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**STRUCTURAL EVALUATION
OF EXISTING PLUTONIUM BUILDINGS
AND AUXILIARY STRUCTURES AT
ROCKY FLATS PLANT**

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Prepared for
ROCKWELL INTERNATIONAL
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Golden, Colorado

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PREFACE

This report was prepared by Agbabian Associates (AA) under Sub-contract No. 69370-D for the Energy Systems Group of the Rockwell International Company at Rocky Flats Plant, Golden, Colorado. The objective of the contract is to evaluate existing plutonium handling and storage buildings at the DOE Rocky Flats Plant for their structural resistance to seismic, tornado, and extreme wind loadings and the impact of tornado-driven missiles. This report presents the summary results of the study for all the buildings included in the study and makes preliminary general recommendations for upgrading where needed. Detailed analyses and backup calculations performed for the several buildings are presented in separate reports.

Project engineer for Agbabian Associates was D.P. Reddy. Principal investigators were: R.R. Boppana (Bldgs. 771 and 715, Group I and Group VII); M. Kundu (Bldg. 774, Group I, Group V, and Group VIII); S.J. Hung (Group II and Group IX); Y.C. Lee (Group III and Group VI); R.W. Dowdy (Group IV); D.G. Yates (Testing); and E.M. Dombourian (Missile Impact). The analysis methodology was formulated by D.P. Reddy for seismic analysis, and R.W. Dowdy for tornado, wind and missile impact analysis. Other personnel who participated in the study were O. Babakhanian and P.S. Hashimoto in the analysis and evaluation of the buildings; J.H. Babcock and D. Rothman in floor responses; F.B. Safford and B. Barclay in testing; and M.B. Balachandra in the preparation of the reports. Project manager was R.W. Anderson.

Marvin Friedman was Contract Administrator for Rockwell International. Victor Terkun of Rockwell International was Technical Monitor for the study.

EXECUTIVE SUMMARY

Agbabian Associates conducted a study for the Energy Systems Group of Rockwell International on the structural resistance of plutonium handling buildings and auxiliary structures at the DOE Rocky Flats Plant, Golden, Colorado, when subjected to seismic, tornado, and extreme wind loadings and the impact of tornado-driven missiles. The structural evaluation is based on mathematical simulations supplemented by limited in-place testing to verify the analytical models. This report presents the summary results of the study and makes preliminary general recommendations for upgrading where needed.

The criteria loadings considered in the study are:

Seismic:	Peak horizontal acceleration of 0.14 g at bedrock level
Tornado:	Tangential wind speed of 200 mph
Nontornadic Wind:	Wind speed of 150 mph
Missiles:	Wooden plank, steel pipe, and automobile

All but nine buildings are safe under the criterion seismic load. The safe allowable bedrock horizontal accelerations for incipient failure for the deficient buildings are:

Building 707A:	0.04 g
Buildings 776/777, 707, 559, and North addition to Bldg 779:	0.06 g
Buildings 771, 774, 708, and 561:	0.10 g

When these levels are exceeded, the degree of anticipated structural damage will be enough to allow possible leakage of contaminants. Building 771 lacks flexural resistance against pressure exerted by the earth embankment on the south wall. The roof panels supporting the stack in Building 774 are overstressed. In the other buildings, the in-plane shear resistance of concrete block walls is inadequate, and connections are overstressed.

All buildings except 771 and 562 are vulnerable under tornado loading, the maximum allowable tornado level for incipient failure varying from 90 mph to 160 mph. Buildings 776/777, 776A, 559, 779, and 989 are the weaker structures under the tornado forces, limited to the 90 mph to 110 mph range. Structural elements contributing to failure include wall systems under out-of-plane flexure, roof systems under upward bending induced by uplift, beams and columns under combined axial and bending loads, and connections.

All buildings except 771, 715, 562, 729, and 875 are vulnerable under wind loading, the predicted damage being similar to that under tornado loading. In many cases, the predicted failure is caused by local intensification of wind pressure at roof corners and edges.

Most of the buildings are vulnerable to missile impact, with parts of the buildings subject to perforation by the plank and pipe missiles and to overall failure under the automobile missile. Vulnerable structural elements include concrete block walls, concrete wall and roof panels, metal deck roofs, metal and transite siding, and twin-tee wall and roof panels.

The following general structural upgrading recommendations are applicable to many buildings. Walls may be strengthened with reinforced gunite for in-plane shear under seismic loading and out-of-plane flexure under tornado and wind loading, thereby also increasing resistance to missile impact. Reinforced gunite may also be used to strengthen beams and columns. Strengthening of roof systems for uplift needs further study but candidate concepts are: tension ties or columns carried to the foundation, an extra topping slab with additional top reinforcing, and steel rods parabolically arched over the stems and supported on precast lightweight concrete struts. Connections may be reinforced through additional welding or the addition of steel plates, anchor bolts, or other inserts.

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SECTION 1

INTRODUCTION

1.1 OUTLINE OF STUDY BY AA

Agbabian Associates (AA) under contract to Rockwell International (R/I), Energy Systems Group, conducted a comprehensive study of existing plutonium-handling and storage buildings at the DOE Rocky Flats Plant to evaluate their structural resistance to seismic, tornado, and extreme wind loads, and tornado-driven missiles. The scope of work is described in Reference 1-1.

The DOE Rocky Flats Plant is located about 30 mi northwest of Denver, halfway between Golden and Boulder, Colorado. The primary function of the plant is the processing, manufacture, research, and recovery of plutonium.

The AA study comprised an analytical evaluation of a total of 24 buildings supplemented by dynamic in-place testing of three of these buildings to determine their dynamic characteristics. The 24 buildings considered in the study are divided into 9 groups based on their functions during operation of the plant. The 9 groups and the buildings included in each group are identified in Table 1-1. Buildings 707, 771, and 776/777 were subjected to impulse tests.

1.2 STUDY OBJECTIVES

The objective of this study is to evaluate the structural capabilities of existing plutonium handling and storage buildings at the Rocky Flats Plant for withstanding seismic, tornado, and extreme wind loads, and the impact of tornado-driven missiles, and thereby determine which structures will maintain their structural integrity and which will sustain damage from any of the postulated natural disasters.

The buildings listed in Table 1-1 include plutonium handling buildings and a limited number of auxiliary structures. For the plutonium buildings, a structural failure occurs if the buildings are unable to (1) shut down safely, (2) confine radioactive material, or (3) provide protection to personnel within the plant site or surrounding communities. For auxiliary structures, failure denotes a degree of building damage, which could result in the inability of plutonium buildings to function as described above. Plutonium buildings and auxiliary structures are identified in Table 1-2.

The buildings and auxiliary structures at the plant site were designed and constructed over a period of 25 years to various codes and standards. Many of the structures were designed for low seismic forces (UBC Seismic Zone 0 or 1 criteria) and for various wind induced pressures (from a low of 20 psf to a high of 100 psf, corresponding to wind speeds of 75 mph to over 120 mph). In addition, because of code modifications in the intervening years, even some buildings that were designed for a certain criterion load may not meet the current code requirements for the same load. Consequently, Rockwell International (R/I), Energy Systems Group, initiated the present study to evaluate the actual capabilities of the existing structures. In cases where the structures are found incapable of withstanding the criteria loads, the study recommends maximum safe allowable loads. In such cases, the study also indicates possible ways of upgrading the structure.

1.3 PRESENTATION OF RESULTS OF AA STUDY

Nine separate reports have been prepared on the analytical portion of the study, one for each group of buildings (Refs. 1-2 through 1-10). These reports address structural evaluation of the buildings under seismic, tornado, and non-tornadic wind loading.* Investigation of all buildings for their vulnerability to tornado-driven missiles is contained in Reference 1-11.

*Group VII, which comprises underground structures, was analyzed under seismic loading only (see Ref. 1-8).

A soil/structure interaction analysis conducted at the outset is reported in Reference 1-12. This report developed magnification factors by which the criterion earthquake ground motion at bedrock level was modified for use as input during the seismic analysis of each building.

Response spectra were generated for several buildings at locations identified by R/I. The spectra are expected to be used for future studies on critical equipment housed in the buildings. Results of this effort are presented in Reference 1-13.

Impulse tests performed on Buildings 707, 771, and 776/777 are described in Reference 1-14, which addresses details of the test setup, instrumentation, data reduction, and correlation of the analytical predictions with test data for the three buildings tested.

1.4 SCOPE AND ORGANIZATION OF THIS REPORT

An overview of the study along with the overall conclusions relative to the structural capabilities of the buildings is contained in this report. The buildings included in the study are briefly described in Section 2. The methodology followed during the analysis is briefly delineated in Section 3 and discussed in more detail in Sections 2 through 6 and Appendix A of the Group II report (Ref. 1-3). Section 4 summarizes the implications of the results of the above analyses relative to the overall structural capabilities of the buildings, and presents preliminary recommendations for upgrading where needed.

Tables and figures are grouped at the end of each section.

1.5 REFERENCES

- 1-1. Rockwell Int'l, Rocky Flats Plant (RI, RFP) *Scope of Work for Engineering Study of Existing Plutonium Buildings and Auxiliary Structures*. Golden, CO: RI, RFP, Sep 1977.
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- 1-3. Agbabian Assoc. (AA) *Structural Evaluation of Group II Buildings at Rocky Flats Plant*, R-7912-4946. El Segundo, CA: AA, Sep 1979.
- 1-4. Agbabian Assoc. (AA) *Structural Evaluation of Group III Buildings at Rocky Flats Plant*, R-7912-4947. El Segundo, CA: AA, Sep 1979.
- 1-5. Agbabian Assoc. (AA) *Structural Evaluation of Group IV Buildings at Rocky Flats Plant*, R-7912-4948. El Segundo, CA: AA, Sep 1979.
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- 1-7. Agbabian Assoc. (AA) *Structural Evaluation of Group VI Buildings at Rocky Flats Plant*, R-7912-4950. El Segundo, CA: AA, Sep 1979.
- 1-8. Agbabian Assoc. (AA) *Structural Evaluation of Group VII Buildings at Rocky Flats Plant*, R-7912-4951. El Segundo, CA: AA, Sep 1979.
- 1-9. Agbabian Assoc. (AA) *Structural Evaluation of Group VIII Buildings at Rocky Flats Plant*, R-7912-4952. El Segundo, CA: AA, Sep 1979.
- 1-10. Agbabian Assoc. (AA) *Structural Evaluation of Group IX Buildings at Rocky Flats Plant*, R-7912-4953. El Segundo, CA: AA, Sep 1979.
- 1-11. Agbabian Assoc. (AA) *Evaluation of Buildings at Rocky Flats Plant for Resistance to Impact of Tornado-Driven Missiles*, R-7912-5014. El Segundo, CA: AA, Jan 1980.
- 1-12. Agbabian Assoc. (AA) *Soil/Structure Interaction Analysis of Building 707 at Rocky Flats Plant*, R-7912-4779. El Segundo, CA: AA, Dec 1978 revised Jan 1979.
- 1-13. Agbabian Assoc. (AA) *Floor Response Spectra for Plutonium Buildings at Rocky Flats Plant*, R-7912-4956. El Segundo, CA: AA, Sep 1979.
- 1-14. Agbabian Assoc. (AA) *Impulse Test Results of Buildings 707, 776/777, and 771 at the Rocky Flats Plant*, R-7912-4954. El Segundo, CA: AA, Sep 1979.

TABLE 1-1. BUILDING GROUPS IN STUDY

Group	Building(s)
I	771, 774, and 715
II	776 and 777
III	707 and 708
IV	559, 561, and 562
V	779, 729, 782, and 727
VI	991, 985, and 989
VII	996, 997, 998, and 999
VIII	886 and 875
IX	331

TABLE 1-2. PLUTONIUM BUILDINGS AND AUXILIARY STRUCTURES

Group	Plutonium Building(s)	Auxiliary Structure(s)
I	771, 774	715
II	776/777	-
III	707	708
IV	559	561, 562
V	779	729, 782, 727
VI	991	985, 989
VII	996, 997, 998, 999	-
VIII	886	875
IX	-	331

SECTION 2

DESCRIPTION OF BUILDINGS

2.1 GENERAL

The 26 buildings evaluated in this study utilize a variety of construction materials and structural framing systems. Some of the buildings were constructed in separate increments over a period of several years and/or have been renovated as required to accommodate new programs. The buildings are briefly described below as to type of construction and are then compared with buildings of similar design.

2.2 BUILDING DESCRIPTIONS

2.2.1 GROUP I

Building 771--Plutonium Recovery

Original building is two-story poured-in-place reinforced concrete construction. Five additions to the original building include steel frame with metal wall panels and metal roof decking or concrete block walls with prestressed, precast twin-tee concrete roof panels.

Building 774--Process Waste Treatment

Structure is multiple level, modified by four additions. Construction varies from poured-in-place reinforced concrete walls and slabs to steel frame with prefabricated metal wall and roof panels.

Building 715--Emergency Generator House

Single-story building is constructed with concrete walls and precast cored concrete roof.

2.2.2 GROUP II

Building 776/777--Manufacturing and Assembly

Two-story combined building utilizes steel frame construction with transite exterior wall panels, steel pan second floor with concrete fill, and metal roof decking. Interior walls are concrete block.

Building 776A

Building construction is similar to Building 776/777.

2.2.3 GROUP III

Building 707--Fabrication-Assembly

Two-story building is constructed of precast concrete columns, beams and girders with precast, prestressed, concrete twin-tee panels for exterior walls, second floor and roof. Interior walls are concrete block.

Building 707A

Building construction is similar to Building 707.

Building 708--Compressor House

Single-story structure utilizes precast, prestressed concrete twin-tee panels for walls and roof.

2.2.4 GROUP IV

Building 559--Plutonium Analytical Laboratory

Single-story building is constructed with reinforced concrete frame, concrete block walls, and precast, prestressed twin-tee roof panels.

Building 561--Filter Plenum

Single-story building utilizes a reinforced concrete frame, precast concrete wall panels, and precast, prestressed twin-tee roof panels.

Building 562--Emergency Generator House

Structure is single-story with concrete block walls and precast cored concrete roof.

2.2.5 GROUP V

Building 779--Plutonium Development

Original building is two-story structure utilizing reinforced concrete frame, concrete block walls, with poured-in-place reinforced concrete second floor and roof; it includes a single-story extension of steel frame and metal deck construction. Single-story addition is constructed of precast reinforced concrete columns and beams with precast, prestressed twin-tee roof.

Building 729--Filter Plenum

Single-story building is constructed of reinforced concrete frame, concrete block walls and precast, prestressed concrete twin-tee roof.

Building 782--Filter Plenum

Single-story structure utilizes a reinforced concrete frame, with precast reinforced concrete wall panels and precast, prestressed twin-tee roof.

Building 727--Emergency Generator House

Structure is single-story with concrete block walls and reinforced concrete poured-in-place roof slab.

2.2.6 GROUP VI

Building 991--Production Control Warehouse

Building is single-story construction of poured-in-place reinforced concrete.

Building 985--Filter Plenum

Single-story building is constructed with a reinforced concrete frame, reinforced concrete precast wall panels and precast, prestressed twin-tee roof.

Building 989--Emergency Generator House

Single-story structure is constructed with concrete block walls and reinforced concrete poured-in-place roof slab.

2.2.7 GROUP VII

Buildings 996, 997, 998, 999--Radioactive Material Storage

Buildings utilize underground poured-in-place reinforced concrete construction for storage of radioactive material.

2.2.8 GROUP VII

Building 886--Nuclear Safety Facility

Single-story building is constructed with concrete block walls with steel roof joists and metal decking. It includes a test cell of poured-in-place reinforced concrete construction.

Building 875--Filter Plenum

Single-story building is constructed with reinforced concrete frame, precast reinforced concrete wall panels and precast, prestressed twin-tee roof construction.

2.2.9 GROUP IX

Building 331--Garage and Fire Station

This one- and two-story building is constructed in increments. Walls are either reinforced concrete or concrete block. Roof is composed of hollow core precast concrete panels, poured-in-place reinforced concrete, steel beams and decking, or precast, prestressed twin-tee panels.

2.3 SIMILARITIES IN CONSTRUCTION

It is evident from the building descriptions provided in Section 2.2 that the plant buildings have utilized a variety of materials and systems for construction. Building walls have been constructed of concrete block; precast concrete panels with concrete frame; precast, prestressed twin-tee panels;

poured-in-place concrete; or steel or transite siding supported by steel framing. Second floors have utilized precast concrete beams and girders with precast, prestressed twin-tee panels; poured-in-place reinforced concrete; or steel framing with metal decking and concrete fill. Roof systems have used the same systems as described above for second floors. It is also noted that in some instances several different materials have been incorporated in a building for the same construction system, e.g., concrete block walls and precast concrete walls, or other combinations of materials.

Some buildings are almost identical in construction and are grouped here for future reference when evaluating the maximum load levels assigned to these structures for seismic, tornado and extreme wind loadings.

<u>Category</u>	<u>Buildings</u>
A.	<u>Emergency Generator Buildings 715, 562, 727, and 989</u> Concrete block walls with precast or poured-in-place roof slab.
B.	<u>Filter Plenum Buildings 561, 985, 782, and 875</u> Concrete frame with precast concrete wall panels and precast, prestressed twin tee roof panels.
C.	<u>Plutonium Recovery Building 771 and Production Control Warehouse Building 991</u> Poured-in-place reinforced concrete construction.
D.	<u>Fabrication-Assembly Building 707 and 707A and Compressor Building 708</u> Precast, prestressed concrete twin-tee walls and roof.
E.	<u>Manufacturing and Assembly Buildings 776/777 and 776A</u> Steel frame construction with transite wall panels, steel pan with concrete-fill intermediate floor and metal roof decking.

- F. Plutonium Analytical Laboratory Building 559, and
Filter Plenum Building 729
Reinforced concrete poured-in-place frame, concrete
block walls, and precast, prestressed twin-tee roof.
- G. Radioactive Material Storage Buildings 996, 997, 998, and 999
Buried poured-in-place reinforced concrete construction.
- H. Remaining Buildings 774, 779, 886, and 331
These structures are one- or multi-story buildings that
incorporate a combination of structural systems and
construction materials not common to other buildings.
They have to be evaluated separately since maximum
allowable load levels are dictated by the specific com-
bination of materials associated with each building.

SECTION 3

DESCRIPTION OF AA STUDY

This section briefly discusses the models and techniques adopted during the analytical evaluation of the buildings at the Rocky Flats Plant. An outline of the test program is also included. Details of the analysis techniques may be found in Reference 3-1, whereas details of the test program and correlation of test data and analytical results are contained in Reference 3-2.

3.1 OVERVIEW OF ANALYTICAL STUDY

The basic approach followed in this study was to obtain the stresses in the structural members of a building under seismic, tornado, and wind loads by analyzing a mathematical model of the building and then use the stresses thus obtained to draw conclusions about the structural capabilities of the building. These analyses utilized the finite element method. The calculations were performed with the TRI/SAC finite element code, described in Section 3.1.1. Modeling considerations are discussed in Section 3.1.2. Seismic, tornado, and wind load analyses are discussed in Sections 3.2, 3.3, and 3.4, respectively. Analysis under missile impact is discussed in Section 3.5. Evaluation of the buildings using the analysis results is discussed in Section 3.6. The test program, which was carried out primarily to verify the significant characteristics of the analytical models, is described in Section 3.7.

3.1.1 FINITE ELEMENT CODE

The TRI/SAC code (Ref. 3-3) was used to compute the seismic response of all the structures in this study.

The TRI/SAC code is a large-capacity, digital computer program for one-, two-, or three-dimensional analysis of structural systems. The element library of the code includes trusses, beams, membranes, plates, shells,

axisymmetric solids, three-dimensional solids, and nonlinear springs and dashpots. More general types of elements may be obtained by combinations of these elements. Types of analyses that can be performed with this code include static response, dynamic response by normal mode method, dynamic response by direct integration method, and response spectrum method. Certain nonlinear dynamic problems can also be solved by direct integration. The output of the code includes a printout of natural frequencies, normal modes, and response time histories, as well as three-dimensional plots of the finite element model, and mode shapes and plots of response time histories.

3.1.2 FINITE ELEMENT MODEL

The finite element model or mesh used in the analysis of each building was made of plate, beam, and other types of elements as appropriate to simulate the structural members of the building. In some of the larger buildings, several physical members were lumped together and represented by a single element in order to minimize the size of the finite element meshes. Beam elements were used to simulate columns and roof and floor beams in the building, whereas plate elements simulated roof, floor, and wall panels. Equivalent properties were assigned to the model elements, taking into account openings and other structural details not explicitly modeled.

The finite element meshes did not include the soil medium surrounding the building. Soil-structure interaction under seismic excitation was accounted for by modifying the input used to drive the model, as described in Section 3.2.

The TRI/SAC code allows each node point of the finite element mesh to have a maximum of six degrees of freedom, three each in translation and rotation. In all analyses within the present study, the node points at the base of the model had their translational degrees of freedom constrained, while the rotational degrees were left free. Each unconstrained node point of the mesh was assigned a concentrated mass. The dead load of the structural members and the weight of equipment were considered in computing the concentrated masses at the node points. Where some structural members and portions

of buildings were omitted from the model, their mass was included and distributed to the appropriate node points. The negative pressure in the plutonium handling buildings was judged much smaller than other loads and was neglected.

Pounding between adjacent structures was not included in the seismic finite element analysis. However, the possibility of pounding was examined by comparing the computed displacements of adjacent structures with the gap provided between them. In those cases where the possibility of pounding was found to exist, the potential for structural damage was evaluated. The possibility of stresses being generated as a result of differential displacement of adjacent buildings connected by corridors or passageways was investigated in a similar manner.

3.1.3 DESIGN BASIS EARTHQUAKE (DBE)

The DBE is defined in Reference 3-4. The horizontal ground spectra for several damping ratios and the postulated horizontal acceleration time history (Ref. 3-5) are shown in Figures 3-1 and 3-2, respectively. The vertical component of the seismic shock is specified to be two-thirds the horizontal DBE levels and to act simultaneously with each of two orthogonal horizontal motions, applied separately. Figures 3-1 and 3-2 define the criterion motion at bedrock level and correspond to a postulated earthquake of Richter magnitude 6.0 at an epicentral distance of 16 mi. The peak horizontal acceleration at bedrock level is 0.14 g (Ref. 3-4).

3.1.4 DBE MODIFICATION THROUGH SOIL/STRUCTURE INTERACTION

In order to assess the importance of soil/structure interaction effects under seismic loads, a soil/structure interaction analysis of Building 707 was undertaken (Ref. 3-6). Based on the results of the analysis, it was agreed to use the following amplification factors in all seismic analyses:

- a. A magnification factor of 1.5 is to be used for horizontal and vertical motions for 25-ft soil deposits. Assume linear

variations of factors from 1.0 to 1.5 for soil deposits between 0 and 25 ft.

- b. For vertical motions, a constant magnification factor of 1.5 is to be used for soil deposits more than 25 ft deep.
- c. For horizontal motions, the magnification factor varies linearly from 1.5 to 2.0 for soil deposits from 25 ft to 40 ft deep.

3.2 SEISMIC ANALYSIS

Seismic response of the buildings at the Rocky Flats Plant was obtained from dynamic finite element analyses using both the time-history method and the response-spectrum method.

In the time-history analysis, the building response was obtained in the form of time histories of accelerations. The response of each building was computed when subjected simultaneously to two horizontal motions and a vertical motion. The input horizontal and vertical motions were obtained from the DBE by applying the magnification factors of Section 3.1.4 corresponding to the depth of the soil deposit under the given building. The response acceleration time histories were used to generate floor response spectra for use in the analysis of equipment in the building (Ref. 3-7).

The response spectrum analyses were used for obtaining maximum responses, which were used for the structural evaluation of the buildings. The total response of the building to seismic loading was obtained through three separate analyses corresponding to two horizontal and the vertical directions. The maximum response in each direction was computed as the square root of the sum of the squares (SRSS) of the individual modal responses. Finally, the total response of the building was computed as the SRSS of the individual response in each of the three directions. The above procedure is consistent with that being used in the analysis of similar structures

(Ref. 3-8) and is approved by the NRC (Ref. 3-9). The stresses induced by gravity loading were added to the stresses under seismic loading calculated above to obtain total stresses for the purpose of structural evaluation.

A damping value of 10% critical was used for all modes and all buildings considered in the study, in both time-history and response-spectrum analyses. This damping value is in accordance with that recommended by Newmark-Rosenblueth (Ref. 3-10). Reference 3-10 recommends a damping of 10% to 15% of critical for reinforced concrete buildings that are stressed beyond the yield point and for bolted steel frames stressed to the yield point. There are no recommendations for masonry structures. A damping of 10% is considered a reasonable and somewhat conservative value for the buildings in this study since they are combinations of steel, concrete, and masonry structures. All the structures at Rocky Flats when subjected to design basis earthquakes are assumed to stress beyond the yield point.

3.3 TORNADO ANALYSIS

3.3.1 TORNADO CRITERIA

Two criteria tornados were used during this study with tangential wind speeds of 300 mph and 200 mph. The characteristics of the 300 mph tornado are described in Reference 3-4 and those for the 200 mph tornado in Reference 3-11. For ease of reference in reports of this study, the 300 mph tornado is termed the Worst Case Tornado (WCT) and the 200 mph tornado the Design Basis Tornado (DBT).

3.3.2 TORNADO LOADS ON BUILDINGS

Static finite element analyses were used to determine the stresses occurring in the buildings as a result of the tornado loadings. Tornado loads used in the finite element analyses were obtained from the criteria specifications of tornado wind velocity and pressure drop.

The total forces applied at each node point of the finite element mesh were obtained by algebraically adding the contributions of the dynamic pressure, the atmospheric pressure drop, and gravity. The dynamic pressure for each tornado criterion was calculated from the specified curve of tangential velocity. An appropriate pressure coefficient was used to take into account the orientation of the given building surface relative to the tornado. In cases where a portion of the wall or roof was shielded from the tornado by an adjacent structure or other obstruction, internal pressure instead of external pressure was applied (see Section 3.4).

3.3.3 ANALYSIS UNDER TORNADO LOADS

The overall response of each building was obtained through finite element analysis. In order to determine the structure stresses under the most critical conditions, the tornado pressure profile was effectively stepped across the structure by consecutively placing the peak pressure of the tornado at each node point along the building wall. Several load cases were thus obtained, and a static finite element analysis was performed for each load case using the three-dimensional finite element model developed for the seismic analysis of that building. Since the buildings are essentially rectangular, the tornado pressure profile was stepped across two mutually orthogonal walls rather than all four walls in order to economize on computer time. The two walls that would yield the most critical loading on the building were selected for this purpose.

For each load case, the finite element mesh was loaded at each node point by concentrated loads consisting of the tornado pressure loads (computed as indicated in Section 3.3.2) and gravity loads. Identical load cases were considered for both the WCT and DBT, the two sets of analyses differing from each other only in the tornado pressure loads applied at the node points.

In addition to the overall building response, the local response of wall and roof panels was also analyzed. Hand calculations were made for this purpose with the wall or roof panel subjected to an equivalent uniform pressure.

3.4 WIND ANALYSIS

3.4.1 ANALYSES UNDER WIND LOADING

All buildings were checked for wind loads related to a nontornadic wind with a speed of 150 mph. Static analyses were performed using the finite element model constructed for seismic analysis. For each building, two wind load analyses were performed, with the wind acting on two mutually orthogonal walls, as in the case of tornado loading. In addition to the finite element calculations, which define the overall building response, hand calculations were made for panel load effects on wall and roof panels and for local effects around the wall corners, eaves, and ridges. The analysis follows the basic guidelines by the ANSI (American National Standards Institute) Code A58.1-1972 (Ref. 3-12). These guidelines are intended for use in design, and the results obtained in this study may be somewhat overconservative in some cases.

3.4.2 WIND LOADS UNDER CRITERION WIND

Pressure loading on the buildings due to the 150 mph wind were computed following the recommendations of ANSI (Ref. 3-12). Thus, the pressure P on the structure is defined by

$$P = q C_p - q_M C_{pi} \quad (3-1)$$

In Equation 3-1, q represents the dynamic pressure of the wind, and the coefficient C_p accounts for the orientation of the structure relative to the wind direction. The second term in Equation 3-1 represents the internal pressure q_i , which is caused by leakage of air either into or out of a building and is defined by $q_i = q_M C_{pi}$. The effective velocity pressure q_M is calculated from wind speed analogous to q . The internal pressure q_i can be either positive or negative depending on the sign of C_{pi} for the wall where leakage occurs. The value of the internal pressure coefficient C_{pi} depends on the distribution of openings in the building wall. A value of $C_{pi} = \pm 0.3$ was used in the present analysis based on the recommendations of Reference 3-12 for uniformly distributed openings with a total area ranging from 0 to 0.3 times the solid area of the wall. The sign of C_{pi} was selected to create the maximum building shear for the computer analysis.

Node point forces were calculated from the wind pressure loads by considering appropriate tributary areas. In cases where portions of walls were shielded from the wind by other structures, only internal pressure was applied. Total node point loads were obtained by adding wind pressure and gravity loads.

3.4.3 CALCULATION OF STRESSES IN STRUCTURAL MEMBERS

Static finite element analyses were performed with the node point forces calculated as above. The results define the overall building response. In addition, hand calculations were performed for panel loads and other local effects. Panel loads were investigated for roof and wall panels by assuming a uniform load calculated as discussed in Section 3.4.2. Local effects under eaves and ridges and around corners were analyzed using a local external pressure coefficient C_p . It may be noted that the values for C_p recommended for local effects by the current version of the ANSI code (Ref. 3-12) may be somewhat conservative. There are indications that some of these values may be revised downward in an upcoming version of the code.

3.5 MISSILE IMPACT ANALYSIS

3.5.1 MISSILE CRITERIA

Independent missile criteria are specified with each of the two criteria tornados used for the analysis of the buildings at the Rocky Flats Plant.

The missile criteria given in Reference 3-4 for the 300-mph tornado (WCT) consider three missiles:

1. Wooden plank (4 in. by 12 in. by 12 ft, 108 lb)

Horizontal velocity	200 mph
Vertical velocity	130 mph

2. Automobile (contact area 20 sq ft, 3000 lb)

Horizontal velocity only 50 mph

3. Steel pipe (3 in. dia. by 10 ft, 76 lb)

Horizontal velocity 100 mph

Vertical velocity 67 mph

For the 200-mph tornado (DBT), Reference 3-11 specified only two missiles, the wooden plank as above and the automobile with a velocity of 40 mph.

3.5.2 BUILDING FAILURE CRITERIA

The present study, the results of which are reported in Reference 3-13, recognizes two failure modes under the action of the criteria missiles:

(1) Perforation, i.e., the passage of the missile through a structural member along with the formation of a hole; and (2) overall failure of a wall panel or other structural member caused by missile impact.

The plutonium-handling buildings include one or more internal pressure zones where negative pressures are maintained with respect to the atmosphere. For these buildings, building failure under missile impact occurs when the building is rendered incapable of sustaining negative pressure with respect to the atmosphere. For the auxiliary buildings, failure occurs when any of the external wall or roof elements exhibits one of the two failure modes.

Perforation of plutonium buildings by the pipe or plank missile may not create a hole large enough to cause rapid loss of negative pressure. However, such a missile entering the building with significant velocity is a potential hazard to equipment and personnel in the building. Accordingly, the analysis here considers building failure under missile impact to occur when an external wall or roof is breached under either the perforation or the overall mode of failure. This definition applies to both the plutonium and the auxiliary buildings.

3.5.3 ANALYSIS METHODOLOGY

The techniques used for the analysis of missile impact are based on the recommendations in Reference 3-14.

3.5.3.1 Analysis for Perforation Failure

The modified National Defense Research Committee (NDRC) formulas are used for calculating the depth to which a target can be penetrated by a given missile. If the thickness of the target is less than the allowable perforation thickness in a given case, the missile passes through the target, i.e., a *perforation* failure occurs. In the present study, penetration depths are calculated for all the wall panels, roofs, etc., found in the several buildings to determine whether perforation occurs. Such calculations are done for the pipe and plank missiles traveling in both horizontal and vertical directions. The missiles are assumed to strike the target end on with velocity normal to the target surface. If perforation occurs, the resulting hole will only be slightly larger than the cross-section of the missile.

When perforation occurs in one or more structural element of a building under the impact of a given missile, an allowable velocity is calculated for that building under that particular missile. The allowable velocity represents the maximum missile velocity for which perforation does not occur in any structural member of the building. In addition to the allowable velocity for the building, the residual missile velocity when the missile strikes the building with the criterion velocity is calculated. The residual velocity is the velocity with which the given missile impacting the building with the criterion velocity would enter the building after perforating the weakest structural member of the building. Therefore, the residual velocity represents an upper bound to the velocity with which the given missile might actually enter the building.

3.5.3.1 Analysis for Overall Failure

Overall failure is defined to occur when the allowable ductility factor in a structural member is exceeded as a result of the impulse imparted to it by the missile impact. Calculations for overall failure are made for the automobile and plank missiles, which are assumed to strike the target with velocity normal to the target surface. These calculations are based on the theory outlined in Reference 3-14.

When one or more structural element of a building exhibits the overall failure mode under the impact of a given missile, an allowable velocity is calculated for that building under that particular missile by scaling down the criterion missile velocity. The weakest structural member is used in this calculation. The allowable velocity represents the maximum missile velocity for which no structural member of the building undergoes overall failure. If these velocities are exceeded, the wall or roof panel subjected to impact will undergo a deformation in excess of that corresponding to the maximum allowable ductility factor. It is not implied that the entire wall, roof, or building will collapse.

3.6 EVALUATION UNDER SEISMIC, TORNADO, AND WIND LOADS

3.6.1 EVALUATION METHODOLOGY

Two basic steps are involved in the evaluation methodology followed for the buildings at the Rocky Flats Plant. The first step is to calculate the margin of safety (M.S.) for each structural component from the allowable stress and the computed stress under seismic, tornado, and wind loads. A negative M.S. value indicates that the structural member (or the particular element of the finite element mesh) considered fails under the imposed load. The second step is to use the M.S. values for the several structural components of a building to draw conclusions on the overall performance of the building and determine if failure occurs according to the criteria laid down in Section 1.2. These two steps are discussed below in more detail.

If it is determined that a building cannot withstand the imposed criterion loading, safe allowable load levels are computed by scaling down the criteria load. The method of calculation of allowable load levels is addressed in Section 3.6.4. Section 3.6.5 summarizes guidelines following which preliminary recommendations are made for upgrading vulnerable buildings.

3.6.2 MARGIN OF SAFETY FOR STRUCTURAL COMPONENTS

Margin of Safety (M.S.) is defined by:

$$M.S. = \frac{\sigma_a}{\sigma_c} - 1 \quad (3-2)$$

where

σ_a = Allowable stress

σ_c = Computed stress

The word *stress* is used in a generalized sense here. As seen below, in some cases σ_a might represent ultimate moment or shear capacity, σ_c being the appropriate computed load. The determination of M.S. for the various structural components relevant to the present study is addressed below.

3.6.2.1 Shear Walls and Roof and Floor Slabs

Norris et al. (Ref 3-15) cite static test results (Ref. 3-16) that showed that the ultimate in-plane shear load of shear walls with a large ratio of length L to height H ($L/H > 2$) is approximately equal to the first cracking load. Since the walls in the buildings studied here have L/H ratios in excess of 2, the ultimate load is taken equal to the first cracking load. For calculating M.S., this implies $\sigma_a = 0.1 f'_c$.

Essentially the same procedure is followed for concrete block shear walls except that the compressive strength of the block (f'_m) is used instead of f'_c . In the absence of data on f'_m , its value had to be assumed based on generic data. ASTM standards (Ref. 3-17) specify a minimum f'_m of 1000 psi

on gross area of block. This converts to an f'_m of 1700 psi on the net area of block. On the other hand, UBC (Ref. 3-18) specifies $f'_m = 1350$ psi for hollow walls and 1500 psi for solid grouted walls, with the allowable working shear stresses further dependent on the type of inspection during construction. Since the actual compressive strength of the block used on the job and the inspection procedures followed are unknown, the ASTM specifications are followed to avoid excessive conservatism. Accordingly, an allowable shear stress of $\sigma_a = 170$ psi is used on the net section plus any filled cells for concrete block shear walls.

For calculating M.S. under in-plane shear for shear walls and roof and floor slabs, the applied in-plane shear load is obtained from finite-element calculations, which modeled shear walls and slabs as plate elements. The computed stress σ_c for use in Equation 3-2 is obtained from the in-plane shear force, accounting for openings in the wall. The M.S. is then calculated using Equation 3-2.

For calculating M.S. of shear walls and roof and floor slabs under flexure, the ultimate moment capacity is identified as σ_a and the applied moment as σ_c . Flexural effects in shear walls and slabs are not adequately simulated in the finite-element model, since plate elements are loaded by concentrated forces at the corner node points. Therefore the applied moments are obtained by hand calculations.

3.6.2.2 Columns and Beams

M.S. values for columns and beams are calculated separately under shear and normal stresses. Under shear stress, the M.S. is calculated from equation 3-2. Under normal stresses the interaction of biaxial bending and axial stresses is considered by constructing a three-dimensional failure envelope. Any vector from the origin to the failure envelope defines the allowable moment-load vector. M.S. is calculated by comparing the applied moment-load vector with the allowable moment-load vector having the same eccentricity.

3.6.2.3 Connections

M.S. values for connections are calculated using the allowable stress in tension or shear depending on the direction of load application and the configuration of the connection.

3.6.3 DAMAGE ASSESSMENT

Finite element analyses performed during this study assume linear elastic behavior of the buildings under seismic, tornado, and extreme wind loadings. However, an actual building does not respond to lateral forces in a purely linear elastic manner, especially when it is subjected to loadings that exceed the values initially used for design. When building elements are stressed to and beyond the yield point of the material, the stiffness of the structure is reduced due to cracking and yielding of the structural (and nonstructural) elements, and the damping of structure response is increased.

A reduction in stiffness or softening of the structure leads to a redistribution of loads as individual elements are deformed into the inelastic range. Since linear analysis does not permit variation in stiffness with load, redistribution of loads is not accounted for. Therefore engineering judgment needs to be exercised to estimate the extent of load transfer from overstressed to understressed regions.

Structural systems can absorb large amounts of energy when stressed beyond the elastic limit by dynamic loading. Since damping represents energy losses due to internal strains, increasing the damping value accounts for increased energy absorption as elements are strained beyond the damage threshold. In this study, a damping value of 10% was used for all buildings. This value is higher than the damping that would be associated with linear elastic response, but it was selected in order to account for the inelastic behavior that is anticipated at the earthquake level being used.

In assessing the structural integrity of the buildings based on the margins of safeties for individual elements, elements with large negative M.S. were identified and related to the specific areas where they occur in the actual structure. When several elements in the same general area of the structure have negative M.S.'s, it was assumed that structural failure* would occur. When adjacent elements had positive or low M.S.'s, engineering judgment was exercised to determine whether a redistribution of load in the area of the element with the high M.S. would minimize potential damage and still satisfy the permissible damage criterion and to establish a level of earthquake motion that the structure would survive.

Pounding between adjacent structures was not included in the seismic finite element analysis. However, the possibility of pounding was examined by comparing the computed displacements of adjacent structures with the gap provided between them. In those cases where the possibility of pounding was found to exist, the potential for structural damage was evaluated. Similarly, in evaluating corridors, duct bridges, and other structures connecting two buildings, the possibility that stresses might be caused by relative displacement between the buildings was investigated. In most cases, however, such passages are not rigidly connected to one or both buildings and relative displacement does not create any stresses.

3.6.4 CALCULATION OF ALLOWABLE LOAD LEVELS

When a building exhibits negative M.S. under one of the criterion loadings, the lowest of the M.S. values for the several structural components defines the M.S. for the entire building. Judgment is exercised in selecting the M.S. for the building in order to avoid picking an overconservative value

*Defined by the work statement as building damage that leads to the release or impedes the containment of radioactive material.

that corresponds to a local failure without serious consequences. The building M.S. is used to scale down the criterion loading level through the following relation:

$$W_a = \sqrt{(M + 1)} W_c \quad (3-3)$$

where

W_a = Allowable load level

W_c = Criterion load level

M = Building M.S.

W_a and W_c represent the peak horizontal acceleration at bedrock level in g's for seismic loading; ^{*} tangential velocity for tornado loading; and wind velocity for nontornadic wind. The criterion loads are therefore: $W_c = 0.14$ g for seismic loading, 200 mph for tornado loading, and 150 mph for wind loading. Note that the allowable tornado level is obtained by scaling down the DBT and not the WCT. The calculation of allowable velocities under missile impact is discussed in Reference 3-13.

3.6.5 RECOMMENDATIONS FOR STRUCTURAL UPGRADING

Preliminary recommendations are made for strengthening buildings based on the extent and location of the predicted damage. Practicality of the proposed recommendations is briefly addressed. Cost estimates and schedules for upgrading are not within the scope of this study.

^{*}For seismic loading, the square root sign does not apply in Equation 3-3.

3.7 IMPULSE TESTING AND DATA CORRELATION

3.7.1 TEST PURPOSE

The impulse tests employed in this project were performed to validate the accuracy of the analytical procedures that were used to determine the response of the structures to earthquake, wind, and tornado loads. Specifically, the function of the test program was to:

1. Use a nondestructive technique to dynamically excite selected buildings.
2. Instrument the buildings to record accelerations induced by the tests.
3. Record and process the data.
4. Analyze the data to derive mode shapes, natural frequencies, and damping.
5. Correlate the modal properties of the test data to the analytical results.
6. Determine the soil flexibility of the fundamental frequency of the building.

3.7.2 TEST CONFIGURATION, ACQUIRED DATA, AND DATA PROCESSING

Three buildings were selected (707, 771, and 776/777) to be experimentally evaluated. Pulse generating devices were placed in the interior of these structures to produce acceleration responses at various locations in the buildings. The input pulse trains consisted of a series of forces that excited building responses in the frequency range between 1 Hz and 40 Hz. Both vertical and horizontal forces were applied to the buildings.

The pulse generation device is shown in Figure 3-3. It consists of a weight on rails that houses a metal cutting device. When the weight is released from its attachment at the top of the test structure, the cutter engages a series of metal rings on a mandrel. By sequentially cutting through these rings, a series of force pulses was produced that was input to the buildings at the point of attachment of the test rig and the building. By careful design of the weight, its drop height, and the angle of inclination, in conjunction with the configuration of the metal cutter and the composition, spacing, and size of the metal rings on the mandrel, the desired pulse train was produced.

3.7.2.1 Test Data Acquisition

The data of interest to this study included the input force pulse and structure acceleration responses. The test apparatus was placed at one or two points on the second floor of each building so that the input force was applied at the intersection of the floor and vertical columns. Response accelerations were measured on the first and second floors as well as on the roofs. The level of input force was a series of pulses of about 24,000 lb each that produced motion throughout the buildings that measured in the range of 10^{-3} to 10^{-2} g's.

Data were captured on a portable signal processing unit that allowed CRT and hard copy display of the time-history signatures as well as their Fourier transforms and the frequency response functions between the force inputs and the acceleration outputs. A potential problem encountered at these low response levels was mechanical noise emanation from myriad pieces of machinery in these buildings. The project investigated the nature of this noise and developed procedures for minimizing it. Basic guidelines were developed so that data contaminated with an unusually large degree of noise would not influence the conclusions of the correlation quality between analytical and experimental data. These investigations are discussed in Reference 3-2. In general, when the signal-to-noise ratio degraded below

about a factor of five, those frequencies were ignored in the correlation process. Fortunately, noise was not a major problem at frequencies corresponding to the first several modes of structural response which govern the determination of structural distress.

One major reason that noise could be contained within manageable bounds was through a procedure called *averaging*. This is the process of performing repeated tests and performing simple averaging of the test outputs. In the averaging process, true structural response remains unchanged while noise is reduced because it tends to be randomly distributed in both amplitude and time and the averaging reduces its significance.

3.7.2.2 Preparation of Data

Analytical data discussed in previous sections of this report were prepared by normal mode methods. In such an analysis, the output from analytical computer codes can be presented as mode shapes and natural frequencies for a specified configuration and independently of damping. The effect of damping can be characterized by showing the frequency response functions between the force input and the acceleration responses at various damping ratios.

Impulse test data can be immediately presented as frequency response functions relating input and output. All of the physical attributes of the structures are manifested in these data. By using a modal extraction method, the experimental frequency response functions can be directly reduced to mode shapes, natural frequencies, and damping. Moreover, if impulsive loads are applied to the analytical models and calculated accelerations are computed at the drive points and response points in the test, the frequency response functions corresponding to the experimental data can be obtained.

3.7.3 CORRELATION OF DATA

For comparison, both analytical and experimental data must be presented in mutually compatible forms consisting of:

1. Mode shapes and natural frequencies, and
2. Frequency response functions relating input forces and output accelerations.

3.7.3.1 Damping

Computer simulations of the tests for the three buildings were performed for the purpose of determining the appropriate damping in the analytical models based on experimental data results. The models were run at damping values of 0, 1%, 5%, and 10% and the frequency response functions thus obtained were compared to experimental data. A typical comparison of these data is shown in Figure 3-4. Based on data of this type, estimates of damping in the models to best match test data are shown in Table 3-1.

3.7.3.1.1 *Conclusions*

In Building 707, the analytical damping that produced best correlation with the experimental results was somewhat lower (4% to 6% with an average of 5%) than observed experimentally (5% to 8% with an average of 7%). Buildings 771 and 776 displayed this same trend. In Building 771, the analytical damping ranged from 2% to 5% with an average of 3.5% while the experimental damping ranged from 4% to 6% with an average of 5%. In Building 776, the analytical damping averaged about 3% with variations between 1% to 6%. The experimental damping averaged about 7% based on data that varied between 5% to 9%.

The somewhat higher experimental damping values are thought to reflect the dynamic interaction of equipment with the parent building. The analytical model did not include this interaction effect. The result is

considered beneficial since analytical models using slightly smaller damping values will produce somewhat conservative results. As excitation levels increase in actual systems, it is thought that damping values also increase. It is postulated that the 10% damping used in the structural analyses is adequate and tends to the conservative side.

3.7.3.2 Mode Shapes and Natural Frequencies

Mode shapes and natural frequencies computed from the analytical models for the three buildings were compared to the mode shapes and frequencies extracted from the experimental data. A comparison of these data sets are shown in Figures 3-5 through 3-7 for various frequencies of response on each building. These data are summarized in Table 3-2, which presents an evaluation of the degree of correlation of the data.

3.7.3.2.1 *Conclusions*

These studies have demonstrated that the fundamental analytical mode shapes and frequencies correlate very well with the fundamental experimental data for Buildings 707, 771, and 776. At increasingly higher mode numbers, the correlation quality tends to degrade although in Building 707 satisfactory correlation exists at all observed frequencies. The disparity between the analytical and experimental data at higher frequencies is ascribed to the following:

1. Noise in the experimental records has been observed to be more pronounced at the higher frequencies so that the true structural response becomes more difficult to discriminate.
2. Interaction between the basic structures and their contained equipments is likely to be more influential at higher frequencies; analytical models do not consider this interaction.

3. Analytical models become less accurate at higher frequencies because the response depends more heavily on details of construction that are not incorporated in the models.
4. Closely coupled modes that are easily extracted from analytical models become difficult to adequately discriminate from test data; modes extracted from test data can be expected to envelope the response of very closely coupled responses.

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TABLE 3-1. DAMPING COMPARISONS--ANALYTICAL AND EXPERIMENTAL

Bldg.	Frequency Analytical, Hz	Frequency Experimental, Hz	Damping Analytical, %	Damping Experimental, %
707	9.2	9.7	5 to 6	7.0
	10.0	10.4	†	7.4
	13.3 to 13.9	13.1	4 to 6	8.5
	16.0	15.1	4 to 5	6.9
	16.2	16.5	4 to 5	4.8
771	9.2	9.5	2 to 5	6.1
	16.0	15.9	2 to 4	4.0
	16.5 to 16.9	17.2	3 to 5	5.1
776	3.4 to 3.6	3.5	1 to 3	9.0
	6.3 to 6.9	6.2	3 to 5	8.3
	8.5, 8.9, 9.1	8.9	3 to 5	6.8
	10.8, 10.9,	10.2	4 to 6	6.9
	12.0 to 13.1	12.8	1 to 2	5.4

†Unable to determine.

TABLE 3-2. ERROR AND DEGREE OF CORRELATION

Building	Frequency Analytic (f_m), Hz	Frequency Experimental (f_e), Hz	Error $\left \frac{f_e - f_m}{f_m} \right \times 100\%$	Qualitative Mode Shape Comparison	Final Degree of Correlation	Comments
707	9.2	9.5	3.3	Excellent	High	Higher modal participation analytically
	10.0	10.45	4.5	Excellent	High	
	13.3	13.13	1.3	Fair	Low to medium	
	13.9		5.5	Excellent	High	
	16.0		15.1	5.6	Fair to good	
	16.2	16.5	1.9	Good	High	
771	9.2	9.5	3.3	Good	High	See section 5.4.3, Ref. 3-2
	16.0	13.9	0.6	Fair	Medium	
	16.5	17.2	4.2	Good	High	
	16.9		1.8	Good		
776	3.4	3.4 to 3.6	0	Excellent	High	See section 5.4.4, Ref. 3-2
	3.6		5.6			
	6.3	6.2	1.6	Poor	Low	See section 5.4.4, Ref. 3-2
	6.9		10	Excellent	Medium to high	

TABLE 3-2. (CONCLUDED)

Building	Frequency Analytic (f_m), Hz	Frequency Experimental (f_e), Hz	Error $\left \frac{f_e - f_m}{f_m} \right \times 100\%$	Qualitative Mode Shape Comparison	Final Degree of Correlation	Comments
776 (con't)	8.5	8.94	5.3	Good to poor	Medium to Low	High modal participation
	8.9		0.4	Poor	Low	Low modal participation
	9.1		1.8	Poor	Low	Low modal participation See section 5.4.4
	10.8	10.24	5.2	Good	High	High modal participation
	10.9		6.1	Fair	Medium	
	12.02	12.8	6.5	Poor	Low	Lower modal participation
	13.10		2.2	Poor	Low	
	13.2		3.0	Poor	Low	
	13.5		5.2	Poor	Low

FIGURE 3-1. HORIZONTAL RESPONSE SPECTRA OF DBE (Ref. 3-4)

FIGURE 3-2. HORIZONTAL ACCELERATION TIME HISTORY FOR DBE LEVEL
(Ref. 3-5)

FIGURE 3-3. IMPULSE DEVICE SHOWING MANDREL AND CUTTERS

(a) 1% Damping

(b) 5% Damping

FIGURE 3-4. TYPICAL OVERLAY OF ANALYTICAL FREQUENCY RESPONSE FUNCTIONS WITH VARIOUS DAMPING FACTORS AND THE RESPECTIVE EXPERIMENTAL FREQUENCY RESPONSE FUNCTION

FIGURE 3-5. MODE SHAPE COMPARISONS OF BUILDING 70/ THIRD FLOOR HORIZONTAL EXPERIMENTAL AND ANALYTICAL DATA

AA10011

AA10010

FIGURE 3-6. MODE SHAPE COMPARISONS OF BUILDING 771 THIRD FLOOR HORIZONTAL EXPERIMENTAL AND ANALYTICAL DATA

AA10111

FIGURE 3-7. MODE SHAPE COMPARISONS OF BUILDING 776 THIRD FLOOR HORIZONTAL EXPERIMENTAL AND ANALYTICAL DATA

SECTION 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 INTRODUCTION

This section of the report summarizes the general conclusions of the study and reconciles these conclusions with the criteria used for the original designs. Allowable load levels are summarized for each building for the seismic, tornado, wind, and missile impact environments. Maximum load levels assigned to buildings of similar construction materials and structural systems are compared for consistency, and significant differences are identified and discussed. A brief description of vulnerable structural components and their predicted failure modes is provided. Recommendations for upgrading the buildings to meet the specified design-basis seismic, tornado, and wind environments are summarized as they relate to similar types of construction.

4.2 GENERAL OBSERVATIONS

The following observations are made concerning structural design limitations as they pertain to maximum allowable seismic, tornado and wind levels and how these limiting load levels relate to the criteria used for design of the structures.

4.2.1 BASIS FOR DESIGN OF EXISTING BUILDINGS

The existing buildings and auxiliary structures at the Rocky Flats plant have been designed and constructed over a period of 25 years. Various codes and standards were utilized for design depending on the regulations in force at the time. In general, the older codes were less stringent than those currently being used for design of new Rocky Flats structures. Due to lack of documentation, it has been difficult to identify the structural criteria actually used as the basis for design of the existing buildings. Rocky Flats personnel have reviewed available construction drawings and supporting documentation during preparation of the Plant SAR and were able to provide the

wind criteria shown in Table 4-1. The only mention of seismic criteria relates to the 3-story interim upgrading of Building 774, where the seismic design criteria was specified as Zone 1 in the 1967 UBC (Ref. 4-1). Available design criteria are reviewed in the following section and compared with current design specifications.

4.2.1.1 Seismic Design

Before discussing the results of the dynamic analyses that were performed to determine the response or behavior of the various buildings when subjected to the Design Basis Earthquake (DBE), it will be useful to explain what is meant by a dynamic analysis and why a dynamic analysis is appropriate.

A building structure, even a small multi-storied one, is a fairly complex, spatially distributed assemblage of columns, walls, and slabs. The vibration of such an assemblage by seismic waves passing through the ground supporting the building occurs in a manner dictated by the characteristics of the ground motion, specific ground-structure interaction, and the vibrational properties of the structure itself. The forces imposed on the building structural elements result from the mass of the building being accelerated in a transient, cyclic manner by the ground motion. The distortions of the building by such motion causes bending and shearing stresses to develop in each member. When this member stress exceeds elastic capability, failure in the member is considered to proceed. Damage usually occurs in one part of the structural system and progresses to greater areal extent as the vibratory loading continues. Sometimes this is catastrophic.

In a building structural system, where we are particularly interested in the ability of the system to resist vibratory loading and remain essentially intact elastically, it is almost mandatory to be able to analyze the structure's system response over the entire time history of ground motion. The capability to perform a truly dynamic analysis of this type has been developed only in recent years, and this is difficult, expensive, and complex. It depends upon the ability to accurately represent the ground motions of a strong earthquake in the epicentral area in time and space. It depends also

on the ability to accurately model or represent the physical structure in its true spatial distribution of mass, stiffness, and strength, and to exercise this model through the transient loading of the ground motions and determine its response.

Engineers now have large-capacity, high-speed electronic computers; matrix methods for numerical analysis; programming languages for addressing computer capacity and capability; computer programs, routines and subroutines for solution of complex vibration problems; and techniques of modeling the structural system, particularly in finite elements, in compatible ways that allow representative spatial distribution of mass and stiffness.

Aseismic design from early in this century up to the present time has largely been accomplished as a problem in statics. Although engineers have understood the dynamic nature of earthquake motion and the dynamic response of a building structure to that motion, the lack of knowledge of the true characteristics of ground motion and the enormous computational capacity necessary to analyze even simple structures dynamically made it necessary to treat the problem statically. The use of a seismic design coefficient to calculate the reactive lateral force upon a building structure was developed in Japan in 1915 and first applied generally in U.S. building codes in 1933. The seismic coefficient is a representation of ground motion acceleration as a percentage of gravity acceleration. The mass of the building multiplied by that coefficient represents the lateral force exerted upon the building. The value of this coefficient was determined from experience, analysis of structural damage sustained in prior earthquakes, and the general seismicity of the region. This value reflected no particular recognition of the particular vibratory characteristics of the structure being designed.

Essentially, most building codes still rely on the use of the statically applied seismic design coefficient. In addition, building codes have been intended to provide *minima* lateral seismic forces that buildings must resist without collapse under the conditions of the most probable severe

earthquake to which the structure would be subjected. Unfortunately, it is clear from the damage that occurred during the 1971 San Fernando earthquake and the 1964 Anchorage, Alaska earthquake that existing building codes could not be relied upon to provide a consistent margin of safety against collapse. Earthquake-resistant design under those code provisions is based on the concept that the structures will experience nonpermanent deflections from the more frequently occurring moderate earthquakes and will yield locally with permanent displacements, but without passing the threshold of collapse, even when subjected to the most severe probable earthquake.

However, the building codes of the time could not have been expected to produce a uniform structural performance or earthquake resistance in all buildings because of the many variables that influenced design, such as architectural plan, structural system, and engineering judgment. In addition, the quality of design and construction differed greatly from one building to another. Since the San Fernando earthquake, many building codes have been revised and are now requiring the use of considerably higher coefficient values for computing lateral forces, as well as requiring recognition of the importance of the critical use or occupancy of a building and consideration for seismic design and anchorage of nonstructural systems, i.e., mechanical and electrical systems, architectural systems, etc. The direction is also towards the requirement for an analysis of building response that considers the time variation of ground motion to validate maximum equivalent static design coefficients.

The Rocky Flats Plant is located in Seismic Zone 1 according to the Uniform Building Code's map of the United States that shows zones of approximately equal seismic probability. Zone 1 includes areas where minor earthquake damage is anticipated; the static lateral load design coefficient specified for this zone is minimum. The 1955 UBC required one-story buildings in Seismic Zone 1 to resist a minimum horizontal seismic force of 0.03 g. Until the late 1970's, this requirement remained essentially at this force level even though the UBC was revised several times during this time interval to include more factors in the formula used to calculate the lateral seismic

force coefficients. The current 1979 Edition of the UBC specifies a lateral force coefficient of 0.04 g for a one-story building in Zone 1.

The DBE specified for the study is defined as the maximum possible earthquake that could occur at the site. The peak horizontal bedrock acceleration related to this earthquake is 0.14 g. This value can not be compared directly to the lateral force coefficient given by the building codes for static design since the bedrock motions interacting with the structure result in horizontal forces that are dependent on the dynamic response characteristics of the structure.

Because of the low lateral seismic forces associated with Zone 1, wind loads were probably the controlling force used in the initial design of the existing plant structures. Results of the seismic evaluation of the buildings are discussed in Section 4.3.1.

4.2.1.2 Tornado Design

The facilities at Rocky Flats were not designed for tornado loads. Building codes do not impose requirements for design and construction of buildings to resist tornadic environments. Damage from tornadoes can be extensive and imposes a real threat to life safety. However, damage normally extends over a very small, localized area as compared to damage from earthquakes and severe windstorms. Requiring ordinary structures to be designed for tornadoes would add appreciably to construction costs and is probably not justified when considering the risk to life-safety based on historical occurrences of tornadoes and their location, path length and intensities.

However, in order to adequately protect public health and safety, nuclear power plants and other plant facilities where nuclear radiation imposes a hazard must be designed and constructed to tornado criteria established by NRC. Current research is being done to obtain realistic data on tornado flow fields and the magnitudes of tornado characteristics, the statistics of tornado occurrence, and the variability of wind and pressure fields in the vortex. Reference 4-2 presents the current practices used in determining tornadic loads for design and evaluation of structures.

The intensities of tornadoes have been classified by the Fujita F-scale based on the maximum wind speed associated with the tornado (Ref. 4-2). Table 4-2 shows the Fujita scale for tornado classification. It is seen that tornadoes have been formed and classified for wind speeds as low as 40 mph. It is interesting to note that the 73 mph windspeed associated with the F1 tornado is normally taken as the beginning hurricane windspeed.

4.2.1.3 Extreme Wind Loads

When considering wind loads acting on buildings it is important to understand the terminology used to describe windspeed and how wind loadings are determined. Windspeed can be described as peak wind, mean wind, fastest-mile wind, or annual extreme fastest-mile wind. These can be summarized as follows:

- Peak windspeed is generally a one- to two-second gust.
- Mean windspeed is commonly referred to as mean hourly or 10 minute average windspeed; wind gusts are fluctuations about the mean value.
- Fastest-mile wind is defined as the average speed of one mile of air passing an anemometer located at 30 ft above the ground; U.S. Codes and Standards use this reference windspeed.
- Annual extreme fastest-mile wind is the highest fastest-mile wind recorded at a weather station in a given year; this figure is used to determine probabilities of occurrence.

Windspeed at a given point fluctuates randomly about a mean windspeed at various frequencies. If the frequencies of the wind fluctuations match structural frequencies of vibration, resonance may occur with significant dynamic amplification of structure response. However, it has been found that the frequencies associated with building components are not likely to result in resonance amplification since the power/energy in the windspeed spectrum at these frequencies is extremely small. Therefore, wind loads are normally applied to structures as quasi-static loads rather than as dynamic loads. Further, a structure will not respond fully to the impact

of a gust which is only a small fraction of the size of the structure. A gust to be fully effective must have sufficient spatial extent to envelop both the structure itself and the flow patterns on the windward and leeward sides which are responsible for the maximum net load on the structure.

The wind flow interacting with a building produces an inward-acting pressure across the windward wall and outward-acting pressures across the two side walls, the leeward wall and the roof. This phenomenon relates to the Bernoulli effect caused by the air flow as it travels the longer distance around or over the structure. Locally, the smooth flow of air cannot negotiate the sharp corners at wall corners, eaves, ridges and roof corners. Thus, the air flow streamlines separate from these surfaces and relatively low pressures occur immediately downstream from these separation points. This causes locally severe, outward-acting pressures across these areas, resulting in the initiation failure as discussed later when evaluating the plant buildings for wind loading.

Design wind pressures are derived from considerations of wind speed, gust factors, and pressure coefficients for various portions of the structure. Starting with the 1964 UBC, wind loads are based on an "allowable resultant wind pressure" shown on a map that is used to create a table that provides wind pressures for various heights above ground. These values are then applied directly to surfaces of the building as horizontal pressures acting on vertical projections of the building or as uplift pressures on the roofs of enclosed buildings, with uplift pressure taken as $3/4$ the horizontal pressure. Wind loads specified in the 1955 UBC and earlier editions were based on a pressure of 15 psf acting on vertical projections of the building less than 60 ft above ground. Uplift pressures were not specified.

The American National Standards Institute's *Building Code Requirements for Minimum Design Loads in Buildings and Other Structures* (ANSI A58.1-1972, Ref. 4-3) utilizes a basic wind speed (fastest-mile winds) selected from

maps for several mean recurrence intervals. Effective velocity pressures are then computed for elements of an enclosed building and for parts and portions of the structure by considering type of exposure and height aboveground and gust factors. This procedure is discussed below and in Appendix A of the Group II report, and was used as the basis for evaluating the buildings for extreme wind loads. The purpose of reviewing it here is for comparison with the methods and loadings used for initial designs.

The effective velocity pressure p is given by

$$p = \frac{1}{2} \rho V^2 \quad (4-1)$$

where ρ = density of air and V = wind speed. For the standard atmosphere at sea level (29.912 in. Hg, 59 °F), the air density is 0.00238 slugs/ft³ (0.0765 lb/ft³), and Equation 4-1 becomes

$$p = 0.00256 V^2 \quad (4-2)$$

where p is in psf and V in mph. At the altitude of the Rocky Flats Plant, approximately 6000 ft, Equation 4-2 for the standard atmosphere would be

$$p = 0.00214 V^2 \quad (4-3)$$

On the other hand, subzero temperatures are common in the Rocky Flats region, and a correction for air density at a temperature of -10 °F yields the relation

$$p = 0.00237 V^2 \quad (4-4)$$

The effective velocity pressures in the ANSI code are based on Equation 4-2 and are used without modification in this study. Furthermore, allowable wind speeds are computed from wind load pressures by reversing Equation 4-2. If Equations 4-3 and 4-4 are used instead, the allowable wind speeds would be increased by 9% and 4%, respectively. In view of the prevailing uncertainties in the analysis, this refinement is of questionable merit and, accordingly, this study reports results based on the ANSI code recommendations, i.e., Equation 4-2.

For the Denver, Colorado area, the Basic Wind Speed maps in ANSI A58.1 specify a windspeed of 80 mph (annual extreme fastest-mile speed) for a 50-year mean recurrence interval and 90 mph for a 100-year interval. These velocities relate to wind pressures of 13 psf and 16 psf for the 50- and 100-year intervals, respectively, at 30 ft above ground, for suburban areas, city outskirts, and rolling terrain (ANSI Code 58.1, Table 5, Exposure B). For open, flat country, wind pressures of 21 psf and 27 psf are specified for the 50- and 100-year intervals, respectively (ANSI Code 58.1, Table 5, Exposure C). The design criteria wind loads shown in Table 4-1 exceed the ANSI wind pressures, indicating that the buildings for which wind criteria loads are known were designed for wind pressures that are higher than those based on building codes and standards normally used.

There is a widespread popular belief that the Rocky Flats area frequently experiences winds of over 100 mph. Measurements at the plant site at heights of 20 ft and 200 ft have shown 1-2 sec *gusts* in excess of 100 mph. However, the same measurements also showed that the 10-min average windspeed was always under 60 mph. This observation should be borne in mind in interpreting the structural evaluations in this study, since they are based on loads derived from average windspeeds.

4.2.1.4 Missile Impact

A tornado will generate a large amount of debris as it moves across the terrain. Debris will consist of building materials that have been torn from structures, materials stored in open areas, telephone poles that have been snapped off, and other similar objects, including vehicles.

When debris strikes a building, it may penetrate the wall or roof structure, either partially or completely, or it may severely crack or spall the member it impacts. Damage caused by missile impact may result in structural failure if the building damage leads to the release or impedes the containment of radioactive material.

4.2.2 BASIS OF DAMAGE ASSESSMENT

The member stresses and resulting margins of safety summarized in the individual group reports are based on finite element analyses that assume linear elastic behavior of the buildings under seismic, tornado, and extreme wind loadings. However, an actual building does not respond to lateral forces in a purely linear elastic manner, especially when it is subjected to loadings that exceed the values initially used for design. When building elements are stressed to and beyond the yield point of the material, the stiffness of the structure is reduced due to cracking and yielding of the structural (and non-structural) elements, and the damping of structure response is increased. A reduction in stiffness or softening of the structure leads to a redistribution of loads as individual elements are deformed into the inelastic range. Structural systems can absorb large amounts of energy when stressed beyond the elastic limit by dynamic earthquake loading. Since damping represents energy losses due to internal strains, increasing the damping value accounts for increased energy absorption as elements are strained beyond the damage threshold. In this study, a damping value of 10% was used for concrete, concrete block, and steel-frame construction. This value is higher than the damping that would be associated with linear elastic response, but it was selected in order to account for the inelastic behavior that is anticipated at the earthquake level being used.

In assessing the structural integrity of the buildings based on the margins of safeties for individual elements, elements with large negative M.S. were identified and related to the specific areas where they occur in the actual structure. When several elements in the same general area of the structure had negative M.S.'s, it was assumed that structural failure* would occur. When adjacent elements had positive or low M.S.'s, engineering judgment was exercised to determine whether a redistribution of load in the area of the element with the high M.S. would minimize potential damage and still satisfy the permissible damage criterion, or to establish a level of earthquake motion that the structure would survive. Some permanent damage/deformation was allowed for when determining the allowable stresses to be used in calculating seismic margins of safety, since the linear dynamic analysis used to evaluate the response of buildings does not account for yielding in any of the elements.

4.3 SUMMARY OF BUILDING EVALUATIONS

The purpose of this section is to summarize results of the analyses and evaluations that were made to establish maximum allowable load levels for seismic, tornado and extreme wind loadings. Table 4-3 indicates the maximum values assigned to each building. A more detailed description of vulnerable structural components, failure modes, and allowable load levels is provided in the following sections. It is seen from Table 4-3 that many of the buildings are not able to resist the design-basis loadings without experiencing unacceptable stress levels and deformations that can result in structural failure defined as building damage leading to the release or impeding the containment of radioactive material. For the Design Basis Earthquake (DBE), eight buildings require seismic strengthening.† Except for Buildings 771 and 562, all buildings are vulnerable to the 200-mph tornado environment, and the majority of buildings are deficient under the 150-mph extreme wind environment. General comments concerning the structural weaknesses of the buildings under the design-basis criteria are given below.

* Defined as building damage that leads to the release or impedes the containment of radioactive material.

† This excludes the northwest addition to Building 779, designated 779C in this study. This addition has very low resistance to lateral loads.

4.3.1 SEISMIC EVALUATION

This section summarizes the results of the seismic analyses and evaluations that were performed to assess the adequacy of the buildings to the DBE. Vulnerable structural components and their predicted modes of failure are identified by buildings. A reduced seismic design level is indicated for buildings that do not incorporate the required seismic resistance based on an overall evaluation of vulnerable members for which negative margins of safety have been calculated. Actual damage that has occurred in past earthquakes to buildings of similar design is discussed and related to the damage predicted for Rocky Flats structures.

4.3.1.1 Predicted Seismic Damage

Structural components that have been identified as vulnerable to the DBE are listed in Table 4-4 by building number. The mode of failure and maximum seismic level predicted for each vulnerable component are also shown in the table. An average maximum earthquake load level was assigned to each building, based on the type of vulnerable building elements, their margin of safety and predicted maximum load level, the number of vulnerable components involved, and the importance of the component to the overall integrity of the structure. Engineering judgment was exercised in making this final assessment considering possible redistribution of load to other members as components start to yield, ductility of the members and their connections, and damage that has occurred to similar construction materials and systems in actual earthquakes. From Table 4-4, it is seen that only eight buildings (excluding the northwest addition to Building 779) are not adequate for the DBE and require a reduction in the seismic load level from the 0.14 g peak bedrock acceleration DBE. The predicted failure of vulnerable components is discussed below:

Building 771

The south exterior building wall is designed to retain an earth embankment and is subjected to lateral soil pressures. The wall does not have adequate flexure resistance under the dynamic soil-structure interaction pressures generated during the DBE. In addition, the wall pilasters in the south wall do not have the capacity to resist the combined flexure and axial loading imposed by the building motions and resulting forces.

Building 774

The welded connections between the precast concrete twin-tee roof panels do not have adequate shear resistance to transfer in-plane diaphragm loads into the building shear walls. In addition, the precast concrete twin-tee roof panels that support the steel stack are overstressed in flexure due to the response of the stack during the earthquake and the reactions imposed on the roof by the stack support system.

Building 776/777

The major portion of the lateral seismic load acting on the structure is resisted by the interior concrete masonry walls, since these elements are much stiffer than the steel-braced frames that comprise the basic building structural system. The in-plane shear resistance of these block walls is not adequate for the seismic loads imposed by the DBE. The maximum load that the walls can resist in shear corresponds to an earthquake of 0.06 g peak bedrock acceleration.

Building 776A

This structure is limited by the metal roof decking and its resistance to flexure under acceleration forces induced by the building motions. The margin of safety calculated for this element was -0.3, limiting the building to an earthquake bedrock motion of 0.10 g. However, based on the predicted failure mode and on the ductility characteristics of the material, it is judged that the structure should be able to withstand the DBE without failure.

Building 707

The lateral load resistance of the building is severely limited by the shear resistance of the welded connections between precast concrete twin-tee panel flanges in the roof, second floor and wall diaphragms, as well as the connections between roof panels and concrete block walls and the connections between precast concrete twin-tee wall panels and the precast floor and roof panels. The in-plane shear resistance of the second floor and roof diaphragms limits the seismic loading that can be transferred to the transverse shear walls. Likewise, the shear resistance of the connections between the precast twin-tee wall panels limits the diaphragm reactions that the walls can carry

down to the foundation. In addition, the capacity of the second-story concrete columns under combined flexure and axial loading is not adequate for the motions and loads imposed by the DBE. The maximum seismic load level assigned to Building 707 relates to a peak bedrock acceleration of 0.06 g.

Building 707A

The construction of this building is similar to Building 707, and its lateral-load resistance to earthquake forces is limited by the second floor, roof, and wall welded flange connections between precast concrete twin-tee panels. A peak bedrock acceleration of 0.04 g was assigned to this structure.

Building 708

The shear resistance of connections at the bottom of the precast concrete twin-tee wall panels limits the lateral-load resistance of this building to 0.10 g.

Building 559

The exterior concrete block walls are not adequate for the DBE as determined by in-plane diaphragm shear resistance and by out-of-plane flexure resistance of the intermediate wall bond beam under lateral inertial forces. As determined for other buildings using the precast concrete twin-tee roof panel construction, connections between flanges and between panel flange and roof edge beam do not have adequate shear resistance to transfer in-plane diaphragm loads to the transverse shear walls. It is judged that the building has an overall allowable seismic level corresponding to an 0.06 g peak bedrock acceleration.

Building 561

The shear resistance of the welded connection between stems of the precast concrete twin-tee panels and the roof edge beam limits the lateral-load resistance of the building to forces associated with a peak bedrock acceleration of 0.10 g.

4.3.1.2 Typical Earthquake Damage

Past earthquakes have demonstrated the structural weaknesses in building systems that are subjected to strong vibratory ground motion. For a specific earthquake, damage to buildings can vary from severe, including building collapse, to minor, depending on many factors. These factors include the frequency content and duration of the ground motion, the type of building configuration and distribution of mass and stiffness, the size of the building, its dynamic characteristics in responding to the earthquake, type of construction, design deficiencies, type of connections and details, and quality of construction.

An earthquake is remarkably effective in sorting out structural weaknesses. The structure must be able to transmit the seismic forces, including all their effects such as shear, torsion and overturning, down through the structure to the foundation. If there is a single zone of weakness in the path, failure of the structure can result unless load can be redistributed to other members or systems that are able to accommodate the additional stresses. Inadequate connections are the most common structural weaknesses in a structure as demonstrated in several major earthquakes, for example, the Anchorage, Alaska earthquake of 1964 and the San Fernando, California earthquake of 1971 (Refs. 4-4 and 4-5). There were numerous examples of connections between precast concrete elements failing, either in the anchorage mechanism (bolts, rebar, etc.) or in the welds. Connections are critical in transferring lateral, in-plane forces in roof and floor diaphragms to transverse shear walls. In the case of precast concrete floor or roof construction, connections between individual elements must be capable of transferring diaphragm shear and tension stresses to the transverse shear-resisting structural systems. Failure of precast diaphragms due to inadequate connections between elements was evident in these earthquakes. As discussed in the previous section, inadequate shear resistance of connections was one of the primary weaknesses that limited the lateral-load resistance of the buildings to seismic forces.

Another important structural system in resisting the seismic forces imposed on a building is the vertical, lateral-load resisting components that carry the roof and floor diaphragm reactions down to the foundation. During earthquake-generated ground motions, the most rigid elements in the structure will receive most of the lateral load. If these rigid elements are incapable of resisting the lateral loading, they will fail and, unless there is another path available for carrying the lateral forces to the foundation, the building will be severely damaged. Most of the buildings at Rocky Flats utilize shear walls as the primary lateral-load resisting element. They are constructed of a variety of materials including concrete block, poured-in-place concrete, and/or precast concrete. Major damage to similar shear wall construction has occurred in past earthquakes.

4.3.1.3 Summary

Except for the Group I Buildings, the maximum allowable load levels predicted for structures vulnerable to earthquakes are governed by the in-plane shear resistance of concrete block walls, by the shear resistance of connections between precast concrete twin-tee panels, and by the shear resistance of connections between transverse shear walls and the floor and roof diaphragms. It is also noted that the maximum lateral loads that the buildings can resist are limited to a peak bedrock acceleration of approximately 0.06 g (except Building 707A, which is limited to 0.04 g). It appears that this load level is probably somewhat higher than the lateral forces that building codes impose on Seismic Zone 1.

Group I buildings have unique design requirements that limit these buildings to a peak bedrock acceleration of 0.10 g. Building 771 is subjected to lateral soil pressures during the earthquake that overstress the south building wall retaining the earth embankment. The steel stack on the roof of Building 774 imposes seismic reactions on the precast concrete roof panels that overstress the members in flexure.

A review of the earlier report by John Blume & Associates (Ref. 4-6) shows that the strongest earthquake experienced by the Rocky Flats Plant is that of August 9, 1967 with Richter magnitude 5.3 and epicentral distance 26.7 mi. A rough calculation shows that the peak acceleration of this earthquake at bedrock level would be under 0.03 g. The weakest of the buildings evaluated by AA is predicted to withstand a horizontal bedrock acceleration of 0.04 g. Therefore, historic seismic data provide no guidance with regard to the actual seismic resistance of the buildings.

4.3.2 TORNADO EVALUATION

This section summarizes the results of the tornado analyses and evaluations that were performed to assess the adequacy of the buildings to the design-basis tornado (DBT). Vulnerable structural components and their predicted modes of failure are identified by buildings. A reduced tornado design level is indicated for buildings that do not incorporate the required resistance for the DBT based on an overall evaluation of vulnerable members for which negative margins of safety have been calculated. Typical damage that has occurred in actual tornadoes to buildings of similar design is discussed and related to the damage predicted for Rocky Flats structures.

4.3.2.1 Predicted Tornado Damage By Type Of Construction

Structural components that have been identified as vulnerable to the 200 mph DBT are listed in Table 4-5 by building number. The mode of failure and maximum tornado level predicted for each vulnerable component are also shown in the table. An average maximum tornado load level was assigned to each building, based on the type of vulnerable building elements, their margin of safety and predicted maximum load level, the number of vulnerable components involved, and the importance of the component to the overall integrity of the structure. Engineering judgment was exercised in making this final assessment

considering possible redistribution of load to other members as components start to yield, ductility of the members and their connections, and damage that has occurred to similar construction materials and systems in actual tornadoes. The overall maximum tornado level assigned to each building is below the 200 mph DBT in all cases except for Buildings 771 and 562. The predicted failure of vulnerable components is discussed below by type of construction, rather than on a building-by-building basis, since the failure modes identified for a construction material in one building are fairly representative of failure modes in other structures with similar construction. It should be noted that variations in the maximum tornado level identified for a particular system may be different from building to building. This is normally the result of different span lengths that affect ultimate capacities, differences in member dimensions or section modulus, or changes in detailing of connections.

Wall Systems

Wall systems are vulnerable to the tornado loadings that apply normal pressures to the wall surfaces, resulting in out-of-plane bending moments and shears. Connections that resist wall reactions from positive and negative pressures must be capable of transmitting these reactions to the supporting building structure. In most cases, walls were found to be inadequate to resist the 200 mph DBT, and lower tornado levels had to be established based on the flexure capacity of the wall elements. Wall systems that were identified as deficient under normal pressures imposed by the DBT criteria included concrete block, precast concrete panels, precast concrete twin-tee panels, poured-in-place concrete, and structural steel framing with transite or sheet metal siding. See Table 4-5 for specific buildings.

Roof Systems

Roof systems are especially vulnerable to tornado negative or uplift pressures. Roof systems of steel beam and girder construction with metal roof decking normally provide the same load capacity in either the downward or upward direction because of equal section modulus (disregarding compression flange limitations, etc.). However, concrete roof systems are usually reinforced for tension stresses induced by dead load and by live loads that act in the downward direction. This pattern of reinforcement does not provide the same flexure resistance in the upward direction and members are not able to provide as much capacity under tornado uplift pressures. This is especially critical for the prestressed, precast concrete single- and twin-tee panels that are used for many plant roof systems and walls. These precast concrete members are prestressed for gravity and vertical roof loads acting downward and are very inefficient in resisting net upward loads. It should also be noted that connections anchoring roof systems to building walls and columns must also be able to resist net upward reactions at supports.

In general, the negative margins of safety associated with uplift forces acting on roof systems that utilize steel joists, girders, and metal decking are larger than the negative M.S. values determined for concrete roof systems. This is caused by the heavy dead loads associated with concrete construction that counteracts the uplift forces. A review of Table 4-5 will show that almost all building roof systems exposed to the tornado forces are vulnerable to uplift forces.

When exterior building walls are designed to span vertically, lateral wall loadings are resisted at the roof and intermediate floor levels. These wall reactions impose loads that must be distributed through the supporting diaphragms to vertical structural systems, e.g., shear walls, braced frames, etc., that are capable of carrying the lateral in-plane diaphragm reactions down to the foundation. In the case of the precast concrete single- and twin-tee panels, the welded insert connections between panel flanges and building walls must be adequate for transferring diaphragm shear and tension stresses into

the shear walls. It was found that in many cases these connections were not able to transmit the diaphragm stresses and reactions imposed by the 200 mph DBT to the transverse shear walls, and a reduced maximum tornado level had to be established. As noted in Table 4-5, buildings where the vulnerability of diaphragm connections limited the lateral loading that is transmitted to shear wall include Buildings 774, 707, 559, 561, 985 and 331.

Structural Framing Systems

It was found that the beam, girder and column elements that comprise the structural framing systems for some buildings were vulnerable to the combined axial and flexure stresses induced by the tornado forces. Buildings in which this occurred include 776/777, 707, 707A, and 991.

4.3.2.2 Typical Tornado Damage

The damage predictions for plant buildings subjected to the 200 mph DBT can be compared to descriptions of actual tornado damage as documented in the literature. For example, Reference 4-7 reports on damage to engineered structures as the result of a series of tornadoes that occurred during the 3rd and 4th of April 1974, covering eleven states throughout the central part of the United States. Estimates of maximum ground level windspeeds in the tornadoes that appeared to be the most intense were in the range of 200 mph to 250 mph. Damage to many engineered structures, such as school buildings and commercial buildings, that resulted from these tornadoes is briefly discussed below as it relates to types of building construction found at Rocky Flats.

Wall Systems

There was extensive damage and/or collapse of concrete block masonry walls, including load-bearing walls as well as filler walls. There was also damage to precast concrete wall panels.

Roof Systems

Damage to roof systems was the primary mode of building failure. Types of roofing systems that collapsed and/or blew away under uplift forces included precast roof panels, precast concrete twin-tee roof panels, open web steel joists, bulb tees and concrete roofing, reinforced concrete panel joist slabs, and other similar types of construction. Damage to roofing was especially severe at building and roof corners and at roof ridges.

Structural Framing Systems

Structural steel and reinforced concrete framing for roof and walls survived the tornadic winds fairly well even though the buildings may have lost roof and wall panels. These systems included steel rigid frames, steel roof trusses, and reinforced concrete and steel braced framing systems.

It is noted that the damage experienced by types of construction found at Rocky Flats is similar to the damage predicted in this study. It is obvious that damage to roof systems under the 200 mph DBT uplift pressures will be extensive. As noted in Table 4-5, every building exposed to the tornadic winds, except Buildings 771 and 562, will be damaged and probably result in failure of the structure, defined by damage that leads to the release or impedes the containment of radioactive material. Since the structures were not designed to resist tornado forces, extensive structural strengthening is required if buildings must be resistant to tornadoes.

4.3.2.3 Summary

As noted previously, except for Building 771 and 562, the overall maximum tornado level assigned to each building is below the 200 mph DBT. The maximum tornado velocities assigned to these buildings varied from 100 to 160 mph. Buildings 776/777, 776A, 559, 779, and 989 appear to be the weaker structures under the tornado forces, limited to the 90 to 110 mph range. According to the Fujita F-scale of tornado damage, this would correspond to the F1 Moderate Damage classification. All other buildings fall in the F2 Considerable Damage classification (velocities between 113 to 157 mph), except for Buildings 771 and 562 that are adequate for the 200 mph DBT.

4.3.3 WIND EVALUATION

This section summarizes the results of the wind analyses and evaluations that were performed to assess the adequacy of the buildings to the 150-mph design basis wind (DBW). Vulnerable structural components and their predicted modes of failure are identified by building. A reduced wind design level is indicated for buildings that do not possess the required wind resistance based on an overall evaluation of vulnerable members for which negative margins of safety have been calculated.

4.3.3.1 Predicted Wind Damage by Type of Construction

Structural components that have been identified as vulnerable to the DBW are listed in Table 4-6 by building number. The mode of failure and maximum wind level predicted for each vulnerable component are also shown in the table. Based on the type of building element, its margin of safety and predicted maximum load level, the number of vulnerable components involved, and the importance of the component to the overall integrity of the structure, an average maximum wind load level was assigned to each building. Engineering judgment was exercised in making this final assessment considering possible redistribution of load to other members as components start to yield, ductility of the members and their connections, and damage that has occurred to similar construction materials and systems when subjected to extreme wind forces. From Table 4-6, it is seen that the overall maximum wind level assigned to the majority of buildings is below the 150 mph extreme wind criteria. The predicted failure of vulnerable components is discussed below by type of construction since the failure modes identified for a construction material in one building are fairly representative of their use in other structures.

Wall Systems

Failure of many concrete block walls were predicted under wind loading that applies normal pressures to the wall surface, resulting in out-of-plane bending moments and shears that exceeded the capacity of the sections. All walls have sufficient in-plane shear resistance for lateral loading.

Roof Systems

Failure of roof panels due to uplift pressures acting near roof corners and edges was predicted for many buildings. This occurs for both steel and precast concrete twin-tee panel construction and requires strengthening these elements for increased flexure resistance under uplift pressures.

Failure of connections between precast roof panels and between panels and building walls was predicted for some buildings.

Individual Elements

The combined axial and flexure resistance of some building columns is not adequate for the loads imposed on the building by the DBW and failure of these elements is predicted.

4.3.3.2 Summary

In general, the predicted damage under the 150 mph wind is similar to that predicted for the 200-mph tornado except the extent of damage is not as great and not as many structures require strengthening.

The local wind pressures acting at roof corners and edges and the area over which they act are very severe as specified by the ANSI code. There are indications that the width of the edge strips for local effects may be reduced significantly in the new ANSI code which is currently in preparation. This may eliminate the negative margins of safety for the roof under the wind loading. It should be noted that the maximum allowable load levels for wind velocities are normally less than the load levels based on tornadic wind velocities. In general, this can be related to the high local pressure coefficients specified in ANSI for roof corners and edges and the area over which they are assumed to act as discussed above.

4.3.4 MISSILE IMPACT EVALUATION

The results of the missile impact analysis for all buildings are contained in Reference 4-8 and are repeated in Tables 4-7 through 4-9. These tables show the safe allowable velocities for the buildings for each criterion missile and are obtained from the weakest structural member of the buildings.

For the timber plank missile, both perforation and overall failure modes were investigated. In all the buildings, however, the allowable missile velocity is determined by perforation failure of concrete block walls, concrete wall and roof panels, twin tee wall and roof panels, and metal deck roofs.

The negative pressure zones of Building 771 and the wall elements of Buildings 729 and 991 are safe under the steel pipe missile. Even in these buildings, however, the pipe missile can enter the building with significant velocity by breaching the steel siding or metal deck roof of Buildings 771, hollow metal doors of Building 729, and louvers of Building 991. All other buildings are vulnerable to perforation by the pipe missile. Vulnerable structural members are typically concrete block walls, concrete wall and roof panels, metal deck roofs, and twin-tee wall and roof panels.

For the automobile impact, also, concrete block walls and concrete wall panels are subject to overall failure. Tornado-missile impact tests were carried out by Sandia Laboratories for Electric Power Research Institute (EPRI). Although automobile crash tests were not included in this program, the results for the other missiles (Ref. 4-9) show that the analytical techniques of Reference 4-10 yield conservative values for overall failure. Therefore, judgmental factors were applied to modify these analytical techniques to decrease the conservatism of the results. In the absence of supportive test data, it is not possible to evaluate the degree of conservatism or otherwise of the results reported here for overall failure under the plank and automobile missiles.

In view of the low allowable velocities of missiles, it is apparent that the buildings at Rocky Flats cannot easily be upgraded to eliminate all possibility of being breached under the criteria missiles traveling at the specified velocities. However, concrete wall and roof panels, concrete block walls, and metal deck roofs have been found inadequate for flexure under other types of loading. The measures suggested for strengthening these structural members (for instance, reinforcement of block walls by gunite) would substantially increase resistance to both overall failure and perforation under missile impact.

Therefore, if the recommendations made for strengthening the buildings to meet the imposed seismic, tornado, and wind criteria are followed, the buildings will also be substantially reinforced against missile impact. However, it is not expected that these measures will render the buildings entirely impenetrable to the criteria missiles.

4.3.5 COMPARISON OF MAXIMUM LOAD LEVELS WITH SIMILAR BUILDING TYPES

In Section 2.3, plant buildings were grouped by similarities in construction. Eight categories were selected to represent similar types of construction. Table 4-10 identifies the buildings assigned to each category and shows the maximum load levels predicted for each building under seismic, tornado, and wind loadings. It is seen that maximum load levels vary somewhat for similar buildings in a given category. In most cases, the spread in load levels is not significant or inconsistent since differences between buildings should be expected because of differences in member dimensions, spans, connection details, etc. In cases where the spread between load levels for a given construction category is large, a more detailed evaluation of the evaluation mode for each building is required to identify and explain the differences. This additional effort was considered outside the scope of this project.

4.4 RECOMMENDATIONS FOR UPGRADING

This section describes structural modifications for strengthening building components for which failure has been predicted. The concepts are discussed by structural components rather than on a building-by-building basis since the proposed strengthening methods are applicable to construction materials and vulnerable building components that are common to many buildings.

4.4.1 SEISMIC STRENGTHENING

As discussed in Section 4.3.1, the maximum allowable load levels predicted for structures vulnerable to earthquakes are governed primarily by the in-plane shear resistance of concrete block walls, the shear resistance of connections between precast concrete twin-tee panels and that of floor and roof diaphragm connections to transverse shear walls.

4.4.1.1 Shear Walls

The in-plane shear resistance of building walls can be increased with reinforced gunite applied to the outside wall surface. The thickness of the gunite and the amount of reinforcing would be determined by the shear force the walls must resist and whether the flexure resistance of the walls must also be increased for out-of-plane, inertial forces generated by the lateral earthquake loading. Dowels inserted into drilled holes and epoxied would provide mechanical anchorage between the gunite and reinforcing steel and the wall. This strengthening method is applicable to either concrete block or reinforced concrete wall construction. The gunite strengthening concept has been used extensively with good success in strengthening buildings damaged during the 1971 San Fernando earthquake and in strengthening old unreinforced masonry load-bearing building walls. Confining the strengthening to the outside wall surface minimizes disruptions to the normal operation of the facility.

4.4.1.2 Connections

Strengthening connections to increase their resistance to shear and tension stresses is required for many of the buildings that incorporate the prestressed precast concrete twin-tee panels for floor, roof, and wall construction (Buildings 707, 707A, 708, 559, and 561). The connection details vary based on the specific building but, in general, they utilize a steel insert embedded in a concrete member that is eventually to be welded to a matching insert in the adjacent element. Steel plates are normally used to join the two inserts, with fillet welds providing the required shear transfer between members. Anchorage of the insert in the concrete member must also have adequate strength to resist the shear and tension forces acting on the connection. It was found that in some instances the additional shear resistance the connections require can be achieved by providing additional welding and/or application of additional steel plates.

If the existing connector cannot be upgraded by additional welding, other means for strengthening the connections between the panels and between the panels and building walls must be developed. For example, additional inserts can be attached to the concrete elements by anchor bolts embedded in the concrete or by use of thru-bolts. The latter detail would drill through the flanges of the precast concrete panels and extend bolts completely through the section. Weld plates would be used across the inserts as presently done to transfer shear. Since connection details vary from building to building, there are other concepts for strengthening connections that may be more appropriate based on the physical requirements at the interface.

4.4.1.3 Individual Structural Members

The strength of concrete beams and columns can be increased to provide additional resistance under combined axial and flexure stresses by use of reinforced gunite. The amount of reinforcing and gunite thickness would depend on the additional capacity required to resist the DBE. Dowels would be used to anchor the reinforcement to the existing construction.

4.4.2 TORNADO STRENGTHENING

The buildings are especially susceptible to tornado loadings since they were not designed initially for this environment. As summarized in Section 4.3.2, the maximum tornado levels that the buildings can resist are governed primarily by (1) the out-of-plane flexure resistance of the exterior walls to normal pressure, (2) the shear resistance of connections that limit the in-plane lateral loads that roof or floor diaphragms can transfer to supporting shear walls, and (3) the uplift forces the roof structures can resist. The majority of the roof systems will require extensive strengthening to avoid catastrophic failures.

4.4.2.1 Exterior Building Walls

The out-of-plane flexure resistance of concrete or concrete masonry building walls that are exposed to tornado pressures can be increased with reinforced gunite applied to the wall surface. See discussion in Section 4.4.1.1 for strengthening shear walls. Because tornado pressures can be positive or negative, it may be necessary to treat both outside and inside wall surfaces. Reinforced gunite can also be used to increase the in-plane shear resistance of concrete or concrete masonry shear walls.

The out-of-plane flexure resistance of steel frame wall construction can be accomplished by adding additional steel members, i.e., columns, girts, stiffeners, etc., or by welding additional steel shapes to existing members to increase their strength, or by a combination of the two schemes.

4.4.2.2 Connections

Welded connections between precast concrete single- and twin-tee roof panels and between the panels and the building edge beams will have to be reinforced as part of the general roof strengthening to ensure adequate transfer of lateral loads to the building shear walls. This also applies to the welded connections between precast concrete twin-tee wall panels since these walls function as shear walls in transferring roof and floor diaphragm reactions down to the foundation. Concepts for increasing the shear resistance of welded connections have been discussed in Section 4.4.1.2 under seismic upgrading requirements.

4.4.2.3 Roof Systems

Strengthening roof systems for tornado uplift loadings will be extensive and difficult to accomplish, especially for the precast concrete single- and twin-tee roof panels. These panels are not prestressed for uplift and the cantilevered flanges of the tee sections are not reinforced for negative bending. It is not obvious at this time how this can best be accomplished and will require further study. Candidate strengthening concepts for further study include the following:

- a. Remove the roofing materials, sandblast the exposed concrete surfaces, and use reinforced concrete to increase the depth of the slab and stem sections of the precast panels. Provide mechanical anchors between the old and new concrete to ensure composite action. The added weight of concrete will help reduce net uplift loads, but the additional dead load will have to be carried by the columns and bearing walls; this may require strengthening these members or adding supplementary columns.
- b. Reduce panel spans by adding tie-down column struts inside the structure that extend down to the foundation. This will be difficult in two-story buildings.
- c. Provide steel rods parabolically arched over the stems and supported on precast lightweight concrete struts. Negative or upward deflection of the roof panels will be resisted by the parabolic tension rods.

Poured-in-place concrete roof slabs that require additional flexure resistance under uplift forces can be strengthened by adding an overlay of reinforced concrete, similar to the concept suggested for the precast concrete roof panels.

Steel-framed roof systems with metal decking will be easier to strengthen for uplift pressures than concrete roof systems. Additional framing members can be added to reduce the loading on existing framing or to reduce spans. The metal roof decking can be attached to new purlins for increased flexure resistance. Rather than adding new roof framing members, the strength of roof members can be increased by welding structural shapes to the existing beams. This will increase the cross-sectional area and section modulus of the members and provide additional axial and flexure resistance. Columns where combined axial and flexure resistance must be increased can also be strengthened by welding structural shapes to the columns.

The steel open-web roof joists that require strengthening for uplift will require a detailed stress analysis to determine the most efficient method of modifying the truss for flexure in the upward direction. Stresses in the web members and the top and bottom chords will be reversed and tension members will be loaded in compression.

It was found that many precast roof panels and steel roof beams or girders are vulnerable to uplift reactions that can not be resisted by tie-down connections at points of supports. This can be corrected by incorporating additional anchorage or hold-down resistance at supports.

4.4.3 WIND STRENGTHENING

As indicated in Table 4-6, many buildings are vulnerable to the 150 mph extreme wind criteria and a reduced maximum allowable load level had to be assigned based on the negative margins of safety calculated for vulnerable components and their mode of failure. As noted in the table, the failure modes are similar to those predicted for tornados. The concepts for strengthening the vulnerable components for tornado forces are also applicable for upgrading the structure for wind forces.

4.4.4 SUMMARY

As discussed previously, candidate concepts for strengthening vulnerable components should consider what impact they would have on building function and operation while they are being implemented. As much work as possible should be accomplished from outside the building.

More important, in developing a strengthening concept, all deficiencies should be considered and integrated into the concept. For example, the walls of some buildings require strengthening for in-plane shear resistance as dictated by the design-basis earthquake. The same walls under the tornado and extreme wind criteria require strengthening for out-of-plane flexure induced by pressures acting normal to the wall. The walls can probably be upgraded to satisfy all three load conditions for a cost that is not much more than upgrading for seismic requirements only. The same philosophy is applicable to connections between the precast concrete roof and floor panels and their connections to building walls, that require strengthening to ensure that lateral forces can be distributed through the roof and floor diaphragms to the building shear walls. In general, the shear resistance of connections critical to diaphragm action require upgrading for the seismic, tornado, and wind environments.

If it is determined that all buildings at Rocky Flats must be able to resist the lateral forces imposed by the DBE, then certain buildings identified in Table 4-4 must be upgraded. It is recommended that, as a minimum:

- Shear walls requiring strengthening for in-plane shear resistance should also be strengthened for out-of-plane flexure resistance required for tornado and wind resistance.
- Connections critical to the transfer of lateral seismic loadings through roof, floor, and wall diaphragms be strengthened to also satisfy tornado and wind deficiencies.

Since uplift deficiencies of the roof structures are unique to the tornado and wind criteria, initial strengthening of buildings to increase seismic resistance would not have to incorporate strengthening requirements imposed by uplift forces.

It is recommended that, if upgrading of the structure for wind independent of tornado is contemplated, the structure should be re-examined in light of the new ANSI code which is currently in preparation. By the time such upgrading would take place, a draft of the new code should be obtainable.

4.5 REFERENCES

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TABLE 4-1. ORIGINAL WIND DESIGN CRITERIA

Group	Building	Year Constructed	Criteria		Applicable UBC	
			Wind Load, psf	Wind Speed, mph ⁽³⁾	Wind Load, psf	Wind Speed, mph ⁽³⁾
I	771 Original Building	1953	1000 ⁽¹⁾	N/A	15	75
	771 Cafeteen and Office Add'n	1962	--	--	15	75
	771 Office Addition	1966	35	120	25	100
	771 Dock Addition	--	--	--	--	--
	771 Maintenance Shop Add'n	1970	30	110	25	100
	771 Waste Packaging Add'n	1972	35	120	25	100
	774 Original Building	1953	100	N/A	15	75
	774 West Side Add'n	1963	--	--	15	75
	774 Storage Facility and Dock	1966	--	--	25	100
	774 3rd Flr Locker Room Add'n	1970	--	--	25	100
	774 3 Story Interior Upgrading	1974	75	170	25	100
			38 ⁽²⁾	--	19	--
	715 Original Building	--	75	170	--	--
II	776/777 Original Building	1957	30	110	15	75
	777 Addition	1960	30	110	15	75
	777 Betatrow Vault Add'n		35	120	15	75
	777 Assembly Plant Add'n	1965	30	110	25	100
III	707, 707A and 708 Building	--	52	140	--	--
			52 ⁽²⁾	--	--	--
IV	559 Original Building	1967	--	--	25	100
	559 Analytical Lab Add'n	1972	43	130	25	100
	561 Original Building	1973	30	110	25	100
			38 ⁽²⁾	--	19	--
	562 Original Building	1973	38 ⁽²⁾	--	--	--

NOTES: (1) Relates to blast pressure.
 (2) Indicates roof uplift pressure.
 (3) Nominal wind speed calculated from pressure p using $p = 0.00256 v^2$

TABLE 4-2. FUJITA SCALE FOR TORNADO INTENSITY

Classification	Wind Speed, mph	Damage
F-	<40	Little or none
F0	40-72	Light
F1	73-112	Moderate
F2	113-157	Considerable
F3	158-206	Severe
F4	207-260	Devastating
F5	261-318	Incredible

TABLE 4-3. SUMMARY OF BUILDING EVALUATIONS

Group	Building	Maximum Allowable Load Level		
		Seismic, g	Tornado, mph	Wind, mph
I	771 ⁽¹⁾	0.10	200	>150
	Plenum Deluge Catch Tank Room	>0.14	120	90
	774 ⁽²⁾	0.10	130	120
	715	>0.14	140	>150
II	776/777	0.06	110	90
	776A	0.14	100	90
III	707 ⁽³⁾	0.06	130	120
	707A	0.04	140	100
	708	0.10	160	120
IV	559	0.06	90	90
	561	0.10	140	120
	562	>0.14	>200	>150
V	779 ⁽⁴⁾	>0.14	100	110
	North addition to 779 (779C)	0.06	90	80
	729	>0.14	150	>150
	782	>0.14	150	140
	727	>0.14	140	130
	Exhaust Duct Tower	>0.14	150	>150
	Passageway between 779 and 777	>0.14	150	>150
VI	991	>0.14	130	120
	985	>0.14	130	130
	989	>0.14	110	130
VII	996	>0.14	NA	NA
	997	>0.14	NA	NA
	998	>0.14	NA	NA
	999	>0.14	NA	NA
	Interconnecting Tunnels	>0.14	NA	NA
VIII	886	>0.14	120	140
	875	>0.14	150	150
	Corridor	>0.14	160	>150
IX	331	>0.14	120	100

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- NOTES: (1) Concrete stack near 771 is safe under criteria loads.
(2) This includes two stacks on top of Building 774.
(3) This includes the structure connecting Buildings 707 and 708.
(4) This does not include north addition, but includes duct bridge between 729 and 779, tunnel between 779 and 782, and stack east of 729.
(5) Underground tunnels connecting Plutonium buildings are all safe under seismic loading.

TABLE 4-4. VULNERABLE STRUCTURAL COMPONENTS UNDER DESIGN BASIS EARTHQUAKE (DBE) AND SAFE ALLOWABLE SEISMIC LOAD LEVELS

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level	Judgment Assessment for Building
I	771	South exterior building wall against earth--2nd floor	Flexure resistance under lateral earth pressure	-0.3	0.10 g	0.10 g
		Pilasters in south exterior wall--2nd floor	Combined flexure and axial loading	-0.4	0.08 g	
	774	Welded connections between precast concrete twin-tee roof panels in west portion of building (774B)	Shear resistance of connections for transfer of diaphragm stresses	-0.2	0.11 g	0.10 g
		Precast concrete twin-tee roof panels supporting steel stack	Flexure resistance under loading from steel stack	-0.3	0.09 g	
II	776/777	Interior concrete masonry walls	In-plane diaphragm shear resistance	-0.6	0.06 g	0.06 g
	776A	Metal roof decking with concrete topping	Flexure strength under acceleration forces	-0.3	0.10 g	0.14 g
III	707	Precast concrete twin-tee panel-to-panel flange connections in roof and 2nd floor	Combined stresses in welded connection	-0.5	0.07 g	0.06 g
		Precast concrete twin-tee panel-to-twin-tee exterior wall connections	Shear resistance of connection	-0.5	0.07	
		Precast concrete twin-tee roof panel-to-concrete block wall connections	Shear resistance of connection	-0.8	0.03	
		Precast concrete twin-tee panel-to-panel flange connections in exterior walls	Shear stresses in welded connections	-0.5	0.07	
		2nd story precast concrete columns	Combined flexure and axial loading	-0.4 to -0.8	0.09 to 0.03	
		Concrete block stairwell wall	In-plane diaphragm shear resistance	-0.2	0.11	
	707A	Precast concrete twin-tee panel-to-panel flange connections in roof, 2nd floor, and exterior walls	Shear resistance of connection	-0.7	0.04 g	0.04 g
IV	559	Exterior concrete block walls	In-plane diaphragm shear resistance	-0.5	0.07 g	0.06 g
		Intermediate bond beams	Flexure resistance under lateral forces	-0.3	0.10 g	
		Precast concrete twin-tee panel-to-panel flange connections and flange-to-roof edge beam connection	Shear resistance of connections	-0.4 to -0.8	0.08 g to 0.03 g	
	561	Welded connection between stems of precast concrete twin-tee panels and roof edge beams	Shear resistance of connection	-0.4	0.09 g	0.10 g

NOTE The north addition to Building 779 has very low lateral resistance and is not considered in this table.

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TABLE 4-5. VULNERABLE STRUCTURAL COMPONENTS UNDER 200-MPH DESIGN BASIS TORNADO (DBT)
AND SAFE ALLOWABLE TORNADO LEVELS

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Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
I	771 Main Building	Poured-in-place concrete roof beams	Flexure resistance under uplift forces	-0.3	160	200
	771 Filter Drainage Tank Housing	Steel siding support girts	Flexure resistance under normal pressure	-0.45	150	120
		Sheet metal siding	Flexure resistance under normal pressure	-0.6	120	
		Steel columns, base plates and anchor bolts	Column flexure resistance, base plate and anchor bolt uplift tension, and shear resistance	-0.6 to -0.75	120 to 100	
		Steel roof girder and girder connections to columns and wall	Girder flexure resistance and shear/tension resistance of girder connections	-0.5 to -0.6	140 to 120	
	774A	Sheet metal roofing	Flexure resistance under uplift forces	-0.75	100	120
		Poured-in-place roof slab	Flexure resistance under uplift forces	-0.3	180	
		Penthouse concrete block walls	Flexure resistance under normal pressure	-0.8	90	
		Penthouse roof steel joist	Flexure resistance under uplift forces	-0.6	140	
	774B	Metal decking over steel-framed portion of building	Flexure resistance under loading from steel stack	-0.2	180	120
		Precast concrete flanges of twin-tee roof panels	Flexure resistance under uplift forces	-0.3	170	
		Precast concrete twin-tee roof beams	Flexure resistance under uplift forces	-0.85	130	
		Precast concrete twin-tee roof panel-to-panel flange connections	Shear resistance of connection for transfer of diaphragm stresses	-0.6	120	
		Concrete block walls	Flexure resistance under normal pressure	-0.6	120	
	774C	Guy wire for steel stack	Tension resistance against overturning	-0.4	150	150
		Precast concrete twin-tee roof panels	Flexure resistance under uplift pressures	-0.9	140	
	715	Poured-in-place 9-in. concrete walls	Flexure resistance under normal pressures	-0.3	170	140
		Concrete block walls	Flexure resistance under normal pressure	-0.6	130	

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TABLE 4-5. (CONTINUED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
11	776/77	8-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.85	75	110
		8-in. concrete block interior walls	In-plane diaphragm shear resistance	-0.4	150	
		12-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.75	100	
		Metal roof decking	Flexure resistance under uplift forces	-0.7	110	
		Steel roof beams	Combined axial and flexure resistance	-0.2 to -0.8	180 to 90	
		Steel roof girders	Flexure resistance under uplift forces	-0.3	170	
		Steel roof beam to girder connections	Shear resistance of welded connection	-0.6	125	
		Exterior wall girts	Flexure resistance under normal pressure	-0.35	160	
		Exterior asbestos cement siding	Diagonal tension shear resistance under normal pressure	-0.5	140	
		First-floor steel columns	Combined axial and flexure resistance	-0.2 to -0.07	180 to 110	
		Second-floor steel columns	Combined axial and flexure resistance	-0.4 to -0.85	150 to 75	
		Second floor steel beams	Combined axial and flexure resistance	-0.3	170	
	776A	8-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.7	110	100
		Metal roof decking	Flexure resistance under uplift pressure	-0.8	95	
		Roof girder connection	Shear resistance of bolted connection	-0.9	65	

TABLE 4-5. (CONTINUED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
III	707	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.7	150	130
		Concrete block exterior wall	Out-of-plane flexure resistance under normal pressure	-0.7	110	
		Precast reinforced concrete 1st and 2nd floor columns	Combined axial and flexure resistance	-0.45	150	
	707A	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.8	160	140
		Precast concrete twin-tee roof panel-to-panel flange connections	Shear resistance in welded connection for transfer of diaphragm stresses	-0.3	170	
		Precast reinforced concrete roof beam to precast concrete column	Shear resistance of welded connection	-0.7	110	
		Precast concrete twin-tee wall panel	Out-of-plane flexure resistance under normal pressure	-0.2	180	
		Precast concrete twin-tee wall panel-to-panel flange connection	Shear resistance in welded connection for transfer of diaphragm stresses	-0.2	180	
		Precast reinforced concrete 1st-story columns	Combined axial and flexure resistance	-0.4	150	
		Precast reinforced concrete 2nd-story columns	Combined axial and flexure resistance	-0.8	90	
	708	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.7	160	160
		Precast concrete twin-tee wall panel bottom connector	Shear resistance of welded connection	-0.4	150	
IV	559	Sections of concrete block exterior wall in north building wall	In-plane diaphragm shear resistance	-0.4	150	90
		Concrete block exterior wall	Out-of-plane flexure resistance under normal pressures	-0.9	60	
		Concrete block bond beam	Out-of-plane flexure resistance from lateral wall reactions	-0.75	100	
		Precast concrete single-tee and twin-tee roof panels	Flexure resistance under uplift forces	-0.7	170	
		Precast concrete roof panel flange connections to roof beams	Shear resistance in welded connection for transfer of diaphragm stresses	-0.85	80	
		Precast concrete roof panel connections to roof beams	Tie-down tension resistance under uplift forces	-1.00	140	
	561	Precast concrete twin-tee roof panel	Flexure resistance under uplift pressures	-0.65	180	140
		Precast concrete twin-tee roof panel flange-to-concrete roof edge beam connection and panel stem to roof beam connection	Shear resistance of welded connections for transfer of diaphragm stresses	-0.40	150	
		Precast concrete wall panels	Out-of-plane flexure resistance under normal pressures	-0.65	120	

TABLE 4-5. (CONTINUED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
V	779A	8-in. concrete block exterior wall along west side of building	Out-of-plane flexure resistance under normal pressure	-0.8	90	100
		Tapered steel roof girder	Flexure resistance under uplift pressures	-0.5	150	
		Tapered steel roof girder	Shear resistance under uplift pressures	-0.8	120	
		Metal roof decking	Flexure resistance under uplift pressure	-0.3	180	
	779B	8-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.8	90	100
	779C	10-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.9	60	90
		Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.85	150	
	779D	8-in. concrete block exterior wall	Out-of-plane flexure resistance under normal pressure	-0.8	90	100
	729	12-in. exterior concrete block cavity wall	Out-of-plane flexure resistance under normal pressure	-0.5	140	150
		Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.6	170	
	782	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.9	150	150
		Precast concrete wall panels	Out-of-phase flexure resistance under normal pressure	-0.5	140	
		Poured-in-place concrete roof slab over air lock	Flexure resistance under uplift forces	-0.2	180	
727	Poured-in-place 5-in. concrete roof slab	Flexure resistance under uplift forces	-0.3	180	140	
	Concrete block window pier	Out-of-plane flexure resistance under normal pressure	-0.7	110		
Auxiliary Structures	Exhaust duct tower walls	In-plane flexure resistance under lateral forces	-0.3	170	150	
		Out-of-plane flexure resistance under normal	-0.5	140		
	Wall framing for passageway between Buildings 779 and 777	Wall flexure resistance under lateral forces	-0.35	160	150	
Column resistance under combined axial and flexure loading	-0.6	125				

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TABLE 4-5. (CONTINUED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph.	Judgment Assessment for Building, mph
VI	991	4-in. poured-in-place concrete roof slab	Flexure resistance under uplift forces	-0.7	115	130
		Poured-in-place concrete roof beams	Flexure resistance under uplift forces	-0.4	150	
		Poured-in-place concrete roof girders	Flexure resistance under uplift forces	-0.6	130	
		Concrete columns	Combined axial and flexure resistance	-0.3	170	
	985	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.8	155	130
		Precast concrete twin-tee roof panel connections to edge beam	Resistance of connections in transferring in-plane diaphragm shear forces to walls	-0.30 to -0.7	170 to 110	
		Precast concrete wall panels	Out-of-plane flexure resistance under normal pressure	-0.2	180	
		Airlock poured-in-place concrete roof slab	Flexure resistance under uplift forces	-0.2	130	
	989	5-in. poured-in-place roof slab	Flexure resistance under uplift forces	-0.6	125	110
		Concrete block walls	Out-of-plane flexure resistance under normal pressure	-0.8	90	
VIII	886	Metal roof decking	Shear resistance under uplift forces	-0.3	170	120
		14-in. steel roof joists	Flexure resistance under uplift forces	-0.5	150	
		18-in. steel roof joists	Flexure resistance under uplift forces	-0.8	100	
		Concrete block building walls	Out-of-plane flexure resistance under normal pressure	-0.7	110	
		Concrete block exterior corridor walls	Out-of-plane flexure resistance under normal pressure	-0.4	155	
	875	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.9	150	150
		Precast concrete wall panels	Out-of-phase flexure resistance under normal pressure	-0.3	170	
		Airlock poured-in-place concrete roof slab	Flexure resistance under uplift forces	-0.2	130

TABLE 4-5. (CONCLUDED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
IX	331	6-in. poured-in-place concrete roof slab	Flexure resistance under uplift forces	-0.4	170	120
		Metal roof decking	Flexure resistance under uplift forces	-0.65	125	
		Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.4	160	
		Precast concrete twin-tee roof panel connections at walls	Tie-down resistance under uplift forces	-0.75	140	
		8-in. poured-in-place concrete walls	Out-of-plane flexure resistance under normal pressure	-0.8	90	
		8-in. concrete block walls	Out-of-plane flexure resistance under normal pressure	-0.6	125	
		10-in. poured-in-place concrete walls	Out-of-plane flexure resistance under normal pressure	-0.6	125	
		12-in. concrete block walls	Out-of-plane flexure resistance under normal pressure	-0.25	175	

TABLE 4-6. VULNERABLE STRUCTURAL COMPONENTS UNDER 150-MPH DESIGN BASIS WIND (DBW)
AND SAFE ALLOWABLE WIND LEVELS

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
I	774A	4-in. concrete block penthouse walls	Out-of-plane flexure resistance under normal pressure	-0.4	120	120
	774B	Precast concrete twin-tee roof panel-to-panel connections	Shear resistance for transfer of lateral in-plane roof loads to shear walls	-0.4	120	120
	774C	Precast concrete twin-tee roof panels at edge of building	Flexure resistance under localized uplift forces	-0.85	110	110
	Plenum Deluge Catch Tank Room	Metal roof decking	Flexure resistance under uplift forces	-0.85	60	90
		Sheet metal siding	Out-of-plane flexure resistance under normal pressure	-0.5	110	
Wall framing steel girts		Out-of-plane flexure resistance under normal pressure	-0.25	130		
Wall framing steel columns		Out-of-plane flexure and base plate shear resistance under normal pressure	-0.4	120		
II	776/777	8-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.8	70	90
		12-in. concrete block exterior walls	Out-of-plane flexure resistance under normal pressure	-0.65	90	
		Sheet metal decking	Flexure resistance under uplift forces (eaves and corner conditions)	-0.6 to -0.8	95 to 70	
		Precast concrete twin-tee roof panels	Flexure resistance under uplift forces (corner condition)	-0.5	120	
		1st floor steel columns	Combined axial and flexure resistance	-0.25 to -0.45	130 to 115	
		2nd floor steel columns	Combined axial and flexure resistance	-0.2 to -0.75	135 to 75	
		Steel roof beams	Combined axial and flexure resistance	-0.2 to -0.70	135 to 85	
		Steel beam-to-girder connection (typical)	Weld shear resistance in connection	-0.75	75	
	776A	Metal roof decking	Flexure resistance under uplift forces	-0.85	70	90
		Girder connection (typical)	Bolt shear resistance in connection	-0.85	70	
8-in. concrete block wall (typical)		Out-of-plane flexure resistance under normal pressure	-0.55	100		

TABLE 4-6. (CONTINUED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
III	707	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.55	120	120
		2nd floor precast reinforced concrete columns	Combined axial and flexure resistance	-0.50	110	
		1st floor precast reinforced concrete columns	Combined axial and flexure resistance	-0.35	120	
	707A	Precast concrete twin-tee roof panel	Flexure resistance under uplift forces	-0.8	115	100
		2nd floor precast reinforced concrete columns	Combined axial and flexure resistance	-0.8	70	
		1st floor precast reinforced concrete columns	Combined axial and flexure resistance	-0.4	120	
IV	559	Precast concrete twin-tee roof panel	Flexure resistance under uplift forces	-0.7	120	120
		Precast concrete twin-tee wall panel bottom connector	Shear resistance of welded connection	-0.4	120	
		Precast concrete single-tee roof panel	Flexure resistance under uplift forces	-0.8	120	
		Precast concrete single-tee roof panel flange	Flexure resistance under uplift forces	-0.99	60	
		Precast concrete twin-tee roof panel	Flexure resistance under uplift forces	-0.8	110	
		Precast concrete twin-tee roof panel flange	Flexure resistance under uplift forces	-0.3	130	
		Concrete block exterior cavity walls	Out-of-plane flexure resistance under normal pressure	-0.3	130	
		Concrete block bond beam at midheight of wall	Out-of-plane flexure resistance under normal wall loading	-0.65	90	
	561	Precast concrete twin-tee roof panel flange connection-to-edge wall beam	Shear resistance of connections for transfer of roof diaphragm reactions to shear walls	-0.3	130	120
		Precast concrete twin-tee roof panel connection-to-roof beams	Tie-down resistance of panels at support beams under uplift forces	-1.00	80	
		Precast concrete twin-tee roof panel flange-to-edge wall beam	Flexure resistance under uplift forces	-0.8	130	

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TABLE 4-6. (CONCLUDED)

Group	Building	Vulnerable Structural Components	Failure Mode	Margin of Safety	Calculated Maximum Allowable Load Level, mph	Judgment Assessment for Building, mph
V	779 ⁽¹⁾	Concrete block walls (typical)	Out-of-plane flexure resistance under normal pressure	-0.5 to -0.7	110 to 90	110
	782	Precast concrete twin-tee roof panels	Flexure resistance under uplift forces	-0.6	140	140
	727	Concrete block window pier	Out-of-plane flexure resistance under normal pressure	-0.3	130	130
VI	991	4-in. poured-in-place concrete roof slabs Poured-in-place concrete columns	Flexure resistance under uplift forces Combined axial and flexure resistance	-0.6 -0.4	100 120	120
	985	Precast concrete twin-tee roof panels Precast concrete twin-tee roof panel connections-to-edge beam	Flexure resistance under uplift forces Shear resistance of connections at building walls	-0.7 -0.5	130 130	130
	989	Concrete block walls	Out-of-plane flexure resistance under normal pressure	-0.3	130	130
VIII	886	18-in. steel roof joists	Flexure resistance under local uplift forces	-0.3	140	140
	875	Edge precast concrete twin-tee roof panel	Flexure resistance under local uplift forces	-0.4	150	150
	331	6-in. poured-in-place roof slab Metal roof decking at corners 8-in. poured-in-place concrete wall 8-in. concrete block walls 10-in. poured-in-place concrete walls	Flexure resistance under uplift forces Flexure resistance under uplift forces Out-of-plane flexure resistance under normal pressure Out-of-plane flexure resistance under normal pressure Out-of-plane flexure resistance under normal pressure	-0.7 +0.8 -0.6 -0.5 -0.4	100 70 100 110 120	100

NOTE: (1) The north addition to Building 779, which has very low lateral resistance, is not considered in this assessment.

TABLE 4-7. ALLOWABLE MISSILE VELOCITIES FOR SURVIVAL UNDER MISSILE IMPACT
CRITERION MISSILE: WOODEN PLANK, 4 IN. X 12 IN. X 12 FT, 108 LB

Building Group	Building Number	Horizontal Missile			Vertical Missile		
		Vulnerable Structural Member	Failure Mode	Allowable Horizontal Velocity, mph	Vulnerable Structural Member	Failure Mode	Allowable Vertical Velocity, mph
I	771	12-in. RC wall, typ	Perforation	182 ⁽²⁾	8-in. RC roof	Perforation	110 ⁽²⁾
	715	8-in. conc. block wall, typ(1)	Perforation	25	8-in. precast hollow core roof, typ	Perforation	48
	774	8-in. conc. block wall, 2d flr	Perforation	25	5-in. RC roof, typ	Perforation	62
II	776/777	8-in. conc. block wall, typ(3)	Perforation	25	22 ga. metal deck	Perforation	18
III	707	Twin tees, typ	Perforation	25	Twin tees with 2-in. topping, typ	Perforation	52
	707A	Twin tees, typ	Perforation	25	Twin tees with 2-in. topping, typ	Perforation	52
	708	Twin tees, typ	Perforation	25	Twin tees with 2-in. topping, typ	Perforation	52
IV	559	10-in. cavity wall, typ	Perforation	41	Single tee roof panel, typ	Perforation	25
	561	6-in. precast panel, typ	Perforation	77	Twin tees with 2-in. topping, typ	Perforation	52
	562	8-in. conc. block wall, typ	Perforation	25	8-in. cored slab, typ	Perforation	48
V	779	8-in. conc. block wall, Line 1	Perforation	25	18 ga. metal deck, low roof, typ	Perforation	27
	729	Hollow metal door	Perforation	57	Twin tees with 4-in. topping, typ	Perforation	57
	782	6-in. precast conc. panel, typ	Perforation	77	Twin tees with 2-in. topping, typ	Perforation	52
	727	8-in. conc. block wall, typ	Perforation	25	5-in. conc. roof, typ	Perforation	62
VI	991	12-in. conc wall	Perforation	182 ⁽²⁾	4-in. conc. roof, typ	Perforation	48
	985	6-in. precast conc. panel, typ	Perforation	77	Twin tees with 2-in. topping, typ	Perforation	52
	989	8-in. conc. block wall, typ	Perforation	25	5-in. RC roof, typ	Perforation	62
VIII	886	8-in. conc. block wall, typ	Perforation	25	22 ga. metal deck, typ	Perforation	18
	875	6-in. precast conc. panels, typ	Perforation	77	Twin tees with 2-in. topping, typ	Perforation	52
IX	331	8-in. conc. block wall, Lines 6 and 9	Perforation	25	Twin tees with no topping	Perforation	25

NOTES:

- (1) All concrete block walls listed in this table are partially filled unless otherwise mentioned.
- (2) These values may be overconservative. Bldg. 771 and walls of Bldg. 991 are judged capable of withstanding criterion impact.
- (3) Transite panel on south wall perforated at 3 mph.

TABLE 4-8. ALLOWABLE MISSILE VELOCITIES FOR SURVIVAL UNDER MISSILE IMPACT
CRITERION MISSILE: STEEL PIPE, 3 IN. DIA. x 10 FT, 76 LB

Building Group	Building Number	Horizontal Missile		Vertical Missile	
		Vulnerable Structural Member	Allowable Horizontal Velocity, mph	Vulnerable Structural Member	Allowable Vertical Velocity, mph
I	771	N.A. (See Note 2)	N.A.	N.A.	N.A.
	715	8-in. conc. block wall, typ	17	8-in. precast hollow core roof, typ	48
	774	8-in. conc. block wall, typ	17	5-in. RC roof, typ	54
II	776/777	8-in. conc. block wall, typ	17	22 ga. metal deck	20
III	707	Twin tees, typ	17	Twin tees with 2-in. topping, typ	40
	707A	Twin tees, typ	17	Twin tees with 2-in. topping, typ	40
	708	Twin tees	17	Twin tees with 2-in. topping, typ	40
IV	559	10-in. cavity wall, typ	32	Single tee roof panel, typ	17
	561	6-in. precast wall panels, typ	76	Twin tees with 2-in. topping, typ	40
	562	8-in. conc. block wall, typ	17	8-in. conc. cored slab, typ	24
V	779	8-in. conc. block wall, Line 1	17	18-ga. metal deck, low roof, typ	30
	729	N.A.	N.A.	Twin tees with 2-in. topping, typ	40
	782	6-in. precast conc. panel, typ	76	Twin tees with 2-in. topping, typ	40
	727	8-in. conc. block wall, typ	17	5-in. solid concrete roof, typ	54
VI	991	N.A.	N.A.	4-in. solid concrete roof, typ	37
	985	6-in. precast conc. panel, typ	76	Twin tees with 2-in. topping typ	40
	989	8-in. conc. block wall, typ	17	5-in. RC roof, typ	54
VIII	886	8-in. conc. block wall, typ	17	22-ga. metal deck, typ	20
	875	6-in. precast conc. panel, typ	76	Twin tees with 2-in. topping, typ	40
IX	331	8-in. conc. block wall, Line 6 & 9	17	Twin tees with no topping	17

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- NOTES: 1. Failure mode under the pipe missile is by perforation.
2. N.A. indicates that building can withstand criterion missile velocity (Horizontal velocity = 100 mph; vertical velocity = 67 mph)
3. Glass windows, louvers, and hollow metal doors excluded in calculating allowable velocities.
4. All concrete block walls listed in this table are partially filled.

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TABLE 4-9. ALLOWABLE MISSILE VELOCITIES FOR SURVIVAL UNDER MISSILE IMPACT

Criterion Missile: Automobile, 3000 lb, Contact Area of 20 sq ft

Building Group	Building Number	Vulnerable Structural Member	Allowable Horizontal Velocity, mph
I	771	12-in. R.C. wall, typ	18
	715	8-in. conc. block wall, (partially filled) typ	6
	774	8-in. conc. block wall, (partially filled) typ	6
II	776/777	8-in. conc. block wall (partially filled), typ	6
III	707	Twin tees, typ	6
	707A	Twin tees, typ	6
	708	Twin tees, typ	6
IV	559	10-in. cavity wall, typ	10
	561	6-in. precast wall panel, typ	21
	562	8-in. conc. block wall (partially filled), typ	6
V	779	8-in. conc. block wall (partially filled) Line 1	6
	729	12-in. hollow wall around bldg.	12
	782	6-in. precast conc. panel, typ	21
	727	8-in. conc. block wall (partially filled) typ	6
VI	991	12-in. conc. outside wall	18
	985	6-in. precast conc. panel, typ	21
	989	8-in. conc. block wall (partially filled) typ	6
VIII	886	8-in. conc. block wall (partially filled) typ	6
	875	6-in. precast conc. panel, typ	21
IX	331	8-in. conc. block wall (partially filled) Lines 6 & 9	6

NOTE: Mode of failure under automobile impact is overall failure

TABLE 4-10. COMPARISON OF MAXIMUM LOAD LEVEL
WITH SIMILAR BUILDING TYPES

Category	Building No.	Maximum Allowable Load Level		
		Seismic	Tornado	Wind
A	715	>0.14	140	>150
	562	>0.14	>200	>150
	727	>0.14	140	130
	989	>0.14	110	130
B	561	0.10	140	120
	985	>0.14	130	130
	875	>0.14	150	150
	782	>0.14	150	140
C	771	0.10	200	>150
	991	>0.14	130	120
D	707	0.06	130	120
	707A	0.04	140	100
	708	0.10	160	120
E	776/777	0.06	110	90
	776A	0.14	100	90
F	559	0.06	90	90
	729	>0.14	150	>150
G	996	>0.14	NA	NA
	997	>0.14	NA	NA
	998	>0.14	NA	NA
	999	>0.14	NA	NA
H	774	0.10	130	120
	779	>0.14	100	110
	886	>0.14	120	140
	331	>0.14	120	100