Seismic Safety Program: Ground Motion and Structural Response

**Prepared for
U.S. Department of Energy
Nevada Operations Office
Under Contract DE-AC08-89NV10733**

**Prepared by
URS/John A. Blume & Associates, Engineers
100 California Street
San Francisco, CA 94111**

May 1993

DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

DISCLAIMER

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represented that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors express herein do not necessarily state or reflect those of the United States Government or any agency thereof.

Printed in the United States of America
Available from
National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Road
Springfield, VA 22161

NTIS price codes

Printed copy: A17
Microfiche copy: A01

Seismic Safety Program: Ground Motion and Structural Response

Prepared for
U.S. Department of Energy
Nevada Operations Office
Under Contract DE-AC08-89NV10733

Prepared by
URS/John A. Blume & Associates, Engineers
100 California Street
San Francisco, CA 94111

May 1993

MASTER

JB
DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

RECEIVED
OCT 15 1993
OSTI

DISCLAIMER

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, nor any of their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, apparatus, product, or process disclosed, or represented that its use would not infringe privately owned rights. Reference herein to any specific commercial product, process, or service by trade name, trademark, manufacturer, or otherwise does not necessarily constitute or imply its endorsement, recommendation, or favoring by the United States Government or any agency thereof. The views and opinions of authors express herein do not necessarily state or reflect those of the United States Government or any agency thereof.

Printed in the United States of America
Available from
National Technical Information Service
U.S. Department of Commerce
5285 Port Royal Road
Springfield, VA 22161

NTIS price codes

Printed copy: A17
Microfiche copy: A01

Seismic Safety Program: Ground Motion and Structural Response

May 1993

**Prepared for
U.S. Department of Energy
Nevada Operations Office
Under Contract DE-AC08-89NV10733**

**Prepared by
URS/John A. Blume & Associates, Engineers
100 California Street
San Francisco, CA 94111**

FOREWORD

Thirty years ago the Nevada Operations Office of the U.S. Atomic Energy Commission began a Seismic Safety Support Program to resolve potential problems arising from the effects of earthquake-like ground motion generated by underground nuclear explosions at the Nevada Test Site. The Program was also extended, on occasion, to similar experiments conducted in areas outside the Nevada Test Site for studies involving peaceful applications of underground nuclear explosions and for some nuclear weapons development tests.

This report sets forth the history of work by private consultants retained by the Nevada Operations Office to implement the Seismic Safety Support Program, under the scientific management of National Laboratories involved in the underground nuclear testing program, and with supervision by the Nevada Operations Office. The report is being published to help increase public knowledge, understanding, and awareness of the extensive public safety program associated with underground nuclear testing.

This effort required fundamental research in seismic instrumentation, ground motion predictions, and structural response to induced ground motion. Because of the obvious value of these studies to scientific and engineering communities, their findings have been made broadly available through published reports and technical journals, and have formed the basis for a number of earthquake engineering design procedures.

This report was prepared by URS/John A. Blume & Associates, Engineers. It is primarily concerned with structural response aspects of the Seismic Safety Support Program, which have been URS/Blume's responsibility throughout the duration of the program. It also includes summary descriptions of the ground motion prediction program conducted by the Environmental Research Corporation and the seismic instrumentation program assigned to the Special Project Party, U.S. Geological Survey. Input from these latter two consultants was essential to the solution of the critical seismic safety problem, i.e., the structural response of buildings exposed to the ground motions generated by underground nuclear explosions. The Seismic Safety Support Program has been eminently successful, due in large part to the close cooperation of the three consultants, the National Laboratories scientific managers, and the Nevada Operations Office supervisors.

Nick C. Aquilina
Manager

ABSTRACT

In 1964, John A. Blume & Associates Research Division (Blume) began a broad-range structural response program to assist the Nevada Operations Office of the U.S. Atomic Energy Commission (AEC) in ensuring the continued safe conduct of underground nuclear detonation testing at the Nevada Test Site (NTS) and elsewhere. Blume's long experience in earthquake engineering provided a general basis for the program, but much more specialized knowledge was required for the AEC's purposes. Over the next 24 years Blume conducted a major research program to provide essential understanding of the detailed nature of the response of structures to dynamic loads such as those imposed by seismic wave propagation. The program's results have been embodied in a prediction technology which has served to provide reliable advanced knowledge of the probable effects of seismic ground motion on all kinds of structures, for use in earthquake engineering and in building codes as well as for the continuing needs of the U.S. Department of Energy's Nevada Operations Office (DOE/NV).

This report is primarily an accounting of the Blume work, beginning with the setting in 1964 and the perception of the program needs as envisioned by Dr. John A. Blume. Subsequent chapters describe the structural response program in detail and the structural prediction procedures which resulted; the intensive data acquisition program which, as is discussed at some length, relied heavily on the contributions of other consultant-contractors in the DOE/NV Seismic Safety Support Program; laboratory and field studies to provide data on building elements and structures subjected to dynamic loads from sources ranging from testing machines to earthquakes; structural response activities undertaken for testing at the NTS and for off-NTS underground nuclear detonations; and concluding with an account of corollary studies including effects of natural forces and of related studies on building response. Appendices list published official reports to DOE/NV and also technical journal publications which were primarily intended to make generally available to the scientific community the valuable information acquired under this program. Reports and journal articles by the Environmental Research Corporation, the Special Projects Party, and by DOE/NV are also listed in these appendices.

The acronym "DOE/NV" is used throughout this report, replacing earlier designations for the federal agency responsible for the underground nuclear testing program, which included the Seismic Safety Support Program.

ACKNOWLEDGEMENTS

The structural response study program conducted by Blume was greatly facilitated by the outstanding qualities and capabilities of the people with whom we worked. In particular we found the DOE/NV staff to be deeply interested in our work and extremely helpful in assisting us to achieve the goals of our program.

Our audiences, reviewers and overseers from the National Laboratories associated with DOE/NV's programs, and from Sandia National Laboratories (SNL) in particular, provided a scientific comprehension of the physical processes and the statistical nature of the fundamental problems involved, which allowed a close interaction between ourselves and laboratory representatives, and an equally close and efficient transfer of information. The Special Projects Party of the U.S. Geological Survey (USGS) and the Environmental Research Corporation (ERC), our consultant-contractor counterparts in the safety program, provided us with excellent support and did so with great good will even when meeting our work requirements detracted from their own needs.

We acknowledge a particular debt of gratitude to the Las Vegas and southern Nevada public officials, building owners, and private individuals who have afforded us their essential cooperation and assistance whenever our work brought us into contact with them.

Individual acknowledgements for this long-running program would be endless, but Richard Navarro, formerly of the U.S. Coast and Geodetic Survey (USC&GS) and USGS, and now on the DOE/NV staff has been particularly helpful, both because of his profound technical awareness of our work and also the assistance he has provided us in meeting our goals. From SNL we have been greatly benefitted by the guidance of Dr. John R. Banister, and of his staff, particularly Dr. D. M. Ellett. We would like to express our appreciation to all these agencies and individuals for allowing us to participate in a project which combined unusual demands with a sense of great achievement and thus a gratifying sense of accomplishment.

This report was compiled by Ken Honda and Roger Skjei. In addition to the input from reports and publications, acknowledgements should go to the present and former members of the project who provided input to this report; in particular to Roger Scholl. We also have had the benefit of John Blume's review and comments throughout the preparation of the report.

ACRONYMS AND GLOSSARY

ADP	Automatic Data Processing
AEC	Atomic Energy Commission
ANFO	Ammonium Nitrate Fuel Oil
Area	A land surface identified by a number suffix, forming a subdivision of NTS into many small segments for various control purposes
ATP	Accelerated Test Program
Blume	John A. Blume & Associates, Engineers
Bu Mines	United States Bureau of Mines
Contractor	An agency or firm providing specialized professional or commercial services under the terms of a contract with DOE/NV or its predecessor
CSC	Computer Science Corporation (Parent company of ERC)
Design Yield	The expected yield of a planned nuclear explosion
DOD	Department of Defense
DOE/NV	Department of Energy's Nevada Operations Office
DRI	Desert Research Institute, University of Nevada
EIS	Engineering Intensity Scale
ERC	Environmental Research Corporation
Event	An underground nuclear explosion
GZ	Ground Zero
High-rise	A building classification used to identify a building of 9 or more stories high
IOCS	Atlantic-Pacific Interoceanic Canal Study Commission
kt	A kiloton. Nuclear explosive energy equivalent to an explosion of one thousand tons of TNT
LANL	Los Alamos National Laboratory
LLNL	Lawrence Livermore National Laboratory

Low-rise	Building of one or two stories
LTBT	Limited Test Ban Treaty
LV	Las Vegas
Maximum Yield	The greatest possible yield of a planned nuclear event
MT	A megaton. Nuclear explosive energy equivalent to an explosion of one million tons of TNT
NOAA	National Oceanic and Atmospheric Administration
NRDS	Nuclear Rocket Development Station, a research area in the southwestern portion of the NTS
NSF	National Science Foundation
NTIS	National Technical Information Service
NTS	The Nevada Test Site, a 1,350 square mile area in southern Nevada in Nye county and about 65 miles northwest of Las Vegas.
Off-site	Outside the boundaries of the Nevada Test Site
Pahute Mesa	A plateau in the northwest corner of the Nevada Test Site
Plowshare	A program (no longer active) for the application of nuclear explosives to develop peaceful uses of atomic energy
Safety Panel	Panel of Safety Consultants
Seiche	A pronounced and repeated water surface rise and fall, generated by basin resonance within a contained area such as a lake or harbor
Settlement	Ground surface depression
Shot	An underground nuclear explosion
SMM	Spectral Matrix Method
SNL	Sandia National Laboratories
Subsidence	Ground surface depression
TEP	Threshold Evaluation Procedure
TNT	Trinitrotoluene - a high explosive

Tsunami	A fast-moving seismic-generated sea wave frequently causing inundation of low-lying coastal areas
UBC	Uniform Building Code
UCSB	University of California at Santa Barbara
UNE	Underground Nuclear Explosions
USC&GS	U.S. Coast and Geodetic Survey
USGS	U.S. Geological Survey
Vela Uniform	A Department of Defense program to improve capability to detect, identify and locate underground nuclear explosions (no longer active)
Weapons Related	A nuclear detonation conducted for the purpose of testing a nuclear device intended for a specific type of weapon system
Yield	The total effective energy release in a nuclear explosion, usually expressed in terms of equivalent tonnage of TNT
Yucca Flat	A valley area in the north central portion of the Nevada Test Site

**SEISMIC SAFETY PROGRAM:
GROUND MOTION AND STRUCTURAL RESPONSE**

CONTENTS

	<u>page</u>
FOREWORD	iii
ABSTRACT	v
ACKNOWLEDGEMENTS	vii
ACRONYMS AND GLOSSARY	ix
SUMMARY SECTION	Page 1
1 INTRODUCTION	1-1
1.1 BACKGROUND	1-1
1.1.1 Nevada Test Site	1-2
1.1.2 Atomic Energy Commission Organization and Responsibilities	1-2
1.1.3 National Laboratories Role	1-4
1.1.4 Nevada Test Site Planning Board	1-4
1.1.5 Safety Panel of Consultants	1-4
1.1.6 Scientific Management Centers	1-5
1.1.7 Contractor Assignments	1-7
1.1.7.1 Special Projects Party	1-7
1.1.7.2 Environmental Research Corporation	1-9
1.1.7.3 John A. Blume & Associates, Research Division	1-13
1.2 INITIATION OF THE STRUCTURAL RESPONSE PROGRAM	1-18
1.2.1 General Objectives and Need	1-18
1.2.2 The Need for Empirical Data	1-19
1.2.3 Some of the Gaps in the Knowledge	1-20
2 OVERVIEW OF THE STRUCTURAL RESPONSE PROGRAM	2-1
2.1 COPE OF THE STRUCTURAL RESPONSE PROGRAM	2-1
2.2 FACTORS THAT SHAPED THE STRUCTURAL RESPONSE PROGRAM	2-3
2.2.1 Operational Scope of Structural Response Evaluations	2-6
2.2.2 Investigative needs of the Structural Response Program	2-7
2.3 GROUND MOTION PREDICTION	2-8
2.4 MOTION DATA ACQUISITION AND PROCESSING	2-10

CONTENTS (continued)

	<u>page</u>
2.5 STRUCTURAL RESPONSE EVALUATIONS	2-11
2.5.1 Surveys and Inventories	2-12
2.5.2 Damage Assessments	2-13
2.5.3 Safety Recommendations	2-14
2.5.4 Shot Monitoring of Structures	2-14
2.5.5 Postshot Damage Complaint Investigation	2-14
2.6 STRUCTURAL RESPONSE INVESTIGATIONS CONDUCTED	2-15
2.6.1 Field Investigations	2-15
2.6.2 Laboratory Testing	2-16
2.6.3 Statistical Evaluations of Damage	2-16
2.6.4 Postearthquake Damage Investigations	2-17
2.7 STRUCTURAL RESPONSE EVALUATION PROCEDURES DEVELOPED	2-17
2.7.1 Engineering Intensity Scale (EIS)	2-18
2.7.2 Spectral Matrix Method (SMM)	2-18
2.7.3 Threshold Evaluation Procedure (TEP)	2-19
3 DATA ACQUISITION	3-1
3.1 SEISMIC DATA ACQUISITION PROGRAM	3-1
3.1.1 Background	3-1
3.1.2 Instruments	3-1
3.1.3 Deployment and Operation	3-5
3.1.4 Record Processing	3-17
3.1.5 Seismic Record Maintenance	3-21
3.2 STRUCTURE DATA COLLECTION	3-24
3.2.1 Structure Reconnaissance and Inventory	3-24
3.2.2 Detailed Structural and Architectural Data	3-26
3.2.3 Structure Condition Survey	3-30
3.2.4 Geotechnical and Soil Foundation Data	3-32
3.2.5 Structural Damage Data	3-33
4 TECHNOLOGY DEVELOPMENT AND APPLICATION	4-1
4.1 LABORATORY DAMAGE AND MOTION PERCEPTION STUDIES	4-1
4.1.1 Cyclic Racking Studies of Nonstructural Wall Panels	4-1
4.1.2 Human Perception of Motion	4-8

CONTENTS (continued)

	<u>page</u>
4.2 FIELD RESPONSE MEASUREMENTS AND STUDIES	4-11
4.2.1 Four-Story Structures	4-11
4.2.2 One- and Two-Story Structures	4-23
4.2.3 Dynamic Testing of Las Vegas Area Single-Story Structures	4-27
4.2.4 Dynamic Characteristics of High-Rise Buildings	4-33
4.2.5 Dynamic Response of Bridges	4-36
4.3 CHEMICAL EXPLOSIONS	4-37
4.3.1 Trinidad, November 1970	4-37
4.3.2 Old Reliable Mine, November 1972	4-39
4.3.3 Zonia, April 1973	4-39
4.3.4 Cerrillos, January 1977	4-41
4.3.5 Other Chemical Explosion Projects	4-45
4.4 POST-EARTHQUAKE INVESTIGATIONS	4-46
4.4.1 Anchorage, Alaska - March 27, 1964	4-46
4.4.2 Dulce, New Mexico - January 22, 1966	4-48
4.4.3 Parkfield, California - June 27 to 29, 1966	4-50
4.4.4 Truckee, California - September 12, 1966	4-52
4.4.5 1967 Colombian Earthquakes	4-53
4.4.6 Fairbanks - Alaska, June 21, 1967	4-57
4.4.7 Mexico City - August 21, 1968	4-59
4.4.8 San Fernando, California - February 9, 1971	4-61
4.4.9 Santa Barbara, California - August 13, 1978	4-81
4.5 STATISTICAL STUDY OF LOW-RISE BUILDING DAMAGE	4-84
4.5.1 Formulation of Damage Parameters	4-84
4.5.2 Results from RULISON	4-85
4.5.3 NTS Events 1966 to 1970	4-86
4.5.4 Results from RIO BLANCO	4-87
4.5.5 Combined Study	4-103
4.6 RESPONSE EVALUATION PROCEDURES	4-90
4.6.1 Guidelines for Computer Modeling	4-90
4.6.2 Identification of Dynamic Characteristics	4-92
4.6.3 Motion-Damage Relationships	4-93

CONTENTS (continued)

	<u>page</u>
4.6.4 The Engineering Intensity Scale	4-99
4.6.5 Spectral Matrix Method	4-99
4.6.6 Threshold Evaluation Procedure	4-104
5 RESPONSE EVALUATIONS FOR NTS EVENTS	5-1
5.1 CHRONOLOGY OF NTS EVENTS	5-1
5.2 CHANGES IN THE AFFECTED AREA	5-3
5.3 EVENT SUPPORT ACTIVITIES	5-3
5.3.1 BILBY through HANDLEY	5-6
5.3.2 After HANDLEY Through The Accelerated Test Program	5-8
5.3.3 After The Accelerated Test Program	5-9
5.4 HIGH-RISE BUILDING CONSTRUCTION IN LAS VEGAS	5-10
5.4.1 Number of High-Rise Buildings	5-10
5.4.2 Types of High-Rise Buildings	5-11
5.5 CHANGES IN UNIFORM BUILDING CODE AND THEIR EFFECTS	5-13
5.6 HIGH-RISE BUILDING DESIGN AND ANALYSIS TECHNIQUES	5-15
5.7 OBSERVATION OF HIGH-RISE BUILDING CONSTRUCTION	5-17
5.7.1 Use of Concrete Masonry	5-17
5.7.2 Use of Pre-Cast Floor System	5-18
5.7.3 Post-Tensioned Flat-Plate System	5-18
5.8 SPECIAL OBSERVATIONS AND STUDIES	5-20
5.8.1 Complex Structural Configuration	5-20
5.8.2 Seismic Strengthening	5-22
5.8.3 Convention Center Canopy Failure	5-27
6 OFFSITE RESPONSE EVALUATIONS	6-1
6.1 MAJOR OFFSITE TESTS	6-1
6.1.1 Central Nevada	6-2
6.1.2 Vela Uniform Program	6-11
6.1.3 Plowshare	6-13
6.1.4 Interoceanic Canal Study	6-30
6.2 STANDARD PRE-SHOT ACTIVITIES FOR OFFSITE EVENTS	6-35
6.3 MONITORING AND POST-SHOT ACTIVITIES	6-38

CONTENTS (continued)

	page
6.4 SPECIAL STUDIES	6-40
6.4.1 SALMON Damage	6-40
6.4.2 RULISON - Harvey Gap Dam	6-45
6.4.3 WAGON WHEEL - Bridges	6-46
7 PARALLEL STUDIES	7-1
7.1 GEOTECHNICAL INVESTIGATIONS	7-1
7.1.1 Compaction Fault Scarp Studies	7-1
7.1.2 Subsidence Problems	7-3
7.1.3 Expansive Soils	7-4
7.2 WIND STUDIES	7-5
7.3 EARTHQUAKE HAZARD IN SOUTHERN NEVADA	7-7
7.4 TERMINAL WASTE STORAGE	7-9
7.5 HISTORIC SITE SURVEY	7-12

APPENDICES

Appendix A Blume Reports	A-1
Appendix B Selected Blume Technical Journal Publications	B-1
Appendix C Selected USC&GS-NOAA-USGS Special Projects Party Reports	C-1
Appendix D ERC Reports and Selected Technical Journal Publications	D-1
Appendix E Selected DOE/NV Reports	E-1

TABLES

2.1 List of Major UNE Events in Structural Response Program	2-2
3.1 Summary of Seismic Motion Records	3-25
3.2 Summary Tabulation of Damage Complaints and Claims	3-34
4.1 Description of Nonstructural Wall Panels	4-4
4.2 Zonia Mine Explosion Data	4-40
4.3 Cerrillos Peak Particle Velocity Calibration Test	4-43
4.4 Cerrillos Peak Particle - Prototype Test	4-44
4.5 Seismological Data, 1967 Earthquakes	4-54

CONTENTS (continued)

	<u>Page</u>
4.6 Peak Ground Motion in Bogota	4-56
4.7 Strong Motion Effects	4-56
4.8 Ground Motion Data - Duckering Hall, University of Alaska	4-58
4.9 Spectral Velocity - Duckering Hall Basement	4-58
4.10 Damage Identification Numbers	4-64
4.11 Number and Percentage of Buildings With and Without Chimneys for Each Building Class	4-64
4.12 Number and Damage Ratio in Percent of Each Damage Type for Each Building Class	4-66
4.13 Number of Chimneys of Different Construction Materials Found at Various Locations on Buildings	4-67
4.14 Comparison of Damage Ratios (in Percent) of Chimneys with Different Construction Materials and Found At Various Locations on Buildings	4-67
4.15 Damage Ratio in Percent for Capped and Uncapped Chimneys	4-68
4.16 Damage Ratios in Percent of Brick Chimneys for One-Story Buildings	4-68
4.17 Mobile Home Survey; San Fernando Earthquake Related Damage	4-71
4.18 Swimming Pool Survey; San Fernando Earthquake Related Damage	4-71
4.19 Distribution of Buildings in Glendale Study Areas	4-75
4.20 Average Predicted 5% Damped Response Spectrum Values for Glendale Study Areas	4-75
4.21 Number of Buildings with Damaged Components: Glendale Study Areas	4-76
4.22 Damage Ratios in Percent for Glendale Study Areas	4-76
4.23 Observed Fundamental Periods	4-78
4.24 Calculated Fundamental Periods	4-78
4.25 Damage Comparisons	4-79
4.26 Components Considered Relevant For Local Response Parameters	4-100
5.1 Summary of NTS Events	5-2
5.2 Population of Communities near the Nevada Test Site	5-5
6.1 FAULTLESS Event - Summary of Peak Acceleration Ratios of Close-In Stations	6-4
6.2 FAULTLESS Event - Comparison of Peak Structural Accelerations	6-5
6.3 FAULTLESS Event - Peak Measured Structural Acceleration and Displacement	6-6
6.4 FAULTLESS Event - Comparison of Predicted and Actual Structural Effects	6-8
6.5 Peak Acceleration Data, Project SALMON	6-12

CONTENTS (continued)

	<u>page</u>
6.6 Predicted and Actual Damage from Project RULISON	6-20
6.7 RIO BLANCO - Peak Ground Motion Values	6-24
6.8 RIO BLANCO Damage Data	6-25
6.9 RIO BLANCO Building Inventory and Building Description	6-26
6.10 Damage Statistics	6-27
6.11 Spectral Amplitude Values	6-27
6.12 Summary of Damage Potential - Significant Structures	6-49
7.1 Summary of Estimated Hardening Costs as Percentage Increase	7-11
7.2 Acceleration-Damage Relationships	7-15

FIGURES

1.1 Southern Nevada Area Map	1-3
1.2 1970 Blume Organization Chart	1-15
2.1 DOE/NV Off-Site Ground Motion Safety Program in 1964	2-4
3.1 Instrument Utilization	3-3
3.2 Frequency Response Curves of Various Seismographs	3-4
3.3 Frequency Response Curve of L-7 System	3-4
3.4 USC&GS Instrument Operation Flowchart	3-7
3.5 BOXCAR Event Close-In Instrumentation	3-11
3.6 BOXCAR Event Velocity Instrument Location	3-12
3.7 FAULTLESS Event Close-In Instrumentation	3-14
3.8 FAULTLESS Event Velocity Instrument Locations	3-15
3.9 ERC ADP System Block Diagram	3-19
3.10 ERC ADP Configuration for Various Processes	3-20
3.11 Seismic Signal Processing with PDP-11/34	3-22
3.12 Seismic Signal Processing with VAX8200	3-23
3.13 Communities Covered in 1965 Survey	3-27
3.14 Building Information Collection Form	3-28
3.14 Building Classification	3-29
4.1 Example Cyclic Force-Displacement Plots	4-3
4.2 Example Backbone Force-Displacement Curve	4-4

CONTENTS (continued)

	<u>page</u>
4.3 Definition of Energy Terms	4-5
4.4 Regression Curve for Period versus Perception Threshold	4-10
4.5 4-Story Concrete Test Structures	4-12
4.6 Experimentally Obtained Fundamental Mode Periods for Longitudinal Motion	4-14
4.7 Experimentally Obtained Fundamental Mode Periods for Transverse Motions	4-15
4.8 Experimentally Obtained Fundamental Mode Damping Ratios for Longitudinal Motions	4-16
4.9 Experimentally Obtained Fundamental Mode Damping Ratios for Transverse Motions	4-17
4.10 Installation of Major Equipment	4-18
4.11 Velocity versus Period for Nondestructive Test, Longitudinal Direction	4-21
4.12 Base Shear versus Roof Displacement for the Destructive Tests of the Original and Epoxy-Repaired Structures	4-22
4.13 NTS Test Structure Locations	4-24
4.14 Maximum Roof Level Accelerations and Period in Chronological Sequence, Building F	4-34
4.15 Log-Log Graph Showing Relationship of Low-Rise Building Complaint Ratio to Spectral Acceleration, All Events	4-95
4.16 Log-Log Graph Showing Relationship of Low-Rise Building Damage Ratio to Spectral Acceleration, All Events	4-96
4.17 Log-Log Graph Showing Relationship of Low-Rise Building Mean Damage Factor to Spectral Acceleration, All Events	4-97
4.18 Log-Log Graph Showing Relationship of Low-Rise Building Coefficient of Variation to Spectral Acceleration, All Events	4-98
4.19 Engineering Intensity Scale Matrix Superimposed with Example Spectrum	4-101
4.20 Iso-EIS Lines for Period Band of $T < 0.4$ Second, Event FAULTLESS	4-102
5.1 Nevada Test Site and Nearby Communities	5-4
5.2 Las Vegas High-Rise Construction (5-Year Period by Type)	5-12
5.3 UBC Seismic Zones	5-14
5.4 Comparison of Base Shear Coefficients	5-16
5.5 Nevada Test Site and Surrounding Area	5-25
6.1 GASBUGGY Area Map	6-15
6.2 RULISON Area Map	6-18
6.3 RIO BLANCO Area Map	6-23
6.4 Location of L-7 Seismic Stations, RIO BLANCO Event	6-28
6.5 Interocean Canal Project Area Map	6-31

CONTENTS (continued)

	<u>page</u>
6.6 SALMON Area Map	6-41
6.7 Project WAGON WHEEL Area Map	6-47
7.1 Physiography of Las Vegas Valley	7-2
7.2 Historic Structure Survey Area	7-14

SUMMARY SECTION

Introduction

In the early 1960's, the Nevada Operations Office of the U.S. Atomic Energy Commission (now the Nevada Operations Office of the U.S. Department of Energy)(DOE/NV) identified a potential safety problem arising from the effects of ground motion generated by underground nuclear testing at the Nevada Test Site (NTS). Among other actions taken to resolve this problem, the Nevada Operations Office entered into a contract in January 1964 with John A. Blume & Associates, Research Division (Blume), for consultation in the field of structural response to ground motion. The basic objective of the contract arrangement was to maintain the capability of DOE/NV for conducting underground nuclear testing, primarily at the NTS, while ensuring the safety of the neighboring population and minimizing damage to public and private structures.

In 1985, Blume began the preparation of an historic account of the steps undertaken during the preceding 20 years to meet the basic objective of the work conducted for DOE/NV. Our subsequent draft submittals have been reviewed by DOE/NV and Sandia National Laboratories (SNL). The original report has been consequently expanded to include discussion of the work of the ground motion contractor and the seismic data contractor. Their participation with Blume in a closely-associated effort under the supervision of DOE/NV and SNL was essential to the effective conduct of the ground motion and structural response safety program.

This report has primarily been summarized from the technical documents issued by the ground motion and structural response contractors during the 1964 - 1984 period. Much more detail is available in the original Blume reports and papers listed in Appendices A and B, and in those of other participants as set forth in subsequent appendices.

Organizations

Blume was one of a number of contractors engaged by DOE/NV for the purpose of ensuring safe conduct of underground nuclear testing. The firm of Roland F. Beers, Inc. (Beers) had previously (1961) been assigned the task of analyzing and predicting ground motion from underground nuclear explosions. Also, the U.S. Coast and Geodetic Survey's Special Projects Party had been responsible since 1961 for acquiring instrument data on ground and structural movement generated by such explosions. Other contractors were responsible for environmental monitoring, public health aspects, and other safety concerns not directly associated with the dynamic ground motion generated by underground

nuclear explosions. The U.S. Bureau of Mines was responsible for evaluating and monitoring effects of the NTS testing program on those mining activities within range of significant ground motion.

These contractors operated directly under the general supervision of DOE/NV, with an assigned DOE/NV technical management supervisor. DOE/NV also established scientific management centers which were assigned to various national laboratories and had responsibility for overseeing the scientific aspects of the contractors' work. Sandia National Laboratories (SNL) was responsible for scientific management of the ground motion and structural response contractors (Blume, Beers, and the Special Projects Party) and for Bureau of Mines work on mine safety aspects. Close cooperation was required between the DOE/NV technical managers, the SNL scientific managers, and the three contractors, so that the appropriate technology for predicting ground motion and structural response could be developed in a concerted manner.

The work of the ground motion and structural response contractors was reviewed by other boards, panels and committees, in particular by the Ground Shock Evaluation Subcommittee of the NTS Planning Board and the Panel of Safety Consultants. The Ground Shock Evaluation Subcommittee was staffed primarily by representatives of the national laboratories. The Panel of Safety Consultants was made up of distinguished senior scientists from various institutions, generally major universities. The Panel's reviews were usually concerned with some specific planned explosion with particular or unusual safety hazard aspects. Typically, such explosions included those at unprecedented yield levels and those in offsite areas with no previous exposure to ground motion from underground nuclear explosions.

Structural Response Program

At the outset of the Blume contractual studies the primary need was for a methodology which would permit accurate and reliable advance knowledge of the probable effects of any particular underground nuclear explosion. Fundamental to this methodology would be the ground motion prediction, in a format suited to the structural response prediction process. This prediction was to be furnished by the ground motion contractor, Roland F. Beers, Inc., (later the Environmental Research Corporation)(ERC). Both ERC and Blume needed reliable instrumental data on ground motion and structural response, with the necessary characteristics to be fully responsive to the specific needs of the contractors. The USC&GS Special Projects Party was assigned responsibility for providing such data.

Major specific needs for the structural response program included information on the detailed dynamic response characteristics of existing structures, the influence of nonstructural elements, internal stress

distribution and structural element interaction, stiffness and energy absorption characteristics, foundation influence, damage threshold, the effects of aging and of repeated motion, etc.. All pertinent knowledge and information was transferred from then-current earthquake engineering practice, but much more had to be obtained, as quickly as possible, from research and from monitoring effects of ground motion on existing structures.

Initial attention was focused on the Las Vegas high-rise buildings because of their amplified response to the NTS-generated ground motion. This amplification arose through a resonance phenomenon, i.e., the matching of fundamental response frequencies of the high-rise buildings with similar strongly-propagated frequencies of ground motion from large NTS tests. For the necessary development of a computer-based prediction methodology, it was essential to develop a reliable procedure for preparing mathematical models of these high-rise structures. Such modeling had to account correctly for the differences between real structures and the structural models in use for earthquake engineering. Modeling procedures also had to be verified by data on actual response behavior. Low-rise structures quickly became of equal importance because of their numbers and proximity to NTS. The great numbers and diversity of these structures made it impractical to develop individualized modeling and prediction methods. Instead a generalized statistical approach was employed to determine average response behavior.

For application of the structural response prediction procedures it was necessary to acquire data on the structural population to be affected by any particular test. This required development and application of survey procedures, varying from quick broad-scale reconnaissance surveys to lengthy detailed inventories of entire areas. The Las Vegas and NTS boundary area, subjected to repeated ground motion from nuclear weapons testing, required up-to-date knowledge of the current condition of major structures. Such knowledge was provided through condition surveys repeated at regular intervals.

Damage assessment and safety recommendations were provided for some tests, usually those at NTS at high yield levels or those in offsite areas. For such tests Blume observers would be stationed at a number of critical locations, such as roof levels of major Las Vegas structures or at low-rise structures in NTS border communities. Observers recorded motion perception, structural behavior, and the occurrence of any damage. Damage complaint investigations were subsequently conducted, as requested by DOE/NV, to provide an engineering evaluation of the damage attributed by the claimant to event ground motion.

Structural Response Investigations

The needed structural response prediction methodology required information and data not yet in existence. These were provided through a combination of field and laboratory studies. Field investigations included the establishment of a number of test structures on the NTS for specific testing programs, and regular monitoring of the dynamic response of existing structures on the NTS, in Las Vegas, and elsewhere. Of the new NTS structures the most important and useful were two 4-story reinforced concrete frames constructed in Area 1 in 1965 and used to test the influence of various non-structural partition materials. The performance of the bare-frame structures was also monitored for several years, including testing one into the damaging range and its subsequent repair and retesting. In Las Vegas existing and newly constructed high-rise structures were monitored in great detail, both for structural response information to be acquired and also for observation of any effects of the ground motion on these buildings. Field investigations were conducted for a number of damaging earthquakes occurring during the earlier years of the structural response program. These were usually directed toward acquiring data on the manifestation of seismic damage, especially at threshold levels, and on the participation of non-structural elements in the building response. Considerable statistical information on low-rise structure response was obtained from these investigations. A few large off-site chemical-explosion projects were similarly monitored, also for dynamic ground motion effects on low-rise structures. Laboratory studies were conducted to obtain data on the behavior of non-structural elements, to determine the levels of motion at which damage would be caused, and also to obtain data on the influence of such materials on overall building response.

Structural Response Prediction Methods

Three formal methods were developed for structural response predictions needs. These formal procedures included the Engineering Intensity Scale (EIS) for general evaluation of broad areas to indicate the incidence and degree of probable damage, the Structural Matrix Method (SMM) for more detailed indication of the extent and probable repair cost for the structural population in a particular area, and the Threshold Evaluation Procedure (TEP) for detailed prediction of response and damage onset in a given structure.

Inherent in and supplementary to the application of these formal procedures was the continual employment of experience and engineering judgement. In many instances, particularly during the early days, engineering judgement was the only practicable way to predict damage onset and severity so that safety procedures and remedial measures could be established. In such instances Blume's long experience in earthquake engineering was invaluable.

Seismic Data

Prediction methods developed by ERC and by Blume needed more and better seismic data than were available at the start of the structural response program. Initially the Special Projects Party used the USC&GS-1 system designed and used for earthquake work. This system did not have the capability to measure motion in periods of one second or longer. Other systems in use at the time (1964) included the portable Sprengnether blastmeter, used primarily for quarry blast monitoring; the CEC-strain gage system, used for measuring high acceleration levels; and the NGC-21 velocity meter system used to measure intermediate-level motion. A year or so later the EV300 and 301 systems were acquired specifically to measure long period motion. In 1967 the L-7 velocity meter system was acquired by DOE/NV. This system provided the necessary capability for data acquisition meeting the ground motion and structural response needs for many years. In the last decade the digital recorders such as the DCS301/302 and PDR-1 have provided a significant advantage over the L-7 system, i.e., the internal ability to make the essential conversion from analog signals into the digital data needed for computer input.

In 1979 the U.S. Geological Survey, which had assumed responsibility for seismic data acquisition from the USC&GS in 1973, relinquished that responsibility. DOE/NV assigned this task to Blume and the work has since been performed by Blume's Las Vegas staff. In 1983 DOE/NV also assigned ground motion prediction responsibility to Blume. Since that date Blume has had entire responsibility for all aspects of the ground motion and structural response safety and monitoring programs.

Special Studies

During the course of the structural response study program, a number of major studies had been carried out that were incidental, to some degree, to the basic program objectives. These were conducted either at DOE/NV's specific direction or with DOE/NV approval of a Blume proposal.

One such study was concerned with establishing motion-damage relationships for low-rise structures. Data from a number of NTS and offsite explosions were analyzed and statistical relationships defined for various motion components.

Another detailed study evaluated the modeling process to determine the validity of assumptions made as to structural characteristics, foundation influences, and structural partition contributions.

Off-NTS events required major structural response efforts primarily because of the circumstance of an

underground nuclear explosion in a previously untested area. The SALMON event of 1964 in Mississippi was, for Blume, the first of such efforts and was followed by a major investigation of the unexpectedly high occurrence of minor damage in the area. The GASBUGGY project in 1967 in northern New Mexico, the 1968 FAULTLESS event in central Nevada, and the 1969 RULISON project in Colorado required a great deal of Blume pre-shot structural inventory as well as substantial response prediction work and shot-time participation. For the 1973 RIO BLANCO project in central Colorado, the commercial sponsor assigned most of the structural response work to other contractors, and Blume had only a minor role. SALMON and RULISON were both followed by extended damage complaint investigation work.

During the latter half of the 1960's Blume personnel expended a great deal of office and field work for the proposed (but never initiated) construction of a nuclear-excavated sealevel canal across the Isthmus of Panama. This would have involved strong ground shaking and probable damage in Panama and Columbia. Lesser efforts were undertaken for a number of other proposed but never executed nuclear excavation projects, mostly in the western United States.

Blume conducted a geological and seismological investigation of the southern Nevada area in an effort to establish a comparison of hazards to the Las Vegas structures from natural earthquakes and from NTS events. A study of wind-generated motion, including wind force and building displacement measurements, was performed to determine relative effects of wind loading and event ground motion. Man-induced vibration studies were conducted by the USC&GS and Blume for a number of the Las Vegas high-rise structures.

Because of extensive earlier structural inventory work in central and southern Nevada, Blume was assigned by DOE/NV in 1977 to conduct a federally-required historic site survey. This study provided data on all known and potentially qualified historic sites within range of possible damage from NTS ground motion. Work was also done on seismic aspects of proposed nuclear waste repositories within the NTS boundaries.

Conclusions

The fundamental objective of the structural response program was to assist DOE/NV in continuing the safe conduct of the planned testing programs, under the dual circumstances of escalating test yields and booming high-rise construction in Las Vegas.

To meet this objective, Blume accomplished the following:

1. Provided initial structural response consultation which permitted safe and timely continuation of the planned test program at the Nevada Test Site, and for those in off-NTS areas.
2. Established and maintained good relationships with private owners of all major structures, to obtain essential structural data and to achieve access to such structures for instrumentation purposes and for ongoing condition surveys.
3. Conducted and verified structural response analyses of all major Las Vegas structures, to provide DOE/NV with accurate and reliable advance predictions of structure response, damage risk, and to conduct any necessary investigations required for continued safety.
4. Maintained a cost effective damage complaint investigation program to eliminate unfounded complaints and to restrict compensation to only justified damage claims.
5. Provided close communication with DOE/NV and internal review organizations on results of risk evaluations for forthcoming events and on major developments in structural response prediction technology.
6. Participated, as requested, in external public hearings and reviews of major testing programs.
7. Maintained close and harmonious cooperation with DOE/NV technical supervisors and with other ground motion effects contractors to achieve a coordinated development of ground motion and structural response prediction technology, defining ground motion characteristics and instrumental characteristics necessary for the development of a reliable and accurate structural response prediction technology.
8. Established and maintained a structural response laboratory to provide essential data on dynamic behavior of structural materials and elements.

9. Designed and operated a field test program with full-scale test structures to obtain essential data on response behavior of structural elements and full size structures.
10. Established and implemented necessary procedures for obtaining structural and instrumental data on all major LV structures and any others necessary for comprehensive, accurate, and reliable advanced predictions of structural response and damage from NTS and offsite events.

In meeting the basic objective of the structural response program, the Blume firm also achieved a number of significant engineering accomplishments. Most of these were inherent in satisfying the fundamental requirement to assist DOE/NV in the safe conduct of underground nuclear testing from the standpoint of controlling ground motion effects. Prediction methodology appropriate for the particular circumstances had to be developed to provide DOE/NV with reliable advance prediction of ground motion and its effects on structures. ERC and Blume worked closely together, under DOE/NV management, to provide these methodologies. In addition to the requisite engineering judgement for any application, formal methods which were devised included the Engineering Intensity Scale, the Structural Matrix Method and the Threshold Evaluation Procedure. These were employed at various times; most notably the Structural Matrix Method was used to predict quite accurately the damage repair costs of the RULISON explosion.

Development of these prediction procedures required acquisition of then-unavailable data on structural response behavior of non-structural building elements, foundation material influence, mathematical modeling of structures, statistical data on motion-damage relationships for buildings and building elements, and the effects of aging and previous inelastic behavior on subsequent response and damage occurrence. Most of these data were acquired from laboratory and field investigations conducted by Blume but earthquake research data was also reviewed continuously for applicable material.

In application of prediction technology it was also necessary to have information on the population of structures to be exposed to any particular ground motion. To gather these data structural inventories had to be conducted, using procedures developed by Blume so that information acquired could be used directly in the prediction procedure. The structural inventory information acquired for central and southern Nevada and for nearby locations in California also formed the basis for subsequent historic site surveys for DOE/NV, required by federal regulations for the safeguard of such historic locations.

Results of the Blume studies were documented primarily in formal reports to DOE/NV. These were normally made available to the public through the facilities of the National Technical Information Service (NTIS) in Springfield, Virginia and the U.S. Department of Energy's Office of Scientific and Technical Information at Oak Ridge, Tennessee.

In the period from 1964 to 1988 Blume published 126 reports. About 62 of these are available to the public through the NTIS. Most of the issued reports were preceded by preliminary versions, issued for technical and patent review purposes.

In addition to making structural response research findings available through reports for public distribution, Blume staff members also actively participated in technical conferences. Through 1975 about 70 technical papers and presentations on findings from the structural response program were made at various technical meetings, and in subsequent years another 30 or so for a total of about 100 in the period covered by this report. Most of these presentations provided reliable and hitherto unavailable data on the dynamic behavior of structural materials, elements, and entire buildings.

An equally large group of publications was issued by ERC, totaling almost 300 reports and technical journal publications. Other reports were issued by the Special Projects Party, and DOE/NV itself published a number of safety program reports and summaries of some of the off-site event findings. Those which are publicly-available are listed in the Appendices, and are also cited as references throughout this text in the form of a letter-number combination with the letter identifying the appropriate Appendix for the title, author, and date information.

1. INTRODUCTION

1.1 BACKGROUND

Early in the course of underground nuclear testing at the NTS, it was recognized at the Atomic Energy Commission's Nevada Operations Office that such testing generated earthquake-like ground motion. The off-site effects of such ground motion required careful consideration as part of DOE/NV's public safety program activities (E3,4,6 - see appendix E). This awareness was strongly enhanced on September 13, 1963 when an underground nuclear test, code-named BILBY, was conducted at the Nevada Test Site (NTS). BILBY was the largest such test, 249 kilotons, to that date at NTS (E10). About 30 seconds after the BILBY device was detonated, workmen on the structural framework of the partially-built Dunes Hotel and Casino high-rise structure in Las Vegas found themselves hanging on while the building swayed strongly in response to the just-arrived ground motion from the BILBY explosion. Because the testing program for the future included a number of BILBY-size or larger underground nuclear explosions and because a high rise building boom was currently underway in Las Vegas, it was clear that structural response aspects of the DOE/NV safety programs were going to require even more consideration.

The setting, the organizations, and the individuals who were important in the conduct of subsequent ground motion and structural response studies, and activities in the DOE/NV are described in this report. The time covered is from the inception of the Blume structural response program in January 1964 through the most active development and implementation stages for the response prediction technology, i.e., roughly the next 24 years. A great deal of detailed information is available in the many reports issued by DOE/NV and its contractors, listed in appendices A through E. References in the text of this report are taken from those appendices and identified by letter-number combinations. The letter identifies the relevant appendix and the number identifies the particular numbered item in that appendix.

1.1.1 Nevada Test Site

Under the U.S. Atomic Energy Act of 1946, the Atomic Energy Commission (AEC) was responsible for conducting experiments and doing research and development work relative to nuclear weapons, and for producing nuclear weapons. The AEC carried out these responsibilities on the basis of nuclear weapons requirements developed by the Department of Defense (DOD) in support of stated United

States policies and in accordance with prerequisite Presidential direction and approval.

In the years immediately following World War II, nuclear weapons tests were conducted at remote Pacific island sites. Testing at those distant sites required an extensive logistic effort and an inordinate amount of time. After review of known information about fallout, thermal, and blast effects, it was determined that, with careful control, a portion of an Air Force bombing and gunnery range in south central Nevada could be used for relatively low-yield nuclear detonations with assurance of public safety.

Originally, an area of 1,760 square kilometers (680 square miles) was withdrawn under Public Land Order 805 dated February 19, 1952 for nuclear testing purposes. This resulted in the formation of approximately the eastern half of the present Nevada Test Site (NTS). Additional land withdrawals in 1961 and 1964 added to the site, and, with the use of Pahute Mesa acquired by Memorandum of Agreement with the DOD in 1967, provide its present size of about 3,500 square kilometers (1,350 square miles).

The NTS is located in Nye County in southern Nevada, with its southernmost boundary about 100 kilometers (65 miles) northwest of Las Vegas as shown in Figure 1.1. The site is made up of federally owned land and varies from 46-56 km (28-35 miles) in width (east-west) and from 64-88 km (40-55 miles) in length (north-south).

1.1.2 Atomic Energy Commission Organization and Responsibilities

From 1952 until early 1962, all nuclear tests at the NTS took place under the management of the AEC's Albuquerque Operations Office. Because of the significantly increased activities resulting from the resumption of weapons testing in the fall of 1961, the Nevada Operations Office of the AEC was established in Las Vegas on March 6, 1962. This office was assigned the AEC's responsibilities for nuclear detonation programs at the NTS, as well as at all other United States test sites.

DOE/NV is headed by a Manager who reports directly to headquarters in Washington, D.C. At the time of the initiation of the Blume structural response program, the Manager was James E. Reeves. His successor in 1969 was Robert E. Miller, who was followed in 1972 by Mahlon E. Gates. Thomas R. Clark took over the post in 1983, and upon his retirement in 1987 the responsibility was assumed by the present Manager, Nick C. Aquilina.

FIGURE 1.1 SOUTHERN NEVADA AREA MAP

DOE/NV administers any nuclear explosive test site under DOE Headquarter authority, and provides a Test Controller who has immediate authority to control on-site and offsite safety conditions of any test. The test operation is performed by a test organization which includes scientific, technical, construction, logistics, and public safety elements. The design of each test is the responsibility of one of the sponsoring scientific organizations such as the national laboratories.

1.1.3 National Laboratory Role

Three laboratories were established by the AEC for the specific purpose of conducting research and development activities relating to nuclear weapons. Two nuclear weapons laboratories were the Los Alamos Scientific Laboratory (now Los Alamos National Laboratory)(LANL) at Los Alamos, New Mexico, and the Lawrence Livermore Laboratory (now Lawrence Livermore National Laboratory)(LLNL) at Livermore, California, both of which were operated by the University of California. The third was Sandia Corporation (now Sandia National Laboratories)(SNL) with major facilities at Albuquerque, New Mexico, and Livermore, California. At the time Sandia was operated by the Western Electric Company. Sandia's work was concerned with weapon systems research and development work, mainly related to non-nuclear portions of nuclear weapons systems.

1.1.4 Nevada Test Site Planning Board

The NTS Planning Board, made up of senior representatives of all organizations with substantial official interests in the use of the NTS and any other nuclear explosive sites, reviewed reports made by its subcommittees and made recommendations to the DOE/NV Manager on public safety matters as well as on budgeting, scheduling, and many other matters. One of the subcommittees of the NTS Planning Board was the Ground Shock Evaluation Subcommittee which reviewed predicted event ground motion and structural effects.

1.1.5 Safety Panel of Consultants

In 1957, a Panel of Safety Consultants was appointed to advise the DOE/NV Manager on certain public safety matters related to the effects of underground tests. Most of the original appointees were recommended by the National Academy of Sciences as experts in their fields of geophysics, hydrology, geology and other disciplines pertinent to safety aspects. During the early years of the Blume structural response contract the members of the Panel, their fields of expertise, and organizational affiliations included Dr. Perry Byerly, seismology, University of California at Berkeley; Dr. Don U. Deere, engineering geology, University of Illinois; Dr. Lydik S. Jacobsen, mechanical engineering, Stanford University; Dr. George B. Maxey, ground water hydrology, Desert Research Institute; Dr. Nathan N.

Newmark, structural engineering, University of Illinois; Mr. Thomas F. Thompson, engineering geology consultant; Mr. Lewis G. Von Lossberg, hydrogeology, Sheppard T. Powell & Associates; and Mr. Stanley D. Wilson, soil mechanics, Shannon & Wilson.

1.1.6 Scientific Management Centers

DOE/NV established Scientific Management Centers to provide technical coordination of those NV contractors assigned to study special effects.

For selected events, an Effects Evaluation Scientist was appointed from one of the Centers and a DOE/NV professional was also assigned. This team prepared a comprehensive effects evaluation prediction based upon output from each of the Centers. This report was then reviewed by 1) the Centers, for accuracy and completeness; 2) NTS Planning Board, particularly by its Ground Shock Evaluation Subcommittee; 3) Panel of Safety Consultants; 4) the Test Manager's staff; 5) a special panel of experts if unusual effects were possible; and 6) the DOE/NV Manager, and scientific advisors from the three National laboratories. For industry-sponsored events, e.g., Project RULISON (E5), another review was made by the DOE/NV Plowshare Advisory Group, comprising laboratory, industry, and Department of Interior representatives.

Much of the review and oversight responsibilities for the seismic safety program were assigned to SNL, which placed a number of its senior scientists in key roles. Dr. Wendell D. Weart of Sandia Corporation (later Sandia National Laboratories) was a member of the Ground Shock Evaluation Subcommittee when it was first formed in October 1963. Dr. Weart was appointed chairman of the Subcommittee in May 1966, and continued as chairman of its successor, the Ground Motion and Seismic Effects Committee, throughout the period of this report. Dr. Weart's degrees are in geology and mathematics from Cornell University and geophysics from the University of Wisconsin. He has been on the staff at Sandia since 1959.

SNL was assigned as the Scientific Management Center for ground motion, structural response, and airblast effects. Dr. John R. Banister of SNL was assigned as Scientific Manager for these programs in 1969. His previous assignment as DOE/NV Technical Safety Director for the 1968 FAULTLESS event (E10) had brought him into close association with the ground motion and structural response contractors. In addition to his continuing position as Scientific Manager throughout the major portion of this history, Dr. Banister also conducted personal investigations into wind loading on structures and on pore pressure and air pressure changes caused by ground motion from underground nuclear explosions. Dr.

Banister's degrees in physics are from Iowa State College. Dr. Banister, who retired from SNL in January 1990, had been on the staff since 1953.

Dr. Banister was assisted in his duties as Scientific Manager by Dr. D. Maxwell Ellett, Dr. Melvin L. Merritt, Jack W. Reed, and Dorris Tendall, all of SNL. Dr. Ellett was primarily concerned with the structural response investigations, and was closely involved with 3 particular areas of the structural response investigations: forced-vibration testing of 4-story concrete frame test structures at the NTS; instrumentation of very large chemical explosions for mineral ore body development programs, of interest because of the information on ground motion and structural response to be obtained thereby; and wind vibration measurement programs in Las Vegas, conducted to investigate structural response from natural causes (e.g. wind loading) as compared to that from underground nuclear explosions. Dr. Ellett was on the SNL staff from 1946, with interim absence for educational purposes from 1949 to 1952, until his retirement in April 1986. His degrees in mechanical engineering are from the University of Virginia and Yale University.

Dr. Melvin L. Merritt of SNL monitored the ground motion and ground instrumentation aspects for the Scientific Management Center. He was particularly concerned with effects evaluation aspects of site selection committee investigating test sites in the 1963 - 1964 period. He was assigned as Effects Evaluation Scientist for underground nuclear explosion (UNE) tests on Amchitka in the 1968 to 1977 period, and was also associated with several ecological and environmental study programs and the preparation of environmental impact statements associated with these studies. Before his retirement in 1988, Dr. Merritt had been on the SNL staff since 1950. His degrees are from the California Institute of Technology.

Jack W. Reed of Sandia provided consultation on a number of programs in which airblast was of particular interest. These included, in particular, studies during the 1960s on airblast propagation from large underground and cratering tests at the NTS. During the middle and late 1960s Mr. Reed was chairman of the Acoustic Wave Project under the Inter-Oceanic Canal Studies Commission. This project was concerned with distant damage potential from multi-megaton nuclear explosive cratering for a proposed sea-level canal through the Isthmus of Panama. Mr. Reed has a BS degree in Mathematics from the University of New Mexico and a MS (equivalent) degree in meteorology from the USAF during World War II. Prior to his retirement in 1989, Mr. Reed had been on the SNL staff since 1948.

Dorris Tendall worked in the field of seismic signal analysis, including yield estimation at intermediate

ranges and the influence of shot and station location on signal strength. Her degrees include a BS in mathematics from Central College, Pella, Iowa in 1946, and an MS in physics from Iowa State College in 1949. Before her retirement in 1979 she had been on the Sandia Staff since 1952.

1.1.7 Contractor Assignments

Various aspects of the effects of underground nuclear testing were assigned by DOE/NV to organizations with specialized capabilities in those aspects.

From the earliest days of contained underground testing, the possible effects of nuclear explosions on ground water had been a source of concern to the test organization and to the public. This problem was assigned to the U.S. Geological Survey, which established a permanent unit at the Nevada Test Site and provided backup from its offices in Denver. To assess possible transport of radionuclides in ground water DOE/NV obtained the services of Hazelton-Nuclear Sciences Corporation. The University of Nevada's Desert Research Institute also provided consulting services on underground water questions.

1.1.7.1 Special Projects Party

As test explosions increased in size in the early 1960s, the regional off-site effects of ground motion became an obvious potential problem. By 1961 the AEC test organization's prediction capability had been enhanced in two ways, providing for seismic instrumentation and for work on ground motion prediction. An expanded network of local and regional seismic recording stations had been started in 1958 through the personnel and facilities of the U.S. Coast and Geodetic Survey (USC&GS). Since 1931 the USC&GS had been responsible for the activities of the Seismological Field Survey (SFS) within the U.S. Department of Commerce. A Special Projects Party was established within the SFS for the specific purpose of acquiring ground shock data from underground nuclear detonations. Later assignments included earthquake and aftershock studies, and ground motion data from chemical explosions. These assignments were related to needed data for ground motion and structural response utilization.

The Special Projects Party was assigned by the USC&GS in 1961 to conduct required seismic instrumentation and data collection for NV. In subsequent organizational changes, the U.S. Coast and Geodetic Survey became part of the National Oceanic and Atmospheric Administration (NOAA) from 1971 to 1973. In 1973, with its transfer to the U.S. Geological Survey, SFS became Seismic Engineering, Office of Earthquake Research and Crustal Studies, U.S. Geological Survey (USGS), Department of the Interior. As part of the USGS, the Special Projects Party was assigned a broadened responsibili-

ty. However, the seismic instrumentation work for NV was still a major part of that responsibility. Consequently for identification purposes the title of Special Projects Party is used throughout this report for the organization responsible for DOE/NV seismic instrumentation from 1964 (the inception of the Blume contract) to 1979. In 1979, the USGS relinquished the responsibility for DOE/NV's seismic instrumentation work, which was then assigned by DOE/NV to Blume.

At the outset of the USC&GS Special Projects Party assignment in 1958, the party was headed by Mr. Thomas H. Pearce, Chief. Mr. Pearce had extensive experience, beginning in 1946, supervising the observance and recording of ground shock effects from underground nuclear detonations. Upon his retirement in August 1966 his position as Chief of the Special Projects Party was assumed by Mr. Kenneth W. King, geophysicist, initially as acting Chief and confirmed in 1968 as Chief. Mr. King was Chief of the Special Projects Party throughout the rapid expansion of the ground motion and structural response aspects of the DOE/NV offsite safety program beginning in 1964. In 1974, upon the transfer of the Special Projects Party to the U.S. Geological Survey, Dr. Walter W. Hays was designated as Principal Investigator for the DOE/NV project functions and Mr. King became Deputy Principal Investigator. Mr. King retained the operational responsibility for the DOE/NV functions of the Special Projects Party.

Richard Navarro, geophysicist, had major responsibilities for the operations of the Special Projects Party, especially in liaison with other ground motion and structural response contractors. The liaison effort became particularly important following the USGS withdrawal from the DOE/NV ground motion and structural response program in 1979 and subsequent transfer of the Special Projects Party responsibilities to Blume. Mr. Navarro was assigned by the USGS to temporary duty with DOE/NV to provide technical and administrative assistance in Blume's assumption of those responsibilities. Mr. Navarro was subsequently transferred permanently to DOE/NV with responsibility for contractual supervision of Blume's work for DOE/NV, among other duties. His background includes a mechanical engineering degree from the University of Texas and a degree in mathematics from Baylor, both in 1961. He worked for the National Geophysical Company before joining the USC&GS in 1963.

Don Calfee was assigned principal responsibilities for field engineering. Mr. Calfee's background included 2 years of academic work at the University of Nevada, Las Vegas. Upon the USGS withdrawal from DOE/NV safety program functions in 1979 Mr. Calfee transferred to Blume to assist with the assumption of technical and administrative responsibilities of Special Projects Party functions by Blume as well as liaison with the building representatives. Other Special Projects Party members who also

transferred to Blume for similar assistance in Blume's assumption of those function were Walter L. Jungblut, field engineer and Leroy J. Foote, Electronics Technician.

Other Special Projects Party personnel with major responsibilities in the data processing and analysis work included Stan Brockman and Al Rogers, geophysists. Data acquisition responsibilities were carried out by field personnel of the Special Projects Party, frequently under difficult schedule and weather conditions. The outstanding quality of performance under these conditions reflects great credit upon the management and personnel of the staff. In addition to those listed above, other major credit is due to Dee E. Overturf, Eugene D. Sembrera, and Gerri Wuollet.

1.1.7.2 Environmental Research Corporation

The firm of Roland F. Beers, Inc., of Alexandria, Virginia (later Environmental Research Corporation) (ERC), had been put under contract in 1961 to interpret the seismic recordings provided by the Special Projects Party, and to provide a predictive capability for ground motions from future tests at various distances. Dr. Roland F. Beers, President of Roland F. Beers, Inc. was a consultant in geophysics for many years prior to forming his firm in 1955. His degrees were from Rensselaer Polytechnic Institute and from the Massachusetts Institute of Technology. He served as a member and later as chairman of a panel of scientific advisors to the AEC's Albuquerque Operations Office of the U.S. Atomic Energy Commission on public safety aspects of underground nuclear detonations.

In 1966 Roland F. Beers, Inc. was acquired by Systems Sciences Corporation (later Computer Sciences Corporation) (CSC). Following Dr. Beers' subsequent retirement the firm of Roland F. Beers, Inc was retitled Environmental Research Corporation (ERC). In 1970 ERC established a branch office in Las Vegas at about the same time as the opening of a Blume branch office also in Las Vegas. These actions were intended to provide maximum liaison between these two contractors, the Special Projects Party and the DOE/NV and Sandia technical supervisors.

Reductions in DOE/NV's safety program requirements took place following the major off-site effects required project RULISON in 1969 and for the HANDLEY, a megaton-yield event in 1970 (last of a series of high-yield NTS events). Corresponding reductions took place in contractor staffing. Despite this reduced staffing ERC research and data analysis had reached a point in 1974 at which an early version of a reliable ground motion prediction manual could be issued. In 1975 a resumption of high-yield testing at NTS accompanied by increased contractor staffing provided a great deal of excellent ground motion data to incorporate into ground motion prediction verification efforts. Subsequent

improvements in ERC's ground motion prediction parameters were published. With the completion of this fundamental objective, ERC staffing was again reduced and in 1976 the company was incorporated into the Las Vegas arm of its parent company, CSC, losing its identification as the ERC. In 1979 the small remaining staff was taken over by Adaptronics Inc, of McLean Virginia. Adaptronics was already supervising the work under subcontract to CSC. Finally, in 1983 the contract function was transferred to Blume.

Prior to the establishment of the ERC branch office in Las Vegas in 1970, major DOE/NV program assignments in ERC's Alexandria Virginia office included those to geophysicist L. B. Luhrs; physicists C. F. Casey, C. R. Cassity, and P. C. Loux; and earth scientists R. H. Barry, L. L. Davis, W. W. Hays and B.G. Weetman.

After 1970, DOE/NV program work was accomplished primarily in ERC's Las Vegas office. Staffing was accomplished largely by personnel transfer from Alexandria to Las Vegas, including several of the above-listed Alexandria office staff. Until his transfer to the USGS in 1973, Dr. W. Hays was responsible for major administrative and technical aspects of ERC's work in Las Vegas for DOE/NV. Dr. Hays' background includes a BS degree in geology from Southern Methodist University, and MA and PhD degrees from Washington University, St. Louis, in 1959 and 1961.

J.A. Lahoud, who had BS and MS degrees in geology from Rensselaer Polytechnic Institute, was responsible for much of the geological and geophysical aspects of ERC's work for DOE/NV. Mr. Lahoud assumed management responsibilities for ERC's contract work for DOE/NV upon its takeover by CSC, its parent corporation in 1976.

J. R. Murphy, a geophysicist and research seismologist for ERC, had BS and MS degrees from Boston College. Mr. Murphy conducted research and published extensively on the nature of UNE-generated seismic waves and their modification by local geology and by source characteristics.

R. A. Mueller, also played an active role in the study of seismic wave phenomenology, especially in the modifications of elastic waves by source and receiving station characteristics.

R. D. Lynch, an applied research geophysicist with a BS in physics from Wittenberg University, accomplished much of the research study leading to the development of a ground motion prediction technology by ERC. Mr. Lynch published extensively on his findings.

L. J. O'Brian, mathematician and statistician from the University of Maryland was responsible for major accomplishments in the acquisition and utilization of ground motion data. His research was a key element in the development of a reliable and accurate ground motion prediction technology.

1.1.7.3 John A. Blume & Associates, Research Division

Ground motion from an underground test was first felt in Las Vegas in September 1963, generated by the BILBY event at NTS. At that time the question of how to translate the ground motion prediction into possible effects on buildings was still unresolved. To obtain better information, the DOE/NV in early 1964 employed John A. Blume and Associates, Engineers, of San Francisco, a firm long familiar with earthquake effects on structures and experienced in designing structures to minimize earthquake damage. The Blume firm was retained to interrelate ground motion predictions and the probable effects of such ground motion on man-made structures. The contract was assigned to the Research Division of the Blume company.

From the inception of the DOE/NV contract work in 1964 until the establishment of a Blume branch office in Las Vegas in 1970, essentially all work from DOE/NV was accomplished in the San Francisco office. Exceptions included frequent field operations involving as many as 20 or more staff personnel, and primarily involving participation in data collection and observation of structural behavior in areas adjacent to large underground nuclear detonations; and a small staff of field personnel resident in Las Vegas and assigned to acquisition of the structural data for high rise buildings. This group was headed by E. C. Arnold who had a great deal of experience in field operations and damage claim investigation. The field group was primarily concerned with acquisition of structural data on new high rise construction, monitoring conditions of existing structures, and with active involvement in field collection for communities bordering the NTS.

DOE/NV project work was roughly divided into two approximately equal areas; the essential research program directed towards achieving an accurate and reliable structural response prediction technology, and the equally essential program of data acquisition and support activity, both for the research effort itself and for the parallel operational aspects of participation in DOE/NV safety programs.

A typical Blume organization chart for about 1970 is shown in Figure 1.2, illustrating the division of the DOE/NV project work into the two major areas described above.

The fundamental technical requirements for the Blume work lay in the field of structural dynamics;

FIGURE 1.2 1970 BLUME ORGANIZATION CHART

specifically in the response of buildings to ground motion generated by underground nuclear explosions. Basic similarities to the existing earthquake field permitted considerable technology transfer from that field, but the differences required a great deal of field, office, and laboratory investigation.

For DOE/NV needs, the interaction and behavior of both structural and non-structural building elements needed to be understood and quantified. The influence of foundation materials was of great importance in the overall response of structures. Instrumental capabilities and limitations in the acquisition and processing of data were critical, and the development of appropriate computer capabilities both for machines and programming was essential to providing adequate structural response prediction capability. Understanding mathematical and probabilistic aspects of the extensive and complicated process involved was also fundamental.

All the information being developed had to be appropriately stored, transferred to reports and papers for the understanding of DOE/NV and National Laboratory personnel and for the technical community. All the work required administrative management in accordance with internal and external standards.

Disciplines and capabilities included civil, structural, and architectural engineering, particularly in the field of structural dynamics; earth sciences, again in the area of dynamic characteristics of earth material and earth structure; materials science; laboratory testing; data acquisition and analysis, and processing, both from the standpoint of the instrumentation capabilities required for the nature and quality of the data needed and from that of the machinery and programming required for its analysis and manipulation into usable information and prediction procedures; and the administrative requirements for project management and control liaison with DOE/NV and other contractors, reporting requirements, and interaction with public agencies and with the public itself, especially in the fields of damage complaint investigation and occasionally at public hearings for major and sometimes controversial projects.

Technical work divided into two major areas; one was a long range research program to provide missing information needed for the development of an accurate and reliable response prediction technology necessary for the detailed needs of the DOE/NV safety program.

An equal and frequently predominating requirement was for participation in DOE/NV event and project programs, on and off the NTS. Both weapons and Plowshare programs were active during the 1960's and early 1970's and requirements for Blume participation often required interruption of other work,

such as long range research.

The project management philosophy which arose from both DOE/NV requirements and from general project needs is illustrated by the preceding organization charts. Dr. Blume's utilization was in two general areas; for establishing general project guidelines and objectives, and for his personal involvement in research aspects, problem areas, and in response for specific DOE/NV requests. It was frequently necessary during the early phase of the DOE/NV project to make use of Dr. Blume's judgement, intuition, and experience in solving difficult structural response problems.

Dr. John A. Blume, President of John A. Blume and Associates, Engineers, received an A.B in engineering in 1932; a Civil Engineering degree in 1934, with a thesis on the structural dynamics of multistory buildings; and a Ph.D. in 1966, with a thesis titled "The Dynamic Behavior of Multistory Buildings with Various Stiffness Characteristics". All degrees were from Stanford University.

In 1945, Dr. Blume founded John A. Blume & Associates, Engineers. The firm was merged in 1970 into the URS Corporation. Prior to 1945, Dr. Blume had been in the employ of Stanford University; the USC&GS, Seismological Division; the State of California Division of Highways on construction of the San Francisco-Oakland Bay Bridge; Standard Oil Company of California on design and supervision of construction; and H. J. Brunnier, as engineer in charge of design of large and complex military facilities and structures.

As shown in Figure 1.2, Dr. R.E. Scholl headed research management which included research, structural engineering, and computer studies. Dr. Scholl's background included a BS in civil engineering from the University of North Dakota in 1960, an MS in structural engineering, also from University of North Dakota, in 1963, and a PhD in Structural Engineering from the University of Oregon in 1969.

Administration, operation, and overall project administration operations were headed by R. E. Skjei. Mr. Skjei's background included a BS in engineering science from the University of California at Berkeley (UCB), in 1958 and an MS in geological engineering, also from UCB, in 1959.

In the middle 1960's a structural dynamics research laboratory was established in San Francisco to provide essential data in the participation of non-structural elements in the overall response of structures. Also, 2 4-story concrete frame test structures were constructed at NTS and heavily instrumented to provide needed detailed data on the response of the bare-framed structures and the response with

various non-structural partitions, to both NTS-event ground motion and to forced vibration by other means. In addition a field laboratory vehicle was acquired and outfitted to obtain data on the response behavior of earth material and earth structures. The field unit and a small group of earth scientists was headed by Mr. F. R. Conwell, a senior engineering geologist.

During the 1970's the previous intense demands imposed by the escalation of weapons test yields and simultaneous activity in off-site testing gave way to broadening necessity to complete building analyses backlogs created by the Las Vegas building boom. Still later, Blume assumed responsibility for seismic data acquisition and ground motion prediction activities previously assigned to the Special Projects Party.

During this period, K. K. Honda, who had previously been responsible for much of the high-rise research studies and for liaison with the Special Projects Party, took over increasing responsibility for overall project management. By the late-1970's he was fully responsible for overall project management, assisted by ongoing technical and administrative consultation by Dr. Blume and Mr. Skjei. Mr. Honda, whose background included a BS in civil engineering from San Jose State College in 1967 and an MS in structural engineering, also from San Jose State, in 1967, made particular contributions in the area of high-rise response and with the improvement of the quality of response data for structural response analysis purposes.

Structural engineering aspects of the research program were assigned to Mr. S. A. Freeman. This work consisted of investigations into the overall aspects of structural response, including the important contributions of non-structural elements in initial levels of response behavior. Mr. Freeman was also responsible for contributions in the area of mathematical modeling of structures, and for particularly important developments in the technique of rapid evaluation of structural response for large groups of buildings. Mr. Freeman had a BS in civil engineering and an MS in structural engineering, both from the University of California, Berkeley.

Mr. Monroe was responsible in 2 areas of research; assisting Dr. Blume in the early development of structural response prediction technology and for the development of computer software for structural response prediction procedures. Mr. Monroe whose degrees were in civil and structural engineering was also assigned to manage the newly-established Blume branch office in Las Vegas in 1970.

The Las Vegas branch office expanded in the early 1970's to include a major share of the personnel

assigned to the DOE/NV project. The growth was in accordance with current DOE/NV desires to have most of the DOE/NV safety program studies in ground motion and structural response conducted in the Las Vegas area. This expansion included staff additions, both by transfer from the San Francisco office and by new hires. Among the transferred personnel from San Francisco were Mr. Roger Skjei, who assumed managership of the Las Vegas office; Dr. R. E. Scholl, Dr. J. D. Raggett and Dr. J. W. Reed, Dr. Raggett's contributions included simplified analytical procedures for high-rise building response.

Dr. J. W. Reed made particular contributions to the understanding of probabilistic aspects of structural responses. Dr. Reed also assumed managership of the Las Vegas office following a staff reduction in the mid-1970's with relocation of most of the surplus staff back to San Francisco. Mr. R. M. Czarnecki joined the Las Vegas office staff in 1973 and took a particularly active role in a series of non-structural and destructive test of the 4-story test structure at NTS. Mr. Czarnecki's background included a BS in civil engineering from the University of Massachusetts in 1971 and an MS in structural engineering from the Massachusetts Institute of Technology in 1973.

From the inception of the Blume contract in 1964 until the early 1970's an active and at times intense field effort was under way almost continuously. These efforts included participation in the Vela Uniform SALMON project, the 1966 and 1967 related to the proposed sea-level canal excavation in Panama, the 1967 GASBUGGY test in New Mexico, the 1968 FAULTLESS event in central Nevada, and the 1969 RULISON project in Colorado. Earthquake investigations in the western U.S., in Alaska and in Colombia were also undertaken during this period. Many of the field projects were conducted under the direction of L. A. Lee and staff personnel from the project staff in San Francisco. Mr. Lee's background included a BS in civil engineering from the University of California in 1951 and extensive field and office experience in design and construction of steel-framed structures.

Other support was provided by Mr. W. H. Nelson whose architectural background, which included a BFA in architecture from the University of Utah in 1954, provided additional insight into the structural response prediction problem. The Blume field unit was usually involved also, providing the necessary information on earth materials, seismicity (seismic background), and other engineering geology questions.

Most of this kind of field activity came to an end by 1970 except for some Blume participation in the RIO BLANCO project in 1973, largely confined to work done by Mr. Lee in relation to structural

bracing of a tall steel ore-processing structure, and an active involvement in earthquake studies following the 1971 San Fernando earthquake. Subsequent field effort was confined largely to southern Nevada and Las Vegas areas, and was related primarily to acquisition of data on new structures. A flurry of activity was created in 1975 by an accelerated testing program at the NTS, requiring some heavy participation by other elements of the San Francisco office. In the later 1970's and with the advent of yield limitations on nuclear testing in 1976 the work of the organization was directed primarily at monitoring new construction activity in Las Vegas and monitoring data acquisition in ongoing low level testing. The Las Vegas branch office managership was taken over by Mr. Gerald Kralik from 1976 to 1982 and by Mr. Joseph Woodruff from 1982 to present. Recent key staff members in Las Vegas include Paul Lum, Donald Calfee, Judith Dickinson, Bruce Bradley, Leroy Foote and Philip Robarts.

Broadened responsibilities were assumed with the transfer of seismic data acquisition and processing work to Blume from the Special Projects Party in 1975 and of the ground motion prediction was responsibility from ERC in 1983.

In recent years the key San Francisco office staff included Dr. Russell Mills, Dr. Takeru Igusa, Dr. Richard Larder, Lunming Yuen and Peter Lum.

During the course of the DOE/NV program work, and especially during the periods of intense activity in the 1960's and 1970's, it was frequently necessary for the assigned Blume staff to be augmented by the personnel and capabilities of other elements of the Blume firm. The DOE/NV work was assigned to the research division of Blume, and assigned personnel usually made up about one-fourth of the total Blume staff. Another major organization element was made up by the Engineering Division, which constituted a large group of experienced civil and structural engineers. Major elements of the DOE/NV program work, e.g. the extensive seismic upgrading of structures at the Nuclear Rocket Development Station (NRDS) at NTS, were assigned directly to the Engineering Division via the existing Blume contract with the Research Division. Mr. J. P. Nicoletti, then a Blume vice president and head of the engineering division, was deeply involved in such programs. Dr. J. P. Jhaveri and Mr. R. T. Yokoyama also made important personal contributions.

These aspects of the Blume work are describe in more detail in the following chapters and are summarized here primarily for background on the Blume personnel who were active in the program and their respective responsibilities at various project stages.

1.2 INITIATION OF THE STRUCTURAL RESPONSE PROGRAM

The most accurate historical perception of the structural response program at the outset of the Blume contract is presented by the following extract from a statement prepared by Dr. John A. Blume in 1964 at the request of the Ground Shock Evaluation Subcommittee of the NTS Planning Board.

1.2.1 General Objectives and Need

The basic long term objective of the structural response program is to develop reliable means of predicting structural response to, and damage if any from ground motion induced by underground nuclear or chemical explosions. The prediction techniques and data should be as simple as possible consistent with satisfactory results. Other objectives and functions are to predict, to the extent possible in the current state of knowledge prior to the completion of the effort, the effects of certain events and to provide for any necessary temporary bracing. There are also certain other services to be provided as required from time to time including consultation, reviews, surveys, services in connection with damage claims, etc.

The program as developed has several phases or divisions of operations, most of which can be conducted simultaneously and independently with coordination and cross-feeding of current information. The principal divisions are: integration into the program of all applicable knowledge from other fields including earthquake engineering; data from air blasts; data from prior underground explosions--military, commercial, and research; structural dynamic analyses aided as indicated by computer programs and electric analog procedures; field testing of certain elements, models and small structures; certain laboratory testing of models and elements to obtain needed data; the obtaining, analysis and interpretation of records of the response of structures to ground motion induced by underground explosion; gathering of needed data on human perception of long period motion and also on damage thresholds; and the integration, coordination, documentation and utilization of the data obtained and the results of the effort.

A pressing short term objective is the determination of reliable damage thresholds for the Las Vegas high-rise buildings. Although this problem has been given the

highest priority since it was put into this response program, the results are not yet conclusive because of the lack of 1964 Las Vegas ground motion associated with adequate instrumentation for events of yield in the range of 40 to 200 or more KT. There is a vital need for adequate structural response instrumentation during important events to obtain empirical response data necessary in the program.

Another requirement is to obtain measured response motion for certain Area 400, NRDS installations, during events of increasing yield so as to integrate the actual response with analyses for the proper safety control of that facility.

A by-product of both the Las Vegas and Area 400 work is the accumulation of data useful in the implementation of the basic long range objective of the structural response program.

Unfortunately, the peak demand for instrumentation for both the ground motion program (Roland F. Beers, Inc.) and the structural response program (John A. Blume & Associates Research Division) occurs at the same time - during major events. In spite of this, the paucity of adequately measured response to significant events and of Las Vegas ground motion, together with the need for such measurements to determine Las Vegas safety thresholds without serious problems, demand that both events and instruments be provided prior to causing ground motion in Las Vegas significantly greater than that resulting from BILBY.

1.2.2 The Need for Empirical Data

It is not considered practical to carry out the structural response program entirely on an empirical basis. There are so many parameters involved that only a great many events in many different areas and with a vast number of types and sizes of structures and types of soil, and of combinations of soil and structure, would yield adequate data. On the other hand, a considerable amount and range of empirical data are essential for the proper implementation of the program and to insure reliability of predictions. To extrapolate the results of minor motion on a few structures into a potentially damaging range without check of scaling factors and many other important parameters could lead to very serious consequences and is not recommended. Instead, and to conduct the program as rapidly as feasible, a

balanced program of adaptation of applicable other knowledge, analysis, field and laboratory testing, and obtaining adequate empirical data is programmed. The scope is not elaborate, conservative, or unreasonable; it embraces the practical utilization of presently available and yet-to-be-obtained data.

There is considerable information available from many years of effort by various persons and agencies in the field of earthquake engineering. Much of this is useful in the structural response effort. However, there are important differences between earthquake response and response to man-induced ground motion that make reliance solely upon earthquake data impractical and unwise. The degree of accuracy necessary to or inherent in the earthquake field is not such as to be adequate for this program or for successful future commercial applications of nuclear explosive energy. No one can be held responsible for an earthquake and for what it might do to the property of others. However, man-made ground motions carry legal, financial, and moral responsibilities. Although there is little room for error, and none for negligence, conservatism could unduly curtail the effective uses of atomic energy. The indicated procedure, therefore, is to conduct certain studies and to gather all possible empirical and test data to provide essential input data and to fill certain gaps in earthquake knowledge to the extent indicated for the welfare of the atomic energy program.

The data available from blast research are also useful, but they have the same type of limitations as the earthquake material. Military and protective design, because of its inherent nature, does not need nor does it provide the accuracy needed for the protection of property in peacetime.

1.2.3 Some of the Gaps in Knowledge

The earthquake literature is replete with photographs and observations of damage, idealized analytical procedures, computer and analog programs, and design techniques and procedures. It also provides some ground motion records and response spectra. The writer is familiar with what has been published and most of the research work in process. However, with 30 years activity in this field the writer knows the limitations in the data and the many assumptions that must be made as to the characteristics of the structures and of future ground motion. The gaps in

earthquake knowledge are not a reflection on the workers in the field, many of whom have worked on their own time, but an indication of the lack of research funds over the years; inability to predict the time, place or scale of earthquakes; and the facts that accuracy is not a basic factor in this field, for the reasons noted above.

The basic need is not for analysis procedures, programs, routines or techniques, but for reliable numbers to use as input and also for the evaluation of certain parameters, many of which are interacting. Some examples of needed numerical data follow:

- (1) Periods of vibration and amplitudes (in acceleration, velocity or displacement as indicated) over a range of yield or ground motion measured at two or more locations in tall building superstructures, at the basement level, and on the "free surface" ground away from building influence. Simultaneous time marks are generally needed. Objective: to determine the dynamic characteristics, mode shapes, shears and unit stresses, scaling factors, etc.*
- (2) Damping values, over a range of amplitude.*
- (3) The interaction of walls and frames in typical buildings.*
- (4) The interaction and relative participation of structural and nonstructural materials and elements up to the point of failure.*
- (5) Interaction phenomena, including period variation and energy feedback, between structure and soil.*
- (6) Data on the effective stiffnesses and energy absorption characteristics of various (typical) assemblies of materials, elements, and framing.*
- (7) Scaling effects on response periods, duration of motion, amplitudes, thresholds of sensation and damage, with variations in site materials, shot area*

materials, yield, distance, etc.

Structural analysis and dynamic analysis will, of course, be utilized to the greatest reasonable extent. Computer programs are developed, or available, for use in this work. But reliable empirical data on the above are needed for input - the program can not be based upon hypothetical or highly idealized systems with assumed numerical values in certain areas, such as noted above.

Dr. John Blume's long involvement in the earthquake engineering field and his pioneering research in structural dynamics made him particularly well qualified to discern the limitations of the existing knowledge and the capabilities of that field to fill the needs of DOE/NV's structural response program. His 1964 statement, therefore, was a good prediction of the work undertaken over next 20 years to provide the technology which was essential to the safe conduct of DOE/NV's underground nuclear explosion programs, at the NTS and elsewhere.

The foregoing assessment and description of needed data was directed specifically at engineered structures, and particularly high-rise buildings. These were correctly perceived to constitute the most immediate and critical problems in Las Vegas in 1964. However, as the program developed, the problem of structural response in low-rise unengineered structures emerged with equal importance. Solutions to the low-rise problem required more judgmental and statistical approaches, although much of the research conducted on materials and elements for the high-rise structures turned out to be of great value in the solution to low-rise structural response problems.

2. OVERVIEW OF THE STRUCTURAL RESPONSE PROGRAM

2.1 SCOPE OF THE STRUCTURAL RESPONSE PROGRAM

The purpose of the structural response program was to provide advice to the DOE/NV on safety aspects of underground nuclear explosion (UNE) tests conducted at the Nevada Test Site (NTS) which might create damaging forces on structures in off-site areas. Because any damage to off-site structures was to be kept within safe limits, the primary DOE/NV need and Blume challenge was to predict the onset of threshold damage, i.e., initial minor damage.

At the outset of the work, heavy reliance was placed on the engineering knowledge and technical skills from the earthquake engineering field, necessarily so because the specialized predictive capabilities required for the DOE/NV needs did not exist. Formidable complexities arose from the need for reliable advance knowledge of the probable effects on a large number of structures to be exposed to such forces. The current status of earthquake engineering capabilities did not provide adequate understanding of seismic motion characteristics, structural response or response-damage relationships. Furthermore, while safety of the public remained the primary consideration, financial and public reaction aspects of potential damage to structures were also important, and needed to be understood and accommodated.

Since the creation of the structural response program in 1964, Blume has been involved in more than 300 underground nuclear explosions, or "events." For programmatic reasons, these events can be categorized as follows: (1) NTS weapons tests, (2) off-site weapons tests, (3) Vela Uniform, and (4) Plowshare. Major events; distinguished as those which posed significant structural response, damage, and remedial measure issues; are listed in chronological order in Table 2.1 (E10).

The weapons testing programs conducted each fiscal year are given identifying names. For instance, Operation NIBLICK was the title for the test series during Fiscal Year 1964 (July 1963 through June 1964). Individual tests are also given names, e.g., BILBY. These names usually have no significance other than as a means of identification, except that some off-site tests were named for local communities. DOE/NV-209 (E10), a publication annually updated by DOE/NV, provides unclassified information on tests and test statistics.

TABLE 2.1 LIST OF MAJOR UNE EVENTS IN STRUCTURAL RESPONSE PROGRAM

Period	NTS Events		Off NTS Events		
	Weapons	Date	Program	Event	Date
7/63 to 6/64	BILBY (249 kt)	9/13/63	Vela Uniform	SHOAL (12 kt)	10/26/63
7/64 to 6/65			Vela Uniform	SALMON (5.3 kt)	10/22/64
7/65 to 6/66	HALFBEAK (365 kt)	6/30/66			
7/66 to 6/67	GREELEY (870 kt)	12/20/66	Vela Uniform	STERLING (0.38 kt)	12/03/66
7/67 to 6/68	BOXCAR (1,300 kt)	4/26/68	Plowshare	GASBUGGY (29 kt)	12/10/67
			Weapons Related	FAULTLESS***	1/19/68
7/68 to 6/69	BENHAM (1,150 kt)	12/19/68			
7/69 to 6/70	JORUM (< 1,000 kt)	9/16/69	Plowshare	RULISON (40 kt)	9/10/69
	HANDLEY (> 1,000 kt)	3/26/70	Weapons Related	MILROW (~ 1 MT)	10/02/69
7/70 to 6/71	CARPETBAG (220 kt)	12/17/70			
7/71 to 6/72			Weapons Related	CANNIKIN (< 5 MT)	11/06/71
7/72 to 6/73	ALMENDRO***	6/06/73	Plowshare	RIO BLANCO (3 @ 33 kt)	5/17/73
7/73 to 6/74					
7/74 to 6/75	TYBO***	5/14/75	Plowshare Plowshare	WAGON WHEEL* WASP*	
	MAST***	6/19/75			
	CAMEMBERT***	6/26/75			
7/75 to 9/76	KASSERI***	10/28/75			
	INLET***	11/20/75			
	MUENSTER***	1/03/76			
	FONTINA***	2/12/76			
	CHESHIRE****	2/14/76			
	ESTUARY*****	3/09/76			
	COLBY*****	3/14/76			
	POOL****	3/17/76			
	STRAIT****	3/17/76**			

* Event planned and structural response evaluation made, but UNE not detonated.

** On March 31, 1976, the Soviet Union and the United States agreed to limit the maximum yield of underground tests to 150 kt.

*** Yield Range of 200 to 1,000 kt.

**** Yield Range of 200 to 500 kt.

***** Yield Range of 500 to 1,000 kt.

Weapons tests, both NTS and off-site, are events that were detonated in connection with the U.S. nuclear weapons development program. The Vela Uniform Program, sponsored by the DOD, was designed to detect, identify, and locate underground nuclear explosions. During the middle and late 1960's, a number of Plowshare projects involving the use of nuclear explosives for peaceful purposes, were under active consideration, some for off-NTS areas. The off-site projects were generally of two categories; one involving cratering explosions for earth-moving purposes and the other involving more deeply buried explosions for the enhancement of mineral production through fracturing of the host rock. Several cratering events were fired at NTS, and 3 gas stimulation experiments, discussed in Section 6.1.3, were conducted in the western United States.

In addition to the Blume structural response contract, the DOE/NV Off-Site Safety program included a ground motion contractor and an instrumental data acquisition contractor (Figure 2.1). Blume worked closely with these two contractors to achieve safety program objectives, i.e., to predict the response of and any damage to structures, and to recommend any remedial measures that should be undertaken to minimize the probability of damage and any accompanying hazard to the public. Although Blume was responsible for only one of three major parts of the DOE/NV Off-Site Safety Program in 1964, the work was sufficiently inter-related to require Blume's substantial involvement in the other two contractors' work. These involvements are described in later sections of this Chapter. Since 1983 Blume has been responsible for all three major functional activities illustrated in Figure 2.1, namely, ground motion, instrumental data acquisition, and structural response.

2.2 FACTORS THAT SHAPED THE STRUCTURAL RESPONSE PROGRAM

Two factors that significantly affected the development, scope, form, and schedule of activities that ultimately composed the structural response program were the limitations in damage prediction capabilities in 1964, and the planned rapid escalation of event yield size for future underground nuclear explosion tests. Damage prediction in connection with structural response to dynamic ground motion was at best a poorly developed science in 1964.

Naturally-occurring earthquakes produce dynamic ground motion that has many similarities to the ground motion generated by underground nuclear explosions. Techniques available from earthquake engineering were therefore useful for predicting response to UNE. In the field of earthquake engineering, however, researchers and practicing engineers were concerned with ensuring that buildings did

FIGURE 2.1 DOE/NV OFF-SITE GROUND MOTION SAFETY PROGRAM IN 1964

not collapse during future earthquakes, and very few were concerned with predicting damage. Predicting the onset of damage in connection with earthquakes was not even being seriously considered in 1964. Only very few earthquake ground motion recordings were available at the time, and therefore little could be done in connection with correlating earthquake ground motion and damage.

Some early studies had been conducted to establish seismic damage thresholds. In the 1940's and 1950's the U.S. Bureau of Mines had conducted field tests to establish ground motion limits for the onset of damage to low-rise buildings in connection with quarry blasting. From this work it was concluded that damage to such buildings was not expected if the peak ground acceleration was less than 0.1g. For the 5.3 kt SALMON event in Mississippi in 1964 this criterion was employed, to predict possible damage out to about 4½ miles from shot point. Recorded SALMON ground motion 20 miles away in Hattiesburg, Mississippi was substantially less than the 0.1g criterion. However, minor damage in this nearest large community was wide-spread (see Section 6.4.1).

While the need for conducting applied research aimed at developing reliable damage prediction procedures was recognized by both Blume and by the AEC at the outset of the structural response program, the 1964 SALMON event dramatically emphasized this need. Research studies subsequently became a substantial part of the Blume structural response program.

Research was needed to develop reliable damage prediction procedures, especially because testing programs called for increasing UNE yields. The BILBY event in 1963, which caused the high-rise buildings in Las Vegas to sway, had a 249 kiloton yield and events of greater yield were included in future test schedules. By 1968, UNEs of over 1,000 kilotons had been conducted (E10), but by that time the prediction technology was sufficiently developed to permit testing at that level of energy with assured safety.

Considering the poorly developed state of the technology for both ground motion and damage prediction that existed in 1964, and the planned increase in the yields of future UNE events, it was essential that the Blume structural response program be tailored to identify critical parameters, obtain data where needed, and develop reliable procedures for predicting damage to structures. Undoubtedly, the most challenging feature of the structural response program was that of having to make damage assessments for planned events, and at the same time conduct research investigations aimed at improving damage prediction technology.

2.2.1 Operational Scope of Structural Response Evaluations

The Blume program was responsible for making response evaluations for all types of structures, natural and man-made. Throughout the course of the program, high-rise and low-rise buildings were generally most important, but other types of structures received significant attention as well. For example, dams and rockslides were important issues for the RULISON event (A33, A34), an oil shale processing structure required special attention for RIO BLANCO (A45), and Wyoming bridges near Pinedale were studied extensively for the planned (but never executed) WAGON WHEEL and WASP projects.

Although the available technology in 1964 was limited, structural response evaluations had to be made with specificity, including expected structure response motion, the possibility of damage, the location and severity of any such damage in terms of risk to occupants and to structures involved, and any remedial measures to be undertaken. All this had to be predicted reliably for a substantial inventory of buildings and other structures. In addition, these buildings and other structures typically represented a wide spectrum of variations in age, quality of construction, extent of engineering design, condition, and foundation materials. Following are overview discussions of the manner in which high-rise and low-rise buildings were dealt with for making response evaluations as well as an elucidation of how these challenges shaped the research investigations conducted in connection with the program.

High-rise Buildings. Initially the program focus was primarily on Las Vegas high-rise buildings because it was the growing response of these buildings to ground motion from NTS which underscored the need for a structural response program (E10). In the Blume structural response program, mathematical models of buildings were developed and used to perform computer simulation evaluations of the response of buildings to dynamic ground motion (A12, A13, A16). The basic concepts of such modeling of simple idealized buildings had been developed in the earthquake engineering field prior to 1964, but modeling real buildings that included as-built structural elements (beams, columns, and shear walls) and nonstructural components (exterior cladding, masonry infilled walls, and interior partitions) had not. In effect, analytical procedures were already reasonably well developed, but little work had been done to determine the reliability of computer model response analyses. This was the motivation for initiating a substantial part of the research conducted in connection with the structural response program.

Low-Rise Buildings. Ground motion from UNE detonated at NTS also affected many of low-rise buildings in southern Nevada. The towns of Tonopah and Beatty near the western boundary of NTS experienced strongly-perceptible motion from the larger UNEs detonated at NTS. Many damage complaints

were made by owners of low-rise buildings in Las Vegas.

Most of the low-rise buildings involved were single-family, wood-frame or concrete block houses. Most of the damage complaints arising from these buildings involved minor architectural wall cracking (A59). Such minor cracking of walls is an ongoing process caused by differential soil-foundation settlement, and by shrinkage and swelling of wood door and window headers. These phenomena induce prestress in the architectural wall materials, and therefore only an insignificant amount of response deformation of the building caused by UNE ground motion may trigger a new crack. Similarly, newly poured concrete floor slabs undergoing normal shrinkage of the concrete will develop interior stress, and may require only minor additional stress from ground motion to produce a crack.

Off-site UNE detonations were particularly important in emphasizing the need to developing reliable procedures for predicting damage to low-rise buildings. These events involved low-rise buildings primarily, and multi-story engineered buildings were few. This circumstance, together with the unexpectedly large incidence of NTS-event damage complaints arising from low-rise structures, led to greatly-increased emphasis on the development of low-rise damage prediction technology.

2.2.2 Investigative needs of the Structural Response Program

At the outset of the structural response program Blume conducted a number of special investigations to provide critically needed information (A6, A13, A19, A25, A27, A28, A61). The objective of these investigations was to develop guidelines and analytical procedures for predicting the effects of UNE generated ground motions on structures.

As noted, the fundamental engineering concepts and principles of engineering mechanics for predicting the response of idealized structures were reasonably well developed by 1964 when the structural response program was begun. Little had been done, however, to develop response and damage predictions for prototype buildings situated on a variety of soil foundation conditions. Useful guidelines for predicting threshold damage, which was of primary concern to the DOE/NV Off-Site Safety Program, were virtually nonexistent.

The basic technology development needs were in obtaining reliable parametric data to be used as input to analytical models which are used for making response evaluations. These needs were enumerated in a Blume statement to the ground shock evaluation subcommittee in 1964, quoted in Section 1.2.1, preceding. Particularly critical needs included those listed following:

1. Damping values for various structural framing systems, construction materials, soil foundation conditions, and response amplitudes.
2. Natural periods of vibration for various construction materials, structural framing systems, response amplitudes, building heights, and soil foundation conditions.
3. Information on the influence of nonstructural elements (partition walls, cladding, and other architectural finishes) on response and damage for various structural framing systems, construction materials, response amplitudes, and soil foundation conditions.
4. Data on threshold damage (onset of damage) including response amplitudes that cause threshold damage, and the manner in which threshold damage is manifested in various structural framing systems, construction materials, and soil foundation conditions.
5. Checks on the validity of the theoretical mathematical modeling procedures for performing structural dynamic analyses.

2.3 GROUND MOTION PREDICTION

Although ground motion prediction was initially the responsibility of another DOE/NV contractor, Roland F. Beers, Inc. (later renamed Environmental Research Corporation), Blume played a significant role in shaping this technology.

At the outset of the structural response program, ground motion predictions for proposed UNEs consisted only of peak values of particle motion (D5). An initial challenge was to establish that peak values of ground acceleration, or velocity, or displacement were poor indicators of structural response. In response to the structural response programs need for frequency-distributed ground motion prediction, Roland F. Beers Inc. developed a band-pass amplitude-frequency prediction method based on band-pass filtering of velocity seismograms at 13 center frequencies in the range from 0.4 to 11.0 cycles per second (D62).

Subsequent analytical studies showed that band-pass-filter data could be predicted as a function of yield and distance at each of the 12 center frequencies.

The band-pass-filter predictions provided greatly improved ground motion information for structural response predictions of the amplitude-frequency relationship data supplied thereby. Even more useful information could be supplied for structural response purposes of predictions of relative velocity and pseudo relative velocity response spectra might be obtained from similar analytical studies (D62). The Environmental Research Corporation developed an analog computer program to calculate relative velocity and pseudo relative velocity response from recorded ground velocity resulting from underground nuclear explosions. The spectra are obtained from the peak response to ground velocity of a single-degree-of-freedom system having a natural period and a damping factor.

Subsequent evaluation of the PSRV data showed that response spectra as a function of event parameters, primarily yield and distance, could be reliably predicted for future events. This development permitted greatly improved structural response predictions for subsequent tests.

The flexible body response of structures to ground motion, which is crucial for evaluating probable damage, is revealed much more explicitly through response spectra than through peak particle ground motion. Most important, response spectra reveal amplified forces in structures caused by harmonic buildup of contiguous cycles of ground motion. From previous work in seismic design criteria for nuclear power plants, Dr. Blume correctly anticipated this to be an important factor for Las Vegas valley ground motions. The Las Vegas valley is a deep soft soil site which is known for its amplification of the response of long-period (high-rise) buildings.

In the first few years of the structural response program, Blume's response spectra predictions were necessarily based on a great deal of judgement founded on knowledge from the earthquake field, because of the sparsity of good recorded UNE motion data. As more data were obtained refinements were based on UNE empirical data, statistical studies of recorded response amplitudes, and natural building periods.

Over the course of the program, substantial ground motion data was recorded from many NTS and off-site events. Systematic evaluation and analysis of this data has resulted in the development of reliable procedures for predicting response spectra, particularly for southern Nevada for which an abundance of data is now available (D54). Relationships between response spectra and structural damage which were developed in this DOE/NV program have been used extensively in the field of earthquake engineering (B78).

Among ERC's many notable accomplishments in developing ground motion prediction procedures were those for relating peak ground motion and PSRV curves to yield and range parameters for NTS and offsite use. Effects of confining pressure (burial depth) on ground motion frequencies are considered in references D96 and D97, and surface wave propagation in reference B57. Ground motion anomalies are discussed in references D18 and D24. After-shock relationships between recording location ground motion records were used to recapitulate main shock motion at important locations for structural response studies (D70).

2.4 MOTION DATA ACQUISITION AND PROCESSING

Instrumentation was a major concern for the structural response program from its outset because of the sparsity of essential response data. Ground motion recordings were needed to improve the reliability of ground motion predictions for future events, and dynamic response measurements of high-rise buildings were needed for improving response and damage prediction reliability. It soon became obvious that additional and more technically advanced instrumentation was required to satisfy both needs.

Initially, Blume's role was to define the instrumentation data needed in connection with structural response, including deployment considerations and instrumental characteristics required to provide reliable data for the frequency and amplitude spectrums of importance. At the same time, the ground motion contractor, similarly specified its needs in connection with recording ground motion.

A Special Projects Party of the USC&GS was responsible for deploying instruments and making the requested recordings. This responsibility was subsequently transferred, in 1973, to the USGS. When the USGS relinquished this contract responsibility in 1979, Blume was assigned primary responsibility for conducting the entire ground motion and structural response instrumentation program for DOE/NV. Blume retained several key personnel from the former Special Projects Party staff to provide necessary technical support for this new responsibility.

Processing the recorded time history response data was also a critical issue early in the Blume program, in that particular motion parameters, e.g., response spectra, were of prime importance for structural response prediction purposes. At the outset of Blume's involvement in the DOE/NV Off-Site Safety Program, ground motion and building response motions were being recorded using recording devices that produced either paper or magnetic tape records in analog form. Data in these forms was quite

satisfactory for determining peak motion values, but was inadequate for the more sophisticated needs of structural response program. These needs also required motion recordings of a quality and character which were not available until acquisition of the L-7 instruments in 1967. Furthermore, calculating the dynamic response of structures, as well as calculating response spectra, involves the solution of complex equations of motion and is practically only done using computers. Similarly the analysis of recorded data, for structural response correlation and ground motion correlation purposes, generally requires the use of computers.

An analog computer was acquired in the mid-1960s primarily for the Environmental Research Corporation's use in analyzing ground motion data. While the analog computer was useful for calculating response spectra of ground motion records, computational resources for the structural response program were being developed from the earthquake engineering field using digital computers. Consequently the analog computer quickly became outmoded.

An important milestone in the program was reached in the late 1960's when a hybrid computer capable of making analog analyses as well as producing digital data records was obtained. This significantly extended the Blume capability for DOE/NV, both for making damage assessments as well as for conducting technology development studies. As instrumentation technology has progressed newer technology has been implemented in the structural response program, and today much of the ground motion and building response data is obtained directly in digital form.

2.5 STRUCTURAL RESPONSE EVALUATIONS

Evaluations of probable damage to structures require knowledge of the engineering characteristics, age and condition, and soil foundation of the structures as well as the response spectrum of the ground motion to which the structures will be subjected. Results of these evaluations generally include estimates of damage, either in terms of expected dollar loss or descriptions of the nature of damage, and safety recommendations describing remedial measures that could be taken to minimize damage. In addition, selected buildings, because of their unique construction or because of their close proximity to shot point were occupied at shot time by Blume representatives to observe the response and behavior during event ground motions. Blume was also heavily involved in postshot damage complaint investigations.

Over the course of the structural response program, Blume was responsible for performing these activities for a wide variety of UNE involving a broad range of structures. Critical and noncritical structures had to be distinguished quickly. Detailed response analysis needed to be performed on critical structures that might experience damage; all of this had to be done initially in the absence of proven techniques for predicting damage.

For planned UNE events, the Blume program included five major activities:

1. Conducting structure surveys and inventories
2. Making damage predictions
3. Making safety and remedial measure recommendations
4. Monitoring structures during UNE events
5. Conducting postshot damage complaint investigations

These are described in the subsections which follow.

2.5.1 Surveys and Inventories

For the systematic identification of structures that would be potentially vulnerable to UNE ground motions, three distinct types of structure surveys and inventories were developed and implemented as part of the structural response program. These are: (1) reconnaissance surveys, (2) structure inventories, and (3) condition surveys.

Reconnaissance surveys involved making a cursory identification of structures to some radial distance from shot point, beyond which distance no damage to structures or landslides was likely to occur. The information gathered was usually limited to noting the azimuth and distance from the shot point, the type of construction (e.g., wood-frame, brick, reinforced concrete) and the number of stories. In addition, any other natural or cultural features that might be damaged or disrupted by ground motion, such as wells, dams, roadways, bridges, industrial facilities, and potential rockfall or landslide areas were included in these reconnaissance surveys. The area to be covered in reconnaissance surveys varied with the UNE yield. For off-site low-yield events, such as the RULISON event in Colorado, the reconnaissance survey (A29) extended to about 35 kilometers from shot point. For the much larger central Nevada FAULTLESS event, the reconnaissance survey covered a radius of about 200 kilometers.

Structure inventories were intended to find and describe all facilities located within the area in which

damage was likely to occur. Inventory data included the engineering characteristics of each facility, such as the type of building and number of stories, or bridge type, etc. In addition, particularly vulnerable features of the facility, were noted, for instance an old weathered chimney on a house, or obvious serious ongoing differential foundation settlement problems that a structure might be experiencing. These inventories were conducted for making total damage assessments for a particular UNE event, as well as for making safety recommendations.

Condition surveys were normally conducted for structures in areas repeatedly exposed to ground motion from NTS testing. These surveys were routinely made for the Las Vegas high-rise structures, both to provide data on any visible or measurable effect of their repeated exposure to ground motion from NTS testing series, and also to provide documentation on existing non-event cracking, etc., for use in responding to any damage complaints that might arise.

In addition to the above mentioned surveys, an ongoing historical site survey was initiated for a large area bordering the NTS, as required by legislation directed at determining effects of activities which might adversely affect sites or structures of historical importance. These surveys were conducted to locate all such sites and structures within range of potential damage from NTS activities and to evaluate their sensitivity to such damage. Results have been documented in reports to DOE/NV.

2.5.2 Damage Assessments

Damage predictions for planned UNE events were made using two distinct methodologies. Structural dynamic response analyses, using detailed computer mathematical models, were made to evaluate probable damage for high-rise buildings and other complex or sensitive structures. Generalized analyses, using simplified computer models were made to estimate damage for low-rise buildings and other facilities.

Structural dynamic response analysis and evaluation were conducted in advance of planned events, with particular emphasis on individual structures which constituted unusually sensitive problems, because of concern about design or construction aspects, or because of the function of the structure and any unusually severe potential effects of its failure. When appropriate, these evaluations included recommendations of remedial measures that might be implemented to mitigate expected damage.

Estimates of damage to low-rise buildings relied upon generalized response analyses procedures based largely on building count, type of building, predicted response spectra, and available motion-damage

relationship algorithms. Because of the large number of low-rise buildings involved in a typical UNE event, response analyses for individual buildings were not practical. In addition, because low-rise buildings are so significantly affected by damage triggering processes (e.g., differential settlement, shrinkage and swelling of wood-framing materials, etc.) a detailed response analysis can not accurately predict the onset of damage. Initial predictions in this regard were generally based on engineering judgement from previous earthquake and blast experience. Empirical analytical procedures were formalized as damage data were accumulated in the Blume structural response program.

2.5.3 Safety Recommendations

Safety recommendations were provided when circumstances and analysis indicated the necessity for a particular action to reduce or eliminate hazard to people in areas to be subjected to ground motion from planned events. These recommendations included such items as notification to building management personnel of impending strongly-perceptible response motion, strengthening measures for some structures, evacuation recommendations for structures or areas to be exposed to potentially dangerous ground motion, and consideration for altering event schedules when weather-related environmental conditions, e.g., slope stability, were important. The Exchange Club in Beatty, Nevada, was regularly evacuated for higher yield tests on nearby Pahute Mesa.

2.5.4 Shot Monitoring of Structures

In connection with major UNE events, both NTS and off-site, Blume representatives monitored the response of various buildings and other facilities. This was done primarily to obtain direct observation of structural response behavior.

2.5.5 Postshot Damage Complaint Investigations

Damage complaint investigation and evaluation of the credibility of such complaints involved application of the prediction technology to determine whether or not the subject motion was at damaging levels. In general, it was also necessary to determine what other damage producing mechanisms, e.g., foundation problems, might have been contributing to the damage involved in the complaint. Besides application of the prediction technology it was in almost every such investigation necessary to use good engineering judgement and experience with various damaging mechanisms in order to arrive at a final evaluation of such complaints.

2.6 STRUCTURE RESPONSE INVESTIGATIONS CONDUCTED

Studies conducted in connection with the Blume structural response program aimed at improving damage prediction reliability can be grouped in four categories: (1) field investigations, (2) laboratory testing, (3) statistical evaluations of damage, and (4) postearthquake damage investigations. A summary of the studies conducted in each of these categories is given in the following four subsections.

2.6.1 Field Investigations

Field investigations conducted in the program included the construction of special test structures at NTS, testing buildings that already existed at NTS, and monitoring the dynamic response and performance of structures in Las Vegas and elsewhere. Detailed discussions of these investigations are set forth in Chapter 4.

In 1965, DOE/NV approved the construction of two identical full-scale 4-story reinforced concrete frame test structures in NTS Area 3. These structures were tested extensively as bare frames, to verify mathematical modeling parameters and to determine variations in response parameters such as periods of vibration and damping. They were also tested with various infill wall materials (hollow-clay tile, gypsum wallboard, plywood, etc.) to determine the influence of nonstructural elements on modeling parameters, overall dynamic response, and damage thresholds. The primary damage indices sought in the early phases included the onset of damage, minor nonstructural damage (cracking) of infilled walls, and minor cracking of beams and columns. The dynamic response of these structures was measured for most of the significant NTS UNE events from the time they were constructed in 1965 through about 1976. In addition, induced vibration tests were conducted, using both vibration generators and the pull-release procedure. These induced vibration tests were typically conducted at times when the structures were changed by inserting a new infill wall material or when the structures had been exposed to severe vibration response from nearby UNE.

Las Vegas high-rise buildings were also an important source of data for filling in gaps in knowledge. Because these buildings were regularly exposed to ground motion from NTS testing, most of the more significant buildings were routinely instrumented with seismometers for major UNE events. In addition, low-amplitude (nondamaging) forced vibration tests were conducted on sensitive buildings, or on buildings that would provide a particular type of needed data. This provided a substantial amount of empirical data on building periods, damping, mode shapes, soil-structure interaction effects, as well as data on the variations of all these parameters with response amplitude.

One-story wood-frame and concrete block test structures, located on various foundation materials throughout the NTS, were instrumented to record ground motion from significant UNE. These structures were surveyed before and after such events to provide data for empirical correlation of ground motion and damage. They also provided the opportunity to determine the influence of a variety of soil-foundation conditions on response and damage.

Forced vibration tests and measurements of response to ground motion were also performed on two 2-story houses at NTS constructed for earlier test programs, as well as on houses in Las Vegas and in Colorado. Data on the response and performance of prototype buildings situated on a variety of soil foundation materials was urgently needed; investigations were conducted wherever opportunities existed, and whenever they could be fit into the ongoing schedule for the structural response program.

2.6.2 Laboratory Testing

In 1964, DOE/NV approved the establishment of a Blume-operated structural dynamics laboratory in connection with the structural response program. The primary objective for establishing the laboratory was to provide damage information on the behavior of nonstructural elements of buildings. Nonstructural components of buildings, e.g., partition walls, architectural finishes, and trim, are typically have the lowest damage threshold levels. Knowledge of the response and performance of nonstructural building components is central to determining damage thresholds, and is therefore crucial to reliable damage predictions needed for the structural response program.

Although the laboratory supported a number of additional investigative activities, the most significant work performed involved conducting cyclic racking tests of 8 ft by 8 ft wall panels representative of typical partition walls in high-rise buildings. Tests were conducted on a variety of wall panels, including gypsum wallboard on wood studs, gypsum wallboard on metal studs, and plywood on wood studs. Results are summarized in Section 4.1.1.

2.6.3 Statistical Evaluations of Damage

The SALMON event in 1964 vividly demonstrated that predicting damage for low-rise buildings was not a simple matter. Subsequently, damage complaints were made in connection with NTS testing, and there was some minor damage, but it was not until the late 1960s that a statistically representative sample of damage was available, both from higher yields at NTS and from offsite Plowshare programs. Because of these testing programs larger numbers of low-rise buildings were being subjected to potentially damaging ground motion, and human response to the associated publicity and increased motion

perception led to large increases in numbers of damage complaints.

In 1970, at DOE/NV's request, Blume began a study of low-rise building damage, specifically focused on developing a statistically based simple but reliable procedure for predicting damage to this class of buildings. Data sources were the 1969 RULISON event, the 1973 RIO BLANCO event, and NTS events prior to 1977. Results of these investigations produced a reliable means of estimating motion-damage relationships for low-rise buildings. A detailed discussion is set forth in Section 4.5.

2.6.4 Postearthquake Damage Investigations

In the field of earthquake engineering, earthquakes are regarded by many as the testing laboratory for manmade structures. Because of the many similarities between earthquake and UNE ground motions, properly interpreted observations of earthquake damage could be used to provide important data for the structural response program.

Accordingly, several damaging earthquakes were investigated as part of the Blume program. Some of these were general investigations that focused on anecdotal observations of damage; others involved conducting detailed damage investigations. Of those investigated, the magnitude 6.6 1971 San Fernando, California, and the magnitude 5.9 1978 Santa Barbara, California earthquakes were the ones most exhaustively studied. Detailed results of these studies are set forth in Section 4.4.

2.7 STRUCTURAL RESPONSE EVALUATION PROCEDURES DEVELOPED

With the experience gained from studying the response behavior of structures to UNE-generated ground motion, natural earthquakes, and from the laboratory studies, significant improvements were made in the methods for predicting damage to structures. Important gaps in knowledge were filled in connection with mathematical modeling parameters needed for performing structural dynamic response evaluations of individually important structures, thresholds of response for nonstructural and structural damage as well as for other important considerations, and damage to low-rise buildings was vastly illuminated.

The results of these findings have been incorporated into three procedures that were developed for predicting damage to structures subjected to ground motion. Although the procedures were developed primarily for applications in connection with UNEs, they apply for earthquake damage prediction as well. These procedures are referred to as: The Engineering Intensity Scale, The Spectral Matrix

Method, and The Threshold Evaluation Procedure. They are discussed in detail in sections 4.6.4, 4.6.5, and 4.6.6.

2.7.1 Engineering Intensity Scale (EIS)

A first step in predicting the effects of UNE ground motion is to estimate the extent of the area in which structures might be damaged. The next step is to make a general evaluation of the incidence and degree of damage that such structures might sustain. Peak ground acceleration (PGA) values have been and still are in general use for making these evaluations. Because of inherent limitations in the usefulness of PGA for damage predictions, the Engineering Intensity Scale (EIS) was developed to provide a better indication of the demand that ground motions impose on structures.

In the formulation of the EIS, ground motion is characterized by 5%-damped response spectrum velocity (S_v), and structures are characterized by their fundamental mode vibration properties. Neglecting mode shape considerations, the important correlation variables for relating motion and damage are S_v , amplitude and building period. The 5%-damping value is used because damping in many real structures varies from about 2% to 10%, and 5% has been a standard reference level in the structural response program.

Engineering intensity numbers are assigned to various spectral velocity amplitude bands. The applicable range of spectral velocities (S_v) and periods (T) applicable to civil engineering structures is divided into a 10 by 9 matrix. There are 10 intensity levels, from 0 through 9, and nine period bands, I through IX, from 0.01 to 10 seconds.

2.7.2. Spectral Matrix Method (SMM)

The SMM is an orderly, standardized procedure for predicting damage to large numbers of structures located in a confined or broad geographical area subjected to phenomena such as UNEs, air blasts, earthquakes, tornados, hurricanes, and floods. For predicting building damage caused by ground motions, the method considers the following factors:

- Amplitude and frequency distribution of ground motion
- Soil foundation conditions
- Elastic strength, capacity, and damping of structures
- Inelastic reserve strength of structures for dissipating excess demand

of communicated energy

- Probabilistic evaluation of the random variables inherent in the prediction problem

Building damage is expressed by SMM in two ways. First, SMM can be used to predict the expected total dollar cost to repair or replace buildings that may be damaged by a UNE. The total number of damaged buildings for which reparation is made, without regard to the amount of damage cost, is a second measure of damage that can be estimated by SMM; for this measure, buildings are classified as being either damaged or undamaged.

2.7.3 Threshold Evaluation Procedure (TEP)

The threshold evaluation procedure for predicting the effects of ground motion on structures involves a systematic and detailed dynamic analysis of each individual building. Threshold evaluation analyses are conducted when it has been previously determined (from less rigorous approximate methods) that structures are expected to be damaged and (1) the structures are of significant economic or social value, (2) the structures pose a hazard to persons, or (3) the structures have a consequential environmental impact. The principal objectives for conducting such a detailed analysis are to identify the potential risk concerning a structure's failure as well as to identify possible modifications that might improve the lateral force resistance of that structure. Because of the substantial effort required to perform the TEP analysis, only a limited number of structures can be analyzed for any particular event.

Response thresholds that have been identified as important in connection with engineered structures (particularly buildings) and UNEs include: (1) Code Required Threshold, (2) Working Stress Threshold, (3) Yield Limit Threshold, (4) Story Failure Threshold, (5) Human Alarm Threshold, (6) Observable Damage Threshold, and (7) Human Hazard Threshold.

The principal steps in a threshold evaluation analysis involve: (1) identifying damage or failure criteria for the structure (capacity thresholds), (2) constructing a mathematical model of the structure, (3) performing a dynamic response analysis of the structure using the mathematical model and appropriate ground motion record or response spectrum to determine response demands, and (4) reconciling response demands with capacity thresholds.

3. DATA ACQUISITION

3.1 SEISMIC DATA ACQUISITION PROGRAM

3.1.1 Background

Seismic data acquisition tasks in support of the DOE/NV program include procurement, maintenance, deployment and operation of seismic equipment; recording of ground and structural response motions resulting from UNE; conversion and processing of the seismic signals, and maintenance of the seismic record library. The Special Projects Party, originally of the U.S. Coast and Geodetic Survey (USC&GS) and later of the U.S. Geological Survey (USGS), had measured ground motion from nuclear events beginning with the Bikini Atoll testing in 1946. The party was permanently assigned to the Las Vegas area in 1961. At the peak of the program, the field party included over 30 persons including geophysicists, physicists, engineering geologists, seismological engineers and electronics technicians. During its period of involvement, the Special Projects Party conducted seismic data acquisition tasks resulting not only from UNE generated motion, but also from earthquake and vibration testing.

When the structural response program began, Blume involvement in seismic data acquisition was limited to providing input to the Special Projects Party for the measurements of response of structures in the affected areas. Similarly, Roland F. Beers, Inc., the ground motion contractor, provided input for the ground motion measurements. Seismic signals recorded and processed by the Special Projects Party were analyzed by the two contractors for their respective investigations and predictions. Seismic data acquisition responsibilities were transferred to the Blume organization in 1979.

3.1.2 Instruments

Over the years of UNE testing at NTS, various types of seismic recording equipment have been used, varying with the types of information required, but more significantly with the state of the technology of the equipment and its availability. These instruments have evolved over the years because of more sophisticated recording requirements of the program and also advancements in electronics and magnetic circuitry techniques. Some of the earlier instruments included the Wood Anderson displacement gauge, EV-300 and 301 displacement gauges, long-and-short period Benioff velocity seismometers, NGC-14 and NGC-19 geophones, and Sprengnether portable blastmeter. Most were brought over from the oil exploration and mine blasting field. Systems that have been in long use and those used in the current program are the C&GS-1 Strong Motion Recorder, the NGC-21 Seismograph System, the L-7 Seismo-

graph System, and the digital cassette recorder (C7).

Figure 3.1 shows the approximate time frame in which these instruments were used. The C&GS-1, Benioff Variable Reluctance seismometers, NGC-14 and NGC-19 geophones were some of the earliest instruments used on the program. The C&GS-1 system, which was brought over from the earthquake program, was used until early 1980. Because the earlier systems did not have the capability to measure long period motion, the EV300 and EV301 systems were acquired specifically to measure long period (or low frequency) portion of ground motion. The Sprengnether Blastmeter was brought over from use in mine and quarry blast monitoring field. It was one of the first portable systems used on the program, and made it possible to record motion at any location with little advance notice. The strain gage accelerometer system was used to measure the acceleration as large as 5 times gravity (5g); other systems recorded motion only up to 1g. The L-7 system evolved from earlier geophones and was specifically built for the Special Projects Party with specification required in support of the DOE/NV program. It had a range of flat response not available in previous systems; it essentially covered the frequency range of all other systems combined. A most recent addition to the program is a portable system with digital cassette recorder. The analog signals from the sensors are converted into digital form and record on cassette tape. The response curves of these systems are shown on Figures 3.2 and 3.3. Following is a more detailed discussion of these systems that had major impact on the program.

C&GS-1 Strong Motion Recorder This was one of the earliest systems, introduced to the program in 1959, and used until 1981. It was a combination of six optical-mechanical instruments that recorded ground acceleration and displacement on a 12-inch photographic paper. The recorder, and seismometer were contained in an aluminum box. Three C&GS accelerometers, and three Carder displacement meters were all mounted orthogonally. The instrument was started either by a motion activated pendulum starting mechanism, or by remote radio signals. The signals from these sensors were recorded on a photographic paper. Timing was provided by an internal or external timer. A small wooden shed was constructed over each instrument box to provide shelter and dark room facility for changing and loading and unloading the photographic paper. A record was brought back to the laboratory and chemically processed for preservation. Oftentimes, when a record from an important station needed to be immediately scaled for peak amplitude, the scaling was performed with the paper still wet from the development process.

NGC-21 Seismograph System The basic analog system consisted of 3 seismometers, a five-stage amplifier, and an analog magnetic tape recorder. It was designed to Special Projects Party specifica-

SYSTEM	APPROXIMATE PERIOD OF OPERATION				
	1965	1970	1975	1980	1985
C&GS1	XXXXXXXXXXXXXXXXXX	X	X	XXXXXXXXXXXXX	X XXX
SPRENGNETHER BLASTMETER	XXXXXXXX				
CEC STRAIN GAGE	XXX	XX	XX		
NGC-21 VELOCITY METER	XXXXXXXXXXXXXXXXXXXXX				
EV-300/301 DIS- PLACEMENT METER	XXXXXXXX				
L-7 VELOCITY METER		XXXXXXX	X	XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX	
DCS-301/302 DIGITAL CASSETTE RE- CORDER				XXXXXXXXXXXX	
PDR-1 DIGITAL CASSETTE RECORDER				XXXXXXXXXXXXXXXXXXXX	

FIGURE 3.1 INSTRUMENT UTILIZATION

FIGURE 3.2 FREQUENCY RESPONSE CURVES OF VARIOUS SEISMOGRAPHS (E4)

FIGURE 3.3 FREQUENCY RESPONSE CURVE OF L-7 SYSTEM (E4)

tions to record velocities as low as 0.000001 cm/sec and as high as 17 cm/sec with flat response from 1 to 100 Hz (Hertz, i.e. cycles per second). For special measurements such as time-correlated recording of building response shape, it was possible to record 12 components of motion simultaneously on a single tape. The recorder had a capability of recording 24 hours of motion. It was used from 1964 through 1974. The signals on the tape could be reproduced by a playback equipment onto a paper for manual scaling or interfaced electronically with a computer. The conversion of signals is discussed further in later sections.

L-7 Seismograph System This analog system included seismometers, integrating amplifiers, a time-code generator, and a frequency modulated (FM) magnetic tape recorder. Using an integrating amplifier, the acceleration signal was amplified and integrated to velocity. The system had a flat frequency response of 0.2 to 100 Hz; a much wider range than any system on the market at the time. Another feature of the system was that the recorder had the capability to record for over 5 days. The system was designed for the USGS for these two features; wider response range and longer period of recording. It was introduced to the program in 1967 and it gradually replaced the NGC-21 system. Several of the L-7 systems are still used today.

Digital Recording System This is a portable system that records the signal on a digital cassette tape. It typically has 3 channels of acceleration input. The signal is amplified and digitized at 100 or 200 samples/sec/channel. The system can be activated manually, with remote radio signal, with impulse motion, or with STA/LTA trigger (algorithm using long-term average of background noise and the amplitude and frequency content of the above background signal). The systems were introduced in 1980 and have replaced all of the C&GS-1s and some of the L-7 systems in support of the off-NTS ground motion monitoring for the DOE/NV.

3.1.3 Deployment and Operation

The Special Projects Party seismic instrumentation program on the NTS began in 1961 as a modest effort with deployment of a limited number of seismic recorders. Initially almost all of the systems were located on the test site, because there was essentially no impact outside of the test site because of the small size of the detonations. A few arrays extended outside of the test site, to gather more distant data on the wave propagation characteristics of ground motion. The instrumentation program was essentially the same through event BILBY in September 1963 (C1). Seismic recorders consisted of C&GS1 Strong Motion accelerographs and geophones such as NGC-14 and NGC-19. While providing the seismic motion instrumentation for the NTS activities, the Special Projects Party also provided

support for the test activities in such remote locations as Arizona, Mississippi, Wyoming, California, Colorado, New Mexico, Central America, and Amchitka (C2, C3, C5, C9, C13, C22, C23).

Instrumentation Procedure For each event the instrumentation procedure consisted of three phases; the planning and coordination, implementation, and data distribution, as shown in the flowchart of Figure 3.4. Since each of the seismic safety contractors and the agencies involved in an event had separate instrumentation requirements to conduct its studies, it was necessary for the Special Projects Party to coordinate these requirements. In Phase One a priority list of instrumentation requirement would be submitted for each event by the contractors and agencies involved. The Special Projects Party would prepare a composite plan based on the requirements and submit it to DOE/NV for approval. Based on instrument availability and the submitted requirements, a final instrumentation plan would be developed and distributed.

In Phase Two, the Special Projects Party implemented the final instrumentation program. For each event an implementation plan and schedule would be developed. Oftentimes, the plan for an event would be the same as that of the previous one. In such instances, the implementation would consist of preparing the system for recording. Where the installation was at a new station, the installation was coordinated with the property owner for the access. After all of the systems were installed, the systems were field-calibrated, checked and readied for recording. During the early part of the program when the procedure was new and there were only a few tests conducted, dry runs would be conducted to test the operation procedure. Last minute changes or adjustment would be made and the systems were activated for recording. An important supplemental operation was the preparation of an event card, which contained relevant information for each instrument that would be deployed. This card became a key document that would be affixed to a recording throughout its deployment, recording, processing and archiving. Other supplemental operations included a checking of radio telemetry equipment used to trigger the recorders, calibration and preparation of seismic record reduction equipment, and calibration of continuous monitoring seismic stations to adjust their sensitivities to optimum recording of the event motion. Based on the information from the test operation center, the instruments were triggered either by radio or by the operator.

The Phase Three operation consisted of retrieval, processing and dissemination of the records. Immediately after an event, paper trace of the records were produced and scaled from each system. A report summarizing the peak amplitude motion was prepared and distributed. The paper records of structural response were provided to Blume and the ground motion records were provided to the ground motion

[illegible]

3 - 7

contractor for their own studies. Consolidated versions of the individual analog magnetic tapes were prepared and sent to the ground motion contractor for signal processing (a detailed discussion of this operation is presented in Section 3.2). Equipment at the continuous seismic monitoring stations was recalibrated and prepared for continuous monitoring.

As the coordinator for the instrumentation, the Special Projects Party became a focal point for much of the event support operations of the contractors.

Instrument Deployment from BILBY through HANDLEY A history of instrument deployment program to support the NTS test program can be best viewed from inspection of the instrumentation deployment for large events conducted at the test site (E10). These were conducted during two distinct periods; BILBY through HANDLEY (a period of yield escalation) and during the Accelerated Testing Program (ATP) conducted prior to the initiation of the 150-kt yield limit in 1976.

Because the ground motion generated by the event BILBY produced perceptible motion in the few high-rise buildings existing or under construction at the time in Las Vegas, a program was initiated to monitor the response of these buildings. In addition, a monitoring program was initiated in the communities surrounding the NTS for the UNE-generated ground motion. The program to obtain measurements in the communities surrounding the test site began with the TURF event of April 24, 1964. For this event the C&GS-1s were situated in the town of Beatty to the west of the test site and Alamo to the east. The first high-rise buildings to be instrumented were three hotels; a 23-story, a 9-story and a 21-story under construction in Las Vegas. The Sprengnether Blastmeters were hand-carried onto the top level of buildings to record the building responses. In spite of instrument limitations and recording difficulties, important data on fundamental modes of vibration for these buildings was obtained. With each new event seismic recorders were installed on additional buildings. For the BYE event of July 16, 1964 five high-rise buildings were instrumented with the C&GS-1s, Sprengnether Blastmeters or NGC-21s. With the use of 12-channel NGC-21 velocity meters, it was then possible to obtain measurements that were time-correlated and the investigators were able to determine deflected shapes and torsional characteristics of buildings. The sensors were located at various parts of a building and were connected to a central recorder using multi-strand cables. The cables were strung along a stairwell or outside of a building. Local fire codes now forbid this procedure and cables must be run through pre-installed conduits. By May 1965 six high-rise buildings in Las Vegas had C&GS-1 seismographs installed, typically on the roofs of buildings; two other buildings were monitored with NGC-21 velocity meters. Except when the equipment was needed to support instrumentation for events outside of the NTS, these

systems essentially became permanent fixtures on most of the buildings.

In addition to the instrumentation for events, the Special Projects Party conducted vibration tests of buildings to obtain their response characteristics. For this test the 12-channel NGC-21 was used. The sensors were located at various levels of a building. The excitation was produced by personnel on the roof exerting a slight but adequate horizontal force on the building by swaying in unison at the building's fundamental period. When a maximum possible amplitude was built up by resonance, the excitation was stopped and the motion was permitted to decay. By analyzing the records, it was possible to obtain such response characteristics such as response mode shapes, periods, and damping of a structure. Tests were conducted in 1964 and 1965 on essentially all of the high-rise buildings that existed at the time.

The next progressively larger event was HALFBEAK, conducted on June 30, 1966 at a yield of 365 kt (E10). From the time of the BILBY event to the HALFBEAK event, a number of additional systems were procured and deployed. Also the instruments were located with major emphasis to obtain recording in the communities surrounding the test site (Figure 1.1). The instrumentation outside of the test site included ground stations in Beatty, Alamo, Tonopah, Twin Springs and Las Vegas; a south-east array extending from NTS to Las Vegas; an array (Hot Creek Line) running north into mountain terrain; and most of the high-rise buildings in Las Vegas that existed at the time. On the test site, the instrumentation included the ground stations near various facilities, several arrays, one-story test houses, and the two 4-story concrete test structures. The C&GS-1 and the NGC-21 were the primary instruments used for seismic measurements for the seismic safety program during this period. Forty-nine records were recovered for HALFBEAK.

A much larger area was instrumented for the GREELEY event, (December 20, 1966, with a yield of 870 kt). In addition to the instruments that were deployed for the HALFBEAK event, instruments were also deployed at Hoover dam; Kingman, Arizona; Bishop, California; and Goldfield, Nevada. A 5-story building in Tonopah built during the gold rush days in early 1900s, was also instrumented. Sixty-four C&GS-1 and NGC-21 records were recovered. Records from 12 high-rise buildings in Las Vegas were obtained. In addition to instrumentation support for the GREELEY and other events at the NTS, the USGS deployed instruments in Hattiesburg, Mississippi in support of the STERLING event of December 3, 1966.

The BOXCAR event conducted on April 26, 1968, was the largest UNE at the time with a yield of 1.3

MT. Instrumentation for BOXCAR covered a large area of Nevada (Figures 3.5 and 3.6), and was also deployed as far away as Salt Lake and Ogden, Utah; Reno, Nevada; and Sacramento, San Francisco, and Los Angeles, California. Many instruments were also deployed in the nearby communities surrounding the test site. The Special Projects Party began the deployment of the L-7 seismograph systems about 6 months prior to this event. Ninety three records were recovered from C&GS-1, NGC-21, and L-7 instruments. Records from 14 high-rise buildings were obtained. This was one of the most hectic instrument deployment periods for the Special Projects Party. In November 1967 the Special Projects Party deployed about 50 recording systems in Farmington, New Mexico in support of the GASBUGGY event, and in December 1967 it deployed over 130 systems for the FAULTLESS event in Central Nevada.

Three other large events conducted on the NTS during this periods were the BENHAM event at 1.15 MT on December 19, 1968, the JORUM event (less than 1 mt on September 16, 1969), and the HANDLEY event, greater than 1 MT on March 26, 1970. By this time essentially all of the NGC-21 systems had been replaced by the L-7 systems. One hundred and eleven records were recovered from the HANDLEY event (C4). Records were obtained from 19 high-rise structures in Las Vegas. While the NGC-21 systems procured in 1963 had been almost entirely replaced by the L-7 systems, the C&GS-1 instruments that had been in operation since beginning of the program were still in use.

Accelerated Test Program (ATP) UNE yields at the NTS were relatively small after the HANDLEY event until DOE/NV conducted a series of 12 large tests during the 10 month period from May 1975 through March 1976. Greatly increased instrumentation requirements for the ATP were created by a major boom in high-rise building construction in Las Vegas that had been going on since 1970. There were about 25 high-rise buildings in Las Vegas in 1972. The number grew to about 65 by 1975 and many more were under construction. During 1973 and 1974, 30 were built. Of the 80 systems that were deployed for the ATP's 200 to 1000 kt CAMEMBERT event on June 26, 1975, about half were located on the roofs of the high-rise buildings.

Instrumentation Program after the ATP The seismic monitoring program after the ATP has been limited to a distance of about 200km from the test site. It has included the ground stations in the communities surrounding the test site with major emphasis on the ground stations and buildings in Las Vegas. Even though the size of an event is limited to 150 kt, the motion resulting from the event generated ground motion is still perceptible in nearby communities and on the high-rise buildings in Las Vegas, about 150km from the test site. Because of the geological setting of Las Vegas, the ground

FIGURE 3.5 BOXCAR EVENT CLOSE-IN INSTRUMENTATION (E4)

FIGURE 3.6 BOXCAR EVENT VELOCITY INSTRUMENT LOCATION (E4)

motion in certain frequency ranges is amplified. Some of these frequency ranges coincide with resonant frequencies of many of the high-rise buildings. This similarity in frequency amplifies building response motion. Although event yields are currently constrained by the 150 kt limitation perceptible response motion frequently occurs in the high-rise buildings. Consequently, most of the instruments are located on these structures. High-rise buildings doubled in number from 65 in 1975 to about 130 in 1988, and are currently being built at a rate of about 6 per year.

Instrumentation Outside NTS From 1963 to 1973, several off-NTS events were conducted in support of the Vela Uniform and Plowshare programs. Event locations included Mississippi, New Mexico, and Colorado. In addition, the Special Projects Party deployed instruments in support of activities at Central Nevada where 130 seismic systems were deployed for the FAULTLESS event, on Amchitka in the Aleutian Islands, for several high-explosive experiments outside of the NTS, for field vibration experiments, and for several post-earthquake investigations.

The SALMON event was conducted near Hattiesburg, Mississippi in October 1964. Most of the instruments used in support of the NTS program were transported and used for this experiment. The instrumentation consisted of Special Projects Party strong motion seismographs. The involvement of the Blume organization in the seismic data collection was limited to operation of 2 Sprengnether blastmeters in a building in Hattiesburg. Blume was not involved in the Long Shot event in Amchitka, Alaska of October 1965 although both ERC and Special Projects Party did have assignments. Blume had an involvement in the STERLING event in Hattiesburg, Mississippi of December 1966 and the GASBUGGY event in Farmington, New Mexico of December 1967. The new L-7 velocity meters were used at GASBUGGY, prior to a heavier involvement for FAULTLESS.

The FAULTLESS event with a yield in the 200 to 1000 kt range was conducted in the Hot Creek valley in central Nevada in January 1968. In addition to the major structure investigation involvement, which is discussed in more detail in Section 6.1.1, Blume's involvement included instrument deployment recommendations and shot-time observation. All available instruments were used for this experiment. The Special Projects Party operated about 130 seismic stations ranging in distance from 4 to over 500 km. Instruments included strain gauge accelerometers, C&GS-1 accelerographs, NGC-21 velocity systems, and L-7 velocity systems. Figures 3.7 and 3.8 show the distribution of the seismic equipment for FAULTLESS.

The 40 kt RULISON event was conducted on September 10, 1969, near Grand Valley, Colorado. The

FIGURE 3.7 FAULTLESS EVENT CLOSE-IN INSTRUMENTATION (E4)

instrumentation for this included L-7 velocity meters operated by the Special Projects Party (C2). The involvement of the Blume organization was primarily in the pre- and post-event structural investigation, operation of Sprengnether blastmeters, and assisting DOE/NV in the investigation of damage claims. A detailed discussion of the structural investigation and the evaluation of effects is presented in Section 6.1.3.

The 80 kt (approximately) LONG SHOT event of October 29, 1965; the 1 MT (approximately) MILROW event of October 2, 1969; and the CANNIKIN event (less than 5 MT) of November 6, 1971; were conducted in Amchitka, Alaska. The Blume organization was not involved in those tests.

The LONG SHOT event was recorded at station SE-6 in Las Vegas by the Special Projects Party. Neither ERC nor Blume had any participation in that event.

For MILROW, the Special Projects Party deployed 10 L-7 instruments; 8 on Amchitka itself at distances of 7 to 28.5 km from shot point and 1 each on the Aleutian islands of Adak and Shemya at distances of 300 and 377 km respectively. One of the Amchitka instruments failed to record. Data from the other 9 were processed by the Environmental Research Corporation to obtain seismograms; pseudo-relative velocity spectra; peak velocity and peak component acceleration, velocity and displacement; arrival times; and other data. This information compared well with pre-shot predictions made by ERC (D37).

In 1971, for the CANNIKIN event, the Special Projects Party again deployed 10 L-7 instruments, 1 strong motion accelograph, and 3 strain gage accelerometers. The L-7 instruments were located generally at the same locations on Amchitka, and the strain gage acceleration were located on Semisopochnoi, Rat and Amatignak islands at respective distances of 57.4, 65 and 126 km from the CANNIKIN shot point.

Instrumental data were processed by ERC to obtain seismograms; peak component and peak vector acceleration, velocity, and displacement; arrival times, and other data. Also, pseudo relative velocity spectra were obtained from the L-7 records. Generally good agreement was observed between the pre-shot ERC predictions and the actual shot data (D49).

The last off-site event was the RIO BLANCO event (three 33 kt devices) conducted near Meeker, Colorado on May 17, 1973. The Special Projects Party deployed L-7 velocity meter systems (C22).

The involvement of the Blume organization in the seismic effects evaluation was limited to the strengthening of the TOSCO tower near Grand Valley, Colorado, and instrumentation recommendations for the structures. A more detailed discussion on the response of the tower is presented in Section 6.1.3.

Non-Event Related Support Activities While the basic task of the Special Projects Party was to provide the instrumentation required in support of the testing activities, other instrumentation programs were also conducted, these included monitoring of earthquakes (C6), high-explosive experiments (C5), forced vibration tests on test structures at NTS, measurements of ambient and human-generated vibrations on high-rise buildings, and measurements of high-rise building response to wind and earthquakes. The Special Projects Party also provided support for refraction surveys and ground motion characterization to determine ground motion amplification of local geology (C12).

3.1.4 Record Processing

The level of processing of recorded motions depended on the type and the use of the record. If a motion was recorded to obtain a peak amplitude, the processing was done by studying the reproduced paper record. If a record was obtained to determine complex characteristics of ground motion or a building response, the record would be converted to a computer compatible format before a more detailed analysis could be performed. However, peak amplitude of motion and the period and duration of motion could be determined directly from paper records. Depending on the recording mode of the seismograph, the seismic motions were recorded on paper (photographic, heat sensitive), analog tape, or digital cassette tape. Digital recording systems currently becoming available store data in computer chips, rather on a cassette tape. The motion recorded on analog and digital tapes can be reproduced on paper using a compatible playback equipment. The records thus obtained were analyzed visually (manual scaling of amplitudes and frequency) and with the aid of the computer.

Visual Scaling Immediately after an event, paper traces of the record were reproduced. An initial scaling of the records included determination of peak amplitude of motion, its corresponding period, and arrival time of various seismic waves. Subsequently, more detailed analyses were performed. For the ground motion records from earthquakes, for example, a determination would be made on the arrival of various waves to calculate a distance from an epicenter. For the building response records, a determination would be made on the building's fundamental periods, corresponding amplitudes, and mode shapes. Section 4.2.4 describes example of data collected over a period of time to show the variation in building response period versus amplitude of motion. While it is commonly assumed that a building behaves fairly linearly at a low amplitude of motion, there are actually considerable varia-

tions, depending upon the structural and architectural systems for the building concerned, as described in Section 4.2.4. Even with the extensive use of computers in the current record analysis technique, manual scaling is still a useful tool to obtain basic seismic motion parameters quickly.

Computer Processing Computer processing of seismic records has been used almost from the beginning of the underground testing at the NTS. The first such procedure was implemented with a digitization of C&GS-1 paper records in September 1962. The C&GS-1 records were digitized by Planetary Sciences, Inc. (PSI) using a Benson-Lehner Model Oscar K digitizer. The output from the digitizer was in a form of punched paper tape. Using computer programs, the punched tapes were converted to IBM cards. Various computer programs were developed to aid in the analysis and interpretation of the records. The Special Projects Party computer programs included error removal, instrument response correction and data smoothing, computation of vector particle trajectory display, Fourier synthesis; statistical analysis of frequency spectra, and integration of record in both time and frequency domains.

The seismic record conversion and processing responsibilities were assigned to ERC installed a highly sophisticated and complex analog, digital and hybrid computers in its home office in Alexandria, Virginia to process the signals recorded on analog magnetic tapes. Figure 3.9 shows the block diagram of the complete ADP system. Analog data and control signals from the analog processing equipment were routed through the master patch panel according to the processing needed. Digital data and control signals from the digital processing equipment were performed through the I/O bus. Each peripheral was interfaced to the digital computer through a controller, which monitored the control signals necessary for its function. The controller also directed the peripheral function with the overall program operation.

In the development of the processing operations, ERC organized the processing to seven phases; preview, correction, analog to digital conversion, digital to analog verification, derivation of companion motions, analog band-pass filter time history derivation, and analog pseudo-relative velocity derivation. For each phase, equipment used for the operation was interfaced using the patch panel. Figure 3.10 shows the configuration for the various processes.

Following an event, the recordings obtained by Special Projects Party on analog magnetic tapes were copied and consolidated onto a dub tape. Because of the limitation of the analog-to-digital conversion equipment, the signal on the dub tape was stretched by a factor of 10 during the copying process. The dub tape was shipped to ERC, which converted and processed the records. The output in a form of

FIGURE 3.9 ERC ADP SYSTEM BLOCK DIAGRAM

Equipment Used for Analog to Digital Conversion of Magnetic Tape Data.

Equipment Used for Digital to Analog Verification Program.

Equipment Used for Analog PSRV Program.

FIGURE 3.10 ERC ADP CONFIGURATION FOR VARIOUS PROCESSES

digital data was sent to the support contractors and agencies. Primarily, the ground motion data were analyzed by ERC and the structural response data were sent to Blume. After the digital data had been verified to be accurate, the records on the original field tapes were erased and the tapes were recycled.

When ERC moved its overall operation to Las Vegas as requested by DOE/NV in 1970 the seismic record conversion and processing equipment was also relocated. Funding reductions in subsequent years reduced ERC's role below levels needed for continued operation. Computer Science Corporation (CSC), parent organization for ERC assumed responsibility for remaining ERC functions. The ground motion analysis and prediction portion of the contract was awarded to Adaptronics, Inc., in Alexandria, Virginia. At that time, the responsibility and the equipment used for the conversion and processing of the seismic records was transferred to the USGS, which had assumed responsibility for the Special Projects Party and the seismic instrumentation program formerly assigned to the USC&GS, and the equipment was subsequently moved to its Golden, Colorado office. The Special Projects Party performed the conversion and processing operation and provided the processed output to Adaptronics, Inc. and Blume for post-processing and analysis.

When the conversion and processing responsibilities were assigned to Blume in 1980, the equipment remained with the Special Projects Party. In order to perform at least the record conversion operation, Blume implemented a modest record conversion system using the analog playback equipment and a PDP-11/34 computer. Figure 3.11 shows the configuration diagram of the signal conversion equipment. The operation was limited to the conversion of the field data to a computer compatible format. The processing and analysis of the converted data were performed on computers at other facilities such as the DOE/NV computer center and Lawrence Berkeley Laboratory. In addition to the conversion of the analog signal, a capability was developed to convert the seismic signal recorded on digital cassette tapes. Over the next several years, processing and analysis software such as filtering of signal, calculation of companion motions, derivation of Fourier analysis, and generation of pseudo relative velocity spectrum were developed to perform much of the operations performed on the previous system. In 1987 the PDP-11/34 computer was replaced by a VAX8200 based system. Configuration for this system is shown in the diagram in Figure 3.12.

3.1.5 Seismic Record Maintenance

The seismic records collected in conjunction with the DOE/NV program include paper traces, analog magnetic tapes, digital cassette tapes, and digital magnetic tapes. The records are maintained for refer-

FIGURE 3.11 SEISMIC SIGNAL PROCESSING WITH PDP-11/34

CONFIGURATION DIAGRAM
SEISMIC SIGNAL CONVERSION & PROCESSING
FY-90

FIGURE 3.12 SEISMIC SIGNAL PROCESSING WITH VAX8200

ence in support of the test program and for documentation purposes. Some of the earlier data, collected prior to 1961, have been stored in the Federal Archives. Prior to the assignment of record maintenance to Blume in 1980, such maintenance was assigned to various DOE/NV contractors. The seismic motion records on photographic paper and on analog magnetic dub tapes were maintained by the Special Projects Party. The ground motion records in digital format were maintained by the ERC. The structural response records in digital format were maintained by Blume. Following assignment of record custody in 1980, Blume has acquired all stored seismic records except those turned over by Adaptronics to the Defense Nuclear Agency, and is currently conducting systematic review and documentation of the data maintained by the previous contractors, in addition to maintaining all of the records collected since it took over the responsibility in 1980.

Table 3.1 is a brief summary of records collected and maintained in conjunction with the test program.

3.2 STRUCTURE DATA COLLECTION

In addition to the motion data, information was collected on many structures and buildings in southern Nevada to fulfill the responsibilities associated with the structural response program. Since damage to structures may result from causes other than UNE motion, other information such as geology, foundation material, construction materials had been collected. A discussion on a detailed geotechnical investigations conducted in Las Vegas to study compaction fault scarps, subsidence problems, and study of expansive soils is presented in Chapter 7.

3.2.1 Structure Reconnaissance and Inventory

In order to assess the effect of ground motion on structures in an area to be subjected to strong ground motion, a possible area of exposure is determined and an inventory of structures is conducted. This information is necessary particularly for tests conducted in a new area or as test yields at the NTS increased, necessitating the investigation of wider area. Investigations conducted in association with tests conducted outside of the NTS such as for events FAULTLESS and RULISON are presented in Chapter 6. For NTS events, efforts were originally concentrated on Las Vegas structures, primarily on the multi-story hotel and office buildings.

Immediately upon receiving the contract award in early 1964, the Blume organization was requested to obtain any currently available data on the high-rise buildings in Las Vegas and assess the potential impact of the NTS ground motion. Structure information was collected on the 7 high-rise buildings that

TABLE 3.1
SUMMARY OF SEISMIC MOTION RECORDS

Fiscal Year	L-7	NGC-21	C&GS-1	DIGITAL CASSETTE	OTHERS
1963	--	22	48	--	3
1964	--	16	24	--	3
1965	--	97	156	--	49
1966	--	239	339	--	133
1967	10	148	194	--	42
1968	188	112	348	--	36
1969	206	53	228	--	58
1970	491	41	297	--	69
1971	361	15	136	--	32
1972	136	--	48	--	18
1973	313	--	78	--	14
1974	18	--	20	--	4
1975	161	--	88	6	4
1976	372	--	183	41	6
1977	171	--	35	6	2
1978	212	--	78	--	--
1979	172	--	26	--	--

existed at the time. They ranged from 9 to 29 stories. There were two under construction (22 stories and 26 stories) and one proposed. Since the immediate problem was to determine the effect on the Las Vegas high-rises and other pressing issues, the reconnaissance and the inventory of other buildings and structures in the surrounding communities was not conducted thoroughly until later.

A reconnaissance survey was conducted as a part of the condition survey program in 1965 (A4). The areas covered were Mercury and NRDS on the NTS, Indian Springs, Lathrop Wells, Beatty, Goldfield, Tonopah, Alamo, Moapa, Glendale, Overton, Las Vegas, Henderson, and Boulder City (Figure 3.13). Several structures were selected for continued survey.

With anticipated increase in NTS test yields, a structural inventory of much wider area was conducted in 1968 and 1969 to supplement the information collected by Blume since 1964. In addition to the physical survey, state and local agencies were contacted to obtain supplemental information. In preparation for the inventory, a form was prepared to document the pertinent information. The information obtained were location, owner, structural data, and estimated replacement cost. The structures were grouped by construction type, using a system developed by the U.S. Public Health Service, and by range of their fundamental periods. An example of the type of information obtained in the survey is shown in Figure 3.14. If there was only one or one type of structure at a location, the information obtained was specific to one structure as in the above example. If there were many structures in different category, the information was grouped for individual building class, as shown in Figure 3.15, and an average replacement cost estimate for each type. The information thus obtained was used in the estimation of damage cost used in one of the prediction techniques described in Chapter 4. In addition to the structure information, foundation information of was obtained. This information was obtained from such sources as geologic maps, geologic reports, foundation report, bore hole logs, and site observation.

In Las Vegas however, much more detailed information was needed, both because of the variability of ground motion throughout the area and the substantial hazard potential presented by the increasing numbers of high-rise structures.

3.2.2 Detailed Structural and Architectural Data

In order to make quantitative assessment of the effect of ground motion on high-rise structures, it was necessary to obtain information in a form of architectural and structural drawings. With this information, an analysis was performed by preparing a mathematical model to determine the structural behav-

FIGURE 3.13 COMMUNITIES COVERED IN 1965 SURVEY

BUILDING INFORMATION

Location No. CL 005G Lat. 36° 03' Long. 115° 16'

Location Name A Ranch House Date Seen 10-28-68

Owners Unknown

Year Built - Condition Now Good

Number of Stories 1 Total Height 12' ±

Length & Width 20' x 60' ± House

Frame Type Wood

Exterior Wall Finish Wood siding

Interior Wall Finish Probably Gypsum Wallboard

Ground Floor Wood

Intermediate Floors None

Roof Mineral surfaced roll roofing on wood frame.

Chimneys? (Type) None

Percent of Wall Openings - N. 25 E. 10 W. 10 S. 25

Construction Cost or Estimated Replacement Cost \$ 20,000

Brief Description of Hazards (Items which are susceptible to Damage) --

A small addition at south end of house has asbestos siding. There
are no other structures at this location. Photographs available.

Bldg. Class	<u>5</u>
Cost	<u>20,000</u>

Loc. No. CL 005G

FIGURE 3.14 BUILDING INFORMATION COLLECTION FORM

Building class, indicated near the bottom of each sheet, is assigned on the basis of the following table.

<u>Bldg. Class Number</u>	<u>Construction Type</u>	<u>Building Period Limits</u>
1	Adobe	0.05 - 0.15
2	Adobe	0.15 - 0.20
3	Brick & Stone	0.05 - 0.15
4	Brick & Stone	0.15 - 0.20
5	Wood Frame	0.05 - 0.15
6	Wood Frame	0.15 - 0.20
7	3-5 Story Comm.	0.20 - 0.40
8	6-8 Story Comm.	0.30 - 0.50
9	6-8 Story Comm.	0.50 - 0.80
10	6-8 Story Comm.	0.80 - 1.00
11	Low L.V. Hi-Rise	0.65 - 0.90
12	Med L.V. Hi-Rise	0.85 - 1.30
13	Hi L.V. Hi-Rise	1.50 - 1.80
14	Salt Lake Hi-Rise	0.65 - 1.80
15	Reno Hi-Rise	0.65 - 1.80

FIGURE 3.15 BUILDING CLASSIFICATION

ior, determine its dynamic behavior, and perform stress calculation. The techniques and procedures for the detailed analysis are presented in Chapter 4.

3.2.3 Structure Condition Survey

A detailed condition survey of individual Las Vegas structures was proposed in 1965. The objectives of the survey were to observe existing conditions and monitor any changes, both to provide a check on the prediction techniques that were being developed, and to provide reference information for evaluations of damage claims. One of the first program was to monitor and study the movement of cracks in buildings in southern Nevada (A4). For this study several representative buildings were selected and a detailed monitoring program was initiated. This detailed monitoring program was terminated in 1969. A more general monitoring program was continued on the high-rise buildings.

Building Crack Movement Study One of the studies initiated was to measure movement of existing cracks in buildings undergoing ground movement from underground nuclear detonations. Data were obtained in conjunction with underground nuclear detonations in southern Nevada (A23) and in the Rulison, Colorado area (A36).

In most buildings, cracks in building elements are the results of one or more of the following types of action:

- o Differential foundation settlement
- o Expansion or contraction of building materials due to climatic variation
- o Lateral forces acting on the structure

Differential foundation settlement usually results from inadequately designed building foundations or from highly expansive foundation soils. Lateral forces commonly result from wind loads or ground motion, e.g., earthquakes or underground nuclear test detonations. All of these forces can be acting simultaneously.

Cracks that result from these various causes are usually located at points of stress concentration, such as at corners of doors and windows, at the intersection of building elements, or at other points of discontinuity.

Types of Gages Gage types included Starrett dial gages (standard 1-inch stroke, readable to 1×10^{-3}

inches) and passive gages (displacement and scratch types). The term passive indicates that the gage has maximum-movement memory in that the maximum movement in the time interval between observations is recorded.

The two-part base was mounted on a building element, straddling a crack. Any opening or closing movement of the crack resulted in relative motion of a stylus attached to a rod anchored in the left-hand portion of the gage. As the stylus moved, it displaced two sliding bars in the right-hand component. At the time field measurements were to be made the bars were first locked in their displaced positions by means of two friction screws. After the displacement was measured with micrometer calipers the friction screws were released, the sliding bars were returned to their closed position, and a reading was made in the closed position.

The two-part scratch gage base was mounted on a building element, straddling a crack. Any movement of the crack -- opening or closing -- resulted in relative motion of a stylus attached to a rod anchored in the right-hand portion of the gage. The stylus point touched a film strip mounted beneath it on the left side of the crack. As the stylus moved it caused a scratch to appear on the film strip. At the time measurements were to be made the film strip was removed and taken to the office to be read. Readings were usually made from contact prints. The scratch gages were designed to monitor crack movements in 2 dimensions, whereas the displacement gages monitored only uniaxial movement.

On the basis of laboratory test results and observations from field data, it was concluded that the threshold of quantitatively reliable field data appeared to be about 10^{-2} inches.

Crack movement data from many types of structures -- high-rise, low-rise, engineered, and unengineered -- were taken during the period of the monitoring program, 1965-1969. Because of the lack of data reliability for the lower-order crack movements associated with differential settlement and climatic variation, the attempt to correlate relative crack movements with event-caused ground motion was eventually abandoned.

In addition to the studies to relate crack movement to various parameters, an investigation was undertaken to relate amount of crack movement with specific event motion on one particular high-rise building. During the periodic condition survey of high-rise buildings in Las Vegas, cracks at joints of concrete frame and architectural blockwalls were observed in this building. Continued monitoring indicated that cracks appeared to be enlarging, with an indication of spalling of mortar in one area.

The most severe cracks were observed between the soffit of the third floor beam and the concrete filler wall below. A deflectometer was installed to measure the relative displacement between the beam and the blockwall (a relative displacement between second and third floor). The dial movement of the deflectometer was photographed using a 16 mm movie camera and the relative movement was recorded. The dial gage reading was digitized and compared with the record from the displacement meter located at the roof of the building. It was found that the crack movement coincided with the movement of the building. More importantly, the relative movement between these two floors was almost 13% of the building movement of this 22-story building. This meant that the relative movement was not uniform throughout the height of the building, but movement at this particular story was larger than the average movement. The result from this and other similar studies became a part of the structural damage parameters used in the evaluation of building damage thresholds.

Much of the detailed monitoring program and other investigations were discontinued with the completion of the large yield test program in 1970. Also the direction of the monitoring program was shifted from the low-rise building to high-rise buildings. In 1970 there were only 20 high-rise buildings in Las Vegas. In 1987 there were over 130 high-rise buildings. The emphasis of condition survey has necessarily shifted to a broader survey program. All of the high-rise buildings are currently surveyed at intervals of 6 to 18 months. The frequency of survey is based on the condition of the building. Those that show deterioration or continued change are surveyed approximately every 6 months and others are surveyed less frequently. Since it is not feasible to gain access to individual rooms, the areas normally inspected are roof, hallways, stairwells, basements, service rooms, and garages. During the survey, photos are taken to document existing deterioration and any change in such conditions between surveys. Regardless of different structural systems, most of the buildings exhibit similar types of architectural deterioration. Common types of deterioration in the interiors of buildings include cracking of drywall butt joints, vertical cracks above door frames, and shrinkage cracks. The exteriors typically show signs of aging through bubbling in roof tar paper, random stucco cracking, exterior wallboard buckling from water penetration, and cracking and spalling of exterior rock-impregnated panels.

3.2.4 Geotechnical and Soil Foundation Data

Geotechnical and foundation data are important in the evaluation of structural response and structure damage. Geotechnical investigations as well as structural investigations were conducted for some test. The first geotechnical investigation was conducted in southern Nevada in support of the test program at the NTS in January 1965. The study identified the deep alluvial sediments of the Las Vegas Valley that produced the amplification of ground motion in the long period range, the chemical composition of

foundation material that contributed to many of the damage claims in the Las Vegas area, and the compaction fault scarps and subsidence problems. These initial findings resulted in an extensive study as presented in Chapter 7. These studies and the data collected during the course of the program provided vital information for the understanding of the structural response and evaluation of damage claims.

3.2.5 Structural Damage Data

To make estimates of damage from a planned detonation, it was necessary to collect and study damage observed from past seismic activities such as earthquakes and UNE events. In addition to the damage data collected in conjunction with earthquakes as discussed in Chapter 4, the damage data resulting from UNEs were collected and studied. From event BILBY in 1963 through HANDLEY, DOE/NV received approximately 500 damage complaints of which 177 resulted from the 1970 HANDLEY event. The total amount paid was about \$24,000 during this period. There were over 700 complaints during the Accelerated Test Program from May 1975 to March 1976 with claims of about \$90,000 paid. A summary tabulation of damage is presented in Table 3.2. A detailed study correlating damage and ground motion is presented in Chapter 4.

TABLE 3.2

SUMMARY TABULATION OF DAMAGE COMPLAINTS AND CLAIMS
(Approximate Number of NTS Events)

Period	< 200KT	> 200KT	No. of Complaints	No. of Claims	No. of Claims Paid	Amount of Claims Paid
FY-64	5	1	0	0	0	0
FY-65	5	0	0	0	0	0
FY-66	14	1	9	1	0	0
FY-67	6	2	32	2	0	0
FY-68	12	1	116	22	2	\$4,805
FY-69	13	1	93	115	3	\$575
FY-70	17	3	243	72	49	\$23,092
FY-71	5	1	13	2	0	0
FY-72	3	0	4	1	0	0
FY-73	6	1	6	2	2	\$1,393
FY-74	2	0	11	5	2	\$464
FY-75	9	3	91	39	17	\$2,602
FY-76	5	9	651	377	246	\$87,116
FY-77	7	0	32	8	5	\$1,596
FY-78	10	0	54	22	11	\$6,417
FY-79	11	0	42	15	9	\$3,977
FY-80	6	0	32	6	6	\$4,974
FY-81	3	0	1	2	2	\$1,942
FY-82	14	0	14	4	2	\$2,575
FY-83	4	0	6	2	2	\$370
FY-84	8	0	15	2	2	\$1,332
FY-85	8	0	16	3	3	\$765
FY-86	9	0	17	1	1	\$945
Totals	--	--	1,498	703	364	\$144,940

4 TECHNOLOGY DEVELOPMENT AND APPLICATION

4.1 LABORATORY DAMAGE AND MOTION PERCEPTION STUDIES

At the start of the DOE/NV structural response program in 1964, then-current earthquake engineering practice was directed toward ensuring building survival under strong lateral loading from some assumed most-likely major earthquake. For this approach the bare structural frame was used as the sole force-resisting element. Interior partitions and in-fill walls were ignored. However, it was recognized that nonstructural walls could resist lateral forces and absorb energy during cyclic applications of forces. Thus, nonstructural walls had to be considered when predicting, for DOE/NV, damage thresholds for buildings subjected to ground vibrations. In order to develop the technology for predicting vibration-induced damage, a series of cyclic racking studies of nonstructural wall panels was conducted in the Blume structural dynamics laboratory (A5, A15, A27, A28).

In addition to predicting building damage thresholds for ground vibrations, it was also important for the DOE/NV safety program to predict thresholds at which occupants of buildings might become aware of vibrations or feel discomfort from the motion. Thus, human perception of motion was also studied in the Blume structural response program.

4.1.1 Cyclic Racking Studies of Nonstructural Wall Panels

While engineered elements (beams, columns, or shear walls) in a structure can usually be modeled using the principles of continuum mechanics, typical nonstructural partition wall panels cannot be so readily described theoretically. Useful information concerning the racking stiffness and energy dissipation characteristics of nailed wood frame, lath, and plaster wall panel construction is most practically determined from testing.

Much static (one-way) racking testing of typical wood-frame construction had been conducted on wall panels in the past. This information was useful in identifying racking stiffness, but it provided virtually no insight concerning damping or the cyclic motion effect on stiffness, knowledge of which is essential for reliable mathematical modeling of structures. At the commencement of the structural response research effort by Blume in the mid-1960's, information concerning the cyclic (two-way) racking behavior of nonstructural wall panels was limited. To support the structural response research effort for DOE/NV, Blume tested 55 panels of some 17 different types to determine panel force-deformation and

energy dissipation characteristics. Examples of cyclic force versus displacement curves are given in Figure 4.1. Table 4.1 provides information on the test panel makeup and materials.

A backbone curve of force versus displacement from cyclic testing is the envelope of the maxima of force and displacement for the various cyclic test amplitudes. Figure 4.2 shows the backbone curve for the cyclic test results shown in Figure 4.1. Of the possible methods for identifying the force versus displacement capacity of panels, the backbone curve, which reflects the effects of positive and negative cyclic deterioration, is the method most representative of the cyclic response behavior of panels in buildings responding to dynamic ground motion.

Of the several possible variables that can be identified for quantifying the energy dissipation for structures or structure elements, the most useful are energy capacity and equivalent viscous damping ratio. From typical cyclic racking tests results, such as those in Figure 4.1, energy capacities are defined by the shaded area specified in Figure 4.3(a), 4.3(b), and 4.3(c).

Summary results of cyclic testing of a few of the more commonly encountered types of panels are given in Table 4.1. The table lists seven types of wall panels typically used in high-rise and low rise building construction. Observations made during cyclic testing revealed that threshold damage to finished gypsum wallboard partitions occurs at racking displacements of about 1/8 inch.

For composite structures consisting of other structural elements in addition to wall panels, a composite equivalent viscous damping ratio, G , may also be found. The total energy dissipated or absorbed, E_d , is given by:

$$E_d = E_{dp} + E_{ds} \quad (4.1)$$

where:

E_{dp} = energy dissipated by the panels

E_{ds} = energy dissipated by the structural elements.

As described earlier in the discussion of Figure 4.3, the equivalent viscous damping ratio λ is represented by:

$$\lambda = \frac{1}{2\pi} \left[\frac{E_d}{U} \right] \quad (4.2)$$

where U is the total linear energy capacity.

Test LR2A, Run 2

FIGURE 4.1 EXAMPLE CYCLIC FORCE-DISPLACEMENT PLOTS

TABLE 4.1 DESCRIPTION OF NONSTRUCTURAL WALL PANELS

Type Number	Stud Material	Wall Material		Stud to Runner Connection
		Side One	Side Two	
1	3 5/8" Metal	1/2" Gypsum Wallboard	1/2" Gypsum Wallboard	None - friction
3	3 5/8" Metal	1/2" Gypsum Wallboard	1/2" Gypsum Wallboard	Pop-rivets
5	2 x 4 Wood	1/2" Gypsum Wallboard	1/2" Gypsum Wallboard	Nailed
7	2 x 4 Wood	1/2" Plywood	1/2" Plywood	Nailed
LR2	2 x 4 Wood	1/2" Plywood	1/2" Gypsum Wallboard	Nailed
LR4	2 x 4 Wood	1"x8" Horizontal Siding	1/2" Gypsum Wallboard	Nailed
LR6	2 x 4 Wood	1"x8" Horizontal Siding w/ Let-in Diagonal Bracing	1/2" Gypsum Wallboard	Nailed

FIGURE 4.2 EXAMPLE OF BACKBONE FORCE-DISPLACEMENT CURVE (Test LR2A, Run 2)

(a) Energy Capacity

(b) Linear Energy Capacity

(c) Energy Absorbed

FIGURE 4.3 DEFINITION OF ENERGY TERMS

If Equation (4.2) is rearranged and substituted into Equation (4.1) for subscripts p , denoting panel properties, and s , denoting structural element properties, and for the following conditions:

$$U = U_p + U_s \quad (4.3)$$

then

$$U = G_p U_p + G_s U_s \quad (4.4)$$

and finally

$$G = G_p \left[\frac{U_p}{U} \right] + G_s \left[\frac{U_s}{U} \right] \quad (4.5)$$

for the composite structure.

For large amplitudes of motion, i.e., greater than 1/8 inch, a linear equivalent viscous damping ratio has little meaning for a panel alone because its behavior is so nonlinear. However, a linear equivalent viscous damping ratio may be very significant for composite structures for which U_p/U is small. If the structural frame alone behaves linearly, even for motions of large amplitude, then the addition of very nonlinear panels will make the structure composite, an equivalent viscous damping ratio defined by Equation (4.2) is a valid approximation.

Conclusions reached during the high-rise panel testing were generally as follows:

- First visible damage to new 8 foot-high wall partitions generally occurred at 1/8 inch relative story displacements. The reopening of patched cracks occurred at 1/16-inch relative story displacements.
- Creaking noises of walls generally began at 1/8- to 1/4-inch relative displacement for gypsum wall board partitions. For stiffer partitions of plywood or plaster the creaking sounds may occur at 1/16 inch.
- The elastic limits for the 8-foot-high wall panels were in the neighborhood of 1/4-inch relative story displacement. The moduli of rigidity decreased by a factor of 2 to 3 when relative displacements were increased from 1/32 inch to 1/4 inch. These values were based on tests at 0.7 Hz, and were expected to be valid for high-rise buildings with periods of 1.5 seconds and longer.

- After partitions had been subjected to large lateral displacements, their stiffness was reduced for small lateral displacement.
- The test samples exhibited high percentages of energy absorption which appeared to increase with larger relative displacements.
- Construction details played an important role in the investigation of partitions for structural response. For example, walls with metal studs firmly attached to the top and bottom runners exhibit lower percentages of energy absorption than walls whose studs were connected to the runners by friction only. Later tests indicated that the rate of loading and its frequency, within the limits of the costing program, did not significantly affect the results.
- The effect of walls on the damping of the entire structure was dependent on both the stiffness and the damping of the structure, respectively. In one example, wall panels with 15% to 20% damping added 1% to 2% to the damping of the structure.

Conclusions from the low-rise structure wall panel tests were as follows:

- The 8-foot panels showed generally linear stiffness until a deflection of 1/8 inch was reached. Beyond this displacement, general panel behavior was nonlinear, softening, and inelastic. For amplitudes of motion 1/8 inch and greater, average stiffness values for the second set of four cycles were 7% less than the average stiffness values for the first four cycles. Furthermore, panel stiffness was greatly reduced for reapplied low-amplitude motions.
- Construction details also played an important role in panel stiffness results. Construction methods, workmanship, and probable variations in material properties caused significant variation (up to about 70%) in panels of the same type with no apparent fabrication differences. Results from panels that required little construction detailing, such as plywood sheathed panels, varied only a few percent. Yet, racking stiffnesses of horizontally sheathed panels, which are highly dependent upon the vertical separation of sheathing, varied significantly.

- Energy absorption caused only by nonlinear hysteretic working was expressed in a dimensionless form, equal to an equivalent viscous damping ratio when applicable. This dimensionless value for all amplitudes of ground motion varied from 0.10 to 0.18, with most values grouped around 0.15 for all panels. Of particular significance was that this dimensionless energy absorption value decreased as the panel deteriorates. For amplitudes of 1/8 inch or greater, the average dimensionless energy absorption value for the second set of four cycles of motion was 10% less than that for the first set of four cycles.

4.1.2 Human Perception of Motion

Prior to Blume studies in the late 1960's, investigations into human perception of motion had been concerned with vibrations primarily induced by machinery in the workplace or on aircraft. Vibrations from these sources are characterized by frequencies greater than 1 Hz and long durations. The results from those studies were not generally useful in the structural response program, because vibrations induced by UNEs and earthquakes have considerably shorter durations (10 seconds to 2 minutes) and lower frequencies (below 1 Hz). People in high-rise buildings are often more aware of these low frequency motions than those outside, because the ground motion frequencies are often close to the natural frequencies of high-rise buildings and cause the buildings to resonate. Thus, a series of tests was devised to obtain quantitative information on human perception of motion for frequencies below 1 Hz (A22).

The objectives of this series of tests were to obtain data pertaining to threshold levels of motion perception for humans under controlled laboratory conditions, and to compare the results with data on motion perception reported previously by observers in high-rise buildings in Las Vegas during UNEs at NTS. The frequencies selected for testing ranged from 1 Hz to 0.2 Hz, because they correspond to periods between 1 sec and 5 sec, which are the approximate fundamental periods of buildings 10 to 50 stories high.

The test apparatus consisted of a plywood enclosure mounted on a plywood platform which was suspended from cables attached to each corner of the platform. This configuration provided a simple pendulum whose period could be adjusted by changing the length of the cables. Test subjects sat or stood inside the enclosure. Motion was induced by releasing the platform from an initial displacement that would cause a 0.001g acceleration. The displacement was then increased in increments equivalent to a 0.0005g acceleration until the subject felt the motion. Ten subjects were each tested in four

positions (sitting and standing, both facing the motion and perpendicular to the motion). Tests were conducted with pendulum periods of 4.70 sec, 2.24 sec, and 1.02 sec (frequencies of 0.21Hz, 0.44Hz, and 0.98Hz respectively). Each subject was tested five times in each combination of position and period.

A statistical analysis of the results showed that there were significant differences in perception levels among individuals. The differences in perception due to position of the subjects were statistically significant, but not nearly as great as the differences between individuals. Using all the laboratory data, a regression for frequency versus perception threshold was developed as shown in Figure 4.4.

These data were also compared to other available data. Over a period of several years prior to the laboratory test, a large amount of motion perception data was collected by Blume and other organizations involved with the DOE/NV operations. Observers were stationed on roofs of various high-rise buildings in the Las Vegas area during UNEs and noted their perceptions of the motion. At the same time motion was measured quantitatively by instruments installed on those buildings. Data for frequencies greater than 2 Hz (periods less than 0.5 sec) were obtained from published literature on shock and vibrations. From a regression analysis using these combined data sets, it was determined that the calculated mean perception threshold is given by:

$$\text{Mean Perception Threshold (g)} = 0.00245 + 0.00025 f$$

where f = frequency (Hz).

The standard deviation about the mean regression is 0.0012g. The mean value for the human perception threshold for frequencies less than 1Hz (periods greater than 1 sec) is 0.00245g.

ANALYSIS OF VARIANCE FOR REGRESSION LINE

SOURCE	DEG. OF FREEDOM	SUM OF SQUARES	MEAN SQUARE
DUE TO REG. LINE	1	0.0000638	0.0000638
DEVIATION ABOUT LINE	198	0.000287	$0.00000145 = S^2$
TOTAL	199	0.000351	

REGRESSION LINE: PERCEPTION = $0.00245 + 0.00025\left(\frac{1}{T}\right)$
 STANDARD DEVIATION OF LINE: $\sqrt{S^2} = \pm 0.0012$

FIGURE 4.4 REGRESSION CURVE FOR PERIOD VERSUS PERCEPTION THRESHOLD

4.2 FIELD RESPONSE MEASUREMENTS AND STUDIES

4.2.1 Four-Story Structures

Introduction As a part of the effort to investigate the response of structures to ground motion caused by UNEs, Blume proposed, in 1965, the construction of two identical full-scale test structures to be located near each other in Area 1 of the NTS. The two test structures, designed by Blume, were subsequently constructed of reinforced concrete and were 12 feet by 20 feet in plan dimensions, with four 9-foot stories (Figure 4.5).

The test structures were designed in accordance with the Uniform Building Code (UBC) of 1961, and the American Concrete Institute Building Code Requirements of 1963. Vertical live loads were set at 100 pounds per square foot, which was approximately equal to the dead load. In the seismic design the live load was included in the calculations of weights, thereby ensuring that if dead weights were added to the structure for purposes of increasing the mass, the structure would still have a design capacity to withstand UBC Zone 3 seismic forces (maximum code-required earthquake loading). Design procedures included provision for ductility and reserve energy absorption capacity, which provided additional resistance to damage from dynamic loading.

Construction of the two buildings was completed in March 1966. A series of low-amplitude vibration tests was conducted over the next four years (A11, A25, B49). Several types of partitions were installed in the structures at various times to study their influence on overall building response behavior. Vibrations for these tests were created by pull-release, vibration generator, and human-induced methods. Responses of the buildings to ground motions caused by UNEs were also measured and evaluated. The objectives of the testing program were to investigate:

- Structural response characteristics of reinforced concrete structures;
- Participation of the reinforcing steel and floor slabs during structural response;
- Differences between two theoretically identical structures;
- Effects of architectural partitions on the structural response of the structures;
- Damage thresholds for architectural partitions due to the distortion of the structural frame during structural response;
- Effects of soil-structure interaction;
- Effects of "age" of a structure on its structural response characteristics.

In 1974 the south structure (bare-frame only) was subjected to a destructive test (A61, B63, B67, B70,

FIGURE 4.5 4-STORY CONCRETE TEST STRUCTURE

B71). This consisted of generating response at the lowest north-south translation mode at intermediate to high force levels during which the structure exhibited inelastic response and suffered major structural damage. This structure was repaired by an epoxy-injection method in 1975, and retested in 1979 (A69).

Low-Amplitude Tests Low-amplitude testing was conducted on both structures beginning immediately after construction, when the frames were fresh and uncracked. This testing took place over a four year period, with various configurations and types of partitions. Pull-release tests were used to generate free vibrations. A small eccentric mass vibration generator was employed to create forced vibrations.

Roof displacements created by these tests were small in comparison with those corresponding to yield capacity of the bare frame. Ground motion from nearby UNEs during this same period caused roof displacements which exceeded the yield-capacity displacement, accelerating the transformation of member cross section from uncracked to cracked sections. The observed effect in the response of the four-story test structures was a gradual increase in period and damping values over the first two years of testing.

The walls of a building are generally classified as structural or nonstructural. Structural walls are usually constructed of reinforced concrete or reinforced masonry and are included in the lateral-force engineering design analyses of such buildings. Nonstructural walls or partitions may consist of reinforced or unreinforced masonry but more commonly are constructed of wallboard on wood or metal studs. Nonstructural wall participation is usually ignored in lateral-forces engineering design analyses. However such walls have a significant effect on both building frequencies and damping as demonstrated by studies of the 4-story test structures. Figures 4.6 and 4.7 show a chronological summary of period measurements for the two principal directions of each building over a four-year period. Figures 4.8 and 4.9 show a similar summary for damping measurements.

Destructive Testing of the South Structure The 1974 destructive testing was designed to acquire data on structural response while undergoing damaging forces. To generate forces high enough to damage the test structure, a special vibration generating system was designed and built by SNL. It consisted of a large oscillating mass driven by a hydraulic piston. This reciprocating-mass vibration generator was capable of producing a maximum force of 12,000 pounds, with a frequency range of 1 to 50 Hz. The installation of this and other major equipment is illustrated in Figure 4.10. The vibration generator, shown on the top floor, was moved to the 3rd floor for some tests. The large concrete mass on the

FIGURE 4.6 EXPERIMENTALLY OBTAINED FUNDAMENTAL MODE PERIODS FOR LONGITUDINAL MOTION

FIGURE 4.8 EXPERIMENTALLY OBTAINED FUNDAMENTAL MODE DAMPING RATIOS FOR LONGITUDINAL MOTIONS

FIGURE 4.9 EXPERIMENTALLY OBTAINED FUNDAMENTAL MODE DAMPING RATIOS FOR TRANSVERSE MOTIONS

FIGURE 4.10 INSTALLATION OF MAJOR EQUIPMENT

ground floor was added to prevent lifting and possible overturning of the structure under strong lateral force. Before and after the destructive test, lower force level tests were conducted using the same vibration generator. The nondestructive tests conducted prior to the destructive test consisted of frequency sweep tests and frequency dwell tests, exciting four translational modes of vibration in the north-south (longitudinal) direction and three modes in the east-west (transverse) direction at low and intermediate force levels. During the destructive test, the lowest north-south translational mode was excited at intermediate and high force levels. The test structure exhibited inelastic response and suffered major structural damage during this test. The series of post-destructive tests used force levels similar to the pre-destructive tests in order to investigate the response of the structure after it had experienced structural damage.

The nondestructive tests demonstrated that the structure had the nonlinear stiffness properties of a softening system: period and damping increase with increasing amplitude of response but tended to be constant at higher nondestructive amplitudes. The damping in the second mode appeared to be higher than in the other modes at all amplitudes within the measurements, suggesting a complex but undefined relationship between frequency, mode shape, amplitude, and damping in such structures. Mode shape of vibration for each mode measured in one direction of motion was found to be nearly constant regardless of levels of excitation. No new damage was found during this test series.

At the start of the high-amplitude, destructive-level vibration test, the structure exhibited an initial constant period of 0.54 second, which was consistent with the high-amplitude range of the nondestructive testing. This was followed by nonlinear response in the inelastic range, which resulted in substantial structural damage and a significant change in the stiffness and damping characteristics of the structure. After the structure suffered major damage, its response was similar to that of a linear system with a reduced stiffness and a constant period of approximately 0.90 second.

The post-destructive tests were conducted to compare the response of the damaged and undamaged structure at similar amplitudes. As expected, the damaged structure was more flexible and demonstrated higher damping values than the undamaged structure. The fundamental period increased as the force levels were increased and varied between 0.81 and 0.90 second. Damping values were approximately 3.8%, 2.8%, and 1.6% for the first, second, and fourth modes, respectively.

Comparing period and damping in the fundamental mode from the present nondestructive tests with the low-amplitude tests conducted before this test series indicated that the response properties of this bare

frame structure were both amplitude-dependent and time-dependent. Degradation of structural stiffness occurred during periods of active vibration, while some recovery of structural stiffness occurred during dormant periods.

- Retest of Epoxy-Repaired Structure Following the destructive testing in 1974, the damaged south structure was repaired by the epoxy-injection technique. The repair work, which used a patented repair process, represented typical postearthquake repairs. The repaired structure provided an opportunity to determine the effectiveness of epoxy-injection techniques in restoring the original physical properties of structures. In 1978 Blume submitted a proposal to the National Science Foundation (NSF) for funding to test the repaired structure using the same procedures used for destructive testing of the original structure. With NSF and DOE/NV approval the epoxy-repaired structure was retested in 1979 using the same reciprocating-mass vibration generator (A69). The program consisted of a series of tests, beginning with low-amplitude vibrations followed by forced vibrations increased into the range of inelastic response of the structure. The test program was similar to the 1974 testing of the original, undamaged structure. The results of the retest program have shown that, for low-amplitude motion, the epoxy-repaired structure was slightly more flexible than the original, undamaged structure, as is illustrated in Figure 4.11 by longer periods for equivalent roof velocities in the epoxy-repaired structure. This additional flexibility was expected because not all cracks could be repaired and because the epoxy compound used was a more flexible material than concrete. The destructive-test data showed that as the amplitude of the structure's response increased the stiffness of the epoxy-repaired structure did not degrade as much as the stiffness of the original structure.

The epoxy-repaired structure suffered less damage than the original structure. Cracking was more widely dispersed and was less concentrated at the joints, as shown by comparison of photographs of damage. This suggests that the epoxy-injection procedure may have created better joints in the repaired structure than previously existed in the original structure, possibly because the epoxy improved the bonding between the reinforcing bars and the concrete in the joint.

The performance of the epoxy-repaired structures was compared with that of the original one by comparing the base shears during the two destructive tests, as shown in Figure 4.12. The curves from both tests seem to be identical up to a roof displacement of 2.5 in. The curves are bilinear in that range, with well-defined change in slope at a roof displacement of about 1.0 in. Published data on damage to beam-column connections provide a good statistical basis for identifying this break point in the plot of the base shear as corresponding to the inception of beam cracking in the structure. A second

FIGURE 4.11 VELOCITY VERSUS PERIOD FOR NONDESTRUCTIVE TEST, LONGITUDINAL DIRECTION

FIGURE 4.12 BASE SHEAR VERSUS ROOF DISPLACEMENT FOR THE DESTRUCTIVE TESTS OF THE ORIGINAL AND EPOXY-REPAIRED STRUCTURES

break in the curve for the base shear in the original structure occurs at about 2.5 in. of roof displacement. The second break point in the base shear curve for the original structure is believed to be the inception of beam yielding. There is no obvious second break point in the base shear curve for the epoxy-repaired structure that might identify yielding in the beams, corresponding to the observation of less damage to beam-column connections and more widespread cracking in the epoxy-repaired structure.

4.2.2 One- and Two-Story Structures

As a part of its ongoing structural response investigations for DOE/NV, Blume obtained data on the dynamic response behavior of a number of one- and two-story buildings (A24, A38, A51, A53, A54). These data were used to develop more precise methods for predicting damage to low-rise buildings caused by vibratory ground motion.

The buildings tested were 6 one-story and 2 two-story structures on NTS, three one-story structures in southern Nevada (not on NTS), and four houses and one free-standing chimney in Sopris, Colorado. All structures were tested by pull-release tests; some were also evaluated using a vibration generator.

The fundamental periods of the test buildings ranged from 0.04 sec to 0.28 sec. Concrete block or brick buildings were found to be stiffer (i.e. shorter periods) than comparable wood buildings. The measurements include the effect of soil deformation around the foundations as well as the deformation of the buildings themselves. The contribution of soil deformations to overall deformations was significantly greater for stiffer buildings than for more flexible ones. Hence, the periods calculated for concrete block and brick buildings were significantly increased by soil deformations. Periods of older buildings, or ones in poor condition, were greater than other test buildings. Damping was determined with less certainty. Computed values ranged from 4% to 15% of critical. Analysis suggested that damping increased as the soil participation increased.

Analysis of the chimneys in Sopris, Colorado, test buildings yielded periods of 0.08 sec to 0.15 sec for the chimneys with damping from 4.5% to 6.5% of critical. The free-standing chimney had a period of 0.18 sec and a damping of 3%. A more detailed discussion of the non-UNE experiment at Sopris, Colorado, and those at other locations is presented in Section 4.4.

NTS One-Story Test Structure Studies Small simulated residential test structures had been constructed at the NTS for studies of the response of such structures to ground motion caused by underground nuclear detonations. Eight of these structures in Yucca Flat and on Rainier Mesa (Figure 4.13) were

FIGURE 4.13 NTS TEST STRUCTURE LOCATIONS

constructed in 1963. These buildings were constructed in pairs, one wood frame and one concrete block building, at each of four locations. Two pairs were located on rock on Rainier Mesa and two pairs were located on alluvium in Yucca Flat. These were located on Pahute Mesa (Figure 4.13) where those events which generated large ground motion were being conducted, and which would therefore produce a higher level of response motion. In early 1964 an investigation of the dynamic properties of the Area 3 structures was conducted. Three broad objectives of this investigation were: (1) to investigate dynamic properties, including damping, of structures of this type, when subjected to ground-transmitted vibrations from underground nuclear explosions; (2) to investigate the effects of the extremely different foundation materials, rock and alluvium, present in the two areas involved in the structural response of practically identical pairs of structures; and (3) to evaluate proposed techniques, instrumentation, scheduling, and other aspects of the Blume investigation of the response of test structures at the NTS (A24).

Dynamic properties of the structures were obtained by measuring response to pull-release tests and to ground impact. Pull-release tests were made by sudden release of a load applied via a cable bridle and load line. Release of the applied load was accomplished by cutting a single-wire link in the load line, or in one case by increasing the load to the breaking point of the wire link. Breaking strengths and manner of failure of these wire links were established by laboratory tests made prior to the field studies. Ground-transmitted shock loading tests were made by measuring the vibrations caused inside the building when a 6-ton dozer blade and side arm assembly was dropped to the ground from a height of about 6 feet, at distances varying from 35 to 50 feet from the building.

Dynamic response testing was conducted in February 1969 and May 1971 on 3 pairs of structures from Yucca Flats (Area 3), Rainier Mesa (Area 11) and Pahute Mesa (Area 19). These 3 pairs were neighboring concrete block and wood-frame structures.

These structures were constructed for the purpose of studying the damaging effects of dynamic ground motion on typical residential buildings. Because of the symmetric, single-room, modular-type configuration, and better than standard construction, it was expected that damage would be seen at higher ground motion intensities for these structures than for real residential buildings. However, the simple configuration of these test houses was expected to be helpful in investigating several factors that affect the dynamic response characteristics of buildings.

The buildings were geometrically quite similar, and it was expected that the most significant factors that

would cause the vibration and seismic damage characteristics of the various structures to differ would be construction material, age, and soil condition. The structures tested were selected to reflect these factors. The test structures were tested dynamically to determine their dynamic response characteristics and statistically to evaluate stiffness.

The two most commonly used procedures for dynamically testing full-scale buildings, the pull-release and the vibration generator, were both employed in the test program. In all cases, the buildings were vibrated in the direction normal to the plane of the wall in which the doorway was located.

The procedure used in performing the pull-release tests consisted of placing a cable around the building at the eave line and applying a lateral static force to the building, deflecting it. After the desired force (read from a dynamometer in the force line) was reached, the force would be released suddenly. The motion of the structure, vibrating freely, would then be recorded until it damped out. For each building, tests were performed at maximum lateral force intensities of one, two, and three kips to investigate motion intensity effects.

Vibration-generated tests determined the steady-state response of the buildings to a sinusoidal exciting force produced by a vibration generator. A vibration generator would be bolted to the floor slab, and a horizontal sinusoidal force with varying intensity and frequency would be created. Such a horizontal force caused the structure to vibrate in unison with the generator at various forcing frequencies. The steady-state response of the structure would then be recorded after transient vibration had died out. For each frequency, the intensity of recorded sinusoidal response provided a point on a frequency-response curve; a series of such point yielded the total frequency-response curve.

In this test series, the rotating mass and its eccentricity were held constant. The tests were begun with a forcing period of about 0.35 seconds and reduced at discrete intervals to a period of about 0.05 seconds, which was the minimum period limit for the vibration generator used.

Static stiffness tests were performed on each of the test structures to determine the force-displacement relationships for the buildings. The force was applied in the same manner as that described above for the pull-release testing. The static stiffness tests included three loading and unloading cycles for each of three maximum lateral load intensities of one, two, and three kips.

These attempts to emulate typical low-rise structures and their response to strong ground motion were

recognized in advance to be severely limited in practical application because of the tremendous variety in types, construction, materials, and condition of actual low-rise buildings. However, useful basic response information was obtained, as set forth in the following.

The fundamental mode response characteristics of the one-story structures tested did simulate the dynamic response behavior of typical low-rise buildings. The range of observed fundamental periods for the test structures was 0.04 to 0.12 seconds. Observed damping was in the range of 4% to 9% at critical. For both period and damping, the block buildings were represented by the lower values within these ranges, while the wood-frame buildings were represented by the higher values.

The percentage contribution of building foundation motion for soil-structure system lateral distortion increased as the ratio of building stiffness to soil stiffness increased. For the tests conducted, soil effects on the dynamic response characteristics were somewhat significant for the block buildings. They were of little consequence for the wood-frame buildings.

The fundamental mode vibration properties of the buildings tested were simulated reasonably accurately by the mathematical model developed and shown in this report.

Reinforcing steel included in concrete block construction was of significant value in maintaining structural integrity for this type of construction in a vibration environment.

4.2.3 Dynamic Testing of Las Vegas Area Single-Story Structures

The dynamic response characteristics of three 1-story Las Vegas buildings were determined from analyses of pull-release tests conducted in late 1971 and early 1972. The subject buildings were constructed of wood, brick, and block materials. The block and wood-frame dwellings were residences, while the brick structure was a commercial shop. The wood-frame house was under construction at the time of the tests, which permitted a pull-release tests series to be performed at an early construction stage and again when the building was completed. All three buildings were instrumented to measure building dynamic motion (including rocking and translation in the soil), from which the fundamental mode frequencies, damping ratios, and mode shapes were determined.

An analytical model was used to represent the overall dynamic properties of the buildings and their supporting foundations. Procedures for obtaining values of the model parameters were developed and applied to the test structures. To reconcile test-derived and model-derived dynamic characteristics,

ranges of building and soil stiffness were calculated, expressed in terms of structure component period and soil shear velocity.

Building 1 was a single-story residential home constructed of concrete block walls with a cement tile roof and wood floor. The interior walls and ceiling were of lath and plaster construction. The block walls were supported on a concrete wall footing. The house was about 20 years old and in good condition.

Building 2 was a single-story residential home constructed of a wood frame covered with particle board insulation (plywood panels at the corners) and redwood siding. The roof was covered with asphalt shingles, and the floor was a concrete slab. The interior walls were gypsum board on wood-frame construction.

Two series of tests were performed on Building 2. During the first series, the house was partly built. The wood frame and roof were complete, but only the particle board insulation and plywood panels were attached. The redwood siding, gypsum board walls, and ceiling had not been constructed. At the time of the second test series, the house had been completely finished.

Building 3 was a single-story commercial building with brick walls. The floor was a 6-inch concrete slab, and the roof was constructed of wood shingles. The partition walls were lath and plaster, and the ceiling consisted of plaster over sheetrock. About 10,000 pounds of equipment were present at the time of the pull-release tests, which increased the mass of the system under consideration.

Model L-7 velocity seismograph instruments were installed in each test building in such a way that wall racking, foundation translation, and foundation rocking vibrations were measured. This was accomplished by placing at least one (usually two) horizontal sensor(s) at the floor and ceiling levels. In addition, two (often three) vertical sensors were positioned at opposite ends of the building on the floor. This arrangement allowed the basic motions (racking, translation, and rocking) to be isolated and compared.

The procedure used in pull-release tests consisted of placing a cable around each building at the eaves line and applying a lateral static force to deflect the building. After the desired force (read from a dynamometer in the force line) was reached, the force was released suddenly using a quick-release hook. The motion of the structure, vibrating freely, was then recorded until it damped out. Tests were

performed at various load levels for each building. A few of the tests were repeated several times at nearly the same load level to ensure adequate data recording.

Based on analysis of the pull-release test data, it was concluded that significant soil-structure interaction may exist for low-rise residential and commercial type buildings. This conclusion was supported by reasonable agreement obtained from a theoretical soil-structure model study. This agreement also suggests that the analytical model could be used to extrapolate to more general low-rise building soil-structure situations.

From the observed and theoretical results it was concluded that stiffer buildings (i.e., brick and block) tend to have greater soil-structure interaction. Also, it was observed for Building 2 (wood-frame) that stiffening the walls increased soil participation. Finally, both model and test results produced translation mode shape ratios that were greater than the rocking components.

Period values were observed to range between 0.09 and 0.28 seconds. Corresponding damping ratios ranged between 0.10 and 0.15 but were determined with less certainty. As shown by this study, it is difficult to make accurate measurements or theoretical predictions of low-rise building damping ratios. The mathematical formulation suggests that damping increases as the soil participation increases. This was neither confirmed nor refuted by the experimental data.

Two-Story Test Structures, Area 1, NTS This study examined the dynamic response characteristics of two 2-story test structures. The study reported the first mode period, damping ratio, and mode shape for each structure and offered some general comments about the response of this class of structures to ground motions.

The two structures considered here had identical plan dimensions. One was of wood-frame construction and the other of brick. Both free vibration, generated by pull-release tests; and forced vibration, created by response to ground motion generated by detonation of UNEs, were used in the dynamic testing of each structure. Free-vibration data are most useful in determining the fundamental periods, damping ratios, and mode shapes. Motion data recorded during nuclear events can be used to verify the dynamic properties determined from the pull-test data; such records are most useful for understanding the overall dynamic behavior of these structures and evaluating response prediction techniques.

Both structures were constructed in the mid-1950s to study the damaging effects of above-ground

nuclear explosions, and were identical except for exterior finishes. Each story level measured 822 square feet. The first and second levels had wood floors. Especially in the brick structure, these wood floors were loosely connected to the rest of the structure, which was probably due to aging and exposure to severe dynamic loading rather than to poor construction. Basement had a 4-inch-thick concrete floor and 8-inch-thick cinder block walls.

Pull tests consisted of placing a cable around a building at the eaves line and applying a lateral static force to the cable, deflecting the building. When the desired force (read from a dynamometer in the force line) was reached, the cable was suddenly released. The motion of the structure, vibrating freely, was then recorded until it damped out. The brick structure was tested in both directions with lateral loads equal to 6, 9, and 12 kips. The wood-frame house was also tested in both directions, but lower lateral loads, 4, 6, and 8 kips, were used because damage to the house was feared.

The fundamental periods, damping ratios, and mode shapes of the two-story brick and wood-frame test structures were typical of the dynamic properties of low-rise buildings. Therefore, these test structures were reasonable representatives of this class of buildings. Based on the lower standard deviations found in the analysis of pull-test data, it was concluded that dynamic properties can be determined with more certainty using pull-test data than if UNE data are used.

Average fundamental periods of 0.0763 seconds and 0.0735 seconds for the longitudinal and transverse directions of the brick structure, and 0.139 seconds for both directions of the wood-frame structure, indicated that the brick house was nearly twice as rigid as the wood-frame house. The pull-test results showed that foundation rocking and basement translation were nearly 10 times more significant in the stiffer structure. The UNE results indicated that, when exposed to long-period ground motion, both structures responded primarily as rigid bodies; however, the more flexible wood-frame structure experienced more dynamic amplification than the rigid brick structure.

Average damping ratios of 0.075 for the brick structure and 0.053 for the wood-frame structure indicated that both structures were relatively highly damped. However, in both cases damping ratios varied over a considerable range of values.

Structure Crack Movement Studies at Mercury, Nevada Studies were conducted to determine the mechanisms for crack movements occurring in a residential structure complex at Mercury, Nevada, on the Nevada Test Site (A7). The basic observations were made during the period from December, 1965

through January, 1967.

This study of cracks in structure elements was concerned principally with the effects of continuous temperature cycling, temperature differentials, and variations in wind, sun, and ground motion. Of these factors, ground motion and wind tend to subject the structure to abrupt motions; i.e., the building structure responds in a very short period of time. On the other hand, the structure and its elements are continuously moving as a result of changes in temperature and humidity, and of other forces such as differential settlement. The expansion or shrinkage due to temperature and humidity changes takes place over a much longer period of time than do the effects from wind or ground motion; e.g., diurnal or seasonal, rather than within seconds.

Of 14 Mercury buildings which had been instrumented and monitored, six which had the most complete and most reliable data were selected. Displacement gauges had been installed on existing building element cracks caused by foundation settlement, material drying and shrinkage, expansion and contraction of building elements, and similar causes. Many of these cracks probably occurred during construction of the building.

To provide a comprehensive and detailed record of crack movement, observations had been made and recorded frequently at each gauge location since installation.

The displacement gauge used was a model which evolved out of a series of field tests and modifications. The two-part base is mounted on a building element, straddling a crack. Any movement of the crack - opening or closing - results in relative motion of a stylus attached to a rod anchored in the left-hand portion of the gauge. As the stylus moves, it displaces two sliding bars in the right-hand component. At the time field measurements are to be made, the bars are locked in their displaced positions by means of two friction screws. After the displacement is measured with micrometer calipers, the friction screws are released, the sliding bars are returned to their closed position, and a reading is made in the closed position.

In investigations of structure damage from nuclear events and earthquakes, it has been found that initial damage occurs as widening and extension of existing cracks. It would be manifested as greater movement of existing cracks than that previously measured during periods between events.

Displacement gauge DG-12 was installed over an expansion joint in the east wall of the Health, Medi-

cine, Safety, and Rad-Safe Building at Mercury. The total movement recorded during event-included periods was less in all cases than those levels previously attained due to periods in which natural forces were predominant. Seven of the ten gauges (on five of the six Mercury buildings studies) showed the same results.

Event GREELEY which was detonated on December 10, 1966, created the largest ground motion measured at Mercury in this report period. Calculations of probable ground motion for all events in the report period, whether or not the ground motion was measured, showed that GREELEY should have created the largest ground motion in the entire period. If crack movement was being caused by event ground motion, it would be expected that gauge readings for the period in which GREELEY was detonated would show largest crack movement. However, gauge readings taken for the period including GREELEY ground motion were exceeded by readings taken for the periods of time just prior to and again just after GREELEY. Small events were detonated in these periods but, again, calculations showed that effects at Mercury, because of the shot parameters and location, would have been negligible.

Findings to date also indicated that damaged areas of building elements underwent changes in crack width more rapidly in the early morning and early evening. This same crack movement varied from season to season, with the greatest activity occurring during abrupt seasonal changes.

Conclusions which may be drawn from this study were as follows:

- Opening and closing of existing cracking resulted from various causes, including ground motion, temperature changes, temperature differentials, wind loading, humidity variations, and differential foundation settlement.
- In the time from December 1965 to January 1967, displacement gage readings on six monitored buildings at Mercury showed that maximum differential movements occurred during periods in which ground motion from nuclear events was not large and natural forces predominated. During periods in which the structure was exposed to strong ground motion from nuclear events, readings, although sometimes large, were not as large as at other times when strong ground motion was not experienced.
- Since crack movement during periods of maximum ground motion was less than at other

times, it was concluded that event ground motion did not contribute significantly to crack movement; i.e., damage was not being caused by event ground motion.

4.2.4 Dynamic Characteristics of High-Rise Buildings

Blume began monitoring and studying the dynamic behavior of the high-rise buildings in Las Vegas in 1964. During the course of the monitoring program, much interesting and useful information was obtained. Noteworthy observations included similar variation of response period with amplitude of motion, the variation in damping ratios, the influence of architectural elements on response and the change in response behavior over time (A60). These observations and the analysis results became the basis for some of the governing parameters used in the prediction methods discussed in section 4.6. Some of these observations were specific to the type of structures, while others were general in nature as presented below:

Response Observation Over Time: Many of the high-rise buildings have been monitored with seismic motion recorder over a considerable period of time. One such series of measurements was made on a 21-story reinforced concrete building (Building F). The building had a typical floor-to-floor height of 9 feet and an overall height of approximately 220 feet. The floor area was typically about 20,000 square feet per floor. The structural system consisted of rectangular columns and 12-1/2-inch-thick waffle slabs formed with hollow concrete blocks left in place. The blocks were omitted along column lines in each direction, resulting in 12-1/2-inch-deep beams of varying width. Architectural elements included drywalls on metal studs as room dividers and concrete block filler walls around the central elevator and stair core.

When the construction of this building was completed in 1964, a C&GS-1 system was placed on its roof, and over 40 UNEs were subsequently monitored. NGC-21 and L-7 systems were also used on occasion to obtain measurement of multi-level response. Figure 4.14 shows a 10-year history of acceleration measurements on the roof and corresponding response period of the building. The figure shows a gradual lengthening of the building period. Earlier measurements showed a period of 1.4 seconds, but later measurements for approximately the same roof acceleration showed a period of about 2 seconds. Building response was measured also during the San Fernando earthquake of February 9, 1971, which had its epicenter about 400 km from Las Vegas. A roof acceleration of 0.016g was recorded at a period of 2.1 seconds. The motion was about one-half of the largest acceleration recorded on this building. Although the bare-frame period was not obtained, it was calculated to be about 3 seconds.

Variation in Period with Amplitude of Motion: A basic assumption in structure design and analysis is that a structure will respond linearly until the force level exceeds the elastic limit of the structural elements. However the field measurements indicate that there can be a considerable amount of variation, even at relatively small amplitude of motion, depending on the structural system and the architectural materials used in the construction. This variation has been observed during the measurements of the 4-story test structures (as discussed in section 4.2) and of the high-rise buildings in Las Vegas.

Influence of Architectural Materials on Response: The influence of architectural elements was mentioned in the above discussion on several buildings. However, a more dramatic influence was observed on measurements taken on a concrete frame building with drywall partitions. Building MM: In typical construction practices, the architectural elements are installed soon after the structural framing of a particular floor is completed. The work generally proceeds simultaneously with the architectural work being performed several floors below newly constructed structural floors. A whole building rarely exists in a bare-frame condition. However, for Building MM, response measurements were obtained when the structural system was completed, but before any architectural partitions were installed and also during the installation of these architectural partitions were installed.

The building was constructed in 1975 as a moment-resisting reinforced concrete space-frame structural systems. It had 15 stories with a typical floor-to-floor height of 9 ft, 16-ft first story, and overall height of approximately 150 ft. The floor area was typically 8,000 ft² per floor. The structural system consisted of square columns and 8-in.-thick reinforced-concrete post-tensioned flatplate floors. The architectural systems were drywall on metal studs spanning from floor to ceiling in the interior and glass curtain walls on the exterior. The arrangement of partitions was identical from the second floor to the fifteenth floor. The first floor had about one-half as many partitions as the upper floors.

The response and ground level motions were monitored by the L-7 velocity meters. The building was first measured for its response to ground motion when the structural framing was completed and before any architectural elements were installed. The next measurements were taken when architectural elements were installed on the second and the third floors only. After the building was completed, several measurements of response were taken. Although period measurements alone do not completely describe dynamic response characteristics of a building, they do indicate change in building characteristics if periods vary from one UNE to another. The period-amplitude relationships were obtained from the recordings (Figure 4.14). The points (circles) on the extreme right represent the observation for the bare-frame building with a period of about 2.6 sec. For the measurements with two floors of architec-

tural elements installed, there was a period reduction of about 10% from the bare-frame. The measurements after the building was completed showed a period of about 1.8 sec, about a 40% reduction in period, reflecting the contribution of architectural partitions to lateral stiffness.

The recordings made on the bare-frame building for the first event showed an unusually long fundamental period of 2.4 sec on the average, with the corresponding damping value at about 2% of critical. The peak roof motion was 2.6 cm/sec, responding predominantly in the fundamental mode of vibration, and the equivalent acceleration and displacement were 0.007g and 1.0 cm, respectively.

The building period was considerably shorter during the second event: the 2.4 sec average period of the bare-frame building had reduced to an average of 1.8 sec when the architectural system was in place--a 25% reduction in period. A corresponding damping value was determined to be about 5% of critical, a significant increase over the damping of the bare-frame building. At this level of motion, the partitions had added mass and stiffness as expected, as well as energy-absorbing capability.

Determination of Damping Values: Response data for Las Vegas high-rises were also evaluated to determine equivalent linear viscous damping ratios. While the concept of equivalent viscous damping is reasonably applicable for systems that are slightly nonlinear, for grossly nonlinear systems such as elastoplastic systems the arbitrary choice of an equivalent linear stiffness can lead to widely varying damping ratio determinations. Since damping determinations for Las Vegas high rises were made from low-strain tests that involve only slightly nonlinear response, they are believed to be reasonably appropriate, at least for use in threshold damage predictions.

4.2.5 Dynamic Response of Bridges

Thirty one (31) bridges that were subjected to relatively high peak acceleration levels ($> 0.1g$) for three UNEs were examined both before and after the events (A52). UNEs were the SALMON event (Mississippi), the GASBUGGY event (New Mexico), and the RULISON event (Colorado).

The surveyed bridges exhibited only normal deterioration and no evidence of event-related damage, confirming the absence of reported damage at the time of the respective events. This suggests that high acceleration levels by themselves are not sufficient to cause damage. Earthquake motion which has caused bridge damage is normally characterized by peak ground accelerations similar to those from the above nuclear events, but having considerably greater displacement amplitudes and longer durations. A more extensive discussion of the bridge study is set forth in Section 6.4.3.

4.3 CHEMICAL EXPLOSIONS

In the period from 1969 to 1977 a number of large chemical explosion projects were planned and conducted in the western United States. These projects involved either direct excavation for civil engineering work or else in-place fracturing of underground ore bodies for enhanced extraction of minerals. Blume's interest, and DOE/NV support for Blume and other DOE/NV consultant involvement, arose from the opportunity to obtain additional data on ground motion effects on structures. In each case of interest small residential or industrial buildings were located in close proximity to the explosion point and real damage was expected to occur.

4.3.1 Trinidad, November 1970

The first of these projects was Trinidad, a dam and reservoir project of the US Army Corps of Engineers near Trinidad, Colorado. Because the town of Sopris, Colorado, would eventually be flooded by this reservoir, the U.S. Government purchased all of the buildings, thus giving Blume the opportunity to perform tests on several residential structures. Prior to the reservoir construction, the U.S. Army Engineers Nuclear Cratering Group conducted four series of chemical explosive cratering experiments for the purposes of improving its capability for excavating large quantities of earth and rock and for obtaining information concerning side effects of cratering detonations.

Test Series Delta, which included three tests designated 1 through 3, was executed in close proximity to Sopris and therefore provided useful ground motion and building response data. The fourth test in Series D was located too far away to provide meaningful data. The residential building dynamic tests and measurements of response to ground motion were obtained during the week of November 17, 1970 (A38, A54). Blume personnel provided the technical direction for obtaining ground and building response data. The Special Projects Party and the U.S. Army Corps of Engineers provided personnel and equipment for measuring and recording the building response and ground motion data (C5).

At the time the cratering experiments were being conducted, four houses in Sopris were dynamically tested to determine their frequency, damping, and mode shape characteristics. To demonstrate the low-rise residential building motions can be predicted from recorded ground motions, the observed building relative responses (between the ceiling and floor levels) were compared with peak relative velocity values calculated using the recorded ground motions as input.

The houses chosen for the testing were about 40 to 50 years old and in poor condition because they had

not been maintained for a number of years. Their original construction involved a high proportion of do-it-yourself work and even when new their structural integrity was not comparable with current construction.

A comparison was made between the measured peak relative responses of the four Trinidad buildings and the responses predicted from the recorded ground motions, to examine how well a known ground motion at a central location could be used to predict the response of a group of buildings. Conveniently, the four Trinidad buildings were grouped near two different ground motion stations. The comparison examined the uncertainty associated with predicting building response and the grouping of buildings with common ground stations.

Major conclusions regarding the dynamic response of the Sopris structures were as follows:

- 1) Building-as-a-unit vibration modes can be clearly seen even for 1-story residential type buildings.
- 2) The observed building-as-a-unit fundamental periods of 0.1 to 0.19 seconds for the four buildings tested appear slightly long. The most probable explanation for these "longer" periods lies in the poor quality construction, the age of the buildings (40 to 50 years), their deteriorated condition, and soil foundation conditions.
- 3) For three of the four buildings tested, the building-as-a-unit observed damping was between 5% and 6% of critical. The fourth building exhibited an anomalously high effective damping, which is most probably atypical.
- 4) Observed damping for chimneys typical of residential buildings range from 3.0% to 6.5% of critical. Two of the four chimneys monitored reflect 3.0% damping.
- 5) Observed chimney response for these dynamic tests showed that the intensity of motion was high relative to associated building motion and that there was a detectable amount of independent motion between the chimney which exhibited an extremely high motion intensity relative to the associated house. The observed independent building and chimney damping were 5% and 3% respectively.

4.3.2 Old Reliable Mine, November 1972

In March, 1972, a chemical explosion of 2,000 tons of ANFO (ammonium nitrate fuel oil) was detonated at the Old Reliable Mine about 45 miles northeast of Tucson, Arizona. This project was conducted under the sponsorship of DuPont, with DOE/NV support through participation by Blume and by the Special Project Party. Observers from Lawrence Livermore Laboratory, the Corps of Engineers, DOE/NV, and SNL were also present. Ground and structural measurements were obtained from the concrete block ranch house at Sister's Ranch near the blast site, and at a tailings dike and fill at Magna Mine near San Manuel, Arizona. SNL also operated microbarographs near the Sister's Ranch structure to obtain airblast measurements for consideration of ground motion and airblast loadings in that structure.

4.3.3 Zonia, April 1973

The McAlester Fuel Company of Kirkland, Arizona, retained the Blume company in 1972 for consideration on structural effects of very large chemical explosions planned for the Zonia Mine, about 20 miles southwest of Prescott, Arizona. Calibration explosions were conducted on December 6 and 7, 1972. The major explosion of 2,000 tons of ANFO (ammonium nitrate fuel oil) took place on April 19, 1973. This explosion was separated by a delay of a few milliseconds so that the maximum instantaneous explosion was reduced to 80 tons of ANFO.

DOE/NV supported participation by a few Blume personnel, the Special Project Party (USGS Special Projects Party from Las Vegas) and by the DOE/NV ground motion contractor, ERC, to obtain data on various kinds of industrial equipment response to strong ground motion (C23). Following the large April 19, 1973 blast two others took place, on February 14 and May 24, 1974.

Instrumentation deployment for the calibration explosions and for all three of the large detonations is described in Table 4.2. Instrumentation was deployed at various times by URS Research Corporation (URS), Blume, U.S. Geological Survey (USGS), and SNL.

Explosion data for the 3 large explosions are set forth in Table 4.2. Millisecond delays were incorporated in these explosions so that the detonation of the entire tonnage was not simultaneous.

In May 1973 the National Science Foundation approved a proposal by Blume to analyze existing data and to acquire additional data from subsequent explosions at the Zonia Mine. The intent of the study was to provide information on the response of industrial structures to strong ground motion, and to

TABLE 4.2
ZONIA MINE EXPLOSION DATA

CALIBRATION SHOTS

Identification	Date	Charge Weight	Instrumentation
#1	December 6, 1972	1 @ 1.25 T	URS
#2	"	15 @ 1.25 T (5 ms delays)	URS, Blume
#3	"	5 @ 3.75 T (5 ms delays)	URS, Blume
#4	"	1 @ 18.75 T	URS, Blume

TESTS

Identification	Date	Charge Weight	Instrumentation
Zonia #1	April 19, 1973	2070 Tons	USGS, SNL, Blume
Zonia #2a	February 14, 1974	445 Tons	SNL ^a , Blume
Zonia #2b	May 24, 1974	772 Tons	SNL ^b

Notes:

URS Velocity meters; 3 orthogonal components at each of 4 stations; ground at plant, SO₂ tower, sulfur tank and sulfur burner.

Blume Sprengnether displacement meter; 3 orthogonal components; ground station.

USGS L-7 velocity meters, 3 orthogonal components (unless indicated otherwise) at each of 11 stations: ground at plant, SO₂ tower, waste heat boiler, pipe from boiler to burner, sulfur burner, sulfur tank, administration building (12 components), 3 free-field stations, VA hospital Prescott, Arizona (6 components).

SNL^a Accelerometers; 3 orthogonal components at each of 3 stations; ground station near sulphur burner, waste heat boiler, pipe from burner to boiler; 2 horizontal components at exhaust end of sulfur burner, 1 horizontal component at injection end of sulfur burner.

SNL^b Accelerometers; 3 orthogonal components at each of 9 stations, 3 ground stations each 0.5 km N, E, and W of shot point; 3 ground station each 1.0 km N, E, W of shot point; 3y ground stations each 2.0 km N, E, and W of shot point.

assist in the design of damage-prevention measures for such structures exposed to similar ground motion. The justification for the NSF study arose from the active programs being conducted and planned for similar mineral production program in the western United States. A summary of the resulting data from the Zonia Mine study was submitted to the National Science Foundation in a letter report dated October 29, 1974.

4.3.4 Cerrillos, January 1977

On January 21, 1977, a pilot test blast was conducted by Occidental Minerals Corporation in the Mina del Terra deposit, north of Cerrillos, New Mexico, in order to fracture an underground ore body to be used as a test of in-situ copper leaching. This prototype blast had a nominal total charge of 50 tons (actually 104,000 lb) separated by delays into several stages with a maximum weight per stage of approximately 10,000 lb. All holes were loaded with ANFO. Because the value of such data to the DOE/NV seismic program, SNL participated in the instrumentation program. Seismic response data were also acquired by Blume for use in DOE/NV structural programs.

During this test, seismic response was recorded to accomplish another purpose of the blast; to provide seismic calibration data for structural response estimates for planned future larger blasts. A series of small calibrator explosions was fired in January, 1975, to provide preliminary information on ground motion response.

A variety of seismic instrumentation was fielded for the blast. Blume blast monitoring equipment consisted of two independent systems, each using six velocity geophones (two triaxial arrays) recording on a six-channel direct-write oscillograph. Blume instruments were deployed to acquire data on response of two adobe structures in Cerrillos -- the Mitchell house and the Adobe Antique Shop. At the Mitchell house, one triaxial array measured the building response at an exterior corner roof location on an older adobe wing; the other triaxial station was located on the ground below to record the ground motion excitation of the structure. The second Blume seismograph system was installed at the Adobe Antique Shop with a similar deployment of the instruments, except that the ground station consisted of one horizontal and one vertical geophone. The Adobe Antique Shop building was relatively new and was selected to provide a comparison between the structural response of adobe structures of different ages. Excellent records of ground motion and structural response were obtained for both structures. In addition, a record of the ground motion and building response generated by a passing Amtrak train were obtained at the Mitchell house.

Four calibration explosions took place in January 1977, prior to the prototype blast. These smaller explosions employed about 5000 lbs of explosive each. SNL recorded ground motions from these calibration shots, and the resulting data were used for instrument planning for the prototype shot.

Because of interest in seismic motion generated by large millisecond-delayed blasts, and to provide a field check of a seismic monitoring system, SNL monitored the prototype blast at five locations, approximately 0.5 and 1.0 kilometers east of the blast, and approximately 0.5, 1.0, and 2.0 kilometers south of the blast. All SNL stations were unoccupied and telemetered their data to a recording van positioned south of Cerrillos on the highway to Madrid, NM. Instrumentation at each station consisted of triaxially mounted accelerometers plus a vertical HS-10 velocity transducer. In addition, the station at 1.0 kilometers east had a redundant set of instrumentation designated as East 1.0 km Special.

Table 4.3 contains data recorded by SNL during the calibration shots. Table 4.4 contains peak data recorded by Blume during the 50-ton prototype blast. Corner roof stations were selected to measure the structural response of the two adobe structures monitored in Cerrillos -- the Mitchell house and the Adobe Antique Shop.

Plotted data for transverse (nominally east-west) motion at the Mitchell house, during the first second of motion showed pronounced amplification of the ground excitation at a period of about 0.13 inches/second, which was excited by about 7 ground motion pulses in this period range, none of which had an amplitude exceeding 0.03 inches/second. This pronounced amplification indicated that the fundamental resonant period of the structure was close to 0.105 seconds, and at these low amplitudes the structure had low damping.

The ground motion records from the 50-ton prototype blast refined and confirmed previous estimates of ground motions that would be produced by any larger blasts in the Cerrillo hills. It appeared that structural response of buildings in the Cerrillos community could be a controlling factor in the design of the larger blasts, partly because of the thick alluvial deposits beneath Cerrillos were causing a marked amplification of the surface waves. Such amplification increased the probability of damage to structures from larger explosions.

The monitoring of two adobe structures in Cerrillos indicated that adobe structures in good condition and with framed roofs responded in a manner similar to other low-rise structures that have been studied for the correlation of ground motion with damage; i.e., their fundamental periods in the horizontal

TABLE 4.3
CERRILLOS PEAK PARTICLE VELOCITY - CALIBRATION TEST

Data recorded by SNL

Sta- tion	Com- po- nent*	Shot #1		Shot #2		Shot #3		Shot #4	
		Dis- tance (ft)	PPV (in/ sec)	Dis- tance (ft)	PPV (in/ sec)	Dis- tance (ft)	PPV (in/ sec)	Dis- tance (ft)	PPV (in/ sec)
1	V R T					1140	0.22 0.28 0.16	1145	0.28 0.30 0.24
2	V R T	660	1.02 1.50 1.18	525	1.14 1.57 1.02	560	2.01 1.81 --	565	2.13 1.69 1.34
3	V R T	1385	0.28 0.24 0.28	1240	0.39 0.29 0.35				
4	V R T	1530	0.26 0.27 0.21	1390	0.22 0.22 0.27				
5	V R T	9850	0.019 0.020 0.021	9775	0.020 0.022 0.026	9650	0.014 0.017 0.015	9575	0.017 0.037 0.032

*Component: V - vertical; R - radial; T - transverse

TABLE 4.4
CERRILLOS PEAK PARTICLE VELOCITY - PROTOTYPE TEST

1. Data recorded by Blume

Station	Component	Location	Distance (ft)	Peak Particle Velocity (in/sec)
1	V R T	Mitchell House	9300	0.044 0.106 0.074
2	V R T	Adobe Antique Shop	9600	0.032 0.098 --
3	V R T	Olson House	7900	0.023 0.018 0.015
4	V R T	Route 14 Fire Station	14840	0.0055 0.0035 0.0040

2. Data recorded by SNL

Station	Component	Location	Distance (ft)	Peak Particle Velocity (in/sec)
0.5E	V R T	--	1450	0.38 0.30 0.57
1.0E	V R T	--	3250	0.07 0.13 0.14
0.5S	V R T	--	1950	0.29 0.25 0.13
1.0S	V R T	--	3800	0.22 0.32 0.20
2.0S	V R T	--	6560	0.070 0.096 0.074

plane were in the range of 0.05 to 0.3 seconds.

4.3.5 Other Chemical Explosion Projects

Besides the project already described, Blume and SNL were involved in a number of other projects involving large chemical explosions creating significant ground motions and airblast for nearby structures. Both Blume and SNL were involved in a near-shore channel excavating project in 1969 in Hawaii entitled TUG BOAT. Data obtained for the U.S. Army Corps of Engineers project were used for technical reporting and were also available to DOE/NV for application to the structural response program being conducted by Blume.

Blume was involved as a structural response consultant to Occidental Mineral Corporation for a planned 1976 blast near Miami, Arizona in 1976. While these explosions were not large, ranging from 3000 to 7500 lbs of explosive, they were located in close proximity to a tailings pond. Possible damage complaint from residents of Miami, Arizona and a possible liquefaction of the retaining dike for the tailings pond were items of concern.

Other similar projects in which Blume was involved included ore body fracturing in Mexico, quarry blasting in Alabama and in Mexico, and oil shale fracturing in Colorado. These projects all provided data for structural response investigations for DOE/NV although no direct participation or sponsorship by DOE/NV was involved. The knowledge acquired from these studies of explosion-induced ground motion and structural response permitted Blume to provide highly-successful consultation to the U.S. Corps of Engineers for replacement of Bankhead Lock on Black Warrior River in Alabama. This project involved excavation blasting in close proximity to the existing locks without any damage to the locks, and with minimal interference with the on-going operation of the existing lock (B88).

4.4 POST-EARTHQUAKE INVESTIGATIONS

Field investigations of earthquake effects were conducted by Blume for a number of earthquakes during the period from 1964 to 1971, as part of the work being undertaken for DOE/NV. The applicability of these investigations arose because of the great similarity between ground motion from earthquakes and that from underground nuclear explosions. The fundamental objective was to evaluate and refine procedures for predicting structural response and damage from ground motion, using data from real seismic damage.

In chronological order, the earthquakes investigated were:

- Anchorage, Alaska, March 27, 1964 (A1)
- Dulce, New Mexico, January 22, 1966 (A2)
- Parkfield, California, June 27 to 29, 1966 (3 separate events) (A6)
- Truckee, California, September 12, 1966 (A17)
- Colombia-Venezuela; February 9, 1967 (Colombia), and July 29, 1967 (Colombia and Venezuela) (A9)
- Fairbanks, Alaska, June 21, 1967 (A26)
- Mexico City, August 2, 1968
- San Fernando, California, February 9, 1971 (A37, A39, A40, A44, A46, A55, B21, B55, B63)
- Santa Barbara, California, August 13, 1978 (A64)

Investigation findings are summarized in the following discussions. A great deal of detailed information on building damage from these earthquakes is available in the original Blume reports on the investigations. Brief summaries are presented in the following discussion, with more extended treatment of the better-recorded San Fernando and Santa Barbara earthquakes.

4.4.1 Anchorage, Alaska - March 27, 1964

The magnitude of the major shock in the March 27, 1964 earthquake was estimated from U.S. seismographic records at about 8.4 to 8.6 on the Richter scale. The epicenter of the earthquake had been established at about latitude 61° north and longitude 147°30' west in northern Prince William Sound on the southern coast of Alaska. Focal depth was estimated to be about 20 kilometers below the surface, a relatively shallow depth.

No strong-motion seismograph instruments were in operation in Anchorage during the earthquake and, consequently, no reliable estimate of ground motion period or amplitude could be made. However, there are some indications that a dominant period of motion may have been about one second.

The epicenter was located about 75 to 80 miles' distance from Anchorage. In spite of this considerable spatial separation, long-period seismic waves, which are least attenuated by distance, arrived at Anchorage with sufficient energy to accomplish extensive damage. The effects of earthquake motion on sedimentary layers underlying the city resulted in a number of landslides in inhabited areas. Tall structures in Anchorage with natural periods of vibrations similar to that of the long-period seismic waves experienced resonant motion which exceeded the strength of some elements, and damage occurred. Shorter structures with higher natural frequencies did not approach resonance and did not incur damage through this mechanism. All structures located in the landslide areas were damaged to some degree.

The Uniform Building Code was the governing building code in the City of Anchorage for some time, but the Zone 3 seismic requirements had been in force only since 1958. Except for the Mt. McKinley building and the 1200 L apartments, which were built in the early 1950's, most of the tall buildings in Anchorage in 1964 had been constructed after 1958 and were therefore subject to the Uniform Building Code Zone 3 requirements.

Anchorage's structures in 1964 represented a wide range of types of construction; concrete and steel frames with and without structural shear walls; light framing with shear elements of masonry block of wood or a combination thereof; box structures, frame structures, three-walled structures (store fronts), horizontally braced systems, and combinations thereof; roof and floor slabs of poured-in-place concrete, metal siding and masonry block.

Damage to the structures resulted primarily from one or more of the following causes:

1. Large permanent ground displacements from landslides.
2. Small permanent ground displacements in the neighborhood of one inch. When a displacement such as this occurs within a building or between two connecting buildings, damage is expected to take place.

3. Location of openings in concrete shear resisting elements which either resulted in weakening the wall directly or resulted in the wall acting as several individual vertical resisting elements connected by horizontal elements.
4. Participation of architectural elements and nonstructurally designed elements in the following sequence; changing the relative rigidity distribution, resisting lateral forces, failing, and then transferring the forces to structural elements. In other cases the architectural elements might have provided the extra resistance required to prevent damage.
5. Large torsional motion created in some structures due to the relatively large distance between the center of rigidity of the resisting elements and the center of gravity of the structure.
6. Inability of connection elements to absorb energy without failing. This applied primarily to structures of precast units. For many cases these connections might have been subject to forces much greater than code design, but, if these connections had been ductile, total collapse of the structures might have been avoided.
7. Connection details weakening structural elements. Openings in a concrete shear wall due to connection details for some precast elements might have reduced the strength of the shear wall enough for it to fail.
8. Quality of workmanship and materials appeared to be a factor in the degree as well as the cause of failure. Failures at construction joints were obvious in many of the damaged structures.

4.4.2 Dulce, New Mexico - January 22, 1966

An earthquake on January 22, 1966 caused minor damage in Dulce and Lumberton, New Mexico. The magnitude of the major shock was estimated, from 15 recording seismographic stations, at 5.5 on the Richter scale. The intensity of the shock in the area of the epicenter was rated at 7 on the Modified Mercalli Index (MMI).

The buildings suffering the greatest amount of damage were those composed chiefly of adobe brick,

concrete block and stucco. Little or no reinforcing was observed in any of the damaged structures where failure had occurred.

Numerous cases of glass breakage and cracking were observed in the town of Dulce, especially in the school buildings. Glass damage in these structures was attributed to the large wall areas weakened by window expanse and placement.

Damage to window glass was observed to occur in two ways, one by bending out of the window wall element and thus popping out of the window glass in a diagonal pattern of shear planes and splintered glass, the other occurred in the form of diagonal cracks that appeared to be the result of racking of the window sash within the wall element.

There were numerous instances of damage to concrete slabs on grade. Damage to these slabs was generally confined to the opening up of cold joints, expansion joints, and weakened plane joints. In finished floor areas, this movement resulted in the tearing up of the floor finish material, generally asphalt tile.

Some plaster ceilings on metal lath fell partially or completely at the Indian Agency (two-story cement brick buildings). Many glass windows (approximately 200) were broken at the Dulce School and at the Indian Agency. Both large and small windows broke, in a random fashion.

Brick walls cracked at corners, around lintels, and over openings. Cracks were typically vertical or diagonal.

Concrete block walls cracked at expansion joints, around lintels, upward diagonally from doors and windows, at intersections of blocks and roof beams, at intersections of partition walls, horizontally at tops of windows and under bond beams, and diagonally above openings.

Very little damage was visible in reinforced concrete slabs except:

- (1) At expansion joints, where tile was dislodged and joints opened, and

- (2) As aggravation of an existing settlement problem. This was seen in downward displacement of floor slabs in areas of pre-existing settlement effects.

Slight vertical cracking was seen in reinforced concrete foundation walls. Openings seen in a few gyp-board panels at typical joints. Beaver-board partitions opened or moved along joints.

Broken lines occurred at connections to boilers. Buried water line leakage is suspected.

Lower stories of two-story structure; low-wall structures and basements, were affected less than upper stories of two-story structures and high wall structures. South walls suffered more damage than north walls. East walls suffered more damage than west walls. Two-story buildings suffered more damage than low-wall one-story buildings.

Primary damage concentration was observed in masonry construction. Extensive cracking occurred, most of which was inconsequential insofar as structural performance is concerned but will still require expensive repair.

Prefabricated steel buildings showed essentially no damage other than a few minor cracks in interior drywall partitions. Frame buildings suffered dislocation of porches, some failure of masonry pier supports, and minor cracking in dry wall partitions.

4.4.3 Parkfield, California - June 27 to 29, 1966

A series of 3 earthquakes occurred near Parkfield, California on June 22 to 29, 1966. The first two occurred at 9:09 and 9:22 Pacific Daylight Time on June 27 and had Richter magnitudes of 5.3 and 5.5, respectively. A major aftershock, of Richter magnitude 5.0, took place at 12:35 PST on June 29. All three had approximately the same epicenter, about 10 kilometers northwest of Parkfield. Modified Mercalli Intensity in the Parkfield areas was rated at VII.

Structural damage was largely confined to broken glass, cracks in masonry and plaster, collapsed brick chimneys, cracked swimming pools, road bridge slabs shifted at abutments, cracks in highway pavements, fallen bric-a-brac, and displaced wooden structures. The main shock was felt over an area of about 20,000 square miles. Sixteen strong motion stations in the vicinity were instrumented with U.S. Coast and Geodetic Survey seismoscopes and Stations 2, 5, 8 and 12 were equipped with UED Model AR-240 accelerographs. Peak accelerations recorded on the accelerographs were near 0.50g, a

remarkably high acceleration value for a relatively small earthquake. Structures in the area were predominantly older rural buildings, constructed without regard to building code requirements.

The nearby small towns of Shandon and Cholame, which are a few kilometers southwest of the surface breakage at its southernmost extent experienced slight damage consisting largely of fallen objects and cracked concrete. Windows were broken in Cholame and between Shandon and Cholame.

In Parkfield, a few reopened or extended brick wall cracks and cracked windows were observed in the State Division of Forestry buildings. Eyewitness accounts indicated that the motion was quite violent.

An exterior cabana attached to a house trailer moved 4 feet away from the trailer. Shelves collapsed in a nearby shed. Extensive damage was caused to a wooden community church. Many brick chimneys suffered damage.

Plaster cracking was reported at the police station in San Luis Obispo. Cracks in plaster finish at a wall-ceiling intersection were observed in the Shandon Valley Cafe. The Shandon postmaster reported cracking in his house.

Windows were broken in Parkfield; in residences, at a forestry station, and in the community church. One injury from flying glass was reported.

Cement brick was cracked (old cracks reopened and extended) at the State Division of Forestry dormitory and office building in Parkfield. The cracks were located at the bottom corners of a window opening. Brick chimneys in Parkfield were extensively damaged.

Displacement and cracking were observed in concrete slabs in the Cholame Cafe, a Cholame garage and a gas station, in several Parkfield road bridge decks, and in a swimming pool deck slab near Parkfield.

Damage was primarily caused to non-structural elements of buildings such as concrete slabs, brick chimneys, and windows. Much falling of objects took place. Relatively little damage occurred to structural elements. However, a wooden church structure in Parkfield experienced major damage, with one wing almost completely separated from the remainder of the building. Plumbing fixtures also were dislodged in the church, tearing loose from their fastenings.

4.4.4 Truckee, California - September 12, 1966

A moderate earthquake of Richter magnitude 5.4 (U.S. Coast and Geodetic Survey) was centered 10 kilometers northeast of Truckee, California on September 12, 1966. The earthquake caused moderate to severe damage to some buildings within a few kilometers of the epicenter and slight damage at Reno and Sacramento. Ground motion was felt as far east as Salt Lake City and as far west as San Francisco.

Calculated Richter magnitudes for the earthquake range from 5.4 (U.S. Coast and Geodetic Survey) to 6.2 to 6.5 (California Institute of Technology). The epicenter was reported by the U.S. Coast and Geodetic Survey (USC&GS) to be located at Latitude 39°24' north, Longitude 120°06' west, approximately at Boca Dam on the Little Truckee River. The focal depth was 8 kilometers.

Because damage to structures in Truckee at 10 kilometers distance and Reno at 27 kilometers distance was relatively light, it was surprising that similar slight damage occurred in Sacramento, at the considerable distance of 150 kilometers southwest of the epicenter.

A strong-motion seismograph record of Sacramento ground motion was obtained from an instrument located in the basement of the 4-story Pacific Telephone and Telegraph building, about 10 kilometers east of downtown Sacramento. Another strong-motion instrument was located on the roof of the same building. Buildings within a few kilometers of the epicenter were 1 to 2 stories in height. Damage to these structures included broken chimneys and windows. Walls and slabs were also damaged.

As usual, dry-wall cracking was the most common evidence of earthquake damage. Reno and Sacramento multiple-story structures incurred most of this damage. In Reno, such cracking was more noticeable on lower story levels, whereas in Sacramento it was more common at the higher levels of the taller structures. Most cracking occurred at wall-wall, wall-frame, or wall-ceiling intersections; or at the corners of wall openings. Unstable objects and shelf merchandise fell, at distances out to about 35 kilometers.

Damage to earth structures included slight cracking and slumping in Prosser Dam about 3 kilometers from the Epicenter, very slight cracking in Boca Dam, incipient landsliding in a highway fill on U.S. 40 near Boca Dam, and rock falls from steep slopes in the area. Some movement in the Southern Pacific Railroad fill was reported in the area.

4.4.5 1967 Colombian Earthquakes:

Blume conducted field investigations of 3 earthquakes in 1967; two in Colombia and a third near Caracas, Venezuela which had some effects in Colombia.

The work was sponsored by DOE/NV under a contractual arrangement for the study of ground shock effects associated with the proposed nuclear excavation of a sealevel interoceanic canal across the Isthmus of Panama. Three earthquakes were investigated; the first, on February 9, 1967, had a Richter magnitude of 7.3 to 7.5. The second, on July 29, had a Richter magnitude of 6.1 to 6.3. Another earthquake, also on July 29 was centered about 50 km northwest of Caracas, Venezuela, with a Richter magnitude of 6.3 to 6.5. While the Caracas earthquake occurred outside the region of interest for the interoceanic canal project, it was recorded in Bogota, and valuable information on the response of South American high-rise structures in both Caracas and Bogota was thereby afforded. The party fielded for the July 29 Colombian earthquake study also made a brief visit to Caracas to acquire this additional information.

The earthquake of February 9, 1967 was centered about 40 km northeast of Neiva in the Department of Huila in southwestern Colombia, while that of July 29, 1967 was centered 35 km southeast of Bucaramanga in the Department of Santander, in north central Colombia. These cities both lie within the valley of the Magdalena River, between the Cordillera Central and Cordillera Oriental, almost equally distant northeast and southwest from Bogota. The epicenter of the July 29 Caracas earthquake was 5 km off the Caribbean coast, about 50 km northwest of Caracas, and a little over 1000 km northeast of Bogota.

Table 4.5 lists some of the standard seismological data for the earthquakes. Most of these data were provided by Dr. Bruce Bolt of the University of California Seismographic Station, Department of Geology and Geophysics, as part of a contractual arrangement for seismological consultation for the interoceanic canal project.

The February 9 earthquake was not well recorded in Bogota. Better records were obtained for the July 29, 1967 Bucaramanga and Caracas earthquakes. At the time of the February 9 earthquake, plans were already in progress to install a number of strongmotion accelerographs in Colombia and Panama. The intent was to record seismic data for assistance in predicting ground effects from the proposed nuclear excavation of a new sea level canal across the Isthmus of Panama. In Bogota, the instrument was located on the ground floor of the two-story National Museum on Carrera 7 between Calle 28 and Calle

TABLE 4.5
SEISMOLOGICAL DATA, 1967 EARTHQUAKES

	February 9 (Neiva)	July 29 (Bucaramanga)	July 29 (Caracas)
Local Time (24-hour clock)	1025	0524	2000
Epicenter: Latitude Longitude	2.9 N 74.9W	6.8 N 73.0 W	10.5 N 67.3 W
Focal Depth	58 km	157 km	10 - 15 km
Richter Magnitude (U.S., Berkeley)	7.3 - 7.5	6.1 - 6.3	6.3 - 6.5
Sn-Pn Time	33 sec	31 sec	10 sec
Distance to Bogata	300 km	270 km	1000 km

29, about 1500 meters due south of the Universidad Javeriana. A record from this instrument was obtained during the July 29 earthquake. Table 4.6 is a summary of peak motions in Bogota from the earthquakes.

Of the three earthquakes discussed, that in Caracas on July 29 caused by far the greatest damage and loss of life. While this was not the strongest earthquake, having a magnitude of 6.3 to 6.5 compared with the Neiva magnitude of 7.3 to 7.5, it was the closest of the three to a major center of population, being at a slant range (hypocentral distance) of about 50 km from Caracas. The Neiva earthquake was at 70 km (slant range) from Neiva, and the Bucaramanga earthquake was at 160 km (slant range) from Bucaramanga. Also, Caracas with a population of almost 2,000,000 offered a much greater exposure of structures than did Neiva with 100,000 or Bucaramanga with about 270,000. Caracas, for instance, has about 1000 structures from 10 to 40 stories in height. Neiva's tallest building, with the exception of the Cathedral, was eight stories; Bucaramanga's tallest was a single 10-story structure.

Table 4.7 presents a summary of strong motion effects for the three earthquakes. The data are derived from newspaper accounts, and from BSSA 1967 and 1968.

The February 9 earthquake caused extensive damage in the Department of Huila, particularly in smaller towns near the epicenter and in Neiva, where a Modified Mercalli rating of IX was assigned. Total damage cost was estimated to be about \$20,000,000; 98 persons died in the earthquake.

In Bogota, thirty structures were seriously damaged, including banks, schools, houses, and churches. At least fifty walls failed. The church of San Juan de Dios was seriously damaged; other churches suffered lesser damage. Modern high-rise structures in Bogota suffered minor damage. Typical "pounding" damage occurred in a number of downtown locations where adjacent buildings of different fundamental periods swayed sufficiently to collide with each other.

The July 29 earthquake, although smaller than that of February 9, caused destruction, injury, and deaths throughout the eastern central portion of Colombia. Twenty persons died in the Departments of Santander, Boyaca, and Tolima. Although there was moderate or severe damage in Fativiva and Zipaquirá, and slight damage in Bogota, no deaths were reported in these cities. Damage also occurred in Medellin and Manizales, as well as in many smaller cities and towns, primarily in the Magdalena River Valley. Total damage costs for the July 29 earthquake are estimated to be \$2 million, with severest damage being incurred by churches.

TABLE 4.6
PEAK GROUND MOTION IN BOGOTA

	February 9, 1967 (Neiva) Rock Alluvium		July 29, 1967 (Bucaramanga) Rock Alluvium		July 29, 1967 (Caracas) Rock Alluvium	
Peak Acceleration, g	0.014*	0.05*	0.004	0.015	0.0002	0.001
Peak Displacement, cm	0.3*	1.5*	0.03	0.15	0.02	0.10

* estimated

TABLE 4.7
STRONG MOTION EFFECTS

	February 9, 1967 (Neiva)	July 29, 1967 (Bucaramanga)	July 29, 1967 (Caracus)
Duration of strong motion, sec	90 +	50 - 60	30 - 40
Total damage cost, US\$	20×10^6	20×10^6	50×10^6
Number of deaths	98	20	250 +
Number of injured	300	150	1500 +
Maximum Mercalli intensity	IX	VIII	IX
Maximum range of perceptible motion, km	1200	800	1000

The Caracas earthquake damage was particularly notable because of several cases of extreme damage to modern structures, supposedly designed and constructed to be earthquake resistant. Four multi-story apartment buildings collapsed in the Los Palos Grande and Altamira sections of Caracas, as did the top four floors of a 10-story apartment building in the seacoast suburb, Macuto, 7 km north of Caracas. Severe damage was also concentrated in the northwest portion of Caracas, where many one- and two-story adobe buildings collapsed. More than 250 people were killed and total damage costs were estimated at \$50,000,000.

Earthquake-resistant design in Caracas was based on certain U.S. standards and practice, with a building code requirement about equivalent to the Uniform Building Code Seismic Zone 2 design requirements. This was a moderately rigorous requirement. Design and construction practices were apparently sound by conventional standards. Opinions have been advanced that a probable cause of the unexpectedly severe damage from the moderate earthquake experienced was that the use of rigid hollow clay tile filler walls in upper stories changed the dynamic characteristics of the structure quite drastically from the assumed bareframe properties on which design was based. Large overturning forces then developed in lower portions of the structure not designed for such loading, and severe damage or collapse ensued.

4.4.6 Fairbanks, Alaska - June 21, 1967

An earthquake with three primary shocks of Richter magnitudes 5.4, 5.6, and 5.4 (U.S. Coast and Geodetic Survey) occurred 15 kilometers east southeast of Fairbanks, Alaska, on June 21, 1967.

The three shocks occurred at 1804:49.5, 1813:02.9, and 1824:45.7, Greenwich Mean Time. A maximum Modified Mercalli Intensity of VII was assigned to the Fairbanks earthquake by the U.S. Coast and Geodetic Survey (USC&GS). The epicenter was at latitude 64°48' north, longitude 147°42' west, with a focal depth of about 16.5 kilometers.

Ground motion data in Table 4.8 are from a seismograph in the basement of Duckering Hall, University of Alaska, which is located on Birch Creek schist bedrock. Downtown Fairbanks, on the other hand, is situated on 200 to 400 feet of river gravels, with permafrost layers at 15 to 30 feet below the surface.

Because the typical amplification of seismic ground motion by such alluvial materials exceeds that observed for bedrock, downtown Fairbanks ground motion was probably twice that measured at Duckering Hall.

TABLE 4.8 GROUND MOTION DATA
DUCKERING HALL, UNIVERSITY OF ALASKA

Shock No.	Date and Time (GMT)	Accel. (g)	Period (sec)	Accel. (g)	Period (sec)	Accel. (g)	Period (sec)
1	June 21 18:05	0.06	0.1	0.06	0.1	0.06	0.1
2	June 21 *	0.07	*	0.03	*	0.03	*
3	June 21 *	0.14	*	0.08	*	0.09	*
4	June 21 *	0.08	*	0.04	*	0.04	*

* estimated values from a faulty record

TABLE 4.9 SPECTRAL VELOCITY, DUCKERING HALL BASEMENT

Building	Stories	Structural Frame	Estimated Period (sec)	S _v (cm/sec)	Damage
Lathrop High School	2	Reinf. Concrete	0.2	7.3	Slight
Haynes Hall	3	Reinf. Concrete	0.3	20.2	Moderate
State Building	4	Reinf. Concrete	0.4	21.8	Moderate
Bassett Hospital	5	Reinf. Concrete	0.5	16.3	Slight
Power Plant No. 2	5 (equiv)	Steel	0.5	16.3	Slight
Northward Building	8	Steel	0.6	18.1	Minor
Polaris Building	11	Reinf. Concrete	0.6	18.1	Slight

The 5-percent damped spectral velocities for the N-S component of the Duckering Hall Basement record have been used in Table 4.9 to reconstruct possible 5-percent damped spectral velocities for downtown Fairbanks.

Using the peak acceleration of 0.06g shown for the first shock only and doubling it for downtown Fairbanks would indicate that the downtown area experienced peak single-component ground accelerations of about 0.12g. This value seems reasonable, since the kind and extent of damage in Fairbanks would be consistent with short-duration seismic motion having peak ground accelerations on the order of 0.10g to 0.15g.

Damage to structures in Fairbanks varied from slight to moderate, with reports of cracked and chipped plaster, horizontal and vertical wall cracks, wall partition separations, broken glass, toppled chimneys, and wall displacement. No street damage was reported in Fairbanks, although there was some earth subsidence in a pond embankment at nearby Ft. Wainwright. In the building suffering the most extensive damage, ceiling sections and light fixtures fell, and bookcases and storage shelves buckled and were toppled.

The most extensive damage from the earthquake occurred in the 4-story concrete frame State Building. In the main entrance concourse, concrete walls were cracked and split at approximately 4-foot heights. The fascia by the entrance canopy was cracked; ceramic tile and grout were removed to prevent any possible injuries. Interior damage included cracked and separated walls, cracked plaster, and toppled bookcases and shelves.

4.4.7 Mexico City - August 2, 1968

An earthquake of Richter magnitude 6.8 occurred at 14:06:43.9 (GMT) August 2, 1968 at latitude 16°36' north, longitude 97°42' west on the Oaxaca- Guerrero border. Structures were damaged in Mexico City, about 370 kilometers north of the epicenter. The earthquake had a focal depth of 40 kilometers and a maximum intensity of VII (downtown Mexico City) on the Modified Mercalli Scale.

This earthquake was the largest (greatest Richter magnitude) in Mexico since the July 1957 earthquake of Richter magnitude 7.5, centered about 275 kilometers south of Mexico City, which caused 125 deaths and damage estimated at \$150 million (U.S. dollars). Motion from the 1968 earthquake was felt over a wide area, at distances out to more than 400 kilometers from the epicenter and including Mexico City and Vera Cruz, 80 kilometers east of Mexico City. Acapulco was shaken perceptibly but no

damage was reported. A small tsunami was reported at Acapulco.

Downtown Mexico City, where most damage occurred, is founded on deep lake bed sediments. The University area, which is founded on rock, experienced no significant damage, nor did the nearby housing areas or facilities under construction for the October 1968 Olympic Games. Modified Mercalli Intensities of II to IV were assigned to these outlying areas. The earthquake caused four deaths in Mexico City, four in the State of Oaxaca, and many serious injuries according to police reports, newspaper accounts, and other sources.

Most of the communities in Oaxaca State, on whose border the epicenter was located, also experienced moderate damage. In Pinotepa Nacional, a town of 10,000 inhabitants in Oaxaca, about 5% of the houses were demolished and 50% damaged beyond safe repair.

Damage to Mexico City structures from earthquakes centered far to the south, e.g., the July 1957 earthquake (275 kilometers) and the August 1968 earthquake (370 kilometers), results from a two-fold resonance situation. The old Lake Texcoco sediment-filled basin which underlies Mexico City has a basin response peak at about 2 seconds and thus tends to reinforce the long-period ground motion which propagates with relatively little attenuation. This amplified motion is then further accentuated in the response of the tall downtown high-rise structures having natural periods near 2 seconds.

Another factor contributing to seismic damage in 1968 was prestressing caused by settlement. Highly compressible bentonitic clays derived from volcanic materials underlie downtown Mexico City, and are largely responsible for extreme settlement problems which have characterized the urban downtown area for years. Seismic damage or aggravation of existing cracking tend to occur at lower motion values when a prestressing mechanism is present, such as this settlement problem.

From observations and from other accounts, it appeared that damage during the August 1968 earthquake was concentrated to a considerable extent in structures ranging from 10 to 14 stories and was characterized by diagonal cracking in partition walls, separation at column-partition intersections, and much window damage. From the structural effects, it can be deduced that the character of motion was such that large relative horizontal displacements occurred in buildings in the downtown area having periods of approximately 1 to 2 seconds. Response spectra for Mexico City from the August 1968 earthquake, from records taken at the Tlatelolco apartment complex in the downtown area verify that the motion was indeed characterized by relatively high spectral displacements, moderate velocities, and low

accelerations. On the other hand, response spectra prepared from the ground motion recordings at the bedrock stations at the National University, and at essentially the same epicentral distance as the Tlateloco complex, showed a complete absence of the strong peaks at about 2 seconds characteristic of the downtown Mexico City spectra. Peak ground motion from the Tlateloco recording ranged up to about 0.05g, whereas those at the University area were just under 0.02g.

Other Mexican earthquakes, with Richter magnitudes of 7.0 or more have occurred since; on June 30, 1973 (7.5), Nov. 29, 1978 (7.8), Mar. 14, 1979 (7.6), Oct 25, 1981, (7.3), June 7, 1982 (7.0), Sept, 19 1985 (8.1) and Sept 21, 1985, (7.5). The 1985 earthquakes caused widespread major damage in Mexico City, with many buildings collapsed and a great loss of life.

4.4.8 San Fernando, California - February 9, 1971

The earthquake that occurred on February 9, 1971, beneath the San Gabriel Mountains, some 10 miles north-northwest of the city of San Fernando, California and 15 miles from downtown Glendale, caused severe structural failures in the immediate vicinity and some severe structural damage in Glendale. The main shock struck at 06:00:41.7 PST with a calculated magnitude of 6.6 on the Richter scale. A provisional seismological epicenter was calculated by the Seismological Laboratory, California Institute of Technology, to be at latitude 34°24.0' N, longitude 118°23.7' W with a depth of 5 to 10 miles.

This earthquake, and its more than 300 aftershocks was of special significance because of the seismic hazard it constituted in a densely populated modern environment, and because many good seismic ground motion records were obtained from this heavily-instrumented area.

Unusually high ground motion accelerations were recorded. Preliminary measurements from many of the seismograph records indicated that generally the amplitudes of the peak vertical ground acceleration components were a large percentage of the horizontal components. Analyses of the accelerogram retrieved from the Pacoima Dam, located on rock near the earthquake epicenter, indicated a single peak acceleration of 1.0g horizontally and a peak of 1.0g for the vertical direction for one cycle, the largest ever recorded from an earthquake. The record showed that during the first 12 seconds, ground motions of 0.5g to 0.7g occurred.

The primary area affected by the initial earthquake shock consisted of greater metropolitan Los Angeles and contiguous areas. Substantial, and in some instances, devastating damage occurred in the communities of Sylmar and San Fernando with lesser damage occurring in Burbank, Van Nuys, downtown Los

Angeles, Pasadena, Glendale, and other nearby cities.

There were hundreds of reports of broken windows, including plate glass in stores and large panes in high-rise buildings. Bricks and plaster cascaded into streets and cement-plaster cornice and parapet failures were prevalent throughout the Los Angeles area. Structural damage occurred to unreinforced brick masonry walls and parapets by severe diagonal cracking, which in the larger structures resulted in massive wall failures and collapse.

Because the San Fernando earthquake was particularly well-recorded by the many seismographs in and around the Los Angeles area, Blume conducted several studies on motion-damage relationship for that earthquake. Three of these studies were concerned with damage in the Glendale area and the fourth was concerned with comparative response of two essentially identical 7-story motor hotels in Van Nuys and Los Angeles.

The first Blume study and report was concerned with chimney damage in Glendale area. Records from a strong-motion instrument located in the basement of the Municipal Service Building at 633 East Broadway, Glendale, California, indicated a peak acceleration of 0.275g horizontally and a peak of 0.142g for the vertical component. Structural damage occurred to unreinforced brick masonry walls and parapets by severe diagonal cracking, which in the larger structures resulted in massive wall failures and collapse.

Because of the concentration of minor to moderate damage in Glendale, that city was of particular interest for structural response purposes. To establish motion-damage relationships it was necessary to collect damage data for the area of interest and to relate that data to the input San Fernando earthquake ground motion. Glendale had only one recording station operating for the San Fernando earthquake, and that station was located in the downtown area. For a closer geographical match between widespread damage data in the city and the causative ground motion, the ERC and the Special Projects Party initiated a study to extrapolate from the single Glendale downtown station to the outlying areas where damage data was being collected.

The approach used was to assume that a consistent relationship existed between ground motion at the Glendale downtown station and that at outlying areas. This relationship could be determined by recording aftershocks at the downtown station and at the outlying stations, analyzing the relationships between spectral ratios for several aftershocks between the downtown station and the outlying stations,

and then using these relationships to extrapolate the spectral composition at each of the outlying stations from the downtown station for the main shock (D70).

Results indicated that significant variability in the ground motion spectra occurred within the study area. The nature of the variability was consistent, however, for the several aftershocks. On that basis it was assumed that local geological response was linear, and consequently response spectra were estimated with some confidence for a number of locations which did not actually record the main shock.

The high incidence of damage to chimneys in Glendale afforded an opportunity to obtain good data for this extremely common form of earthquake damage. It was of particular interest to DOE/NV because of the prevalence of this kind of damage in off-site testing areas.

Buildings with chimneys in Glendale were idealized using two class approximations. Class 1 included all wood frame buildings that have fundamental periods between 0.05 and 0.1 seconds. Wood frame buildings with fundamental periods in the range of 0.1 and 0.2 seconds were in Class 2. Since the sample size for masonry buildings was small (24 masonry buildings), they were not considered in this investigation.

An identification number was assigned to each observed damage type. Table 4.10 shows the identifiers. The damage types were grouped according to their severity. Damage types 1, 2, and 3 were considered as slight and damage types 4 and 5 were regarded as moderate. Toppled chimneys were identified as damage types 6 and 7 and were grouped in the severe damage category.

There were 1,494 buildings in the study area, excluding masonry buildings. Some of these buildings had more than one chimney. For this study, a building with n chimneys was substituted by n similar buildings, each having only one chimney. Therefore, the total sample size was 1,769 buildings. The sample size was divided into buildings with chimneys and without chimneys, and into two building classes (Class 1 and Class 2) described above and shown on Table 4.11. About two-thirds of all buildings under consideration fell into Class 2 (period band of 0.1 to 0.2 seconds) and more than half of the buildings had chimneys.

Damage data was presented in terms of a parameter called Damage Ratio. Damage Ratio (DR) is defined by:

TABLE 4.10 DAMAGE IDENTIFICATION NUMBERS

SLIGHT			MODERATE		SEVERE	
LOOSE BRICKS	HORIZONTAL CRACKS		OFFSET SHIFT	CHIMNEY-HOUSE SEPARATION	CHIMNEY TOPPLED	
	EAVE	OTHERS			EAVE	BELOW EAVE
1	2	3	4	5	6	7

TABLE 4.11 NUMBER AND PERCENTAGE OF BUILDINGS WITH AND WITHOUT CHIMNEYS FOR EACH BUILDING CLASS

TYPE	NUMBER OF BUILDINGS *			PERCENTAGE		
	ALL BUILDINGS	WITH CHIMNEY	WITHOUT CHIMNEY	ALL BUILDINGS	WITH CHIMNEY	WITHOUT CHIMNEY
CLASS 1	597	447	150	34	25	9
CLASS 2	1172	581	591	66	33	33
TOTAL	1769 **	1028	741	100	58	42

* Masonry buildings, which are 1.7% of all buildings, have not been considered

** A building with n chimneys is replaced by n similar buildings, each having one chimney

$$DR = \frac{\text{Number of Damaged Chimneys}}{\text{Number of Chimneys}}$$

DR can be defined for all chimneys of the sample data as well as a particular group of chimneys such as brick chimneys, severely damaged chimneys, etc.. Field investigation showed that some types of chimney damage were more common than others as summarized in Table 4.12. The column indicating average damage ratio in Table 4.12 shows that most damaged chimneys cracked, toppled at their connection to buildings, or separated from the houses leaving residual gaps, in the wake of the earthquake. This type of damage was partly caused by the impact exerted by the structure to the chimney at their connection. Although the structure and the chimney might vibrate as a unit for a few cycles, because of a weak connection their oscillations become gradually more independent. Therefore, the impact phenomenon is inevitable in the case of relatively high amplitude motion.

Table 4.13 summarizes information concerning the number of chimneys of different construction materials found at various locations on buildings. Table 4.14 compares their damage ratios.

The chimney-building connection has great influence on the system response and consequently affects the extent of damage to chimneys. The impact of the building with the chimney is more damaging if the chimney is located at the perimeter of the building. Table 4.15 shows that, irrespective of the type of chimney, damage to center chimneys was much less than that of chimneys located at the end or at the eaves of houses.

Another important factor influencing damage ratio is the condition of the chimney. The force carrying capacity of a chimney in poor condition may not be adequate to resist the applied dynamic lateral loads. Weathering is one of the main causes of decreasing the force carrying capacity of chimneys. Capped chimneys are less susceptible to weathering than uncapped chimneys. Table 4.15 presents the damage ratios for capped and uncapped chimneys and the ratios of the damage ratio of the uncapped over capped chimneys. In all cases, regardless of the variation of factors such as location and materials, the uncapped chimneys were damaged more frequently than capped chimneys.

Damage ratio is certainly a function of chimney height. Less damage occurs to chimneys that terminate near the roof line of a structure than those that extend high above the roof line. Damage ratios for one-story buildings having brick chimneys of various heights are given in Table 4.16. The table shows that

TABLE 4.12 NUMBER AND DAMAGE RATIO IN PERCENT OF EACH DAMAGE TYPE
FOR EACH BUILDING CLASS

DAMAGE TYPE		NUMBER OF DAMAGED CHIMNEYS		DAMAGE RATIO IN %		AVERAGE DAMAGE RATIO IN %
		CLASS 1	CLASS 2	CLASS 1	CLASS 2	
SLIGHT	1	7	16	1.4	2.6	2.0
	2	42	37	8.2	6.0	7.0
	3	12	2	2.3	0.3	1.2
MODERATE	4	18	17	3.5	2.8	3.1
	5	45	44	8.8	7.1	7.9
SEVERE	6	40	61	7.8	9.9	9.0
	7	9	4	1.8	0.7	1.2
OVERALL		173	181	33.8	29.4	31.4

TABLE 4.13 NUMBER OF CHIMNEYS OF DIFFERENT CONSTRUCTION MATERIALS
FOUND AT VARIOUS LOCATIONS ON BUILDINGS

LOCATION	CLASS 1			CLASS 2		
	BRICK	STONE	PLASTER-COAT- ED BRICK	BRICK	STONE	PLASTER-COAT- ED BRICK
CENTER	78	9	18	61	10	18
END	39	6	27	102	6	43
EAVE	105	10	66	141	15	78

TABLE 4.14 COMPARISON OF DAMAGE RATIOS (IN PERCENT) OF CHIMNEYS WITH
DIFFERENT CONSTRUCTION MATERIALS AND FOUND AT VARIOUS
LOCATIONS ON BUILDINGS

LOCATION	CLASS 1			CLASS 2		
	BRICK	STONE	PLASTER-COAT- ED BRICK	BRICK	STONE	PLASTER-COAT- ED BRICK
CENTER	9	*	6	21	*	*
END	28	*	52	31	50	19
EAVE	34	*	61	28	47	46

TABLE 4.15 DAMAGE RATIO IN PERCENT FOR CAPPED AND UNCAPPED CHIMNEYS

LOCATION	CLASS 1			CLASS 2		
	CAPPED	UNCAPPED	RATIO OF DAMAGE RATIOS	CAPPED	UNCAPPED	RATIO OF DAMAGE RATIOS
Center	5.4	30.9	5.7	2.8	34.3	12.3
End	10.0	60.0	6.0	13.2	41.4	3.1
Eave	17.3	65.7	3.8	14.6	41.9	2.8

TABLE 4.16 DAMAGE RATIOS IN PERCENT OF BRICK CHIMNEYS FOR ONE-STORY BUILDINGS

LOCATION	TOPPLED	HEIGHT OF BRICK CHIMNEY ABOVE ROOF (IN FEET)				
		1 & 2	3 & 4	5 & 6	7, 8, & 9	1 to 9
Center	2	9	7	14	*	8
End	15	18	17	33	*	17
Eave	18.5	40	20	16	11	18.5

chimneys extending 5 and 6 feet above the roof line were more susceptible to damage than those of other heights.

General conclusions from the chimney damage study were as follows:

- Most residential buildings in the Glendale control area had periods in the range of 0.1 and 0.2 seconds.
- More than half of the buildings had chimneys.
- Approximately one-third of the chimneys were damaged.
- Approximately one-third of damaged chimneys were slightly damaged, one-third severely cracked or separated from the buildings, and one-third toppled.
- Brick chimneys were damaged less than stone and plaster-coated brick chimneys.
- The damage to chimneys located at the building center was less than that of chimneys located at the end or eave of the structure. The chimney damage for those located at the end of one-story buildings was less than that of chimneys at the eave.
- Capped chimneys were damaged less than uncapped chimneys.

The next Blume investigation was concerned with damage to mobile homes (trailers), swimming pools, and commercial buildings in Glendale, and in adjacent areas of Los Angeles.

In general, there was no damage to mobile home parks in the area of Glendale. Damage severity increased in locations north of Glendale culminating in all of the units being damaged in parks in Sylmar and San Fernando. Mobile homes in the area were mounted on any of three types of supports:

1. Precast concrete piers with steel leveling screws
2. Concrete block units
3. Aluminum tripod jacks

The most severe damage was the result of mobile homes being displaced from pier supports.

- Penetration by precast concrete piers through mobile home floors was reported in the Sylmar-San Fernando area. This damage occurred when the mobile homes were dislodged from their supports during the earthquake. This kind of damage required extensive repair of the floor systems, releveled of the mobile homes on supports, and in some instances the replacement of piers.
- Numerous reports were received regarding the necessity of releveled mobile homes following the ground motion shock, yet few structures were damaged. Other reports indicated the failure of metal jacks because of lateral ground movement, requiring replacement of many aluminum jacks.
- Expandable mobile homes are homes that can extend a side portion of the unit into an additional living space. This telescoping addition is fragile in that the unit is cantilevered from the main housing and simply supported. At the intersection of the expanded portion with the main housing, damage can easily be caused by lateral movement.
- Similar problems occur in mobile homes that have attached patio covers. These covers are generally light gage aluminum attached to the mobile home at the roof line and supported at the outer edge by attachment to metal poles. The poles or light gage pipe columns are rigidly secured to the ground. Lateral ground motion causes the simply supported mobile home to move on its piers, which results in damage to the rigidly anchored patio cover.
- Damage to utility attachments occurred in the same way as to rigidly attached patio covers. Upon dislodgement of the mobile homes from their piers during the earthquake, damage occurred to the entrance connection between the mobile home housing and the rigidly supported utility line. Damage was generally minor and readily repaired.
- The most commonly reported damage in mobile homes in the Glendale area was dish breakage.

Table 4.17 presents statistics of mobile home damage.

Pool damage resulting from severe ground motion of the San Fernando earthquake was reported severe in the Sylmar area to minor in the Glendale area. Except for structural concrete pool slab ruptures in the Sylmar area, the majority of instances of pool damage were cracks in plaster, coping, and decking. The reported crack damage varied from hairline to one-inch widths. Repairs were generally accomplished by the addition of a plaster coat and cement pointing of tile splash and copings. Most damage associated with cracking was obviously earthquake caused because of the fresh ragged configuration of the edges. The nature and instances of swimming pool damage are set forth in Table 4.18.

Field information was collected on the effects of the earthquake on 23 commercial buildings in the city of Glendale that were posted as unsafe to occupy, albeit minor damage was sustained by many of the approximately 760 commercial buildings (mostly one- and two-story) in the city. Observed damaged included severely cracked brick masonry walls predominantly in a diagonal or crossed diagonal pattern, walls out of plumb especially at the front portion of stores, walls loosened from floor and roof elements, failed connections, and broken store windows.

Most Glendale commercial building failures were in one-and two-story unreinforced brick structures with wood structural members framing into exterior masonry-bearing walls. The one or two buildings over two stories in height that suffered severe damage were of similar construction type.

Building characteristics that were believed to be responsible for the severe damage which was observed were listed following:

- Use of lime mortar, which weakened with age, resulting in lack of bond.
- Lack of full-bedded mortar joints during masonry construction.
- Structural supporting masonry walls, two-wythes thick but poorly bonded by insufficient mortar bond or lack of header bond at a minimum distance of six-course common bond construction (A wythe or withe is a partition, one brick-width thick). This condition allowed the two-wythe section of the bearing wall to act separately during lateral stress resulting in delamination of the wall by allowing the face brick wythe to crack and fall from the back-up or interior wall portion.

TABLE 4.17 MOBILE HOME SURVEY, SAN FERNANDO EARTHQUAKE RELATED DAMAGE

Description of Damage	Percentage Requiring Repair				
	Sylmar	San Fernando	Van Nuys	Burbank	Glendale
Patio Canopy	1	2	0	0	0
Bric-a-brac	100	100	15	34	5
Utilities	36	7	68	8	0
Pier foundations	--	--	--	--	--
Releveling	100	100	15	10	0
Reblocking	100	51	0	2	0
Number of Mobile homes surveyed	244	286	128	270	118
Approximate peak horizontal ground acceleration	0.5-0.9	0.4-0.8	0.25-0.5	0.2-0.45	0.15-0.35

TABLE 4.18 SWIMMING POOL SURVEY; SAN FERNANDO EARTHQUAKE RELATED DAMAGE

Description of Damage	Number Repaired
Coping and splash tile cracks	1,486
Side-plaster cracks	1,063
Bottom-plaster cracks	761
Decking cracks	531
Supply system	13
Drainage system	21
Heating system	33
Filter system	33

- Insufficient or nonexistent ties into masonry bearing walls at the roof level between wood structural members and parallel wall surfaces. This resulted in the lack of vertical support for the masonry wall, allowing the wall to sag or bulge. This was observed in numerous brick structures exhibiting unreinforced parapet failures, wall failures at roof lines, and delamination of face brick wythes at roof levels.
- Masonry bearing walls were reduced to single-wythe walls above the bearing of the roof-supporting structural members. The reduced section of this masonry wall, laterally unsupported, extended upward to form the unreinforced parapet. The inadequate structural support allowed this portion of the wall to be battered by the structural members during the earthquake, resulting in wall damage and failure.
- Excessive use of openings through unreinforced masonry walls resulted in impaired strength of wall piers between the openings. This condition allowed for the formation of continuous cracks between points of accumulated stress. The two-wythe wall weakened by excessive openings, in addition to the poor bonding methods in the articulated wall system, produced the condition for cracking failures.
- Heavy stone rubble veneer walls having interior wood structural members were damaged or failed because of lack of bond between the two surfaces. Insufficient structural ties to unite the two units allowed the components to respond individually to seismic forces, resulting in stone veneer mortar joint cracking or failure.
- The failure of double-wythe unreinforced masonry-bearing wall construction resulted in cracking or dislodgement of architectural cosmetic elements attached to the wall surface. Stucco, plaster, and paint cracked, peeled, and fractured when brick walls failed. Window glass cracked and terra cotta friezes and plaques cracked and failed during brick wall failures. All elements attached directly to masonry-bearing walls were liable to damage.

The extensive earthquake damage occurrence in Glendale also offered an opportunity to obtain extensive data on seismic motion-damage relationships. Blume conducted a survey of Glendale building owners, via questionnaire, to acquire this data set. To attempt an evaluation of foundation condition influence, this survey was divided into two areas; one (Area 1) in the valley section of Glendale with a thin

alluvium cover and the other (Area 2) in the flatland where deep alluvium prevailed. Structures were grouped according to expected dynamic response characteristics. Group 1 included wood-frame buildings with fundamental periods between 0.005 and 0.1 seconds. Wood-frame buildings with fundamental periods between 0.1 and 0.2 seconds were in Group 2.

Table 4.19 summarizes the distribution of buildings in study areas and Table 4.20 the response spectrum values for various building groups.

To investigate the effects of various parameters on damage, the concept of a damage ratio was devised. Damage Ratio (DR) is defined:

$$DR = \frac{\text{Number of Damaged Buildings}}{\text{Number of Buildings}}$$

Tables 4.21 and 4.22 summarize damage components and damage ratios for the Glendale studies. Conclusions of this study were as follows:

- Group 1 buildings (period range 0.05 to 0.1 seconds) were more vulnerable than Group 2 buildings (period range 0.1 to 0.2 seconds).
- Buildings in Area 1 suffered more damage than buildings in Area 2, because of stronger ground motion in Area 1.
- The interior walls and chimneys are more susceptible to damage than other building components. This is partially because most interior walls are plastered walls, which are both brittle and weak. Chimneys such as these studied that are neither reinforced or laterally restrained are very vulnerable to lateral forces.
- Foundations suffer less damage than any other component. This is partially because most foundations in the two study areas are of good quality concrete stem wall construction.

Another study of San Fernando earthquake damage was concerned with damage occurrence to 2 seven-story structures, one in Van Nuys and the other in Los Angeles.

TABLE 4.19

DISTRIBUTION OF BUILDINGS IN GLENDALE STUDY AREAS

Location	Number of Buildings		
	Group 1	Group 2	Group 3
Area 1	322	172	494
Area 2	245	304	549
Total	567	476	1043

TABLE 4.20

AVERAGE PREDICTED 5% DAMPED RESPONSE SPECTRUM
VALUES FOR GLENDALE STUDY AREAS

Location	Structure Group	North-South		East-West		Envelope	
		Vel. (cm/sec)	Accel. (g)	Vel. (cm/sec)	Accel. (g)	Vel. (cm/sec)	Accel. (g)
Area 1	Group 1	3.66	0.29	6.22	0.55	6.22	0.55
	Group 2	25.29	1.00	26.46	1.16	26.46	1.16
Area 2	Group 1	3.32	0.27	3.72	0.30	3.72	0.30
	Group 2	13.70	0.55	13.21	0.64	13.21	0.64

TABLE 4.21 NUMBER OF BUILDINGS WITH DAMAGED COMPONENTS: GLENDALE STUDY AREAS

Location		Chimney	Interior Wall	Exterior Wall	Foun- dation	Window	Other	Overall
Area 1	Group 1	96	137	44	10	18	97	174
	Group 2	39	99	44	6	11	43	114
Area 2	Group 1	45	61	16	2	8	29	91
	Group 2	32	68	20	4	11	36	100

TABLE 4.22 DAMAGE RATIOS IN PERCENT FOR GLENDALE STUDY AREAS

Location		Chimney	Interior Wall	Exterior Wall	Foun- dation	Window	Other	Overall
Area 1	Group 1	42.1	42.5	13.7	3.1	5.6	30.1	54.0
	Group 2	40.2	56.9	25.6	3.5	6.3	24.7	65.5
Area 2	Group 1	29.6	24.9	6.5	0.81	3.3	11.8	37.1
	Group 2	24.1	22.4	6.6	1.3	3.6	11.8	32.9

The structures were both Holiday Inn motor hotels which were essentially identical. One was located about 13 miles and the other about 26 miles from the epicenter of the February 9, 1971, San Fernando earthquake. Appreciable nonstructural damage as well as some structural damage was observed. Strong-motion seismic records were obtained for the roof, intermediate story, and ground floor of each structure. The analyses were based on data from the Structural drawings, architectural drawings, photographs, engineering reports, and seismogram records obtained before, during and after the San Fernando earthquake.

Both structures experienced motion well beyond the limits of the building code design criteria. A change in fundamental period was observed for each structure after several seconds of response to the earthquake, which indicated nonlinear response.

Data on fundamental periods of the 2 structures prior to, during, and after the earthquake are set forth in Table 4.23. Changes in the dynamic behavior of the structures as indicated by period lengthening, as observed in the motion records, indicated that the elastic capacity of some structural members was exceeded. Idealized linear models were constructed to approximate response at various time segments.

Table 4.24 presents period data on some of these idealized linear models.

Comparison of damage costs, shown in Table 4.25, is based on cost estimates of the repairs made to the structures.

Observed structural damage that was repaired was relatively minor. For the Van Nuys structure, this included cracking of a second floor spandrel-to-column connection at the northeast corner, cracking of typical exterior spandrel beam-to-column connections, and cracking of architectural fins on structural columns. For the Los Angeles structure, there was structural damage to the beam-column connection at the intermediate stair landing between the 1st and 2nd floors. Some drywall panels had to be replaced at the Van Nuys structure but not at the Los Angeles structure. This is consistent with estimated interstory displacements of 1 inch for the Van Nuys structure versus 1/2 inch for the Los Angeles structure. At the Van Nuys structure, 45 bathtubs and 12 water closets had to be replaced because of damage from interaction of slab, fixture, and tile walls. Only nine bathtubs had to be replaced at the Los Angeles structure.

**TABLE 4.23 OBSERVED FUNDAMENTAL PERIODS
(SECONDS)**

		Transverse		Longitudinal	
		V.N.	L.A.	V.N.	L.A.
Pre-Earthquake V.N. 2/16/67 L.A. prior to 4/8/68		0.48	0.49	0.52	0.53
Earthquake 2/9/71	Early Portion	0.70	0.63	0.70	0.60
	Late Portion	1.6	1.15	1.5	1.1
Post-Earthquake 2/71 and 2/71		0.68	0.63	0.72	0.64
Aftershock 3/31/71		1.1	--	1.2	--
Post-Repair 5/27/91		0.58	--	0.64	--

**TABLE 4.24 CALCULATED FUNDAMENTAL PERIODS
(SECONDS)**

	Transverse	Longitudinal
Structural Frame Only*	0.88 (T-2)**	
With Partitions Modeled as Diagonal Struts	0.83 (T2-P)	
Partitions as Struts and Cement-Plaster as Shear Wall	0.54 (T2-PW)	0.68 (L2-PW)
Additional Cement- Plaster Wall Included	--	0.63 (L2-PW2)

* Based on beams and slabs @ gross concrete section, slab width @ 60% tributary width, and columns @ gross transformed section.

** Letters in parentheses identify the model being used.

TABLE 4.25 DAMAGE COMPARISONS

	VAN NUYS	LOS ANGELES
Total Cost of Damage Repair (% of initial construction cost)	\$145,000 (11%)	\$95,000 (7%)
Cost Per Square Foot	\$2.30	\$1.50
Cost of Structural Repair	\$2,000	\$2,500
Drywall Partitions	Some Replacement of Panels required	Damage Repairable
Bathtubs Replaced	45	9
Water Closets Replaced	12	None

Although the damage repair cost ratio of the Van Nuys structure to the Los Angeles structure was about 1.5, the actual damage severity ratio was probably higher, possibly greater than 2.0. For example, if a wall has to be repainted or repapered because of partition damage, the cost of paint or wallpaper is the same whether there are a small number or a large number of cracks. Costs of the structural repairs were relatively insignificant. A dollar cost comparison does not reflect a structural damage comparison because of the nature of the damage and the nature of the repair.

Conclusions from the study are summarized below:

- The structures were nearly identical.
- Both structures exceeded their elastic limits during the earthquake and have suffered permanent losses of stiffness.
- The closer structure (Van Nuys, 13 miles) experienced almost twice the peak lateral displacement and damage than did the further structure (Los Angeles, 26 miles). The Van Nuys structure exhibited a greater loss of stiffness (greater increase of period) during the earthquake than the Los Angeles structure.
- The second mode response might have played a significant part in initial partial loss of nonstructural stiffness of the Van Nuys structure, but not of the Los Angeles structure.
- The yielding of beams prevented possibly serious column failures. The structures responded as ductile frames, although not specifically designed as such. Based on partially cracked concrete sections, average beam ductility ratios were calculated at 8 for the Van Nuys structure and 4 for the Los Angeles structure. Some beam ductility ratios were calculated to be greater than 20. If ductility is based on fully cracked sections, these values would be reduced, possibly by a factor of 2.
- On the basis of analysis results, one would expect more damage than was actually observed. Peak accelerations were 4 to 5 times and peak displacements were from 8 to 15 times design levels.
- The cost of repair for the Van Nuys structure was roughly 1.5 times the cost of damage

repair for the Los Angeles structure. However, visual observations indicate that the Van Nuys damage was closer to twice that of the Los Angeles structure.

- Responses of these structures to future earthquakes will vary considerably from their responses to the February 9, 1971, earthquake. Their periods will be longer. Their amplitudes of response could be greater or smaller for a similar sized earthquake, depending on the characteristics of the ground motion.
- This study shows that the nonstructural elements of a building can play a significant role in the amplitudes of response of buildings to earthquakes. The partitions took 60% or more of the force before cracking. They reduced the response period by about 30%. The investigation of nonstructural elements and their effects on the response of a structure should be included as part of the design criteria.
- The periods obtained from very small amplitudes of motions must be used cautiously when compared with periods obtained from moderate-or-strong-motion amplitudes.
- This study also demonstrates the need to define ductility in design. The basis for calculating periods, displacements, stresses, etc., will greatly affect apparent ductility factor calculations.

4.4.9 Santa Barbara, California - August 13, 1978

On August 31 and September 1, 1978, Blume conducted a field study of damage inflicted on structures in the Greater Santa Barbara area by a magnitude of 5.1 earthquake that occurred there at 15:54 PDT on the afternoon of Sunday, August 13, 1978. Damage and heavy shaking were reported throughout the Santa Barbara area, but were most noticeable in Goleta, which is located roughly 7 miles to the west of downtown Santa Barbara. The epicenter was located in the Santa Barbara Channel, about 4 miles south of downtown Santa Barbara and about 7 miles southeast of Goleta. Peak ground accelerations recorded in Santa Barbara were approximately 0.2g while those recorded in Goleta ranged from 0.35 to 0.45g.

The objectives of the field study were (1) to determine the general nature and extent of damage to all types of structures and (2) to obtain specific information concerning damage to mobile homes. Other structures in the Greater Santa Barbara area that were included in the field study are: the University of

California (UCSB) buildings, Santa Barbara County government buildings, and other various private buildings.

A disproportionate number of mobile homes, in comparison to conventional dwellings, office buildings, and other structures, suffered considerable damage as a result of the earthquake. Although damage typical of a moderate earthquake, such as items falling from shelves and bookcases tipping over, was found throughout the Santa Barbara region, mobile homes generally sustained more serious damage.

The mobile home parks of particular interest are located immediately west of the City of Santa Barbara and in the Goleta area, further west. The parks that reported the bulk of the damage are in the Goleta area, where they are typically located on gently sloping sites. Most of the damage to mobile homes had already been repaired at the time of this study.

The total number of mobile homes in Santa Barbara and the Goleta area was 1,414 at the time of the earthquake; of these, 254 sustained damage. The least costly damage to repair occurred when the coach or its supporting jacks shifted a small amount, requiring releveled of the home and repositioning of any porches and overhangs.

An intermediate level of damage occurred when some of the jacks either tipped over or were damaged, but the coach had not actually fallen to the ground. Repairs were more expensive because workmen had to remove the failed jack and reseal another before the leveling process could be started. This type of failure usually also damaged the skirting around the coach.

The most severe damage level occurred when all or most of the jacks were damaged or tipped over, allowing the coach to fall to the ground. Nearly all of the jacks need to be replaced, and the coach home itself usually required repairs as a result of its fall. Damage may also occur to the walls of the coach. Other aspects of this highest damage level included the puncturing of the floor by jacks remaining vertical as the coach fell and the severing of the utilities to the home, such as gas, water, sewage, and electrical. One major fire at a coach was reported due to a severed gas line.

Most buildings at UCSB suffered either no structural damage or insignificant structural damage. Much of the damage to buildings was nonstructural or architectural, such as cracking of plaster and wood stud walls and broken windows. Some buildings, however, did sustain significant structural damage, and in several cases it was severe. University officials estimated at the time of this study that it would cost

about \$4 million to repair structural and architectural damage. UCSB suffered significant damage to its mechanical equipment, most notably to elevators, and to building contents. Damage cost estimates were \$1 million for mechanical and electrical repairs plus a possible addition of \$2 million for laboratory equipment.

The campus is constructed on a shale rock mesa overlain by 8 to 20 ft of alluvium. Some of the most expensive damage was in the category of nonstructural damage. In the library alone, 350,000 books fell from the shelves, representing about 80% of the books in the two-story portion and 10% of the books in the eight-story portion. In nearly all UCSB laboratories where there were no restraints on the shelves, chemicals, expensive equipment, and other items fell to the floor or tipped over.

Some ventilating duct grates, light fixtures, and veneer tile fell into heavily traversed areas. In an auditorium, steel plates which covered light wells in the ceiling, fell down on the seats below. These plates were about 48 in. long, 8 in. wide, and weighed about 5 lb. each. They fell a distance of about 20 ft.

Eighteen of the 41 elevators throughout the campus were made inoperable by the earthquake. Others experienced leakages in their hydraulic systems but remained in operation. Some of the elevators jammed when counterweights came out of their guides due to the lack of seismic restraints, thus snarling cables and breaking other structural mounts, etc.. University officials have estimated damage to elevators to be \$100,000.

Other mechanical equipment, located mainly on the top levels of the taller structures on campus, was damaged. Extensive mechanical damage occurred in the penthouse above the steel eight-story portion of the library, where boilers, pumps, heat exchangers, and other heavy piping were mounted on spring isolation mounts that lacked lateral seismic restraints. In many cases, the equipment vibrated out of these mounts. In several cases where equipment was rigidly connected to the concrete pad, the mounting brackets sheared off. Pounding between mechanical equipment, pipes, and walls of the penthouse was evident. University officials stated that other buildings on campus such as South Hall (six stories) suffered similar mechanical equipment damage, but to a somewhat lesser extent.

No further earthquake damage studies were conducted by Blume for DOE/NV after the Santa Barbara study of 1978, primarily because adequate data for the structural response program were available by that time.

4.5 STATISTICAL STUDY OF LOW-RISE BUILDING DAMAGE

Blume conducted a number of studies on the occurrence and extent of damage to low-rise (one and two story) buildings subjected to ground motion generated by UNE's (A24, A51). Damage from two high-yield events in Colorado Project RULISON in 1969; and Project RIO BLANCO in 1973 was the subject of several studies (A33-A36, A42, A59, A75). Low-rise building damage from ground motions at lower amplitudes was explored in studies that used damage data from southern Nevada for selected NTS and central Nevada events from 1966 to 1976 (A59) and from other off-site tests (A65, B20, B24).

4.5.1 Formulation of Damage Parameters

Damage information was obtained from DOE/NV's damage complaint and payment records. To establish the validity of damage, each complaint was first investigated by an insurance adjuster. If there was any question as to whether damage was caused by an underground nuclear explosion, an engineering investigation was conducted by Blume. In analyzing these data, a distinction was made between complaints and credible damage, defined as follows:

Complaint: Any complaint made concerning property damage whether or not formalized as a claim.

Credible Damage: Any of the above-described complaints acknowledged as credible by investigators.

To identify the various aspects of damage that are important for prediction purposes, three parameters were defined:

Complaint Ratio (CR):

$$CR = \frac{\text{Number of Buildings from Which Complaints Were Received}}{\text{Total Number of Buildings}}$$

Damage Ratio (DR):

$$DR = \frac{\text{Number of Damaged Buildings}}{\text{Total Number of Buildings}}$$

Mean Damage Factor (m_{DF}):

$$m_{DF} = \frac{1}{N} \sum_{i=1}^N DF_i = \frac{1}{N} \sum_{i=1}^N \frac{\text{Damage Repair Cost for a Building}}{\text{Replacement Value of that Building}}$$

where N equals the number of buildings in a particular exposed sample (e.g., the number of buildings in a town) and DF_i is the damage factor for building i .

The initial studies of the RULISON data used Damage Cost Factor (DCF) instead of m_{DF} :

$$DCF = \frac{\text{Damage Repair Cost of Buildings}}{\text{Value of Buildings}}$$

The mean damage factor, m_{DF} , has about the same meaning as the damage cost factor, but the former is more directly applicable to the development of a statistical damage prediction procedure and therefore the damage cost factor was eventually replaced.

In order to evaluate the scatter in the damage factor data, the standard deviations and variances for the damage factors (σ_{DF} and σ_{DF}^2), as well as their coefficients of variation (V_{DF}), were also calculated. The parameters σ_{DF}^2 and V_{DF} are defined below:

$$\sigma_{DF}^2 = \frac{1}{N-1} \sum_{i=1}^N (DF_i - m_{DF})^2$$

$$V_{DF} = \sigma_{DF}/m_{DF}$$

4.5.2 Results from RULISON

The RULISON experiment provided the first good opportunity to conduct an effective statistical analysis because of the large data set associated with the project and the ground motion much larger than experienced from the NTS events. Over 450 damage complaints were filed. Although about half were not used because of the lack of nearby ground motion records, the remaining half provided a sufficiently large data base to conduct meaningful analysis.

In the first study in 1971 involving the RULISON data, various methods for characterizing the ground

motion were correlated with the damage prediction parameters to identify the method that yielded the best correlation. When comparing correlations using peak velocity with correlations using peak acceleration, no definite conclusion could be made as to which of these two peak parameters of ground motion provided the better indicator of damage. However, use of response spectral values produced better correlations than either of the peak parameters. Further studies with different response spectrum parameters showed that the best correlation was obtained by using the 5% damped response spectrum acceleration for the vector sum of the two horizontal components (S_{av}). However, because no procedure had been developed for predicting S_{av} , it was not a convenient variable for damage prediction.

A convenient and applicable ground motion characterization for predicting damage to low-rise buildings is the envelope spectrum, defined by the envelope of the two horizontal-component response spectra. A comparison of the envelope spectrum and the spectrum of the horizontal vector for several earthquake and underground nuclear explosion ground motions showed that the envelope spectrum (S_{ae}) approximates S_{av} well in the period range of 0.05 to 0.2 seconds -- the fundamental period range of most low-rise buildings. Therefore S_{ae} values were used in all studies after the initial study of RULISON data.

4.5.3 NTS Events 1966 to 1970

From 1966 through 1970, DOE/NV conducted several UNE tests at the NTS that generated ground motion amplitudes high enough to cause damage complaints to be filed and to cause damage in towns surrounding the NTS. Complete and useful data (damage was claimed) were available for seven events and three communities: Las Vegas, Tonopah, and Beatty.

The damage information used in this study was obtained from DOE/NV's damage complaint and payment records. To establish the validity of damage, each complaint was first investigated by an insurance adjuster and if there was any question as to whether or not damage was caused by a UNE test, an engineering investigation was conducted, as noted earlier.

The motion-damage statistics obtained from the lower spectral motion amplitude NTS testing fitted well with the similar RULISON damage statistics obtained for higher spectral motion amplitudes.

At low spectral motion amplitudes, the shape of the motion-damage relationship curve is such that damage approaches zero at a finite spectral motion amplitude. Theoretically this may or may not be the case, depending upon the particular circumstances involved.

Although it was generally concluded from these studies that damage can be related to ground motion, a large scatter in the data was noted for testing events that generated low-amplitude ground motion. The studies indicated that the level of damage produced by some of the low-amplitude events was much greater than that produced by others. Some factors that could contribute to this large variability are variations in ground motion amplitude and the presence of certain geological conditions in the study area, the time of day when the explosion was detonated, the community temperament at the time of detonation, and other sociological factors.

Studies by others have shown that there is significant variation in ground motion amplitude throughout the Las Vegas area. In addition, investigations of damage complaints in the Las Vegas area have revealed that various geological conditions (in particular, the shrinking and swelling of surficial soils due to changes in ground water levels) have caused damage to low-rise building. Because both of these factors could be highly relevant to Blume's ongoing investigations, a further statistical analysis was conducted in 1981.

4.5.4 Results from RIO BLANCO

The RIO BLANCO event provided additional opportunity to evaluate the damage-motion relationships of low-rise buildings. For the entire area, there were 169 damage claims registered by May 1975 (two years after the detonation). Of these, 114 were acknowledged as justified. Data from five towns in the area were used in this study. For these 5 towns there were 70 damage complaints and 54 justified damage settlements.

Structure inventory information was obtained from several sources. For the town of Rangely, the data were collected from on-site observations conducted from the street. Inventories of the towns of De Beque, Grand Valley, and Rifle were available from a similar damage study conducted in connection with Project RULISON. Because little change was noted, the previously collected information was used. General inventory data for the town of Meeker was obtained from CER Geonuclear Corporation (the industrial sponsor of the experiment).

With the limited number of damage-motion relationships from the test and the narrow band of ground motion values associated with the data, a correlation study of the data was not performed on the RIO BLANCO data alone. However, a comparison with results from other events showed a reasonable correlation.

4.5.5 Combined Study

The data from the RULISON and RIO BLANCO projects provided statistical descriptions of the incidence and degree of damage to low-rise buildings subjected to intermediate amplitude ($0.1g < S_{ae} < 1.0g$). Damage to low-rise buildings was also being reported for very low-amplitude ground motions ($0.01g < S_{ae} < 0.1g$) in various communities adjacent to the NTS during the high-yield test program conducted from 1966 to 1970.

For this final study the data base was expanded by including data from additional damaging and several non-damaging events that took place at NTS from 1966 to 1976. Motion-damage relationships specifically relevant to the Las Vegas area were established by separately analyzing the data from all NTS events in the revised data base.

This study produced definitive results concerning the statistical correlation of ground motion and associated damage produced by NTS events. A relationship was established between the damage parameters CR, DR, and m_{DF} and the ground motion parameter S_{ae} (see Section 4.6.3).

For the low-amplitude events, a significant scatter or variability in damage existed. Statistically, the scatter can be interpreted to mean that damage is not highly dependent on ground motion in the low-amplitude range, which suggests that credible damage may have been the result of other factors, such as variations in ground motion amplitude, wind, or geological conditions.

The geographical correlation of the distribution of low-rise damage from NTS events in Las Vegas with that of ground motion amplitudes and geological conditions was not conclusive. From the study of the areal distribution of ground motion, it was found that interstation ratio values have a tendency to vary randomly in short-period intervals (less than 3 sec) in the Las Vegas area, thus indicating either that ground motion is probably not the sole cause of damage or that other factors cause the ground motion to appear randomly distributed when it is not. The tendency of short-period ground motion amplitudes to appear randomly distributed can be attributed to three factors:

- Distortion of instrument records caused by extraneous vibratory noise, such as that caused by cultural sources
- Effects of local near-surface material conditions on the ground motion in the short-period range recorded at the various stations

- The random nature of ground motion generated by each event

It was found that damage and geological parameters could not be unequivocally correlated using the damage subregions developed for this study and the presently available large-scale surficial geological maps of the Las Vegas area. However, the study did show that damage was concentrated in the downtown Las Vegas area and along geological lineations and that the largest amount of subsidence resulting from groundwater withdrawal, which can cause damage, was located in an area slightly northeast of downtown Las Vegas. This degree of correlation gave support to the contention that the various active geologic conditions in the Las Vegas Valley induced stresses in building components and that damage was triggered by low-intensity ground motion.

4.6 RESPONSE EVALUATION PROCEDURES

4.6.1 Guidelines for Computer Modeling

The response of an individual building can be predicted by computer modeling. During the process of using computer models to predict response, several questions arose:

- Can the dynamic characteristics of a multi-story building be determined from an approximate model without resorting to a detailed lumped-parameter model? If so, this would result in significant savings in time and effort.
- When modeling low-rise buildings, is soil-structural interaction a significant effect?
- What are the most appropriate models for representing the dynamic response of non-structural partitions?

The first question was addressed by a study to determine if the periods, mode shapes, and participation factors for a multi-story building could be computed approximately by a uniformly continuous elastic system (A47). It was determined that this is true for high-rise structures which are generally uniform with height. The continuous model allows a shearing deformation corresponding to a relative interstory displacement, a section rotation corresponding to an average joint rotation for a given floor, and a section rotation corresponding to an overall floor rotation.

Mode shapes and periods obtained using this approximate model were compared to those obtained from a more exact, lumped-mass model for 40 cases. Both models are representations of actual high-rise structures in Las Vegas. Thus, the non-uniformities in mass and stiffness in these models are realistic. The approximate continuous model yields periods with an average of 2% high with a standard deviation of 14% about the mean.

The second question, soils-structure interaction as a significant factor when modeling low-rise buildings, was addressed in another Blume report (A57). This report presented the formulation and results of a low-rise building soil-structure interaction parameter study that considers 1- and 2-story structures typical of residential construction.

Considering the uncertainty associated with each step in predicting damage to low-rise buildings, the

possible error that might occur by neglecting the system amplification effect of soil-structure interaction was determined to be a secondary consideration. It was concluded that neglecting the system amplification effect of soil-structure interaction when predicting damage for typical low-rise buildings subjected to UNE-induced ground motions was reasonable, and that it was sufficient to know the system period and damping ratio values, which for a particular building could be easily determined experimentally.

The third question, regarding modeling nonstructural partitions, was addressed in a third Blume report (A43). That report focused on the influence of nonstructural partitions on the dynamic behavior of two, identical, four-story, reinforced concrete test structures and compared their behavior to that predicted from a linear theory. The two structures are located at NTS and discussed in Section 4.2.1. The partition types studied were gypsum board, plywood, unreinforced concrete block, and unreinforced hollow clay tile.

The bare frame was modeled according to accepted practice: one model using uncracked sections, one using beams and columns cracked but slabs uncracked, and one using all cracked sections. The gypsum board and plywood partitions were modeled as single shearing elements while the masonry partitions were modeled as equivalent struts. Only the fundamental periods were used to compare the theoretical to observed behavior.

The stiffness of the bare frames was remarkably variable. Approximately two years after the structures was completed, the generalized stiffness of the frames were reduced to 43% of their original values. The fresh structures were approximated well by the totally uncracked frame, while after extensive vibration history the structures were approximated well by a fully cracked model.

The stiffness effects of the gypsum board and plywood partitions were slight and approximated well using the single shear elements and shearing stiffnesses obtained from laboratory experiments. Results of laboratory experiments are given in Section 4.1.

The stiffness of the frame with masonry partitions was extremely variable. The stiffness of the frame with fresh partitions was approximated well by the equivalent strut model. After extensive vibration history, the partition stiffness was effectively reduced to zero. Specifically, the combined frame and partition stiffness was reduced to 16% of its fresh value.

The question on modeling of nonstructural partitions was also addressed in still another Blume report

(A48). It had been observed in certain cases that the bond between the frame and filler panels has been destroyed by prior motion of the building, poor construction techniques, or similar causes. The result could be gaps between the filler panels and the adjacent beams and columns of the frame. During subsequent response of the structure, such as response to an earthquake or underground nuclear explosion, the structure might displace enough for the frame to contact the panels and thereby close the gaps. Repetitive closing and opening of the gaps could occur during the response of the structure.

An analytical model and a computer program were developed to perform nonlinear dynamic analyses of a plane frame with filler panels and gaps between the sides and tops of the panels and the frame. In addition, an analytical model and program were developed for linear elastic analyses of a frame with monolithic panels, that is, with no gaps. The framing members of the structure were represented by conventional line elements in bending with axial deformations, and the filler panels were represented by finite element meshes using the Oakberg-Weaver rectangular plane stress element having 16 nodal displacement (Gere, J.M. and W. Weaver, Jr., *Analysis of Framed Structures*, van Nostrand, Princeton, New Jersey, 1965). Special elements were developed for panel corners by the use of a coordinate transformation procedure, and static condensation techniques were used to eliminate certain degrees of freedom and thereby reduce the problem size.

Modal superposition techniques were used to solve the equations of motion for the linearly elastic case, and the incremental linear acceleration method was used for the nonlinear case. Criteria were developed for the monitoring of the opening and closing of gaps, and a procedure was developed for the modification of the stiffness matrix upon closing and opening of gaps.

Linear and nonlinear examples were analyzed to illustrate the application of the techniques and computer programs that were developed, and to illustrate the effects of varying certain parameters in the analyses. It was found that the presence of gaps between the frame and panels can significantly affect the dynamic response of a structure. In addition, rather different responses were obtained for gaps of different widths.

4.6.2 Identification of Dynamic Characteristics

In order to evaluate the dynamic response of structures, Blume recorded motions on numerous structures which were vibrated by the pull-release method (free vibration), vibration generator, and UNEs. It was often necessary to identify dynamic response characteristics; periods, mode shapes, and viscous damping ratios; for a given structure on the basis of the recorded motions. An analytical procedure was

developed to obtain these dynamic characteristics from an input-output pair of time histories (A31). The input time history represents the input motion to the structure and could be a random input motion, harmonic input motion, or no input motion (free vibrations). The output time history is the structure's output motion, correlated in time with the input time history.

The procedure developed was a simple curve-fitting method in the time domain. Specifically, a modal response must first be removed from the total measured response. A filtering scheme was used that had been successful in isolating modal responses. The response of a single-degree-of-freedom, viscously damped oscillator was then generated from the measured input and fitted in a least squares sense to the isolated modal response. The generated response was fitted as a function of natural frequency, viscous damping ratio, scale factor, initial displacement, and initial velocity. The dynamic response characteristics of the best-fit oscillator response were then assumed to equal the dynamic response characteristics of the isolated mode. The overriding assumption made was that the structure to be analyzed may be modeled well by a viscously damped linear model; therefore, constant natural periods and viscous damping ratios have meaning.

This approach was accurate because the dynamic response characteristics were obtained from an exact, well defined response that in turn is fitted to the raw data. Background noise and inconsistencies from an exact linear theory were essentially filtered from the raw data by this fitting procedure. The computer program that performs this analysis was called RAP.

4.6.3 Motion-Damage Relationships

Low-rise building damage data from 17 NTS events, one central Nevada event and two Colorado events were analyzed for statistical correlation between motion and damage (A65). Damage was characterized by three parameters: complaint ratio (CR), damage ratio (DR) and mean damage factor (m_{DF}), as defined previously in Section 4.5.1. The ground motion was characterized by the envelope of two horizontal components of the response spectrum acceleration (5% damping), designated S_{ae} . To arrive at a single-valued number characterizing the ground motion at a particular station for low-rise buildings, the S_{ae} spectrum was averaged over the period band of 0.05 to 0.2 sec, and a banded spectral acceleration envelope value, designated S_{ae}^- , was obtained. The period band covers the periods of fundamental mode of most low-rise buildings.

The relationships between the three damage parameters with respect to S_{ae}^- for all events are shown in

Figures 4.15 through 4.17. These relationships were developed by parabolic best-fit analyses in the log-log domain that excluded nondamaging events. The nondamaging events (those having damage parameters with a value of zero) are represented on the log-log graphs by an asterisk on the S_{ae} coordinate axis, which indicates the S_{ae} values only. The variability in damage is depicted in the V_{DF} to S_{ae} curve shown in Figure 4.18. This shows that the statistical variability in damage becomes very large for low-amplitude ground motion.

Because many high-rise buildings are located in Las Vegas, it would be useful to have motion-damage relationships for high-rise buildings due to UNEs. Very little damage had occurred to high-rises from the high-yield testing program; therefore meaningful motion-damage relationships could not be obtained. However, data on damage to building components were available from laboratory tests. Using these data, damage to a given building component could be predicted as a function of the demand imposed on the component by the overall building response. The demand on the component could be described by a relevant local response parameter, such as interstory drift. This parameter could be calculated for a given ground motion from a standard linear or nonlinear analysis of the structure. Thus, the available published laboratory test data for various high-rise building components were collected and used to develop damage functions.

The high-rise building components studied included:

- Reinforced concrete beams
- Reinforced concrete columns
- Reinforced concrete shear walls
- Steel frames
- Masonry walls
- Drywall partitions
- Glazing

The data that could be obtained for other components, including suspended ceilings, exterior cladding, mechanical systems, piping, and elevators, were insufficient for statistical analysis.

A component damage function is the relationship between the expected value of the component damage factor (DF) and a peak local building response parameter (e.g., peak interstory drift or peak floor

Note: Nondamaging events excluded from statistical analysis

FIGURE 4.15 LOG-LOG GRAPH SHOWING RELATIONSHIP OF LOW-RISE BUILDING COMPLAINT RATIO TO SPECTRAL ACCELERATION, ALL EVENTS

Note: Nondamaging events excluded from statistical analysis

FIGURE 4.16 LOG-LOG GRAPH SHOWING RELATIONSHIP OF LOW-RISE BUILDING DAMAGE RATIO TO SPECTRAL ACCELERATION, ALL EVENTS

FIGURE 4.18 LOG-LOG GRAPH SHOWING RELATIONSHIP OF LOW-RISE BUILDING COEFFICIENT OF VARIATION TO SPECTRAL ACCELERATION, ALL EVENTS

acceleration). The high-rise building components considered in this study and the most relevant local response parameters affecting them are shown in Table 4.26.

4.6.4 The Engineering Intensity Scale

A first step in predicting the effects of UNE ground motion was to estimate the extent of the area in which structures might be damaged. The next step was to make a general evaluation of the incidence and degree of damage that such structures might sustain. The Engineering Intensity Scale (EIS) was developed by Dr. John A. Blume to accomplish both steps (A32, A58, B13).

In the formulation of the scale, ground motion was characterized by 5%-damped spectral velocity (S_v), and structures were characterized by their fundamental mode vibration properties. Neglecting mode shape considerations, the important correlation variable for relating motion and damage are S_v , amplitude and building period. The 5%-damping value was used because damping in many real structures varied from about 2% to 10%, and 5% had been made a standard reference level in the DOE/NV nuclear event structural response program.

Engineering intensity (EI) numbers were assigned to various spectral velocity bands. The applicable range of spectral velocities (S_v) and periods (T) applicable to civil engineering structures was divided into a 10 by 9 matrix, as shown in Figure 4.19. There were 10 intensity levels, from 0 through 9, and nine period bands, I through IX, from 0.01 to 10 seconds.

Iso-EIS maps can be constructed if sufficient spectral data are available. It is possible to prepare a map for each of the nine columns of the EIS and to show specific data for each of the nine period bands, but a convenient alternative is to use three period band, e.g., $T < 0.4$ second, $0.4 \text{ second} < T < 2.0$ seconds, and $T > 2.0$ seconds.

Another alternative is to construct maps for only certain significant narrow-period bands. Spectral response curves from the motion of Event FAULTLESS, which was detonated on January 19, 1968, in central Nevada, have been calculated for several stations in Figure 4.20 (A58), which shows iso-EIS lines for the period band $T < 0.4$ second.

4.6.5 Spectral Matrix Method

The Spectral Matrix Method (SMM) was an orderly, standardized procedure for predicting damage to

**TABLE 4.26 COMPONENTS CONSIDERED RELEVANT
FOR LOCAL RESPONSE PARAMETERS**

Component	Relevant Local Response Parameter
Reinforced Concrete Frames	Tangential Interstory drift
Beams	Cantilever Beam Deflection
Columns	Cantilever Column Deflection
Steel Moment-Resisting Frames	Tangential Interstory Drift
Reinforced Concrete Shear walls	" " "
Masonry Walls	" " "
Drywall Partitions	" " "
Glass	" " "
Suspended Ceiling	Floor Acceleration
Mechanical Equipment (Boilers, Fans, Pipes, Elevators, etc.)	Floor Acceleration or Tangential Interstory Drift

FIGURE 4.19 ENGINEERING INTENSITY SCALE MATRIX SUPERIMPOSED WITH EXAMPLE SPECTRUM

FIGURE 4.20 ISO-EIS LINES FOR PERIOD BAND OF $T < 0.4$ SECOND, EVENT FAULTLESS

structures subjected to seismic ground motion. This method, based on observed data and theoretical considerations, was developed by Dr. John A. Blume (A35, A58, B61). Originally, SMM treated ground motion demand as a matrix of discrete points obtained from the pseudo relative velocity curve. A later version treated ground motion demand as a continuous function. For predicting building damage caused by ground motions, the method considered the following factors:

- Amplitude and frequency distribution of ground motion
- Foundation conditions
- Elastic strength, capacity, and damping of structures
- Inelastic reserve strength of structures for dissipating excess demand of communicated energy
- Probabilistic evaluation of the random variables inherent in the prediction problem

Building damage was expressed by SMM in two ways. First, SMM could be used to establish the total dollar cost to repair or replace buildings that might be damaged by an expected ground motion. The total number of damaged buildings for which reparation was made, without regard to the amount of damage cost, was a second measure of damage that could be estimated by SMM; for this measure, buildings were classified as being either damaged or undamaged.

Two theoretical SMM parameters, quantitatively defined, were used to predict damage to buildings from ground motions: the damage state, DS, measures whether a building was damaged or not; the damage factor, DF, measures the degree of damage in terms of the ratio of repair cost to building replacement value.

Because of the many uncertainties associated with predicting building damage caused by ground motion, a pre-event forecast could only approximately predict the state and degree of damage. This lack of precise prediction occurred because of the necessary modeling assumptions and the fundamental uncertainties present in evaluating demand and capacity. These uncertainties required demand and capacity to be treated probabilistically, which caused both DF and DS to be ultimately expressed in statistical terms.

The damage factor, DF, and the damage state, DS, parameters were treated above as deterministic quantities. In reality, the underlying variables were uncertain, and it follows that damage would not be predicted exactly. The damage parameters depend on the demand, D, the capacity, C, ultimate ductility

capacity, and an empirical economic factor. All of these factors were uncertain, and a rigorous approach would treat each probabilistically. However, the most sensitive variables were demand and capacity, and because of the lack of reliable information for the other variable, the SMM damage parameter probability models treated only demand and capacity as random variables. This limitation was not serious, as the model has been shown to fit empirical motion-damage data for both low and high-rise buildings.

Whether considering damage to a single building, a group of buildings, or every building in the vicinity of a blast, the underlying assumption was that stable probability functions of damage exist. For example, the damage probability functions for both a single building and total damage to many buildings will have the same general shapes.

Demand as quantified by response spectral ordinates was assumed to be lognormally distributed. A lognormal probability distribution was completely defined by a median value and a geometric standard deviation. The capacity probability distribution used in SMM was the Weibull distribution. A Weibull distribution was selected because theoretical studies indicated that material strengths were represented well by this function. In addition, the theoretical damage estimates, using a Weibull distribution, produced the best fit to empirical motion-damage data. The Weibull distribution requires three parameters for definition.

The SMM provide rational predictions of the dollar damage and subsequent number of paid damage settlements that might occur from UNE ground motion. Rather than predicting only single damage numbers, the SMM provided could predict statistics of damage probability distributions to describe an uncertain situation realistically. The same procedures, when suitably calibrated, could be applied to predicting damage from earthquakes.

4.6.6 Threshold Evaluation Procedure

The Threshold Evaluation Procedure (TEP) for predicting the effects of dynamic ground motion on structures involved a systematic and detailed dynamic structural analysis of individual buildings (A58). Threshold evaluation analyses were conducted when it had been previously determined (from less rigorous approximate methods) that certain structures were expected to be damaged and (1) these structures were of significant economic or social value, (2) the damage posed a hazard to persons, or (3) the result had a consequential environmental impact. The principal objectives for conducting such a detailed analysis were to identify the potential risk concerning a structure's failure as well as to outline

possible modifications that might improve the lateral force resistance of that structure. The principal steps in a threshold evaluation analysis were (1) the identification of damage or failure criteria (capacity thresholds), (2) structure modeling and response analysis (spectral demand), and (3) reconciliation of demand and capacity.

Many mechanisms of distress and failure, or capacity thresholds, needed to be determined for a high-rise building or other civil engineering structure. It was essential, either explicitly or implicitly, to consider several if not all of them. Among the many elements that made up a structure, the failure of some were much more important than others. An upper story beam overstressed in flexure, for example, was less important than a base story column overstressed in compression or in shear.

The Spectral Matrix Method, described in preceding section 4.6.5, used statistical determinations of building capacity. On the other hand, the TEP was based on a deterministic computation of the subject building's capacity. This difference arose both because it was practicable to establish specific capacities for individual structures and also because the TEP was intended to predict levels at which various response thresholds would be reached, rather than the accumulative damage predicted by the SMM.

The following seven capacity thresholds were usually the most useful in evaluating a building:

- Code Required Threshold was based on the base shear force from the applicable Uniform Building Code under which the building was designed. The greater value obtained from the seismic or wind requirements controlled this threshold.
- Working Stress Threshold was the ground motion-generated lateral force that caused a structural member to reach the allowable working stress as defined by applicable building codes. Structural members included major elements such as beams, columns, shear walls, and slabs.
- Yield Limit Threshold was the ground motion-induced lateral force that causes a structural member to reach yield stress. Unless more specific data were available, standard code-defined yield stresses were used.
- Human Alarm Threshold corresponds to a level of building motion that might alarm occupants. It was set at a value corresponding to a peak acceleration at the top of the

building equal to 0.05g. The value was based on the results from laboratory tests and field observation.

- Observable Damage Threshold was based primarily on the probable occurrence of non-structural cracking or minor structural damage in the form of concrete spalling for minor structural elements. It was normally set at a value corresponding to an average interstory displacement of 1/16 inch. This was based on the wall panel racking tests which indicated that the partition damage occurred at about 1/8-inch relative interstory displacement. The 1/16 inch threshold was used because the maximum interstory displacement of a high-rise building in a fundamental mode of vibration was observed to be approximately twice the average interstory displacement. This value was adjusted for special building situations.
- Human Hazard Threshold corresponded to a level of building motion that may cause building occupants to be injured by either personal overreaction or falling objects. This threshold was set at twice the Human Alarm and Observable Damage Thresholds, i.e., an average interstory displacement of 1/8 inch or a peak acceleration at the top of the building equal to 0.10g, whichever was less. These criteria were also adjusted for special building situations.
- Story Failure Threshold was based on either a brittle story shear failure or a story mechanism failure. A mechanism mode of failure occurs when all the columns on a floor level begin to yield. The brittle shear mode of failure was based on an ultimate shear stress failure of the major lateral shear-resisting members or elements for a story.

The capacity thresholds were initially in the form of the base shear coefficients that correspond to the given threshold condition. However, equivalent spectral values were determined to facilitate comparison of capacity thresholds to demand.

Either a time-history solution or a response spectrum analysis could be used to develop expressions for the response of the structure to the demands from dynamic ground motion. However, the response spectrum approach was used because the technology for estimating the variability of predicted ground motions was available only in the response spectrum form for the DOE/NV program. The demand on a structure was expressed probabilistically as a variation from a median spectral value (usually pseudo

relative velocity). With demand, D , and capacity, C , expressed in like units, the probability of demand exceeding capacity was determined using a lognormal probability density function for the demand.

If a threshold was predicted to be exceeded, the consequences of that exceedance had to be identified before the risk could be evaluated. In most cases, this would vary from structure to structure. If only the Code Required Threshold was exceeded, there was little chance that damage will result; any damage that did occur was expected to be minor.

Exceeding the Human Hazard Threshold for a building might or might not have an associated small monetary loss and might include chance personal injury, but it was not likely to include life loss. The principal risk involved in exceeding the observable damage threshold would be small monetary loss.

Exceedance of the Story Failure Threshold would imply substantial damage (monetary loss) and life loss. The life loss risk, of course, could be precluded by simply evacuating the building. The particular circumstances of the architectural detailing and occupancy of a building would have to be considered in specific evaluations of the risk associated with exceeding the various thresholds.

5. RESPONSE EVALUATION FOR NTS EVENTS

5.1 CHRONOLOGY OF NTS EVENTS

All announced U.S. nuclear tests and their dates, locations, types, purposes, and yields or yield ranges are listed in Reference E10, available from the National Technical Information Service. Events conducted at the NTS can be grouped generally into three yield level groups, less than 20 kilotons (kt), 20 to 200 kt (20 to 150 kt after March 1976), and greater than 200 kt (see Reference E10 for exact yield ranges). Events of less than 20 kt had no seismic impact on the communities outside of the NTS, and Blume had little to do in terms of supporting the test program, except to collect data used for development of motion prediction. Essentially all of Blume's studies and investigations were directed toward providing support for the safe conduct of the test for the events greater than 200 kt, especially those in the megaton (1000 kt) range. To describe the structural response program in support of the DOE/NV test program, the period can be grouped into four time frames, from the BILBY event through the HANDLEY event, after HANDLEY until the start of the Accelerated Test Program, the period of the Accelerated Test Program, and after the Accelerated Test Program.

Blume became involved with the seismic safety program for DOE/NV (E4, E6) after the 249 kt BILBY event of September 13, 1963, the first underground test reported felt in Las Vegas. During Fiscal Year 1964 (from July 1963, through June 1964), there was one event with a yield greater than 200 kt and five in the 20 to 200 kt range (Table 5.1). During the next fiscal year (FY-65), there were 5 events in the 20 to 200 kt range and none greater than 200 kt. This lull in the test program gave Blume and other seismic safety contractors time for the development of techniques and procedures in the prediction of effects for the larger events to be conducted in the future. During FY-66, one event with a yield greater than 200 kt was conducted; the 365 kt HALFBEAK event. In addition 14 tests in the 20 to 200 kt range were conducted. The GREELEY event with a yield of 870 kt was conducted in FY-67, along with 6 tests in the 20 to 200 kt range and one at 250 kt. At least one test in the megaton range was conducted during each of the next three years. After the HANDLEY event in March 1970, the test program was scaled down in size as well as in the number of tests conducted.

During the period from FY-71 through FY-74, a limited number of tests were conducted at the NTS. There were two tests greater than 200 kt and 16 tests in the 20 to 200 kt range. Testing outside of the NTS was similarly reduced. Work and staffing requirements for the contractors were drastically

TABLE 5.1 SUMMARY OF NTS EVENTS

<u>PERIOD</u>	<u>20 TO 200KT</u>	<u>GREATER THAN 200KT</u>
FY-64 7/63 - 6/64	5	BILBY (249KT)
FY-65 7/64 - 6-65	5	-
FY-66 7/65 - 6/66	14	HALFBEAK (365KT)
FY-67 7/66 - 6/67	6	GREELEY (870KT), COMMODORE (250KT)
FY-68 7/67 - 6/68	12	BOXCAR (1.3MT)
FY-69 7/68 - 6/69	13	BENHAM (1.15MT)
FY-70 7/69 - 6/70	17	JORUM (<1MT), PIPKIN (200-1000KT), HANDLEY (>1MT)
FY-71 7/70 - 6/71	5	CARPETBAG (220KT)
FY-72 7/71 - 6/72	3	-
FY-73 7/72 - 6/73	6	ALMENDRO (200-1000KT)
FY-74 7/73 - 6/74	2	-
FY-75 7/74 - 6/75	9	TYBO, MAST, CAMEMBERT (200-1000KT)
FY-76 7/75 - 9/76	5	KASSERI, INLET, MUENSTER, FONTINA (200-1000KT); CHESHIRE, ESTUARY, POOL, STRAIT (200-500KT); COLBY (500-1000KT)
FY-77 10/76 - 9/77	7	-
FY-78 10/77 - 9/78	10	-
FY-79 10/78 - 9/79	11	-
FY-80 10/79 - 9/80	6	-
FY-81 10/80 - 9/81	3	-
FY-82 10/81 - 9/82	14	-
FY-83 10/82 - 9/83	5	-
FY-84 10/83 - 9/84	8	-
FY-85 10/84 - 9/85	8	-
FY-86 10/85 - 9/86	9	-
FY-87 10/86 - 9/87	8	-

reduced, creating some problems in gearing up for the following sudden and steep increase in testing activity, which constituted the Accelerated Test Program (ATP).

During the 10 month period of the ATP in FY-75 and FY-76, 12 tests with yields greater than 200 kt were conducted and 14 tests in 20 to 200 kt range. Subsequently, from FY-77 to the present, all of the tests have been less than 150 kt. The number of tests in the 20 to 150 kt range have varied from 3 to 14 per year.

5.2 CHANGES IN THE AFFECTED AREA

The Nevada Test Site is located in Nye County of southern Nevada. Nearby communities include Tonopah, Goldfield and Warm Springs located to the north; Caliente, Pioche and Alamo to the east; Beatty, Furnace Creek and Scotty's Castle to the west; and Indian Springs and Pahrump to the south. However, the area that had the most impact on the test program was the Las Vegas valley (Figure 5.1).

As shown in Table 5.2, over 95 percent of the population in the affected area is in Clark County. In 1960 the population of the Las Vegas area was about 100,000. By 1987 the population had increased to about 550,000. Major changes in population in the nearby small communities have been those of Tonopah which serves as a mid-point between Las Vegas and Reno and in which the population fluctuates with the mining industry, and of Pahrump which has made several attempts to develop a residential community. A more recent development is the sudden growth of Laughlin located near the southern tip of Nevada with construction of two high-rise buildings and several more under development.

When the underground testing program began, the residential and commercial building population covered a fairly small area of Las Vegas. Presently, the building population covers a much wider area and additional growth is forecast. A more significant impact to the test program has been the construction of high-rise buildings in Las Vegas. The number of high-rise buildings in Las Vegas has grown from 8 in 1964 to over 130 in 1988. A more detailed discussion on growth of high-rise building population is presented in Section 5.4.

5.3 EVENT SUPPORT ACTIVITIES

All tasks performed by the seismic safety contractors were directed toward the safe conduct of the DOE/NV testing program (E3, E4, E6). Some were on-going activities with long range objectives

FIGURE 5.1 NEVADA TEST SITE AND NEARBY COMMUNITIES

TABLE 5.2
POPULATION OF COMMUNITIES NEAR THE NEVADA TEST SITE

<u>County (City)</u>	<u>1960</u>	<u>1970</u>	<u>1980</u>	<u>1986</u>
Esmeralda (Goldfield)	619	629	777	1,380
Nye (Tonopah, Warm Springs, Beatty, Pahrump)	4,373	5,599	9,048	14,970
Lincoln (Alamo, Caliente, Pioche)	2,341	2,257	3,732	4,200
Clark (Las Vegas, No. Las Vegas, Henderson, Laughlin)	127,016	273,388	463,087	616,778

while others provided support for specific tests. The long range on-going activities performed by Blume are discussed earlier in this report, but these activities provided input which allowed evaluation and prediction of the effects of ground motion for specific tests. Three major activities performed by Blume were the development and application of structural response procedures, the conduct of field monitoring program, and the damage complaint investigations. The level of effort for these activities varied with the program depending on test program as well as the progress in the development of the structural response program. For example, in the early part of the program, major efforts were directed toward the collection of information and data and the development of prediction techniques and procedures. To review the history of the test support activities, the following discussion is divided into three periods: from the initiation of the program in 1964 through the 1970 HANDLEY event, the greatly reduced activity after HANDLEY followed by sharply increased requirements for the 1975-1976 ATP, and the period following the ATP and continuing until the present.

5.3.1 BILBY Through HANDLEY

At the start of the structural response program, the first order of business for Blume was to obtain all available information on the high-rise buildings that existed at the time. A survey of high-rise buildings was made, but Blume had yet to establish a working relationship with the building owners to obtain necessary detailed structural information. Blume's access to the hotel-casinos was hindered partly because of building management's concerns arising from then-current publicity regarding supposed wiretapping activities within the casinos. With continued liaison efforts, the building owners eventually cooperated fully in providing the architectural and structural information needed to perform structural analyses. In early 1964, a request was made to DOE/NV to obtain measurements of structural response records from the high-rise buildings for the upcoming TURF event. Blume personnel were not able to participate in the pre-event planning and in field observations for the event because the staff had not yet been granted necessary security clearances. However, the Special Projects Party operated three portable Sprengnether Blastmeters to measure response of high-rise buildings for Blume. The records obtained from TURF were scaled to determine the response of the buildings. Although the amplitudes of motion and the response periods of the buildings were beyond the capability of the instruments, some useful information was obtained. In addition to the evaluation of the high-rise buildings, Blume was requested to conduct a damage investigation resulting from the BILBY event of September 13, 1963. These activities set the tone for the event support activities of the Blume organization for the years to come.

Structural Response Program While the structural response research program continued, the event support activities included the development and implementation of data acquisition and processing

programs, the analysis of structures to predict structural response, and the evaluation of response and effects of the ground motion on the structures. For each significant event a prediction of event ground motion was provided by ERC. Based on the ground motion information, a prediction of structure effects was made. For major events, the impact of the event as well as any recommended remedial measures was presented to DOE/NV and its panels and committees. After an event, a post-event evaluation was performed and presented.

Field Monitoring and Event Participation Since the high-rise building population was quite small during the early years of the program, the field monitoring program was limited to obtaining information for specific on-going programs, and all office activities were conducted at the Blume home office. A field office with a small staff was established in Las Vegas in late 1964, to coordinate field activities, perform liaison with the building owners so that building information could be obtained, conduct surveys of buildings and their conditions, and conduct special measurements to supplement the seismic measurements. Field parties were dispatched on occasion from the home office to participate as observers on the high-rise buildings for the major events. As size of the tests increased along with a steady increase in high-rise building construction during this period, the field participation by Blume increased. Major participation efforts by Blume were required in, (1) response to testing opposition by Howard Hughes who had purchased several high-rise buildings in Las Vegas shortly before the BOX-CAR event of April 1968, (2) for response to a lawsuit on an alleged damage resulting from a test, (3) for the seismic strengthening of a high-rise building in Las Vegas and structures at NRDS for the HANDLEY event, and, (4) to conduct an assessment of impact and for coordination with a building contractor on problems in the construction of a particular high-rise building. Beginning in 1970, Blume was required by DOE/NV to relocate a major part of the technical support operation to Las Vegas and to perform, in that office, essentially all of the research and developmental work and for the completion of the structural response prediction guidelines. Similarly, ERC also relocated its technical staff to Las Vegas to conduct its work and to develop ground motion prediction guidelines. With the completion of the 1970 HANDLEY event which was envisioned as the last of the major tests and the completion of prediction guideline reports, Blume contract efforts and event support activities were scaled sharply down, with a matching reduction in the technical staff.

Damage Complaint Investigations As noted above, the first damage complaint investigation was conducted in 1964 for a complaint resulting from the BILBY event. The investigation was investigated by an architect, a structural engineer, and a geologist from Blume. Numbers of complaints and subsequent formal claims have increased as the size of events increased.

The damage complaint investigation procedure had remained essentially the same. Initially, the complainant would make a complaint to DOE/NV, usually by telephone. The General Adjustment Bureau (GAB) performed the initial contact and evaluation, and would provide a form if the complainant wished to submit a formal claim. If a claim could not be resolved or required a more detailed investigation by professionals, Blume was requested by DOE/NV to provide the investigative support. The claims were investigated and the results of the investigation were submitted to DOE/NV for its use. The GAB used the information to aid in the settlement of the claims.

5.3.2 After HANDLEY Through The Accelerated Test Program

With completion of the HANDLEY event and reduction in the size and number of events, the scope of work of the Blume efforts was reduced. From the end of the HANDLEY event and the start of the Accelerated Test Program (ATP) in May 1975 (a period of approximately 5 years), only two tests greater than 200 kt and 28 tests in the 20 to 200 kt range were conducted at the NTS. In addition there were very few off-NTS support activities performed. By 1974 the staff of the Blume organization in support of the program was reduced to less than half of the staffing during the HANDLEY period. Event support activities were limited to coordinating with the Special Projects Party for the instrumentation of high-rise buildings to obtain measurements, for scaling and analysis of the recorded data, for supplementing the structural response prediction procedures with the new data, for collection of structural information for new construction and limited analysis of the structures, and for providing damage claim investigation support.

With initiation of the Accelerated Test Program (ATP) in 1975, an immediate increase in the support activities for the structural response program was necessary. The ATP consisted of a series of closely-spaced high yield NTS events. An anticipated US-Soviet agreement on limiting test yields would prevent such tests, and the program expedited all planned high yield events to complete them before the US-Soviet agreement was to be enacted.

In order to determine the effects of ground motion generated from the planned series of high-yield detonations, Blume was required to conduct studies to provide effects prediction, with a lead time of about 9 months. Unlike the tests conducted in the past with the large events were conducted at about 6 months interval permitting Blume to complete analyses of any new high-rise buildings for their potential damage thresholds, the ATP required that many high-rise buildings had to be analyzed and ground motion effects determined within a short period. Complicating this process was a major reduction in technical staff during the early-1970's lull in the test program and a major construction boom that took

place during 1973 and 1974 with a construction of about 30 high-rise buildings. In addition many of the buildings were of new construction using post-tensioned flatplate slabs, pre-cast slabs, and concrete masonry shearwalls. Only limited information was available relative to the strength of these materials and systems. With the limited time available to perform a detailed analysis of all of the new buildings, it was necessary to develop an effective procedure to identify those structures that might be adversely impacted by the ground motion. This was done by performing a preliminary evaluation of all of the structures to conduct assessment of the vulnerability of the structures to the event-generated ground motion. Those structures judged to be impacted most from the ground motion were analyzed in detail to determine structural thresholds. Based on the ground motion prediction provided by ERC, estimates of impacts on the high-rise buildings as well as on the other structures were determined. A presentation was made to the Ground Shock Evaluation Committee for each major test conducted during this period. There were 12 tests with yields greater than 200 kt conducted during the 10-month period from May 1975 through March 1976(E10).

With the increase in the size of the detonations and in the building population in the communities surrounding the NTS, there was a large increase in the number of damage complaints and claims during this period. The Blume organization conducted investigations of most of the complaints and any resulting claims.

5.3.3 After The Accelerated Test Program

With the completion of the ATP and the agreement between the Soviet Union and the United States to limit the maximum yield of underground tests to 150 kt, the structural response program was again reduced. The scope of work also changed. During the period from BILBY through HANDLEY, the program's emphasis was on the development of the techniques and procedures for the effective prediction and assessment of the impact. With techniques and procedures pretty much established, the emphasis during the next phase was in their application. Although the structural analysis of the new high-rise construction and the damage complaint investigation support has continued, current emphasis is on the collection and documentation of the information and on the maintenance of the information and data collected in the past.

When the USGS and its special projects party relinquished the seismic instrumentation program in 1979, Blume was assigned the instrumentation, record processing and record maintenance responsibilities for the DOE/NV seismic safety program. With these added responsibilities, the event support activities became a large part of the Blume effort. The new responsibilities in terms of event support activities

include liaison with the high-rise building managers in regard to the event motion observation and to the deployment of instruments, coordination of instrumentation requirements, maintenance and deployment of the equipment, operation of the equipment during the events, operation of the processing equipment and the processing of the records, and maintenance of the seismic record library. In 1983 DOE/NV made Blume also responsible for the ground motion prediction responsibilities formerly assigned to ERC. This effort includes providing DOE/NV a prediction of ground motion for each event and a development of ground motion prediction equations at new locations as required by DOE/NV.

5.4 HIGH-RISE BUILDING CONSTRUCTION IN LAS VEGAS

The BILBY event of September 13, 1963, at the Nevada Test Site, generated ground motion that caused a clearly perceptible motion on the upper floors of tall buildings in Las Vegas. This perception of motion from the event accelerated a monitoring program to document, evaluate, and determine the impact of seismic ground motion on the buildings. A part of the responsibilities assigned to Blume was to keep abreast of the high-rise construction in Las Vegas. During the course of the Blume involvement from 1963 to the present, the Las Vegas area changed from a small city to a major metropolitan area. In the early 1960s the population of the Las Vegas area was approximately 100,000. By 1987 the population had grown to over 550,000. More dramatically, the number of high-rise buildings had increased from 8 to 130 during the same period. Also the types of construction had changed from a traditional concrete frame structure with shearwalls to construction of different materials and techniques. There had also been a radical change in design and analysis techniques with use of modern computers and computer software. In addition, the building design and construction requirements had changed. All of these changes brought about dramatic changes in the structural response program. Public and state government attitudes regarding NTS activities have also undergone significant changes in recent years.

5.4.1 Number of High-Rise Buildings

The first high-rise building was constructed in Las Vegas in 1955. From 1955 to 1963 seven additional high-rises were built. At the time of BILBY event in 1963, there were 8 high-rise buildings with the tallest being a 24-story hotel. There were two buildings under construction at the time. The number of new high-rise structures completed averaged about two per year during this period until early 1970. During this period a number of UNEs were conducted at the NTS with 6 events larger than the BILBY event. These were HALFBEAK (6/30/66), GREELEY (12/20/66), COMMODORE (5/20/67), BOX-CAR (4/26/68), BENHAM (12/19/68), JORUM (9/16/69) and HANDLEY (3/26/70). By the time of

the HANDLEY event, there were about 25 high-rise buildings in Las Vegas.

After the HANDLEY event and until TYBO event of May 1975, there were only 2 tests greater than 200 kt at the NTS. All other tests conducted were less than 200 kt. During this period, funding for the testing support was drastically reduced, precluding any new structural analyses. During 1974 and 1975, there were about 30 high-rise buildings constructed, with a resulting total of about 65 in Las Vegas. The structural response investigation efforts were reactivated in 1974 to determine the effects of the ground motion from the forthcoming ATP, a series of high-yield detonations at the NTS.

After the ATP, all of the tests conducted at the NTS have been less than 150 kt. Building construction has continued. By mid-1988, there were over 130 high-rise buildings, and construction continued at a rate of about 6 buildings per year.

5.4.2 Types of High-Rise Buildings

In addition to the increased number of high-rise buildings constructed in Las Vegas since the start of the UNE test program at the NTS, there have been major changes in the types of construction. Figure 5.2 shows the different types of high-rise buildings constructed in Las Vegas. The earlier buildings were of more traditional concrete construction. Over the years new construction types have been introduced that take advantage of modern construction materials and techniques.

When the program began, all 8 high-rise buildings that existed at the time were of concrete construction. Vertical loads were carried by the reinforced concrete frames and the lateral loads were resisted by reinforced concrete shearwalls. In 1965 two structures were constructed that were reinforced concrete frames without shearwalls and one structure of steel framing with reinforced concrete shearwalls in one direction. Although the two concrete frame structures did not have any shearwalls designed as such, the architectural concrete block walls acted as shearwalls in response to lateral forces.

The next change occurred in 1970 with the construction of a concrete masonry block wall building. The concrete block walls carried the vertical load as well as resisting the lateral loads. Floor slabs were constructed of precast concrete slabs. Another structure with similar construction was built in 1972. This type of construction peaked during 1974 and 1975 with construction of 12 buildings. In this type of construction, the walls are located at each frame line and also function as partitions for individual rooms. The walls are continuous in most cases from the first floor to the roof and contribute greatly to the lateral force resisting capability of a building. However, in order to accommodate shops and

FIGURE 5.2 LAS VEGAS HIGH-RISE CONSTRUCTION (5-YEAR PERIOD BY TYPE)

casinos, some walls become discontinuous or are replaced with columns at the first floor. This discontinuity has introduced more flexible first floors and in some cases introduced torsional behavior of the structure. In 1975 a building was built with precast slabs as well as precast walls.

The next major change occurred in 1971 with the construction of a concrete frame structure with post-tensioned flatplate floor slabs. This type of floor slab is an economical system to resist the vertical loads. Prefabricated concrete forms (termed "flying form") that can be used from floor to floor provide an efficient way to construct this type of structure. However, this type of structural system has very little lateral force resisting capability. In most buildings of this type, the lateral force is designed to be resisted by reinforced concrete or masonry shearwalls. Another building of this type was constructed in 1973. Nine buildings of this type were built during 1974 and 1975. This type of construction became quite popular and became one of the major construction techniques in many of the high-rise buildings in Las Vegas.

5.5 CHANGES IN UNIFORM BUILDING CODE AND THEIR EFFECTS

The Uniform Building Code (UBC) is a national building standard, promulgated by the International Conference of Building Officials (ICBO), which can be adopted by local government agencies to provide minimum requirements to ensure public safety in buildings. The UBC is revised periodically to reflect technological changes and advances in the field. The trend in the seismic design requirements has generally been toward increased seismic forces, increased complexity, and extended scope of applicability. Clark County and the city of Las Vegas have historically adopted this code and made its provisions the governing standards. The first high-rise building in Las Vegas was constructed in 1955 in conformance with the 1952 provisions of the UBC. Since 1955 the trend in seismic design requirements referred to above has imposed gradually more restrictive standards for code-designed structures in Las Vegas. In addition, a 1970 change in the geographical seismic zoning for the Las Vegas area, as shown in Figure 5.3, doubled force design requirements for that area.

The UBC includes a formula for determining the minimum total lateral seismic force assumed to act on a structure. This total lateral force, V , also called design base shear, is calculated from several basic parameters. These parameters account for the seismic activity of the area as determined from the assigned seismic zoning, the type of structure framing system, the fundamental period of vibration of the structure, a site factor which accounts for potential amplification of structural response by foundation soil characteristics, an occupancy factor reflecting the potential threat of structural damage to

A. 1952 - 1969

B. 1970 - 1975

C. 1976 - 1987

D. 1988

FIGURE 5.3 UBC SEISMIC ZONES FOR SOUTHERN NEVADA

occupants and to the vicinity, and the total weight of the structure.

A parameter called the base shear coefficient, i.e., the design base shear for a structure divided by the total weight of the structure, is shown plotted in Figure 5.4 for purposes of illustrating the effect of code changes on Las Vegas design requirements. Curves shown are for the UBC changes from 1955 to 1988.

The UBC requires all structures to be designed to resist seismic forces as described in the previous discussions. However, the code also stipulates that where prescribed wind loads produce higher stresses, those loads shall be used in lieu of the seismic forces. Prior to 1970 when Code seismic forces dramatically increased, wind forces often governed design, particularly for the axis perpendicular to the broad face of the building.

Some of the other impacts and observations are discussed in subsequent sections.

5.6 HIGH-RISE BUILDING DESIGN AND ANALYSIS TECHNIQUES

Much progress has been made in the analysis and design techniques over the years. With the aid of modern computers and applicable analysis and design programs, engineers can design a building much more quickly and precisely. This improvement in the technique and procedure brought about much refinement in the amount of materials used, thereby allowing cost savings in the construction of buildings and the construction of much more complex structural systems. A result is that there is less of the conservatism that was inherent in the earlier designs. Consequently, even though a building may be designed according to a higher seismic coefficient because of the doubling of the zoning coefficient in 1970, a building may not necessarily be stronger by a factor of two as it might seem.

Design of buildings to resist lateral force is dictated by the UBC with the use of a base shear coefficient. The base shear coefficient is a function of the seismic zone, structure type, facility factor, soil factor, and fundamental period of building. Most of the coefficients are fairly well defined or specified by the code. However, in prior editions of the UBC, the designer was given the choice of empirical building periods or a period calculated by any rational method. Since the calculation of the building's period, particularly for framed buildings, is sensitive to the assumptions made as to fixity of the base, centerline versus clear dimensions, rigidity of the floor system, and other judgmental considerations, a wide range of periods could be calculated for any given building. Since the lateral force coefficient, C ,

FIGURE 5.4 COMPARISON OF BASE SHEAR COEFFICIENTS

was calculated by $C=1/(15 \cdot t^{0.5})$, an imprudent building designer could use a period that would require a substantially reduced base shear as compared to that from the code-specified empirical building periods. The net effect was that the some of the new buildings, constructed since the Seismic Zone of southern Nevada was doubled from 1 to 2, were designed essentially as those constructed before the Zone change in 1970.

The 1988 edition of the UBC was recently issued. The application of this code would alleviate many of the problems and shortcomings observed in the past for the construction of new buildings in the area. For example the new code restricts the period coefficient to 80% of the value determined using the standard method. Of course, it would have no effect on buildings that were designed and built under the previous codes.

5.7 OBSERVATION OF HIGH-RISE BUILDING CONSTRUCTION

The changes in the building code generally increased the seismic design requirements thereby increasing the lateral force resisting capability of new buildings. Some of the offsetting factors were the use of new construction techniques and materials, the improvement in analysis and design techniques, and the interpretation of requirements by design engineers. Some of the impacts of these changes are discussed following.

5.7.1 Use of Concrete Masonry

Use of concrete masonry as structural walls had been a common practice in building construction, but its use in high-rise buildings has been more recent. Since 1969, its use in place of reinforced concrete has become popular in high-rise building construction. As in any new construction with use of new materials, the tests necessary to evaluate and determine their strength and failure mechanism lagged behind those of concrete construction. Consequently, the design of masonry structures had been based on the working stress capacity of the materials. The use of ultimate strength design had been just recently introduced. There had been very little guidance and restriction by the building codes on its use as a major lateral force resisting elements. In the design of shearwalls with concrete, the code requires extensive use of reinforcing steel on the ends of shearwalls to carry the overturning moment and for the confinement of concrete. In concrete masonry design, however, similar placement of reinforcing steel is not possible due to the restrictive space for the steel. Consequently, shearwalls built with reinforced concrete masonry will have less ductility than those built with reinforced concrete.

5.7.2 Use of Pre-Cast Floor System

In conjunction with the introduction of concrete masonry as a structural material for high-rise buildings, precast concrete planks were introduced as floor slabs. Typically, such planks are prefabricated in a factory under controlled condition and shipped to the construction site for installation. The precast planks are placed on top of the concrete masonry walls and anchored by means of reinforcing steel between the wall and the slab or with welding of metal plates placed in the slab.

Differential Movement Vertical deformations of the floor slab depend on dimensions and properties of the slab. If different loads were placed on two identical slabs, the one with the heavier load would deflect more than the other. Similarly, when the same unit floor loads are applied to slabs with different lengths, a longer length slab would deflect more than the other. To minimize the differential vertical movement, connectors are installed in the precast slabs and are connected to each other to prevent this independent movement. Omission or failure of these connectors has been found in some of the construction, resulting in cracks in floor slab along the joints as evidenced by plaster cracks in the ceilings below these slabs.

Anchorage The lateral forces from the floor slabs must be transmitted to the adjacent walls through connectors. Without the connectors, the only means of force transmission is through friction between the slab and the walls. To transmit the force, connectors or reinforcing steel dowels are typically used. Observations made during the construction of these types of building indicate that some lack adequate anchorage to transmit the lateral load.

Field Fabrication of Slabs The precast slabs are typically and most often fabricated under controlled conditions in a factory to standard dimensions or as specified by the designer. When irregular sizes of precast slabs are used, or when construction dictates, the slabs may be fabricated in the field. To fabricate the slabs in the field, field casting beds are established, normally with capability to control the environmental conditions. It has been observed that slabs have been fabricated in the field during the peak summer months in Las Vegas without controlled conditions.

5.7.3 Post-Tensioned Flat-Plate Construction In traditional concrete construction, the vertical load is carried by a reinforced concrete floor slab in such a manner as to take advantage of the compressive capacity of concrete and the tensile capacity of reinforcing steel. In order for the floor slab alone to carry the vertical load and to transfer the load from the slab to widely spaced vertical load carrying elements such as columns and walls, the slab might need to be relatively thick. However, to reduce the

amount of concrete, reinforced concrete beams, column capitals, and waffle slabs have been used to supplement the vertical load carrying capability of slabs. To further reduce the amount of concrete and to simplify construction, a floor slab using post-tensioned tendons was introduced. The use of the post-tensioned flatplate is an economical way to carry the floor load. The first use of this concept was introduced in Las Vegas high-rise construction in 1970. In this construction, post-tensioned tendons (stranded wires) are anchored on one end in plastic sleeves and draped across the floor slab. After the concrete is poured and cured, the tendons are pulled from one end with a specified force, and then anchored. This in turn produces an uplift force on the floor slab. With the use of this technique the distance between columns has been increased from a traditional distance of about 15 to 18 feet to a distance of 25 to 30 feet. In addition, a typical floor slab thickness has decreased from 8 to 10 inches to about 6 to 7 inches. This method of construction is a cost-effective way to carry the vertical load, but it has certain limitations. One, in order for the post-tensioning to be effective, the amount of "mild" reinforcing steel must be limited; otherwise the post-tensioning will be compressing the steel instead of the concrete and only a limited force will be exerted on the concrete. Two, with limited thickness of the floor slab, the cross-sectional area available to transfer the load from the floor slab to the column is limited. Three, because of limited amount of steel and the thickness of the slab, there is little lateral force resisting capability. The lateral force must be resisted by other means. The promoters of this construction technique suggest shearwalls to resist the lateral force, and reinforced concrete or masonry shearwalls are typically used in most cases. Unfortunately, however, there have been a few cases in which buildings of this type in Las Vegas were constructed without such shearwalls. The construction of post-tensioned flat-plate system without shearwalls was limited to the period from early to mid-1970s.

Several other characteristics were observed in this type of construction. The floor slab of the top floor that is exposed to the sun, as in a parking garage, may exhibit excessive deflection because of temperature effects. On one building the floor slab had deflected almost 2 inches. This deflection had caused the underside of the floor slab to crack. In some cases such cracks were about 1/4 inch wide. Cracks have been observed along the mid-span between the column lines through the entire length of the floor. In addition cracks have been observed on the top of the slab above a column.

In instances where shearwalls are located at the ends of a building on a post-tensioned flat-plate system, diagonal cracks have been observed in the shearwalls. This condition appeared on the lower floors. Studies indicated the cracks were caused by a diagonal tension force produced by a combination of the post-tension force and the restraint of the wall. On the other hand, if this type of building is construct-

ed without the shearwalls, the post-tensioning process introduces horizontal forces resulting in cracking in the supporting columns. An observation has been made in which the post-tensioning introduced cracks in the supporting columns. These conditions have been observed on several buildings in mid-1970s and are still observed today in similar construction.

5.8 SPECIAL OBSERVATIONS AND STUDIES

During the course of the structural response program, observations have been made on the construction of many of the buildings in Las Vegas. Most are constructed in a traditional and typical manner. However there have been a few with unique and unusual features in which it has been possible to observe the influence of architectural components on the structural response, on how the ground motion affect structural response, and on how the structural characteristics affect the response. Particularly unusual situations included a high-rise building in which the architectural partitions provided 50% of the overall stiffness and another in which the building period lengthened about 50% due to a loss of stiffness caused by separation of an infill block wall from the concrete framing.

From 1974 through 1976 the ATP brought about a major flurry in structural response evaluations because of the increase in high-rise building from 25 in 1970 to 65 in 1975; 30 of these were completed during the two year period from 1974 through 1975. During this period, construction changed from conventional concrete shearwall and/or concrete frame and shearwall to either concrete flat-plate buildings without shearwalls or concrete masonry buildings. Additionally, many of the buildings were built in a new area where very little foundation information from the NTS events was available.

5.8.1 Complex Structural Configuration

In Las Vegas, as in many other large cities, the expansion of a building complex often takes place in the form of building additions on top of an existing structure or adjacent to it. The addition on top is normally considered during the initial stage and the structure is designed to accommodate the future addition. The configuration of the structure with the addition remains essentially the same as the original structure. When completed the structure behaves as a single structure, although it will have new dynamic response characteristics. When an addition is made adjacent to an existing structure, the addition is typically connected structurally at each floor to the original structure or is structurally independent with seismic joints separating the structures. The seismic joint is covered architecturally to hide the separation without hindering the movement of the two structures. Occasionally, however, the way in which the addition is connected to the original structure causes likelihood of observable damage,

complex structural response, interaction between the two structures, and other unusual behavior. Some observations regarding building additions in Las Vegas are briefly described in the following paragraph.

During the course of about 10 years in one complex, structures were built adjacent to the original structure. The height and construction materials of all of the structures were essentially the same and each addition was structurally connected to its adjacent structure. What made the construction unusual was that the structures were connected only on a limited number of floors, and the structural systems in the lower floors varied on a few of the buildings. On a few of the buildings the shearwalls were continuous from the ground to the roof, while on others the shearwalls were replaced by beam-column framing in the lower floors. These incompatibilities caused various portions of the structures to respond differently to ground motion.

On another complex, additional stories were added to one of the structures. During the original phase of construction, two structures were built that were approximately the same height. One was a moment-resisting frame system while the other was a frame system with shearwalls. One structure was completed during the first phase and the second structure was to be constructed to approximately the same height with an addition to be built later. There was a 10-inch separation between the structures. The main elevator core was built as a part of the first building and located between the two structures. The elevator core housed 6 elevators and was structurally supported up to the top of this building by 10 reinforced concrete columns and shearwalls. Several years later, the addition to the second structure was made. When the addition was made to the second structure, the elevator core of the first building was extended. For the floors above the height of the first structure, the elevator core was then connected to the second structure. To accommodate the transition and the potential differential movement between the two towers, sliding bearing plates were installed in the columns at the transition location. In case of a large lateral differential movement between the two structures, the sliding bearing plates are designed to permit the independent movement of the structures without inducing an extremely high stress on the columns; without it the differential movement of the structure from the first floor to the transition point (over 100') would be resisted by the columns at the particular floor. On the other hand by permitting the two structures to move independently, the elevator may not be able to function if a large lateral movement occurs at the transition point.

Even though the adjacent portions of the same building may be structurally independent, the architectural finish has not always accommodated this problem. When two unconnected structural portions are constructed adjacent to each other, they should be constructed to move independently and the joints

between the two portions of the structure should be constructed structurally and architecturally to accommodate the movement. The opening may be filled with flexible filler and a sliding coverplate can be placed at the joint to cover the opening and yet allow differential movement. Occasionally, however, wallpaper or other finishes are applied directly over the opening. When a lateral movement takes place, the working of the two structures causes the joint move and produces a crack on the finish material. This is one of the most common conditions observed when two structures are built adjacent to each other.

5.8.2 Seismic Strengthening

During the course of the testing program several structures were strengthened to resist expected forces generated by the tests. Three structures that were subjected major study and seismic strengthening were a high-rise building in Las Vegas, structures at the Nuclear Research Development Station (NRDS), and the TOSCO tower in Colorado. The discussion of the TOSCO tower is presented in Chapter 6.

Las Vegas High-Rise Building Prior to the HANDLEY event in March 1970, the shearwall on a 10-story building in Las Vegas was strengthened. A decision by DOE/NV to have this strengthening performed included the first major application of the Threshold Evaluation Procedure (TEP) described in Section 4.6.6, which included a detailed analysis of the structure and its lateral load resisting system and a use of prediction of ground motion and the associated statistics; evaluation by panels of experts which provided guidance to DOE/NV; and coordination with building owner and contractors. The concept of TEP had been used in evaluation of buildings soon after Blume was hired as the structural contractor. A detailed description and a formal procedure was introduced in 1969 (A58). In 1969 there were fewer than 20 high-rise buildings in Las Vegas, most of which had undergone structural analysis -- those with low threshold or unusual features were analyzed in more detail than some others.

This particular building had an unusual lateral load resisting system which had contributed to the its vulnerability. The building was a 10-story steel frame structure with reinforced concrete shear walls at each end. The ten stories consisted of four lower stories of parking with a plan dimension of roughly 125 ft by 140 ft. The six upper stories consisted of office space with a plan dimension of 69 ft by 140 ft. The east and west end walls were reinforced concrete which encased the structural steel frame. The west wall was a windowless 8-inch thick concrete wall with a large cut-out on each side between the second and the fifth floor. Part of the cut-out area was covered with 8-inch concrete block. The east end of the structure had two 32-foot wide reinforced concrete walls connected by spandrel beams below a window at each floor. One of these walls was continuous for the full height of the building and the

other was continuous from the fifth floor to the roof. The structural steel framing in the transverse direction of the building consisted of six frames with the exterior frames encased in the east and west concrete walls. The longitudinal structural steel frames in the tower portion of the building consisted of two exterior frames which were designed as rigid frames and one interior and one transverse frame which were not designed as a rigid frame but had partial lateral force resisting capabilities.

A mathematical analysis of the building was performed. Examination of resulting axial loads, bending moments, and shear forces, showed that the critical lateral force resisting segment of the building was located in the west wall between the 2nd and 5th floors. This was where the 8-in. reinforced concrete wall was reduced to a 22-ft length for a height of 24 ft with an intermediate parking level. Analysis showed that if failure occurred, it would begin with excessive bending stresses which would cause reinforcing steel to yield. As the lateral forces increased, this portion of the wall would fail in shear manifested as diagonal tension cracks.

A structural analysis of the mathematical model identified motion levels at which key structural thresholds were reached. From the point of view of structural integrity, the key thresholds were the motion levels at which the elastic limit and the ultimate limit would be reached. Applying the predicted ground motion and its statistics, it was determined that there was a 10% chance of reaching the elastic limit threshold and a 0.1% chance of reaching the ultimate limit threshold; respectively, about 2.2 and 3.1 geometric standard deviations above the median value of the ground motion prediction for the HANDELEY event. Although the probability of reaching these thresholds was quite small, a decision was reached by DOE/NV to approach the owner with recommendations for structural modifications.

The modification was performed in March 1970, consisting of installing a reinforced concrete wall in the total cut-out area, but leaving the existing concrete block in place. The modification essentially transformed the west wall into a uniform wall, thus eliminating the critical section. During the modification it was observed that essential reinforcing provided in the original design was missing from the wall as it had originally been constructed. Quality of concrete construction was also poor.

The following observations were made:

- o Base shear coefficients for various damage thresholds (a measure of the ground motion required to produce damage) were increased 3 to 4 times by the west wall modification; i.e., the ground motion required to produce damage would have to be 3 or 4 times larger, after modification, than that which would have produced the same damage

before modification.

- o Stiffening of the building as a result of the modification was calculated to reduce the fundamental period for HANDLEY-level motion about 10 to 15%. Actual measured period reduction for HANDLEY-level motion was about 10%.
- o The stiffened west wall deflected only about 50% as much as the east wall for HANDLEY. Prior to the modification the west wall displacement was about 80% that of the east wall.
- o No damage to the remodeled building was expected from event HANDLEY, and none occurred.

Structures at the NRDS Although structures within NTS boundaries were outside Blume's scope of work, exceptions were made when particular problems arose. Most of the Blume involvement occurred with structures at the NRDS in the southwestern portion of the NTS (Figure 5.5), and specifically within the 1966-1970 time period. Several large and complex structures were located at the NRDS and initial concerns were related to the question of possible damage from increasingly large yields from test programs underway at the time. Later in the period particular concern was engendered by the discovery of anomalous ground motion, characterized by high accelerations, from some test shots on Pahute Mesa.

Initial concern regarding structural response at NRDS arose in early 1966. In June of that year, the Ground Shock Subcommittee of the NTS planning board requested that Blume conduct an investigation of water tanks at the NRDS. This request arose from concern regarding the effects of the HALFBEAK event ground motion on these tanks.

Blume's evaluation indicated no damage was likely to occur. As expected the 365 kt HALFBEAK event of June 30, 1966 did not cause any damage to the tanks (A14). However, concern continued, especially for Test Cell C structures and the forthcoming GREELEY event which had a predicted yield level of more than twice that of HALFBEAK. Concern over possible damage from GREELEY also extended to other NRDS structures and facilities, and a broadened investigation encompassing most of the NRDS installation was proposed and implemented during the fall and early winter of 1966. Recommendations were provided for personnel safety measures and some water level reductions to reduce damage hazard to ground-based water tanks.

The GREELEY event at 870 kt, was fired on December 20, 1966. Damage was negligible consisting

FIGURE 5.5 NEVADA TEST SITE AND SURROUNDING AREA

only of extension of pre-existing cracks in concrete structures at ETS-1, Test Cells A and C, and the R-MAD and E-MAD buildings.

Prior to the GREELEY event Blume strongly urged the standardization of seismic design provisions at NRDS through the mechanism of a seismic design manual for NRDS. Blume subsequently prepared such a manual, and submitted it late in 1966.

Analyses of the GREELEY records were conducted in early 1967 and results were used to help prepare a report on damage threshold ground motions for NRDS facilities. That report was completed and submitted in March, 1967. Surveys of NRDS subsequent to the 250 kt COMMODORE event of May 20, 1967 and the 155 kt SCOTCH event of May 23, 1967 showed only negligible damage consisting of minor crack extension in the ETS-1 test stand and the E-MAD building.

The need for thorough seismic review of NRDS and possible upgrading of seismic capabilities of the NRDS structures was recognized prior to the GREELEY event, based on plans for escalating yield levels in future NTS programs. In view of the low damage threshold ground motion levels predicted for some NRDS structures, a thorough study of all NRDS facilities by Blume was authorized in August of 1967, to include design of seismic bracing as necessary. Design and construction took place in late 1967 and early 1968.

Additional Blume instrumentation was requested for ETS-1 at NRDS for the planned high-yield BOX-CAR event. That event, at a yield level of 1.3 MT was executed on April 26, 1968. No damage was reported, with the exception of possible cracking in the granite slope covering of an embankment at ETS-1. An unpublished report on the Blume studies at NRDS was submitted in October 1968.

Following the BOXCAR event, a smaller event entitled CHATEAUGAY, was detonated on June 28, 1968 at a yield of 20 to 200 kt. Anomalous ground motion levels were reported from a station near the ETS-1 facility at NRDS. The anomaly was characterized by unusually high ground motion acceleration levels, which caused additional concern regarding structural response at NRDS from planned large Pahute Mesa Tests. A study was conducted by ERC (D65) using Special Projects Party data from a number of stations in the NRDS area. This study indicated that characteristics of the anomaly included a spectrum unusually enriched in high frequency pulse-like first arrivals with a peak vector amplitude considerably greater than typically observed at the distance for stations with similar geologic settings. The anomalous response was greatest at ETS-1 and decreased in all directions away from it. Analysis

of observed data suggested that the anomaly was related to physical parameters of the travel path rather than solely to parameters of the source or receiving station environments.

Additional studies at ETS-1 and Test Cell C were authorized in early November 1968 and completed the same month. Modifications were completed in December, prior to the December 19 execution of the 1.15 MT BENHAM event (E10). Only minor new cracking or slight aggravation of older cracking was observed from the BENHAM event. In early January 1969 Blume was advised by DOE/NV that additional ETS-1 bracing studies were required for two series of possible high yield levels at Pahute Mesa event locations. These were to be permanent bracing installations, as compared to temporary bracing installed during early 1969, with various modifications resulting from by modified ERC predictions of expected NRDS ground motion.

In late 1969 field work on installation of permanent bracing for ETS-1 began, with Blume providing resident engineering support for this effort. Particular attention was required in some areas for the forthcoming HANDLEY event of early 1970. However, motion from the HANDLEY event was less than the criteria for which bracing had been installed. No evidence of any damage attributable to the that event was reported.

A seismic design aspect study for NRDS essentially completed by mid-1970 and only occasional minor assignments, e.g., reviews reports by others and brief considerations of minor structural problems took place thereafter. All this work was completed by early 1971.

5.8.3 Convention Center Canopy Failure

One unusual damage occurrence involved the collapse of the hung ceiling of the covered exterior walkway and marquee at the Las Vegas Convention Center. This failure occurred late in the afternoon of May 15, 1970. Event CORNICE, with a yield of between 20 and 200 kt (E10), had been detonated earlier in the day. Building representatives alleged that the failure was in some measure caused by ground motion from testing at the NTS.

A detailed study conducted by Blume indicated that the failure had resulted from progressive pull-out of hanger wires supporting the hung ceiling. These wires were led upward from the ceiling and through holes in the overhead metal deck. They were then bent at right angles and covered with a layer of asphalt tar, insulation, and 5-ply roofing. Additional loading at the area of initial failure was caused by 4 30-pound electrical transformers supported on the hung ceiling.

Wind effects were also considered to be potentially significant because of aerodynamic pressure variations on the exposed surface of the walkway ceiling, calculated to be about the same order of magnitude as the static load on the hanger wires. High ambient temperatures were also believed to be a contributing effect through softening of the tar embedment anchorage for the bent ends of the hanger wires.

Calculations of maximum ground motion loading on the hanger wires for Event CORNICE and also for Event HANDLEY, detonated in March 1970, and at over 1 megaton one of the largest NTS events, indicated that these loadings were far below those from static loads. Aerodynamic effects were also calculated to be much greater than those from ground motion. Blume concluded that on the basis of these calculations, the failure was not caused by ground motion effects. SNL's review of the Blume report and its own independent calculations concurred that the ground motion effects were insignificant in comparison with other factors (support mechanism, wind, temperature effects) and consequently ground motion could not be appealed to as the damage mechanism for the collapse of the hung ceiling.

As shown by the foregoing discussions in Section 5, response evaluations conducted by Blume were strongly influenced by the nature and pace of underground nuclear testing and, equally, by the progress and individual characteristics of structural construction in Las Vegas. These combined at times in ways that required utmost effort by the seismic safety program consultants and reviewing agencies to ensure that testing could continue with adequate assurance of public safety.

6. OFFSITE RESPONSE EVALUATIONS

Underground nuclear explosion programs at locations away from the Nevada Test Site were conducted for various purposes, including weapons tests, improved detection of foreign nuclear explosions abroad, and nonmilitary applications of nuclear explosive energy.

Offsite programs necessarily required substantial response evaluation efforts. Because such tests took place in new locations each automatically required the collection and utilization of a complete set of basic structure inventory data for that area.

6.1 MAJOR OFFSITE TESTS

Blume was involved in several major offsite projects during the period of this report. The earliest of these were the 1964 SALMON (A10) and 1966 STERLING (A18, C8) events in central Mississippi, sponsored by the Vela Uniform program which was concerned with the detection and identification of distant underground nuclear explosions. From 1965 through 1968 a great deal of field and office work was expended on the investigation of potential structural response to ground motion and air blast from the planned (but never executed) nuclear explosion excavation of a sea level canal across the Isthmus of Panama (A8, A9). The 1968 FAULTLESS (E10) event in central Nevada was a calibration shot for a proposed new weapons test area. The 1967 GASBUGGY (A21) event in northern New Mexico and two following central Colorado projects, RULISON (A30, A33, A34, C2, C3, D25-D28, D34, E5, E7) in 1969 and RIO BLANCO (A75, C22, D45, D55) in 1973, were part of the Plowshare program for nonmilitary applications of nuclear explosions, in these cases for enhanced production of natural gas. The planned WAGON WHEEL and WASP projects in western Wyoming, for which Blume conducted extensive field investigation in 1969, would also have been Plowshare gas-stimulation projects.

These events are grouped, for discussion, into 4 parts; the Central Nevada Testing Area evaluation (FAULTLESS); the Vela Uniform Program (SALMON and STERLING); Plowshare (GASBUGGY, RULISON, RIO BLANCO, and several unexecuted events); and the Atlantic-Pacific Interoceanic Canal Study Commission investigation.

6.1.1 Central Nevada

The growing incidence of damage complaints and verified minor damage from the larger NTS events in

the later 1960's led to serious consideration of alternative testing locations both within and outside the continental U.S. One such location inside the U.S. was in Hot Creek Valley of south-central Nevada, a sparsely populated area northeast of Tonopah, and about 100 kilometers distant from each of the 3 nearest towns; Austin, Eureka, and Tonopah (See Figure 3.8).

A seismic calibration event, FAULTLESS, was planned for the new prospective testing area. Because of planned follow-on testing and the uncertainty of ground motion characteristics throughout this new area, particular efforts were made to locate and evaluate all structures within the range of possible damage. In addition to the structural inventory, extending out to 100-kilometers distance from the shot point, the geologic and foundation conditions were determined for the communities and for particular structures, e.g. dams, within the surrounding area. The seismic history of these areas was also reviewed to determine what, if any, previous exposure to strong ground motion might have existed.

Because of the possible extended use of the prospective new testing area for additional future tests at potentially high yields, the structural response evaluation included consideration of distant high-rise structures, in Las Vegas, Reno, Salt Lake City and San Francisco. Instrumentation was deployed widely, to acquire the necessary calibration data from FAULTLESS to permit reliable extrapolation of ground and structure response for any future tests in this area.

The level of activity for FAULTLESS was considerably greater than would normally be appropriate for two reasons; location of the test in an area not previously exposed to ground motion from underground nuclear event, and for calibration of motion of the event as a precursor to other and potentially larger events in the new area. The alternative testing area already under consideration was on Amchitka in the Aleutian islands. That location would entail much greater logistic expenditures, and therefore it was essential to conduct a careful evaluation of the Central Nevada Testing Area.

Approximately sixty seismic instruments were dispersed within a 100-km radius of the detonation to record ground motion. Since all structures in this range were low 1- and 2-story buildings, none were instrumented for structural response.

Comparisons of the ratios between predicted ground motion and recorded ground motion are set forth in Table 6.1. Within the range of 52 kilometers, in which all predicted damage was confined, the comparison of particular interest is that of the acceleration values. The actual ground accelerations were from 15 to 55% of the predicted values, with most of the peaks occurring in longer periods instead of the

TABLE 6.1

FAULTLESS EVENT-
SUMMARY OF PEAK ACCELERATION RATIOS OF CLOSE-IN STATIONS

Station (Distance)	Ratio of Max. Recorded Accel. to Predicted Accel.	Ratio of Peak Accel. at Short Periods to Max. Recorded Accel. (Component)	Ratio of Peak Short Period Accel. to Predicted Peak Accel.
Moore's Station (6.9 km)	0.33	0.45 (R) 0.34 (T)	0.15 0.11
Pretchard's Station (16.3 km)	0.50	0.61 (R) 0.57 (T)	0.30 0.28
Hot Creek Ranch (17.6 km)	0.44	0.17 (R) 0.41 (T)	0.07 0.18
Fish Lake Ranch (22.5 km)	0.37	0.25 (R) 0.29 (T)	0.09 0.11
Blue Jay Station (29.1 km)	0.52	0.75 (R) 0.27 (T)	0.39 0.14
Clear Creek Ranch (30.8 km)	0.25	0.29 (R) 0.49 (T)	0.07 0.12
Tybo (33.7 km)	0.15	0.68 (R) 0.40 (T)	0.10 0.06
Lockes (39.6 km)	0.50	0.91 (R) 0.77 (T)	0.45 0.38
Barley Creek Ranch (42.4 km)	0.55	0.40 (R) 0.57 (T)	0.22 0.31
Snow Ball Ranch (45.3 km)	0.50	0.32 (R) 0.44 (T)	0.16 0.22
Twin Springs (47.9 km)	1.8 1.0	0.10 (R) 0.10 (T)	0.18 0.18
Stone House Ranch (51.2 km)	0.67	0.21 (R) 0.45 (T)	0.14 0.30
Duckwater (53.2 km)	0.50	0.54 (R) 0.48 (T)	0.27 0.24

short-period range (0.1 to 0.2 second) which would be the important dynamic response range for low buildings in the region of predicted damage.

Within 50 to 200 km, the structures ranged from one-story buildings and residential structures to 5- and 6-story hotels. The two largest communities were Tonopah (110 km) and Ely (135 km), with approximate 1968 populations of 1680 and 4100, respectively.

Structures in Tonopah included two 5-story hotels (Belvada and Mizpah) and various other 1-, 2-, and 3-story brick and stone masonry structures. The Belvada Hotel and the Ramona Hotel (3 stories) were instrumented for structural response, and peak accelerations are tabulated in Table 6.2. An unexplained disturbance appeared on the Tonopah records about 17 seconds after the start of the motion, and continued for about 10 seconds. The response from the initial portion of the record is tabulated in the column headed FAULTLESS, and the response from the disturbance is tabulated in the column, labeled "QUESTIONABLE" FAULTLESS. The disturbance appeared to have had a greater effect on the Belvada Hotel than on the Ramona Hotel.

Beyond 200 miles from shot point, the closest metropolis, Las Vegas, is located approximately 290 km due south of ground zero. Other major cities are Reno (350 km), Salt Lake City (440 km), Sacramento (460 km), Los Angeles (550 km). Recordings of structural response to FAULTLESS ground motion were obtained from 16 structures; 13 in Las Vegas, 1 in Reno, 1 in San Francisco, and 1 in Beatty. Table 6.3 sets forth the maximum absolute accelerations and displacements of these structures.

In Las Vegas, the largest absolute acceleration from FAULTLESS was recorded by Landmark Tower (N/S component of building axis), followed by Mint (transverse direction of building axis), Dunes (longitudinal), and First National Bank (longitudinal). In general, the structures with the longest natural periods (0.8 to 2 seconds) registered higher accelerations than those with periods between 0.3 and 0.8 seconds.

The response spectrum obtained from Salt Lake City recordings showed comparatively high amplitudes. While response spectrum amplitudes normally decrease with distance from shot point, the response spectrum of the Salt Lake City ground motion was actually almost identical with that of Reno, while the distances were 440 km to Salt Lake City and 330 km to Reno. The distance to Sacramento was about the same as to Salt Lake City, but the Salt Lake City response spectrum was approximately three times that of Sacramento.

TABLE 6.2

FAULTLESS EVENT -
COMPARISON OF PEAK STRUCTURAL ACCELERATIONS

Structure (Component)	FAULTLESS		"QUESTIONABLE" FAULTLESS	
	Absolute Acceleration (g)	Predominant Acceleration (g)	Absolute Acceleration (g)	Predominant Acceleration (g)
Belvada (L) (T)	0.013 0.013	0.19/0.34 0.17/0.65	0.016 0.016	0.34 0.60
Ramona (L) (T)	0.024 0.017	0.15/0.88 0.18/0.31	0.017 0.013	0.14/0.31 0.15/0.87

TABLE 6.3

FAULTLESS EVENT - PEAK MEASURED STRUCTURAL ACCELERATION AND DISPLACEMENT

Structure	Component	Peak Acceleration (g)	Predominant Period (sec)	Peak Displacement (cm)	Predominant Period (sec)
Bank of Nevada	L	0.0059	1.21	0.23	1.21
	T	0.0049	0.6	0.66	0.6
Caesar's Place	L	0.005	0.79	0.15	0.84
	T	0.0056	0.9	0.18	0.92
Clark County Courthouse	L	0.0028	0.5	0.083	1.7
	T	0.004	0.44	0.022	0.45
Desert Inn	L	0.0046	0.66	0.079	0.67
	T	0.0047	0.54	0.12	1.71
Dunes	L	0.0091	1.88	0.74	1.87
	T	0.0067	1.78	0.33	1.81
First National Bank	L	0.0094	1.28	0.6	1.29
	T	0.0062	0.59	0.17	0.54
Fremont	L	0.0074	0.82	--	--
	T	0.0055	0.93	0.17	1.03
Landmark Tower	N/S	0.0192	1.89	0.85	1.88
	E/W	0.0085	1.79	0.54	1.79
Mint	L	0.0092	0.89	0.24	0.91
	T	0.0186	1.15	0.71	1.17
Sahara 200	L	0.0029	0.76	0.16	0.79/2.01
	T	0.0041	0.81	0.26	0.87
Sahara 400	L	0.0068	1.18	0.31	1.18
	T	0.0074	1.12	0.28	1.13
Sands	N/S	0.0071	1.17	0.26	1.18
	E/W	0.0056	1.21	0.31	1.25
Stardust	L	0.0034	0.54	0.116	4.11
	T	0.002	0.39	0.116	1.94
Arlinton Tower (Reno)	L	0.0061	1.47	0.35	1.51
	T	0.0072	1.72	0.47	1.77
Exchange Club (Beatty)	L	0.0011	0.39	--	--
	T	0.0107	0.41	0.043	0.43
Wells Fargo Bank Building (San Francisco)	N/S	--	--	0.014	1.3
	E/W	--	--	0.043	1.3/3.9

In San Francisco the 43-story Wells Fargo Bank building was instrumented for FAULTLESS, using 2 displacement-recording Sprengnether instruments. The larger readings, shown in Table 6.3, were obtained by extrapolating an off-trace record from the instrument located near the end of the building and are largely torsional displacements. The other instrument, on the same upper level but located closer to the center of rigidity of the building, recorded a peak displacement about one-third that shown in Table 6.3, and is primarily a translational displacement.

All the damage predicted to occur from FAULTLESS was expected to take place inside the area within 52 kilometers from the shot point. Within this area, the structures were all low buildings, frequently of stone construction, whose response is best predicted by using ground motion acceleration. As shown in Table 6.1, the actual maximum recorded ground acceleration were from 15 to 55% of the predicted values. Most of these peaks occurred in the longer period range instead of the short period range where they should occur for such distances. Accelerations in this short period range, which is the important dynamic response range for low buildings in the region of predicted damage, varied from about 10 to 45% of the predicted peak accelerations. Since damage potential decreases rapidly with decreasing acceleration, actual damage was well below that predicted using pre-shot ground motion forecasts.

Table 6.4 summarizes predicted and actual effects from the FAULTLESS event, including perceptibility ratings from an index based generally on the Modified Mercalli Index. These ratings provided a means for relating perception from UNEs to that from earthquakes.

Following the FAULTLESS event, continuing interest in additional future tests led to the conduct of a detailed structural inventory for the central Nevada area. Possible increases in yield level of several times that of FAULTLESS event extended the area of concern to about 175 kilometers from the center of the area. Also, high-rise structure evaluations were conducted in Salt Lake City, Reno, and Las Vegas, as well as special-structure evaluations in Tonopah, Ely, and Eureka. However, the widely-felt structural response from the FAULTLESS event, particularly that in the high-rise buildings in Salt Lake City, together with anticipation of adverse public reaction to future large events in this area, led to a decision to abandon further testing at the Central Nevada Site. Two large follow-on events, MILROW at about 1 megaton on October 2, 1969 (D29, E10); and the CANNIKIN event at less than 5 megatons on November 6, 1971 (D47, D49, E10), were conducted at a testing site established on Amchitka in the Aleutian Islands. An earlier event, Project LONG SHOT (E10), at about 80 kilotons yield, had been conducted in October 1965, for the Vela Uniform Program.

TABLE 6.4

FAULTLESS EVENT - COMPARISON OF PREDICTED AND ACTUAL STRUCTURAL EFFECTS

Location	Distance from Shot Point (km)	Predicted Effect	Observed Effect
Moore's Station	7	Severe Damage	Minor Damage
Morey Mine	8	Severe Damage	No Damage
Pritchard's Station	15	Moderate Damage	No Damage
Hick's Station	16	Abandoned Ruins	No Damage
Hot Creek Station	18	Moderate Damage	No Damage, Perceptible (V)
Fish Lake Ranch	23	Minor Damage	No Damage
Blue Jay Maintenance Station	31	Minor Damage	No Damage, Perceptible (V)
Clear Creek Ranch	31	Minor Damage	No Damage
Tyto	35	Minor to Moderate Damage	No Damage
Lockes Ranch	38	Minor Damage	No Damage, Perceptible (II)
Barley Creek Ranch	42	Minor Damage	No Damage
Snowball Ranch	44	Minor Damage	No Damage
Willow Creek (South) Ranch	47	Minor Damage	No Damage
Twin Springs Ranch	50	Minor Damage	No Damage
Indian Springs Ranch	52	Very Minor Damage	No Damage
Stone House Ranch	52	Very Minor to Moderate Damage	No Damage, Perceptible
Bar-n-bar Ranch	53	Very Minor Damage	No Damage
Duckwater, Nevada	53	Minor Damage	No Damage Perceptible (V)
Warm Springs, Nevada	53	Very Minor Damage	No Damage
Pine Creek Ranch	55	No Damage	No Damage
Belmont, Nevada	58	No Damage	No Damage, Perceptible (III)
Martin Ranch	58	Very Minor Damage	No Damage

TABLE 6.4 - continued

Location	Distance from Shot Point (km)	Predicted Effect	Observed Effect
Railroad Valley	58	Very Minor Damage	No Damage, Reported Oil Spillage
Blue Eagle Ranch	60	No Damage	No Damage
Nyala Ranch	60	Very Minor Damage	No Damage
Pott's Ranch	62	No Damage	No Damage
Currant Motel-Cafe	63	Very Minor Damage	No Damage
Blue Diamond Mine	65	No Damage	No Damage
Bull Creek Ranch	68	No Damage	No Damage
Clark's Station, Nevada	68	No Damage	No Damage
Fish Creek Ranch	72	No Damage	No Damage
Wilson's Ranch	72	No Damage	No Damage
Currant Creek Maintenance Station	75	Negligible Damage	No Damage
Round Mountain, Nevada	75	Negligible Damage	No Damage, Perceptible (V)
Manhattan, Nevada	76	No Damage	No Damage
Ardan Ranch	78	No Damage	No Damage
Reynolds Creek Ranch	83	No Damage	No Damage
Highway Maintenance Station, Highway 8A	85	No Damage	No Damage, Perceptible
Kelly Creek Ranch	87	No Damage	No Damage
R.O. Incorporated Ranch	88	No Damage	No Damage
Willow Creek (North) Ranch	93	No Damage	No Damage
Gardner Ranch	95	No Damage	No Damage
Heffern Ranch	95	No Damage	No Damage
Eureka, Nevada	100	Very Minor Damage	No Damage, Perceptible (IV)
Birch Creek Ranch	105	No Damage	No Damage
Tonopah, Nevada	110	No Damage	No Damage, Perceptible (V)

TABLE 6.4 - continued

Location	Distance from Shot Point (km)	Predicted Effect	Observed Effect
Austin, Nevada	120	No Damage	No Damage, Perceptible (IV)
Ruth, Nevada	150	--	No Damage, Perceptible (IV)
Ely, Nevada	150	--	No Damage, Perceptible (IV)
Wells-Fargo (43rd Floor), San Francisco	550	--	No Damage, Perceptible (II)

PERCEPTION INDEX (USC&GS)

- I. No felt.
- II. Questionable.
- III. Felt.
Direction of motion uncertain.
Non-observers do not notice or question.
- IV. Felt.
Noticed by some non-observers.
- V. Felt distinctly.
Observer can define directions of motion with clarity.
Most non-observers at rest will feel or react to the motion.
- VI. Felt by all.
Sense of balance affected.
Non observers may exaggerate reports of motion experienced.

6.1.2 Vela Uniform Program

The Vela Uniform Program, designed to provide data on discrimination between earthquakes and ground motion from underground nuclear explosions, was sponsored by the Advanced Research Projects Agency of the Department of Defense. Event SHOAL at 12 kt was fired on October 26, 1963, near Fallon, Nevada and LONG SHOT at about 80 kt on October 29, 1965, on Amchitka were two of the earliest Vela Uniform tests (E10). Lawrence Livermore National Laboratory was responsible for technical direction of Project DRIBBLE, a third Vela Uniform Program.

SALMON Event On October 22, 1964, a nuclear device of 5.3 kilotons was detonated in the Tatum Salt Dome, about 825 meters below the ground surface in Lamar County near Baxterville, Mississippi. This was the SALMON event of Project DRIBBLE, Peak ground accelerations for selected SALMON instrument locations are shown in Table 6.5.

Ground motion resulting from the detonation was felt over a considerable area. Motion was felt by most of the people in Hattiesburg, the nearest metropolitan area. About 220 separate damage claims were submitted, mostly for minor damage such as cracking of plaster or masonry walls. By November 1967, about \$182,000 had been paid in restitution for claimed damage. Details of the SALMON damage are discussed in Section 6.4.1. As noted in that discussion, there may have been some contributing effects from Hurricane Hilda which had recently passed through the vicinity.

STERLING Event Event STERLING on December 3, 1966 involved the detonation of a 380 ton nuclear device in the center of the SALMON cavity in the Tatum Salt Dome. The purpose of the STERLING test was to measure the extent to which the adjacent airspace would decouple the explosion.

Nineteen structures in cities and towns surrounding the test site were selected for observation. Residences were chosen for close-in structure monitoring because of the lack of other building types; commercial or institutional structures were selected for the outlying cities and towns.

Detailed condition surveys were made on all selected structures, and all information gathered -- including photographs -- was compiled into a field report that was utilized by observers during the pre-shot, shot-time, and post-shot surveys.

Perceptibility reports submitted by Blume observers indicate that motion was felt slightly at only four of the nineteen monitored structures. This observed motion was recorded in the vicinity of residential

TABLE 6.5
PEAK ACCELERATION DATA, PROJECT SALMON

Station	Distance (Slant range, km)	Peak Acceleration (g)	Component
1 East	1.46	1.50	Vertical
5 East	4.08	0.39	Vertical
Baxterville	9.68	0.08	Transverse
Purvis	14.5	0.04	Vertical
Lumbertown	17.7	0.03	Radial
Hattiesburg, Ground Floor 8th Story	33.7	0.01* 0.03*	Vertical Transverse

- Derived from a displacement record, using a Sprengnether Blastmeter. All other data are recorded directly on USC&GS strong motion accelerometers.

structures located with a 4.5-mile radius of shotpoint. Reports from other observers indicated no perceptible motion.

There were no indications of STERLING-generated damage.

MIRACLE PLAY Series A series of follow-on non-nuclear detonations entitled MIRACLE PLAY was conducted, using the SALMON cavity. Blume had little involvement in MIRACLE PLAY. The test series encountered a number of operational difficulties. No useful data were obtained for structural response purposes and no reports were prepared or issued.

6.1.3 PLOWSHARE

During the middle and late 1960's, several projects involving the use of nuclear explosives for non-military purposes were under active consideration. These tended to be of two categories; one involving deeply buried explosions for the enhancement of mineral production through fracturing of the host rock, and the other involving explosions for earth-moving purposes.

The first category, underground explosions for enhanced mineral production actually proceeded to completion for 3 separate projects; GASBUGGY in northern New Mexico, and RULISON and RIO BLANCO in central Colorado. Blume had a substantial role in 2 of the 3 projects, as discussed below. Two other projects involving similar underground detonations for enhanced mineral production were under consideration in 1969 for locations in northwestern Wyoming. A considerable effort was expended by Blume in mid-1969 in conducting structural inventories and evaluating potential hazards for these two projects, entitled WAGON WHEEL and WASP. Blume was also involved during the 1972-1974 period in conducting evaluations of concerns expressed by others with regard to potential effects on bridges near the project locations. This is discussed further in a later part of this chapter.

The earth-moving explosions were usually intended for improvement of transportation by excavating passageways for either land-borne or water-borne traffic. Blume involvement generally consisted of participation in early planning phases, followed by a brief field reconnaissance of the prospective test area. This reconnaissance provided a preliminary evaluation of the potential structural response problems posed by the nature and extent of development in the area to be exposed to strong ground motion from the detonation. This evaluation was normally conducted by a two-man team consisting of an architect or engineer and a engineering geologist. Several such preliminary investigations were conducted for Project SLOOP in Arizona in 1965; in 1967, for Projects BRONCO and DRAGONTAIL

in Colorado, for Project EXCAVATOR in several potential locations in the western U.S., and for Project KETCH in Pennsylvania. Brief office studies took place in 1965 for Project SCHOONER in Idaho and in 1966 for Projects POON and SCROLL.

An extensive effort was conducted in the 1965-1970 period for the proposed nuclear excavation of a sea level canal across the Isthmus of Panama. That investigation is reported in more detail in a following section of this report.

Project GASBUGGY GASBUGGY was a joint effort of the U.S. Atomic Energy Commission, the Bureau of Mines, and El Paso Natural Gas Company. Technical assistance was provided by Lawrence Radiation Laboratory.

The explosive was emplaced at 4240 feet (1290 meters) below ground surface, in Rio Arriba County, New Mexico, about 55 miles (88.5 km) east of Farmington New Mexico (Figure 6.1). It was detonated on Sunday, December 10, 1965, with a yield later determined to be 29 kilotons.

The GASBUGGY project area was sparsely populated with only a few ranches and gas-production facilities in immediate proximity to the shot point. However the lack of previous experience with underground nuclear testing in the area made it necessary to conduct detailed surveys of the region for identification of any structure or facility that might pose particular hazards resulting from the predicted ground motion.

Two locations were of particular concern. The first of these was the La Jara Lake ranch house, 17 kilometers from shot point. This 2-story 100 year old structure was in poor condition with extensive cracking in its thick adobe walls. The Blanco gas production facility at Bloomfield, 69 kilometers from shot point, was also of concern because of the tall processing structures at this facility and the probability of their amplified response to longer period ground motion, with potential leaking gas hazards.

Ground motion generated by the GASBUGGY event was recorded out to a distance of 90 kilometers. Thirty-six recording stations were monitored by the Special Projects Party. The ground motion records were processed and analyzed by the Special Projects Party and the Environmental Research Corporation (ERC).

To study the structural response from this event, three buildings were instrumented -- the Jicarilla

FIGURE 6.1 GASBUGGY AREA MAP

Indian Agency building and the Dulce School in Dulce, New Mexico, and the Avery Hotel in Farmington, New Mexico. Recording instruments for structural response consisted of sets of three velocity meters oriented mutually perpendicular to one another and located on the ground nearby. The instruments on the structures and on the ground were oriented with the instrument axes parallel to the building axes.

Using the ground motion predictions supplied by ERC and the results of the Blume preliminary reconnaissance and detailed condition surveys, various structures and locations were analyzed to predict the property damage that could be caused by the event. Utilization of maximum yield ground motion predictions resulted in a recommendation that the La Jara Ranch House and the Blanco Plant at Bloomfield should be temporarily evacuated at shot time.

For GASBUGGY, the damage prediction estimate, based on engineering judgement, was \$21,500. Blume's Spectral Matrix Method (SMM) of damage prediction indicated that the total damage would amount to about \$35,000. using predicted mean values of maximum yield ground motion. Generally, only slight damage such as plaster crack extension with some development of hairline plaster cracks was expected to occur.

Three damage complaints were investigated, at distances of 23, 40, and 45 kilometers from the detonation point. Of these, only the one at 40 kilometers assessed as being credible, and that only to the extent of slight additional aggravation of existing damage from a major settlement problem.

ERC analysis of GASBUGGY ground motion data indicated that the larger depth of burial for GASBUGGY, as compared to a typical Nevada Test Site event of the same yield, caused a significant increase in peak acceleration levels. This resulted from a shift in spectral content of the ground motion to the higher frequency end of the spectrum.

From these studies, ERC developed a general model for scaling ground motion spectra from underground nuclear detonations. The scaling relationships derived from the source function indicated that the yield, depth of burial, and emplacement medium of the nuclear device played a significant role in the scaling of seismic signals. Comparisons made with the results obtained from a large body of measured spectral data showed good agreement with behavior predicted on the basis of the new general model (D96, D97).

Project RULISON RULISON was a joint experiment sponsored by Austral Oil Company Incorporated, Houston, Texas; the U.S. Atomic Energy Commission; and the U.S. Department of the Interior. Program management was provided by CER Geonuclear Corporation of Las Vegas, Nevada, under contract to Austral. Los Alamos Scientific Laboratory provided the explosive device and the firing process.

RULISON, like its predecessor GASBUGGY, was a gas production stimulation experiment in an area not previously exposed to ground motion from underground nuclear testing. RULISON, however, took place in Garfield County, Colorado, a considerably more populated area with several small towns inside a 30 kilometer distance from the shot point (Figure 6.2).

Structural safety evaluations were based on a design yield of 40 kilotons for RULISON in contrast to a predicted 31 kiloton yield for GASBUGGY. Other concerns were raised at RULISON by the presence of a number of dams, including several old earth dams, in the area; the use of unreinforced brick masonry for many older structures with that material's record of poor behavior during earthquakes; a history of slope failures during wet seasons such as during the late spring May date originally proposed for RULISON; and the high incidence of deteriorated brick chimneys in the area with an even worse earthquake history than that for unreinforced brick masonry construction. For these reasons, RULISON studies were extensive, and followed quite closely the entire gamut of off-site pre-shot studies set forth in Section 6.2.

Initial evaluations of these problem areas were sufficiently disturbing to raise serious safety questions, especially those related to Harvey Gap Dam (A30). This was one of the old earth dams within range of potentially damaging ground motion, and also was upstream from inhabited areas near Silt, Colorado. A special study was conducted and is reported in Section 6.4.2, following.

Major industrial facilities outside of the 25-kilometer radius but within a 35-kilometer radius included the Oil Shale Research Center at Anvil Points, and the Union Carbide Plant at Rifle. The major dams were Rifle Gap Dam, Harvey Gap Dam, Bonham Dam, and Vega Dam. Smaller dams were located on Battlement Mesa, approximately 3 kilometers south of shot point. Of the four major dams, Harvey Gap, above the town of Silt, presented an apparent hazard in view of its age and obvious disrepair of outlot works, a potential liquefaction hazard because of underlying sand lenses, and a downstream flood hazard because of high water levels in the reservoir in early spring. However, because of the potential Harvey Gap Dam hazard, the detonation date was delayed from May 1969 to September 1969. The

FIGURE 6.2 RULISON AREA MAP

reservoir was nearly empty at shot time and no hazard existed. Further discussion of the Harvey Gap dam studies is presented in Section 6.4.2.

Predicted Effects Direct ground motion hazards were predicted to exist within a radius of 7.4 kilometers of shot point because peak horizontal ground accelerations were predicted to equal or exceed 0.3g within this area. It was estimated that ground motion at this level might cause persons to lose balance and fall. Consequently this area was temporarily evacuated at shot time.

Spectral acceleration for structures of 0.2-second period, at 5% damping, was estimated to be 0.8g. At these levels of motion, some slight shifting on foundations could occur. Damage to chimneys might also occur as a result of differential motion or banging of the chimney against the house. Other damage could occur to bric-a-brac, tall standing objects (e.g., a grandfather clock), and to hanging mirrors and pictures. In some locations it was recommended that chimneys be removed, bracing installed, and certain utility services be shut off to prevent damage.

Damage of the extent which could pose a potential hazard to persons was considered a possibility to structures in the range from 7.4 to 14 kilometers. This region includes the town of Grand Valley, the Anvil Points Research Station, and a number of small ranches. Within this area, it was recommended that people be outside and two building heights away from structures.

The town of Grand Valley was located about 10 kilometers northwest of shot point. Structures in the commercial area were generally old brick 1- and 2-story buildings. The largest structure was the school, which was constructed of brick. There were a few brick veneer houses and a number of all-metal mobile houses; the rest were of all-wood construction. The peak 5% damped spectral acceleration prediction for Grand Valley was 0.4g for structures with a period of 0.2 seconds; peak horizontal ground motion was predicted to be about 0.13g. Under these conditions only moderate damage was anticipated, mostly in the older 2-story brick commercial structures.

Actual Effects Table 6.6 presents a comparison of predicted and actual effects. Predicted effects correspond to the predicted motion for the 40 kiloton design yield.

The peak horizontal ground motion within 7.4 km of shot point was predicted to be in excess of 0.30g. Depending on the location, the actual values of peak horizontal ground motion did exceed 0.30g.

TABLE 6.6

PREDICTED AND ACTUAL DAMAGE FROM PROJECT RULISON

Name	Approximate Distance & Direction from Shot Point (km)	Predicted Effect	Actual Effect
Rulison	9 N	Moderate damage	Moderate to minor Damage
Grand Valley	10 NW	Moderate damage	Moderate to minor Damage
Anvil Points	13 N	Moderate to minor damage	Possible minor road damage
Microwave Station	14 NW	No damage	No damage
Ranches	14 - 18 SE	Minor damage	Minor damage
Union Carbide	18 NE	Minor damage	Minor damage
Collbran	19 S	Minor damage	Minor damage
Rifle	20 NE	Minor damage	Minor damage
DeBeque	23 SW	Minor damage	Minor damage
TOSCO Tower	29 NW	No damage	No damage
Vega Dam	25 SE	No damage	No damage
Rifle Gap Dam	30 NE	No damage	No damage
Silt	30 NE	Minor damage	Slight damage
Mesa	32 SW	Minor damage	Minor damage
Harvey Gap Dam	32 NE	No prediction	No damage
New Castle	40 NE	No damage	No damage
Glenwood Springs	56 E	No damage	No damage
Grand Junction	56 E	No damage	Minor claim
Delta	75 S	No damage	No damage

In the hills surrounding shot point some dust was raised on the slopes as a result of minor rockfalls and slides at close-in areas of Battlement Mesa, and some rock slides occurred on the Roan Cliffs. Observers in Grand Valley reported seeing the shock front reflected in local vegetation movement.

Chimney damage and wall cracking occurring within the 7.4 to 14-km range appeared to coincide closely with observer reports of strong perception on that area.

Instances of falling masonry, generally from chimneys or parapets in Grand Valley justified the safety measure requiring that inhabitants be outside and two building heights away from their residence.

Motion in the Rifle area (18 to 20 km) was strongly perceptible, predominantly vertical, and generally lasted from 20 to 30 seconds. At Collbran (18 km) the perceived motion was less intense but similar in direction. A review of ground motion data again confirms the motion perceptions and damage observations of the observers.

Rockfalls resulting from the event-related ground motion were small in size and few in number. No extra efforts by highway maintenance personnel were required.

The damage to low-rise buildings from the RULISON event was of particular interest because of the applicability of the resulting data to predictions for similar future events. Project RULISON also provided the best and, to a large extent, the only example of real building damage in a populated area from an underground nuclear explosion, with adequate ground motion recording. The only other instance of extensive minor damage, in Hattiesburg, Mississippi from the SALMON event of October 22, 1964, was not well documented with respect to ground motion recording in Hattiesburg.

Blume's pre-shot damage cost prediction for RULISON totaled \$123,000. As of October 31, 1970, actual damage payments were \$117,500; remarkably good agreement with the pre-shot prediction.

Project RIO BLANCO RIO BLANCO was, like its predecessors GASBUGGY and RULISON, a gas-reservoir stimulation experiment. Project RIO BLANCO was a joint experiment sponsored by the U.S. Atomic Energy Commission and the CER Geonuclear Corporation of Las Vegas, Nevada. Lawrence Livermore Laboratory provided the explosive device and the firing process. The location was in the Piceance Basin of Rio Blanco county, Colorado (Figure 6.3), about 58 kilometers northwest from the preceding RULISON shot point, and some of the information gathered from RULISON was useful in

predicting RIO BLANCO effects.

The industrial sponsor employed other consultants for much of the pre-shot effects prediction work and post-shot evaluations. However, the DOE/NV consultants; ERC, the Special Projects Party, and Blume all had some involvement in RIO BLANCO effects studies.

Structural response prediction and evaluation work was done largely by Kenneth Medearis and Associates of Fort Collins, Colorado. Of particular interest and concern was the possibility of damage to the historic Rock School, about 11 kilometers northeast of shot point, and that of rock falls from the many unstable rock slopes in the area.

Pre-shot remedial measures at Rock School included regrouting its rock masonry walls to a depth of about 6 inches from the outside, as well as installing heavy interior bracing. Post-shot inspection disclosed only minor visual indications of ground motion effects at the Rock School. Rock falls did occur primarily along roads in the Piceance Basin near shot point. Falls were reported at 30 locations within the Basin, and scattered small rock falls occurred at a few more distant points.

Ground motion data for RIO BLANCO are set forth in the following table (Table 6.7).

A Blume study of RIO BLANCO damage, conducted for the National Science Foundation under grant GI-38090 (A75), determined statistical relationships for the damage data from 5 communities in the region, 4 of which had reported damage. Tables 6.8 through 6.11 summarize the findings from that study.

RIO BLANCO was unique in that 3 nuclear devices, separated vertically at about 400 foot intervals, were detonated simultaneously in this experiment. Three devices were located at depths of from 5840 to 6690 feet below the ground surface in Rio Blanco county, Colorado, in low-permeability gas-bearing sandstone, and the objective of the development of the was to create a vertically-extended fracture zone for increased permeability and resulting increase in production of natural gas from this zone.

The Special Projects Party deployed 36 L-7 velocity-meter systems as shown in Figure 6.4. Locations were chosen primarily for ground motion documentation and structural response studies.

In addition to the NSF-sponsored post-shot damage study, Blume conducted a pre-shot study and

FIGURE 6.3 RIO BLANCO AREA MAP

TABLE 6.7

RIO BLANCO - PEAK GROUND MOTION 9 VALUES

Town	Station	Component	Acceleration (g)	Velocity (cm/sec)	Displacement (cm)
De Beque	NSD-1	Z	0.0091*	0.314	0.0179
		R	0.0097*	0.216	0.0233
		T	0.0121*	0.246	0.0238
De Beque	NSD-2	Z	0.0087	0.291	0.0408
		R	0.0085	0.246	0.0300
		T	0.0114	0.244	0.0342
De Beque	RB-15	Z	0.0112	0.342	0.0234
		R	0.0086	0.190	0.0176
		T	0.0083	0.238	0.0160
Grand Valley	NSG-1	Z	0.0185	0.484	0.0333
		R	0.0253	1.00	0.0432
		T	0.0145	0.467	0.0361
Grand Valley	NSG-2	Z	0.0087*	0.309	0.0209
		R	0.0122*	0.435	0.0274
		T	0.0060*	0.308	0.0267
Grand Valley	RB-12	Z	0.0111	0.298	0.0239
		R	0.0144	0.598	0.0247
		T	0.0130	0.477	0.0402
Meeker	NSM-1	Z	0.0204	0.585	Not readable
		R	0.0301	0.645	
		T	0.0212	0.560	
Meeker	NSM-2	Z	0.0275	0.555	0.0347
		R	0.0318	0.645	0.0333
		T	0.0238	0.560	0.0309
Meeker	RB-13	Z	0.0178	0.455	--
		R	0.0228	0.641	0.0262
		T	0.0093	0.241	0.0263
Rangely	NSA-1	Z	0.0108*	0.356	0.0211
		R	0.0185*	0.441	0.0204
		T	0.0109*	0.305	0.0297
Rangely	NSA-2	Z	0.0215	0.587	0.0369
		R	0.0307	1.00	0.0571
		T	0.0238	0.684	0.0648
Rangely	RB-24	Z	0.0108	0.308	0.0339
		R	0.0196	0.347	0.026
		T	0.0136	0.308	0.0391

TABLE 6.7 - continued

Town	Station	Component	Acceleration (g)	Velocity (cm/sec)	Displacement (cm)
Rifle	NSR-1	Z	0.0344*	0.746	0.0490
		R	0.0191*	0.803	0.0408
		T	0.0186*	0.684	0.0725
Rifle	NSR-2	Z	0.0203	0.489	0.0275
		R	0.0182	0.716	0.0420
		T	0.0144	0.737	0.0566
Rifle	RB-16	Z	0.0079*	0.281	0.0218
		R	0.0132*	0.386	0.0236
		T	0.0073*	0.260	0.0361
Rifle	RB-17	Z	0.0155*	0.345	0.0339
		R	0.0184*	0.386	0.0234
		T	0.078*	0.281	0.0312

* Peak values uncertain due to high frequency noise.

TABLE 6.8

RIO BLANCO DAMAGE DATA

Town	Number of Buildings with:		Justified Damage Cost (\$)
	Complaints	Justified Damage	
Grand Valley	2	1	25
Meeker	56	39	10,948
Rangely	6	4	145
Rifle	7	5	612

Complaint: Any complaint of damage to an individual structure or associate appurtenance.

Justified Damage: Any of the above complaints acknowledged as justified by the responsible investigators.

TABLE 6.9

RIO BLANCO BUILDING INVENTORY AND BUILDING DESCRIPTION

Town	Number of Buildings	Percent of Buildings by Construction Type and Height						
		Wood Frame			Masonry			
		1 & 1½ Story	2 Story	≥ 3 Story	1 & 1½ Story	2 Story	≥ 3 Story	Other
Grand Valley	154	61	5	0	17	2	0	15
Meeker	651	66	5	0	12	2	<1	15
Rangely	488	78	1	<1	13	1	<1	7
Rifle	812	63	0	0	19	5	0	8

Town	Percent of Buildings by Use			Percent of Buildings by Age				
	Residential	Commercial	Institutional	0-5	6-10	11-20	21-40	> 40 Years
Grand Valley	78	18	4	<1	10	12	29	49
Meeker	82	16	2	15	8	32	30	15
Rangely	72	20	8	3	11	22	60	4
Rifle	83	15	2	2	16	20	27	35

TABLE 6.10 DAMAGE STATISTICS

Town	Complaint Ratio, CR (%)	Damage Ratio, DR (%)	Mean Damage Factor (m_{DF})	Standard Deviation (σ_{DF})
Grand Valley	1.3	0.65	1.08×10^{-5}	1.34×10^{-4}
Meeker	8.6	6.30	5.54×10^{-4}	4.27×10^{-3}
Rangely	1.2	0.82	1.20×10^{-5}	1.60×10^{-4}
Rifle	0.86	0.62	4.14×10^{-5}	6.42×10^{-4}

$$CR = \frac{\text{Number of Buildings for Which Complaints Were Received}}{\text{Total Number of Buildings}}$$

$$DR = \frac{\text{Number of Buildings Damaged}}{\text{Total Number of Buildings}}$$

$$m_{DF} = \frac{1}{N} \sum_{i=1}^N DF_i = \frac{1}{N} \sum_{i=1}^N \frac{\text{Damage Repair Cost for a Building}}{\text{Replacement Value of that Building}}$$

$$\sigma_{DF} = \left[\frac{1}{N-1} \sum_{i=1}^N (DF_i - m_{DF})^2 \right]^{1/2}$$

TABLE 6.11 SPECTRAL AMPLITUDE VALUES

TOWN	SPECTRAL ACCELERATION $S_{ao}(g)$	SPECTRAL VELOCITY $S_{vo}(cm/sec)$
Grand Valley	0.04080	0.8500
Meeker	0.07810	1.5300
Rangely	0.05610	1.0500
Rifle	0.04200	0.7900

FIGURE 6.4 LOCATION OF L-7 SEISMIC STATIONS, RIO BLANCO EVENT

structural bracing design for the Colony Tower on Parachute Creek.

The Colony Tower was a 180-foot tall steel-frame experimental oil shale processing retort structure with heavy process equipment on various levels. The shot point for RIO BLANCO was approximately 29 kilometers northwest of the tower. The Project RULISON shot point was about the same distance southeast of the tower. Instrumentation was located on the tower for the RULISON detonation.

After RULISON, a review of the motion recordings from the Colony Tower indicated that the 1st mode responses were less than predicted but the 2nd and 3rd mode responses were substantially greater than anticipated, and may have resulted in stresses close to allowable design code stresses. As part of a structure bracing program for Project RIO BLANCO, the industry-member of the sponsor team, CER-Geonuclear (CER), employed the services of Blume, through an existing contractual relationship with DOE/NV, to conduct structure response analyses and to design and implement a structural bracing program for the Colony Tower.

Selection of design criteria for the modification of the tower structure was made with a view toward assuring that no damage would occur to the structure. Consequently, these criteria established a reasonable but conservative approach to the prediction of response and stress in tower elements.

The results of the analysis of the existing structure showed that the median predicted motion would have caused overstress at the top level in the north-south direction only. However, motion at a higher statistical level would have caused overstress at the top three levels and the bottom three levels in the north-south direction and top three levels and bottom level in the east-west direction. To provide an adequate margin of safety, CER directed Blume, through the DOE/NV contractual relationship, to proceed with a program to design the installation of adequate permanent bracing and to modify existing bracing to assure that no significant overstress would occur at the upper statistical level considered appropriate for safety purposes.

Stearns-Roger, Inc., was selected by CER as the contractor because the firm had worked extensively on earlier tower modifications and security agreements were still in effect. On April 19, 1973, the Blume resident engineer met with representatives from Stearns-Roger at the Colony Development at Parachute Creek to inspect the tower and the facilities available to the contractor.

During the tower inspection, it was decided to stiffen the existing double angle bracing members by the

addition of steel channels, tees, and wide-flange shapes as appropriate for each particular vertical brace panel. All fastening of stiffening members to brace members would be accomplished by welding, with details supplied by the Resident Engineer.

In general, basic sizes and lengths of stiffening members were determined in office studies. Final sizes were dependent on the immediate availability of structural steel shapes from suppliers in Grand Junction. All steel was shipped unfabricated but cut to length and marked for the particular bracing panel. The first load arrived on May 2, after initiating the steel order on Monday, April 30, and the final load arrived Friday, May 4. All fabrication and replacement of stiffening members was accomplished in the field from sketches developed by Resident Engineer. The work progressed according to schedule and with a 100% safety record. The work involved fabricating and placing approximately 21 tons of structural steel shapes (134 pieces) and 750 pounds of welding electrode to strengthen the existing vertical bracing system for the tower.

The work was completed on a 10-hour, 7-day working week schedule and was completed on May 14. The Project RIO BLANCO detonation occurred at the scheduled time on May 17, 1973.

Following the detonation of RIO BLANCO, the tower structure was closely inspected by Blume's Resident Engineer. As expected no damage was found.

6.1.4 Interoceanic Canal Study

From 1965 to 1970 Blume participated in a feasibility study concerned with a proposed sea-level canal across the Isthmus of Panama, to be excavated by cratering nuclear explosions (Figure 6.5). The feasibility study was conducted under the auspices of the Atlantic-Pacific Interoceanic Canal Study Commission (IOCS). Blume's participation was arranged under a contract with the Nevada Operations Office. A Ground Shock Technical Working Group provided DOE/NV with technical assistance on outlining and reviewing work done on ground motion and structural response aspects of the feasibility study.

Initial specifications for the Blume study were as follows:

"The United States Atomic Energy Commission, in connection with feasibility studies for an Interoceanic Canal, has a requirement for a reconnaissance study of the structural response of various types of buildings and certain special structures in Panama and northwestern Colombia."

FIGURE 6.5 INTEROCEAN CANAL PROJECT AREA MAP

"The major part of this project is a study of both engineered and non-engineered structures in Colombia and Panama to determine their susceptibility to ground motion. Prior to the initiation of field work, a large amount of general information can be obtained from the Panama Canal Company (PCC). PCC staff members are very familiar with local building practices, and are also in a position to readily accumulate statistical data on relative numbers of various types of structures in selected towns and cities of interest."

"The field studies will be designed to gather data and to develop criteria essential to make meaningful damage prediction estimates in both dollar value and percent of structures affected. Information will be gathered to calculate the dynamic response of several typical structures to dynamic seismic loadings. Specific failure criteria (both structural and architectural) will be determined for selected types of buildings. The information will be presented in a way that will facilitate recalculation of relative levels of damage in accordance with any change in predicted ground motion spectra."

"Concurrent tabulation of data on foundation conditions, potential landslide areas, and local construction and repair costs will be carried out during the study."

"Specific consideration must be given to the present canal and associated structures. Also, unique structures of local sentimental or historical value should be given a preliminary appraisal whenever possible. Protective measures and costs of same will be outlined to meet projected safety requirements."

"The Panama Canal Company will be able to provide a certain amount of local support including local transportation in Panama, guide-interpreter, and some assistance in collecting general data."

"The contractor is specifically authorized to contract the services of consultants for this study. It may be desirable to augment the onsite studies with one or more experts in particular fields, thereby broadening the contractors' area of expert knowledge. Although the prime purpose of this study is to provide information for the Interoceanic Canal Study Program, it is also desirable that the work be conducted in such a manner as to foster good relations locally and make the optimum use of local information. Therefore it is suggested that the contractor secure the services of a local engineer-contractor in each country to assist in data collection and evaluat-

ing local problems."

"A status report is due July 1, 1967, and the draft of the final report on December 1, 1967."

Subsequent additional tasks assigned to Blume included the compilation of seismological history data for Colombia and Panama, specific analysis of 2 Colombian high-rise structures, investigations of earthquake effects for 2 earthquakes in Colombia in 1967, and the management of a study on sea waves that might be generated by the nuclear explosions.

As stated in the initial DOE/NV specifications, the major part of the project was to be a study to determine susceptibility to ground motion of both engineered and non-engineered structures in Panama and Colombia. In fulfillment of this requirement, two field trips to Panama and Columbia were made, one in February and March, 1967 and the other in August. Use was also made of information obtained on a prior reconnaissance trip to Panama in December 1965. Attention was directed primarily towards accumulating data on structures.

Other data acquisition efforts included frequent consultations and correspondence with the Office of Canal Studies in Panama and in Las Vegas.

A second requirement in the specifications was to gather data and to develop criteria essential to make meaningful damage prediction estimates in terms of both dollar value and percent of structures affected. Information was to be gathered to calculate the dynamic response of several typical structures to dynamic seismic loadings. Specific failure criteria, both structural and architectural, were to be determined. For selected types of buildings, the information was to be presented in a manner that would facilitate recalculation of the relative level of damage for any change in predicted ground motion spectra.

To fulfill this group of associated requirements, Dr John A. Blume devised the Spectral Matrix Method (SMM) for damage prediction. An initial report on the method was published in March 1968 (A14). SMM was used for dollar value predictions of both total damage and damage to individual structures for the yield levels on two selected canal routes.

Major conclusions were that damage to structures would be caused by the detonations, that this damage would become extensive at the higher yields, and that extensive structural safety studies and evacuation

programs would be required to minimize the possibility of injury to persons. Prevention of injury would be best assured by evacuating occupants from critical areas where the extent of damage was forecast to be at hazardous levels.

Blume's first formal IOCS report (excluding the 1967 preliminary structural effects report) was a compilation of available seismic data from Colombia, South America. The basic purpose of this study was to determine the seismic environment of Colombia. This was expected to provide insight on the history of seismic exposure and response to seismic forces on that population of Colombian structures that might be affected by the ground motion from the canal excavation explosions.

The second formal report in the IOCS report series was a compilation of seismic data on Panama. The format was the same as that employed for the Colombian seismic data compilation.

The third in the series of Blume reports on the inter-oceanic canal feasibility studies was concerned with modeling and dynamic analyses of two high-rise structures in Bogota, Colombia. These buildings had been exposed to ground motion from an earthquake on July 29, 1967, causing slight cracking in one building but no visible damage to the other. The fundamental purpose of the work was to examine and validate analytical techniques that could be used for reliable predictions of structural effects from the proposed nuclear excavation of an interoceanic canal.

As part of the feasibility studies for the proposed nuclear excavation of an interoceanic canal, a study of possible damaging water waves, of the seiche (waves in a confined body of water) or tsunami (seismic sea waves) types was assigned to Blume in December 1966, with the following specifications:

"The United States Atomic Energy Commission, in connection with feasibility studies for an interoceanic canal, has a requirement for a study of the possibility that ground motion from nuclear detonations will generate seiches or tsunamis. The area of interest includes Panama and northwestern Columbia.

"The United States Atomic Energy Commission and the Lawrence Radiation Laboratory will provide the contractor with preliminary predictions of ground motions and pertinent reports on the area.

"The study should include (1) an explanation of the problem as it applies to the subject area,

(2) evaluate the possibility that water waves might be generated and predict the possible height of such waves, (3) if the possibility of damaging waves exists, briefly indicate the nature and extent of such waves and (4) outline the type of study that would be necessary to provide a detailed evaluation of the problem.

"The report will be required within three months after the start of the study.

After being assigned responsibility for the study Blume arranged, with DOE/NV's permission, for Professor R.L. Wiegel of the University of California at Berkeley, a recognized expert in water wave problems, to conduct the study. Blume provided Professor Wiegel with field data, consultation on current predictions for ground motion from the proposed excavation, field investigations of the region affected, and assistance in report preparation. Professor Wiegel's report was appended to Blume's report on this study (A8).

During the course of the Blume IOCS studies in 1967, 3 major earthquakes occurred, 2 in Colombia and 1 in Venezuela. The effects of these earthquakes were of great interest and pertinency to the IOCS studies because of the occurrences of seismic damage to structures in the area of interest. On-site studies were conducted as part of Blume's IOCS program (A9).

Much investigative effort was conducted and reported on by Blume and other consultants over the period from 1965 to 1970. In general, Blume's results indicated that a great deal of additional study and much remedial effort would be required for safe conduct of the proposed excavation. The project was subsequently terminated, for various reasons. There have been expressions of renewed interest in subsequent years, although these have not included any nuclear excavation aspects.

6.2 STANDARD PRE-SHOT ACTIVITIES FOR OFFSITE EVENTS

In contrast to NTS events that took place in a region of increasingly well-documented basic structural data, the off-site events were characterized by an initial absence of fundamental information on the structural population of the area involved. Consequently the structural response effort was dominated during the pre-shot period by intensive data gathering activities. These were primarily in the field, centered on the shot location and extending as far out as significant ground motion was predicted to occur.

Initial studies by Blume, ERC, and Special Projects Party generally began simultaneously so that preliminary evaluations supplied to DOE/NV, including estimates of follow-on requirements, could be integrated and costs established. ERC ground motion predictions were particularly important in determining the extent and intensity of structural response study efforts.

Normally, Blume pre-shot efforts began with a planning study of readily available data on the location and nature of the experiment, to arrive at a preliminary plan for the structural response program. Using ERC's initial ground motion predictions, the distance from shot point at which structural response could be considered to reach damaging levels was calculated.

The seismic history of the area was reviewed to provide estimates of the magnitude of past seismic ground motion as a possible basis for evaluating damage threshold values for structures in existence at that time. Once these preliminary office studies and planning had been completed, the next step was usually to conduct a reconnaissance survey of the study area, to provide information on which to base subsequent field and office effort and to prepare a preliminary report on expected structural response aspects. Typically the reconnaissance survey was directed at obtaining data on structure numbers, types, ages, conditions, design and construction practice, and foundation material influences. Unusually hazardous conditions were also noted. Data on important structures, local building costs, seismic history, distribution of closein utilities, and similar information was obtained. Potentially hazardous buildings and special structures such as earth dams, steep slopes in developed areas, and tailings ponds possibly requiring special analysis were identified. Photographic coverage was widely used and the best available maps were utilized. Reasonably thorough coverage was conducted close-in, but appropriately less coverage at greater distance from shot point. Aerial reconnaissance was also used if appropriate or required.

Sources of published data on the structural population and general setting of the area were also sought out during the pre-shot period. These typically included governmental agencies with local jurisdiction, and also local professional individuals or agencies associated with building trades.

Utility and transportation agencies were also interviewed, to identify potential problems that might arise with their facilities as a result of ground motion.

Foundation material influence, slope stability problems, and similar material-structure response phenomena were considered, particularly with respect to any seasonal influences such as increased instability

during wet seasons.

The data obtained during these early activities were analyzed to provide the basis for a preliminary hazards evaluation. For this preliminary investigation, the damage cost estimates usually encompassed a relatively broad range and special problem structures were generally defined rather than specifically evaluated.

The major tasks for pre-shot reporting included structure inventory; condition surveys; analysis of data to predict structural response; identification of hazards and development of appropriate safety recommendations; and damage predictions.

Depending upon the circumstances for any particular off site event, such as the number of structures and the degree of damage expected, there could be additional work and reporting for better definition of the structural response problems foreseen and to outline safety measures to be taken. For most offsite events some remedial structural bracing or replacement of particularly hazardous elements, e.g. deteriorated brick chimneys, was recommended and implemented. For the RULISON event in 1969, the influence of major instability of saturated earth and rock slopes and high reservoir water levels at the initially planned May date led to our recommendation that the shot be delayed to a late summer period when these hazards would be greatly reduced.

Because of the importance of the basic structural data acquired during pre-shot activities a structural inventory report was normally prepared for major offsite events. The structure inventory was conducted to compile pertinent data on all structures of interest. The location and sufficient information to classify the structures for computer-based damage prediction processing was adequate for most buildings. Photographic coverage was carried out to an extent that would allow supplementary information to be obtained from the photos without special field visits. Assessor data was used for estimating the present value of buildings.

In cases where analysis indicated a high probability of damage for a structure, the structure was examined to determine if modifications or temporary bracing were economical or afforded a means to eliminate a potential hazard.

The structural inventory part usually included estimate of expected damage to each structure within damaging ground motion range.

If it appeared that significant damage would result from the event ground motion, a substantial post-shot damage investigation program was planned. An effort was made pre-shot to document existing conditions for selected typical structures within the expected damage area. This was done in order to provide comparative information on pre-shot and post-shot damage conditions, both for evidence in the nature of damage occurrence from event ground motion and as a basis for evaluating damage complaints.

Reporting pre-shot activities and their results was made in accordance with schedules established at the outset of the Blume work. For major events more than one report was issued. Generally, the lengthy structure inventory reports were printed separately from the structural response pre-shot reports. These latter reports included safety hazard investigations, damage cost predictions, remedial measure recommendations, etc. Frequently they were supplemented by informal letter reports and presentations to review panels.

Just prior to event time, observers were stationed at selected locations for personal observations of event motion, damage occurrence, etc. These observers made required investigations of those structures selected for pre-shot and post-shot comparisons of damage occurrence.

Instrumentation was deployed for off-site events to accomplish 3 major objectives; to monitor ground motion for structural response evaluation purposes and also for a basis for damage complaint investigations; to monitor structural response work, again for both structural evaluations and for damage complaint investigation purposes; and to record ground and structure motions for documentation and for structural response record purposes. Deployment was based on structural studies during the pre-shot period. Instrument types and range-setting were based on these studies and on the pre-shot predictions of ground and structural response motions. Post shot reports incorporated results of analysis of the recorded data. These data and analysis were also used for subsequent research reports.

6.3 MONITORING AND POST-SHOT ACTIVITIES

During and after an offsite event, structural response activity occurred in 3 time phases; event time monitoring during the period of perceptible ground motion, immediate post-shot activities during a period of a few days after the event, and long-range investigations and studies which extended to as much as a few years post-shot.

Event-time monitoring included visual and physiological observations by the experienced observers assigned to selected locations such as those where damage was expected to occur. These observations included reporting motion perception in accordance with criteria set forth at the end of Table 6.4.

The Special Project Party was responsible for deploying and operating instruments in accordance with ground motion and structural response needs. The Special Projects Party processed recorded data in accordance with program requirements, and furnished the processed data to the ground motion and structural response contractors for their analysis and reporting work.

The Environmental Research Corporation conducted extensive analyses of ground motion data, to be used both for general documentation and in furtherance of the development of a comprehensive and reliable ground motion prediction technology.

For structural response purposes Blume's immediate post-shot work was concentrated on the observation and recording of damage occurrence, both for subsequent use in damage complaint investigations and for research analysis of motion-damage relationships. Photographic recording and note taking were the primary means of documentation.

Post-shot activities included damage complaint investigation for up to one year after event-time, data analysis and evaluation for both documentation and for motion-damage research, and final reporting. Damage complaint statistics were obtained at the completion of the complaint investigation period. These were used for prediction purposes for future off-site events. These predictions provided a basis for estimating complaint activity post-shot and for assessing probable complaint investigation costs as well as for public relations purposes. Long-range post-shot activity also included analysis of event effects in relation to the ground and structural motion that generated those effects. Initial analyses were reported in final event reports issued after these studies were completed. These data were also used in many long-range research studies on motion damage relationships incorporating the results of a number of events. These studies were particularly valuable for seismic damage research purposes. Similar data from earthquakes were sparse because of the absence of extensive instrumental data as compared to that from the planned underground nuclear event program. In later years, increased deployment of seismic instrumentation at seismically active areas, such as for the 1971 San Fernando earthquake, largely overcame this problem.

6.4 SPECIAL STUDIES

For some of the major off-site events, special studies were conducted because of unusual or particularly important aspects of the event. Three such studies were the extensive and unexpected damage resulting from the 1964 SALMON event; the detailed investigation of the Harvey Gap dam for the RULISON project; and a study of the event effects on bridges, arising from the concerns expressed for the potential damage to bridges in the vicinity of the planned WAGON WHEEL project in western Wyoming.

6.4.1 SALMON Damage

Event SALMON, which involved the detonation of a 5 kt nuclear device at a depth of about 2,700 feet in the Tatum Salt Dome, 23 miles from Hattiesburg, Mississippi, took place on October 22, 1964 (Figure 6.6).

SALMON was one of the planned detonations of Project DRIBBLE of the Vela Uniform program, a national research and development effort to find improved techniques for detecting underground nuclear detonations. The SALMON event was sponsored by the Advanced Research Projects Agency (ARPA) of the Department of Defense and was executed by the U. S. Atomic Energy Commission (AEC) and its contractors. Blume's involvement, preshot, consisted primarily of an observer role.

Prior to the detonation, the following was included in a public information document relative to the SALMON Event:

"It is believed there is a possibility of structural damage to residence and outbuildings located within a radius of about 1.5 miles from the explosion point, with diminishing damage possible at distances out to about 4.5 miles from the explosion point. Based on studies of earthquake damage, maximum credible damage in the close-in zone could include cracking of masonry walls, unstable chimneys sheared off at the roof line (some falling through the roof), frame structures shifted on foundations, and windows broken. In zones further from the explosion point, the maximum credible damage would be confined to some cracks in plaster and masonry, and some broken windows."

The pre-shot damage predictions (in which Blume did not have an involvement) for the SALMON event were based on a threshold damage criterion of 0.10 g ground acceleration, which had been established by a Bureau of Mines study of damage resulting from quarry blasts. The predicted damage zone was a

FIGURE 6.6 SALMON AREA MAP

circular area within a 4.5-miles radius measured from shot point. Beyond this area damage was deemed improbable. Other criteria, such as 10 cm/sec peak particle velocity at ground surface, for estimating the threshold of damage to plaster walls and ceilings were also considered, but these generally placed the limit of damage closer than 4.5 miles from shot point.

After detonation of SALMON, the AEC received a large number of reports of damages to structures in areas outside the predicted damage zone, such as in Purvis (10 miles), Lumberton (12 miles), Columbia (17 miles), and Hattiesburg (20 to 23 miles). Prompt investigation of these structures revealed a high incidence of new cracks in plaster walls and in ceilings, stucco, concrete block and masonry construction. Other relatively minor damage of recent origin was also reported, including damage to water wells. There was, however, no collapse or impending collapse of buildings, nor any complaint pertaining to radiation damage. There was severe damage to a few chimneys close to shot point.

Hurricane Hilda passed through the general area in early October, 1964. The rainfall and winds associated with that occurrence may have had a contributory effect, although none of the official evaluations made any attribution to this source.

As of March 17, 1966, 1,189 formal claims had been made for minor damage such as cracking of plaster or masonry walls, and about \$548,000 had been approved for payment in restitution for claimed damage in this area.

In late 1964, the Nevada Operations Office of the AEC requested Holmes & Narver, Inc. to prepare an outline of a study program, to coordinate efforts of study participants, and to assume technical direction of the study.

The diverse nature and broad scope of the study required that the work be implemented by different agencies, not only from the standpoint of achieving the greatest efficiency but also to accomplish the required work in the available time.

On April 13, 1965, a general meeting of all participants, consultants, and scientific advisors was held at DOE/NV in Las Vegas. After all information had been presented and discussed, the consultants and scientific advisors met in a closed session to consider the evidence.

The following statement of conclusions was prepared and signed by the consultants and advisors.

- "(1) *The SALMON Event did indeed cause damage to structures located outside of the predicted possible-damage area, namely a circular area centered on ground zero with a radius of 4.5 miles.*
- "(2) *The explanation for the occurrence of damage to structures outside of the predicted area has not been established, although data point to various factors playing important roles. These factors include (not in order of importance): Differential settlement due to poor foundation construction, to construction on poorly compacted fill or on poor foundations, or to nonuniform foundation conditions; poor design and construction techniques; inherent vulnerability of masonry structures; asymmetry in seismic wave propagation due to regional geologic differences; pre-shot stress condition of structures; and structure orientation and response to excitation.*
- "(3) *Because of factors such as those listed in (2) and the large number and diverse character of the buildings, the previously accepted damage threshold criteria are not applicable to the damage resulting from the SALMON Event.*
- "(4) *In our judgement, further limited studies should be undertaken in order to better understand the damage mechanism. "*

A formal AEC report on the SALMON damage (E2) set forth other conclusions, observations and recommendations as follows:

- (1) *Minor damage to structures did occur beyond the predicted limit for damage of 4.5 miles, measured from shot point. In some directions, damaged structures were observed as far as 25 miles from shot point.*
- (2) *The damage to these structures was indeed real.*
- (3) *The damage done to these structures was caused by the SALMON Event.*

The Blume study results, appended to the formal AEC report along with those of other investigators, were separated into 2 parts; one with respect to structures and foundations, and the other regarding soils. Conclusions with regard to the study of structures and foundations were as follows:

- (1) *Forces caused by wind or SALMON ground motions could not, in themselves, cause stresses in materials to approach the inelastic range. An exception might be relative movement across a caulked joint of two dissimilar elements (for example, cabinets hung on a wall).*
- (2) *Comparison of calculated results shows that 40 mph winds create greater lateral forces on the structures studied than the SALMON ground motion.*
- (3) *Buildings with damage claims generally show evidence of prior settlement and/or shrinkage damage.*
- (4) *The majority of claimed damage is to brittle, non-yielding wall materials, such as concrete block, plaster and stucco. These types of materials are capable of supporting large loads without yielding. Failure of these highly stressed walls will occur generally for either of two reasons: (a) A brittle material stressed close to its ultimate capacity will sustain the load for a period of time, but then with very little warning fail without any additional load. (b) A brittle material stressed close to its ultimate capacity will fail suddenly under a slight increase in load. Therefore, the small stresses resulting from the SALMON event could be this 'small increase' need to cause a failure. But, it should be noted that the effects of time may have caused the same damage although at a slower rate.*

In addition, because the claimed damage is mostly due to vertical displacement (settlement), the slight increase of effective weight of the structure due to the vertical component of the SALMON event may have created the slight increase of load required for failure if the inherent stresses were at 99% of ultimate strength. Vertical ground acceleration in Hattiesburg was recorded at approximately 1% gravity.

Conclusions with respect to the soils study and soil-structure interaction were:

- (1) *Present conception of the mechanism of sudden settlement under dynamic loading requires that the soil be a loosely-packed, fine-grained, poorly-graded, clean, saturated sand or silt. Soils of this type can liquefy when shaken and cause large displacements. However, our review of preliminary incomplete data from the soils study and the borings logs together with the dynamic tests made, seem to rule out this mechanism as an explanation of the Hattiesburg and other distant damage claims.*

- (2) *An evaluation of the blow count and shear strengths of the Hattiesburg soils has indicated that the heavy brittle wall construction typical in Hattiesburg and the subject of many damage claims, imposes loads which are at or beyond the ultimate strength of these soils. Consequently, a pattern of past settlement and failure damage would be expected. It is expected that continuing failure of the soil under those loads will be creating stresses in the overlying structure that would be nearing failure thresholds in some elements. A slight additional dynamic load might trigger already imminent failure of weak brittle elements. This would explain damage claim concentration on overloaded soils whether they are sands or clays.*

6.4.2 RULISON - Harvey Gap Dam

The RULISON event was initially scheduled for May 1969. Blume studies and field work began in January 1969. In view of the expected increased instability of earth and rock slopes in the RULISON area during the spring thaw and wet weather a considerable effort was expended in evaluating potential problems arising from such instability. Dams were given particular attention because of expected high water levels at shot time. Early in the progress of the evaluation Blume field investigators observed and became concerned about the old Harvey Gap Dam, about 30 kilometers from the RULISON shot point and approximately 3 kilometers upslope from the town of Silt, Colorado (see Figure 6.2).

Harvey Gap Dam and Reservoir, also known as Grass Valley and Grass Valley Antlers, was originally constructed in 1891. The dam was of the earth-fill type, 49 feet in height above the foundation with crest length 580 ft; width of crest 10 ft; width of base 225 ft; downstream slope 2:1; upstream slope 3:1. The original dam failed in 1895 and was rebuilt in 1909. In 1921 the dam was rehabilitated and raised to a height of 60 ft with a crest length of about 800 feet. The gross reservoir capacity was increased to 5058 acre-feet with a high water surface area of 206 acres. Harvey Gap Reservoir supplied water for irrigated lands on Harvey Mesa north of the town of Silt.

Blume's concern regarding potential RULISON effects on Harvey Gap Dam was expressed in early progress reports to DOE/NV and led to a much more extensive study of the possible potential hazard presented by the Harvey Gap Dam. A separate study and report was prepared by Drs. H. B. Seed and J. L. Sherard, assisted by Woodward-Clyde and Associates (A30).

This study was conducted between April 16 and 26, 1969. Field work included observations by Dr. J. L. Sherard, thirteen borings with 2 drillings and hand augers, 2 continuous shallow trenches on the downstream-slope of the dam and 6 test pits. Field and laboratory tests were made of the earth materi-

als in the dam.

The Seed, Sherard, and Woodward-Clyde & Associates study concluded that a remote probability of damage to the Harvey Gap Dam existed, and recommended certain precautionary measures. These were amplified by the Safety Panel (DOE/NV Panel of Safety Consultants), an independent board of experts who regularly reviewed the progress reports made for DOE/NV, and who were particularly concerned about potential liquefaction of possible sand lenses below the dam. The Safety Panel recommended certain action for specific situations of reservoir water level. The study contained recommendations for provisions for timely evacuation if the water level was 15 to 25 feet below the dam crest, and no immediate evacuation requirements if the water level was more than 20 feet below the crest of the dam. Damage to downstream stock-watering ponds, utility lines (gas, electric, telephone), irrigation ditches, roads, the Denver and Rio Grande railway, irrigation ditches and cultivated lands, as well as farm buildings, could accompany serious flooding.

Subsequently, the RULISON shot date was postponed to September 1969. From discussions with persons in the area familiar with water usage and reservoir levels, it was predicted that the reservoir water level in September would be about 40 feet below the dam crest, assuming normal summer rainfall and runoff from the local drainage area, and that the reservoir would be almost dry. This prediction essentially eliminated all further concern about problems arising from the Harvey Gap Dam, and only instrumental and visual surveys were planned for the new shot date. As predicted the reservoir was almost empty at the September 10, shot date. Damage was not expected and none was observed.

6.4.3 WAGON WHEEL - Bridges

In June 1969, Blume conducted reconnaissance surveys of an area in western Wyoming, site of two proposed underground natural gas stimulation projects called WASP and WAGON WHEEL. The WAGON WHEEL location was about 32 kilometers south of Pinedale, Wyoming, in a sparsely-populated ranching area (Figure 6.7). El Paso Natural Gas Company, commercial member of the sponsoring group, had conducted an inventory of structures in the vicinity of the project location. The inventory together with the Blume reconnaissance data, was used for a preliminary structural effect evaluation report issued in November 1969. Tentative upper level damage cost for a 100 kiloton nuclear detonation at a 10,000 ft depth of burial was estimated to be \$200,000.

El Paso Natural Gas Company subsequently contracted with other consultants to conduct final effect evaluations for project WAGON WHEEL. Work done by these consultants in the 1972 - 1974 period

FIGURE 6.7 PROJECT WAGON WHEEL AREA MAP

was presented in reports issued in 1972 and 1974. Table 6.12, extracted from the 1972 report, indicates the possibility of considerable damage to nearby structures especially bridges.

As directed by DOE/NV in early 1972, Blume reviewed the consultant reports, and expressed an opinion that the equivalent static loading analysis approach used by the consultant tended to overpredict damage from the ground motion caused by underground nuclear detonations. Blume did acknowledge that the multiple-detonation approach planned for WAGON WHEEL was unprecedented and might tend to invalidate previous experience with underground nuclear detonation effects. To assist in a final resolution of this difference of opinion, Blume proposed to revisit the SALMON, GASBUGGY and RULISON locations for detailed inspections of bridges in the close proximity to these project locations and to verify previous determinations that no damage had been experienced by these bridges.

These visits and re-evaluations took place in February 1972. Blume's subsequent report to DOE/NV (A52) noted that the surveyed bridges exhibited only normal deterioration and no evidence of event-related damage, confirming the absence of reported damage at the time of the respective events. This suggested that high acceleration levels by themselves were not sufficient to cause damage to bridges. Earthquake motion that had caused bridge damage in the past was normally characterized by peak ground accelerations similar to those from the above nuclear events, but having considerably greater displacement amplitude and longer duration.

Blume's April 1972 report to DOE/NV on its review of the El Paso consultants' work indicated that nuclear detonation experience showed bridge damage had not occurred at ground acceleration levels similar to those for WAGON WHEEL. However, the proposed bracing and other remedial measures recommended by the El Paso consultants seemed to be adequate even for the perhaps overly conservative analysis used.

Project WAGON WHEEL did not proceed to completion and it remains a matter of speculation as to what damage, if any, might have resulted from the multiple detonation sequence planned for the project.

TABLE 6.12

SUMMARY OF DAMAGE POTENTIAL - SIGNIFICANT STRUCTURES

Structure		Location		Description	Damage Potential (A)		
Map No.	Index No.	Miles to G.Z.	Azimuth Degrees		Mean Acceleration		
					Existing Condition	Preventive Work	Improved Condition
2	B5-1	5.5	272	6 Span Concrete Bridge	5	Yes	4
6	B6-3	6.8	15	5 Span Concrete Bridge	4	Yes	2
9	T8-1	8.9	252	6 T.V. Towers	4	Yes	2
15	B10-5	10.2	7	3 Span Concrete Bridge	4	Yes	1
17	B10-7	10.6	258	3 Span Concrete Bridge	4	Yes	1
25	B12-4	12.8	37	3 Span Concrete Bridge	2	Yes	1
26	B13-1	13.0	354	1 Span Wood Bridge	1	No	-
28	B13-3	13.4	66	3 Span Concrete Bridge	2	Yes	1
29	B13-4	13.4	352	5 Span Wood Bridge	2	Yes	2

(A) Key:

1. No expected damage.
2. Slight damage (no loss of use).
3. Some damage (No significant loss of use before repair).
4. Serious damage (Probably short term loss of use before repair).
5. Severe Damage (Probable collapse; unusable).
6. Presently failed but being used. Collapse possible from any cause.

7 PARALLEL STUDIES

7.1 GEOTECHNICAL INVESTIGATIONS

7.1.1 Compaction Fault Scarp Studies

The City of Las Vegas is located in the southern and widest part of Las Vegas valley, on a broad sloping surface of unconsolidated alluvial sediments (Figure 7.1). This alluvial slope extends eastward for a distance of approximately 12 miles from the base of the Spring Mountains and terminates in a relatively flat-lying valley bottom sloping to the south towards Las Vegas Wash, which drains southeasterly into Lake Mead.

The valley bottom consists of unconsolidated deposits of silt and clay that interfingered with the layers of gravels, sands, silts and clays of the alluvial slope to the west, and to a lesser extent the alluvial material from the Sunrise-Frenchman Mountain area to the east.

Most of the older portion of the city of Las Vegas is situated on the alluvial slope to the west of the flat-lying valley floor. Recent residential developments have expanded out into the valley floor in the east part of the city and up the alluvial slope on the west and northwest part of the city.

The alluvial slope is interrupted by two prominent and several smaller scarps. These are relatively abrupt changes in elevation, stepped down to the east and trending in a northerly direction through the city (A67). They have been entitled compaction fault scarps because they are commonly believed to have originated through differential compaction between coarser sediments to the west and finer sediments to the east of these scarps. The westernmost of the two major scarps forms a boundary between gravels to the west, and sands and silts to the east. The eastern major scarp is a boundary between the sands and silts to its west, and silt and clay to the east.

The compaction fault scarps in Las Vegas became subjects for investigation because of the possibility that ground motion from NTS testing might trigger sudden additional subsidence through release of built-up stresses from continuing differential compaction along the scarps. Movement across the fault scarps had been noted in 1957 and 1961, especially across the easternmost. A primary cause for movement across these fault scarps was concluded to be the continued differential compaction of sediments resulting from excessive groundwater withdrawal.

FIGURE 7.1 PHYSIOGRAPHY OF LAS VEGAS VALLEY

A measurement program was conducted by Blume, beginning in 1965 and continuing for several years, to determine whether or not any differential movement might be occurring on the compaction fault scarps as a consequence of ground motion from NTS. This program included vertical and horizontal distance measurements between reference points situated on either side of compaction fault scarps where some indication of recent motion was visible, including large open cracks in a section of North Las Vegas.

Repeated measurements of vertical and horizontal distances between those reference points did not disclose any measurable differences before and after ground motion occurrences from large tests at the NTS. It was concluded that the NTS testing program and resulting ground motion during the period of study was not causing any measurable ground changes at the study location.

7.1.2 Subsidence Problems

In addition to the localized compaction reflected by elevation changes along the compaction fault scarps, other subsidence was occurring throughout Las Vegas Valley. This was disclosed as a consequence of ground surface elevation change studies conducted in association with the construction of Boulder Dam and the creation of the resulting reservoirs.

Leveling by the United States Coast and Geodetic Survey (USC&GS) was initiated in 1935 to ascertain the effects of deformation of the earth's crust as a result of the weight of water impounded in Lake Mead. Leveling was repeated in 1940-41, 1949-50 and in 1963. Portions of this level net included Las Vegas Valley and were first-order (precise) surveys. A second order survey was run over selected benchmarks in Las Vegas Valley in 1957.

Comparison of these level data for Las Vegas Valley revealed a broad area of surface land subsidence, about 200 square miles in area. While the subsidence was largely due to compaction of sediments at depth as a result of excessive ground water withdrawals, some minor additional subsidence had been attributed to tectonic causes and to the additional load from the Lake Mead Reservoir. A rising perched (isolated near-surface) water table in some parts of the valley and the resultant saturation and weakening of surficial sediments also caused some local subsidence.

Continuing subsidence from 1935 to 1963 had resulted in a cone-shaped depression of the land surface centered between the downtown area of Las Vegas and the western edge of North Las Vegas. In 1957 and 1961 fissuring, believed to be an expression of subsidence, developed along the easternmost compaction fault scarp that cuts through this area of subsidence. Other fissuring along this scarp had

occurred in the recent past but time of occurrence was unknown.

Problems arising from this subsidence and fault scarp movements had primarily been restricted to utility lines within the area and to curbs and pavement, although a few residential structures had shown some differential subsidence effects attributed to these mechanisms.

Blume's involvement in the subsidence problem was generated by the need to discriminate between damage to structures arising from natural causes, such as differential settlement caused by subsidence, and that which might occur directly as a consequence of ground motion from NTS activities. In some cases ground motion from NTS testing was determined to have triggered a release of stress primarily caused by differential settlement or to have extended cracks that already existed from differential settlement problems. In most cases the analyses of damage depended primarily on the evidence of recency of crack generation, with those obviously of immediately recent origin being attributed in some degree to NTS ground motion aggravation or stress release. In any case the knowledge of existing subsidence problems was of considerable importance in analyzing claimed damage, in that such knowledge was incorporated into the damage evaluation process. A damage complaint from a house in a subsidence area became more credible because of the prospect of such damage having resulted from a triggering of built-up stress caused by subsidence.

7.1.3 Expansive Soils

During the course of investigation of foundation problems and their contribution to new or existing structural damage around 1965, Blume briefly studied a soils problem arising from chemical composition of some of the foundation materials.

Clays in the younger alluvial sediments in the eastern portion of the valley contained appreciable quantities of water soluble salts such as the various mineral compounds of sodium sulfate. These salts take water into their crystal structure at certain critical temperatures, with corresponding increases in volume. These volume changes commonly occur twice each year in seasonal response to air temperature changes. The availability of water is also a critical factor in the volume change. Uplift pressures in these sulfate-rich soils range between 700 and 2300 pounds per square foot. Such pressures have had disastrous effects on some "slab-on-grade" construction.

Since the problem was soon found to have no conceivable relationship to NTS testing activities, it was not further investigated by Blume. A solution to the difficulty was devised by local soils engineers,

consisting basically of a removal of the top foot or two of such problem soil and replacing it with a crushed rock or gravel fill.

7.2 WIND STUDIES

Blume studies have been conducted of the actual and potential response of Las Vegas buildings to wind loading. These studies, together with those on earthquake effects, were intended to provide DOE/NV with a comparison of the effects of such naturally-occurring forces on structures with those from NTS activities.

Wind-induced motions of tall structures are normally oscillatory about some large mean deflection, i.e., a down-wind lean. This quasistatic mean response is generally of the same order as, or larger than, expected fluctuating responses. Because structure damage is directly a function of stress or interstory drift, the quasistatic response must be evaluated and considered along with the fluctuating response.

As a consequence of the typical large wind-induced quasistatic response, experimental data are difficult to evaluate. Wind-induced building motions must therefore be measured with displacement meters rather than velocity meters or accelerometers; and it is necessary also to obtain a true at-rest position after the wind loading has ceased.

An extensive study was conducted of the wind response of a selected high-rise structure in downtown Las Vegas, Nevada (A70). Las Vegas had relatively frequent high wind occurrences of moderate intensity and was located in a zone having a specific high wind speed, for structural design purposes, of approximately 70 mph, as specified by the Uniform Building Code.

The subject of this full-scale experimental investigation was a 16-story office building located in downtown Las Vegas, Nevada. The building was completed in 1975 and was representative of modern design and construction practices. The structure was relatively simple in configuration, permitting reliable analyses of response characteristics.

The building was located approximately 15 km north of McCarran International Airport. As a consequence, climatological parameters, such as mean wind speed and direction, obtained at the station were applicable to the building site, provided corrections were made for differences in terrain roughness and elevation.

An anemometer and vane assembly mounted on a guyed mast near the center of the building approximately 10 m above the top of the parapet wall provided continuous output of wind speed and direction. Ideally, this device should have been at least 15 m above the wall to reach the undisturbed flow above the building, but this was prohibited by building regulations. Consequently, the wind measurements obtained from this instrument were somewhat affected by the aerodynamic influence of the building.

Microbarographs, provided through participation by SNL personnel, provided an integral measure of wind pressure at the lower exterior surface of each parapet wall. Each set of three pressure ports along a building face was connected to a common manifold and microbarograph, producing an average pressure measurement across that face.

Two orthogonal arrays of velocity sensors positioned at the north and south ends of the roof permitted extraction of the translational and torsional components of dynamic building response.

A low-power, alignment-quality laser mounted at the foundation level was directed vertically through a central elevator shaft to a dual-axis, position-sensing photodiode attached to a roof-level structural member. This instrument permitted measurement of the quasistatic response component. Because only one of these instruments was used, it was not possible to obtain information concerning quasistatic torsional response.

Output from all transducers was recorded continuously on a 14-channel, FM magnetic tape system. Continuous recording was necessary to obtain a low-wind record near each storm occurrence to serve as an instrument tare for the pressure and displacement transducers. Absolute time was recorded on one tape channel to assist in the subsequent location of desired data segments.

Numerous moderate intensity storms and a few extreme wind occurrences were recorded during the course of this study. Five of these storms, recorded between November 1980 and August 1981, were selected for detailed analysis. Of these five storms, three produced response amplitudes of moderate amplitude. The other two storms were relatively severe, which contributed to some malfunctions of the instrumentation system. As a consequence, only abbreviated data records were available from these two storms.

The conclusions of the study (A70) confirmed that full-scale experimental studies were applicable to the evaluation of methodologies being established at the National Bureau of Standards, and that reasonable

comparisons existed between analytical and experimental data. The experimental observations suggested that quasistatic response is overestimated by the numerical procedure, at least for storms of low to moderate intensity. The experimental data also illustrated that torsion is an important element in wind-induced response and was produced by both aerodynamic and structural phenomena.

7.3 EARTHQUAKE HAZARD IN SOUTHERN NEVADA

Blume conducted two studies of the geology and seismology of Southern Nevada as part of its investigation of the potential effects of foundation materials and natural earthquakes on Las Vegas structures (A62, A67). This study was part of a broad investigation of the effects of naturally-occurring forces on Southern Nevada structures compared with those generated by NTS activities.

Earthquakes presented the most obvious comparative natural hazard because of their similarity to ground motion from NTS underground nuclear detonations. To evaluate potential earthquake effects an investigation was conducted of the natural seismicity of Southern Nevada. The probabilities of earthquake ground motion in Las Vegas were derived from that study by determining seismic exposure and combining it with statistical analyses of maximum probable peak ground accelerations, using an acceleration-magnitude-distance relationship developed in this study.

The first step in the assessment of probable earthquake ground motion in Las Vegas was to examine the geology of the Basin and Range province and adjoining provinces to gain a long-term perspective of regional tectonism. Las Vegas is located in the southern portion of the Great Basin physiographic province, which approximately coincides with the northern portion of the Basin and Range tectonic province.

The Las Vegas area lies within a zone of transition between contrasting styles of tectonic behavior. To the west lies the belt of distributed right-lateral deformation between the Sierra Nevada front and the Death Valley - Furnace Creek fault system. In the Mojave Desert, to the southwest, deformation was predominantly right-lateral along northwest-trending faults. To the east of Las Vegas lies the relatively thick, stable Colorado Plateau, bounded by young normal faults (a normal fault has one side displaced downward with respect to the other). On the north is the Basin and Range tectonic province of central Nevada, in which generally east-west extension of the crust was occurring.

A major conclusion of this study was that since Miocene time the rate of crustal extension has been much

greater in the northern part of the Basin and Range province than in the southern part. The transition zone between these two different regimes was a broad, roughly east-west belt uniquely characterized by Pliocene and younger lateral faulting and by normal faulting, which might have been a secondary response to left-lateral shear at depth. The zone was some 100 km wide, trended about N 70° E, and was transversed by a major lineament that extended northeast from the Garlock fault and included the Pahrnagat left-lateral system (facing fault, opposite side is displaced to the left). This zone was significant because it separated regions of widely different Holocene and present-day seismotectonic activity. Las Vegas lies in the southern, much less active region.

Long-term seismotectonic activity was assessed on the basis of post-Miocene fault patterns, topographic relief, crustal thickness, and late Quaternary faulting; the assessment led to a subdivision of the Basin and Range province into several seismotectonic subprovinces. Crustal strain rates were estimated for each of the subprovinces using data on late Quaternary faulting and geodetic strain measurements. Average historic seismicity rates were derived for each of the subprovinces and were found to correlate very well with the late Quaternary crustal strain rates. It was concluded that historic seismicity rates were representative of a period more than 10,000 years long.

A computer program, HAZARD, was used to calculate probabilities of exceedance of various peak ground accelerations in Las Vegas. The calculations were based on the historic seismicity rates that characterize six seismotectonic source regions within 300 km of Las Vegas; the San Andreas fault was also included as a source. Results indicate that Las Vegas was subject to low seismic risk relative to much of the western United States.

Las Vegas is located in a region of locally low seismicity that is flanked by regions of much higher seismicity. It was concluded that accelerations from 0.01g to 0.02g were as likely to result from small, local shocks as from large, distant earthquakes. Thus, ground motion events of 0.02g were apt to have predominant periods of either 0.5 sec or 0.2 sec and durations of either 30 sec or 0.3 sec. The events with longer period and duration had a mean return period of some 100 years.

Data on site-dependent spectral amplification in Las Vegas showed that pseudo-relative spectral velocities (PSRVs) in the downtown area might be from two to seven times as large as those in outlying areas for periods between 0.5 and 6.0 sec. The lowest observed PSRVs appeared to represent average alluvial sites, as did the computed peak ground accelerations. Hence, longer-period spectral motions in the downtown area might be much larger than those normally associated with the peak ground motions cited

above.

7.4 TERMINAL WASTE STORAGE

As part of an investigation conducted for the Nevada Operations Office to assess the feasibility of establishing a nuclear waste repository at the Nevada Test Site, Blume studied costs associated with strengthening the repository for seismic forces (A63) and also considered the procedures appropriate for developing seismic design criteria for such a facility (A66).

The cost study presented a preliminary estimate of the costs required for hardening terminal waste storage at candidate NTS locations against ground motion caused by earthquakes and underground nuclear explosions (UNEs). These hardening costs were the additional costs beyond static design and construction costs.

No appropriate conceptual design for the waste repository was available at the initiation of the study in 1978. Thus, it was necessary to assume a configuration that would represent a prototype facility and would be applicable to a variety of sites with different geologic conditions. Five proposed repository concepts were reviewed: three bedded-salt waste-isolation facilities and two retrievable surface storage facilities. The concept selected for study was based primarily on the bedded-salt waste-isolation facilities and was developed for storage of high-level waste (HLW) only.

The structures of the repository were classified according to the three seismic categories generally used in the nuclear industry. Category I, which includes all structures whose continual functioning or integrity is essential for the safe containment of waste materials, is generally the only category whose structures require hardening beyond the requirements of local building codes.

All of the surface structures in this category were assumed to be reinforced concrete, shear-wall structures. It was further assumed that tornado design criteria that apply to safety-related nuclear power plant structures would similarly be required for surface repository structures.

Four geologic materials, characteristic of the following candidate construction sites, were reviewed:

1. Eleana Formation argillite (hereafter called shale)
2. Climax stock granite

3. Jackass Flats tuff
4. Jackass Flats alluvium

Engineering geology information obtained from data available at the NTS was used to provide a basis for the preliminary design of support systems for the underground repository structures. Data on the candidate materials taken from bore holes and existing tunnels were assumed, for the analysis, to be uniform for all sites at the NTS where the material occurred. Properties were then assigned to the materials on the basis of the acquired data. Evaluation of the Jackass Flats alluvium was discontinued at this point in the investigation because the available data for the material were insufficient for even a conceptual design.

The Eleana shale was found generally to have low hardness and strength properties. Some portions of the shale exhibited high slaking potential and closely-spaced fracturing. Portions could be difficult to mine and support because squeezing ground might be encountered. The tuff and the granite had high strength properties.

The peak ground motion accelerations assumed for this study were specified by SNL as follows:

1. From earthquakes -- 0.3g, 0.5g, 0.7g.
2. From UNEs -- 0.5g, 0.7g, 1.0g.

For dynamic structural analysis, the frequency content of ground motion, as well as the peak ground acceleration, is usually required. This information can be provided by response spectra, which are idealized peak response plots. For the purposes of this study, the response spectra recommended by NRC Regulatory Guide 1.60 were adopted for the surface structures, for both earthquakes and UNEs. A value of 7% of critical damping -- a typical value for concrete structures at high stresses -- was used.

Response spectrum analyses with single-degree-of-freedom models were conducted for the surface structures. Approximate fundamental frequencies and seismic forces and stresses were computed for the specified ground motions. Wherever the seismic stresses exceeded the allowable stresses, the structures were theoretically hardened and then were reanalyzed; this process was iterated until the stress criteria were met. The added costs for hardening were then computed.

Thick exterior walls required for tornado hardening for surface structures also provided major resistance

to seismic motion; therefore, the added costs for hardening the surface structures were small.

A simplified but conservative approach was taken for the investigation of stresses and strains at depth. Dilatational and shear waves were assumed to be propagating through the medium in the directions that imposed maximum strains on underground structures. Various combinations of these waves were then considered in order to obtain the largest possible strain. The additional hardening requirements for underground structures were based not only on incremental seismic stresses (which must be compared to the in-situ static stresses in the rock formation) but also on a practical and experience-oriented evaluation of the effects of seismicity on underground openings. It was concluded that the primary effort in hardening tunnels and shafts must be to improve construction details to achieve a more coherent system between the rock and the tunnel. With this methodology, a program for the incremental improvement of the underground support structures was developed.

At the initiation of the study, it had been planned to compute only the added costs for hardening. However, as the work progressed, it became clear that hardening costs could not be entirely separated from static construction costs. Therefore, the static costs were estimated for all structures. These cost estimates were based on data for conventional structures (such as rapid transit tunnels) that do not involve such costs as licensing, nuclear quality assurance, security, etc. The approach used in presenting study results was to calculate the percentage cost increase over conventional construction costs for the various seismic motion levels and for the three candidate repository site earth materials. These are shown in Table 7.1 below.

TABLE 7.1

SUMMARY OF ESTIMATED HARDENING COSTS AS PERCENTAGE INCREASE

Maximum PGA (g)	Cost Increase per Linear Foot (%)					
	Tunnels			Four Shafts Combined		
	Granite	Tuff	Shale	Granite	Tuff	Shale
0.3	0	0	0	0	0	0
0.5	3	18	1	1	1	1
0.7	27	26	13	1	1	1
1.0	34	35	13	1	1	10*

* sudden sharp increase occurs between 0.7 and 1.0g

In conclusion, this study showed that, on the basis of the assumptions that were made, no significant added costs for hardening against ground motion were expected for surface structures or for underground structures at or below 0.3g peak ground acceleration. However, the added costs for hardening became significant at 0.5g or above and could increase the total cost by as much as 35%. These cost estimates were quite sensitive to the criteria, assumptions, and procedures used in the design for seismic hardening. In particular, the final performance and design criteria selected for the terminal waste repositories would strongly influence total costs.

The second Blume study was intended to provide a discussion of the needed elements for ultimate development of appropriate seismic design criteria for nuclear waste repository at NTS. The scope of the study included identifying appropriate configurations for the surface and underground structures, reviewing potentially applicable existing criteria, consideration of ground motion and rock property parameters needed for seismic analyses of underground structures and criteria development, defining risk aspects, and proposing design philosophies appropriate to the facility. Consideration and recommendation for additional studies were also required.

The report presented results of the various reviews and identifications required by its scope. Its major conclusion was that while qualitative assessments could be made, the current state of knowledge and technical progress in underground facility design was such that quantitative assessment was not presently achievable. For this reason the major product of the report was a list of recommendations for needed additional studies and data acquisition programs to reach a point at which quantitative design criteria could be provided. The report presented these recommendations, primarily for the operational phase of the repository development design during which waste is received and stored. The decommissioned phase, after the repository is filled and no further activity other than monitoring of the stored waste is taking place, required only continued research on seismic effects and on the question of long-term barriers to the escape of radioactive materials from the repository.

7.5 HISTORIC SITE SURVEY

In compliance with a series of federal directives, enacted during the period from 1966 to 1974 and concerned with the preservation of historic resources, DOE/NV directed Blume to conduct a survey of historic sites within a specified distance from the NTS. These historic sites were those that were listed in the National Register of Historic Places or were candidates for such listing.

In 1968 and 1969 Blume had conducted comprehensive structural inventories of central and southern Nevada regions within those areas adjacent to the NTS that might experience potentially damaging ground motion from planned explosions at the proposed Central Nevada Testing Area and at the NTS. The resulting inventory data provided much of the needed information on location, content, and condition of the sites and their historic and cultural features. The basic historic site survey was conducted during the spring, summer, and fall of 1977.

A specific purpose of the survey was to permit determinations of the probable effects of NTS testing activities, primarily of the ground motion generated by underground nuclear detonations, on the physical features of the sites, particularly any structures or ruins still standing.

This purpose fitted well with Blume's primary responsibility in its work for DOE/NV; i.e., the determination of structural response effects on structures outside the NTS. Cultural resources in Nevada and California within a radius of approximately 140 kilometers (87 miles) of the NTS, but exclusive of the test site, were investigated (Figure 7.2).

The test site itself was the subject of an ongoing cultural resources inventory conducted for DOE/NV by the Desert Research Institute of the University of Nevada.

Cultural resources were defined as (a) sites that were listed in the National Register of Historic Places or were candidates for such listings, (b) sites designated by a State Historic Preservation Office as eligible for listing, and (c) sites that came to the attention of Blume personnel during the course of the investigation. Data compiled by Blume were derived from libraries, museums, public agencies, regional experts, and the company's records of previous site inventories as well as from onsite inventories to determine the present condition of natural or man-made historic structures. Many of the sites included in the survey had not been visited previously. Their existence and location were determined through communication with personnel of the Nevada and California historic preservation offices, the Bureau of Land Management, the Nevada State Historical Society, the Nevada State Museum, the National Park Service, and the archaeological districts within the area of study or through search of the files made available to Blume by these agencies.

Field investigations were conducted to determine the location of each site and to document the kinds and conditions of structures then present. Site and structure inventory forms were developed for the study. These were augmented by photographs of the site and its significant structures, and by supplemental field

FIGURE 7.2 HISTORIC STRUCTURE SURVEY AREA

notes where necessary.

As each site was investigated, information was recorded first for the overall site: geographic location and description, topography, a summary of buildings (by number, use, height, and construction material), and a list of related structures. In most instances, one or more significant structures were then selected for detailed description that included approximate date of construction, dimensions, orientation, types of building materials, foundation type, framing, fenestration and other nonstructural details, and present condition. In some cases, a structure was selected for detailed description because of its exceptional historical value; in other cases, because it typified structures at the site or because of its condition.

The factors that influence the effects of ground motion on structures include the amplitude and frequency characteristics of ground motion, the type and condition of the structure and its elements, and the characteristics of the foundation material. Structures of interest in the evaluation of historic sites were man-made objects that could be affected by ground motion produced at the site by distant underground nuclear explosions. The ranges of acceleration as shown in Table 7.2 below were adapted from previous ground motion effect studies and used to determine the study area.

TABLE 7.2
ACCELERATION-DAMAGE RELATIONSHIPS

Peak Horizontal Ground Motion Acceleration (g)	Probable Occurrence of Damage
0.3 or above	extensive
0.1 to 0.3	limited
0.05 to 0.1	occasional
0.005 to 0.05	rare

As stated, sites investigated were those within a 140-kilometer (87-mile) radius of the NTS; that is, sites within the area that might be exposed to measurable levels of strong ground motion from UNE at the test site. The level of ground motion is a function of both UNE magnitude and distance from the source of

energy release; the area of investigation was based on a hypothetical 1,000-kiloton (kt) UNE detonated at Pahute Mesa, in the NTS. Because the level of ground motion would be higher in the area nearest to the NTS, nearby sites were inventoried in more detail than those at the fringe of the study area.

APPENDIX A

BLUME REPORTS

REF. NO.	REPORT NUMBER	TITLE
A1	NVO-99-09	Report on Structural Damage in Anchorage, Alaska Caused by the Earthquake of March 1927, 1964, January 1966.
A2	NVO-99-10	Report on Structural Damage in Lumberton and Dulce, New Mexico, Caused by the Earthquake of January 1922, 1966, March 1966.
A3	NVO-99-11	Report on Computer Program for Response Spectra, May 1966.
A4	NVO-99-12	Report on Structural Condition and Monitoring in Las Vegas, NTS, and Southern Nevada Area, May 1966.
A5	NVO-99-15	First Progress Report on Racking Tests of Wall Panels, August 1966.
A6	NVO-99-19	Structural Response Observations: Parkfield, California, Earthquake - June 27, 1966, September 1967.
A7	NVO-99-20	Structural Condition Survey: Movement of Building Element Cracks in Six Mercury, Nevada Structures, December 1967.
A8	NVO-99-24	Seiche and Tsunami Effects from Nuclear Excavation of an Interoceanic Canal, February 1968.
A9	NVO-99-25	1967 Colombian Earthquakes, May 1968.
A10	NVO-99-27	Spectral Response to Ground Displacement at Hattiesburg Resulting from Nuclear Event SALMON, March 1968.
A11	NVO-99-29	Concrete Test Structures: First Progress Report on Structural Response, March 1968.
A12	NVO-99-30	Dynamic Characteristics of Multistory Buildings, January 1968.
A13	NVO-99-31	Structural Dynamics of Cantilever-Type Buildings, January 1968.
A14	NVO-99-33	The Spectral Matrix Method of Damage Prediction: Description and Status, March 1968.
A15	JAB-99-35	Second Progress Report on Racking Tests of Wall Panels, July 1968.
A16	JAB-99-36	Three-Dimensional Dynamic Analysis of High-Rise Buildings, August 1968.

A17	JAB-99-37	Structural Effects of the Truckee, California, Earthquake of September 12, 1966, August 1968.
A18	VUF-1044 (JAB-99-38)	Structural Response: STERLING Event, October 1969.
A19	JAB-99-39	Stochastic Simulation of Site Response to Earthquake and Underground Explosions - Phase I, November 1968.
A20	JAB-99-40	A Direct-Reading Scaler for Measuring Recorded Wave Data, October 1968.
A21	PNE-1012 (JAB-99-42)	Project GASBUGGY: Final Report on Structural Response, February 1970.
A22	JAB-99-47	Motion Perception in the Low-Frequency Range, July 1969.
A23	JAB-99-48	Monitoring Study of Building Element Crack Movements, July 1971.
A24	JAB-99-49	Dynamic Response Characteristics of One-Story Test Structures, July 1972.
A25	JAB-99-50	Concrete Test Structures: Second Progress Report on Structural Response, July 1971.
A26	JAB-99-51	Structure Effects of Fairbanks, Alaska Earthquake of June 21, 1969, March 1970.
A27	JAB-99-54	Third Progress Report on Racking Tests of Wall Panels, November 1971.
A28	JAB-99-55	Fourth Progress Report on Racking Tests of Wall Panels, September 1974.
A29	JAB-99-61	Project RULISON: Pre-Shot Investigations and Structural Hazard Evaluations, August 1969 (Revised February 1971).
A30	JAB-99-63	Harvey Gap Dam Safety Study, May 1969.
A31	JAB-99-70	Approximate Procedure for Determining Building Dynamic Response Characteristics, October 1971.
A32	JAB-99-73	An Engineering Intensity Scale for Earthquakes and Other Ground Motion, September 1969.
A33	JAB-99-76	Structural Effects of the RULISON Event, December 1969.
A34	JAB-99-78	Structural Response Studies for Project RULISON, March 1971.
A35	JAB-99-81	The Spectral Matrix Method of Predicting Damage from Ground Motion, September 1971.
A36	JAB-99-87	Statistical Correlation of Observed Ground Motion with Low-Rise Building Damage: Project RULISON, September 1971.

- A37 JAB-99-88 Observations of Chimney Damage to Residential Buildings in Glendale, California: San Fernando Earthquake, February, 1971.
- A38 JAB-99-89 Observed Dynamic Response Characteristics of Residential Structures: Project TRINIDAD, May 1972.
- A39 JAB-99-90 Progress Report: Response of a Seven-Story Structure to the San Fernando Earthquake of February 199, 1971, June 1971.
- A40 JAB-99-91 Observations of Damage to Swimming Pools, Mobile Homes, and Glendale Commercial Buildings: San Fernando Earthquake, February 1971, October 1972.
- A41 JAB-99-92 Low-Rise Building Damage Caused by an Earthquake and an Underground Explosion, October 1973.
- A42 JAB-99-93 Statistical Correlation of Observed Ground Motion with Low-Rise Building Component Damage: Project RULISON, January 1973.
- A43 JAB-99-94 Influence of Nonstructural Partitions on the Dynamic Response Characteristics of Structures, July 1972.
- A44 JAB-99-96 Investigation of Ground Motion - Damage Relationships for Residential Buildings in Glendale, California: San Fernando Earthquake, February 9, 1971, August 1972.
- A45 JAB-99-97 Modification of the Colony Tower for Project RIO BLANCO, February 1974.
- A46 JAB-99-98 Response of Two Identical Seven-Story Structures to the San Fernando Earthquake of February 9, 1971, October 1973.
- A47 JAB-99-99 Buildings Modeled as Continuous Elastic Systems, October 1972.
- A48 JAB-99-100 Nonlinear Dynamic Analysis of Frames with Filler Panels, November 1972.
- A49 JAB-99-101 Production of Time Dependent Response and Band Pass Filter Spectra, September 1972.
- A50 JAB-99-103 Time Domain Analysis of Structure Motions, April 1973.
- A51 JAB-99-104 Dynamic Response Characteristics of 2-Story Test Structures, October 1974.
- A52 JAB-99-107 Survey of Bridges Subjected to Strong Ground Motion from Three Underground Nuclear Explosions, May 1972.
- A53 JAB-99-109 Dynamic Response Characteristics of Three One-Story Structures, October 1974.
- A54 JAB-99-110 Dynamic Response Reconciliation of Residential Structures: Project TRINIDAD, October 1974.

A55	JAB-99-111	Correlations Between Ground Motion and Building Damage (Engineering Intensity Scale Applied to the San Fernando Earthquake of February 1971), April 1977.
A56	JAB-99-112	Structural Effects Prediction Guidelines for Plowshare Projects, June 1974.
A57	JAB-99-114	Low-rise Building Soil-Structure Interaction Parameter Study, October 1974.
A58	JAB-99-115	Effects Prediction Guidelines for Structures Subjected to Ground Motion, July 1975.
A59	JAB-99-117	Low-Rise Building Damage from Low Amplitude Ground Motion, April 1974.
A60	JAB-99-118	Statistical Study on the Strength of Structural Materials and Elements July 1975.
A61	JAB-99-119	Vibration Tests of a 4-Story Reinforced Concrete Structure, January 1976.
A62	JAB-99-120	Probable Earthquake Ground Motion as Related to Structural Response in Las Vegas, January 1981.
A63	JAB-99-123	Nevada Test Site Terminal Waste Storage Program, Sub-task 1.3: Facility Hardening Studies Design Cost Scoping Study, April 1978.
A64	JAB-00099-124	Santa Barbara Earthquake of August 13, 1978, Field Report, August 1979.
A65	JAB-00099-126	Low-Rise Damage Data Analysis, December 1981.
A66	JAB-00099-128	Consideration for Developing Seismic Design Criteria for Terminal Waste Storage Repositories, April 1980.
A67	JAB-10145-1	Geology of the Las Vegas Area, May 1982.
A68	JAB-10145-2	Development of Damage Functions for High-Rise Building Components, October 1982.
A69	JAB-10145-3	Vibration Testing of an Epoxy-Repaired Reinforced Concrete Test Structure, June 1983.
A70	JAB-10343-3	Wind Response of a Tall Building with Full Scale Observations, February 1988.
A71	JAB-10343-TM2	Overview of Seismic Signal Processing Equipment and Procedures, January 1986.
A72	JAB-10733-1	Wind Response of a Tall Building with Full-Scale Observations, Phase II, August 1992
A73	JAB-10733-TM3	Overview of Seismic Signal Processing Operations, March 1991

- A74 JAB-10733-TM6 Processed Seismic Motion Records from Little Skull Mountain, Nevada, Earthquake of June 29, 1992, Recorded at Stations in Southern Nevada, December 1992
- A75 JAB-NSF-01 Project RIO BLANCO Low-Rise Damage Study, December 1975.
- A76 JAB-10733-TM7 Processed Seismic Motion Records from Desert Hot Springs, California, Earthquake of April 22, 1992, Recorded at Stations in Southern Nevada, April 1993
- A77 JAB-10733-TM8 Processed Seismic Motion Records from Little Skull Mountain, Nevada, Earthquake Aftershocks of July 5, 1992 and September 13, 1992, Recorded at Stations in Southern Nevada, April 1993
- A78 JAB-10733-TM9 Processed Seismic Motion Records from Landers, California, Earthquake of June 28, 1992, Recorded at Stations in Southern Nevada, April 1993
- A79 JAB-10733-TM10 Processed Seismic Motion Records from Big Bear, California, Earthquake of June 28, 1992, Recorded at Stations in Southern Nevada, April 1993
- A80 JAB-10733-TM11 Processed Seismic Motion Records from St. George, Utah, Earthquake of September 2, 1992, Recorded at Stations in Southern Nevada, April 1993.

APPENDIX B

SELECTED BLUME TECHNICAL JOURNAL PUBLICATIONS

REF.

NO. JOURNAL ARTICLES

- B1 Blume, John A., discussion of "Effect of Site Conditions on Earthquake Intensity" (John H. Wiggins, Jr.), Journal of the Structural Division, ASCE, December 1964.
- B2 Blume, John A., discussion of "Resonance Testing of Full-Scale Structures" (Donald B. Hudson), Journal of the Engineering Mechanics Division, ASCE, February 1965.
- B3 Blume, John A., discussion of "Vibrational Transfer from Shear Buildings to Ground" (Lydik S. Jacobsen), Journal of the Engineering Mechanics Division, ASCE, February 1965.
- B4 Blume, John A., discussion of "Probabilistic Design to Resist Earthquakes" (Emilio Rosenblueth), Journal of the Structural Mechanics Division, ASCE, June 1965.
- B5 Blume, John A., discussion of "Steady-State Versus Run-Down Tests of Structures" (N. Norby Nielsen), Journal of the Structural Division, ASCE, August 1965.
- B6 Blume, John A., "Dynamic Characteristics of Multistory Buildings," Journal of the Structural Division, ASCE, Vol. 94, ST2, February 1968.
- B7 Blume, John A., "Spectral Response to Ground Displacement at Hattiesburg Resulting From Nuclear Event SALMON," Bulletin of the Seismological Society of America, Vol. 59, No. 1, February 1969.
- B8 Blume, John A., Abstract, "Structural Response to Underground Nuclear Explosions," Transactions, American Geophysical Union, Vol. 50, No. 4, April 1969.
- B9 Blume, John A., discussion of "Inelastic Response of Tall Steel Frames," (Subhash C. Goel and Glen V. Berg), Journal of the Structural Division, ASCE, Vol. 95, ST6, June 1969.
- B10 Liu, S. C., and Dilip Jhaveri, "Spectral and Correlational Analysis of Ground-Motion Accelerograms," Bulletin of the Seismological Society of America, Vol. 59, No. 4, August 1969.
- B11 Liu, S. C., and Dilip Jhaveri, "Spectral Simulation and Earthquake Site Properties," Journal of the Engineering Mechanics Division, ASCE, Vol. 95, EM5, October 1969.
- B12 Blume, John A., "Response of High-Rise Buildings to Ground Motion From Underground Motion," Bulletin of the Seismological Society of America, Vol. 59, No. 6, December 1969.
- B13 Blume, John A., "An Engineering Intensity Scale for Earthquakes and Other Ground Motion," Bulletin of the Seismological Society of America, Vol. 60, No. 1, February 1970.

- B14 Blume, John A., "The Motion and Damping of Buildings Relative to Seismic Response Spectra," Bulletin of the Seismological Society of America, Vol. 60, No. 1, February 1970.
- B15 Blume, John A., "Building Columns Under Strong Earthquake Exposure," Journal of the Structural Division, ASCE, Vol. 97, ST9, September 1971.
- B16 Scholl, Roger E., "Dynamic Analysis of Three-Dimensional Frames," Journal of the Structural Division, ASCE, Vol. 98, St1, January 1972.
- B17 Blume, John A., "High-Rise Building Characteristics and Responses Determined from Nuclear Seismology," Bulletin of the Seismology Society of America, Vol. 62, No. 2, April 1972.
- B18 Nicoletti, Joseph P., and James M. Keith, "Structural Response to Seismic Motion Generated by Conventional Explosives," Military Engineer, September-October 1972.
- B19 Blume, John A., and Roger E. Skjei, "Structural Response to Seismic Motion Generated by Underground Nuclear Explosions," Military Engineer, January-February 1973.
- B20 Scholl, Roger E., and Iraj Farhoomand, "Statistical Correlation of Observed Ground Motion with Low-Rise Building Damage," Bulletin of the Seismological Society of America, Vol. 63, No. 5, October 1973.
- B21 Scholl, Roger E., "Statistical Analysis of Low-Rise Building Damage Caused by the San Fernando Earthquake," Bulletin of the Seismological Society of America, Vol. 64, No. 1, February 1974.
- B22 Kost, E. Garrison, "Nonlinear Dynamic Analysis of Frames with Filler Panels," Journal of the Structural Division, ASCE, April 1974.
- B23 Czarnecki, Robert M., "Dynamic Testing of Buildings Using Man-Induced Vibration," Sound and Vibration magazine, October 1974.
- B24 Scholl, Roger E., "Low-Rise Building Damage from Low-Amplitude Ground Motions," Bulletin of the Seismological Society of America, Vol. 64, No. 6, December 1974.
- B25 Blume, John A., S. A. Freeman, K. K. Honda, and L. A. Lee, "Modification of the Colony Tower for the RIO BLANCO Detonation," Nuclear Technology, Vol. 27, No. 4, December 1975.
- B26 Page, R. A., J. A. Blume, and W. B. Joyner, "Earthquake Shaking and Damage to Buildings," Science, Vol. 189, No. 4203, August 1975.

CONTRIBUTIONS TO COLLECTIONS

- B27 Blume, John A., "The Building Vibrator," Earthquake Investigations in the Western United States 1931-1964, U.S. Department of Commerce, Coast and Geodetic Survey, Publication 41-2, (also "Fore word").

- B28 Blume, John A., "Structural Response to Ground Motion," Chapter VII of Report NVO-28, Safety Involving Detonation of Nuclear Devices, U.S. Atomic Energy Commission, Nevada Operations Office, May 1966.
- B29 Blume, John A., "A Structural Dynamic Analysis of an Earthquake-Damaged Fourteen-Story Building, The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks, Vol. II, Part A, U.S. Department of Commerce, Environmental Science Services Administration, Publication 10-3, 1967.
- B30 Blume, John A., "Structural Response to Nuclear Induced Ground Motion," Chapter 10 of Report NVO-40 (Rev. 2), Technical Discussion of Offsite Safety Programs for Underground Nuclear Detonations, U.S. Atomic Energy Commission, Nevada Operations Office, May 1969.
- B31 "Ground Motion," Part d. of "Side effects from Cratering Explosions," IOCS Summary Report, Annex V, Appendix 3, Section C, U.S. Atomic Energy Commission, Nevada Operations Office.
- B32 "Ground Motion," Part 4 of IOCS Summary Report, Annex V, Appendix 12/13, Section F, U.S. Atomic Energy Commission, Nevada Operations Office.
- B33 Blume, John A., "Seismic Signal and Structural Response," Section 2:11, Education for the Peaceful Uses of Nuclear Explosives, Lynn E. Weaver, ed., The University of Arizona Press, 1970.
- B34 Blume, J. A., "On the Prediction of Building Damage from Ground Motion," Proceedings, Symposium on Earthquake Engineering with Nuclear Explosives, TID-4500, Vol. 2, American Nuclear Society, Hinsdale, Illinois, 1970.
- B35 Blume, John A., "Design of Earthquake-Resistant Poured-In-Place Concrete Structures," Chapter 18 of Earthquake-Engineering, Prentice-Hall, Inc., New Jersey, 1970.

PAPERS PRESENTED BEFORE TECHNICAL SOCIETIES

- B36 Blume, John A., "Soil-Structure Earthquake Design Considerations," Proceedings, Earthquake and Geologic Hazards Conference, The Resources Agency, State of California, 1965.
- B37 Czarnecki, R. M., "Dynamic Testing of Buildings Using Man-Induced Vibration," Sound and Vibration, Vol. 8., No 10, October 1974.
- B38 Blume, John A., "Earthquake Ground Motion and Engineering Procedures for Important Installations Near Active Faults," Proceedings, Third World Conference on Earthquake Engineering, New Zealand, 1965.
- B39 Blume, John A., "Remarks on Goals of Engineering Education," Proceedings, Fourteenth Annual Convention, California Council of Civil Engineers & Land Surveyors, 1966.

- B40 Blume, John A., "Comments on Structural Response to Earthquake Motion as Related to Damage Risk," Paper presented at U.S. Department of Commerce Meeting on Seismology and Engineering Seismology, Rockville, Maryland, January 1967.
- B41 Blume, John A., "Earthquake Resistance of Brick Masonry," Proceedings, 37th Annual Convention, Structural Engineers Association of California, October 1968.
- B42 Blume, John A., and Dilip Jhaveri, "Time-History Response of Buildings with Unusual Configurations," Proceedings, Fourth World Conference on Earthquake Engineering, Chile, 1969.
- B43 Blume, John A., "Structural Dynamics of Cantilever-Type Buildings," Proceedings, Fourth World Conference on Earthquake Engineering, Chile, 1969.
- B44 Blume, John A., "Ground Motion Effects," Proceedings, Symposium on Public Health Aspects of Peaceful Uses of Nuclear Explosives, Southwestern Radiological Health Laboratory, Las Vegas, Nevada, April 1969.
- B45 Blume, John A., "Behavior of Buildings Subjected to Ground Motion from Underground Nuclear Explosions," Proceedings, 38th Annual Convention, Structural Engineers Association of California, October 1969.
- B46 Lee, Lloyd A., and Roger E. Skjei, "Effects of RULISON Event on Buildings and Other Surface Structures," Vol. II, Proceedings, Symposium on Engineering with Nuclear Explosives, American Nuclear Society, January 1970.
- B47 Blume, John A., "On the Prediction of Building Damage from Ground Motion," Vol. II, Proceedings, Symposium on Engineering with Nuclear Explosives, American Nuclear Society, January 1970.
- B48 Blume, John A., "An Engineering Intensity Scale for Earthquakes and Other Ground Motions," Bulletin of the Seismological Society of America.
- B49 Freeman, S. A., "Structural Response Investigation of Full-Sized Test structures Subjected to Ground Motion Generated by Underground Nuclear Explosions," ASCE National structural Engineering Meeting, Portland, Oregon, April 1970.
- B50 Blume, John A., "High-Rise Building Characteristics and Responses Determined from Nuclear Seismology," American Nuclear Society/Seismological Society of America Joint Symposium at SSA Annual Meeting, University of California, Riverside, March 1971.
- B51 Blume, John A., and Roger Skjei, "Seismic Motion Thresholds for Damage to Structures," American Nuclear Society/Seismological Society of America Joint Symposium at SSA Annual Meeting, University of California, Riverside, March 1971.
- B52 Raggett, Jon D., "Structural Dynamic Effects Due to Wind Loading," ASCE Structural Division Annual Meeting, Baltimore, Maryland, April 1971.

- B53 Farhoomand, Iraj, and Roger E. Scholl, "Statistical Analysis of RULISON Low-Rise Building Damage," American Nuclear Society, Technical Group for Nuclear Explosive Engineering, Miami, Florida, October 1971.
- B54 Scholl, Roger E., "RULISON-Glendale Damage Statistics Comparison," National Conference on Earthquake Engineering, Los Angeles, California, February 1972.
- B55 Freeman, Sigmund A., "Response of Two Identical Seven-Story Structures to the San Fernando Earthquake of February 9, 1971," National Conference on Earthquake Engineering, Los Angeles, California, February 1972.
- B56 Blume, John A., "Civil Structures and Earthquake Safety," Proceedings of Conference on Earthquake Risk, September 22-24, 1971, Joint Committee on Seismic Safety of the California Legislature (published 1972).
- B57 Blume, John A., "Structural Dynamic Theory," section of Report NVO-118, Proceedings, of the Structural Response Seminar, August 25, 1971, U.S. Atomic Energy Commission, Nevada Operations Office, August 1972.
- B58 Scholl, Roger E., "Nature of Damage Prediction for Low-Rise Buildings," section of Report NVO-118, Proceedings of the Structural Response Seminar, August 25, 1971, U.S. Atomic Energy Commission, Nevada Operations Office, August 1972.
- B59 Raggett, Jon D., "Influence of Nonstructural Partitions on Structural Dynamic Response Characteristics," section of Report NVO-118, Proceedings of the Structural Response Seminar, August 1971.
- B60 Farhoomand, Iraj, "Statistical Analysis of RULISON Low-Rise Building Damage," section of Report NVO-118, Proceedings of the Structural Response Seminar, August 25, 1971, U.S. Atomic Energy Commission, Nevada Operations Office, August, 1972.
- B61 Monroe, Robert E., "Spectral Matrix Method of Damage Prediction," section of Report NVO-118, Proceedings of the Structural Response Seminar, August 25, 1971, U.S. Atomic Energy Commission, Nevada Operations Office, August 1972.
- B62 Blume, John A., "Analysis of Dynamic Earthquake Response," State-of-the-Art Report No. 3, Technical Committee 6, Earthquake Loading and Response, Proceedings of the ASCE-IABSE International Conference on the Planning and Design of Tall Buildings, August 1972.
- B63 Freeman, Sigmund A., "Comparisons of Results of Dynamic Seismic Analyses of Two Identical Structures Located at Two Different Sites, Based on Site Seismograms from the San Fernando Earthquake," Proceedings of the 41st Annual Convention of the Structural Engineers' Association of California in October 1972.
- B64 Blume, John A., "Probability of Earthquakes and Resultant Ground Motion," Proceedings of the World Mining International Tailing Symposium, Tucson, Arizona, November 1972.

- B65 Blume, John A., and Roger E. Scholl, "Damage Prediction for Low-Rise Buildings," Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, June 1973.
- B66 Blume, John A., "Elements of a Dynamic-Inelastic Design Code," Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, June 1973.
- B67 Blume, John A., and Sigmund A. Freeman, "Structural Response to Man-Induced Ground Motion," Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, June 1973.
- B68 Blume, John A., Lloyd A. Lee, Sigmund A. Freeman, "Modification of the Colony Tower for the RIO BLANCO Detonation," American Nuclear Society, San Francisco, November 1973.
- B69 Scholl, Roger E., "Statistical Description of Observed Low-Rise Building Damage for Low Amplitude Ground Motions," Seismological Society of America Conference, Las Vegas, Nevada, March 1974.
- B70 Freeman, Sigmund A., and Roger E. Scholl, "Structural Response Investigation of Reinforced Concrete Frame Structures Subject to High Magnitude Lateral Forces," Research Session of ACI Committee 115, American Concrete Institute annual Convention, San Francisco, April 1974.
- B71 Raggett, Jon D., "Time Domain Analysis of Structure Motions," ASCE Structural Engineering Convention, Cincinnati, Ohio, April 1974.
- B72 Reed, John W., "Motion Perception and Discomfort in Tall Buildings," ASCE Structures Division Meeting, Cincinnati, Ohio, April 1974.
- B73 Blume, John A., and John W. Reed, "Predicting Damage Probabilistically for Buildings Subjected to Ground Motions," ASCE Conference on Probabilistic Methods in Engineering, Stanford University, Stanford, California, June 1974.
- B74 Raggett, Jon D., "Estimating Damping of Real Structures," ASCE annual meeting, Structural Division, Kansas City, October 1974.
- B75 Blume, John A., "San Francisco Seismic Safety Element Study - Geology and Structures," Engineering Foundation, Earthquakes and Lifelines Conference, Asilomar, California, December 1974.
- B76 Blume, John A., R. M. Czarnecki, S. A. Freeman, and R. E. Scholl, "Four-Story Test Structure Destructive Test," ACI Conference, Boston, April 1975.
- B77 Freeman S. A., "Racking Tests of High-Rise Building Partitions, meeting Preprint 2462, ASCE National Structural Engineering Convention, New Orleans, April 1975.
- B78 Freeman, S. A., J. P. Nicoletti, and J. V. Tyrrell, "Evaluation of Existing Buildings for Seismic Risk-- A case Study of Puget Sound Naval Shipyard, Bremerton, Washington," EERI National Earthquake Conference, Ann Arbor, Michigan, June 1975.
- B79 Czarnecki, R.M., S. A. Freeman, Roger E. Scholl, "Destructive Tests of a 4-Story Concrete Structure," Fifth European Conference on Earthquake Engineering, Istanbul, September 1975.

- B80 Scholl, Roger E., "Four-Story Test Structure Destructive Test," Structural Engineers' Association of Central California Meeting, Sacramento, September 1975.
- B81 Freeman, S. A., "Session 1A, Introduction to New Features of the SEAONC Blue Book," Seminar Papers, New Earthquake Design Provisions, ASCE/SEAONC Professional Development Committee, October-November 1975.
- B82 Chen, C. K., and S. A. Freeman, "Dynamic Response of Steel Industrial Storage Racks," International Specialty Conference on Cold-Formed steel Structures, St. Louis, November 1975.
- B83 Nicoletti, J. P., and S. A. Freeman, "Session II: Methods for Analyzing/Designing for Structural Response," CERL Design Seminar, Champaign, Illinois, for Publication in 1976.
- B84 Freeman, S. A., "Testing Wall Panels for Earthquake Response," ASCE/EMD Specialty Conference Dynamic Response of Structures, University of California, Los Angeles, March 1976.
- B85 Freeman, S. A., C. K. Chen, and R. M. Czarnecki, "Dynamic Response Characteristics of Reinforced Concrete Structures," ASCE/EMD Specialty Conference Dynamic Response of Structures, University of California, Los Angeles, March 1976.
- B86 Honda, K. K., "Measurements and Evaluation of High-Rise Building Response to Ground Motion Generated by Underground Nuclear Explosions," ASCE/EMD Specialty Conference Dynamic Response of structures, University of California, Los Angeles, March 1976.
- B87 Honda, K. K., "Measurement and Evaluation of Building Response to Ground Motion at Various Stages of Construction," National Structural Engineering Conference Proceedings, ASCE, Madison, Wisconsin, August 1976.

OTHER

- B88 URS/John A. Blume & Associates, Engineers, Engineering Studies on Blasting Excavation for Excavation, Bankhead Project, Black Warrior River, Alabama, prepared for mobile District, Corps of Engineers, U. S. Army, June 1975.

APPENDIX C

SELECTED USC&GS-NOAA-USGS SPECIAL PROJECTS PARTY REPORTS

REF. NO.	REPORT NUMBER	TITLE
C1	NP-13867	Seismic Data Summary, Nuclear Detonation Program - 1961 through 1963, January 1964.
C2	CGS-746-2	RULISON Seismic Effects. King, Kenneth W. (Coast and Geodetic Survey, Las Vegas, NV. Special Projects Party.) February 1970, (PNE-R-23)
C3	CGS-746-5	Seismic Activity in September 1969 Near the RULISON Nuclear Test Site. Navarro, R.; Brockman, S. R. (Coast and Geodetic Survey, Las Vegas, NV. Special Projects Party.) April 1970.
C4	CGS-746-7	Seismic Station Data HANDLEY Event. (Coast and Geodetic Survey, Las Vegas, NV. Special Projects Party.) April 1970.
C5	CGS-746-9	Project TRINIDAD delta series Numbers 1, 2, and 3 Sopris, Colorado. King, Kenneth W. (National Ocean Survey, Rockville, Md.) December 1970.
C6	CGS-746-12	Earthquakes on and Around the Nevada Test Site 1950-1971. King, Kenneth W.; Bayer, Kenneth C.; Brockman, Stanley R., August 1971.
C7	CGS-C-106	U.S. Coast and Geodetic Survey Seismograph Systems. Navarro, Richard. June 1967.
C8	CGS-E-101	Event STERLING Seismic Report CGS-E-101, Hattiesburg, Mississippi. January 1967.
C9	CGS-P-117	Investigation and Instrumentation of Panama and Colombia Strong-Motion Seismographs. September 1967.
C10	USGS-474-182	Catalog of Seismic Stations Operated in Support of the ERDA/Nevada Operations Office. January 1964 through June 1976, May 1977 (USGS Open File Report 77-404).
C11	USGS-474-183	Catalog of Seismic Records Obtained in support of the ERDA/Nevada Operations Office, October 1963 through June 1976, September 1977 (USGS Open File Report 77-653)
C12	USGS Bull 1279	Use of Seismic Intensity Data to Predict the Effects of Earthquakes and Underground Nuclear Explosions in Various Geological Settings, 1969.

- C13 ERL 235-ESL 20 Recorded and Processed Seismic data from the 1971 San Fernando Earthquake and Aftershocks by the Special Projects Party, NOAA Technical Report, June 1972.
- C14 NVO-746-TM3 Earthquakes Record by a Seismograph Network Located in the Southern Nevada Region, January 1 to December 22, 1971, May 1972.
- C15 NVO-746-2 Navarro, R., and G. M. Wuollett, L-7 Velocity Seismograph Shaking Table Results, National Oceanic and Atmospheric Administration, Earth Sciences Laboratory, 1972.
- C16 NVO-746-3 Seismicity of the Southern Nevada Region, December 22, 1971 to July 1, 1972, July 1972.
- C17 NVO-746-4 A Preliminary Seismicity Study of the Southern Nevada Region for the Month of July 1972, September 1972.
- C18 NVO-746-5 A Preliminary Seismicity Study of the Southern Nevada Region for the Month of August 1972, October 1972.
- C19 NVO-746-7 A Preliminary Seismicity Study of the Southern Nevada Region for the Month of September 1972, November 1972.
- C20 NVO-746-8 A Preliminary Seismicity study of the Southern Nevada Region for the Month of October 1972, December 1972.
- C21 NVO-746-9 A Preliminary Seismicity study of the Southern Nevada Region for the Month of November 1972, January 1973.
- C22 PNE-RB-53 RIO BLANCO Seismic Effects.
- C23 NVO-474-2 Structural and Free Field Measurements of Zonia Mine Blast, Arizona, Navarro, R., February 1974.

APPENDIX D
ERC REPORTS AND
SELECTED TECHNICAL JOURNAL PUBLICATIONS

REF. NO.	REPORT NUMBER	TITLE
D1	NVO-1163-10R	Summary Report of Predictions - DRIBBLE Series, Revised, June 1964.
D2	NVO-1163-34	Geophysics Analysis System for Seismogram Analysis, IBM 7090 Programs, November 1964.
D3	NVO-1163-38	Analysis and Interpretation of Squared Fourier Spectra, January 1965.
D4	NVO-1163-41	Analog Computer Analysis of Magnetic Tape Seismograms, January 1965.
D5	NVO-1163-49R	A Summary of Ground Motion Prediction Procedures and Equations, Revision 1, March 1965.
D6	NVO-1163-51	Investigation of SALMON Ground Motions and Effects, Final Report, April 1965.
D7	VUF-1026 (NVO-1163-59)	Analysis of Ground Motion and Containment, SALMON Event, Project DRIBBLE, Final Report, August 1965.
D8	NVO-1163-63	Interim Report - Pre-SCHOONER II, October 1965.
D9	NVO-1163-64	Diffusion of Seismic Energy in the Near Range, November, 1965.
D10	NVO-1163-67	Analog Computer Study of Building Response to Ground Motion, March 1967.
D11	NVO-1163-80	Magnification of Love Waves, October 1967.
D12	NVO-1163-86	Signal Analysis Using Auto- and Cross-Correlation Techniques and Associated Processes, Final Report, February 1967.
D13	NVO-1163-88	A Statistical Relationship Between Band-Pass Filter and Pseudo-Relative Velocity Spectra, August 1967.
D14	NVO-1163-98	Ray Theory Seismic Pulse (RATS Pulse) Mathematics Report, January 1967.
D15	NVO-1163-105	Ray Theory Seismic Pulse (RATS Pulse) - Program Report, March 1967.
D16	VUF-1035 (NVO-1163-107)	Analysis of Ground Motion and Containment, STERLING Event, January 1968.

D17	NVO-1163-115	Environmental Research Corporation - Procedure for NC-21 Instrument Correction, June 1967.
D18	NVO-1163-130	Amplification of Seismic Body Waves by Low-Velocity Surface Layers, December 1967.
D19	NVO-1163-138	Effect of Ground Motion on Sedan Crater Slopes from an Underground Nuclear Explosion in the Vicinity, May 1969.
D20	NVO-1163-141	On the Duration of Seismic Pulses, June 1968.
D21	NVO-1163-156	An Application of Non-Stationary Processes to Generate Synthetic Seismograms, December 1968.
D22	NVO-1163-157	Identification of Elastic Wave Types on Seismograms from Underground Nuclear Explosions, October 1968.
D23	NVO-1163-165	Models for Calculating Close-In Motions from Underground Nuclear Explosions, March 1969.
D24	NVO-1163-175	Amplification of Rayleigh Waves in a Surface Layer of Variable Thickness, February 1969.
D25	NVO-1163-180	Predictions of Seismic Motion and Close-In Effects - RULISON Event, August 1969.
D26	NVO-1163-181	Amplitude and Frequency Characteristics of Elastic Wave Types Generated by Underground Nuclear Explosions, April 1969.
D27	NVO-1163-195	Seismic Spectrum Scaling of Underground Detonations, November 1969.
D28	NVO-1163-197	Observed Seismic Data - RULISON Event, November 1969.
D29	NVO-1163-199	Observed Seismic Data - MILROW Event, December 1969.
D30	NVO-1163-202	Elastic Wave Propagation in Heterogeneous Media, March 1970.
D31	NVO-1163-203	Seismic Response Characteristics at Las Vegas, Nevada from Underground Nuclear Detonations, July 1970.
D32	NVO-1163-204	Seismic Response Characteristics at Tonopah, Nevada from Underground Nuclear Detonations, June 1970.
D33	NVO-1163-205	Effects of Local Geology on the Amplitude of Seismic Waves, June 1970.
D34	NVO-1163-206	Analysis of Ground Motions and Close-in Physical Effects, RULISON Event, April 1970.

D35	NVO-1163-207	Observed Seismic Data, JORUM Event, May 1970.
D36	NVO-1163-208	Elastic Energy and Magnitude Determinations for Underground Detonations in Tuff and Rhyolite, May 1970.
D37	NVO-1163-209	Analysis of Seismic Data, MILROW Event, May 1970.
D38	NVO-1163-210	Observed Seismic Data, PIPKIN Event, May 1970.
D39	NVO-1163-211	An Analysis of the Applicability of Existing Analytic Models for the Description of Observed Seismic Amplification Effects, September 1970.
D40	NVO-1163-216	Observed Seismic Data, HANDLEY Event, May 1970.
D41	NVO-1163-217	Models for Generating Synthetic Seismograms, August 1970.
D42	NVO-1163-218	Observed Seismic Data, CALABASH Event, February 1971.
D43	NVO-1163-225	Calculated Compressional Wave Arrivals from Underground Nuclear Detonations, August 1971.
D44	NVO-1163-227	Preliminary Considerations of Ground Motion from a Vertically Oriented Multiple Detonation in a Half Space, November 1971.
D45	NVO-1163-228	Predictions of Ground Motion - RIO BLANCO Event, November 1971.
D46	NVO-1163-229	Ground Motions Recorded in Las Vegas from Seven Underground Nuclear Events, November 1971.
D47	NVO-1163-230	Observed Ground Motion Data, CANNIKIN Event, December 1971.
D48	NVO-1163-231	Observed Seismic Data, MINIATA Event, A Plowshare Development Test, February 1972.
D49	NVO-1163-232	Analysis of CANNIKIN Seismic Data, June 1972.
D50	NVO-1163-233	Observed Seismic Data, FLASK Event, March 1971.
D51	NVO-1163-235	Inelastic/Elastic Calculations of Ground Motion and Physical Effects from Nuclear Detonations in Tuff, January 1973.
D52	NVO-1163-237	Topographic Effects of Ground Motion for Incident P Waves: A Model Study, May 1973.
D53	NVO-1163-238	Raleigh Waves from a Nuclear Source in a Half-Space, May 1973.

D54	NVO-1163-239	Predicted Ground Motion Characteristics of Underground Nuclear Detonations, March 1974.
D55	NVO-1163-240	Observed Seismic Data RIO BLANCO Event, March 1974.
D56	NVO-1163-241	Inelastic Calculations in Tuff and Spectrum Scaling Considerations, March 1974.
D57	NVO-1163-242	Amplitude and Propagation characteristics of Short-Period Seismic Surface Waves in Las Vegas Valley, March 1974.
D58	NVO-1163-244	Observed Seismic Data ALMENDRO Event, March 1974.
D59	NVO-1163-TM8	PSRV Prediction Equations for Pahute Mesa Events, May 1969.
D60	NVO-1163-TM11	One-Dimensional Earth Motion Calculations, May 1970.
D61	NVO-1163-TM15	The Cavity Formed by a Contained Underground Nuclear Detonation, February 1970.
D62	NVO-1163-TM16R	Basic Aspects of BPF, PSRV, and Fourier Amplitude Spectra, Revised, January 1972.
D63	NVO-1163-TM18	Methods to Predict Ground Motions from Future Underground Nuclear Detonations in the Piceance Creek Basin, Colorado, January 1970.
D64	NVO-1163-TM20	Analysis of Selected Effects on Peak Ground Motion Generated by Underground Nuclear Detonations, June 1970.
D65	NVO-1163-TM22	Seismic Response Characteristics at NRDS from Underground Nuclear Detonations, December 1970.
D66	NVO-1163-TM23	Photo Interpretation of Close-In Physical Effects of an Underground Nuclear Detonation, September 1970.
D67	NVO-1163-TM24	Covariance Analysis of Response Spectra, May 1971.
D68	NVO-1163-TM25	Variability of Ground Motion Models, The Transmission Path, April 1971.
D69	NVO-1163-TM28	Basic Elements of Two-Dimensional Wave Propagation in Homogeneous, Isotropic Layered Elastic Media, August 1971.
D70	NVO-1163-TM30	Predicted San Fernando Earthquake Spectra - Glendale Area, August 1971.
D71	NVO-1163-TM32	Comparison of Response Spectra from Yucca Flat and Pahute Mesa Underground Nuclear Detonations, June 1972.

- D72 NVO-1163-TM33 Analysis of Seismic Waves Recorded in Las Vegas, Nevada, From Underground Nuclear Detonations, June 1972.
- D73 NVO-1163-TM34 The Question of Ground Motion Duration for Nuclear Detonations and Earthquakes, July 1972.
- D74 NVO-1163-TM37 Time Dependent Spectral Analysis of Ground Motion, March 1973.
- D75 NVO-1163-TM38 Distance Attenuation of Response Spectra from Underground Nuclear Detonations, July 1973.
- D76 NVO-1163-TM40 Effect of Fractured Zone on Ground Motions, February 1974.
- D77 NVO-1163-TM41 An Analysis of Surface and Subsurface Seismic Measurements Demonstrating the Effects of Near Surface Geology, March 1974.
- D78 NVO-0624-1 Observed Ground Motion data from Pahute Mesa Events Detonated between May 1975 and March 1976, March 1979 (CSC Report)

SELECTED ERC TECHNICAL JOURNAL PUBLICATIONS

REF. NO.	REPORT NUMBER	TITLE
D79	PNE 1010	Analysis of Ground Motion and Close-in Physical Effects, GASBUGGY Event; Foote, R. Q., Hays, W. W., and Klepinger, R. A., October 1969.
D80		Smith, S. B., "Plowshare Comes of Age", <u>The Military Engineer</u> , July-August, 1969.
D81		Cassidy, C. R., "The Complete Solution of the Fifth-Order Runge-Kutta Equations", <u>Journal of Numerical Analysis</u> , Vol. 6, September, 1969.
D82		Hayes, W. W., "Amplitude and Frequency Characteristics of Elastic Wave Types Generated by the Underground Nuclear Detonation, BOXCAR", <u>Bulletin of the Seismological Society of America</u> , December, 1969.
D83		Lynch, R. D., "Response Spectra for Pahute Mesa Nuclear Events", <u>Bulletin of the Seismological Society of America</u> , December, 1969.
D84		Mueller, R. A., "Seismic Energy Efficiency of Underground Nuclear Detonations", <u>Bulletin of the Seismological Society of America</u> , December, 1969.
D85		Murphy, J. R., "Discussion of Sterling Experiment: A Coupling of Seismic Waves by a Shot-generated Cavity", <u>Journal of Geophysical Research</u> , December, 1969.

- D86 Murphy, J. R., and J. A. Lahoud, "Analysis of Seismic Peak Amplitudes from underground Nuclear Explosions", Bulletin of the Seismological Society of America, December, 1969.
- D87 Beaudet, P. R., "A Non-Stationary Procedure to generate Synthetic Seismograms", Bulletin of the Seismological Society of America, October, 1970.
- D88 Beaudet, P. R., "Digital Filters for Response Spectra", Bulletin of the Seismological Society of America, June, 1970,
- D89 Beaudet, P. R., "Elastic Wave Propagation in Heterogeneous Media", Bulletin of the Seismological Society of America, June, 1970.
- D90 Murphy, J. R., et al, "Amplification of Seismic Body Waves by Low Velocity Surface Layers", Bulletin of the Seismological Society of America, Vol. 61, No1, February, 1971.
- D91 Loux, P. C., "Ground Motions Predictions", Proceedings: Symposium on Public Health Aspects of the Peaceful Uses of Nuclear Explosives, April, 1969.
- D92 Mueller, R. A., "Predictions of Seismic Motion from Contained and Excavation Nuclear Detonations", Proceedings: Symposium on Engineering with Nuclear Explosives, CONF-700101, May, 1970.
- D93 Loux, P. C. "Seismic Motions from Project RULISON", Proceedings: Symposium on Engineering with Nuclear Explosives, CONF-700101, May, 1970.
- D94 Hakala, W. W., "Subsidence Caused by an Underground Nuclear Explosion", Proceedings: Symposium on Engineering with Nuclear Explosives, CONF-700101, May, 1970.
- D95 Hayes, W. W., and J. R. Murphy, "The Effect of Yucca Fault on Seismic Wave Propagation", Bulletin of the Seismological Society of America, June, 1971.
- D96 Mueller, R. A., and J. R. Murphy, "Seismic Characteristics of Underground Nuclear Detonations, Part I; Seismic Spectrum Scaling", Bulletin of the Seismological Society of America, December, 1971.
- D97 Mueller, R. A., and J. R. Murphy, "Seismic Characteristics of Underground Nuclear Detonations, Part II; Seismic Spectrum Scaling", Bulletin of the Seismological Society of America, December, 1971.
- D98 Hayes, W. W., "Examples of the Effects of Source and Recording Site Parameters on the Seismic Response Observed from Plowshare Projects, GASBUGGY and RULISON", Geophysics, 1972

- D99 Orphal, D. L., "Seismic Motion Recorded from Milrow Detonation in the Distance Range 7-377 Kilometers", Bulletin of the Seismological Society of America, October, 1971.
- D100 Hayes, W. W., "A Review of Ground Motion Predictions for Plowshare Underground Engineering Applications", Proceedings, American Nuclear Society, October, 1971.
- D101 Hayes, W. W., "Comparison of Ground Motion from the San Fernando Earthquake and the RULISON Nuclear Detonation", Proceedings, National Conference of Earthquake Engineering, February, 1972.
- D102 Davis, L. L., and L. R. West, "Preliminary Evaluation of the Effects of Topography on Ground Motion", Bulletin of the Seismological Society of America, 1973.
- D103 Murphy, J. R., "Calculated Compression-Wave Arrivals from Underground Nuclear Detonations", Bulletin of the Seismological Society of America, 1972.
- D104 Perret, W. R., "Seismic Source Energies of Underground Nuclear Explosions", Bulletin of the Seismological Society of America, 1972.

APPENDIX E
SELECTED DOE/NV REPORTS

REF. NO.	REPORT NUMBER	TITLE
E1	NVO-20 (Revision 1)	Nevada Operations Office Technical Information Guide for Contractors. (Nevada Operations Office (AEC), Las Vegas, NV), October 1971.
E2	NVO-22 (Revision 1)	Study Related to Reports of Damage from the SALMON Event, Project Dribble, (Holmes & Narver, Inc., Las Vegas, NV, On-Continent Test Div.), March 1967.
E3	NVO-28	Safety Involving Detonation of Nuclear Devices. (Reynolds Electrical & Engineering Co., Inc., Mercury, NV, Management Engineering Dept.) May 1966.
E4	NVO-40 (Revision 2)	Technical Discussions of Offsite safety Programs for Underground Nuclear Detonations. (Nevada Operations Office (AEC), Las Vegas, NV), May 1969.
E5	NVO-43	Effects Evaluation for Project RULISON. (Nevada Operations Office (AEC), Las Vegas.) June 1969.
E6	NVO-48	Reviews to Insure Safety of Nuclear Operations. (Nevada Operations Office (AEC), Las Vegas.) July 1969.
E7	NVO-61	Project RULISON: Post-shot Plans and Evaluations. (Nevada Operations Office (AEC), Las Vegas.) December 1969.
E8	NVO-87	Seismology, Aftershocks, and Related Phenomena Associated with Under Ground Nuclear Explosions: A Bibliography of Selected Papers with Abstracts. (Nevada Operations Office (AEC), Las Vegas.) May 1971.
E9	NVO-118	Proceedings of the Structural Response Seminar, October 1971.
E10	DOE/NV 209	Announced United States Nuclear Tests, July 1945 through December 1989, Revision 10, April 1990 (Updated Annually)

DISTRIBUTION LIST: JAB-10733-2

U.S. Department of Energy
Nevada Operations Office
P.O. Box 98518
Las Vegas, NV 89193-8518

Richard Navarro
J.E. O'Donnell
Technical Information
Resource Center

Lawrence Livermore National Laboratory
University of California
P.O. Box 808
Livermore, CA 94590

F.J. Tokarz
Technical Library

U.S. Department of Energy
Office of Scientific and
Technical Information
P.O. Box 62
Oak Ridge, TN 37831

(2 Copies)

Sandia National Laboratories
P.O. Box 5800
Albuquerque, NM 87185

J.R. Banister
W.D. Weart - 6340
J.S. Phillips - 9514
Technical Library - 3141

Los Alamos National Laboratory
University of New Mexico
P.O. Box 1663
Las Alamos, NM 87545

K.H. Olsen
Technical Library

Defense Nuclear Agency
Las Vegas, NV

J.W. LaComb, M/S 573

END

**DATE
FILMED**

12/02/93