420	Berryville, Virginia	Fire station
Bothell VSAB*	Bothell, Washington	MERS** garage and office
Denton Federal Regional Center	Denton, Texas	Communications center
Denton VSAB-Old	Denton, Texas	MERS garage and office
Denton VSAB #2	Denton, Texas	MERS garage and office
Maynard Federal Regional Center	Maynard, Massachusetts	Communications center
Maynard VSAB	Maynard, Massachusetts	MERS garage and office
Olney Federal Support Center	Olney, Maryland	Communications center

* VSAB: Vehicle Storage and Administration Building
**MERS: Mobile Emergency Response System

4. Buildings for Exceptionally High Risk Designation

Section 3.1.1 of RP5 recommends that agencies identify all of their "exceptionally high risk" (EHR) buildings for evaluation. According to the guidance on identifying such buildings in Section 3.1.1 of TR-17, the following buildings are identified as EHR buildings.

- Bothell VSAB at Bothell, Washington
  (MERS garage in a high seismic zone.)

- Maynard VSAB at Maynard, Massachusetts
  (MERS garage in a moderate seismic zone.)

Both buildings are essential buildings that house emergency response vehicles, and have unreinforced and partially reinforced concrete masonry walls.

5. Selection of Buildings for Evaluation

5.1 Screening Process

Buildings were screened after the completion of a Data Collection Sheet for each building. The information on the Data Collection Sheet was compiled during site visits and by telephone conversations with the site personnel. Exempt buildings were identified using the exemption criteria listed in Section 2.2.4 of TR-17. If a building met one of these exemption criteria but was on an historical registry or eligible to be on an historical registry, was designated as an essential building, or performed an industrial function (e.g. sewage pumping station), the building was not exempted. Forty five buildings are classified as "Exempt" for evaluation and are identified with "Exemption Code" of other than **E0** in the inventory sheets (Attachment A).

5.2 Selection Process of Buildings for Evaluation

Section 3.1 of TR-17 recommends that agencies identify buildings for seismic evaluation in two categories. The first category is those buildings designated by the agency as "exceptionally high risk" (EHR). The EHR buildings have been identified in Sect. 4.

The second category of buildings to be identified for evaluation is a representative sample of the remaining non-exempt population. The guidance states that buildings in the low seismic areas may be excluded from this group. However, because the majority of FEMA's buildings are in the low seismic areas, they are included in developing the representative sample.

FEMA owns two buildings in a moderate seismic area. These buildings are the **Maynard Federal Regional Center** and the **Maynard VSAB**. Both of these buildings have been recommended for seismic evaluation as the moderate area sample. Also, both are representative of the underground Regional Centers and the VSAB garages at other sites.

In the low seismic areas, FEMA owns 89 non-exempt buildings. In order to identify buildings for seismic evaluation, these buildings were divided into model building type and site. A total of six buildings were chosen between the Emmittsburg, Maryland site and the Berryville, Virginia site because the majority of buildings are located at these sites. Each specific building was chosen as a representative sample of the buildings on that particular site with that particular model building type. Whether or not a building was historic was also considered. Therefore, the following ten buildings at four sites were identified for evaluation:

Table 2 - Buildings Recommended for Seismic Evaluation

BUILDING NAME	LOCATION	STRUCTURE	FUNCTION	SIZE (m^2)	YEAR BUILT
Building 411	Berryville, Virginia	steel light frame	office and conference center	819	1974
Building 420	Berryville, Virginia	unreinforced masonry	fire station	703	1955
Building 431	Berryville, Virginia	unreinforced masonry	office	1517	1974
Building 704	Berryville, Virginia	unreinforced masonry	office	1848	1955
Bothell VSAB	Bothell, Washington	comb. rein. masonry & steel frame with metal cladding	MERS garage and office	2787	1983
Building D	Emmittsburg, Maryland	unreinforced masonry	dormitory	2665	1924
Building J	Emmittsburg, Maryland	concrete frame with infill shear walls	classrooms and offices	4243	1965
Building O	Emmittsburg, Maryland	unreinforced masonry - historic	chapel	1428	1839
Maynard Federal Regional Center	Maynard, Massachusetts	underground reinforced concrete bunker	communications center and office	7432	1968
Maynard VSAB	Maynard, Massachusetts	steel light frame with URM walls and metal cladding	MERS garage and office	3716	1988

6. Seismic Evaluation of Buildings

All four sites were visited by the NIST team. A complete set of architectural and structural drawings were not available for all ten buildings. Particularly, drawings for old buildings such as Buildings "D" and "O" at Emmittsburg, Maryland show only general architectural layout of the buildings.

At each site, the NIST team met a representative who is responsible for the site. The team was briefed about the general history of the building including any remodeling and expansions since the original construction. Both structural and non-structural systems were visually examined. Absence or presence of the lateral load resisting systems and load transfer paths were checked and noted, and a quick evaluation was made at the site to determine the adequacy of the system. Supporting methods for electrical fixtures, suspended ceilings, and air conditioning ducts were examined visually. The condition of mortar of masonry walls was examined by scratching the surface with a nail. No attempts were made to remove any part of the structure to ascertain information on the anchorage and bearing condition of structural members. The exterior of the building was examined to note the general condition of the building, geologic site hazards, adjacency, and soil characteristics.

The buildings were evaluated in accordance with RP4 using the procedure presented in FEMA 178. To clarify evaluation procedures, FEMA 310 (*Handbook for the Seismic Evaluation of Buildings-A Prestandard*) was also referenced in some cases. The checklists given in Appendix B of FEMA 178 were the basis for evaluation and determination of further analysis if needed. If the structure is not compliant for one of the check list items, further analysis of the structure was carried out to determine whether the structure would satisfy the "life safety" requirement. For those cases where no engineering data are available, conservative assumptions were made on material properties and dimensions based on field observations and measurements.

For each of the ten buildings evaluated, the field data, the evaluation statements (checklists),and if applicable, structural calculations, and costs estimates for rehabilitation are given in Attachment B.

The results of the structural evaluation are given in Table 3.

Table 3 - Results of Seismic Evaluation

BUILDING NAME	LOCATION	STRUCTURE	SEISMICITY	STRUCTURAL EVALUATION
Building 411	Berryville, Virginia	MB05 steel light frame	Low	Pass*
Building 420	Berryville, Virginia	MB15 unreinforced masonry	Low	Pass
Building 431	Berryville, Virginia	MB15 unreinforced masonry	Low	Pass*

Building 704	Berryville, Virginia	MB15 unreinforced masonry	Low	Fail
Bothell VSAB	Bothell, Washington	MB05 steel rigid frame	High	Pass
Building D	Emmittsburg, Maryland	MB15 unreinforced masonry	Low	Pass
Building J	Emmittsburg, Maryland	MB10 conc. frame with infill walls	Low	Pass
Building O	Emmittsburg, Maryland	MB15 unreinforced masonry	Low	Fail
Maynard Federal Regional Center	Maynard, Massachusetts	MB16 underground RC bunker	Moderate	Pass
Maynard VSAB	Maynard, Massachusetts	MB05 steel light rigid frame	Moderate	Fail

* Marginal Pass

Berryville, Virginia

Building 411

The floor plan of this one-story building is rectangular. It is 24 m (80 ft) wide and 34 m (110 ft) long. A large unobstructed interior space can hold 200 to 250 people for meetings and conferences. The vertical load resisting system is comprised of pre-engineered and pre-fabricated rigid steel frames. Z-shape purlins spanning between the rigid frames support the metal roof deck. In the plane of the frame, lateral loads are resisted by frame action. The rigid frames are designed for 40 m/s (90 mph) wind load, and they are adequate for the seismic loads in a low seismic zone.

In the direction perpendicular to the plane of the rigid frames (the longitudinal axis), there is only one pair of diagonal bracing in place between the columns of two adjacent rigid frames along one of the exterior walls. On the opposite side, a section of field stone masonry wall about 6 m (20 ft) long balances lateral load resistance in the longitudinal direction of the building. In general, most of the exterior walls are clad with metal siding.

If the bracing were to fail, the lateral load in the longitudinal direction of the building would be resisted by the masonry wall alone, and consequently, torsion would be developed. Since the building is located in a low seismic region and the lateral load produced by an earthquake is much smaller than the same produced by the design wind load, the likelihood of failure of the diagonal-bracing is small. Thus, this building is judged to have no structural deficiency. However, it is recommended that additional diagonal bracing be installed for improved seismic safety.

Building 420

This building is a fire station, and designated as an essential building. The floor plan of the building is rectangular, 19 m (62 ft) wide and 37 m (122 ft) long. This one-story URM (unreinforced masonry) building is comprised of 300 mm (12 in) partially reinforced concrete masonry walls with continuous bond beams at mid-height and at the top of the four exterior walls. Horizontal reinforcement was placed at all horizontal masonry joints. The roof framing is comprised of steel joists spanning between the exterior masonry walls and a row of steel beams supported on reinforced masonry columns located along the centerline of the building. Built-up roofing is applied on 45 mm (2 in) concrete roof planks. Steel angle bridgings between steel joists are placed at about 0.8 m (24 in) on center. This building is judged to have no deficiency.

Building 431

The building, constructed in 1974, is currently being used for office and storage. The original floor dimensions were 18 m (60 ft) wide and 49 m (160 ft) long. An addition in 1977 increased the width to 31 m (102 ft). The vertical load resisting system is comprised of long-span joists supported on square tubular steel columns. The roof load is carried by metal deck on Z-shape purlins spanning between the long-span steel joists. The joists span between square tubular columns spaced at 6 m (20 ft) on center in both directions. When the building was originally built, the perimeter tubular columns were imbedded in the exterior unreinforced masonry walls on all four sides which provided lateral load resistance. When new sections were added to make the building 31 m (102 ft) wide, light steel frames and metal siding replaced the two masonry end walls in the transverse direction. In addition, gypsum wall board partitions framed with 2x4[1] lumber replaced one of the masonry walls in the longitudinal direction. Visual inspection did not reveal any diagonal braces between tubular columns within the wall board partitions. Research has shown that gypsum board walls perform well for in-plane shear loading. Thus, it is judged that even without any diagonal braces, the gypsum wall board partitions are adequate to resist the seismic force generated by light roof load of 1.9 k Pa (40 psf) which includes 1.4 k Pa (30 psf) of snow load.

---

[1] 2x4 is a designation of dimension lumber 38 mm x 89 mm (1½ in x 3½ in) in cross section.

In the transverse direction, the lateral load is resisted by frame action provided by steel joists and tubular columns. Static analysis of the structure indicates that the columns can resist the seismic force with a small margin of safety. Thus, in the transverse direction, the lateral load resistance capacity of this building is marginal. Although the building is judged to have no deficiencies, it is recommended that this building be rehabilitated to improve its expected seismic performance.

Building 704

The plan of this two-story building is rectangular, 8.5 m (28 ft) wide and 55 m (180 ft) long. The building was originally constructed in 1955 as a dormitory and was remodeled in 1984. The building framing is comprised of wood above the first floor. The exterior walls are unreinforced concrete masonry. Continuous reinforced concrete bond beams of 200 mm x 355 mm (8 in x 14 in) and 140 mm x 355 mm (5.5 in x 14 in) are placed around the entire perimeter of the building at the second floor and roof level, respectively. The interior partitions are constructed of 2x4 wood studs. The 2x8[2] floor joists are spaced at 400 mm (16 in) on center and the 2x6[3] ceiling joists are spaced at 610 mm (24 in) on center. At the first floor, the joists are supported on concrete beams. The 2x8 roof rafters are spaced at 610 mm (24 in). The ceiling joists are anchored by metal plate to 2x6 top plates on the masonry wall. In turn, the top plates are anchored to the masonry wall with 16 mm (5/8 in) diameter steel bolts at 1.2 m (4 ft) on center. No specific details are shown on the drawing about the anchor condition of floor joists in the masonry walls.

Because of a large aspect ratio (7.2) of the floor plan, the effectiveness of the wood floor diaphragm is checked. The chord is comprised of concrete bond beams. Since the building is located in a low seismic region, the force developed in the chord is relatively small, and analysis shows that the bond beams would function safely as chords. Extreme fiber bending stresses in the plywood floor sheathing is very low 0.7 MPa (103 psi). Analysis shows that the plywood sheathing would be overstressed in shear if the diaphragm resists the total lateral load on the second floor.

In order for the floor to function as a diaphragm, the floor joists must be anchored adequately in the masonry walls or to the bond beams. A cross section of the building shows that the joists have fire-cut ends at the wall with about 90 mm (3 ½ in) to 100 mm (4 in) bearing. Analysis showed that this bearing length may not be adequate for the joists to remain supported in the wall when the floor deflects during an earthquake. Because of inadequacies found in the horizontal load path, both in stiffness and shear

---

[2,3] 2 x 8 and 2 x 6 are designations of dimension lumber 38 mm x 190 mm (1½ in x 7½ in) and 38 mm x 140 mm (2½ in x 5½ in) in cross section, respectively.

capacities, this building is judged to be deficient.

Bothell, Washington

Bothell VSAB (Vehicle Storage and Administration Building)

The floor plan of this one-story garage is an L-shape. The main garage portion is 30 m (100 ft) wide and 60 m (200 ft) long. The primary structural frame is comprised of pre-engineered rigid frames. The frames are spaced at 9 m (30 ft) on center along the length of the building. There is a two-story office building built at one end of the garage. The two-story steel frame building is structurally independent from the rigid frames. The structure was designed in accordance with the 1982 Army Manual 5-809-10 (Tri-Service Manual). This structure was designed in accordance with the seismic design provisions developed after the 1976 Uniform Building Code which incorporated modern seismic design procedures. Therefore, the design of the structures may be considered adequate.

The lateral load resisting system is comprised of diagonal braces between rigid bent columns on one side of the building and partially reinforced masonry wall on the opposite side along the length of the building. Structural ties are provided between the footings which support the rigid bent columns, thereby preventing relative spread of the column bases.

The field investigation identified that the lateral load resisting system in the north-south direction of the west-end bay may be inadequate as the end wall has four large garage doors without any lateral bracing system. The structural framing of this bay is comprised of steel-channel columns and light I-shape beams. Assuming that the resistance to lateral displacement at the top of the end bay is provided by the roof framing, the lateral displacement is computed. The computed value is very small (6 mm) due to relatively light weight of the structure. Analysis shows that the roof diaphragm has adequate capacity to resist the shear load generated by the lateral displacement of the end bay. Thus, the deficiency of the end bay as identified in the preliminary evaluation using the checklist is removed, and the structure is judged to have no deficiencies.

Emmittsburg, Maryland

All four buildings evaluated do not have architectural or structural drawings.

Building D

The floor plan of this building is rectangular, 14 m (45 ft) wide and 60 m (198 ft) long. It is a three story unreinforced brick masonry structure built in 1924. The exterior walls are stone masonry and the interior walls are brick. The first floor is comprised of reinforced concrete slab on steel beams spaced about 3.6 m (12 ft) on center. The second and third floors and the third floor ceiling are comprised of concrete slab on timber beams. This building has a most unusual roof framing in that it consists of concrete trusses and concrete slab made of fly ash concrete. This results in a large concentration of mass at the roof level. The ratio of the roof mass to the mass of the third floor is about 3.5. This would be a major concern if the building is located in a high seismic region. For a massive brick masonry structure in a low seismic region, it is reasonable to assume that the structure would respond in a first mode of vibration. A shear stress check in the masonry wall at the roof level and at the first floor indicates that the wall has adequate strength to resist horizontal shear. Thus, this building is judged to have no deficiency.

Building J

Building J is one wing of a complex of the three separate buildings (two wings and an auditorium) with connecting sections. In general, all three buildings consist of concrete frames and infill shear walls. The buildings were constructed in 1963-1965 and remodeled in 1992-1994. All floors and the roof are comprised of concrete joists. Building J, 18 m (59 ft) wide and 51 m (169 ft) long, has a partial basement comprised of reinforced concrete slab and exterior walls. A major concern of this building is the presence of gaps between the infill shear walls and concrete columns along the entire length of the building. Windows are placed in these gaps. No portion of the infill wall was removed during the field investigation to obtain information on wall anchor details to the concrete slab above and below. Due to the existence of the gaps and lack of information on the wall anchor details, the building is rated initially to have deficiencies. Since the building is located in a low seismic region, analysis was made to check whether the concrete frame alone could resist lateral loads without the aid of infill shear walls. The results of a linear elastic finite element analysis show that story drifts are relatively small and the columns have adequate strength to resist the seismic load. Based on the analysis, the building is judged to have no deficiency.

Building O

This chapel was constructed in 1839 and is on the Historic Register. The building is about 21 m (68 ft) wide and 38 m (124 ft) long. The exterior foundation of the building is stone and brick. The exterior walls are 600 mm (24 in) thick stone masonry and the interior walls are 450 mm (18 in) thick brick masonry. Timber columns and beams are used in the structure. Timber trusses support the wood ceiling over the chapel. The wood lath and plaster ceiling is suspended from the bottom chords of the trusses. The building has an over 10 m (33 ft) high steeple of wood construction. The basement of the building was renovated in the late 1970s, and the timber columns in the basement below the altar were replaced with steel columns. Although the exterior masonry walls have many large window openings which may reduce the shear capacity of the walls, a check of shear stresses in the walls showed that the exterior walls have adequate capacity to resist seismic loads. Careful examination during the field investigation showed that there is no effective load path from the steeple to the foundation. Positive load path must be provided for the steeple to remain stable during an earthquake. For improved seismic safety, it is also recommended that the wood lath and plaster ceiling be replaced with one of lighter mass.

Maynard, Massachusetts

Federal Regional Center

This is a two-story underground reinforced concrete structure. The outer dimensions of the structure are 36.5 m (120 ft) and 43 m (14 ft). It was designed for nuclear blast loading. All interior fixtures are mounted on springs and shock absorbing cushions. All suspended ceilings are rigidly attached to the concrete slab above. At the present time, there is no generally accepted routine procedure to determine earthquake loading on a buried structure. Review of the structural drawings indicate that structural members, inter-member joints and connections have adequate reinforcement to provide adequate strength and ductility. Since the structure is designed for an event of nuclear blast, it is reasonable to postulate that the structure can be occupied during and after moderate seismic events. This structure is judged to have no deficiency.

Maynard VSAB (Vehicle Storage and Administration Building)

The floor plan of this one-story garage is L-shape. The main part of the garage (the longer leg of the L) is about 36 m (120 ft) wide and 82 m (270 ft) long. The primary structural system is comprised of pre-engineered rigid steel frames. A square steel tubular column supports the ridge of the rigid frame. At one end of the garage, two-story office spaces are framed using steel beams and columns. The office spaces are enclosed with partially reinforced infill concrete masonry walls. The walls along the building (perpendicular to the plane of the rigid frame) have large garage door openings between two bents. This allows large vehicles to drive through the building between two rigid bents. As a result there are no diagonal braces. Thus, in the direction perpendicular to the plane of the rigid frame (the longitudinal axis of the building), the garage portion of the structure relies on the masonry walls to resist the lateral load. The masonry walls have reinforced concrete bond beams at two levels, one at the top of the first story and the other at the top of the second story.

The roof is comprised of steel decks on Z-shape purlins which span between the rigid frames. The out-of-plane stability of the rigid bents is provided by the steel deck and purlins plus steel rope X-bracing in one bay at the roof level. This building lacks a complete load path from the roof to the foundation for the load acting in the longitudinal direction of the building. Analysis shows that the purlins do not have adequate tension capacity to transfer the lateral load generated by the garage portion of the structure to the office portion (masonry walls). Therefore, the roof is deficient in transferring the lateral load to the vertical load resisting members (masonry walls).

7. Rehabilitation Costs of Evaluated Buildings

The costs for rehabilitation of the evaluated and seismically deficient buildings are determined according to the instructions given in Section 4 of RP5 following cost estimating Option II in *Second Edition-Typical Costs for Seismic Rehabilitation of Existing Buildings,* Volume I, FEMA -156 (1994) for structural costs, and procedures in Volume II of *Second Edition-Typical Costs for Seismic Rehabilitation of Existing Buildings*, FEMA-157(1995) for non-structural costs.

The following assumptions are used in estimating the rehabilitation costs.

1. The rehabilitation cost for historical buildings are estimated by multiplying the cost estimate obtained for the same building assuming "non-historical" by a factor of 3 (Sect. 1.6, FEMA-157).

2. The finishing costs are determined using the values obtained from the difference between "none" and "minimal" columns in Tables 1.1, 1.2, 1.3, and 1.4 of FEMA-157.

3. The project costs are determined by multiplying the sum of the structural, non-structural and finishing costs by 0.3.

Three of the ten buildings selected for evaluation failed, one of which is a historical building. The cost estimates for failed buildings are given in Attachment B. As required by TR-17, the estimated costs are divided into four categories: structural costs, nonstructural costs, finishing costs, and project costs.

The total rehabilitation cost of the evaluated buildings is $3 843 000.

8. Rehabilitation Costs of Non-Evaluated Buildings

Of 137 buildings in the inventory, 45 building are exempted from seismic evaluation. All eight underground structures in a low seismic zone are assumed to have no deficiencies. Including the Federal Regional Center at Maynard, Massachusetts, nine underground structures are removed from the inventory for seismic evaluation. The inventory has three buildings which are designated as "historical buildings." The rehabilitation costs for these buildings are treated separately. Since eight buildings (non-historic and non-underground buildings) have been evaluated, the rehabilitation costs of 72 non-evaluated buildings (137-45-9-3-8 = 72) need to be estimated.

8.1 Assumptions Made for Cost Estimate

The inventory of buildings revealed that the non-exempt FEMA buildings can be classified into nine different model building types. If an underground bunker and a historic building are treated separately, the evaluated buildings fall into three different model types. They are tabulated below with the associated floor areas. The floor areas in both columns do not contain the areas corresponding to the underground structures and historical buildings.

Non-Evaluated Bldg. (Area in m^2)	Evaluated Buildings (Area in m^2)
MB 01 (2 995)	
MB 04 (16 138)	
MB 05 (2 683)	MB 05 (7 322)
MB 08 (492)	
MB 10 (6 648)	MB 10 (4 243)
MB 13 (1 014)	
MB 14 (702)	
MB 15 (111 049)	MB 15 (6 733)
MB 16 (1 342)	
Total Area 143 063 m^2	18 298 m^2

Only three model building types are evaluated. The non-evaluated buildings that do not correspond to the evaluated building types are MB 01,04,08,13,14, and 16. However, these types represent a small portion of the total floor area of the non-evaluated buildings (16 % = 22 638 m^2 ÷ 143 063 m^2). It should be noted that none of the buildings in these types are exceptionally high risk buildings, and that all the buildings are located in a low seismic region. Therefore, it is reasonable to assume that the non-evaluated buildings of MB 01,04,08,13,14, and 16 do not need rehabilitation.

8.2 Procedure Used for Cost Estimate

1. Since all non-evaluated MB 05 buildings are in Virginia, the evaluation result of Berryville Building 411 is applied to this group of buildings. Thus, their rehabilitation costs are zero. The VSAB buildings at Maynard, MA and Bothell, WA are garages located in a moderate and a high seismic area, respectively. They are structurally different from the MB 05 buildings in Virginia.

2. Since all non-evaluated MB 10 buildings are located in Emmittsburg, Maryland, and the MB 10 building at that site which was found to pass, it is assumed that the non-evaluated MB 10 buildings do not require rehabilitation.

3. All non-evaluated MB 15 buildings are located in Maryland and Virginia. Excluding one historical building, one of the four evaluated buildings "failed." The area of the "failed" building (Building 704) is about 27 % of the total area of the evaluated MB 15 buildings. The average rehabilitation cost per square meter for the MB 15 buildings is determined by dividing the rehabilitation cost of Building 704 by the total area of the MB 15 buildings, which is ($41 795/6 733 m^2 = $62.07/$m^2$).

4. All three historical buildings are located at Emmittsburg, Maryland. Thirty three percent of the average rehabilitation cost of Emmittsburg Building O (chapel) is applied to Buildings N and Q as one is an office building and the other is a barn.

8.3 Rehabilitation Cost Estimates

The rehabilitation cost of the evaluated buildings is:

Berryville, Building 704	$ 418 000
Emmittsburg, Building O	$2 471 000
Maynard VSAB	$ 954 000
Total	$3 843 000

The rehabilitation cost of the non-evaluated buildings is:

MB 05	$ 0
MB 15	$ 6 892 800 ($62.07 /$m^2$ x 11 1049 m^2)
Others	$ 0
Historic	$ 3 114 000
Total	$ 10 006 800

The total estimated rehabilitation cost for the FEMA buildings is:

Evaluated Buildings:	$ 3 843 000
Non-evaluated Buildings:	$10 007 000
Total	$13 910 000

9. Building Inventory Data Base

All pertinent data required by RP5 are entered in the database forms according to the instructions given in TR-17. The hard copies of the database forms are attached (Attachment C). The electronic form of the database is also provided in a diskette.

Attachment A: Building Inventory

Agency Code	Unique Identifier	State Code	County Code	Seismicity	Area	Number of Buildings	Exemption Code	Occupancy Class Code	Essential Building Code	Historic Building Code	Year of Construction	Model Building Type Cod	Number of Stories Code	Comments
5800	Boathouse	24	021	L	46	1	E1	80	Z2	H2	1960	MB13	N01	Boathouse
5800	Building A	24	021	L	3091	1	E0	30	Z2	H2	1965	MB10	N03	
5800	Building B	24	021	L	541	1	E0	80	Z2	H2	1956	MB15	N01	
5800	Building C	24	021	L	2492	1	E0	30	Z2	H2	1956	MB10	N03	
5800	Building C-West	24	021	L	4923	1	E7	30	Z2	H2	1995	MB14	N03	
5800	Building D	24	021	L	2665	1	E0	30	Z2	H2	1924	MB15	N03	Eligible for historic registry but not officially registered.
5800	Building E	24	021	L	3252	1	E0	10	Z2	H2	1923	MB15	N03	Contains an auditorium which can seat 500; Eligible for historic registry but not officially registered.
5800	Building F	24	021	L	1875	1	E0	30	Z2	H2	1926	MB15	N03	Eligible for historic registry but not officially registered.
5800	Building G	24	021	L	649	1	E0	30	Z2	H2	1948	MB15	N02	
5800	Building H	24	021	L	1871	1	E0	10	Z2	H2	1923	MB15	N03	Contains recreation area (swimming pool, basketball court, weight room)
5800	Building I	24	021	L	3344	1	E7	50	Z2	H2	1996	MB07	N02	Design looked at Map Area 1 in BOCA and NEHRP
5800	Building J	24	021	L	4243	1	E0	23	Z2	H2	1965	MB10	N02	Contains an auditorium and offices as well.
5800	Building K	24	021	L	3786	1	E0	23	Z2	H2	1890	MB15	N03	Contains a cafeteria which seats about 350; eligible for historic registration but not officially registered.
5800	Building L	24	021	L	1065	1	E0	30	Z2	H2	1959	MB10	N03	
5800	Building M	24	021	L	678	1	E0	23	Z2	H2	1960	MB14	N02	
5800	Building N	24	021	L	4449	1	E0	10	Z2	H1	1870	MB15	N04	
5800	Building O	24	021	L	1428	1	E0	80	Z2	H1	1839	MB15	N02	
5800	Building P	24	021	L	280	1	E0	80	Z2	H2	1960	MB16	N01	Log Cabin; Can hold 150-200 people for recreational purposes.
5800	Building Q	24	021	L	948	1	E0	40	Z2	H1	1880	MB15	N02	
5800	Building R	24	021	L	459	1	E0	23	Z2	H2	1950	MB15	N01	
5800	Building S	24	021	L	626	1	E0	80	Z2	H2	1926	MB15	N01	Eligible for historic registry but not formally registered; Currently undergoing major renovations; Will be used as a computer simulations laboratory.
5800	Building T	24	021	L	110	1	E0	10	Z2	H2	1960	MB15	N01	
5800	Building U	24	021	L	156	#	E1	80	Z2	H2	1982	MB16	N01	12x14 precast concrete buildings used as arson labs; Built from 1982-1996.

5800	Building V	24	021	L	90	1	E7	60	Z2	H2	1992	MB13	N01	Security Station
5800	Federal Support Center	24	031	L	6039	1	E0	29	Z1	H2	1970	MB16	N00	Building is underground and designed for nuclear blast; building is reinforced concrete encased in steel; building houses offices, communications center, overall agency network
5800	Fire Pump Station	24	021	L	372	1	E0	50	Z2	H2	1981	MB16	N00	Building is underground and constructed of poured concrete.
5800	Morton Buildings	24	021	L	316	2	E1	40	Z2	H2	1980	MB02	N01	
5800	Olney Storage	24	031	L	0	2	E1	40	Z2	H2				
5800	Sewage Pumping Station A	24	021	L	15	1	E0	50	Z2	H2	1940	MB16	N00	Building is underground and constructed of poured concrete.
5800	Sewage Pumping Station B	24	021	L	15	1	E0	50	Z2	H2	1995	MB16	N00	Building is underground and is constructed of poured concrete.
5800	Region 1 Center	25	017	M	7432	1	E0	29	Z1	H2	1968	MB16	N00	Building is underground and designed for nuclear blast; building is reinforced concrete with 2 rooms in steel enclosures; building is communications center for Region 1 and also serves as regional conference center.
5800	Region 1 MERS	25	017	M	2903	1	E0	50	Z2	H2	1988	MB05	N02	Building contains some office space.
5800	Federal Regional Center	48	121	L	5110	1	E0	10	Z1	H2	1964	MB16	N00	Underground Reinforced Bunker
5800	Reception and Breakroom	48	121	L	285	1	E3	60	Z2	H2	1964	MB05	N01	
5800	Storage Building - East	48	121	L	223	1	E1	40	Z2	H2	1990	MB04	N01	
5800	Storage Building - West	48	121	L	223	1	E1	40	Z2	H2	1990	MB04	N01	
5800	VSAB - Old	48	121	L	4738	1	E0	10	Z2	H2	1985	MB04	N02	Garage and Office
5800	VSAB #2	48	121	L	1858	1	E7	10	Z2	H2	1993	MB04	N02	Garage and Office
5800	Building 104	51	107	L	1014	1	E0	40	Z2	H2	1955	MB16	N04	Structure is reinforced poured concrete walls and roof; designed for blast loading.
5800	Building 105	51	107	L	936	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 106	51	107	L	347	1	E1	40	Z2	H2	1955	MB16	N07	Structure is poured concrete walls.
5800	Building 110	51	107	L	1292	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 114	51	107	L	1398	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 123	51	107	L	22	1	E0	80	Z2	H2	1955	MB15	N02	Building is a Control Tower (Heliport)
5800	Building 127	51	107	L	24	1	E0	60	Z2	H2	1955	MB16	N01	Building is a Security Gatehouse; Structure is reinforced poured concrete and cinder block.
5800	Building 140	51	107	L	75	1	E0	50	Z2	H2	1955	MB13	N02	Sewage Treatment Plant
5800	Building 146	51	107	L	28	1	E1	40	Z2	H2	1955	MB15	N01	
5800	Building 201	51	107	L	691	1	E1	40	Z2	H2	1985	MB05	N01	
5800	Building 205/211/230	51	107	L	2464	3	E0	30	Z2	H2	1955	MB15	N02	
5800	Building 217	51	107	L	821	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 218	51	107	L	874	1	E0	80	Z2	H2	1986	MB13	N01	
5800	Building 219	51	107	L	348	1	E0	10	Z2	H2	1989	MB05	N01	
5800	Building 219A	51	107	L	678	1	E0	10	Z2	H2	1993	MB05	N03	
5800	Building 310	51	107	L	440	1	E0	60	Z2	H2	1955	MB15	N01	Building is a Motorpool.

5800	Building 311	51	107	L	33	1	E0	50	Z2	H2	1955	MB15	N01	Building is a Fire Pumping Station
5800	Building 312/313	51	107	L	35	2	E1	40	Z2	H2	1955	MB15	N01	
5800	Building 315	51	107	L	344	1	E0	50	Z2	H2	1955	MB15	N01	Building is a maintenance shop.
5800	Building 317	51	107	L	42	1	E1	40	Z2	H2	1955	MB15	N01	Structure is cinderblock construction.
5800	Building 320	51	107	L	346	1	E1	40	Z2	H2	1955	MB15	N01	
5800	Building 320A	51	107	L	302	1	E0	50	Z2	H2	1988	MB05	N01	Building is a maintenance shop; Structure has a mezzanine.
5800	Building 321	51	107	L	22	1	E1	40	Z2	H2	1995	MB14	N01	
5800	Building 327	51	107	L	190	1	E1	40	Z2	H2	1955	MB01	N02	
5800	Building 329	51	107	L	669	1	E0	40	Z2	H2	1955	MB05	N01	
5800	Building 331	51	107	L	161	1	E0	50	Z2	H2	1955	MB15	N01	Building houses Emergency Power.
5800	Building 400	51	043	L	96	1	E0	10	Z2	H2	1955	MB15	N01	
5800	Building 401	51	043	L	65	1	E0	60	Z2	H2	1975	MB13	N01	Building is a Guardhouse.
5800	Building 403	51	043	L	358	1	E0	10	Z2	H2	1955	MB15	N01	Building contains health unit.
5800	Building 404	51	043	L	11	1	E0	50	Z2	H2	1974	MB15	N01	Building houses electrical equipment - transformer.
5800	Building 405	51	107	L	929	1	E0	10	Z2	H2	1900	MB01	N04	
5800	Building 406	51	107	L	394	1	E0	80	Z2	H2	1974	MB01	N01	Building is a covered walkway between buildings.
5800	Building 408	51	043	L	462	1	E0	50	Z2	H2	1955	MB05	N01	Building is a Maintenance Shop.
5800	Building 409	51	107	L	779	1	E0	10	Z2	H2	1974	MB05	N01	
5800	Building 410	51	043	L	568	1	E0	50	Z2	H2	1900	MB01	N02	Building is a Maintenance Shop.
5800	Building 411	51	107	L	819	1	E0	10	Z2	H2	1974	MB05	N01	Building has conference capacity for 200-250.
5800	Building 413	51	107	L	1104	1	E0	10	Z2	H2	1900	MB01	N04	
5800	Building 415	51	107	L	132	1	E1	50	Z2	H2	1955	MB15	N01	Maintenance Building
5800	Building 417/425	51	107	L	57	2	E1	60	Z2	H2	1955	MB12	N02	Guardhouses
5800	Building 418	51	107	L	4	1	E1	60	Z2	H2	1955	MB15	N01	Guardshack
5800	Building 420	51	107	L	703	1	E0	60	Z1	H2	1955	MB15	N01	This is the only firestation which serves the site.
5800	Building 426	51	107	L	202	1	E1	40	Z2	H2	1955	MB13	N01	
5800	Building 429	51	107	L	1468	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 430	51	107	L	1336	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 430A	51	107	L	1778	1	E5	10	Z2	H2	1990	MB13	N02	
5800	Building 431	51	107	L	1517	1	E0	10	Z2	H2	1974	MB15	N01	
5800	Building 431A	51	107	L	90	1	E0	10	Z2	H2	1974	MB04	N01	
5800	Building 435	51	107	L	2585	1	E0	60	Z2	H2	1955	MB15	N02	Building is a cafeteria which seats about 250-300 people.
5800	Building 444	51	107	L	3826	1	E0	10	Z2	H2	1990	MB04	N02	
5800	Building 500	51	043	L	39	1	E0	80	Z2	H2	1960	MB15	N02	Heliport.
5800	Building 501	51	043	L	5	1	E0	60	Z2	H2	1972	MB15	N01	Used for Communication.
5800	Building 505	51	043	L	14	1	E1	80	Z2	H2	1992	MB01	N01	Picnic Shelter
5800	Building 604	51	043	L	5626	1	E0	10	Z2	H2	1986	MB04	N02	
5800	Building 701	51	043	L	347	1	E1	40	Z2	H2	1955	MB16	N07	Structure is poured reinforced concrete walls.

5800	Building 702	51	043	L	1014	1	E1	40	Z2	H2	1955	MB16	N04	Structure is reinforced poured concrete walls and roof; designed for blast loading.
5800	Building 703	51	043	L	109	1	E1	40	Z2	H2	1955	MB01	N01	
5800	Building 704	51	043	L	1848	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 706	51	043	L	392	1	E0	80	Z2	H2	1990	MB15	N02	Firing Range
5800	Building 707	51	043	L	749	1	E1	40	Z2	H2	1990	MB01	N01	Polebarn
5800	Building 708	51	043	L	1046	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 709	51	043	L	86	1	E0	50	Z2	H2	1987	MB15	N01	Generator
5800	Building 710	51	043	L	114	1	E1	80	Z2	H2	1989	MB15	N01	Trash Collection
5800	Building 712	51	043	L	1778	1	E0	10	Z2	H2	1955	MB15	N02	
5800	Building 713	51	043	L	88	1	E1	40	Z2	H2	1992	MB08	N01	
5800	Building 713A	51	043	L	131	1	E1	40	Z2	H2	1993	MB01	N01	
5800	Building 718	51	043	L	25	1	E0	50	Z2	H2	1955	MB15	N01	Generator Building
5800	Building 720	51	043	L	492	1	E0	50	Z2	H2	1955	MB08	N03	Water Plant
5800	Building 721+	51	043	L	8424	9	E0	30	Z2	H2	1955	MB15	N02	
5800	Building 752	51	043	L	24	1	E0	60	Z2	H2	1955	MB16	N01	Building is a Security Gatehouse; Structure is reinforced poured concrete and cinder block.
5800	Building 754	51	043	L	103	1	E3	80	Z2	H2	1985	MB01	N01	Picnic Shelter
5800	Building 781	51	043	L	24	1	E0	50	Z2	H2	1955	MB14	N01	Pumping Station - mostly underground
5800	Building 800	51	043	L	29	1	E0	50	Z2	H2	1955	MB14	N00	River Intake Station - underground
5800	Building 810	51	043	L	77	1	E0	50	Z2	H2	1955	MB13	N00	Generator Building - underground
5800	Building 820/830	51	043	L	171	2	E0	50	Z2	H2	1955	MB13	N00	Booster Pumping Station - underground
5800	Bothell VSAB	53	061	H	2787	1	E0	50	Z2	H2	1983	MB05	N01	Garage and offices

Attachment B: Seismic Evaluation and Rehabilitation Cost Data

Building Designation : 411

Location: Berryville, VA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1974 Year(s) remodelled: –
Date of Evaluation: 8/5/98
Area, (sq. ft.) 8816 Length 110' Width 80' Photo Roll No. ____

CONSTRUCTION DATA

Roofframing: Z-purlins + metal deck
Intermediate floor framing: –
Ground floor: Concrete Basement: None
Exterior walls: Masonry Openings: –
Columns: Steel Foundations: Spread footing + Wall footing
General condition of structure: Very Good
Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB 05	MB 05
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

Response Modification Coefficient, R: 4.5

EVALUATION DATA

A_a = 0.05 A_v = 0.05

Site soil profile type: S1 Site soil coefficient, S = 1.0

REMARKS

Pre-engineered steel rigid frames.
Bldg designed for 90 mph wind load.

Bldg 411 , Berryville , VA

EVALUATION STATEMENTS FOR BUILDING TYPE 5: STEEL LIGHT FRAME

These buildings are pre-engineered and pre-fabricated with transverse rigid frames. The roof and walls consist of light-weight panels. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames, with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of tension-only rod bracing, or a combination of panels and bracing.

> Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.
>
> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F DETERIORATION OF STEEL: There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical or lateral-force-resisting systems. (Sec. 3.5.3)

MOMENT FRAMES

T F STRESS CHECK: The building satisfies the Quick Check of the stress in the diagonals. (Sec. 6.1.1)

T F BEAM PENETRATIONS: All openings in frame-beam webs have a depth less that 1/4 of the beam depth and are located in the center half of the beams. (Sec. 4.2.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

CONNECTIONS

T F STEEL COLUMNS: The columns in the lateral-force-resisting system are substantially anchored to the building foundation. (Sec. 8.4.1)

WALL AND ROOF PANELS

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS ROOF PANELS: All light-gage metal, plastic, or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center. (Sec. 8.6.1)

T F WALL PANELS: All wall panels (metal, fiberglass, or cement asbestos) are properly connected to the framing. (Sec. 8.6.2)

Wind load check.

Designed for 90 m/h.

BOCA

$$p = P_v I \left[K_z G_h C_p - K_h (GC_{pi}) \right]$$

$P_v = 20.7$ #/ft

$I = 1.0$

$K_z = 0.8$ (0 – 15 ft).

$G_h = 1.32$

$C_p = 0.8$

$K_h = 0.8$

$GC_{pi} = \begin{matrix} +0.75 \\ -0.25 \end{matrix}$

$$p = 20.7 \left[0.8 \times 1.32 \times 0.8 - 0.8(-0.25) \right]$$

$$= 20.7 (0.845 + 0.2) = 20 (1.045) = 20.9 \text{ \#/ft}$$

Wind load

$$H = \frac{80 \times 14}{2} \times 20.9 = 11704 \text{ \#}$$

Lateral load due to earthquake

Upper half of the wall

$$W_w = \frac{11.5}{2} \times 102' \times 15 \text{ \#/ft} \times 0.9 = 7917 \text{ \#}$$

Roof

$$W_d = 40' \times 102' \times 20 \text{ \#/ft} = 81600 \text{ \#}$$

$$A_v W_d = 0.1 \times 81600 \text{ \#} = 8160 \text{ \#}$$

$0.1 \times W_w = 791$ #
$\underline{8160}$
8951 # < 11704 #

Wind Bracing is O.K.

OPTION 2 COST ESTIMATION FORM

COST ESTIMATION OPTION 2

1. GROUP MEAN COST
   - Group:
     - ☐ URM ☐ S1
     - ☐ W1, W2 ☒ S2, S~~5~~3
     - ☐ PC1,RM1 ☐ S5
     - ☐ C1, C3 ☐ C2, PC2, RM2, S4
   - Cost Coefficient C_1 from Table 4.3.2. — C_1 = 7.23

2. AREA ADJUSTMENT FACTOR
   - Area
     - ☒ Less than 10K sq. ft. ☐ 10K - 50K sq. ft.
     - ☐ 50K - 100K sq. ft. ☐ 10K - 50K sq. ft.
   - Cost Adjustment Factor C_2 from Table 4.3.3 — C_2 = 1.18

3. SEISMICITY/PERFORMANCE OBJECTIVE FACTOR ADJUSTMENT
   - SEISMICITY
     - ☒ Low (NEHRP 1 or 2) ☐ Moderate (NEHRP 3 or 4)
     - ☐ High (NEHRP 5 or 6) ☐ Very High (NEHRP 7)
   - PERFORMANCE OBJECTIVE
     - ☒ Life Safety ☐ Damage Control ☐ Immediate Occupancy
   - Cost Adjustment Factor C_3 from Table 4.4.2 — C_3 = 0.61

4. LOCATION ADJUSTMENT FACTOR
   - City / State Berryville, VA
   - Cost Adjustment Factor C_L from Table 4.3.4 or Table 4.3.5 — C_L = 0.84

5. TIME ADJUSTMENT FACTOR
   - Year 1998
   - Inflation Rate 2 %
   - Cost Adjustment Factor C_T from Table 4.3.6 — C_T = 1.10

TYPICAL STRUCTURAL COST
($C = C_1 \times C_2 \times C_3 \times C_L \times C_T$) — C = 4.81

Building Area (Square Foot) : A = 8816

Estimated Structural Cost (A x C) C_S = 42,405

Non-Structural Cost ($C_1 \times C_L \times C_T$) C_{NS} = 0

Finishing Cost (estimated) C_F = 5,000

Total (Structural + Non-Struc + Finishing) C_{ST} = 47,405

Project Cost (C_{ST} x 0.3) C_P = 14,221

Total Cost ≈ 61,600

Building Designation : 420

Location: Berryville, VA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1955 Year(s) remodelled: -

Date of Evaluation: 8/5/98

Area, (sq. ft.) 7567 Length 122 Width 62 Photo Roll No. ____

CONSTRUCTION DATA

Rooframing: Steel Joists

Intermediate floor framing: -

Ground floor: None Basement: None

Exterior walls: Conc. Masonry Openings: ____

Columns: CMU PIERS Foundations: Conc. Wall and Col. footings

General condition of structure: Very Good

Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB 15	MB 15
Building period, T:		

Unreduced base shear,

$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R: 1.5 ____

EVALUATION DATA

A_a = 0.05 A_v = 0.05

Site soil profile type: S_2 Site soil coefficient, S = 1.2

REMARKS

Soil : weathred rock, clay silt mixture

Reinf. conc. bond beams at the top and mid levels.

Fire station

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F ADJACENT BUILDINGS: There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated. (Sec. 3.4)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

> For buildings with wood diaphragms and unreinforced masonry bearing and enclosure walls at the perimeter, complete the evaluation using the procedure given in Appendix C. For other buildings, continue with the following evaluation statements.

MASONRY WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (Sec. 5.4.1)

T F PROPORTIONS: In areas of high seismicity (A_v greater than or equal to 0.2), the height-thickness ratio of the unreinforced masonry wall panels is as follows: (Sec. 5.5.1; also see Appendix C)

- One-story building: $h_w/t < 14$
- Multistory building:
  - Top story: $h_w/t < 9$
  - Other stories: $h_w/t < 20$

T F MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls have negligible voids. (Sec. 5.4.2)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than *50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

T F SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Sec. 7.2.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

CONNECTIONS

T F MASONRY WALL ANCHORS: Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

Building Designation : 431

Location: Berryville, VA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1974 Year(s) remodelled: 1977
Date of Evaluation: 8/5/98
Area, (sq. ft.) 16330 Length 160 Width 102 Photo Roll No. ____

CONSTRUCTION DATA

Roofframing: steel joists, metal deck
Intermediate floor framing: None
Ground floor: Conc Basement: None
Exterior walls: Masonry Openings:
Columns: Tubular steel Foundations: Conc wall & spread ftg.
General condition of structure: FAIR
Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB16	MB15
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R:	1.5	

EVALUATION DATA

A_a = 0.05 A_v = 0.05

Site soil profile type: S1 Site soil coefficient, S = 1.0

REMARKS

Original CMU wall removed, replaced with 2x4 partitions.

Bldg. 431 , Berryville , VA

EVALUATION STATEMENTS FOR BUILDING TYPE 15: UNREINFORCED MASONRY BEARING WALL BUILDINGS

These buildings include structural elements that vary depending on the age of the building and, to a lesser extent, the geographic location of the structure. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In buildings built after 1950, unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. More recently, in regions of lower seismicity, these buildings can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can have the effect of reducing diaphragm displacements.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1) one-story struct.

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2) one-story struct.

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F ADJACENT BUILDINGS: There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated. (Sec. 3.4)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

> For buildings with wood diaphragms and unreinforced masonry bearing and enclosure walls at the perimeter, complete the evaluation using the procedure given in Appendix C. For other buildings, continue with the following evaluation statements.

MASONRY WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (Sec. 5.4.1)

T F PROPORTIONS: In areas of high seismicity (A_v greater than or equal to 0.2), the height-thickness ratio of the unreinforced masonry wall panels is as follows: (Sec. 5.5.1; also see Appendix C)

- One-story building: $h_w/t < 14$ $\frac{114}{8} = 14.25$ o.k.
- Multistory building:
  - Top story: $h_w/t < 9$
  - Other stories: $h_w/t < 20$

T F MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls have negligible voids. (Sec. 5.4.2) N/A

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1) N/A

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than *50 percent of the building width in either major plan dimension. (Sec. 7.1.3) N/A

T F SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Sec. 7.2.4) $\frac{140}{60} = 2.3$

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

CONNECTIONS

T F MASONRY WALL ANCHORS: Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

Check square tubular column strength

Column size : 6 x 6 x 1/4

steel : A 36

Length of col : 12'- 6"

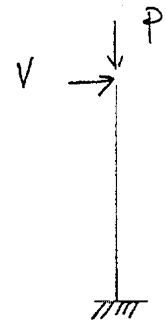
Tributary area for each col. : 20' x 20'

- Assume no moment is transferred from steel joists or Z purlins to the top of column.
- Lateral load is applied to the top of column.

Load : Roof 10 psf

Snow 30 psf

40 psf



Vertical load on col.

$$40 \times 20' \times 20' = 16{,}000^{\#} \text{ or } 16\text{ K}$$

Assume $T = 0.1$ sec.

$$C_s = \frac{2.12\ A_a}{R} = \frac{2.12 \times 0.05}{2} = 0.053$$

$$V = C_s W = 0.053 \times 16 = 0.85\text{ k}$$

Bending $M = 0.85 \times 150" = 127.5$ k-in

$P = 16$ K

$KL = 2L$

$$\frac{kL}{r} = \frac{2.0 \times 150}{2.33} = 129$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2\ 29000}{36}} = 126.1$$

$$\frac{kL}{r} > C_c \qquad F_a = \frac{12\pi^2 E}{23\left(\frac{kL}{r}\right)^2} = 9\ ksi$$

$$F_b = 0.66 F_y = 24\ ksi$$

$$F_e' = \frac{12\pi^2 E}{23\left(\frac{kL}{r}\right)^2} = 9\ ksi$$

$$f_a = \frac{P}{A_g} = \frac{16}{5.59} = 2.86\ ksi$$

$$f_b = \frac{M}{S} = \frac{127.5}{10.1} = 12.62\ ksi$$

$$C_m = 0.85$$

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_a'}\right) F_b} = \frac{2.86}{9} + \frac{0.85 \times 12.62}{\left(1 - \frac{2.86}{9}\right) \times 24}$$

$$= 0.318 + 0.655 = 0.973 < 1.0$$

Marginally O.K.

Building Designation : 704

Location: Berryville , VA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1956 Year(s) remodelled: ________

Date of Evaluation: 8/5/98

Area, (sq. ft.) 19892 Length 180' Width 28' Photo Roll No. ____

CONSTRUCTION DATA

Roofframing: 2×8 Wood rafters @ 24" oc

Intermediate floor framing: 2×8 Wood joists @ 16" oc

Ground floor: 2×8 Wood Jois Basement: Partial / Concrete

Exterior walls: CMU Openings: Windows

Columns: Steel pipes Foundations: Concrete spread footing

General condition of structure: FAIR

Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB 15	MB 15
Building period, T:		

Unreduced base shear,

$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R: 1.5 ________

EVALUATION DATA

A_a = 0.05 A_v = 0.05

Site soil profile type: S2 Site soil coefficient, S = 1.2

REMARKS

Bldg 704 Berryville, VA

EVALUATION STATEMENTS FOR BUILDING TYPE 15: UNREINFORCED MASONRY BEARING WALL BUILDINGS

These buildings include structural elements that vary depending on the age of the building and, to a lesser extent, the geographic location of the structure. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In buildings built after 1950, unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. More recently, in regions of lower seismicity, these buildings can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can have the effect of reducing diaphragm displacements.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F ADJACENT BUILDINGS: There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated. (Sec. 3.4)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

> For buildings with wood diaphragms and unreinforced masonry bearing and enclosure walls at the perimeter, complete the evaluation using the procedure given in Appendix C. For other buildings, continue with the following evaluation statements.

MASONRY WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (Sec. 5.4.1)

T F PROPORTIONS: In areas of high seismicity (A_v greater than or equal to 0.2), the height-thickness ratio of the unreinforced masonry wall panels is as follows: (Sec. 5.5.1; also see Appendix C)

- One-story building: $h_w/t < 14$
- Multistory building:
  - Top story: $h_w/t < 9$
  - Other stories: $h_w/t < 20$

T F MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls have negligible voids. (Sec. 5.4.2)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than •50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

T F SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Sec. 7.2.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than •2 to 1. (Sec. 7.2.1)

CONNECTIONS

T F MASONRY WALL ANCHORS: Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (Sec. 8.2.3) Not Known

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4) Not Known

Weight of Building

Long. Walls		180' × 10' × 55 =	99000	(8" hollow c.b)
		180 × 9.3 × 80 =	133920	(12½" hollow c.b)
End walls		28 × 10 × 55 =	15400	
		28 × 9.3 × 80 =	20832	
Partitions	2nd fl	2 #/□ × 180 × 28 =	10080	
	1st fl	2 × 180 × 28 =	10080	
Floor	2nd fl	5 #/□ × 180 × 28 =	25200	
	1st fl	5 #/□ × 180 × 28 =	25200	
Ceilings	2nd fl	1 × 180 × 28 =	5040	
	1st fl	1 × 180 × 28 =	5040	
Roof (Wood trusses) Rafters + Joists		5 × 180 × 28 =	25200	
Roofing		2 × 180 × 28 =	10080	
L.L.	2nd fl only	50 × 180 × 28 =	252000	
			637,072 #	

h/t ratio

Top story $= \frac{112}{8} = 14 > 9$ o.k. for a low seismic region.

1st story $= \frac{112}{12} = 9.33 < 20$.

$C_w = \frac{2.12\, A_a}{R} = \frac{2.12 \times 0.1}{1.5} = 0.141$

$V = 0.141 \times 637072 = 89827^{\#}$

Net Area $= 180' \times \frac{12''}{12} \times 2 = 360^{\square'} = 51840^{\square''}$

$v = \frac{89827}{51840} = 1.73$ psi ✓

CRITICAL SECTION THRU WINDOWS

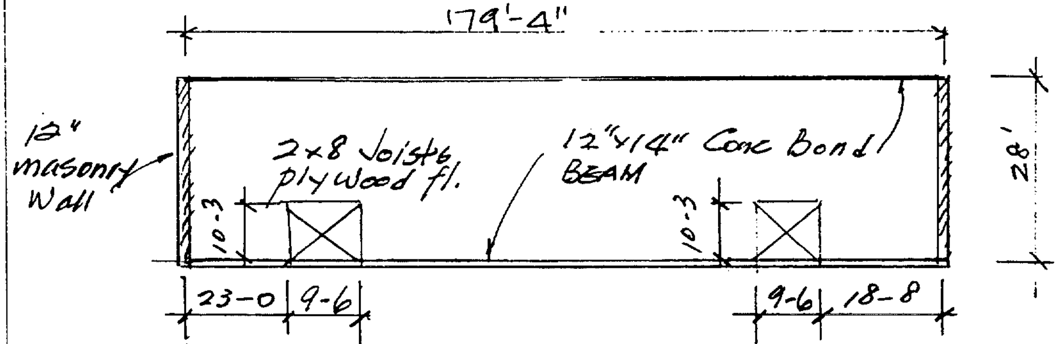
$(16 \times 6.0') = 22.3'$

$(180 - 22.3) \times \frac{12}{12} \times 2 = 315.4^{\square'} = 45417^{\square''}$

$v = \frac{89827}{45417} = 2$ psi ✓ very low.

Even w/ snow load acting v would be very low.

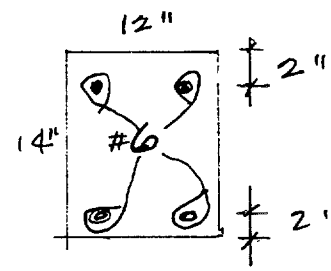
CHECK DIAPHRAGM ACTION AT 2nd FL.



$f'_c = 3000$ psi

$f_y = 20{,}000$ psi

$f_{wood} = 1100$ psi



LATERAL LOAD ON CHORDS (BOND BEAMS)

DL + LL = 40320 + 252 000 = 292,320 #

DL :	Floor	25200 #
	Partition	10080 #
	ceiling	5040
		40320 #
LL :		252000 #

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$$

$$T_a = \frac{0.05 \; h_n}{\sqrt{L}} \qquad L = 180' \quad h_n = 20'$$

$$= \frac{0.05 \; 20}{\sqrt{180}} = 0.07 \text{ sec}$$

UBC (94) 1628.2.2 $\quad T = C_t (h_n)^{3/4} = 0.02 (20)^{3/4} = 0.189$ sec

9.46

Building Period

USE 0.13 sec. $k = 1$

Snow load

$P_f = C_e I P_g = 0.7 \times 1.0 \times 25 = 17.5$ #/□

$P_s = C_s P_f = \left[1 - \frac{(22.6-30)}{40}\right] 17.5 = 1.185 \times 17.5 \cong 20$ #/□

Roof level :

$$\begin{array}{r} w_r = 25200 \\ 10080 \\ 5040 \\ \hline 40320 \text{ \#} \end{array}$$

$h_r = 20'$

2nd fl level : $29232\# + \overset{wall}{99000} + \overset{wall}{15400} = 143632\#$

$h_r = 10'$

$$C_{vx} = \frac{143632\,(10)^1}{(40320 \times 20) + (143632 \times 10)} = \frac{1436320}{2242720}$$

$= 0.64$

$F_x = 0.64 \times 89827 = 57528$ #

$f_x = \frac{57528}{179.25} = 321$ #/ft

$M = \frac{321\,(179.25)^2}{8} = 1289239$ f-#

TRANSFORMED SECTION of bond beams

$\#6 = 0.44\ in^2$

Σ steel area = $4 \times 0.44 = 1.76$

$E_c = 57000\sqrt{3000} = 3122.$ ksi

$n = \frac{30}{3.122} = 9.6$

Total Conc Area $= (12 \times 14) + (9 \cdot 6 - 1)\, 1.76 = 168 + 15.14$
$= 183 \ in^2$

$$I \approx 2Ad^2 = 2 \times 183 \times [(14 \times 12) - 6]^2 = 9714455 \ in^4$$

$$\frac{Mc}{I} = \frac{1289239 \times 12 \times 162}{9714455} = 258 \text{ psi} < 328 \quad o.k.$$

Tensile strength of Conc

$$f_t = 6\sqrt{f'_c} = 6\sqrt{3000} = 328 \text{ psi.}$$

Buckling Possibility Check

$$C = 268 \times 12 \times 14 = 45024 \ \#$$

$$P = \frac{\pi^2 EI}{L^2}$$

$$I = \frac{bh^3}{12} + 2Ad^2 = \frac{14(12)^3}{12} + 2(183)(4)^2$$

$$= 2016 + 5856 = 7872 \ in^4$$

$$P = \frac{(3.14)^2\, 3122000 \times 7872}{(2152)^2} = 52323 \ \# > 45024 \ \# \quad o.k.$$

No Buckling.

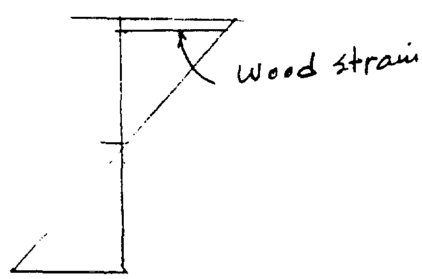
Shear Stress in Plywood

$E_w \cong 1.2 (M \text{ psi})$

$$f_w = (1.2 \times 10^6) \times \left(\frac{258}{3 \times 10^6}\right)$$

$= 103$ psi o.k

$$\varepsilon_c = \frac{\sigma}{E} = \frac{258}{3 \times 10^6}$$



End Wall Shear Stress at 2nd Fl. and 1st Fl. levels,

CHECK SHEAR AT MID HEIGHT of 2nd story.

DL.: Roof rafters & Joists : 5 #/ф
Roofing : 2 #/ф
Ceiling : 1 #/ф

8 #/ф

Total load = 8 × 180 × 28 = 40320 #

LL : Snow : 20 #/ф :

Total snow load = 20×180×28 = 100800 #

w/ snow

$C_w = 0.141$

$V = \frac{0.141}{2}(40320 + 100800) = 9949\ \#$

$\text{Net area} = [(28' - 12') \times 12''/1] \times 8'' \times \frac{57.1\ (\text{net})}{119.1\ (\text{Gross})} = 736\ in^2$

$v\ (\text{shear stress}) = \frac{9949}{736} = 13.5\ psi$ << 50 psi allowable. O.K.

$\frac{M}{Vd} = \frac{5}{28} = 0.12$

CHECK SHEAR AT MID HEIGHT of 1st story.

Roof : 40320
Snow : 100800

No. of window openings

Lg. Walls (99000×2) − 55 × 2(6×5)×17 :
= 198000 − 56100 = 141900 #

Tr. Walls (15400×2) − 55 × 2(6×5) × 3
= 30800 − 9900 = 20900 #

Partition : 10080 #

2nd fl : 25200
Ceiling : 5040

2nd fl LL = 25200 #

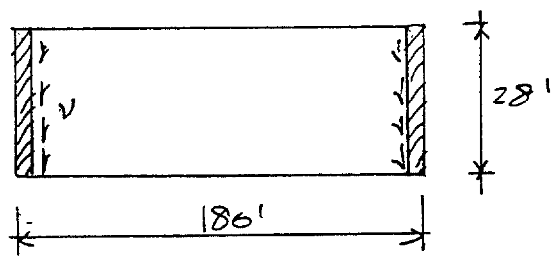
Total load (including snow) = 596240 #

$V = \frac{0.141}{2} \times 596240 = 42035$ #

net area of wall = $(28' - 12') \times 12 \times 12'' \times \frac{82.5}{181.6} = 1047 \text{ in}^2$

$v = \frac{42035}{1047} = 40$ psi < 50 psi o.k.

Check shear in the plywood diaphragm @ 2nd fl



LOAD :

D.L. of floor	:	5 #/ft² × 180' × 28' =	25200 #
L.L.		25% × 50 #/ft² × 180 × 28' =	63000
Partition		10 #/ft² × 180 × 28' =	50400
			138600 #
Exterior wall	8"	180' × 5.5' × 55 =	54450 #
	12"	180 × 4.6 × 80 =	66240
			120,690 #

a) Assume all lateral loads are resisted by wood diaphragm

UBC 97. $f_p = 1.0\ C_a I w_p$ $C_a = 0.12$ (S_D soil, Zone 1)

w_p = floor load + wall load

= 138600 + (2 × 120690) = 379980

2 walls

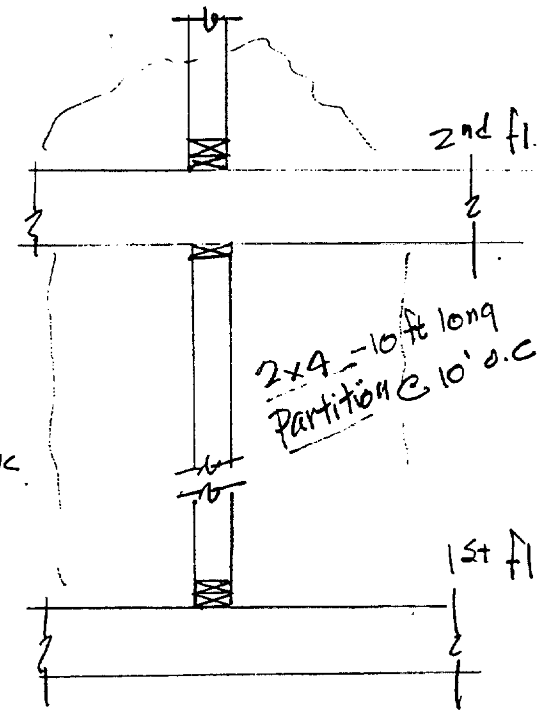
$f_p = 1.0 \times 0.12 \times 1.0 \times 379980 = 45600$ #

$$v = \frac{45600}{2 \times 28} = 814 \text{ \#/ft} \gg 675 \text{ \#/ft.}$$

(FEMA 178, page C-10)

2) Assume wood partitions participate in resisting lateral load by 30%

There are over twenty 2x4 Gypum wall partitions in transverse directions. 2x4 plates are nailed to wood joists and to roof rafters.



$V = 70\% \times 814$

$= 570 \text{ \#/ft} < 675 \text{ \#/ft}$ o.k.

3) CHECK MID SPAN Deflection.
(ATC 7, Page 167; FEMA 303)

No chord slip.

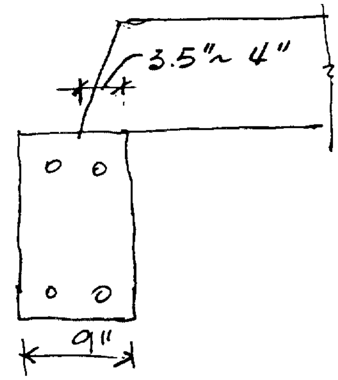
$$\Delta = \frac{5 v \ell^3}{8 w EA} + \frac{v \ell}{4Gt} + 0.188\, \ell e_n + \frac{\Sigma (\cancel{\Delta_c X})}{2W}$$

$\ell = 180'$
$w = 28'$
$E_c = 3{,}222{,}000$ psi (conc)
$A_c = 8 \times 14 = 112 \text{ in}^2$
$G = 60{,}000$ psi
$e_n = 0.077$ (Assumed 10d nail @ 3") 240#/nail (Largest value in the table)
$t = 3/4"$

$$\Delta = \frac{5 \times 814 \times (180)^3}{8\,(28)(3222000)\,112} + \frac{814\,(180)}{4\,(60{,}000)(0.75)} + 0.188\,(180)\,0.077$$

$$= 0.29'' + 0.81'' + 2.61'' = 3.71''$$ N.G. too close.

Controling factor is nail deformation



4) Assume interior patitions participate in resisting lateral deformation

$$\Delta = (0.29 + 0.81)\frac{570}{814} + 2.61 = 0.77 + 2.61 = 3.38''$$

It is concluded that

1. Diaphagm has enough strength to resist the lateral load.

2. It is most likely that either the joists fall off from the wall or the wall would move away from the joists.

OPTION 2 COST ESTIMATION FORM

COST ESTIMATION OPTION 2

1. GROUP MEAN COST
   - Group:
     - [x] URM
     - [x] W1, W2
     - [ ] PC1,RM1
     - [ ] C1, C3
     - [ ] S1
     - [ ] S2, S5
     - [ ] S5
     - [ ] C2, PC2, RM2, S4

   $\frac{15.29 + 12.29}{2} = 13.79$

   - Cost Coefficient C_1 from Table 4.3.2. — $C_1 = 13.79$

2. AREA ADJUSTMENT FACTOR
   - Area
     - [ ] Less than 10K sq. ft.
     - [x] 10K - 50K sq. ft.
     - [ ] 50K - 100K sq. ft.
     - [ ] 10K - 50K sq. ft.

   $\frac{1.00 + 1.02}{2} = 1.01$

   - Cost Adjustment Factor C_2 from Table 4.3.3 — $C_2 = 1.01$

3. SEISMICITY/PERFORMANCE OBJECTIVE FACTOR ADJUSTMENT
   - SEISMICITY
     - [x] Low (NEHRP 1 or 2)
     - [ ] Moderate (NEHRP 3 or 4)
     - [ ] High (NEHRP 5 or 6)
     - [ ] Very High (NEHRP 7)
   - PERFORMANCE OBJECTIVE
     - [x] Life Safety
     - [ ] Damage Control
     - [ ] Immediate Occupancy
   - Cost Adjustment Factor C_3 from Table 4.4.2 — $C_3 = 0.61$

4. LOCATION ADJUSTMENT FACTOR
   - City / State: VA
   - Cost Adjustment Factor C_L from Table 4.3.4 or Table 4.3.5 — $C_L = 0.84$

5. TIME ADJUSTMENT FACTOR
   - Year: 1992
   - Inflation Rate: 2 %
   - Cost Adjustment Factor C_T from Table 4.3.6 — $C_T = 1.10$

TYPICAL STRUCTURAL COST
($C = C_1 \times C_2 \times C_3 \times C_L \times C_T$) — $C = 7.85$

Building Area (Square Foot) : A = 19892

Estimated Structural Cost (A x C)	C_S =	156200
Non-Structural Cost ($C_I \times C_L \times C_T$) \$ 6.00/ft x 0.84 x 1.10 = 5.54	C_{NS} =	110200
Finishing Cost 1/2 x 110200	C_F =	55100
Total (Structural+Non-Struc+Finishing)	C_{ST} =	321500
Project Cost (C_{ST} x 0.3)	C_P =	96450
Total Cost		417,950

Building Designation : Bothell USAB

Location: Bothell , WA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1985 Year(s) remodelled: ________
Date of Evaluation: 8/14/98
Area, (sq. ft.) 30000 Length 200 Width 100 Photo Roll No. ____

CONSTRUCTION DATA

Rooframing: Z-purlins + metal deck
Intermediate floor framing: Steel framing + Conc slab
Ground floor: Conc. Basement: None
Exterior walls: CMU/metal Openings: Overhead doors
Columns: Steel Foundations: Conc spread footings, Piles
General condition of structure: Good
Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB05	MB05 + MB07
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R: 5.5 ________

EVALUATION DATA

A_a = 0.2 A_v = 0.2

Site soil profile type: S_2 Site soil coefficient, S = 1.2

REMARKS

Designed for 90 MPH wind load (20 psf / ANSI A58.1)
Seismic design: Zone 3 Army Manual TM 5-809-10 (1982)

Bothell USAB , Bothell, WA

EVALUATION STATEMENTS FOR BUILDING TYPE 5: STEEL LIGHT FRAME

These buildings are pre-engineered and pre-fabricated with transverse rigid frames. The roof and walls consist of light-weight panels. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames, with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of tension-only rod bracing, or a combination of panels and bracing.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T (F) LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1) No lateral load resisting elements in the N-S dir. at the west end bay

(T) F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

(T) F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

(T) F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

(T) F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F DETERIORATION OF STEEL: There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical or lateral-force-resisting systems. (Sec. 3.5.3)

MOMENT FRAMES

T F STRESS CHECK: The building satisfies the Quick Check of the stress in the diagonals. (Sec. 6.1.1)

T F BEAM PENETRATIONS: All openings in frame-beam webs have a depth less that 1/4 of the beam depth and are located in the center half of the beams. (Sec. 4.2.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

CONNECTIONS

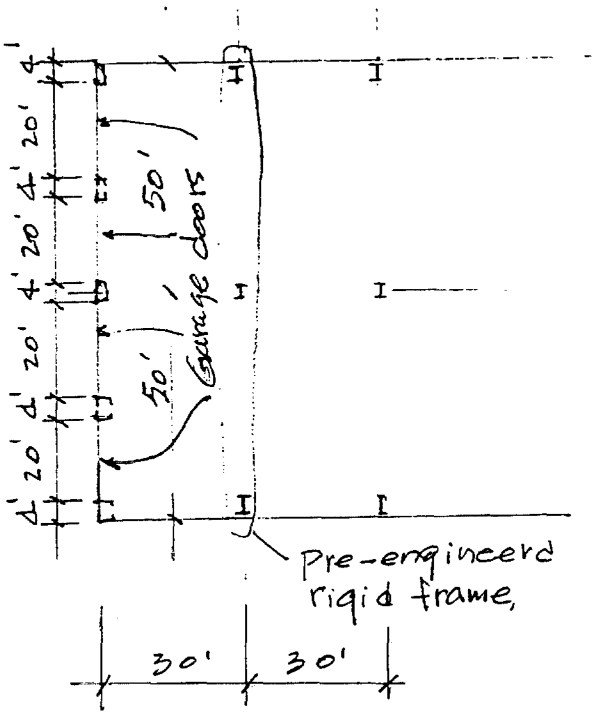
T F STEEL COLUMNS: The columns in the lateral-force-resisting system are substantially anchored to the building foundation. (Sec. 8.4.1)

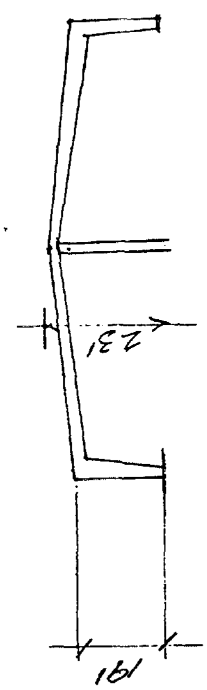
WALL AND ROOF PANELS

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS ROOF PANELS: All light-gage metal, plastic, or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center. (Sec. 8.6.1)

T F WALL PANELS: All wall panels (metal, fiberglass, or cement asbestos) are properly connected to the framing. (Sec. 8.6.2)

CHECK THE WEST END BAY OF THE VEHICLE BARN





THE RIGID frame was designed for Z = 3.
USING TM 5-809-10 (1982)

BASE SHEAR $V = Z\,I\,C\,K\,S\,W$

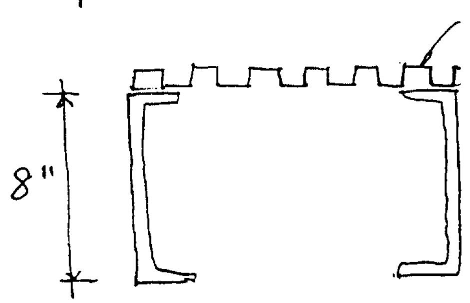
$Z = 0.75$
$I = 1.5$
$K = 1.33$
$CS = 0.14$

MASONRY WAS DESIGNED in accordance with TM-5-809-10.

THE RIGID frames and masory walls are judged to be designed for UBC zone 3.

The West end wall has 4 large garage doors and judged to have inadequate lateral load resisting strength.

The end wall is comprised of 10 8" [in the vertical direction. spaced 4' between two adjacent chanels.



1) Assuming each channel is acting independently.

$C \; 8 \times 11.5$

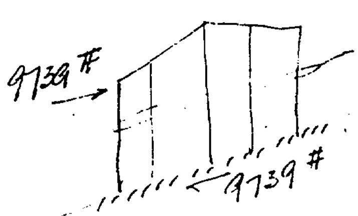
$I_{y-y} = 1.32 \; in^4$

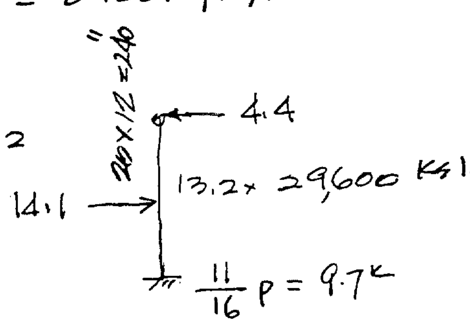
<u>Lateral load.</u>

Roof DL :	4.5 psf × 100' × 15'	=	6750 #
Roof LL :	25 × 100 × 15	=	37500
End Wall	15 psf × 100 × $\frac{19+23}{2}$	=	31500
			75750 #

$$V = \frac{ZIC}{R_w} W$$

$$= \frac{0.3 \times 1.5 \; (2)}{7} \times 75750 = 0.128 \times 75750 = 9739 \;\#$$





Check simple cantilever beam defl.

$$\Delta = \frac{P\ell^3}{3EI} = \frac{9.7 \times 240^3}{3 \times 13.2 \times 29600} = 114" = 0.475\,\ell \;(\text{big})$$

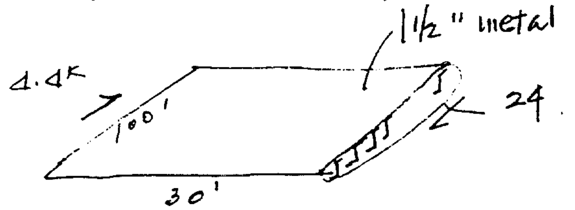
Participation of the roof in resisting the endwall deflection.

8" Z section @ 4'-0" o.c.

24 Z purlins.

$I_{y-y} = 0.306 \text{ in}^4$

$\Sigma I_{y-y} = 0.306 \times 24 = 7.344$



$A_z = 0.706 \text{ in}^2$

$Ad^2 = 2(0.706 \times \overline{600}^2) + 2(0.706 \times \overline{552}^2) + 2(0.706 \times \overline{456}^2)$
$+ 2(0.706 \times \overline{408}^2) + 2(0.706 \times \overline{384}^2) + 2(0.706 \times \overline{336}^2)$
$+ 2(0.706 \times \overline{288}^2) + 2(0.706 \times \overline{240}^2) + 2(0.706 \times \overline{192}^2)$
$+ 2(0.706 \times \overline{144}^2) + 2(0.706 \times \overline{96}^2) + 2(0.706 \times \overline{48}^2)$

$= 1,355,520 + 430,242 + 293605$
$+ 235047 + 208202 + 159409$
$+ 117117 + 81331 + 52052$
$+ 29279 + 13013 + 3253$

$= 2978070 \text{ in}^4$ Say effective on 30 %

$\Delta = \dfrac{Pl^3}{3EI} = \dfrac{4.4 \times (12 \times 30')^3}{3 \times 29600 \times 2978070 \times 0.3} = 0.24''$ Very small.

Diaphragm Shear Transfer

$V = 1.5\, A_v\, C_p\, W_d$
$= 1.5 \times 0.2 \times 0.6 \times 44250\# = 7965\#$ O.K.

Table C 6.1.1a
Capacity = 1800 #/ft × 100 ft = 180000 #

Building Designation : D

Location: Emmittsburg , MD

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1929 Year(s) remodelled: ________
Date of Evaluation: 6/23/98
Area, (sq. ft.) 28687 Length 212' Width 45' Photo Roll No. ____

CONSTRUCTION DATA

Roofframing: Concrete TRUSSES
Intermediate floor framing: Wood beams
Ground floor: Concrete Basement: Concrete
Exterior walls: Brick Masonry Openings: Windows
Columns: None Foundations: Stone + Brick Masonry
General condition of structure: FAIR / Good
Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB 15	MB 15
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R: 1.25 ________

EVALUATION DATA

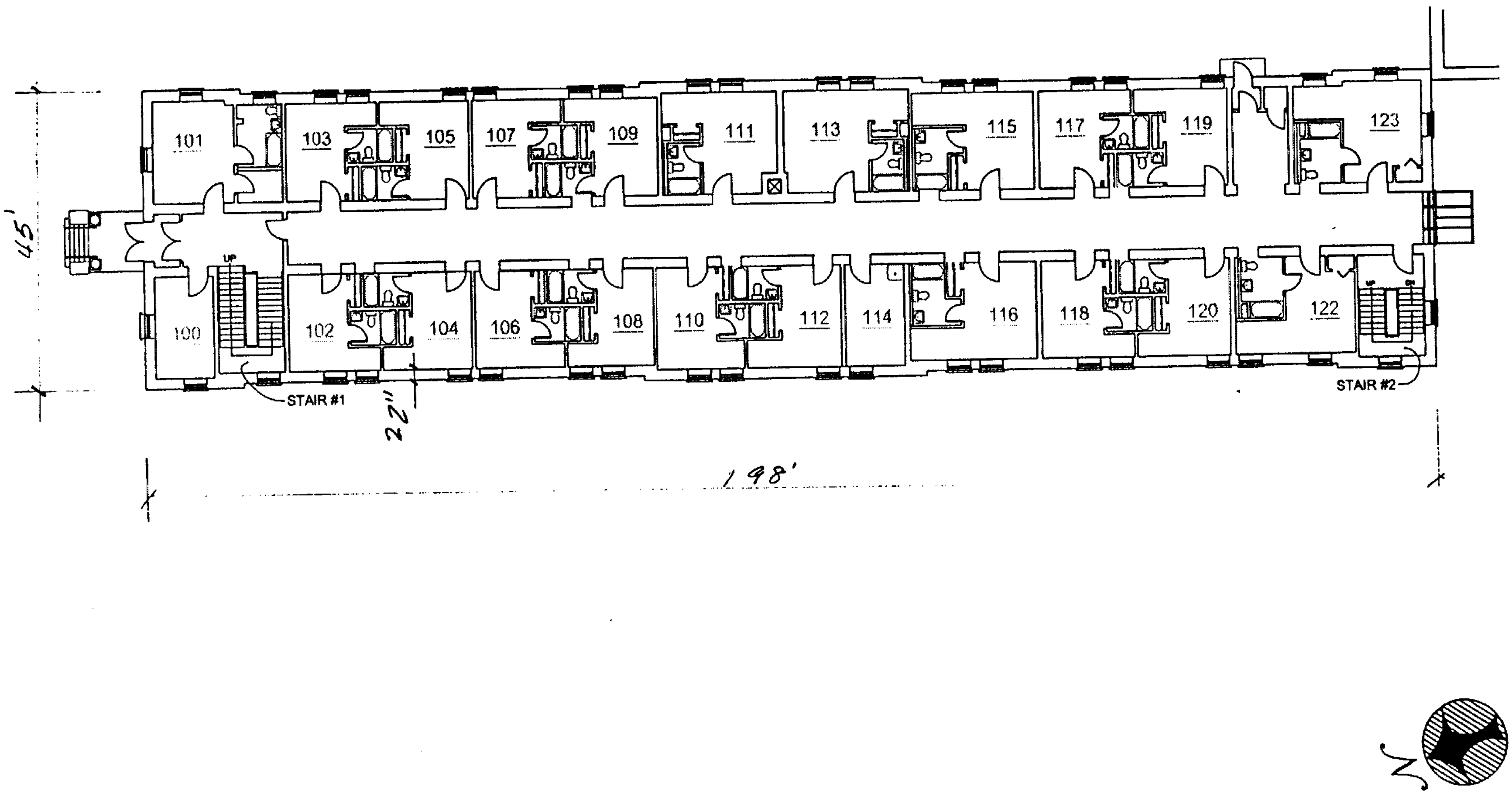
A_a = 0.05 A_v = 0.05

Site soil profile type: S2 Site soil coefficient, S = 1.2

REMARKS

Being used as dormitory

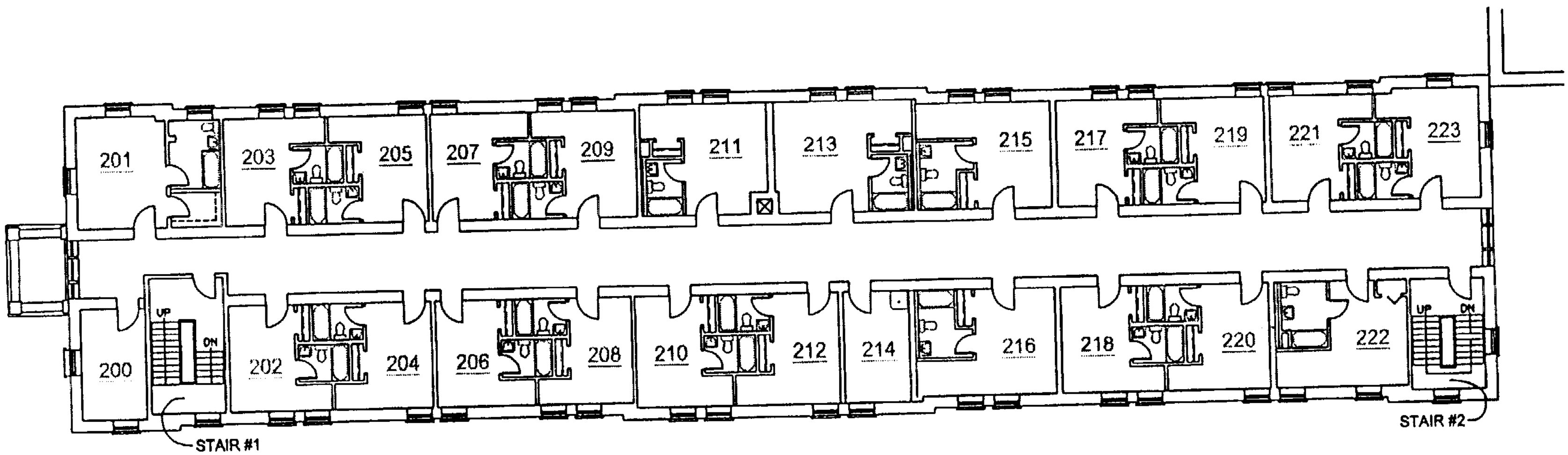

BUILDING "D" - WEST ELEVATION
BUILDING "D" - EAST ELEVATION
BUILDING "D" - NORTH ELEVATION
BUILDING "D" - SOUTH ELEVATION
NATIONAL EMERGENCY
TRAINING CENTER
EMMITSBURG, MARYLAND
BUILDING "D"
SETON HALL
ELEVATIONS
SITE KEY PLAN
1/8" = 1'-0"
1 OF 1

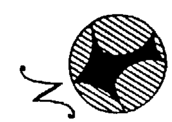


NATIONAL EMERGENCY TRAINING CENTER

BUILDING D FIRST FLOORPLAN

PSB 12/17/97 N.T.S.

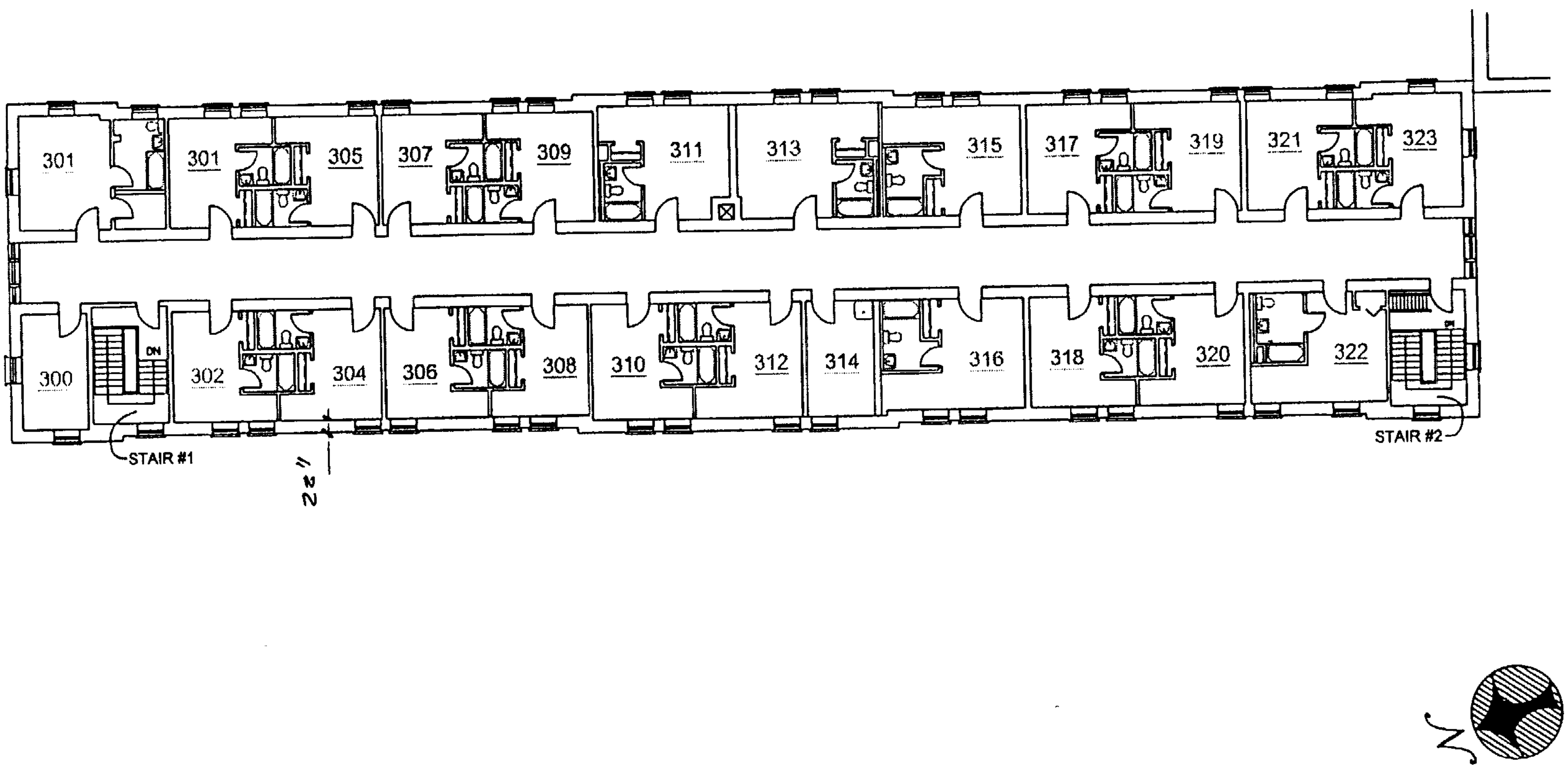




NATIONAL EMERGENCY TRAINING CENTER

BUILDING D SECOND FLOORPLAN

PSB 12/17/97 N.T.S.



NATIONAL EMERGENCY TRAINING CENTER

BUILDING D THIRD FLOORPLAN

PSB 12/17/97 N.T.S.

Building "D" Emmittsburg, MD

EVALUATION STATEMENTS FOR BUILDING TYPE 15: UNREINFORCED MASONRY BEARING WALL BUILDINGS

These buildings include structural elements that vary depending on the age of the building and, to a lesser extent, the geographic location of the structure. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In buildings built after 1950, unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. More recently, in regions of lower seismicity, these buildings can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can have the effect of reducing diaphragm displacements.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

(T) F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T (F) MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

(T) F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

(T) F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

(T) F ADJACENT BUILDINGS: There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated. (Sec. 3.4)

(T) F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

(T) F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

For buildings with wood diaphragms and unreinforced masonry bearing and enclosure walls at the perimeter, complete the evaluation using the procedure given in Appendix C. For other buildings, continue with the following evaluation statements.

MASONRY WALLS

(T) F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (Sec. 5.4.1)

(T) F PROPORTIONS: In areas of high seismicity (A_v greater than or equal to 0.2), the height-thickness ratio of the unreinforced masonry wall panels is as follows: (Sec. 5.5.1; also see Appendix C)

- One-story building: $h_w/t < 14$
- Multistory building:
  - Top story: $h_w/t < 9$
  - Other stories: $h_w/t < 20$

(T) F MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls have negligible voids. (Sec. 5.4.2)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than *50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

T F SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Sec. 7.2.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

CONNECTIONS

T F MASONRY WALL ANCHORS: Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (Sec. 8.2.3)

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4)

CHECK h/t ratio.

top story $= \frac{144}{22} = 6.54 < 9$ o.k

1st story $= 6.54 < 20$ o.k

Wgt of Bldg.

Longitudinal Wall : $198 \times 36 = 7128$

Windows : $(19 \text{ windows} \times 28) \times 3 = 1596$

NET AREA 5532 ⌀

TRANSVERSE (END) $45 \times 36 = 1620$

$(12 \times 28) \times 3 = 1008$

Net AREA $= 612$ ⌀

Wall surface area:

$(5532 + 612) \times 2 = 12288$ ⌀

Σ Weight 12288×243 #/⌀ $= \boxed{2,985,984}$ #.

Partition (12 1/2" Conc Block).

$194' \times 10' = 1940$ ⌀'

Doors $17 \times 3.5' \times 8' = 476$

1464 ⌀/story $\times 80$ #/⌀ $\times 2 = 233,600$ #

INTERIOR Part. (8")

$18' \times 10' \times 17 \times 55$ #/⌀ $= 168300$ #/story

3 stories $\times (233600 + 168,300) = \boxed{1,205,700}$ #.

Roof : Conc.

CEILING

$110\ \#/ft^3 \times 45' \times 198' \times 0.83' = 813,483\ \#$

Roof

$110 \times 50' \times 198 \times 0.67 = 694,465\ \#$

TRUSS $694\,465 \times 0.2 = 139\,293\ \#$

$1,647,241\ \#$

2nd & 3rd fl

$43' \times 196' \times 8\ \#/\square = 67424\ \#$

67424
134,848

Total DEAD Load at 1st FL Level

2985984
1205700
1647241
134848
5,973800 #

$V = C_s W = 0.17 \times 5973800 \simeq 1,015,500\ \#$

$C_s = \frac{2.12 A_a}{R} = \frac{2.12 \times 0.1}{1.25} = 0.17$ (short period bldg)

$Net\ A = \left[(198' - 19 \times 4) \times \frac{22}{12}\right] 2 = 447\ \square$

$v = \frac{1015500}{447 \times 12 \times 12} = 15.8\ psi \times 1.25 = 19.75\ psi$

$V_m = 0.56\, V_w + \frac{0.75\ P_D}{A} = 0.56 \times 60 + \frac{0.75\ (5973800)}{447 \times 144}$

$= 33.6 + 69.6 = 103\ psi$ OK

CHECK SHEAR AT Roof line

Total Roof + Ceiling load = 1647 241 #

Available Wall cross section = $[(198 - 19 \times 4) \times 18/12]2 = 366$ #

$$v = \frac{1647241}{366 \times 144} = 31.2 \text{ psi}$$

$$v_m = 0.56 \underset{\uparrow \atop 60}{\sqrt{f'_m}} + \frac{0.75\,(1647241)}{366 \times 144} = 33.6 + 23.4$$

$$= 57 \text{ psi}$$

CHECK MASS IRREGULARITY

Mass at 3rd floor ceiling level : 1647241 #

3rd floor + Walls (3rd story) + partitions : 469324 #

67424 + 233600 + 168 300

$$\text{Ratio of Masses} = \frac{1647241}{469324} = 3.5 > 1.5$$

Building Designation : J

Location: Emmittsburg, MD

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1965 Year(s) remodelled: 1992 (non-structural)
Date of Evaluation: 6/23/98
Area, (sq. ft.) 45673 Length N/A Width N/A Photo Roll No. ____

CONSTRUCTION DATA

Roofframing: Concrete joists
Intermediatefloorframing: Concrete joists
Ground floor: Conc. Joists Basement: Concrete
Exterior walls: Masonry infill Openings: ____
Columns: Concrete Foundations: Concrete wall and spread footing
General condition of structure: Very Good
Evidence of settling: none

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB 10	MB10
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

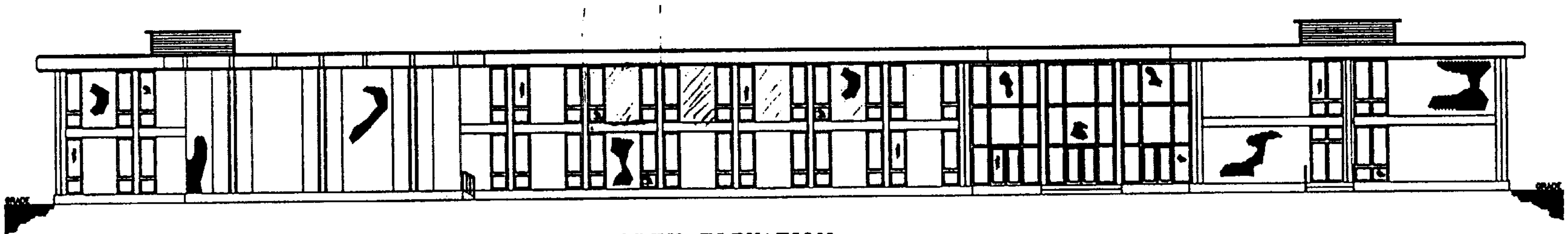
Response Modification Coefficient, R: 5 ____

EVALUATION DATA

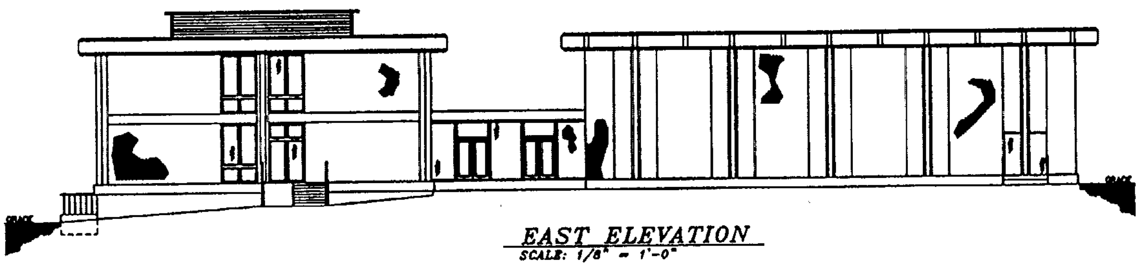
A_a = 0.05 A_v = 0.05

Site soil profile type: S2 Site soil coefficient, S = 1.2

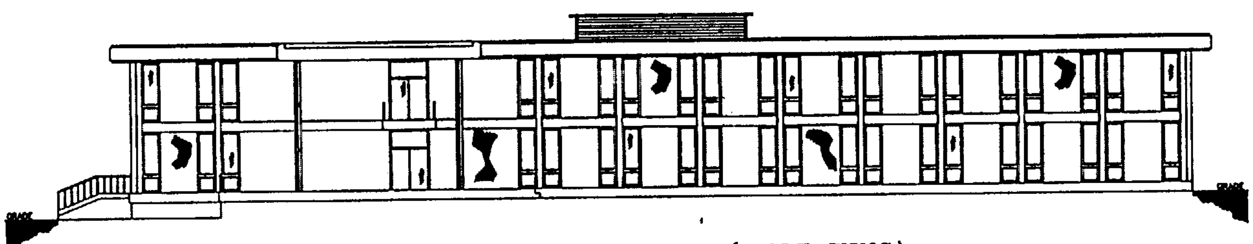
REMARKS



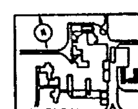
NORTH ELEVATION
SCALE: 1/8" = 1'-0"



EAST ELEVATION
SCALE: 1/8" = 1'-0"



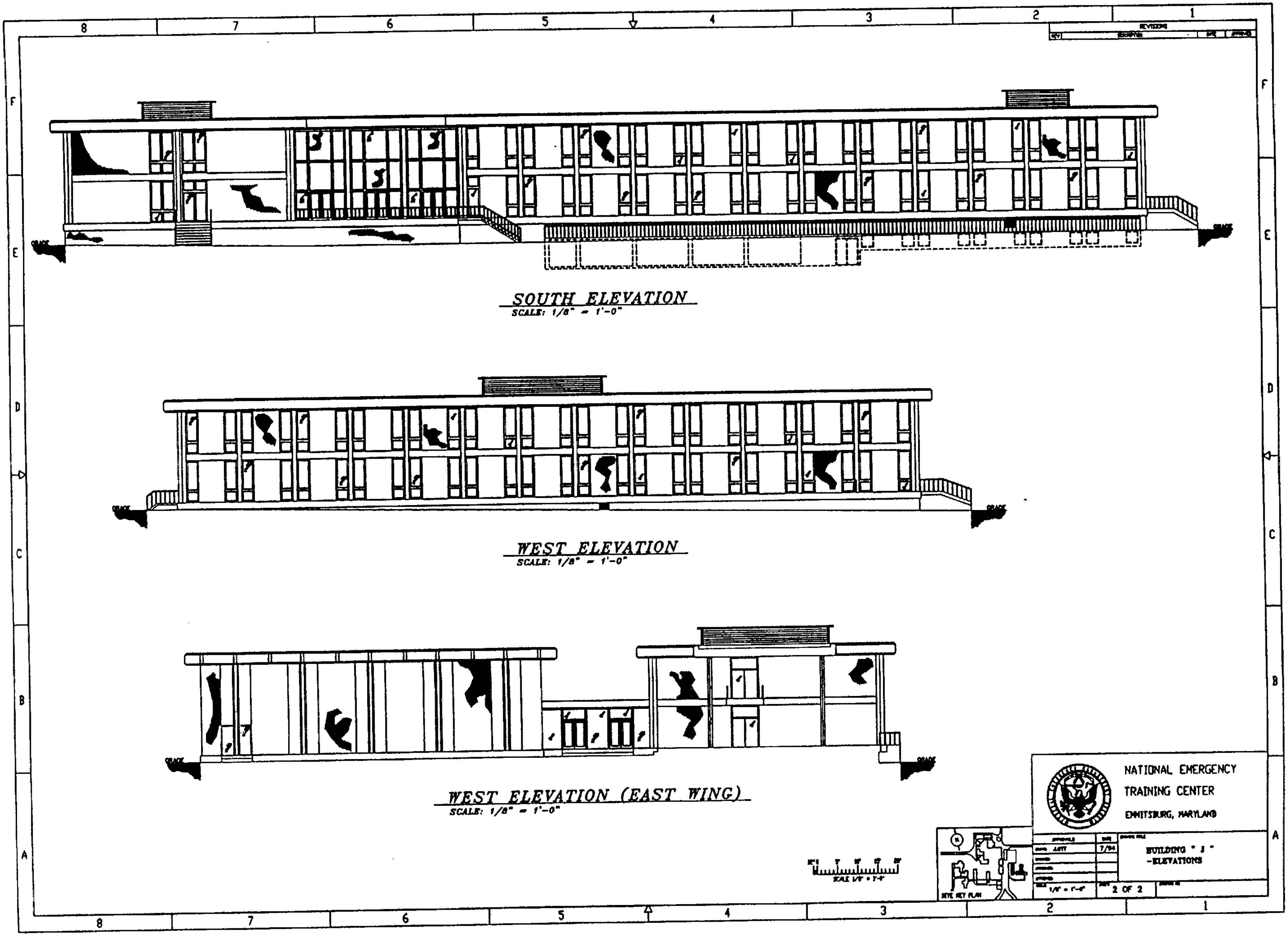
EAST ELEVATION (WEST WING)
SCALE: 1/8" = 1'-0"

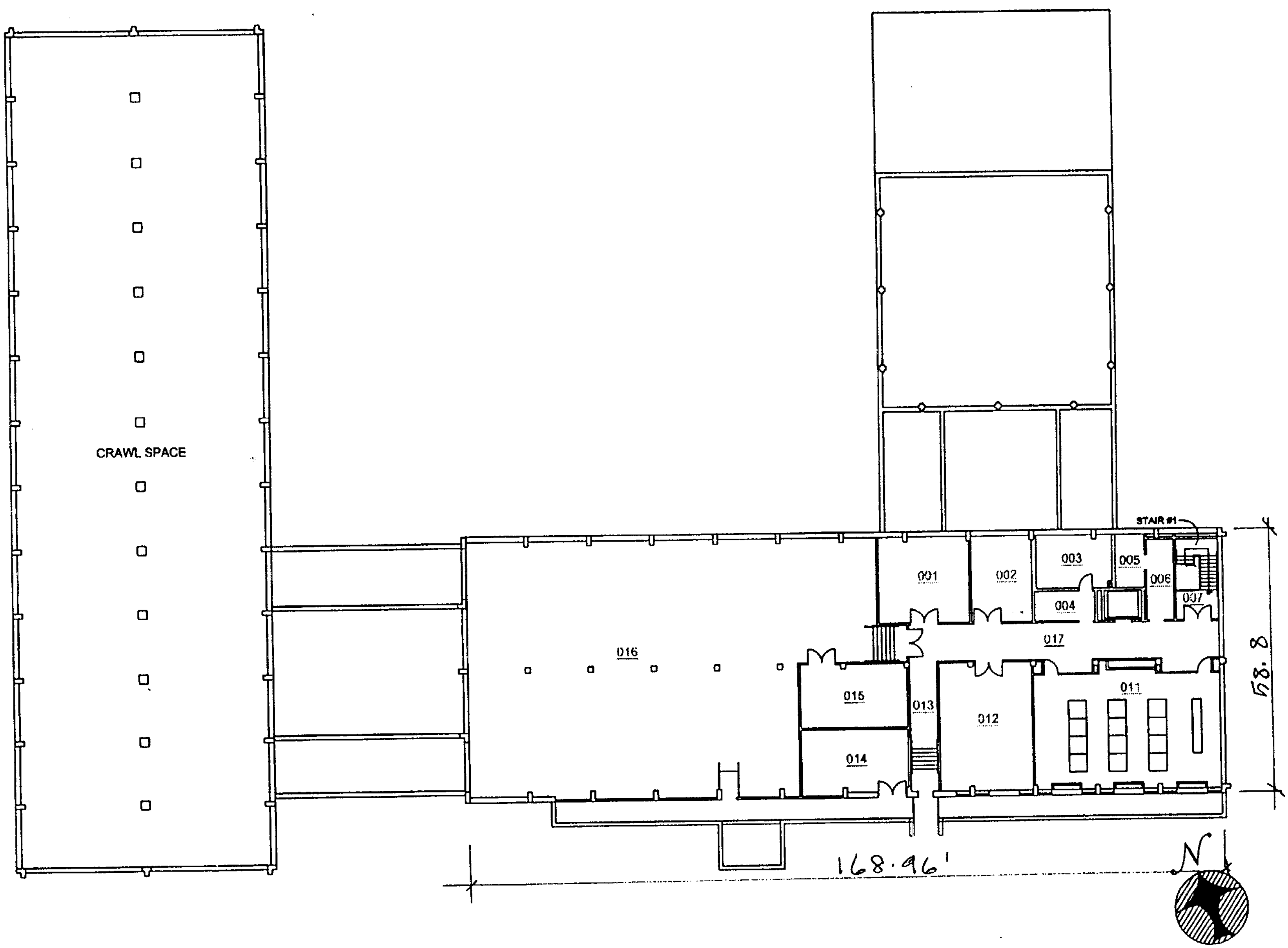


NATIONAL EMERGENCY
TRAINING CENTER
EMMITSBURG, MARYLAND

BUILDING "J"
-ELEVATIONS

APPROVALS	DATE	
DRAWN ...	7/94	
SCALE 1/8" = 1'-0"	1 OF 2	

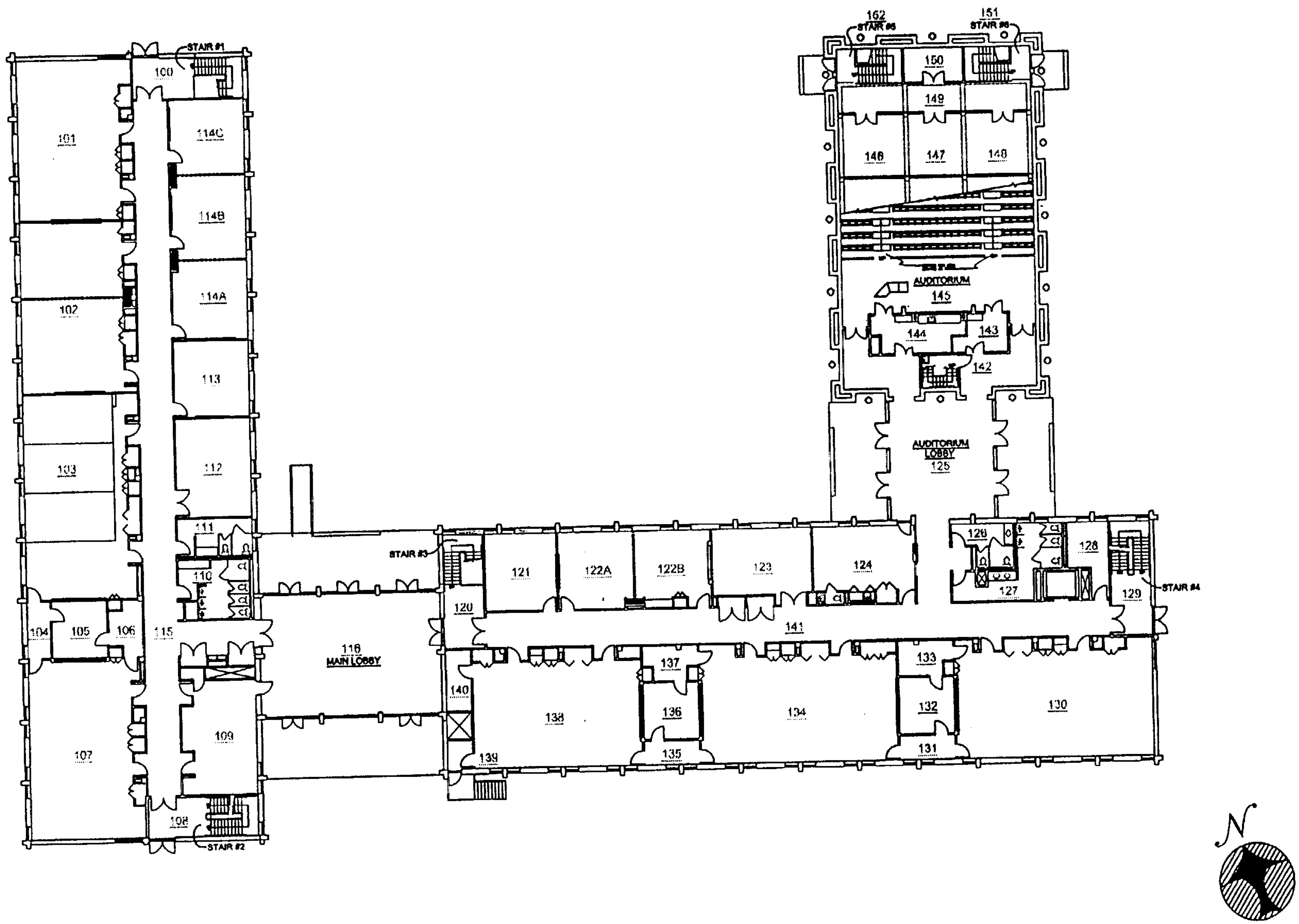

SOUTH ELEVATION
SCALE: 1/8" = 1'-0"
WEST ELEVATION
SCALE: 1/8" = 1'-0"
WEST ELEVATION (EAST WING)
SCALE: 1/8" = 1'-0"
NATIONAL EMERGENCY
TRAINING CENTER
EMMITSBURG, MARYLAND
SITE KEY PLAN
7/94
BUILDING " J "
-ELEVATIONS
1/8" = 1'-0"
2 OF 2



NATIONAL EMERGENCY
TRAINING CENTER

BUILDING J BASEMENT FLOORPLAN

PSB 01/20/98 - N.T.S.

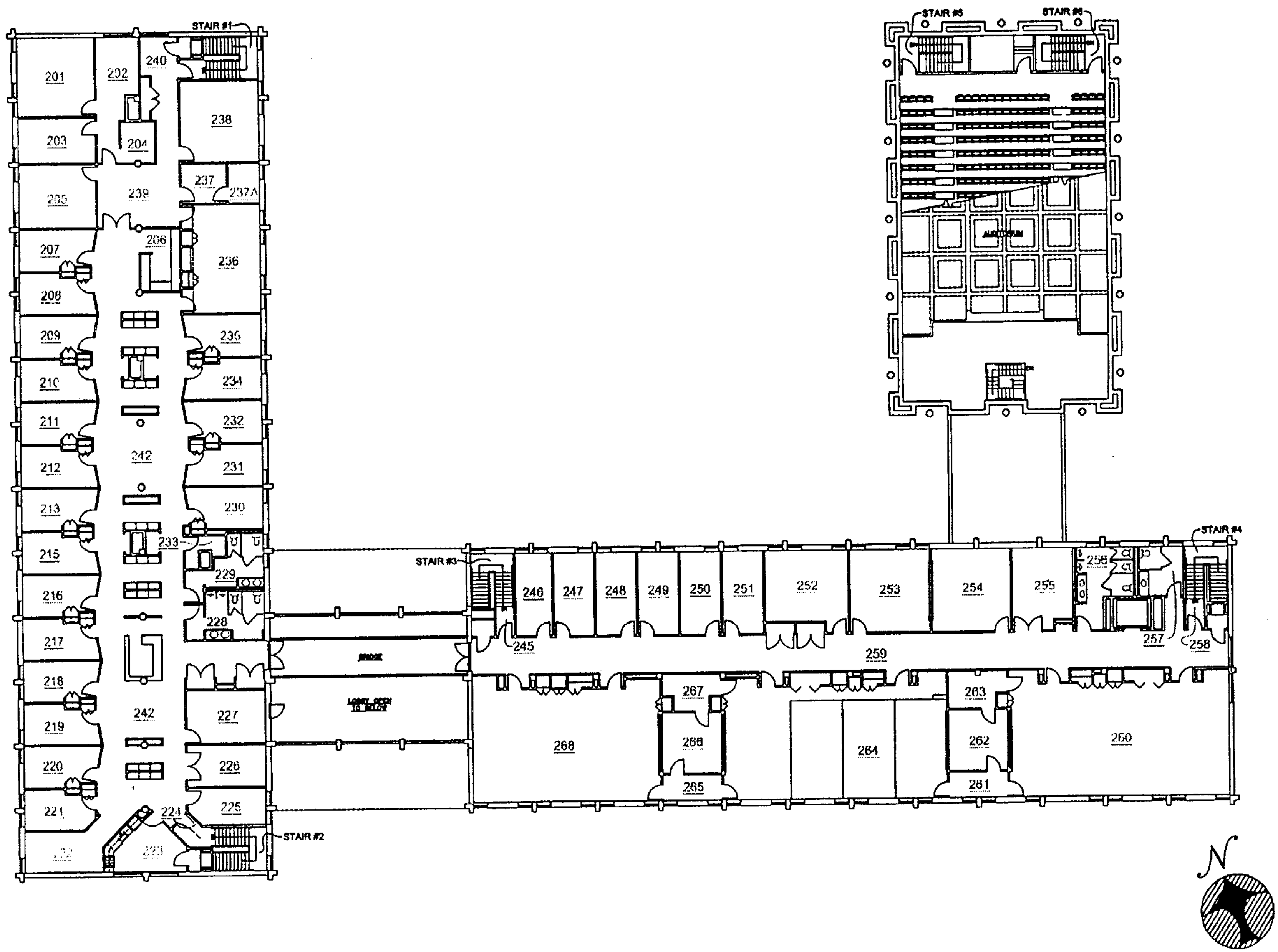


NATIONAL EMERGENCY TRAINING CENTER

BUILDING J FIRST FLOOR FLOORPLAN

PSB 8/20/97 N.T.S.

Building J Second Floor Floorplan.dwg



NATIONAL EMERGENCY TRAINING CENTER

BUILDING J SECOND FLOOR

PSB 01/20/98 - N.T.S.

Bldg J, Emmittsburg, MD

EVALUATION STATEMENTS FOR BUILDING TYPE 10: CONCRETE FRAME WITH INFILL SHEAR WALLS

These buildings are similar to Type 7 except that the frame is of reinforced concrete. The analysis of this building is similar to that recommended for Type 7 except that the shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. Research that is specific to confinement of the infill by reinforced concrete frames should be used for the analysis.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITIES: All infill walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

(T) F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

(T) F CRACKS IN INFILL WALLS: There are no diagonal cracks in the infilled walls that extend throughout a panel or are greater than 1.0 mm wide. (Sec. 3.5.11)

(T) F CRACKS IN BOUNDARY COLUMNS: There are no diagonal cracks wider than 1.0 mm in concrete columns that encase the masonry infills. (Sec. 3.5.7)

SHEAR WALLS

(T) F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the masonry infill walls. (Sec. 5.3.1 for reinforced masonry; Sec. 5.4.1 for unreinforced masonry)

(T) F PROPORTIONS: In areas of high seismicity (A_v greater than or equal to 0.2), the height-thickness ratio of the unreinforced masonry wall panels is as follows: (Sec. 5.5.1; also see Appendix C)

- One-story building: $h_w/t < 14$ $\frac{12 \times 12}{12} = 12 < 14$
- Multistory building:
  - Top story: $h_w/t < 9$
  - Other stories: $h_w/t < 20$

(T) F SOLID WALLS: The infilled walls are not of cavity construction. (Sec. 5.5.2)

(T) F CAVITY WALLS: The infill walls are continuous to the soffits of the frame beams. (Sec. 5.5.3)

T (F) WALL CONNECTIONS: All infill panels are constructed to encompass the frames around their entire perimeter. (Sec. 5.5.4)

(T) F REINFORCING: In areas of high seismicity (A_v greater than or equal to 0.2), the total vertical and horizontal reinforcing steel in reinforced masonry walls is greater than 0.002 times the gross area of the wall with a minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 inches; and all vertical bars extend to the top of walls. (Sec. 5.3.2) N/A

MOMENT FRAMES

(T) F COMPLETE FRAMES: The concrete frames form a complete vertical load carrying system. (Sec. 4.5.1)

DIAPHRAGMS

(T) F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1) N/A

(T) F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. (Sec. 7.1.3) N/A

T F SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Sec. 7.2.4)

CONNECTIONS

T F WALL ANCHORAGE: The exterior concrete or masonry walls are anchored to each of the diaphragm levels for out-of-plane loads. (Sec. 8.2.2)

Not known

Wgt of Bldg

Roof : 74#/sq ft × 169' × 59' = 737,854 #

2nd FL
78 × 169 × 59 = 777,738

L.L effect + equipment etc = 233321 (30% D.L).

WALL.

1.25' × 14.08' × 12' = 211.2 ft3

24 (211.2 × 150) = 760,320 #.

CEILING

1+1 #/sq ft

Partition

14 #/sq ft

15 #/sq ft × 169 × 59 = 159,536#

Roofing
6 #/sq ft × 169 × 59 = 59,826#

Total D.L + L.L at the 1st fl. level.

59,826
737,854
777 738
233 321
760, 320
760, 320
159, 536
3,488, 915 #

$C_s = \frac{2.12\ A_a}{R} = \frac{2.12 \times 0.1}{5} = 0.0424$

$V = 0.0424 \times 3488915 = 147,930$ #

Available wall area = 24 × 15" × 78" = 28080 □"

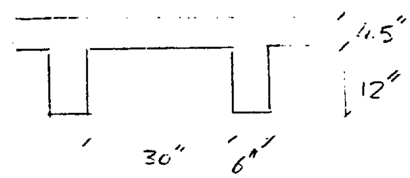
$$v = \frac{147930}{28080} = 5.27 \text{ psi}$$ O.K.

$$v_{allowable} = 2\sqrt{f'_c} = 2\sqrt{3000} = 109.5 \text{ psi}$$

Emmitsburg, MD

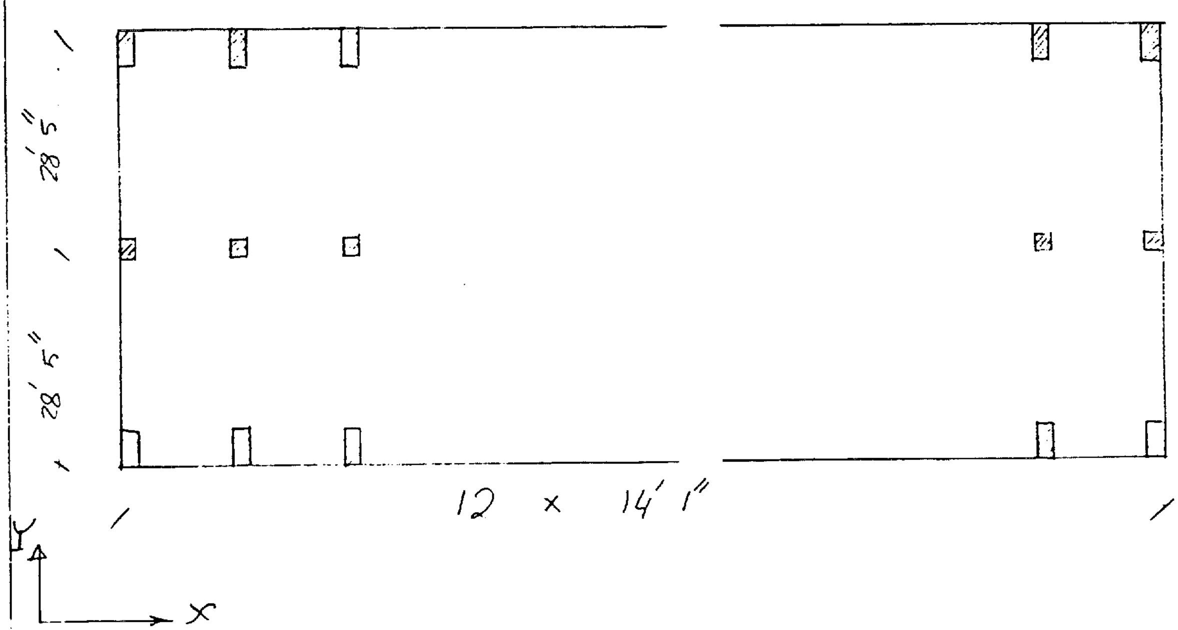
Assumptions:

- $f'_c = 4000$ psi
- Floors: Concrete joist Construction 12 + 4.5 x 6 + 30



- Columns: Perimeter 12" x 24"
  interior 12" x 12"
- Live Load: 50 psf
  Snow Load 30 psf
- $E_{Conc.} = 3,640,000$ psi
- floor height = 12'
- Floors act as rigid diaphragms

PLAN:



Loads / floor :

Joist floor :	85 psf
bridging :	2
Partitions :	20
Ceiling / mech	12
DL	119 psf

Lateral Force Estimation

* Roof weight including 30 psf snow load:

$W_1 = (119 + 30) * 57 * 12 * 14.1 = 1,437,016$ lbs
$= 1437^k$

* 1st floor weight including 25% of L.L.

$W_2 = (119 + \frac{50}{4}) * 57 * 12 * 14.1 = 1,268,239$ lbs
$= 1268^k$

$W = 1437 + 1268 = 2705^k$

* Assume natural Period of Building = 0.2 s

* $C_s = \dfrac{2.12\ A_a}{R}$

Maryland $\rightarrow A_a = .05$
Ordinary moment frame $\rightarrow R = 2$
$C_d = 2$

$C_s = \dfrac{2.12 \times .05}{2} = 0.053$

* Base Shear $V = C_s W = 0.053 * 2705 = 143.365^k$

* Force at top floor $= \dfrac{1437 * 2}{1437 * 2 + 1268} * 143.365 = 99.5^k$

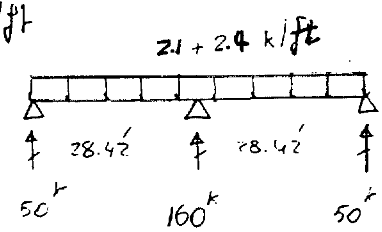
" " 1st floor $= 43.9^k$

* Vertical Reactions in Columns

Roof load $= (119 + 30) * 14.1 = 2101$ p/ft $= 2.1$ k/ft

floor load $= (119 + 50) * 14.1 = 2383$ p/ft $= 2.4$ k/ft

interior Col. } Ground level 160 k
exterior Col. } Ground level 50 k



Frames in Y - direction

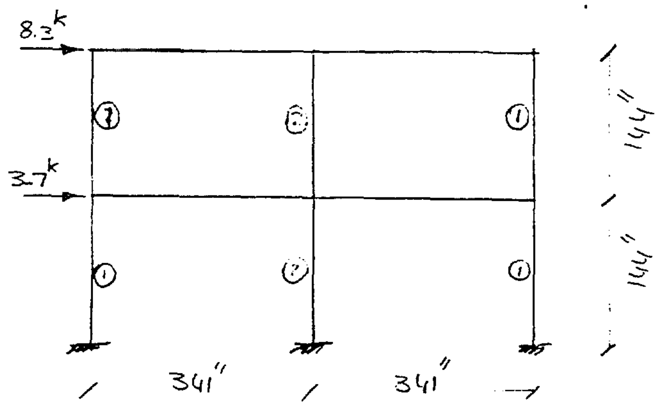
for an intermediate frame, force at top floor = $99.5/12 = 8.3^k$

" " 1st " = $43.9/12 = 3.7^k$

$A_{Col.1} = 12 * 24 = 288\ in^2$ | $I_{Col.1} = \dfrac{12(24)^3}{12} = 13{,}824\ in^4$

$A_{Col.2} = 12 * 12 = 144\ in^2$ | $I_{Col.2} = \dfrac{12(12)^3}{12} = 1{,}728\ in^4$

$A_{beam} = 12 * 16.5 = 198\ in^2$ | $I_{beam} = \dfrac{12(16.5)^3}{12} = 4{,}492\ in^4$

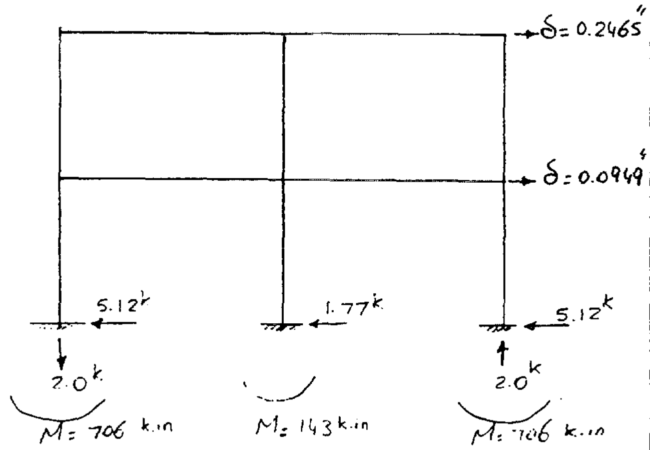


Analysis Results:

Lateral drift

top floor $\Delta = 2 * .2465 = 0.493''$

1st floor $\Delta = 2 * .0949 = 0.190''$



$f'_c = 4$ ksi
$f_y = 60$ ksi
$\gamma = 0.8$
$\rho = 0.01$

* interior Column:

$P_n = 1.6 * 160 = 256$ k.

$M_n = 143$ k. in.

$P_n / f'_c A_g = 256 / (4 * 12 * 12) = 0.444$

from interaction diagram $\frac{(M_n)_{max}}{f'_c A_g h} = 0.14$

$(M_n)_{max} = 0.14 * 4 * (12 * 12) * 12 \cong 970$ k. in.

Safe

* Exterior Column:

$P_n = 1.6 * 50 = 80^k$

$M_n = 706$ k. in.

$P_n / f'_c A_g = 80 / (4 * 12 * 24) = 0.07$

$\frac{(M_n)_{max}}{f'_c A_g h} = 0.08$

$(M_n)_{max} = 0.08 \times 4 * (12 * 24) * 24 = 2212$ k. in.

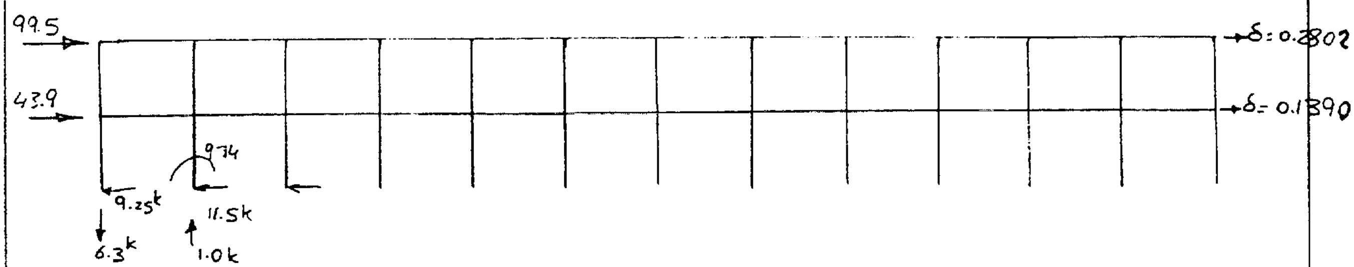
Safe

Frames in X-direction

The 3 Column Lines are lumped in 1 frame

beams $\quad A = 3 * 12 * 16.5 = 594 \text{ in}^2 \qquad I = 3 * \frac{12(16.5)^3}{12} = 13476 \text{ in}^4$

Columns $\quad A = 2(12 * 24) + 12 * 12 = 720 \text{ in}^2 \qquad I = 2\left[\frac{24(12)^3}{12}\right] + \frac{12(12)^3}{12} = 8640 \text{ in}^2$



Max. Moment = 974 k. in

Analysis Results:

Lateral drift : top floor $\Delta = 2 * .28 = 0.56''$

1st floor $\Delta = 2 * .139 = 0.278''$

Moment / exterior Col. $= \frac{24(12)^3/12}{8640} * 974 = 390$ k. in.

Moment / interior Col. $= \frac{12(12)^3/12}{8640} * 974 = 195$ k. in.

* Interior Column:

$$P_n / f'_c A_g = 0.444$$

$$M_n = 195 \text{ k.in.}$$

$$(M_n)_{max.} = 970 \text{ K.in.}$$

Safe

* Exterior Column:

$$P_n / f'_c A_g = 0.07$$

$$\frac{(M_n)_{max}}{f'_c A_g h} = 0.08$$

$$M_n = 390 \text{ k.in.}$$

$$(M_n)_{max} = 0.08 * 4 (12 * 24) * 12 = 1106 \text{ k.in.}$$

Safe

Building Designation : "O"

Location: Emmittsburg, MD

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1839 Year(s) remodelled: 1970's
Date of Evaluation: 6/23/98
Area, (sq. ft.) 15370 Length 124' Width 68' Photo Roll No. ____

CONSTRUCTION DATA

Rooframing: Timber Trusses
Intermediate floor framing: Timber members
Ground floor: Timber Basement: Concrete
Exterior walls: Stone masonry Openings: Large windows
Columns: Brick Foundations: Stone masonry
General condition of structure: FAIR
Evidence of settling: Not Noticeable

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB15	MB15
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R:	1.25	

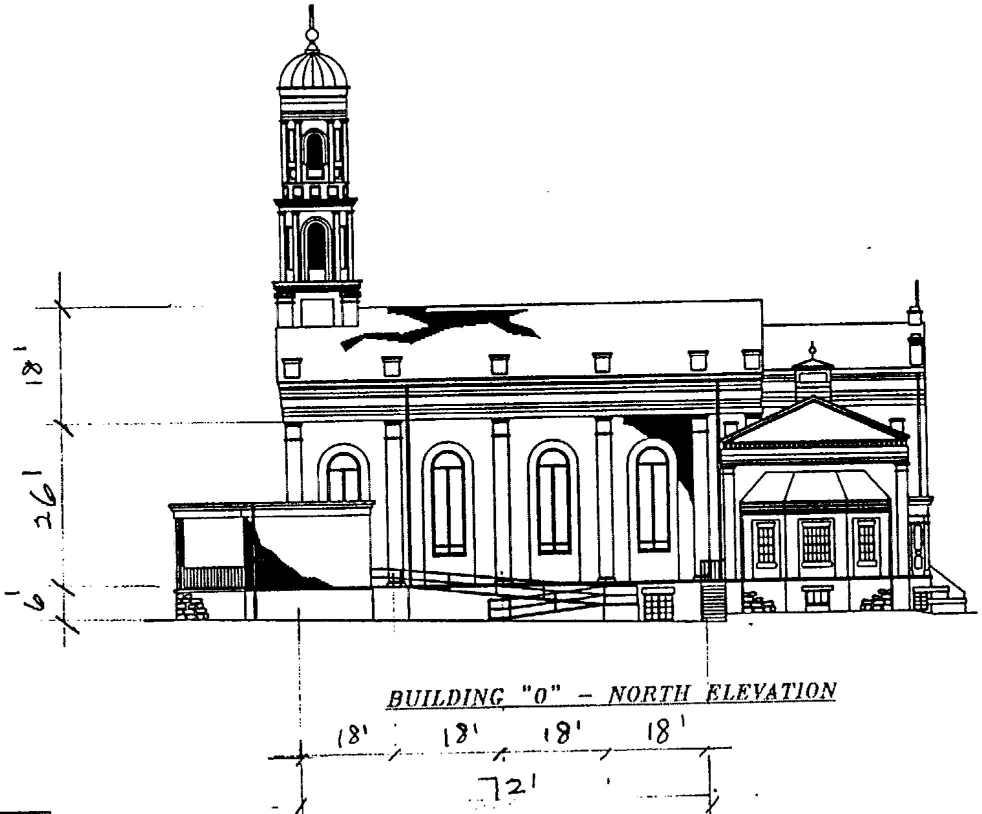
EVALUATION DATA

A_a = 0.05 A_v = 0.05

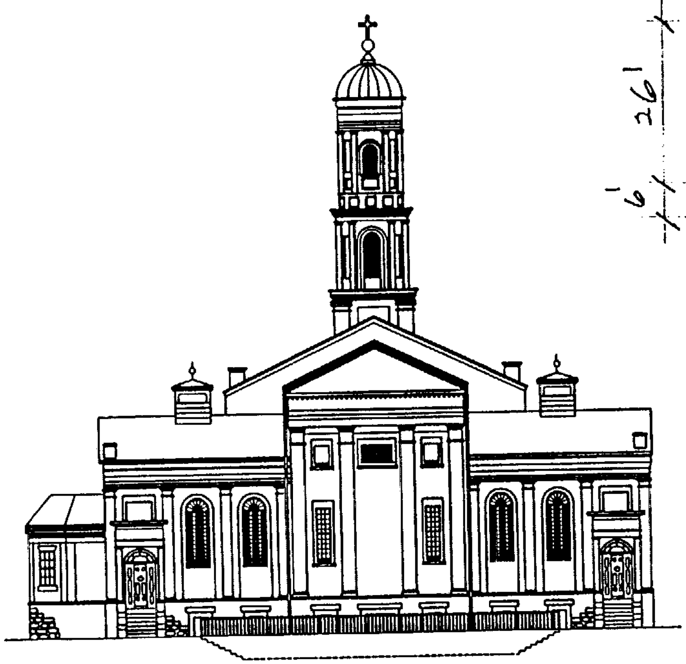
Site soil profile type: S2 Site soil coefficient, S = 1.2

REMARKS

Historic resister.
Timber steeple needs a special attention.

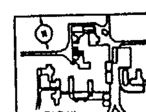


BUILDING "O" - NORTH ELEVATION



BUILDING "O" - WEST ELEVATION

SCALE 1/8" = 1'-0"



NATIONAL EMERGENCY
TRAINING CENTER
EMMITSBURG, MARYLAND

J. OTT

BUILDING "O"
ELEVATIONS

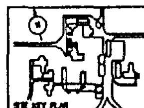
1/8" = 1'-0"

2 of 2

BUILDING "O" - SOUTH ELEVATION

18' 18'

BUILDING "O" - EAST ELEVATION



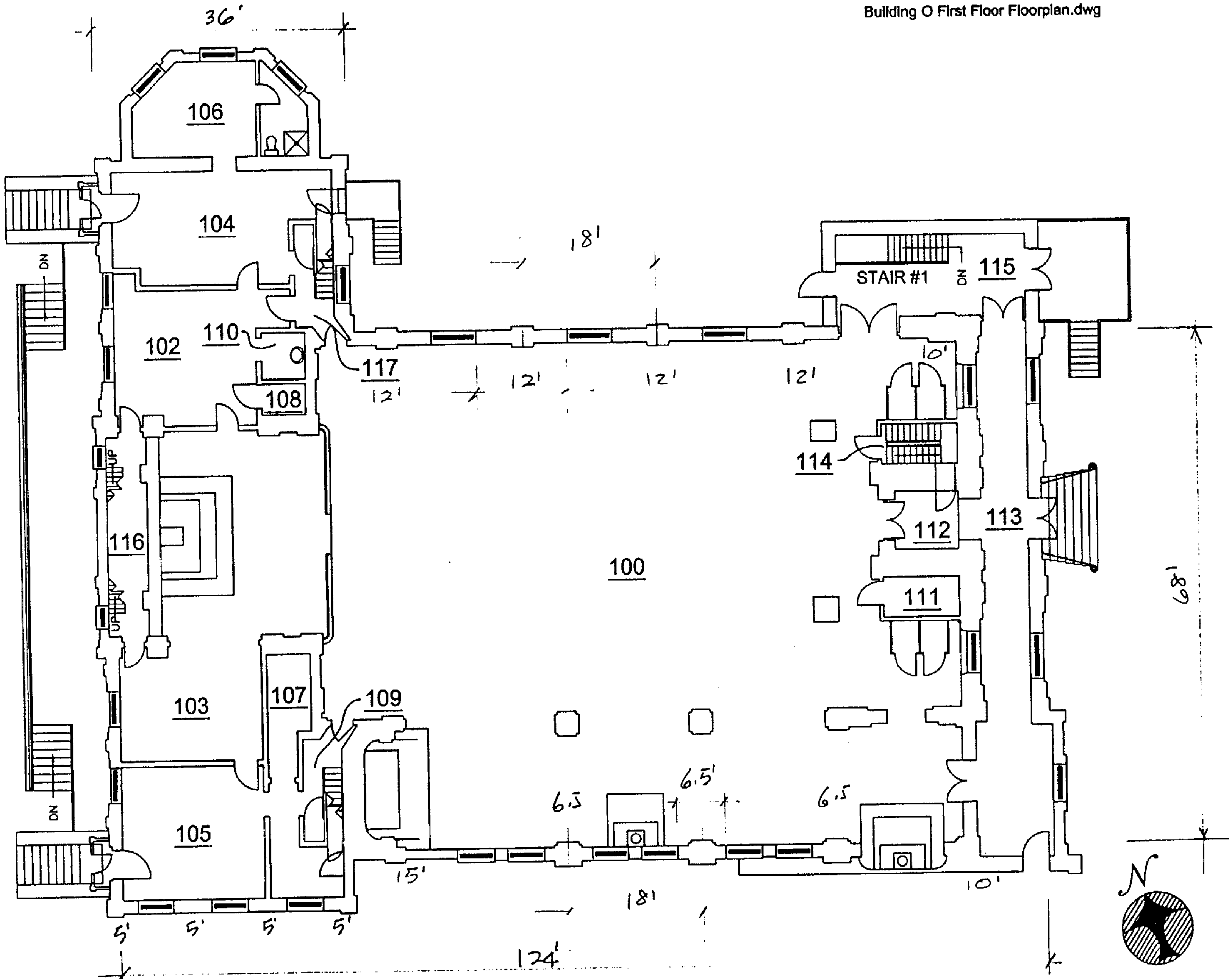
NATIONAL EMERGENCY
TRAINING CENTER
EMMITSBURG, MARYLAND

BUILDING "O"
ELEVATIONS

1/8" = 1'-0"

1 OF 2

Building O First Floor Floorplan.dwg



NATIONAL EMERGENCY TRAINING CENTER

BUILDING O FIRST FLOOR

PSB 01/09/98 - N.T.S.

Building "O", Emmittsburg, MD

EVALUATION STATEMENTS FOR BUILDING TYPE 15: UNREINFORCED MASONRY BEARING WALL BUILDINGS

These buildings include structural elements that vary depending on the age of the building and, to a lesser extent, the geographic location of the structure. In buildings built before 1900, the majority of floor and roof construction consists of wood sheathing supported by wood subframing. In large multistory buildings, the floors are cast-in-place concrete supported by the unreinforced masonry walls and/or steel or concrete interior framing. In buildings built after 1950, unreinforced masonry buildings with wood floors usually have plywood rather than board sheathing. More recently, in regions of lower seismicity, these buildings can include floor and roof framing that consists of metal deck and concrete fill supported by steel framing elements. The perimeter walls, and possibly some interior walls, are unreinforced masonry. The walls may or may not be anchored to the diaphragms. Ties between the walls and diaphragms are more common for the bearing walls than for walls that are parallel to the floor framing. Roof ties usually are less common and more erratically spaced than those at the floor levels. Interior partitions that interconnect the floors and roof can have the effect of reducing diaphragm displacements.

> Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.
>
> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1) No effective means of transferring the steeple mass to foundation.

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1) Many large window openings

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITIES: All shear walls are continuous to the foundation. (Sec. 3.3.5)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F ADJACENT BUILDINGS: There is no immediately adjacent structure that is less than half as tall or has floors/levels that do not match those of the building being evaluated. A neighboring structure is considered to be "immediately adjacent" if it is within 2 inches times the number of stories away from the building being evaluated. (Sec. 3.4)

T F MASONRY UNITS: There is no visible deterioration of large areas of masonry units. (Sec. 3.5.10)

T F MASONRY JOINTS: The mortar cannot be easily scraped away from the joints by hand with a metal tool, and there are no significant areas of eroded mortar. (Sec. 3.5.9)

> For buildings with wood diaphragms and unreinforced masonry bearing and enclosure walls at the perimeter, complete the evaluation using the procedure given in Appendix C. For other buildings, continue with the following evaluation statements.

MASONRY WALLS

T F SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in the unreinforced masonry shear walls. (Sec. 5.4.1)

T F PROPORTIONS: In areas of high seismicity (A_v greater than or equal to 0.2), the height-thickness ratio of the unreinforced masonry wall panels is as follows: (Sec. 5.5.1; also see Appendix C)

- One-story building: $h_w/t < 14$
- Multistory building:
  - Top story: $h_w/t < 9$
  - Other stories: $h_w/t < 20$

T F MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls have negligible voids. (Sec. 5.4.2)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than *50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

T F SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing. (Sec. 7.2.4)

T F SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater than *2 to 1. (Sec. 7.2.1)

CONNECTIONS

T F MASONRY WALL ANCHORS: Wall anchorage connections are steel anchors or straps that are developed into the diaphragm. (Sec. 8.2.3) Not known

T F ANCHOR SPACING: The anchors from the floor and roof systems into exterior masonry walls are spaced at 4 feet or less. (Sec. 8.2.4) Not known

Buiding O - Emmittsburg, MD 7-27-98

$A_v = 0.1$, $A_a = 0.1$

Roof : straight sheating $v = 300$ #/ft.

Length of shear wall at 1^{st} floor $\approx$ 198'

$h_w/t = \frac{26}{1.5} = 17.3 < 14$ O.K. $A_v \leq 0.1$.

Bldg weight

130 #/ft^3 for masonry

$250' + 250' + 71' + 100' \simeq 670'$ perimetr

vol = $670' \times 1.5'$ (thickness) $\times 24' = 24120$ ft^3

W (masonry) = $130 \times 24120 \simeq$ $\underline{3130\ K}$

Roof : Timber $\simeq 45$ #/ft

$45 \times 124 \times 68 \simeq$ $\underline{380}^{K}$

Ceiling : Wood lath + plaster : 20 #/ft

$20 \times 124 \times 68 \approx 170^{K}$.

W (Σ Dead weight) = $3130 + 380 + 170 = 3680^{K}$.

$V = C_s W$.

$C_s = \frac{2.12\ A_a}{R}$ for short period buildings.

$$C_s = \frac{2.12 \times 0.1}{1.25} = 0.17$$

$$V = 0.17 \times 3680^K \cong \underline{626^K}$$

LL effect

$$V = \frac{626\,000^{\#}}{34128} \cong 18 \text{ psi} \times 1.25 = 22.5 \text{ psi}$$

$$V_m = 0.56 \underset{\text{say } 40}{\underset{\uparrow}{V_w}} + \frac{0.75\, P_D}{A} = 0.56 \times 40 + \frac{0.75\,(3680\,000)}{34128}$$

$$= 22.4 + 8.08 = 30.5 \text{ psi} > 22.5 \text{ psi} \quad \underline{\underline{O.K.}}$$

Bldg "O"

OPTION 2 COST ESTIMATION FORM

COST ESTIMATION OPTION 2

1. GROUP MEAN COST
   - Group:
     - ☒ URM ☐ S1
     - ☐ W1, W2 ☐ S2, S5
     - ☐ PC1,RM1 ☐ S5
     - ☐ C1, C3 ☐ C2, PC2, RM2, S4
   - Cost Coefficient C_1 from Table 4.3.2. — C_1 = 15.29

2. AREA ADJUSTMENT FACTOR
   - Area
     - ☐ Less than 10K sq. ft. ☒ 10K - 50K sq. ft.
     - ☐ 50K - 100K sq. ft. ☐ 10K - 50K sq. ft.
   - Cost Adjustment Factor C_2 from Table 4.3.3 — C_2 = 1.00

3. SEISMICITY/PERFORMANCE OBJECTIVE FACTOR ADJUSTMENT
   - SEISMICITY
     - ☒ Low (NEHRP 1 or 2) ☐ Moderate (NEHRP 3 or 4)
     - ☐ High (NEHRP 5 or 6) ☐ Very High (NEHRP 7)
   - PERFORMANCE OBJECTIVE
     - ☒ Life Safety ☐ Damage Control ☐ Immediate Occupancy
   - Cost Adjustment Factor C_3 from Table 4.4.2 — C_3 = 0.61

4. LOCATION ADJUSTMENT FACTOR
   - City / State Emmittsburg, MD
   - Cost Adjustment Factor C_L from Table 4.3.4 or Table 4.3.5 — C_L = 0.98

5. TIME ADJUSTMENT FACTOR
   - Year 1998
   - Inflation Rate 2 %
   - Cost Adjustment Factor C_T from Table 4.3.6 — C_T = 1.10

TYPICAL STRUCTURAL COST
($C = C_1 \times C_2 \times C_3 \times C_L \times C_T$) — C = 10.05

Building Area (Square Foot) : A = 15370

Estimated Structural Cost (A x C) — C_S = (154,500)

Historical 300 % — 463,500

Non-Structural Cost ($C_1 \times C_L \times C_T$)

\$16/ft × 0.98 × 1.1 = \$17.25/ft × 3 — C_{NS} = 790,800

Finishing Cost \$42.05/ft × 15370 — C_F = 646,300

Total (Structural + Non-Struc + Finishing) — C_{ST} = 1,900,600

Project Cost (C_{ST} x 0.3) — C_P = 570,180

Total Cost ≈ 2,471,000

Building Designation : Maynard Federal Regional Center

Location: Maynard, MA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1968 Year(s) remodelled: ______
Date of Evaluation: 9/8/98
Area, (sq. ft.) 80,000 Length 140' Width 120' Photo Roll No. ____

CONSTRUCTION DATA

Rooframing: Concrete beams and slab
Intermediate floor framing: Concrete beams and slab
Ground floor: N/A Basement: N/A
Exterior walls: N/A Openings: ______
Columns: Concrete Foundations: Concrete footing
General condition of structure: Very Good
Evidence of settling: None

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB 16	MB16
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

______

Response Modification Coefficient, R: N/A ______

EVALUATION DATA

A_a = 0.10 A_v = 0.10

Site soil profile type: S_2 Site soil coefficient, S = 1.2

REMARKS

Underground structure designed for nuclear blast.

Building Designation : MAYNARD VOAB

Location: MAYNARD , MA

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1988 Year(s) remodelled: –
Date of Evaluation: 9-8-98
Area, (sq. ft.) 40,000 Length 272' Width 147' Photo Roll No. ____

CONSTRUCTION DATA

Rooframing: metal roof deck
Intermediate floor framing: ____
Ground floor: Conc Basement: None
Exterior walls: metal Openings: ____
Columns: steel Foundations: spread footing
General condition of structure: ____
Evidence of settling: ____

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	MB05	
Building period, T:		

Unreduced base shear,
$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$

____

Response Modification Coefficient, R: 5.5 ____

EVALUATION DATA

A_a = 0.10 A_v = 0.10

Site soil profile type: S2 Site soil coefficient, S = 1.2

REMARKS

Pre-engineered rigid frames

MERS GARAGE
MAYNARD, MA

EVALUATION STATEMENTS FOR BUILDING TYPE 5: STEEL LIGHT FRAME

These buildings are pre-engineered and pre-fabricated with transverse rigid frames. The roof and walls consist of light-weight panels. The frames are designed for maximum efficiency, often with tapered beam and column sections built up of light plates. The frames are built in segments and assembled in the field with bolted joints. Lateral loads in the transverse direction are resisted by the rigid frames, with loads distributed to them by shear elements. Loads in the longitudinal direction are resisted entirely by shear elements. The shear elements can be either the roof and wall sheathing panels, an independent system of tension-only rod bracing, or a combination of panels and bracing.

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.

Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1)

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F TORSION: The lateral force resisting elements form a well balanced system that is not subject to significant torsion. Significant torsion will be taken as any condition where the distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F DETERIORATION OF STEEL: There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical or lateral-force-resisting systems. (Sec. 3.5.3)

MOMENT FRAMES

T F STRESS CHECK: The building satisfies the Quick Check of the stress in the diagonals. (Sec. 6.1.1)

T F BEAM PENETRATIONS: All openings in frame-beam webs have a depth less that 1/4 of the beam depth and are located in the center half of the beams. (Sec. 4.2.3)

DIAPHRAGMS

T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

CONNECTIONS

T F STEEL COLUMNS: The columns in the lateral-force-resisting system are substantially anchored to the building foundation. (Sec. 8.4.1)

WALL AND ROOF PANELS

T F LIGHT-GAGE METAL, PLASTIC, OR CEMENTITIOUS ROOF PANELS: All light-gage metal, plastic, or cementitious roof panels are properly connected to the roof framing at not more than 12 inches on center. (Sec. 8.6.1)

T F WALL PANELS: All wall panels (metal, fiberglass, or cement asbestos) are properly connected to the framing. (Sec. 8.6.2)

ORIGINAL Design Loads

Wind = 90 mph

Snow = 35 psf

L.L. (office) = 80 psf.

Weight of Building.

Roofing (metal deck + insulation) = 3.5 psf

Purlin 1.0

Roof weight = 4.5 x 39600 = 178.2 kips

Snow = 35 x 39600 = 1386.0 kips.

Suspended Ceiling in office areas.

1 psf x 9000 = 9 kips per floor.

2nd fl slab (4" Conc)

48.3 psf x 9000 = 435 kips.

Interior walls 55 psf x 120' x 26.5' = 175 kips

Interior partitions

1st fl : CMU 55 x 11' x 630' = 381 kips

Metal Stud 4.5 x 11 x 185 = 92

2nd fl. : CMU 55 x 15 x 350 = 289 kips

MSP 4.5 x 8 x 510 = 18.4

2nd fl steel framing : 70.5 kips.

Exterior walls

NORTH

Col 1 thru 7	CMU	21.8	kips.
	Steel	24.0	
	Doors	23.0	
Col 7 thru 10	CMU	167	kips.

South

Col 1 thru 5	CMU	14.5	kips
	Steel	4.9	
	Doors	15.4	
Col 5 thru 8	CMU	10.9	kips
	Steel	7.5	
	Doors	11.5	
Col 8 thru 10	CMU	124.6	kips

EAST

Col A thru E	CMU	275	kips
Col E thru G	CMU	36.3	kips
	Steel	7.4	

WEST	CMU	109	kips
	Steel	27.9	

Pre-engineered steel frames: 143 kips

D.L.: Roof 178.2 kips

Interior Walls partitions ect. 1571

Exterior Walls 891.2

Frames 143

2783 kips

Snow : 1386 kips

L.L : 720 kips

W = DL + LL + SNOW $\cong$ 4900 kips

BASE SHEAR

$V = C_s W \times I.$

$C_s = \dfrac{0.80 A_v S}{R T^{2/3}} = \dfrac{0.80 \times 0.1 \times 1.2}{5.5 \ (2.5)^{2/3}} = \dfrac{0.096}{10.13} = 0.0095$

$T = 0.2 \ (29)^{3/4} = 2.5$ sec.

$V = 0.0095 \times 4900 \times 1.25 = 58.2$ kips

Assume all shear is carried by Masonry Walls

$f'_m = 3000$ psi

E_s : 29 000 ksi

E_m = 2250

n = 12.9

#4 @ 2'-8" o c

$\rho = 0.2 / 32 \times 8 = 0.00078$ o.k UBC 2106.1

$k = 0.132$

$j = 1 - \frac{0.132}{3} = 0.956$

$b = 8''$, $d = 48'$

$f_v = \frac{V}{bjd}$

About 60% of the base shear is carried by a shorter wall.

↑ BASED on tributary area

$V = 0.6 \times 58.2$

$f_v = \frac{0.6 \times 58.2}{8\,(0.956)(48 \times 12)} = 7.9$ psi

FEMA 310 3.5.3.3

$v = \frac{1}{m}\left(\frac{V}{A_w}\right)$

$= \frac{1}{3.0}\left(\frac{58.2^{K}}{8 \times 60 \times 12 \times 75\%}\right) = 4.5$ psi   o.k. for shear wall

CHECK SHEAR TRANSFER to the west direction

Roof	$3.5 \times (180 \times 120 + 60 \times 120)$ 28800	= 100.8 kips
snow	35×28800	= 1008 kips
Wall	N $(21.8 + 24 + 23)\frac{1}{2}$	= 34.4
	S $(14.5 + 4.9 + 15.4)\frac{1}{2}$	= 17.4
	$(19.9 \times 2)\frac{1}{2}$	= 19.9
		1180.5 kip

Total Cross section of Z's

$24 \times 0.706 = 16.94^{\square''}$

$f = \frac{1180.5}{16.94} = 69.7$ ksi > 36 ksi

Not adequate to transfer tension due to lateral movement in the E-W direct.

Maynard VSAB

OPTION 2 COST ESTIMATION FORM

COST ESTIMATION OPTION 2

1. GROUP MEAN COST
   - Group:
     - ☐ URM ☐ S1
     - ☐ W1, W2 ☒ S2, S3
     - ☐ PC1,RM1 ☐ S5
     - ☐ C1, C3 ☐ C2, PC2, RM2, S4
   - Cost Coefficient C_1 from Table 4.3.2. — C_1 = 7.23

2. AREA ADJUSTMENT FACTOR
   - Area
     - ☐ Less than 10K sq. ft. ☒ 10K - 50K sq. ft.
     - ☐ 50K - 100K sq. ft. ☐ 10K - 50K sq. ft.
   - Cost Adjustment Factor C_2 from Table 4.3.3 — C_2 = 1.12

3. SEISMICITY/PERFORMANCE OBJECTIVE FACTOR ADJUSTMENT
   - SEISMICITY
     - ☐ Low (NEHRP 1 or 2) ☒ Moderate (NEHRP 3 or 4)
     - ☐ High (NEHRP 5 or 6) ☐ Very High (NEHRP 7)
   - PERFORMANCE OBJECTIVE
     - ☐ Life Safety ☐ Damage Control ☒ Immediate Occupancy
   - Cost Adjustment Factor C_3 from Table 4.4.2 — C_3 = 1.4

4. LOCATION ADJUSTMENT FACTOR
   - City / State Maynard, MA
   - Cost Adjustment Factor C_L from Table 4.3.4 or Table 4.3.5 — C_L = 1.10

5. TIME ADJUSTMENT FACTOR
   - Year 1998
   - Inflation Rate 2 %
   - Cost Adjustment Factor C_T from Table 4.3.6 — C_T = 1.10

TYPICAL STRUCTURAL COST
($C = C_1 \times C_2 \times C_3 \times C_L \times C_T$) — C = 13.72

Building Area (Square Foot) : A = 40,000

Estimated Structural Cost (A x C) C_S = 548,800

Non-Structural Cost ($C_1 \times C_L \times C_T$)
\$3/ft² x 1.10 x 1.10 = 3.63 C_{NS} = 145,200

Finishing Cost \$1 /ft² C_F = 40,000

Total (Structural + Non-Struc + Finishing) C_{ST} = 734,000

Project Cost (C_{ST} x 0.3) C_P = 220,200

Total Cost ≈ 954,000

Attachment C: Building Inventory and Rehabilitation Cost Database

Agcy Co	Name of Building	St Cod	Cty Cod	Seismic	Area (sq	No of Bld	Exmpt	Occu Class	Essen Bldg	Hist Bldg	Yr of Con	Model Bldg Typ
5800	Boathouse	24	021	L	46	1	E1	80	Z2	H2	1960	MB13
5800	Bothell VSAB	53	061	H	2,787	1	E0	50	Z1	H2	1985	MB05
5800	Building 104	51	107	L	1,014	1	E0	40	Z2	H2	1955	MB16
5800	Building 105	51	107	L	936	1	E0	10	Z2	H2	1955	MB15
5800	Building 106	51	107	L	347	1	E1	40	Z2	H2	1955	MB16
5800	Building 110	51	107	L	1,292	1	E0	10	Z2	H2	1955	MB15
5800	Building 114	51	107	L	1,398	1	E0	10	Z2	H2	1955	MB15
5800	Building 123	51	107	L	22	1	E0	80	Z2	H2	1955	MB15
5800	Building 127	51	107	L	24	1	E0	60	Z2	H2	1955	MB16
5800	Building 140	51	107	L	75	1	E0	50	Z2	H2	1955	MB13
5800	Building 146	51	107	L	28	1	E1	40	Z2	H2	1955	MB15
5800	Building 201	51	107	L	691	1	E1	40	Z2	H2	1985	MB05
5800	Building 205/211/230	51	107	L	2,464	3	E0	30	Z2	H2	1955	MB15
5800	Building 217	51	107	L	821	1	E0	10	Z2	H2	1955	MB15
5800	Building 218	51	107	L	874	1	E0	80	Z2	H2	1986	MB13
5800	Building 219	51	107	L	348	1	E0	10	Z2	H2	1989	MB05
5800	Building 219A	51	107	L	678	1	E0	10	Z2	H2	1993	MB05
5800	Building 310	51	107	L	440	1	E0	60	Z2	H2	1955	MB15
5800	Building 311	51	107	L	33	1	E0	50	Z1	H2	1955	MB15
5800	Building 312/313	51	107	L	35	2	E1	40	Z2	H2	1955	MB15
5800	Building 315	51	107	L	344	1	E0	50	Z2	H2	1955	MB15
5800	Building 317	51	107	L	42	1	E1	40	Z2	H2	1955	MB15
5800	Building 320	51	107	L	346	1	E1	40	Z2	H2	1955	MB15
5800	Building 320A	51	107	L	302	1	E0	50	Z2	H2	1988	MB05
5800	Building 321	51	107	L	22	1	E1	40	Z2	H2	1995	MB14
5800	Building 327	51	107	L	190	1	E1	40	Z2	H2	1955	MB01
5800	Building 329	51	107	L	669	1	E0	40	Z2	H2	1955	MB05
5800	Building 331	51	107	L	161	1	E0	50	Z1	H2	1955	MB15
5800	Building 400	51	043	L	96	1	E0	10	Z2	H2	1955	MB15
5800	Building 401	51	043	L	65	1	E0	60	Z2	H2	1975	MB13
5800	Building 403	51	043	L	358	1	E0	10	Z2	H2	1955	MB15
5800	Building 404	51	049	L	11	1	E0	50	Z2	H2	1974	MB15
5800	Building 405	51	107	L	929	1	E0	10	Z2	H2	1900	MB01
5800	Building 406	51	107	L	394	1	E0	80	Z2	H2	1974	MB01
5800	Building 408	51	043	L	462	1	E0	50	Z2	H2	1955	MB05
5800	Building 409	51	107	L	779	1	E0	10	Z2	H2	1974	MB05
5800	Building 410	51	043	L	568	1	E0	50	Z2	H2	1900	MB01
5800	Building 411	51	107	L	819	1	E0	10	Z2	H2	1974	MB05

No of Stories	Exceptionally High Risk	Evaluation Procedure	Soil Type	Found Type	Eval Outcome	Struct Deficiency	Nonstruct Deficiency
N01							
N01	R1	P1	S2	FT1	OK	PS	PN
N04							
N02							
N07							
N02							
N02							
N02							
N01							
N02							
N01							
N01							
N02							
N02							
N01							
N01							
N03							
N01							
N01							
N01							
N01							
N01							
N01							
N01							
N01							
N02							
N01							
N01							
N01							
N01							
N01							
N01							
N04							
N01							
N01							
N01							
N02							
N01	R2	P1	S2	FT1	OK	PS	PN

Geo Site Hazard Defcy	Adj Defcy	Structural Cost	Nonstruct Cost	Finishing Cost	Project Cost	Source of Cost Est
PG	PA					
PG	PA	$42400	$0	$5000	$14200	C2

Comments
Boathouse
Pre-engineered steel frame with reinforced masonry walls.
Reinforced poured concrete
Poured concrete walls.
Control Tower (Heliport)
Security Gatehouse; Reinforced poured concrete and cinder block.
Sewage Treatment Plant
Motorpool
Fire Pumping Station
Maintenance Shop
Cinderblock construction
Maintenance Shop with mezzanine
Emergency Power
Guardhouse
Contains Health Unit
Electrical equipment - transformer
Covered walkway between buildings
Maintenance Shop
Maintenance Shop
Struct. passed marginally. Rehab cost is for improved performance.

Agcy Co	Name of Building	St Cod	Cty Cod	Seismic	Area (sq	No of Bld	Exmpt	Occu Class	E-sor Bldg	Hist Bldg	Yr of Con	Model Bldg Typ
5800	Building 413	51	107	L	1,104	1	E0	10	Z2	H2	1900	MB01
5800	Building 415	51	107	L	132	1	E1	50	Z2	H2	1955	MB15
5800	Building 417/425	51	107	L	57	2	E1	60	Z2	H2	1955	MB12
5800	Building 418	51	107	L	4	1	E1	60	Z2	H2	1955	MB15
5800	Building 420	51	107	L	703	1	E0	60	Z1	H2	1955	MB15
5800	Building 426	51	107	L	202	1	E1	40	Z2	H2	1955	MB13
5800	Building 429	51	107	L	1,468	1	E0	10	Z2	H2	1955	MB15
5800	Building 430	51	107	L	1,336	1	E0	10	Z2	H2	1955	MB15
5800	Building 430A	51	107	L	1,778	1	E5	10	Z2	H2	1990	MB13
5800	Building 431	51	107	L	1,517	1	E0	10	Z2	H2	1974	MB15
5800	Building 431A	51	107	L	90	1	E0	10	Z2	H2	1974	MB04
5800	Building 435	51	107	L	2,585	1	E0	60	Z2	H2	1955	MB15
5800	Building 444	51	107	L	3,826	1	E0	10	Z2	H2	1990	MB04
5800	Building 500	51	043	L	39	1	E0	80	Z2	H2	1960	MB15
5800	Building 501	51	043	L	5	1	E0	60	Z2	H2	1972	MB15
5800	Building 505	51	043	L	14	1	E1	80	Z2	H2	1992	MB01
5800	Building 604	51	043	L	5,626	1	E0	10	Z2	H2	1986	MB04
5800	Building 701	51	043	L	347	1	E1	40	Z2	H2	1955	MB16
5800	Building 702	51	043	L	1,014	1	E1	40	Z2	H2	1955	MB16
5800	Building 703	51	043	L	109	1	E1	40	Z2	H2	1955	MB01
5800	Building 704	51	043	L	1,848	1	E0	10	Z2	H2	1955	MB15
5800	Building 706	51	043	L	392	1	E0	80	Z2	H2	1990	MB15
5800	Building 707	51	043	L	749	1	E1	40	Z2	H2	1990	MB01
5800	Building 708	51	043	L	1,046	1	E0	10	Z2	H2	1955	MB15
5800	Building 709	51	043	L	86	1	E0	50	Z2	H2	1987	MB15
5800	Building 710	51	043	L	114	1	E1	80	Z2	H2	1989	MB15
5800	Building 712	51	043	L	1,778	1	E0	10	Z2	H2	1955	MB15
5800	Building 713	51	043	L	88	1	E1	40	Z2	H2	1992	MB08
5800	Building 713A	51	043	L	131	1	E1	40	Z2	H2	1993	MB01
5800	Building 718	51	043	L	25	1	E0	50	Z2	H2	1955	MB15
5800	Building 720	51	043	L	492	1	E0	50	Z2	H2	1955	MB08
5800	Building 721+	51	043	L	8,424	9	E0	30	Z2	H2	1955	MB15
5800	Building 752	51	043	L	24	1	E0	60	Z2	H2	1955	MB16
5800	Building 754	51	043	L	103	1	E3	80	Z2	H2	1985	MB01
5800	Building 781	51	043	L	24	1	E0	50	Z2	H2	1955	MB14
5800	Building 800	51	043	L	29	1	E0	50	Z2	H2	1955	MB14
5800	Building 810	51	043	L	77	1	E0	50	Z2	H2	1955	MB13
5800	Building 820/830	51	043	L	171	2	E0	50	Z2	H2	1955	MB13

No of Stories	Exceptionally High Risk	Evaluation Procedure	Soil Type	Found Type	Eval Outcome	Struct Deficiency	Nonstruct Deficiency
N04							
N01							
N02							
N01							
N01	R2	P1	S2	FT1	OK	PS	PN
N01							
N02							
N02							
N02							
N01	R2	P1	S2	FT1	OK	PS	PN
N01							
N02							
N02							
N02							
N01							
N01							
N02							
N07							
N04							
N01							
N02	R2	P1	S2	FT1	OK	PS	PN
N02							
N01							
N02							
N01							
N01							
N02							
N01							
N01							
N01							
N03							
N02							
N01							
N01							
N01							
N00							
N00							
N00							

Geo Site Hazard Defcy	Adj Defcy	Structural Cost	Nonstruct Cost	Finishing Cost	Project Cost	Source of Cost Est
PG	PA	$0	$0	$0	$0	
PG	PA	$55200	$45200	$22600	$36900	C3
PG	PA	$0	$0	$0	$0	

Comments
Maintenance Building
Guardhouses
Guardshack
Firestation
Struct. passed marginally. Recommended for rehabilitation.
Cafeteria - seats 250-300
Heliport
Communication
Picnic Shelter
Poured reinforced concrete walls
Poured reinforced concrete walls and roof
Firing Range
Polebarn
Generator
Trash Collection
Generator Building
Water Plant
Security Gatehouse; Reinforced poured concrete and cinder block
Picnic Shelter
Pumping Station - mostly underground
River Intake Station - underground
Generator Building - underground
Booster Pumping Station - underground

Agcy Co	Name of Building	St Cod	Cty Cod	Seismic	Area (sq	No of Bld	Exmpt	Occu Class	Essen Bldg	Hist Bldg	Yr of Con	Model Bldg Typ
5800	Building A	24	021	L	3,091	1	E0	30	Z2	H2	1965	MB10
5800	Building B	24	021	L	541	1	E0	80	Z2	H2	1956	MB15
5800	Building C	24	021	L	2,492	1	E0	30	Z2	H2	1956	MB10
5800	Building C-West	24	021	L	4,923	1	E7	30	Z2	H2	1995	MB14
5800	Building D	24	021	L	2,665	1	E0	30	Z2	H2	1924	MB15
5800	Building E	24	021	L	3,252	1	E0	10	Z2	H2	1923	MB15
5800	Building F	24	021	L	1,875	1	E0	30	Z2	H2	1926	MB15
5800	Building G	24	021	L	649	1	E0	30	Z2	H2	1948	MB15
5800	Building H	24	021	L	1,871	1	E0	10	Z2	H2	1923	MB15
5800	Building I	24	021	L	3,344	1	E7	50	Z2	H2	1996	MB07
5800	Building J	24	021	L	4,243	1	E0	23	Z2	H2	1965	MB10
5800	Building K	24	021	L	3,786	1	E0	23	Z2	H2	1890	MB15
5800	Building L	24	021	L	1,065	1	E0	30	Z2	H2	1959	MB10
5800	Building M	24	021	L	678	1	E0	23	Z2	H2	1960	MB14
5800	Building N	24	021	L	4,449	1	E0	10	Z2	H1	1870	MB15
5800	Building O	24	021	L	1,428	1	E0	80	Z2	H1	1839	MB15
5800	Building P	24	021	L	280	1	E0	80	Z2	H2	1960	MB16
5800	Building Q	24	021	L	948	1	E0	40	Z2	H1	1880	MB15
5800	Building R	24	021	L	459	1	E0	23	Z2	H2	1950	MB15
5800	Building S	24	021	L	626	1	E0	80	Z2	H2	1926	MB15
5800	Building T	24	021	L	110	1	E0	10	Z2	H2	1960	MB15
5800	Building U	24	021	L	156	10	E1	80	Z2	H2	1982	MB16
5800	Building V	24	021	L	90	1	E7	60	Z2	H2	1992	MB13
5800	Denton Federal Regional	48	121	L	5,110	1	E0	29	Z1	H2	1964	MB16
5800	Denton VSAB #2	48	121	L	1,858	1	E0	50	Z1	H2	1993	MB04
5800	Denton VSAB-Old	48	121	L	4,738	1	E0	10	Z1	H2	1985	MB04
5800	Fire Pump Station	24	021	L	372	1	E0	50	Z2	H2	1981	MB16
5800	Maynard Federal Region	25	017	M	7,432	1	E0	29	Z1	H2	1968	MB16
5800	Maynard VSAB	25	017	M	3,716	1	E0	50	Z1	H2	1988	MB05
5800	Morton Buildings	24	021	L	316	2	E1	40	Z2	H2	1980	MB02
5800	Olney Federal Support C	24	031	L	6,039	1	E0	29	Z1	H2	1970	MB16
5800	Olney Storage	24	031	L	139	2	E1	40	Z2	H2	1955	MB15
5800	Reception and Breakroo	48	121	L	285	1	E3	60	Z2	H2	1964	MB05
5800	Sewage Pumping Statio	24	021	L	15	1	E0	50	Z2	H2	1940	MB16
5800	Sewage Pumping Statio	24	021	L	15	1	E0	50	Z2	H2	1995	MB16
5800	Storage Building - East	48	121	L	223	1	E1	40	Z2	H2	1990	MB04
5800	Storage Building - West	48	121	L	223	1	E1	40	Z2	H2	1990	MB04

No of Stories	Exceptionally High Risk	Evaluation Procedure	Soil Type	Found Type	Eval Outcome	Struct Deficiency	Nonstruct Deficiency
N03							
N01							
N03							
N03							
N03	R2	P1	S2	FT1	OK	PS	PN
N03							
N03							
N02							
N03							
N02							
N02	R2	P1	S2	FT1	OK	FS	FN
N03							
N03							
N02							
N04							
N02	R2	P1	S2	FT1	NG	FS	FN
N01							
N02							
N01							
N01							
N01							
N01							
N01							
N00							
N02							
N02							
N00							
N00	R2	P1	S2	FT3	OK	PS	PN
N01	R1	P1	S2	FT1	NG	FS	FN
N01							
N00							
N01							
N01							
N00							
N00							
N01							
N01							

Geo Site Hazard Deficy	Adj Defcy	Structural Cost	Nonstruct Cost	Finishing Cost	Project Cost	Source of Cost Est
PG	PA	$0	$0	$0	$0	
PG	PA	$0	$0	$0	$0	
PG	PA	$463500	$790800	$646300	$570200	C3
PG	PA	$0	$0	$0	$0	
PG	PA	$548800	$145200	$40000	$220200	C3

Comments
Recreation Building
Eligible for historic registry but not registered
Auditorium seats approx. 500; Eligible for historic registry but not registered
Eligible for historic registry but not registered
Contains recreation area (swimming pool, basketball court, weight room)
Design looked at Map Area 1 in BOCA and NEHRP
Contains an auditorium and offices
Cafeteria seats about 350; eligible for historic registration but not registered
Chapel; historical building
Log Cabin; Holds 150-200 people for recreational purposes.
Eligible for historic registry but not registered; Planned renovations for compu
12x14 precast concrete buildings used as arson labs; Built from 1982-1996.
Security Station
Underground reinforced concrete structure.
Garage and Office
Garage and Office
Underground; Poured concrete
Underground reinforced concrete structure designed for nuclear blast.
Underground Reinforced Bunker; Code 29 for office/communications
Former firehouse being used for storage
Underground; Poured concrete
Underground; Poured concrete