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TESTING OF SHELTER DESIGN AND INDUSTRIAL HARDENING CONCEPTS AT --ETC(U)
JAN 82 R S TANSLEY, J V ZACCOR EMW-C-0611

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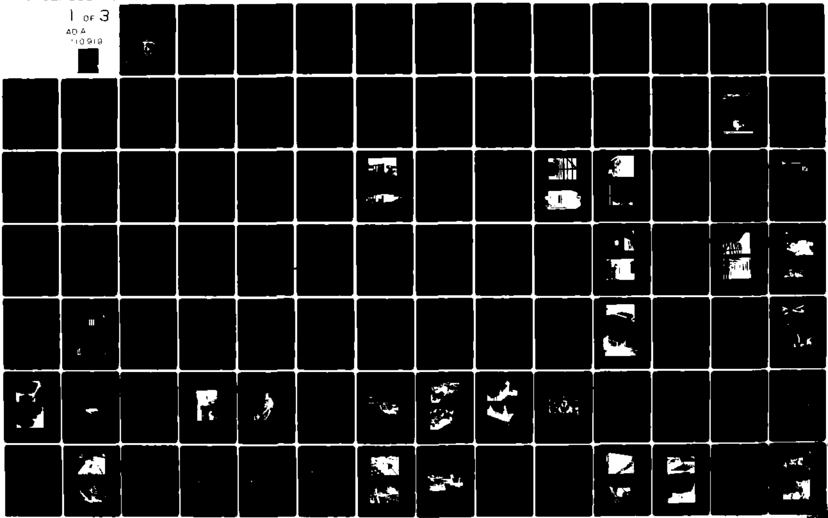

SSI 8115-4

January 1982

Testing of Shelter Design and Industrial Hardening Concepts at the Mill Race Event

FINAL REPORT

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Contract No. EMW-C-0611
Work Unit 1128D

SCIENTIFIC SERVICE, INC.

SSI Testing of Shelter Design and Industrial Hardening Concepts

REPORT DOCUMENTATION PAGE

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		key work shelters
MILL RACE	industrial hardening	expedient shelters

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The report presents the results of the experiments conducted for the Federal Emergency Management Agency at the MILL RACE high explosive test. on September 16, 1981 at White Sands Missile Range. The MILL RACE event provide a simulated nuclear weapon airblast and ground motion environment to demonstrate, using full-scale structures and test objects, the validity and practicality of a number of blast upgrading and industrial hardening concepts that will support current Civil Defense planning. The design, —

Block 20.

construction, and objectives of each of the experiments are described and illustrated, and the test data and conclusions are presented. The experiments included host area and key worker shelters, expedient shelters, and industrial hardening experiments at the 2, 20, and 40 psi peak overpressure levels.

(DETACHABLE SUMMARY)

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by

R.S. Tansley and J.V. Zaccor

for

Federal Emergency Management Agency
Washington, D.C. 20472

Contract No. EMW-C-0611, Work Unit 1128D

FEMA REVIEW NOTICE:

This report has been reviewed in the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency.

Scientific Service, Inc.
517 East Bayshore, Redwood City, CA 94063

(Detachable Summary)

TESTING OF SHELTER DESIGN AND INDUSTRIAL HARDENING CONCEPTS AT THE MILL RACE EVENT

The following is a summary of the report prepared by Scientific Service, Inc. (SSI) on the experiments conducted at MILL RACE under the sponsorship of the Federal Emergency Management Agency (FEMA).

The MILL RACE event was a high explosive test conducted on September 16, 1981 at the White Sands Missile Range in New Mexico. This test was conducted by the Defense Nuclear Agency (DNA) and consisted of the detonation of 600 tons of an ammonium nitrate-fuel oil (ANFO) mixture that provided a simulated nuclear weapon airblast and ground motion environment for experiments conducted by various government agencies. SSI, under the sponsorship of FEMA, designed and conducted experiments in upgrading existing structures for both host area and key worker shelters, industrial hardening, and expedient shelter development. The objective of these experiments was to demonstrate, using full-scale structures and test objects, the validity and practicality of shelter upgrading and industrial hardening concepts in support of crisis relocation planning.

The design, construction, and objectives of each of the experiments are described and illustrated, and the test data and conclusions for each are presented and discussed. Instrumentation included load cells, pressure gauges, displacement transducers, soil pressure gauges, still and cine photography, and anthropomorphic dummies. The limitations imposed by the short duration of the blast effects from this event were recognized, and the resulting test data were analyzed accordingly and are believed to be consistent with the expected performance with increased weapon size. Following is a brief description of the experiments, the test results, and the conclusions.

Host Area Shelters - Three buildings, one wood frame and one masonry constructed on grade, and one masonry basement structure constructed below grade, were located at 2 psi peak overpressure. Both buildings on grade and one half of the below-grade structure were shored to survive this blast loading, with earth cover and berms added after shoring for simulated radiation protection. The shored areas survived with little or no damage, while the unshored portion of the floor in the basement structure collapsed. It is of interest to note that this unshored portion of the floor partially failed prior to the blast loading solely because of the earth cover. The performance of the roof and floor shoring was consistent with current prediction methodology. Considerable data were developed on wall upgrading, closures, and resource expenditures (labor and materials required to accomplish upgrading), much of it pointing to a need for further research.

Key Worker Shelter - One below-grade basement structure, constructed with the floor above composed of three different types of concrete construction (flat slab, two-way slab, and prestressed precast slabs) was located at 40 psi. In order to obtain maximum data, these floor constructions (which also had earth cover) were shored to survive various blast loadings using different shoring configurations and materials. Six test walls were also incorporated into the experiment, two unreinforced masonry, two reinforced masonry and two precast concrete. The portions of the floor upgraded to 40 psi, with the exception of the prestressed concrete slabs, exhibited little damage, as anticipated. Those portions of the floor that were either shored to less than 40 psi or left unshored collapsed to varying degrees under the blast loading. The six test walls remained in place with only two (precast concrete) exhibiting any cracking. The various floor shoring tested as anticipated, and valuable data were acquired showing that some types required considerable expenditure of upgrading resources. Further investigation is required in the areas of closures and soil/structure interaction.

Industrial Hardening - The industrial hardening experiments investigated two areas: the use of buried concrete vaults as expedient shelters, and the protection of industrial equipment and machinery, secured and hardened by various methods. The vaults, one at 20 psi and one at 40 psi, sustained no damage. The industrial hardening experiments, all at 20 psi, consisted of stabilization, securing, and/or other schemes to protect lightweight machine tools and steel drums, and provided valuable information that has immediate application. The damage that some of the tools sustained was the result of impact, not pressure; elimination of the opportunity for these tools to slide, overturn, or tumble would eliminate most damage. The stabilization techniques used on the steel drums provided significant data towards an alternative to anchoring; the techniques developed, however, do not lend themselves to extrapolation to other items without further investigation and testing.

Key Worker Expedient Shelter - A below-grade shelter constructed of dimension lumber was placed at 40 psi and covered over with earth. The shelter performed adequately and exhibited no structural distress. The labor and materials required to construct this shelter were considerable, and accordingly, further research on these types of shelters in the area of dynamic loading of soils is needed so that a more accurate and sophisticated design methodology may be developed.

In general, all of the experiments conducted by SSI at MILL RACE provided excellent data. Areas the need additional investigation and/or require a more definitive or an expanded test program in the future were clearly pointed out. The upgrading of both the host area and key worker shelters verified existing prediction data for floors and roofs, but indicated that further investigation of walls, closures, resource requirements, structural connections, and soil interaction is needed. The expedient shelter experiments utilizing the concrete vaults were favorable, while the dimension lumber shelter test indicated that further testing and investigation are required. Valuable information was obtained from the industrial hardening experiments, much of it having immediate application, but also pointing out some areas requiring future exploration.

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Scientific Service, Inc.
517 East Bayshore, Redwood City, CA 94063

ACKNOWLEDGEMENTS

This report describes experiments conducted by Scientific Service, Inc. for the Federal Emergency Management Agency at the Defence Nuclear Agency's MILL RACE event, September 16, 1981, at White Sands Missile Range, New Mexico. The authors wish to take this opportunity to thank all those involved in the project.

John R. Lavis General Contractor, Inc., El Paso, Texas, constructed the experiments; particular thanks are due Steve Brunson, their job superintendent, whose cooperation and assistance were invaluable to the success of the project. Los Alamos Technical Associates, Inc. personnel, primarily Bill Meyers, assisted with liaison and construction inspection.

Special mention should be made of the Field Command, DNA personnel - Lieutenant Commander Gary Reid, Lieutenant Roy Garcia, Captain Fred Jaeger, and Mr. Jim Wilson - for their cooperation, assistance, and patience.

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Finally, the authors would like to acknowledge the support of the SSI staff members both in the field and at the home office. Those at the site included: R.D. Bernard, D. Boyes, J. Boyes, Dr. B.L. Gabrielsen, G.M. Shephard, Jr., C. Wilton, L. Wilton, and G. Zaccor. Those who supported the project at the home office included G.J. Cuzner, M. Ineich, E.P. Kaplan, M. Reeder, and Dr. T.C. Zsutty.

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METRIC CONVERSION TABLE

Conversion Factors for U.S. Customary
to Metric (SI) Units of Measurement

To convert from:	To:	Multiply by:
inch	meter (m)	2.540×10^{-2}
foot	meter (m)	0.3048
yard	meter (m)	0.9144
square inch	meter^2 (m^2)	6.452×10^{-4}
square foot	meter^2 (m^2)	9.290×10^{-2}
pound	kilogram (kg)	0.4536
pounds per linear foot (plf)	newtons per meter (N/m)	14.5939
kip	newton (N)	4.448×10^3
kips per foot	kilonewtons per meter	14.5932
pressure (psi)	pascal (Pa)	6.894×10^3
pounds per square foot (psf)	pascal (Pa)	47.88
ksi	pascal (Pa)	6.894×10^6
kips per square foot (KSF)	pascal (Pa)	4.788×10^3
inch-pounds	meter-newtons	0.1129848
inch-pounds per foot	meter-newtons/meter	0.370682
degrees Fahrenheit	degrees Celsius	$(t_{°F} - 32)/1.8$

Section 1
INTRODUCTION

BACKGROUND

Current Civil Defense planning in the United States is based largely on the policy of "Crisis Relocation." This policy presumes that a period of crisis buildup or international tension would precede any future major war. This period of crisis would allow time to accomplish a number of activities to protect the civilian population and industry from attack. These activities would include:

1) Evacuation of most of the population out of risk areas to host areas where only fallout and possibly low-level blast protection would be required.

2) Development of shelters in the risk area for a relatively small contingent of key workers who would remain behind to maintain essential services; e.g., fire protection, communications, medical services, and military production.

3) Hardening and protection of industry.

Scientific Service, Inc., under the sponsorship of the Federal Emergency Management Agency (FEMA), is at present conducting three interrelated programs that support crisis relocation planning. These programs include the development and testing of shelter design options for both key worker and host area shelters, the development and testing of an industrial hardening manual, and the development and implementation of shelter plans for three test communities.

The MILL RACE event offered a unique opportunity for FEMA to demonstrate, using full-scale structures and test objects, the validity and practicality of a number of shelter upgrading and industrial hardening concepts that will support crisis relocation planning. MILL RACE was a high explosive test conducted on September 16, 1981 at the White Sands Missile Range in New Mexico. This test was sponsored by the Defense Nuclear Agency (DNA) and consisted of the detonation of 600 tons of an ammonium nitrate-fuel oil (ANFO) mixture that provided a simulated nuclear weapon airblast and ground motion environment for numerous experiments conducted by various government agencies. The airblast effects from detonation of 600 tons of ANFO are approximately equivalent to the detonation of a one kiloton nuclear device or 500 tons of TNT.

OBJECTIVES

As part of the above mentioned programs, Scientific Service, Inc., has produced a number of technical reports and manuals on the subjects of shelter upgrading and industrial hardening. These publications were the result of extensive testing and analysis, but were limited because of the lack of viable data in areas such as wall performance, the interaction of walls with floors and roofs, connections, closures for doors and windows, surcharge loads on walls caused by blast loading, and the performance of industrial equipment and machinery under blast loading. Although some data were available from previous high explosive and nuclear tests, most were derived from static rather than dynamic testing. Accordingly, the MILL RACE event presented FEMA with the opportunity to address a number of these problem areas under more realistic loading conditions. The design of the experiments for MILL RACE was expressly directed toward developing data in these "gray" areas of structural response under blast loading. Following is a brief outline of the objectives of the experiments and their interrelation to the ongoing programs in support of crisis relocation planning.

The Industrial Protection Manual, SSI 8011 (Ref. 1), consists of ten booklets directed toward the development of an industrial hardening plan for the purpose of assisting in plant equipment and personnel survival, post-crisis recovery, and operation subsequent to a disaster. The last four booklets in this manual address the vulnerability, priority, and blast ratings, the hardening of industrial equipment, and the sheltering of key workers. A considerable amount of the data presented in this manual has been determined analytically, supplemented by a few small-scale tests when practical. The industrial experiments at MILL RACE investigated the performance of industrial machinery and equipment, secured and hardened by various methods, at several overpressure levels, and subjected to varying degrees of debris translation. The capability of expedient key worker shelters was also included in the experiments.

The Shelter Upgrading Manuals, one for host area shelters, SSI 7815-8 (Ref. 2), and one for key worker shelters, SSI 8012-7 (Ref. 3), were both supported extensively by full- and small-scale testing, primarily static, supplemented by statistical analysis. The manuals contain sketches of typical building elements and possible expedient shelter options, charts for the selection of upgrading methods and shore sizing, and worksheets to assist the user. Four technical reports were the primary basis for the information presented in these manuals: SSI 7618-1 (Ref. 4), SSI 7719-4 (Ref. 5), SSI 7910-5 (Ref. 6), and SSI 8012-6 (Ref. 7). The tests discussed in these reports were conducted basically on building components; i.e., sections of floors or walls, and small concrete slab sections used for punching shear data. Although these types of tests are technically valuable and cost effective in determining the static load capacity of these components, under the "as built" and shored conditions, they do not address all of the aspects required to determine the dynamic load capability of the total structure. Other important areas that required investigation were the loading transmitted to basement walls by a blast overpressure surcharge, connection integrity, wall upgrading, closure evaluation, continuous floor or roof spans, performance of various shore types, the effect of debris translation on shoring integrity, and manpower and resource requirements for various upgrading schemes. The MILL RACE test provided the opportunity to investigate some of these effects on typical structures. Thus, the shelter experiments conducted at MILL RACE were four structures, all designed using typical building codes currently in use, and constructed by a civilian contractor using the methods and materials that he would normally use. Incorporated in these four structures were as many of the desired conditions for evaluation as it was possible to include without having one experiment influence an adjacent one.

At the request of FEMA, an experiment was included of a key worker expedient shelter constructed of dimension lumber. This shelter was similar in overall size to a shelter described in the "Expedient Shelter Handbook", ORNL-4941, by Oak Ridge National Laboratory (Ref. 8). Valuable data were derived from this experiment with respect to the resource and manpower requirements for this type of shelter.

In summary, the eight FEMA experiments approved by DNA for inclusion in the MILL RACE event, which are covered in this report, are: three host area shelters (DNA Nos. 5001, 5002, 5003), one key worker shelter (DNA No. 5201), three industrial hardening experiments (DNA Nos. 5101, 5102, 5103), and a key worker expedient shelter (DNA No. 5301). Figure 1-1 shows the test bed layout and the location of these experiments relative to each other and to ground zero. Figure 1-2 shows the bags of ANFO explosive (600 tons) stacked prior to detonation; Figure 1-3 shows the cloud formed by the explosion immediately after detonation.

LIMITATIONS

Although the MILL RACE event produced the equivalent of an approximate one kiloton nuclear explosion, the short duration of the blast effects required considerable attention be directed toward the design of the experiments and the interpretation of the resulting test data. In the initial layout of the experiments, each was oriented such that the short dimension was normal to the radials, thus reducing the differential overpressure between the front of the experiment (side closest to ground zero) and the back (side furthest from ground zero) to a minimum. Additionally, the structural performance of the experiments was analyzed with these limitations in mind in order to assess, to the degree possible, the probable performance of long duration blast effects from megaton explosions.

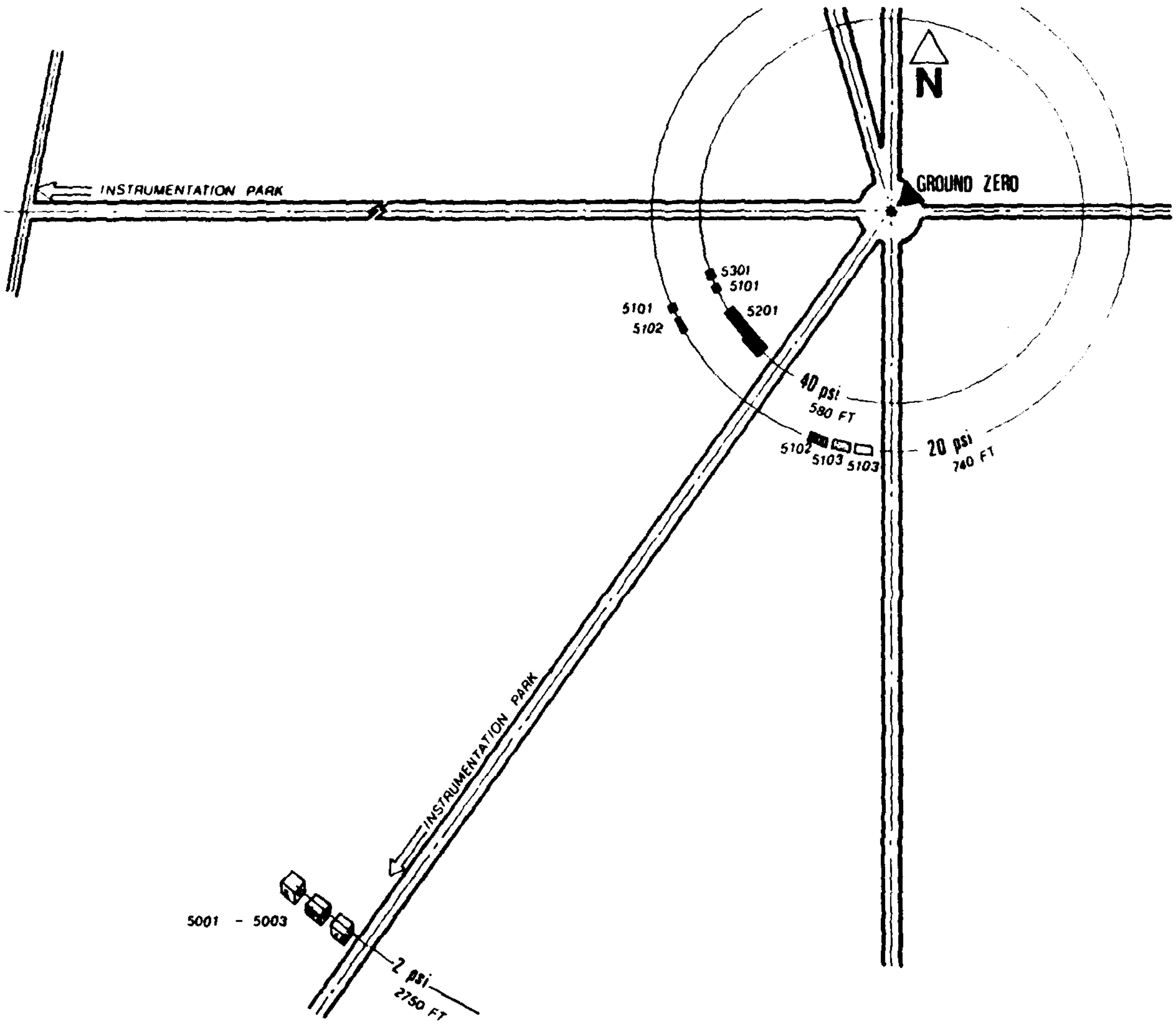


Fig. 1-1. Test Bed Layout.

Fig. 1-2. ANFO Explosive Stacked Prior to Detonation.

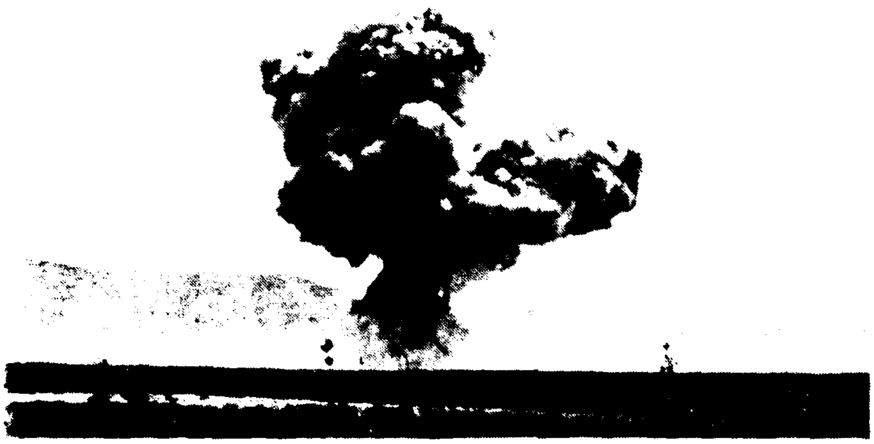


Fig. 1-3. Explosion Soon After Detonation.

REPORT ORGANIZATION

This report describes all of the experiments in detail, outlining the specific objectives of each, the design parameters used, the construction methods and materials, the upgrading methodology and materials, the instrumentation, and the test results, observations, and conclusions. The report is supplemented with instrumentation readings and observations, photographs, and sketches and drawings.

The remainder of this report is organized as follows:

Section	2	DNA No. 5001, Host Area Shelter
Section	3	DNA No. 5002, Host Area Shelter
Section	4	DNA No. 5003, Host Area Shelter
Section	5	DNA No. 5201, Key Worker Shelter
Section	6	DNA No. 5101, Industrial Hardening - Expedient Shelters
Section	7	DNA Nos. 5102, 5103, Industrial Hardening
Section	8	DNA No. 5301, Key Worker Expedient Shelter
Section	9	Program Summary
Section	10	References

Section 2

DNA NO. 5001

HOST AREA SHELTER

INTRODUCTION

This experiment was one of three 24-ft-by-16-ft buildings constituting the host area shelter array at the 2 psi environment, 2,750 ft from ground zero. This particular building was a one-story wood-frame structure with interior partitions, two doors, and five windows. It was constructed on grade on a concrete slab.

OBJECTIVE

This structure simulated a typical one-story residential or motel type of building of light construction. It was bermed on all four sides with earth, and the roof was covered with approximately 18 in. of earth in order to simulate fallout radiation protection equal to a PF of 100. The object of the experiment was to shore the roof, shore or brace the walls, and close off the windows and doors, to the degree required to support the earthfill and withstand the 2 psi overpressure, and evaluate the performance of the structure, shoring, and closures.

DESIGN

The structure was designed in accordance with the 1979 edition of the Uniform Building Code (Ref. 9). The timber roof and wall framing was based on the conventional framing requirements of Chapter 25, the concrete slab on Chapter 26. The roof was designed to support a 30 psf live load and a 15 psf dead load in accordance with the code recommendations for these types of structures.

CONSTRUCTION

The roof consisted of 2-in.-by-8-in. timber joists, spaced 16 in. on center, sheathed on the top with 3/8-in. thick plywood, and on the bottom with 1/2-in. thick gypsum wallboard. The joists spanned one-half the length of the building, or 12 ft, and were supported at that location by the interior partition and/or a timber header beam. The walls and interior partitions were constructed with 2-in.-by-4-in. wood studs located at 16 in. on center. The walls were sheathed with 3/8-in. thick plywood on the exterior, and on the interior with 1/2-in. gypsum wallboard. The interior partitions were sheathed on both sides with the gypsum wallboard. The concrete slab on grade was 4 in. thick throughout with a monolithically poured footing (12 in. by 12 in.) around the periphery. The design strength of the concrete at 28 days was 3,000 psi, and 6x6-W1.4xW1.4 welded wire fabric was located at the mid-depth of the slab. The wall sole plates were secured to the concrete slab with 1/2-in. diameter anchor bolts at 8 ft on center maximum. The completed building measured approximately 16 ft by 24 ft and was 8 ft high. Figure 2-1 shows the floor plan, and Figure 2-2 the roof framing plan of this building. Figure 2-3 is an elevation of the building. A section through an exterior wall is shown in Figure 2-4. Figure 2-5 is a photograph of the building under construction, and Figure 2-6 shows the completed building.

UPGRADING

The roof shoring consisted of two timber stud walls, located at the midspans of the 12-ft roof joist spans, and running the full 16-ft width of the building. The stud walls were constructed of 2-in.-by-4-in. studs, spaced 16 in. on center, with a double 2-in.-by-4-in. top plate and a single 2-in.-by-4-in. sole plate. The horizontal and diagonal bracing was of 1-in.-by-4-in. timber. One of the stud wall shores had to be constructed in three sections because of the interference of the interior partitions. The shores were shimmed to fit tightly between the floor and ceiling. The material sizes and locations of these shores were obtained from the Shelter Upgrading Manual: Host Area Shelters (Ref. 2). Figure 2-7 shows the location of the stud wall shores on the building floor plan.

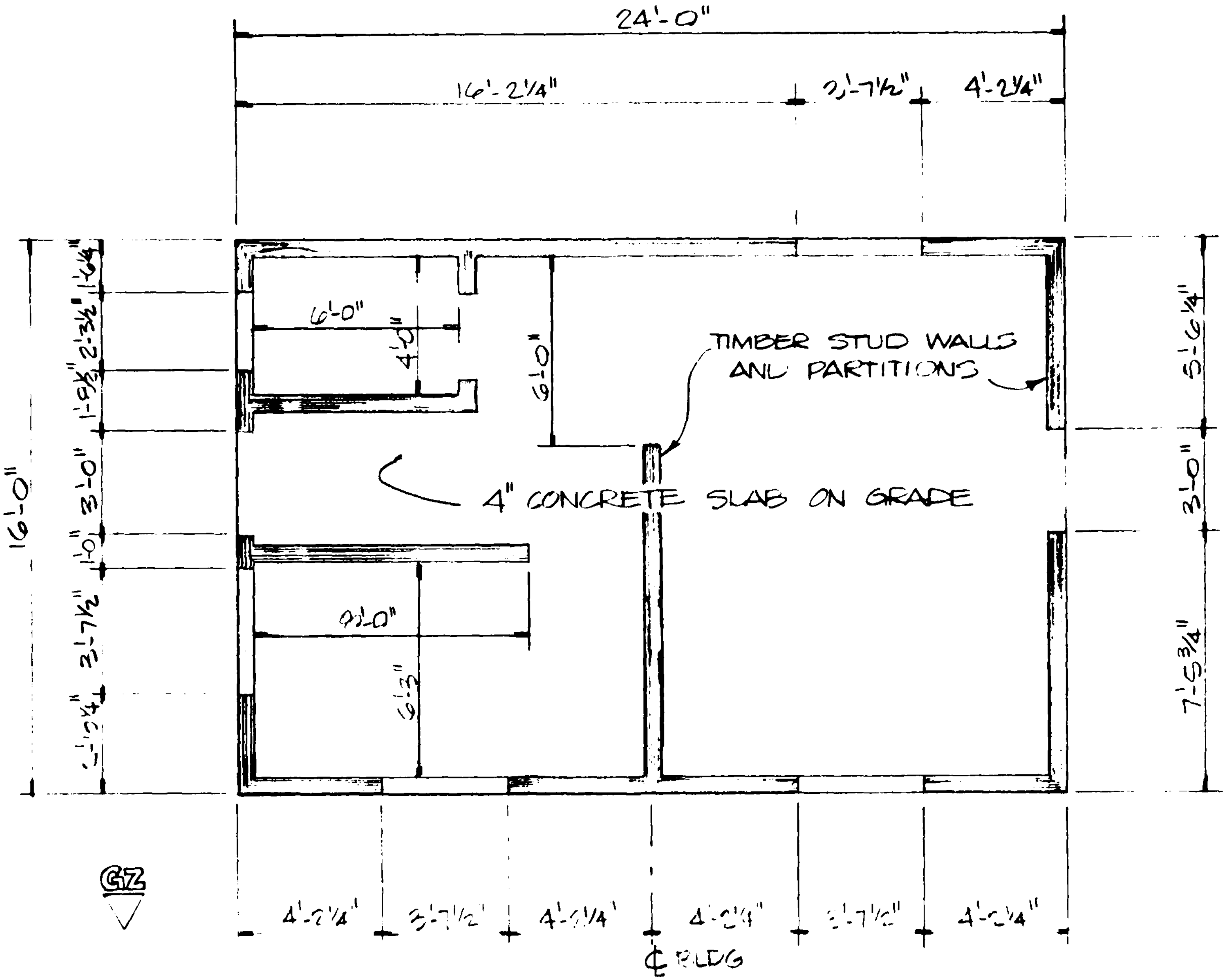


Fig. 2-1. Floor Plan, DNA No. 5001.

DNA No. 5001

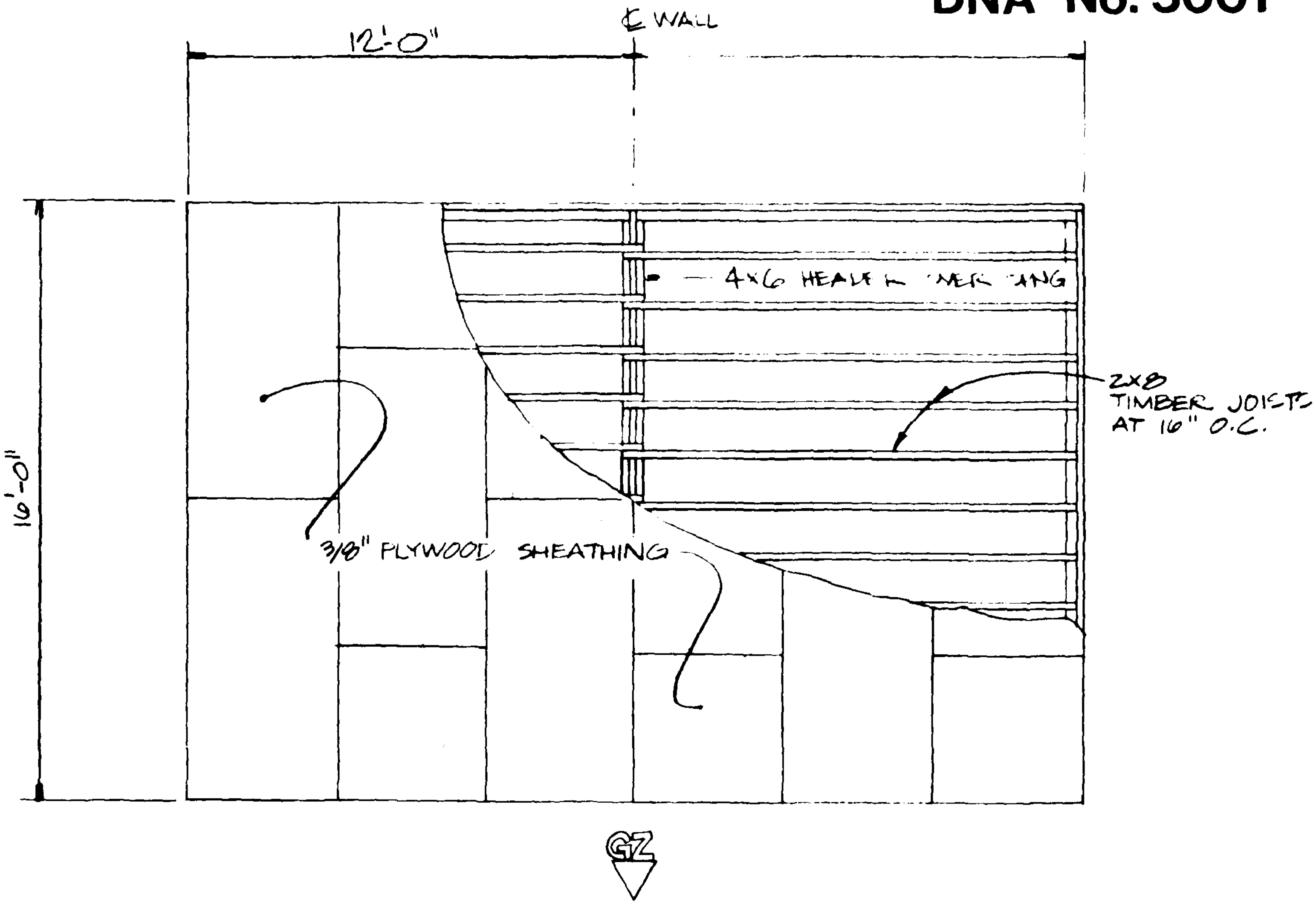
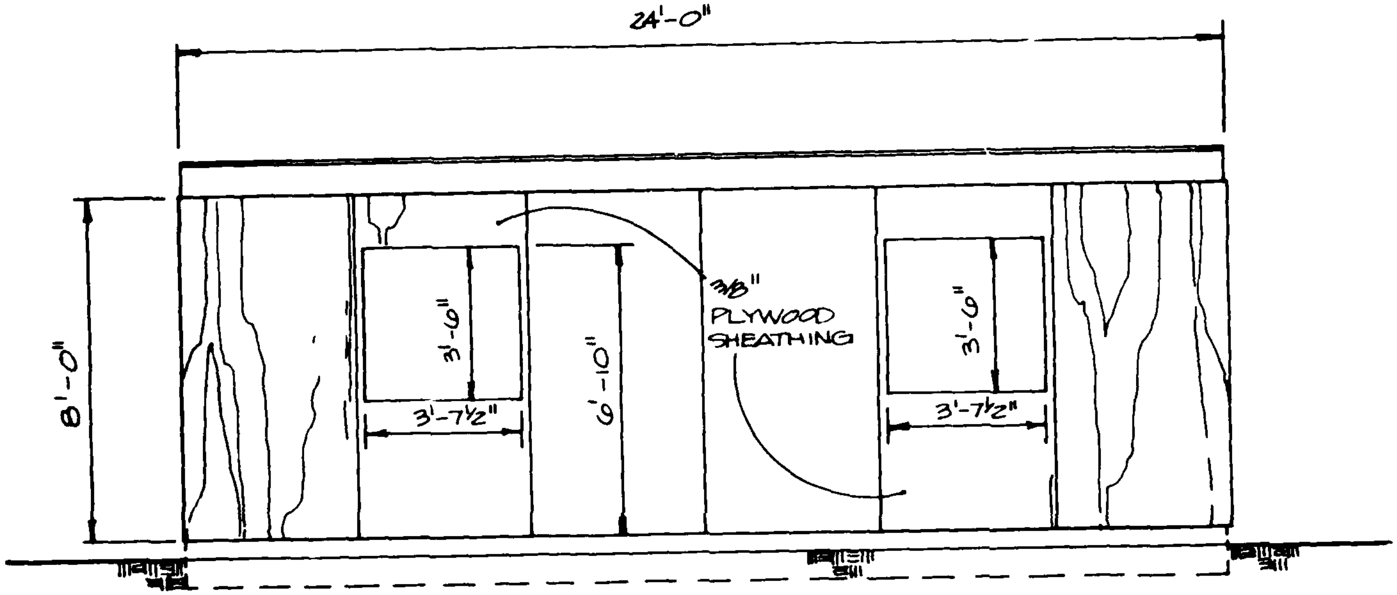


Fig. 2-2. Roof Framing Plan, DNA No. 5001.



DNA No. 5001

Fig. 2-3. Elevation, DNA No. 5001.

DNA No. 5001

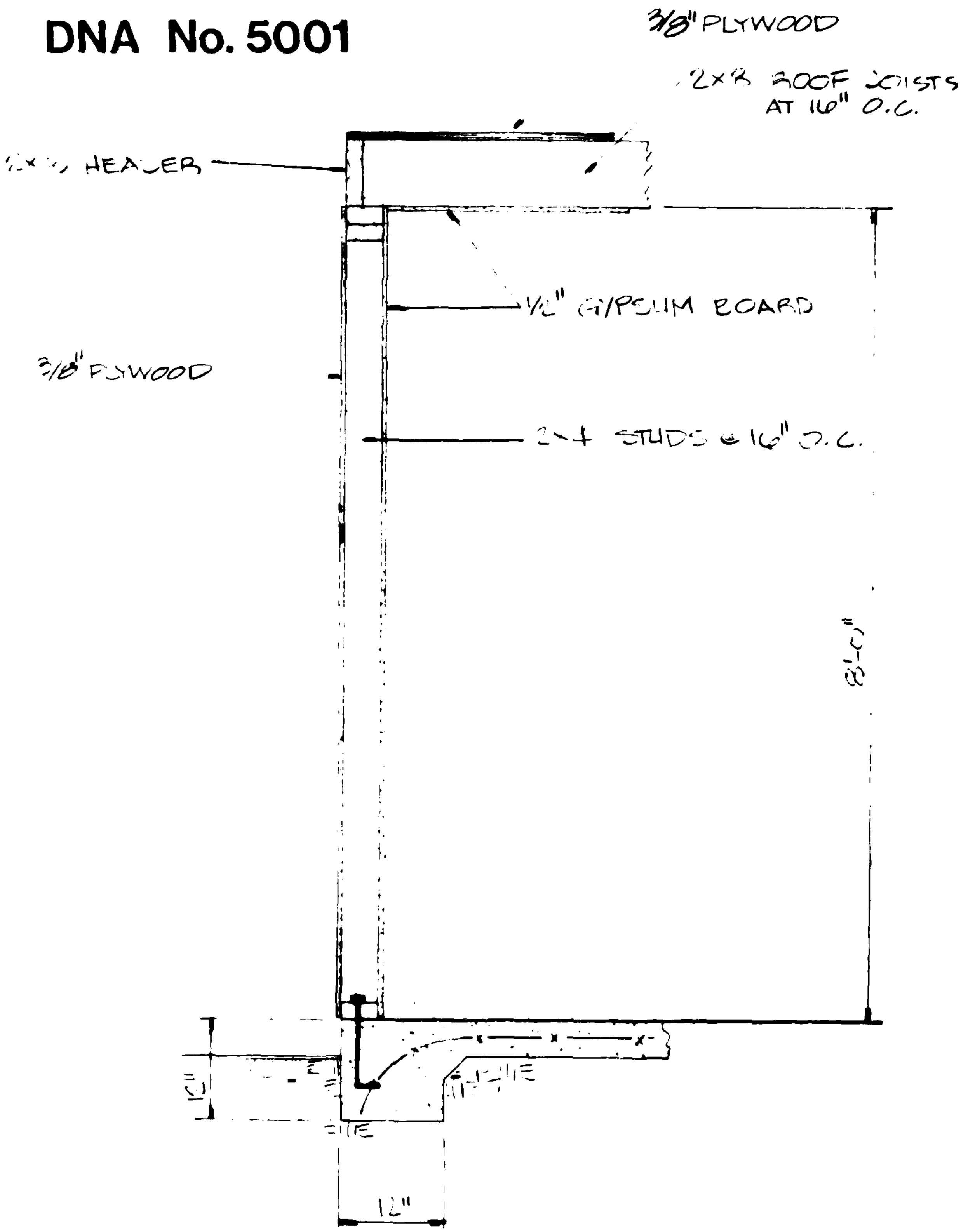


Fig. 2-4. Section Through Exterior Wall, DNA No. 5001.

Fig. 2-5. Building Under Construction.

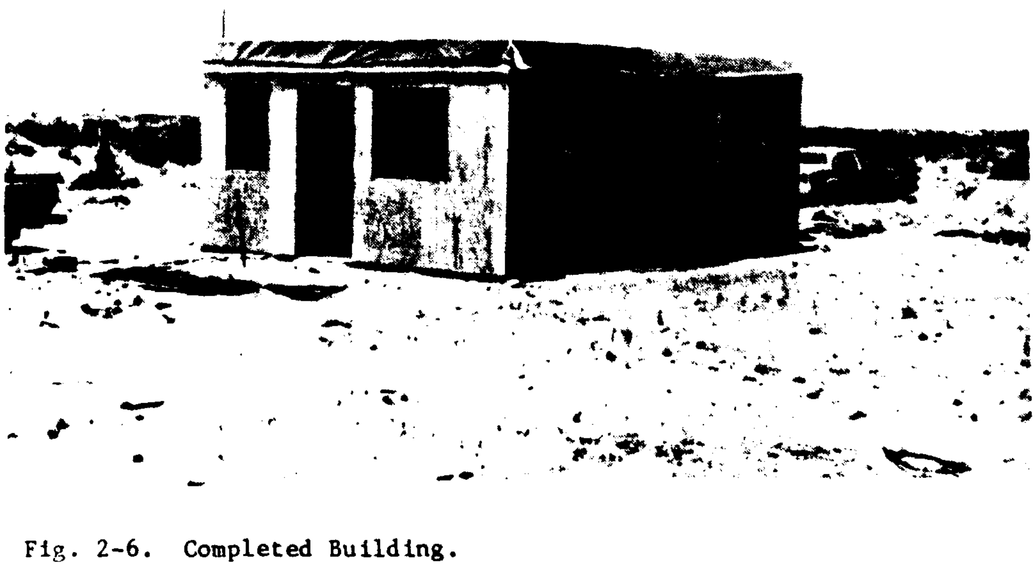


Fig. 2-6. Completed Building.

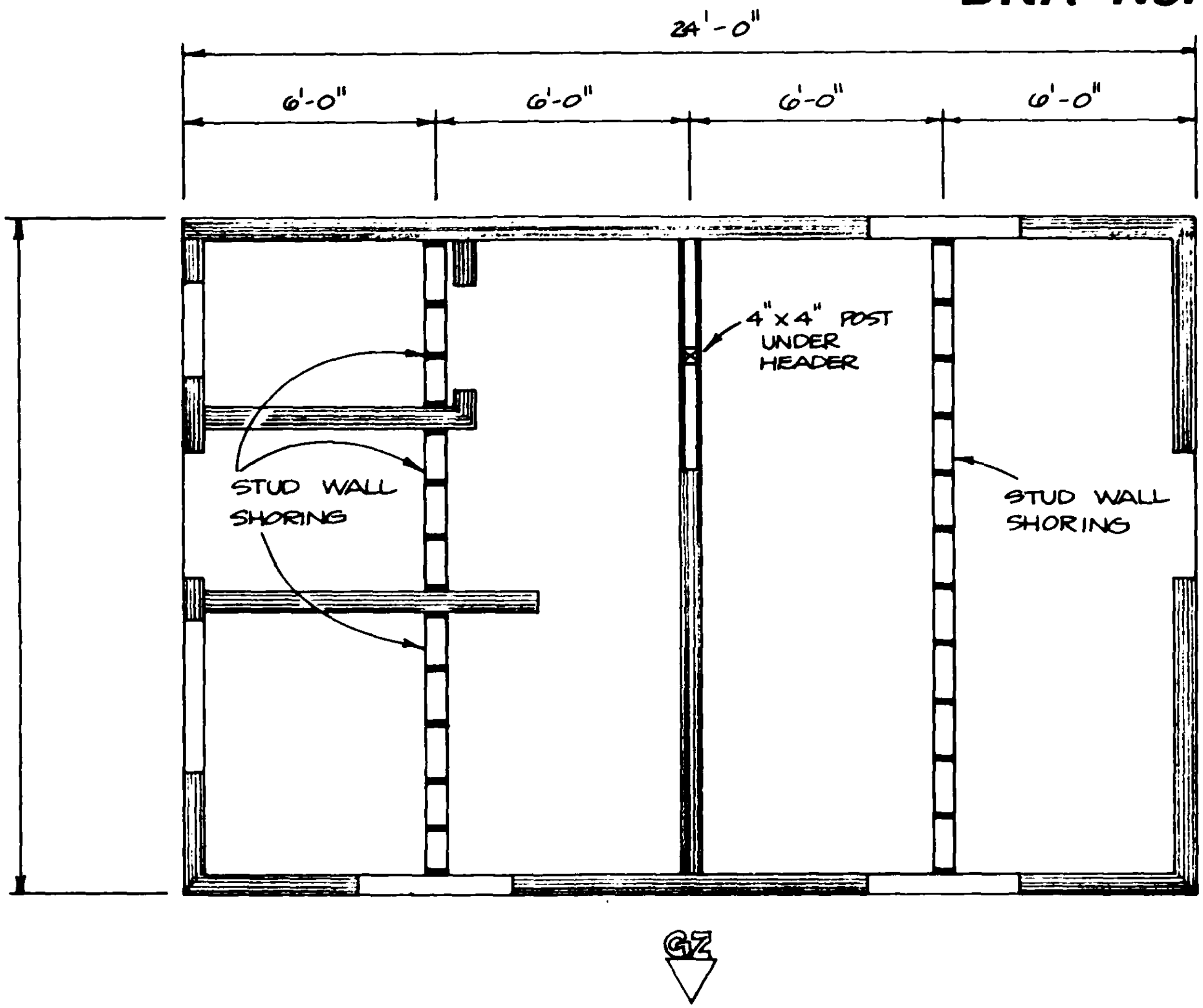


Fig. 2-7. Stud Wall Shoring Location.

The wall upgrading consisted of required additions to both the interior and exterior of the walls. On the exterior of all four walls, an additional layer of plywood, 3/8 in. thick, was tacked to the existing sheathing. This upgrading plywood was placed with the 8-ft length horizontal as opposed to the sheathing, which was placed with the 8-ft length vertical. In typical construction, placing the plywood vertical, or with the grain parallel to the stud supports, is not unusual, but this orientation is detrimental to its ability to withstand loads of the magnitudes applied by earth berms and blast overpressure. Plywood will carry up to five times more load when the grain is perpendicular to the supporting members (in this case, the vertical 2x4 wood studs). Accordingly, for upgrading purposes it was necessary to add the additional plywood layer. If a structure such as this was for some reason constructed in an "untypical" manner; i.e., the sheathing installed horizontal, this additional layer would not be required for upgrading. We believed that it was advantageous to the test program to construct all the experiments "typical".

The upgrading on the interior of the walls used two different configurations. On the 16-ft walls, perpendicular to the roof joists, 2-in.-by-4-in. studs, 4 ft long, were placed parallel to and against each stud at midheight of the wall, and kicked back to blocks with 2x4's secured to the roof joists at the top and the concrete slab at the bottom. On the 24-ft walls, parallel to the roof joists, it was not desirable to apply kick braces to these joists since the load would be applied perpendicular to the joists. Therefore, 8-ft long, 2-in.-by-8-in. timbers were placed parallel to and against each wall stud full height. These timbers were secured at their ends with bracing fastened to the floor on the bottom and the across the full width of the building at the top. Figure 2-8 shows the stud wall shoring in place.

The closures for both the windows and doors were constructed similar to the original walls; i.e., 2x4 timber studs at 16 in. on center with 3/8-in. thick plywood sheathing on the exterior. The gypsum wallboard was not used on the interior. Each closure was fabricated individually to fit into each of the openings, and was braced from the inside similar to the walls. Figure 2-9 shows the door and window closures partially in place, and Figure 2-10 shows the additional layer of exterior plywood sheathing being installed. Figure 2-11 shows the interior of the building with the shoring and closures completed.

Fig. 2-8. Stud Wall Shoring In Place.

Fig. 2-9 Door and Window Closures Partially In Place.

Fig. 2-10. Additional Layer of Exterior Plywood Being Installed.

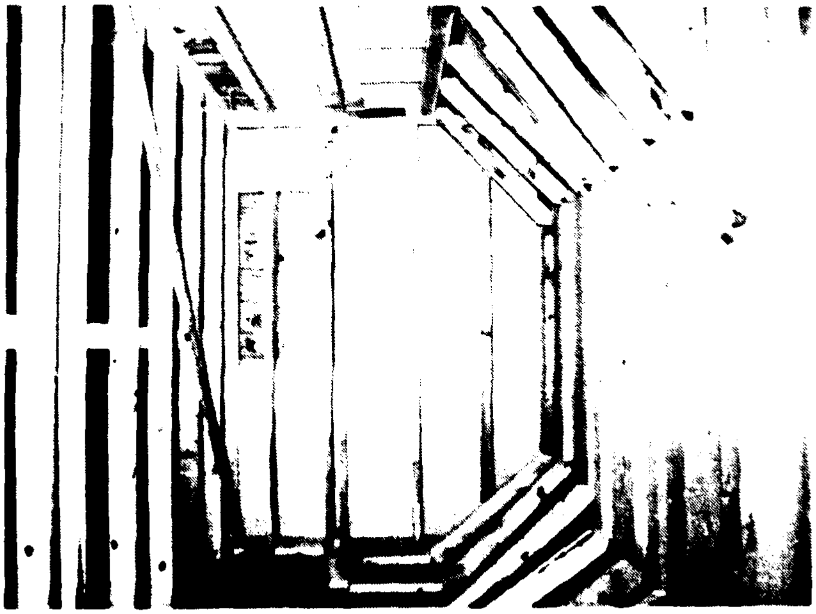


Fig. 2-11. Interior of Building With Shoring and Closures.

Figure 2-12 shows the building completely bermed and covered with approximately 18 in. of earth.

INSTRUMENTATION

The instrumentation for this experiment was still photography.

TEST DATA

The peak free-field overpressure at this location, as recorded and supplied by DNA, was 1.9 psi.

Immediately after the test, close observations of the earth surrounding the building indicated apparent movement. Fissures or cracks were present near the top of the berms on all four sides, and a slight dishing of the earth over the roof was noted.

Six days after the test, earth was removed from one end of the building, a door closure removed, and the building inspected. No apparent structural damage was observed. A slight increase (approximately 1/16 in.) was noted in the width of the joints between the sections of gypsum wallboard secured to the ceiling, and several of the vertical studs in the stud wall shoring system were slightly bowed. The interior of the building was clean and free of dust and debris.

Figure 2-13 shows the exposed end wall of the building after the test, and Figure 2-14 is an interior view showing the ceiling and walls with the shoring intact.

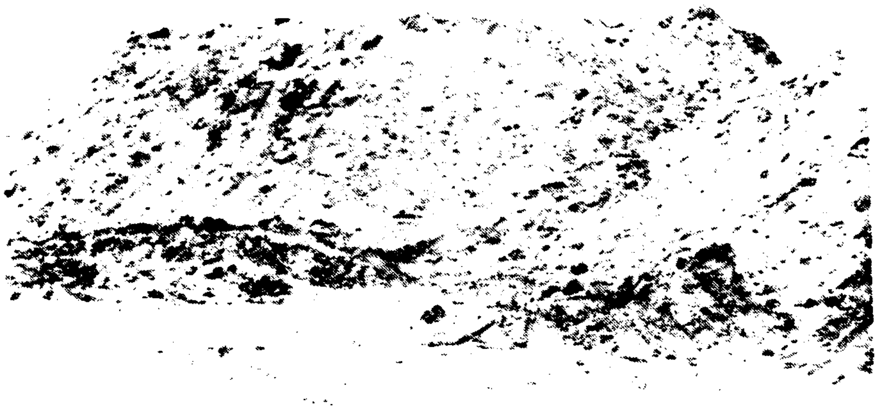


Fig. 2-12. Building Covered and Bermed.

DNA No. 5001

Fig. 2-13. Exposed End Wall of Building, Posttest.

Fig. 2-14. Interior View of Ceiling and Walls, Posttest.

DISCUSSION AND CONCLUSIONS

This upgraded structure performed quite well in the 2 psi range, and when upgraded in this manner, would likely be adequate for overpressures of 2 psi even with increased weapon size. The disadvantages associated with this experiment appeared to lie primarily in the construction and installation of the upgrading system; i.e., the labor and materials required to perform these tasks.

As evidenced by the performance of the walls, the wall bracing as designed and constructed was more than adequate, and it might be concluded that it was significantly overdesigned. The design methodology employed for the wall bracing was based on basic timber design and the best available data on earth loading of walls. Unfortunately, there is a paucity of data on the specific subject of pressures transmitted to bermed walls by dynamic or blast loading, and this area requires significant further research and testing.

The two lines of stud wall shoring performed well. It appeared that this system was approaching its ultimate load limitation, as evidenced by the bowing of several of the vertical members. As mentioned above, the design of this system was taken directly from the Shelter Upgrading Manual: Host Areas Shelters (Ref. 2).

With respect to the construction and installation of the upgrading system, the labor and materials expended on the wall bracing appear to be excessive and not consistent with the concept of "Crisis Relocation". Four man-days (32 hours) were expended on the wall bracing for this building. A great amount of this time was spent in sawing the various size pieces to length, and cutting the required angles on the ends. The application of the explosively driven inserts, used to secure the braces to the floor, required one man-day.

The sizes of the door openings required the stud wall roof shoring to be constructed inside of the building. It was anticipated that this type of shoring system would be material and labor effective, and although the material was readily obtainable, the actual construction of the walls was somewhat hindered by the small working space and the interference of the already installed wall bracing. It was judged that if the upgrading construction sequence had been reversed — the wall bracing had been installed after the roof shoring — this condition would not have improved, since it would have then been necessary to pass the bracing timber around and through the erected wall. The time required to construct and install the stud wall shoring was 2½ man-days.

The window and door closures required 1½ man-days to construct and install. These closures were rather complex (the same construction as the walls) and were constructed to fit the openings more precisely than required, or than would be done typically. Other types of closures of similar integrity are available and would be suggested in the future.

The additional layer of exterior plywood sheathing was installed without problems in 1½ man-days. It should be noted, however, that this sheathing was easily nailed to the existing sheathing, but if the as-built exterior were some other material, such as stucco, metal siding, or brick veneer, this upgrading scheme might be less easily installed.

The total time required for the upgrading, excluding the berming and covering of the roof with earth, was 9½ man-days (76 hours). The wall bracing system was the most detrimental factor in this time frame, not only because of its individual complexity, but because of the effect it had on other parts of the system, such as the roof shoring and closures. Also, there is some degree of structural interdependence between the walls and the roof of the building, and the fact the wall bracing may have been overdesigned could have directly affected the roof performance. Additional study and testing are required and recommended in the areas of dynamic earth pressures on walls and wall bracing systems.

Section 3

DNA NO. 5002

HOST AREA SHELTER

INTRODUCTION

This experiment was the second of the three 24-ft-by-16-ft buildings in the host area environment of 2 psi. This building was a one-story masonry structure with a precast prestressed concrete slab roof, and containing interior partitions. The building contained two doors and five windows, and was constructed on grade on a concrete slab.

OBJECTIVE

This structure simulated a typical one-story retail store or motel type of building of light masonry construction. It was bermed on all four sides with earth and the roof was covered with approximately 18 in. of earth to simulate fallout radiation protection equal to a PF of 100. The object of the experiment, as in DNA No. 5001, was to shore the roof, shore and brace the walls, and install closures for the windows and doors, to support 18 in. of earthfill and withstand the 2 psi overpressure, and to evaluate the performance of the structure, shoring, and closures.

DESIGN

The structure was designed basically in accordance with the 1979 edition of the Uniform Building Code (Ref. 9). The masonry walls were based on the minimum requirements for hollow unit masonry from Chapter 24, and the interior wood stud partitions on Chapter 25. The concrete slab on grade was designed in accordance with Chapter 26, and the precast prestressed concrete hollow-core slabs in accordance with Chapter 26 and Chapter 18 of the Building Code Requirements for Reinforced Concrete (ACI 318-77), Ref. 10. The roof was specified to be designed to support 30 psf live load and 15 psf dead load in accordance with the code recommendations; however, because this was a very short span for this type of slab, the prestressed concrete manufacturer was not able to produce a section with such a low load capacity. Accordingly, using his minimum section and minimum reinforcing, the safe load capacity of the slabs actually supplied and used was 293 psf. This, again, is a typical situation in that the manufacturers of these types of products are not in a position to supply all of the various configurations conforming to the exact specified loading, but instead concentrate on the ones most used in their geographical areas.

CONSTRUCTION

The concrete roof slabs were 6 in. thick, 48 in. wide and approximately 12 ft long. They spanned one-half of the length of the building, or 12 ft, and were supported at the center of the building by an interior block partition and a structural steel header beam. The slabs were designed, manufactured, and delivered to the test site by a precast concrete manufacturer familiar with these types of products. To obtain maximum data on masonry units from this experiment, one-half of the walls were constructed with hollow masonry units (8 in. x 8 in. x 16 in. standard concrete block) ungrouted, and one-half with common 4 in. x 4 in. x 8 in. brick. The interior partition was of concrete block. Both types of units were installed in a running bond pattern. The top course of all the concrete block was grouted full. The lintels over the windows and doors in the block walls were constructed of two courses of grouted block with two No. 3 steel reinforcing bars in each course. The lintels in the brick walls were structural steel tees. The concrete slab on grade was 4 in. thick throughout with a monolithically poured 12-in.-by-12-in. footing at the periphery. The design strength of the concrete at 28 days was 3,000 psi, and welded wire fabric (6x6-W1.4xW1.4) was located at mid-depth of the slab. Typical of this type of unreinfored masonry construction, there was no positive connection between the slab and the masonry walls.

The completed building measured 16 ft by 24 ft and was 8 ft high. Figure 3-1 shows the floor plan, and Figure 3-2 the roof framing plan of this building. An elevation of the building is shown in Figure 3-3. Figures 3-4 and 3-5 show sections through the exterior block and brick walls. Figure 3-6 shows the partially completed building, and Figure 3-7 shows the interior after completion.

UPGRADING

The roof shoring consisted of two timber stud walls, each located at the mid-span of the 12-ft roof slab span, and running the full 16-ft width of the building. The stud walls were constructed of 2-in.-by-4-in. studs, spaced 16 in. on center, with a double 2-in.-by-4-in. top plate and a single 2-in.-by-4-in. sole plate. The horizontal and diagonal bracing was of 1-in.-by-4-in. timber. One of the stud wall shores was constructed in sections because of interior partition interference. No shoring was used in the small 3½-ft-by-6-ft corner area. The material size and location of the shores was obtained from the Shelter Upgrading Manual: Host Area Shelters (Ref. 2). Figure 3-8 shows the building floor plan with the shoring locations.

The same upgrading configuration was used on all four walls. Timbers, 2 in. by 8 in., and 8 ft long, were located vertically 16 in. on center and secured at the top and bottom with blocks fastened to the roof and floor. Pretest views of the stud wall shoring and wall bracing are shown in Figure 3-9.

The closures for both the windows and doors were constructed similar to those in Experiment No. 5001, 2x4 timber studs at 16 in. on center with 3/8-in. thick plywood sheathing on the exterior. Each of the closures was fabricated individually to fit into the openings, and was braced from the inside similar to the walls.

Figure 3-10 shows the building being bermed, and Figure 3-11 shows the building just prior to the test, completely bermed on all four sides and the roof covered with earth.

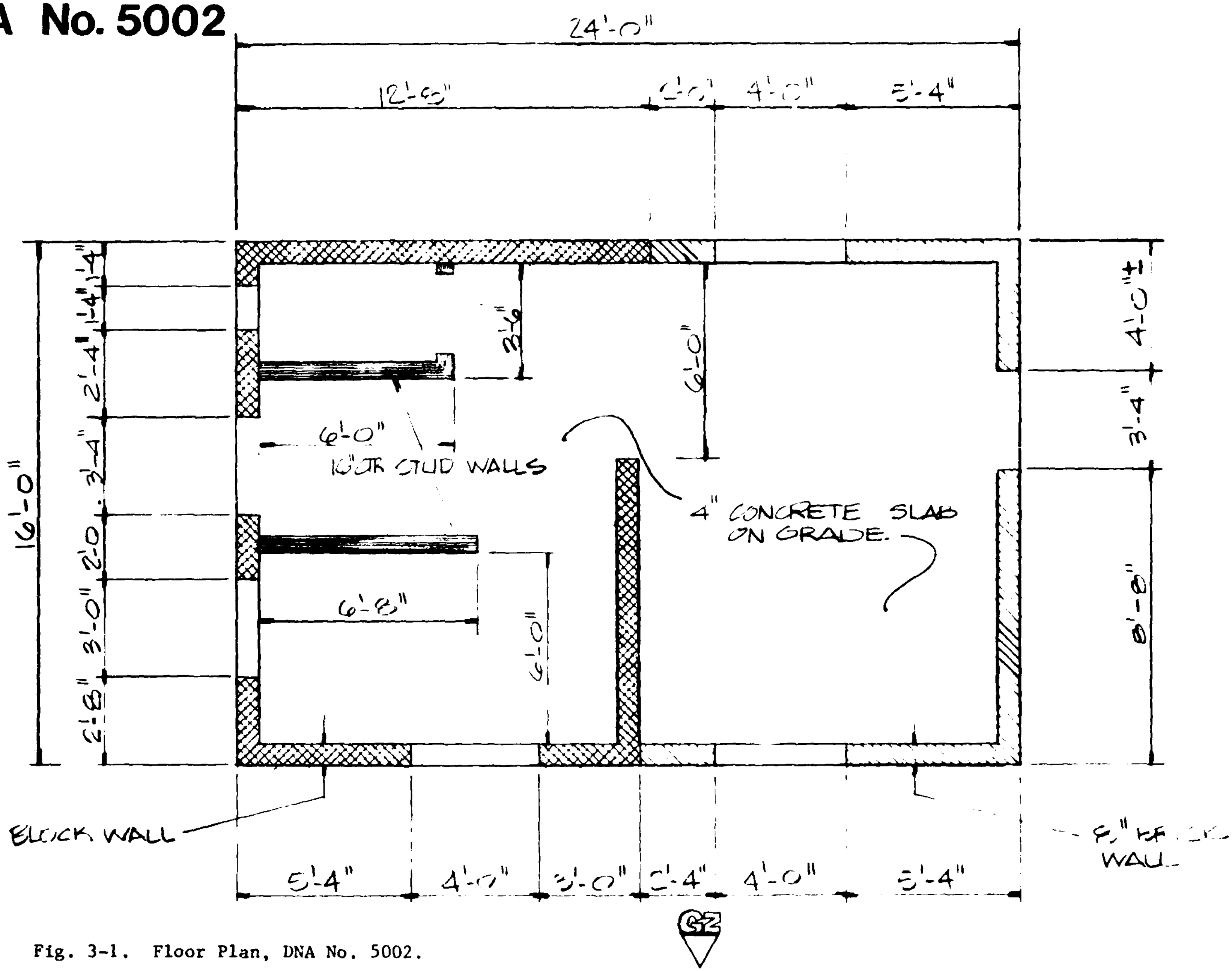


Fig. 3-1. Floor Plan, DNA No. 5002.

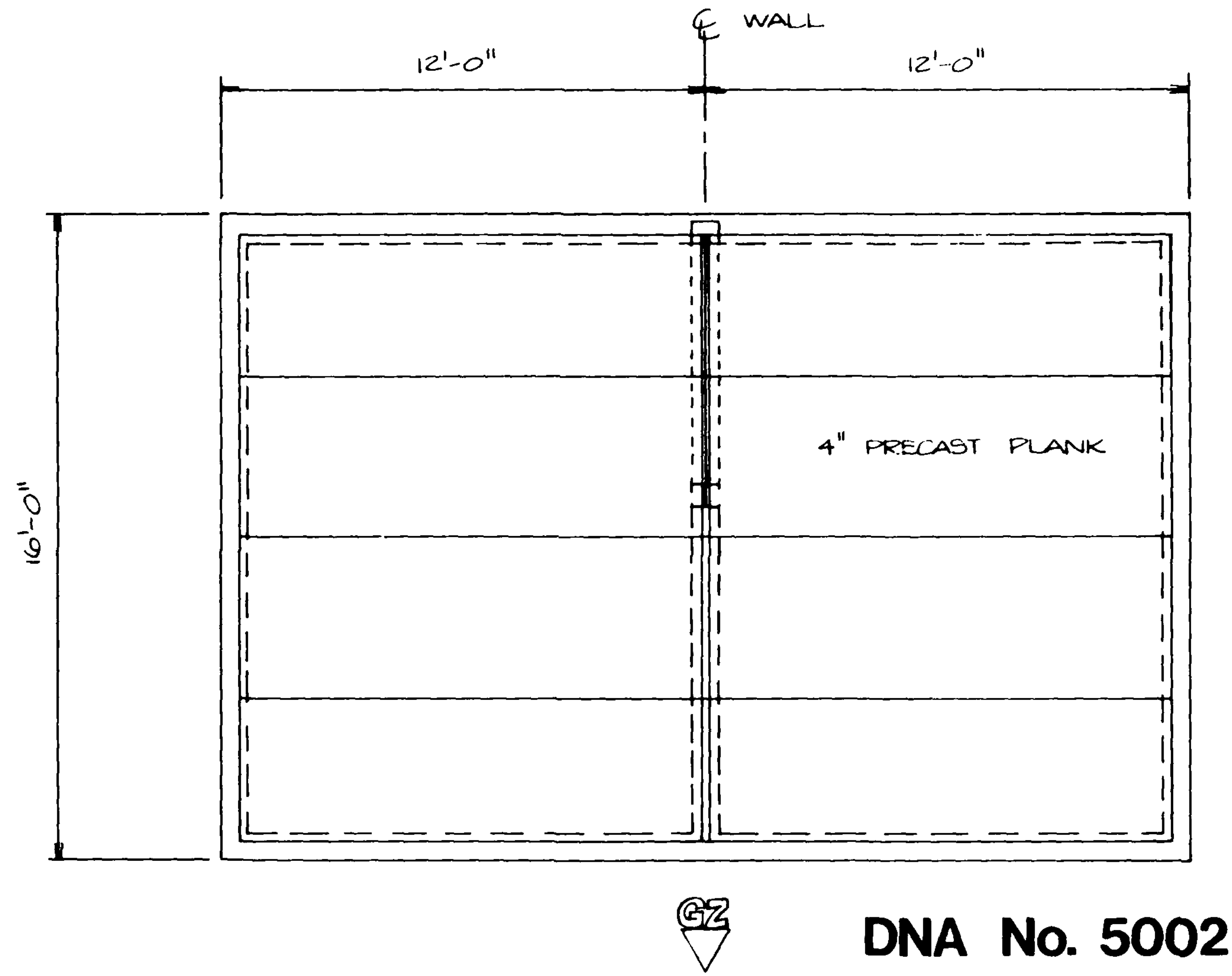


Fig. 3-2. Roof Framing Plan, DNA No. 5002.

DNA No. 5002

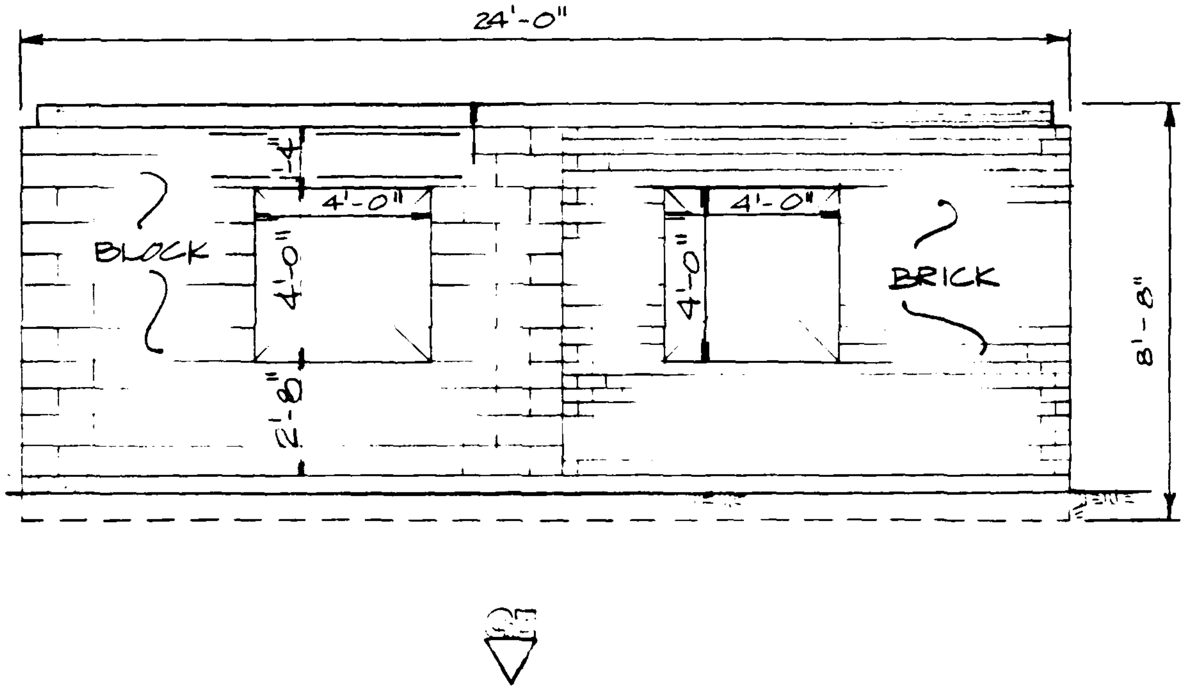
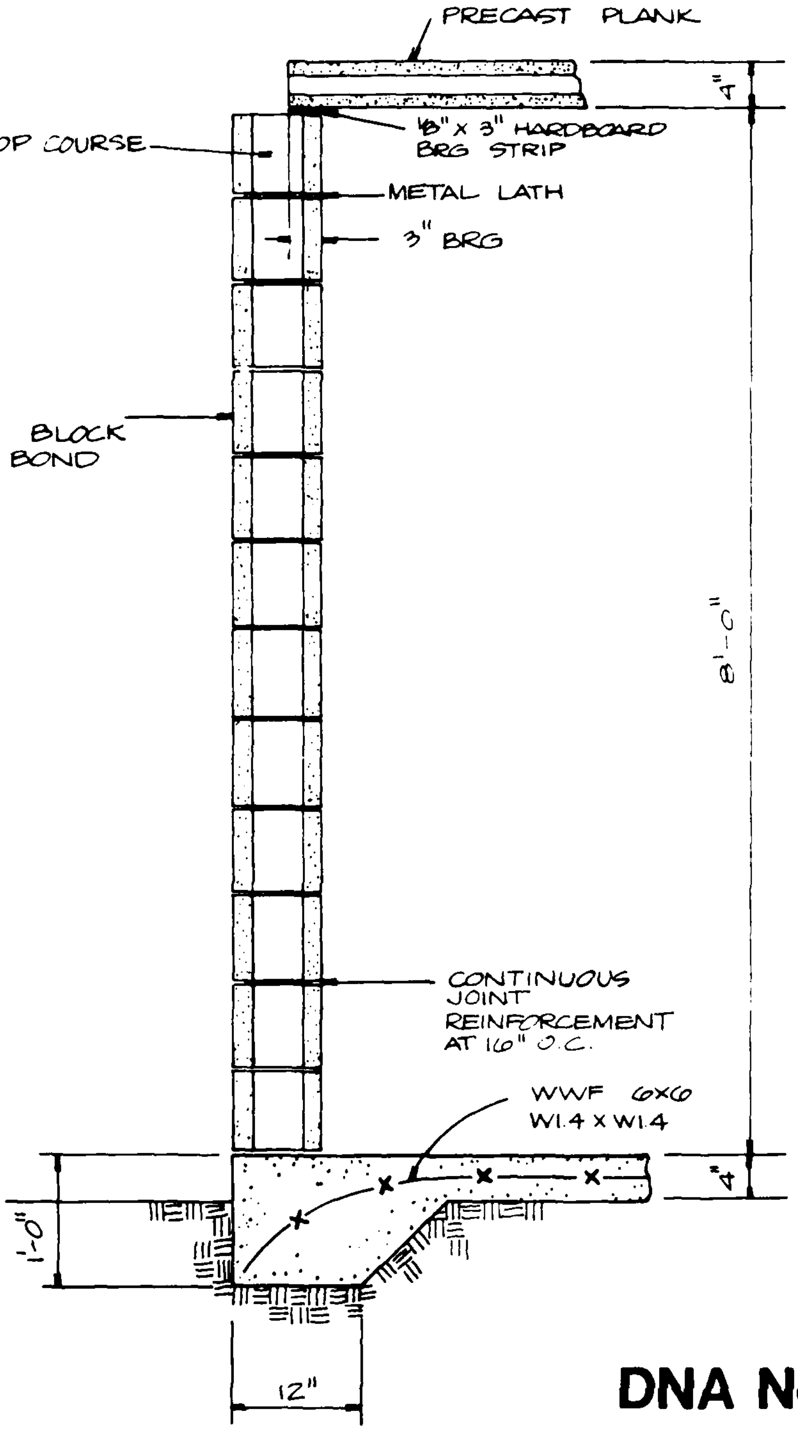


Fig. 3-3. Elevation, DNA No. 5002.



DNA No. 5002

Fig. 3-4. Section Through Exterior Block Wall.

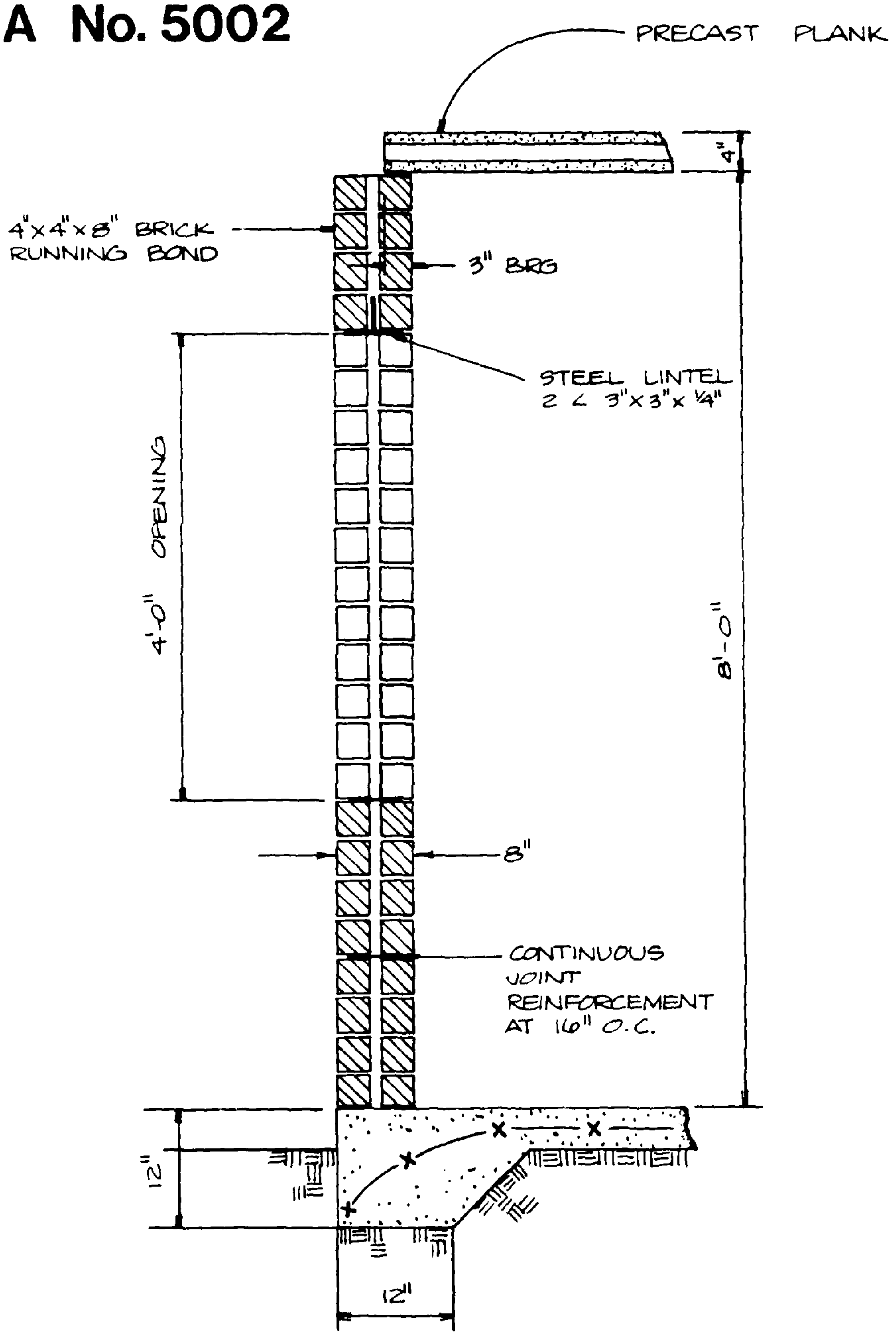


Fig. 3-5. Section Through Exterior Brick Wall.

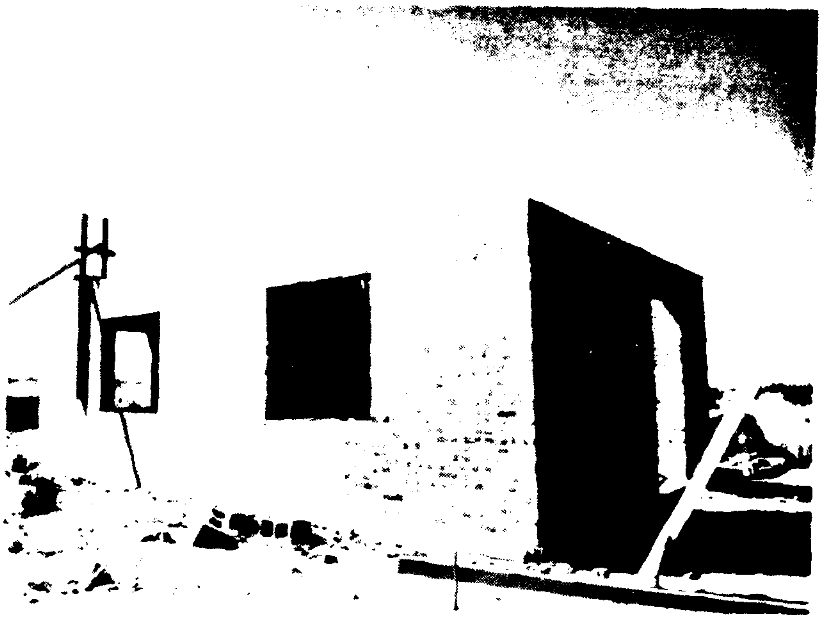


Fig. 3-6. Building Partially Completed.

Fig. 3-7. Completed Interior.

DNA No. 5002

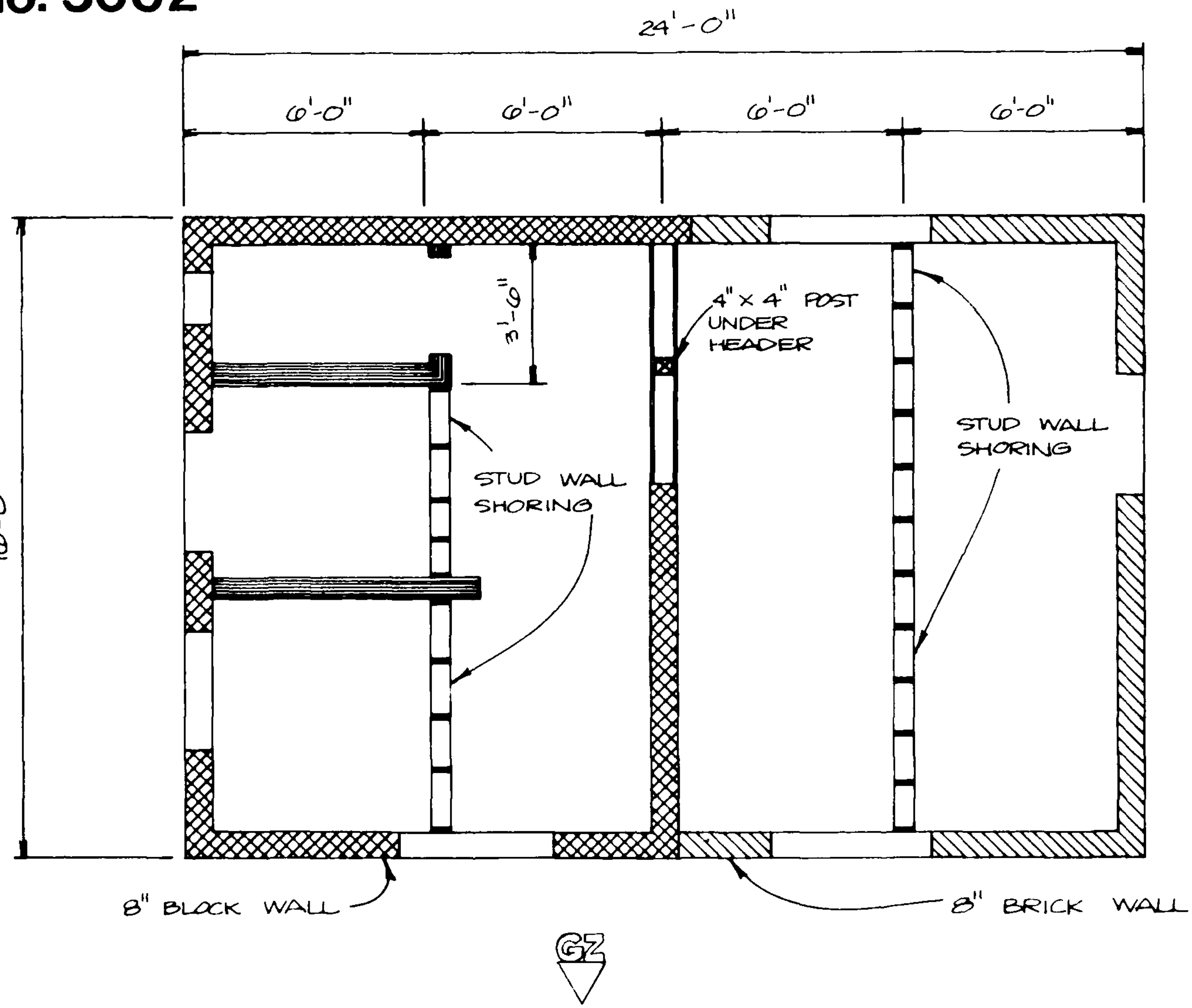


Fig. 3-8. Shoring Locations, DNA No. 5002.

Fig. 3-9. Stud Wall Shoring and Wall Bracing, Pretest.

DNA No. 5002

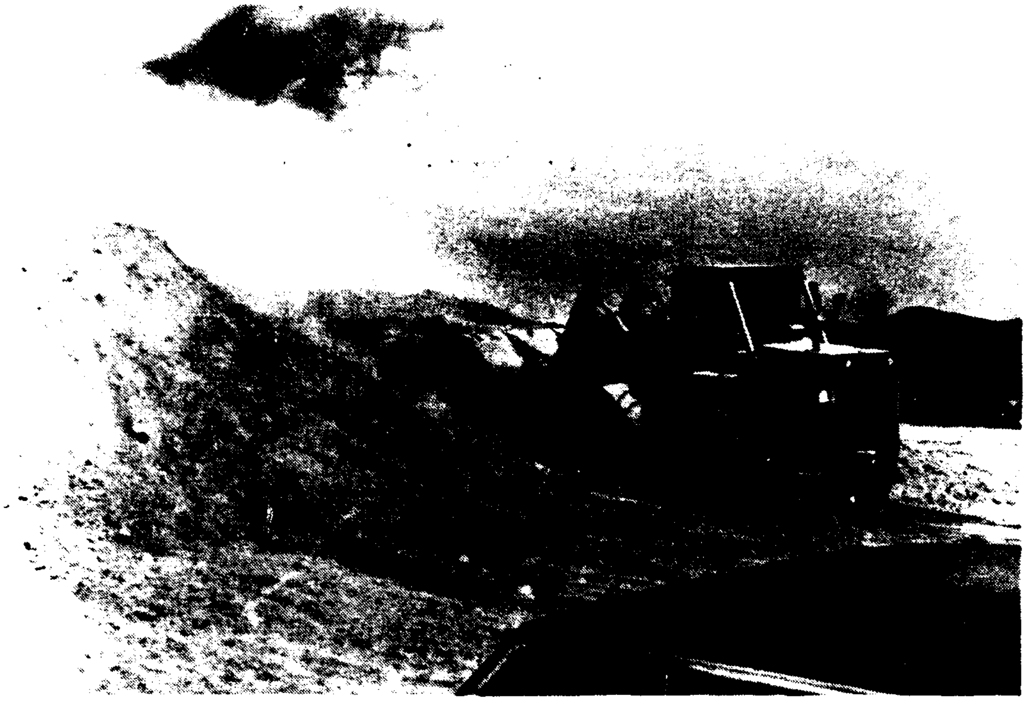


Fig. 3-10. Building Being Bermed.

Fig. 3-11. Building Completely Covered and Bermed Prior to Testing.

INSTRUMENTATION

The instrumentation for this experiment consisted of still photography.

TEST DATA

The peak free-field overpressure at this location, as recorded and supplied by DNA, was 1.9 psi.

As with Experiment No. 5001, fissures in the earth were noted near the tops of the berms around the perimeter of the building indicating that some movement had occurred. This earth movement was the only change observed on the exterior.

Six days after the test, one end wall of the building was cleared of earth and a door exposed. Figure 3-12 shows this exposed end wall with the door closure partially removed. An examination of the interior of the building did not indicate any structural distress in the building or the shores. No cracks were noted in either the brick or block walls, or in the concrete slab ceiling. The interior was clean and free of debris. Figure 3-13 is an interior view of the building after test, showing the ceiling slabs, wall bracing, and stud wall shoring.

DNA No. 5002

Fig. 3-12. Exposed End Wall With Door Closure Partially Removed.

Fig. 3-13. Interior View of Building, Posttest.

DISCUSSION AND CONCLUSIONS

The upgraded structure performed well in the 2 psi range, and would be expected to perform in a similar manner at that overpressure with a larger weapon. The same disadvantages as previously discussed under Experiment No. 5001 (see Section 2), although not as pronounced, were associated with the construction and installation of the upgrading system. The labor and materials required to perform these tasks were considered excessive.

As mentioned above, neither the concrete masonry nor the brick walls exhibited any cracking or deflection, thus indicating that the wall bracing system exceeded what was required. This bracing was designed, as in No. 5001, using the best available data, again reinforcing the need for research and testing in these critical areas.

The stud wall roof shoring performed well, as expected, and was probably approaching its load limitation. The design of this stud wall shoring system was, again as in No. 5001, taken directly from the Shelter Upgrading Manual: Host Area Shelters (Ref. 2).

The construction and installation of the wall bracing system required 4 man-days (32 hours), which appears to be excessive for this size and type of building. The majority of the time was again expended in sawing and cutting the lumber to size, and in securing the kick blocks to the floor and ceiling with explosively driven inserts.

As in No. 5001, the size of the door openings again precluded the prefabrication of the stud wall roof shoring outside of the building. However, in this experiment, the construction and installation was considerably easier since the wall bracing consisted of vertical members only, and not the diagonal type of braces used in No. 5001. Accordingly, much more floor space was available for laying out and constructing the stud walls, and less interference between the ceiling and wall upgrading systems was encountered. The time required to construct and install the stud wall shoring in this building was 1½ man-days as compared to the 2½ man-days required on No. 5001.

The window and door closures were identical to those used in No. 5001, and were constructed and installed in the same amount of time, 1½ man-days. It is again believed that more expedient types of closures are available.

The total time required for upgrading this building, excluding the berming and the covering of the roof with earth, was 7 man-days (56 hours). The wall bracing system accounted for four of those days, and again, appeared to be neither very expedient nor practical. Because of the type of construction and the absence of positive connections between the roof slabs and the walls, it is not believed that the roof performance was influenced by the performance of the walls. As mentioned previously, research and testing in the area of dynamic earth pressures on walls and wall bracing systems is recommended.

Section 4
DNA NO. 5003
HOST AREA SHELTER

INTRODUCTION

This experiment was the third of the three 24-ft-by-16-ft buildings located at the 2 psi environment. This was a basement structure of concrete block and wood frame construction. It contained one interior concrete block partition (which contained one door) running the full width at the center of the building. The exterior walls had no doors or windows. The floor above contained a hatch for entry and exit. The building was constructed on a concrete slab 7 ft below grade.

OBJECTIVE

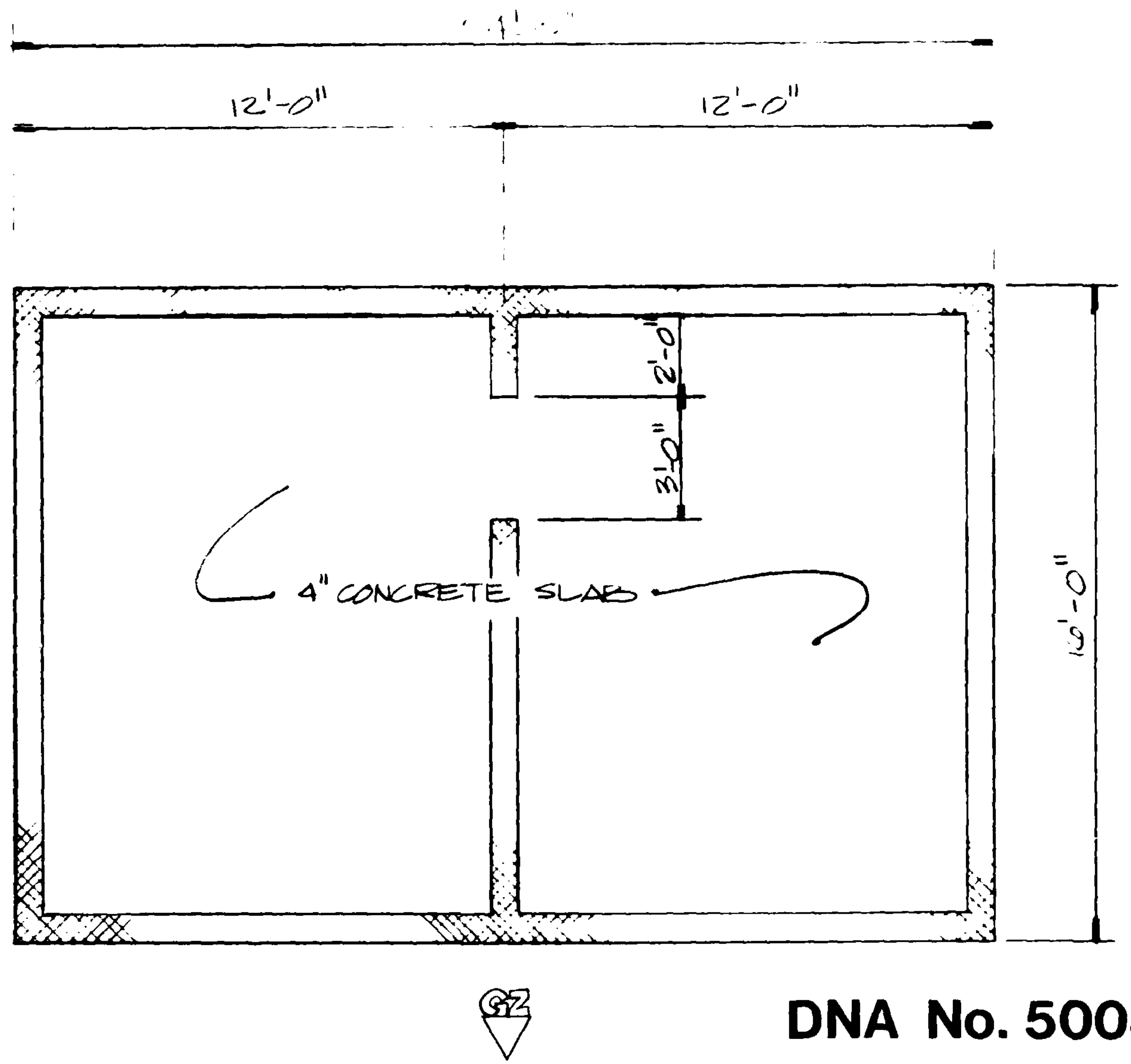
This structure simulated a typical concrete block basement. The basement walls were backfilled with granular backfill, and the floor above was covered with approximately 18 in. of earth to simulate fallout radiation protection equal to a PF of 100. The object of the experiment was to effectively close off the floor hatch and shore one-half of the floor to withstand the earthfill and the 2 psi overpressure. The other half of the floor above was not shored or upgraded in order to investigate and evaluate the degree and the mode of failure, and debris translation at this overpressure environment.

DESIGN

This structure was designed in accordance with the 1979 edition of the Uniform Building Code (Ref. 9), Chapters 24 through 26. The masonry walls were based on the minimum requirements for reinforced hollow unit masonry from Chapter 24, and the timber floor framing on the conventional framing requirements of Chapter 25. The concrete slab was designed in accordance with Chapter 26. The timber floor was designed to support 40 psf live load and 10 psf dead load in accordance with the code recommendations.

CONSTRUCTION

The exterior walls and the interior partition were constructed of hollow masonry units (standard concrete block, 8 in. by 8 in. by 16 in.) with all cells fully grouted. Each of the cells contained one No. 4 steel reinforcing bar, Grade 60, and continuous horizontal steel joint reinforcement was placed 16 in. on center, or every other horizontal joint. The lintel over the door in the interior partition contained two No. 3 steel reinforcing bars in each of the two courses above. The floor above consisted of 2-in.-by-8-in. timber joists, spaced 16 in. on center, and sheathed on the top with 5/8-in. thick plywood. The joists spanned one-half the length of the building, or 12 ft, and were supported at that location by the interior concrete block partition. The concrete floor slab, constructed at -7.00 ft elevation, was 4 in. thick throughout with a monolithically poured 12-in.-by-12-in. footing around the periphery. The design strength of the concrete at 28 days was 3,000 psi, and welded wire fabric (6x6-W1.4xW1.4) was located at the mid-depth of the slab. The concrete block walls were secured to the foundation with No. 4 steel reinforcing bars at 8 in. on center, located so as to lap the main vertical steel in each block cell. The sill plates, which supported the timber roof, were connected to the block wall with two 1/2-in. diameter steel anchor bolts per wall embedded 18 in. into the grouted cells. The floor hatch, which was built into the timber floor, provided a clear 4 ft 0 in. by 3 ft 9-3/4 in. opening for access. The finished floor was at an elevation of +2.14 ft; the completed building measured approximately 16 ft by 24 ft, with an interior height of 8 ft. All four of the exterior walls were backfilled to grade level with granular material. Figure 4-1 shows the basement plan, Figure 4-2 the roof framing plan of this building. Figure 4-3 is an elevation of the building. Figure 4-4 is a section through an exterior wall. Figure 4-5 is a view looking down into the excavation prior to placement of the concrete floor slab, and Figure 4-6 shows the grouting of the concrete block wall.



DNA No. 5003

Fig. 4-1. Basement Plan, DNA No. 5003.

DNA No. 5003

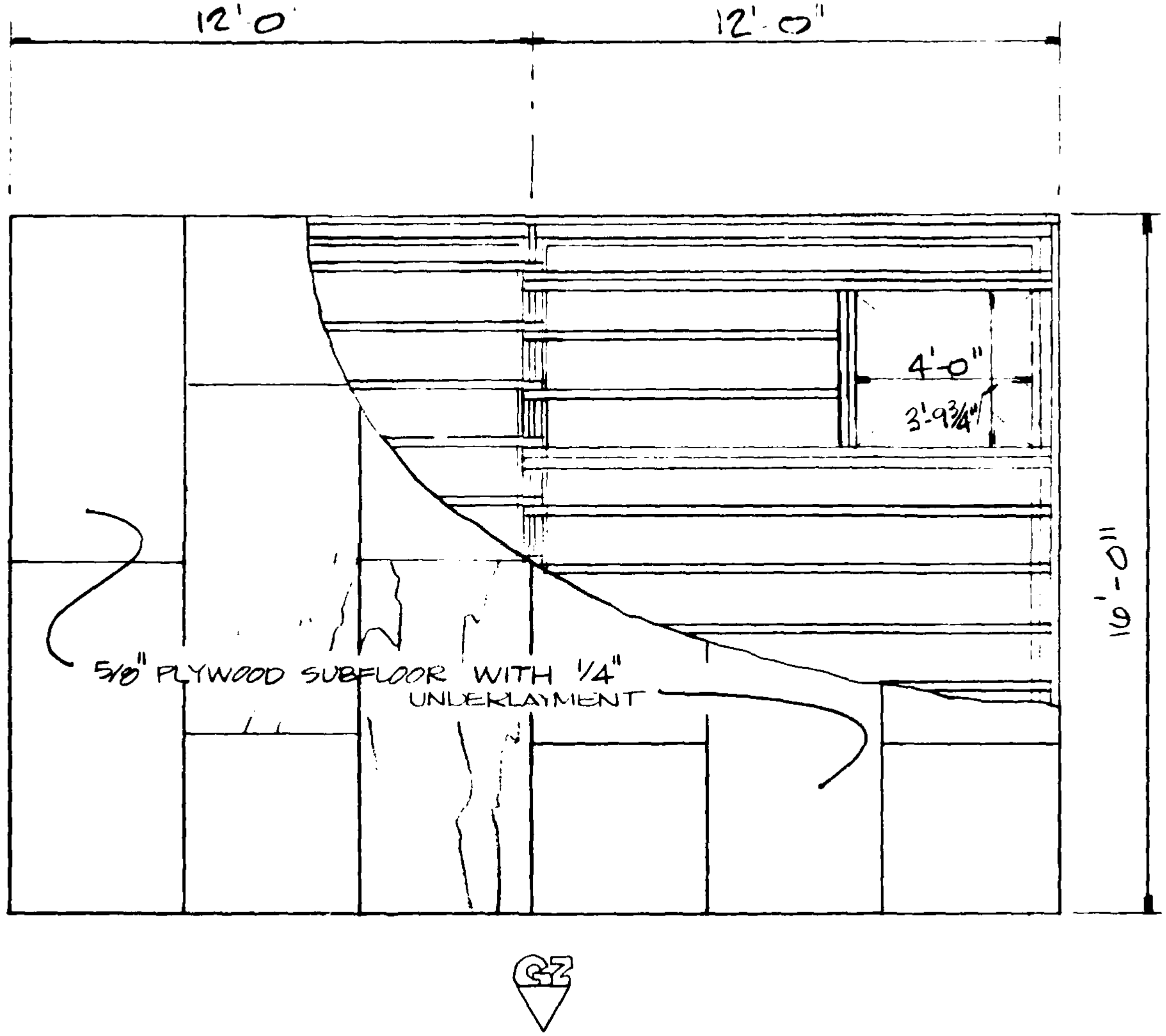
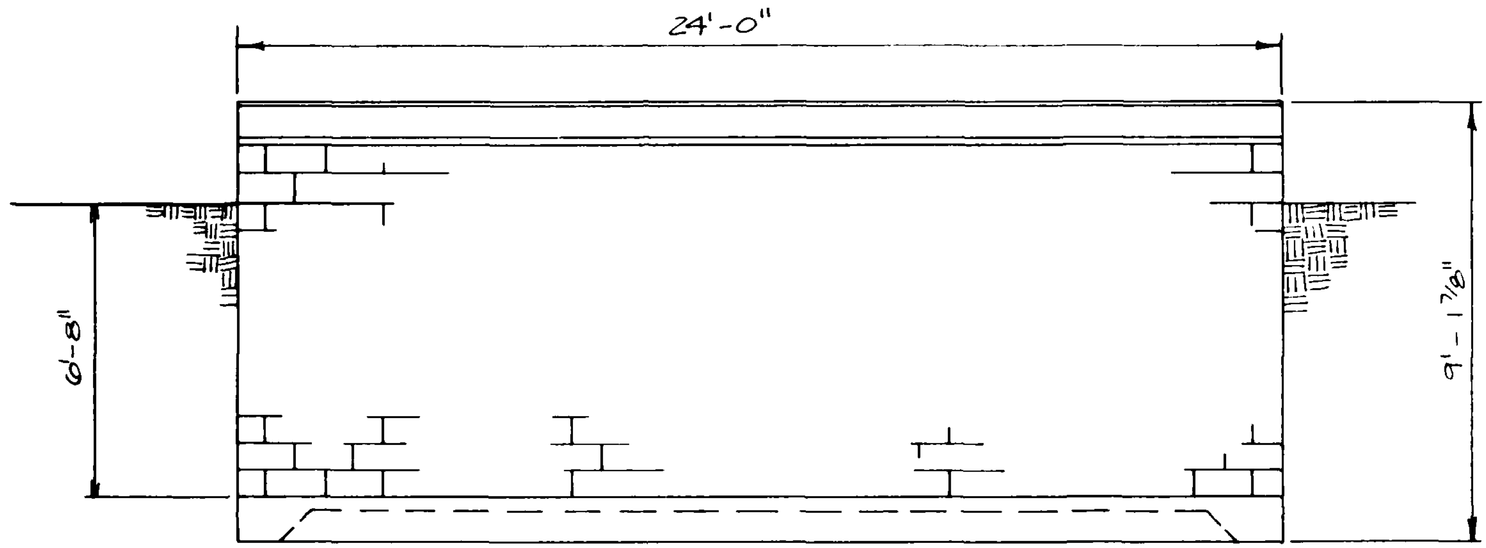


Fig. 4-2. Floor Framing Plan, DNA No. 5003.



DNA No. 5003

Fig. 4-3. Elevation, DNA No. 5003.

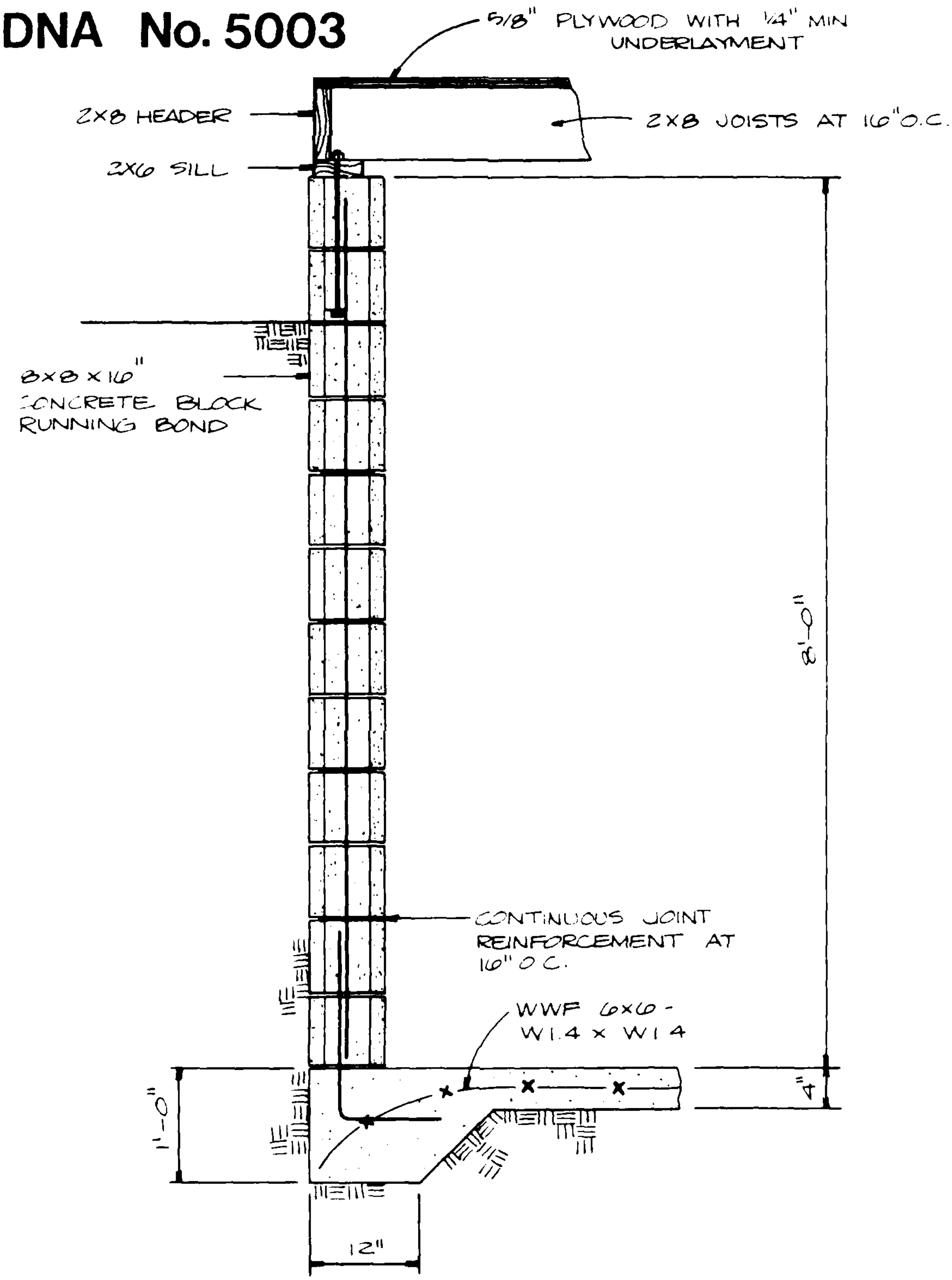


Fig. 4-4. Section Through Exterior Wall.

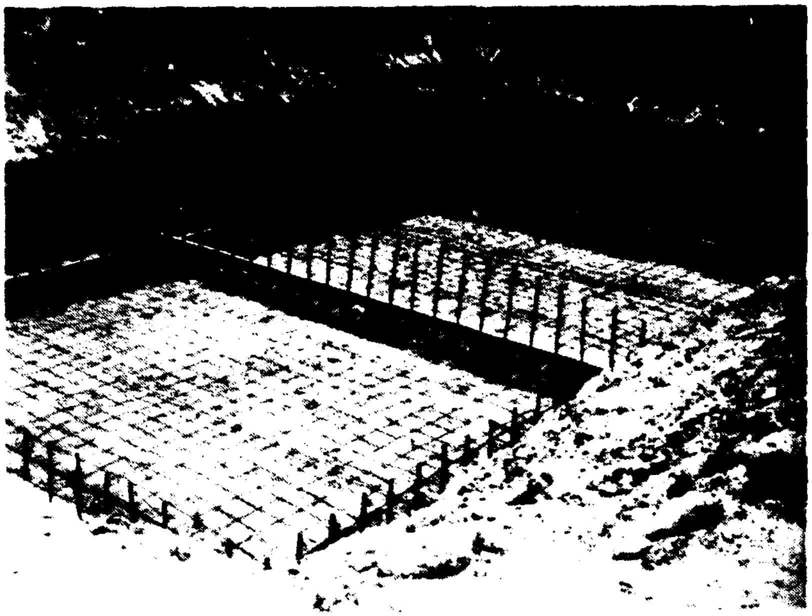


Fig. 4-5. View of Excavation Prior to the Placement of the Concrete Floor Slab.

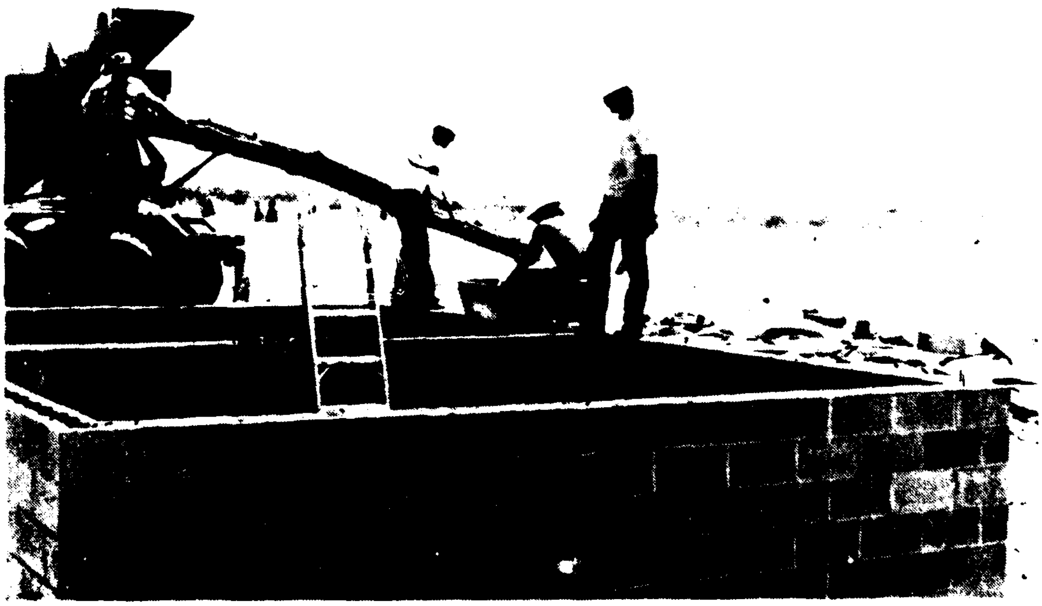


Fig. 4-6. Grouting of Concrete Block Wall.

UPGRADING

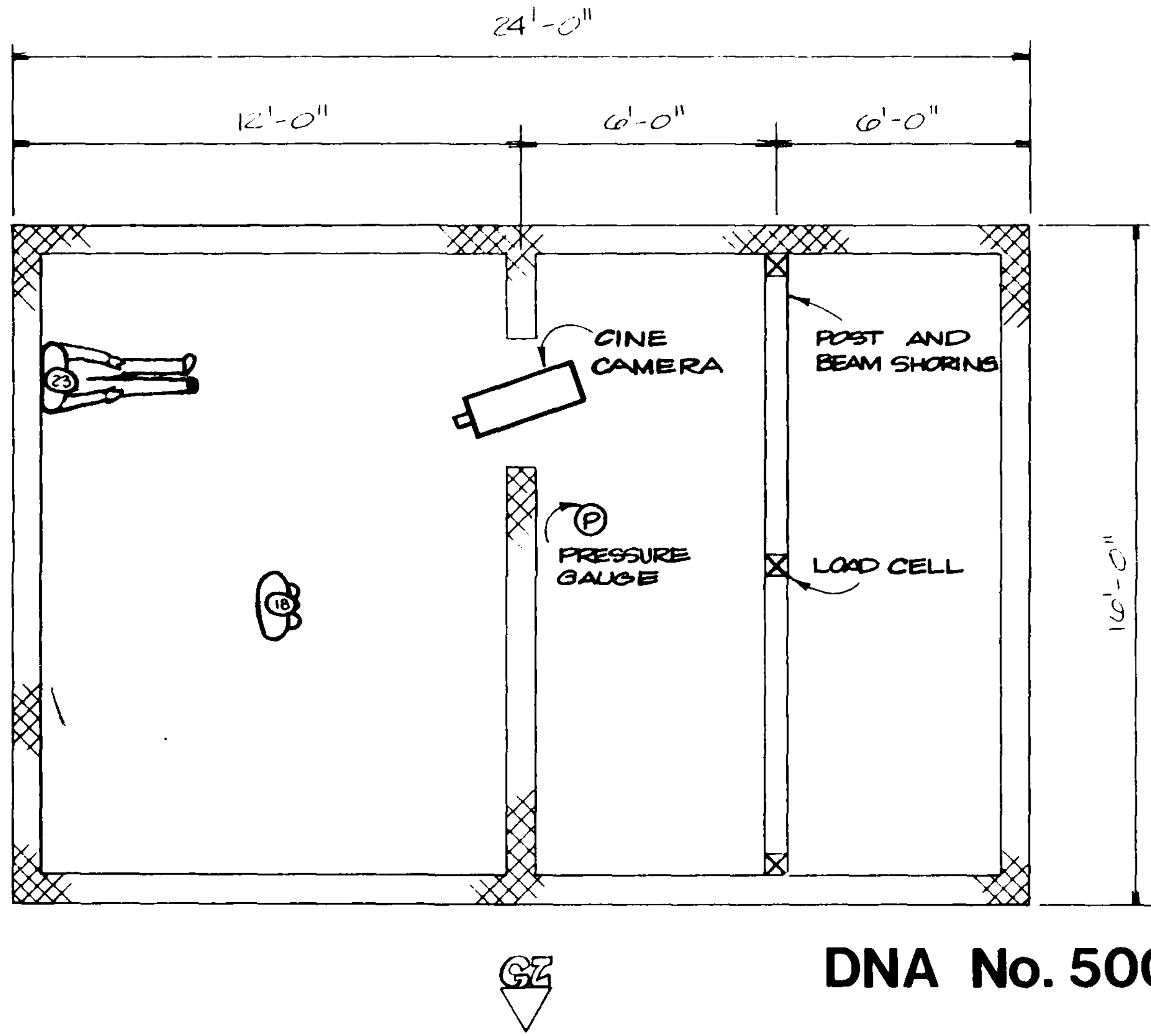
As mentioned above, only one-half of the structure was upgraded. The upgrading of the floor was accomplished by one line of posts and beams extending the full width of the building, 16 ft, and positioned at the midspan of the 12-ft timber joists. Two 8-ft long, 6-in.-by-10-in. timber beams were used to span the width, and were supported by three 6-in.-by-6-in. timber posts. The extreme ends of each beam were supported by a post at the walls, and one post, located at the center of the room, was used to support the other ends of the beams. A standard metal post/beam connector was used to secure the ends of the beams to the center post. Figure 4-7 is the basement plan and shows the location of the post and beam shore. Figure 4-8 shows the post and beam in position. Figure 4-9 shows the building being backfilled.

Soon after the backfilling operation was completed, it was noticed that seven of the 2-in.-by-8-in. timber joists in the unshored section of the building had failed. These failed joists were replaced prior to the test. Figure 4-10 shows the failed portion of the ceiling prior to repair.

The reinforced masonry walls did not require upgrading to withstand the backfill or the 2 psi environment.

The entry hatch was closed off with a two-section cover, which was constructed of 2-by-4-in. timber and 3/4-in. thick plywood and secured in place by covering with sandbags. Figure 4-11 shows the hatch cover partially in place just prior to the test.

The sizes of the posts and beams and their locations were obtained from the Shelter Upgrading Manual: Host Area Shelters (Ref. 2).



DNA No. 5003

Fig. 4-7. Post and Beam Shoring and Instrumentation Locations.

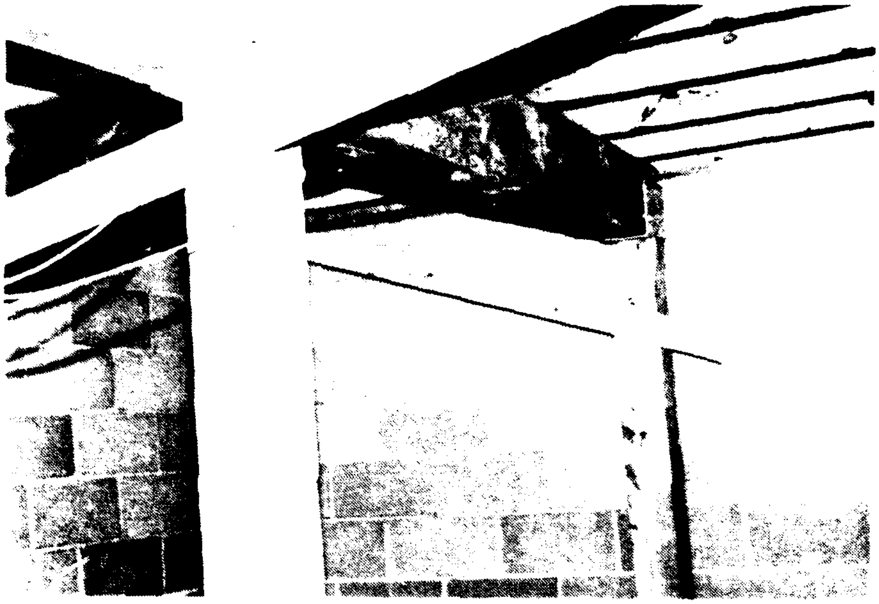


Fig. 4-8. Posts and Beam in Position.

Fig. 4-9. Backfilling of Building.

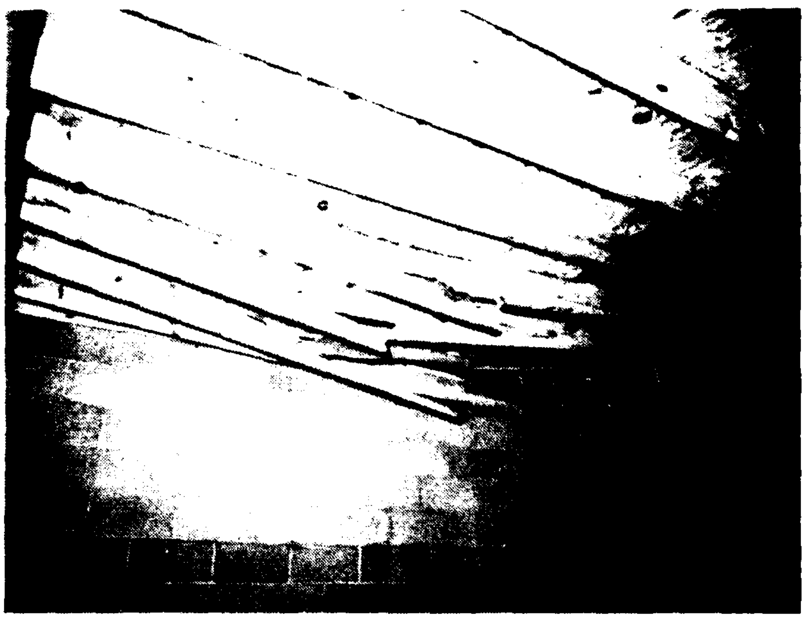


Fig. 4-10. Pretest Joist Failures.

Fig. 4-11. Hatch Cover Partially in Place, Pretest.

INSTRUMENTATION

The instrumentation utilized on this experiment is described below:

Pressure Gauge

One pressure gauge was installed in the interior of the structure to measure and record the overpressure level. The gauge was located on the floor, as shown in Figure 4-7. This gauge, a PCB Model 101A02, was used in conjunction with a PCB Model 494A06 amplifier.

Load Cell

One load cell was located at the bottom of the center 6x6 post, see Figure 4-7, in the upgraded portion of the building for the purpose of measuring the loads transmitted to this timber post shore. The load cell, Model 3500-200, was manufactured by Houston Scientific, Inc., and had a 200 kip capacity and was amplified by an LM 741C op-amp.

Photography

One cine camera was located in the interior doorway of the building, and oriented toward the non-upgraded portion. The location of this camera is shown in Figure 4-7. The camera was positioned 6 ft above the concrete slab floor to cover the ceiling and the entire opposite wall, which contained reference targets. The camera was installed and operated by personnel under the direction of DNA; it was a 16 mm camera with a 5.3 mm lens, operated at a speed of 400 frames per second. The coverage was from -5 to +12 seconds with a light period of -1 to +10 seconds. The cine photography was supplemented by still photography.

Anthropomorphic Dummies

Two anthropomorphic dummies, each with a self-recording accelerometer, were in the unshored portion of the building, located in the positions shown in Figure 4-7. One was held in a standing position near the center of the bay, while the other was seated against the exterior wall. Figure 4-12 shows the standing dummy (18) and Figure 4-13 shows the one seated (23).

Fig. 4-12. Standing Anthropomorphic Dummy No. 18.

DNA No. 5003

Fig. 4-13. Seated Anthropomorphic Dummy No. 23.

TEST DATA

The peak free-field overpressure at this location, as recorded and supplied by DNA, was 1.9 psi.

The initial observations of the building after the test indicated complete collapse of the unshored bay, and very little structural distress in the shored bay. The basement walls sustained no structural damage and no cracks were in evidence. The closure hatch, which was located in the shored bay, had been blown off, scattering both hatch section and the sandbags. Figure 4-14 shows the hatch after test. Figure 4-15 shows the failed bay looking toward the unfailed bay.

In the foreground of the lower photograph of Figure 4-15, completely covered with earth and debris, is one of the anthropomorphic dummies (18). This dummy was entirely covered to an average depth of 3.5 ft with earth and roof materials. It was found lying face down on the floor with its head near the pretest location, its feet and body displaced toward ground zero, and its legs bent forward at the knees. It had sustained multiple lacerations and extensive crushing injuries, including the breaking of both knee joints. The accelerometer reading indicated 10 g.

Figure 4-16 shows views of the failed bay looking down toward the exterior wall. Both of these figures show dummy No. 23. Figure 4-17 is a closeup of dummy No. 23. The location of this dummy was the same as pretest, with earth and roof materials covering both legs to an average depth of 1.5 ft. The dummy was found to have sustained two deep lacerations on the head, the chest was crushed, and both legs were broken at the knee joints. The accelerometer reading on this dummy also indicated 10 g.

Close inspection of the interior of the shored bay indicated that two of the timber floor joists had cracked. These two joists had deflected down a maximum of 15/16 in. from their initial pretest position. The other floor joists were in good condition, exhibiting no structural distress. With the exception of the cracked joists, the entire ceiling system was deflected from its original position a maximum of 5/16 in., with approximately half of the joists indicating zero displacement.

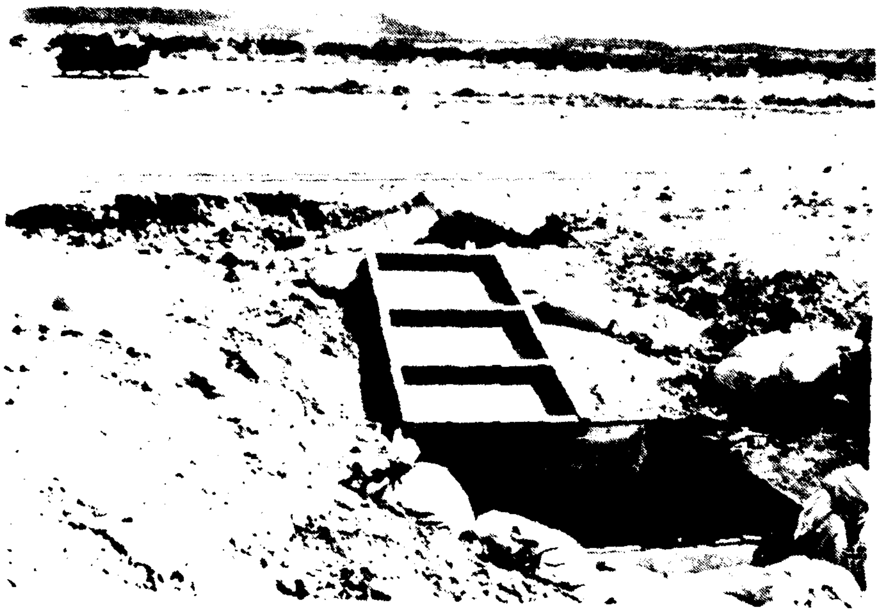


Fig. 4-14. Hatch, Posttest.

DNA No. 5003

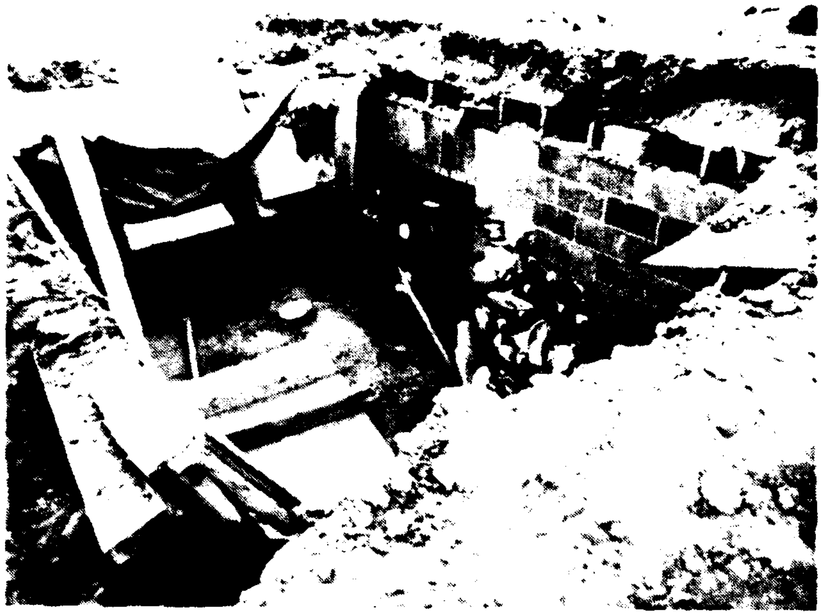


Fig. 4-15. Views of Failed Bay Looking Toward Unfailed Bay.

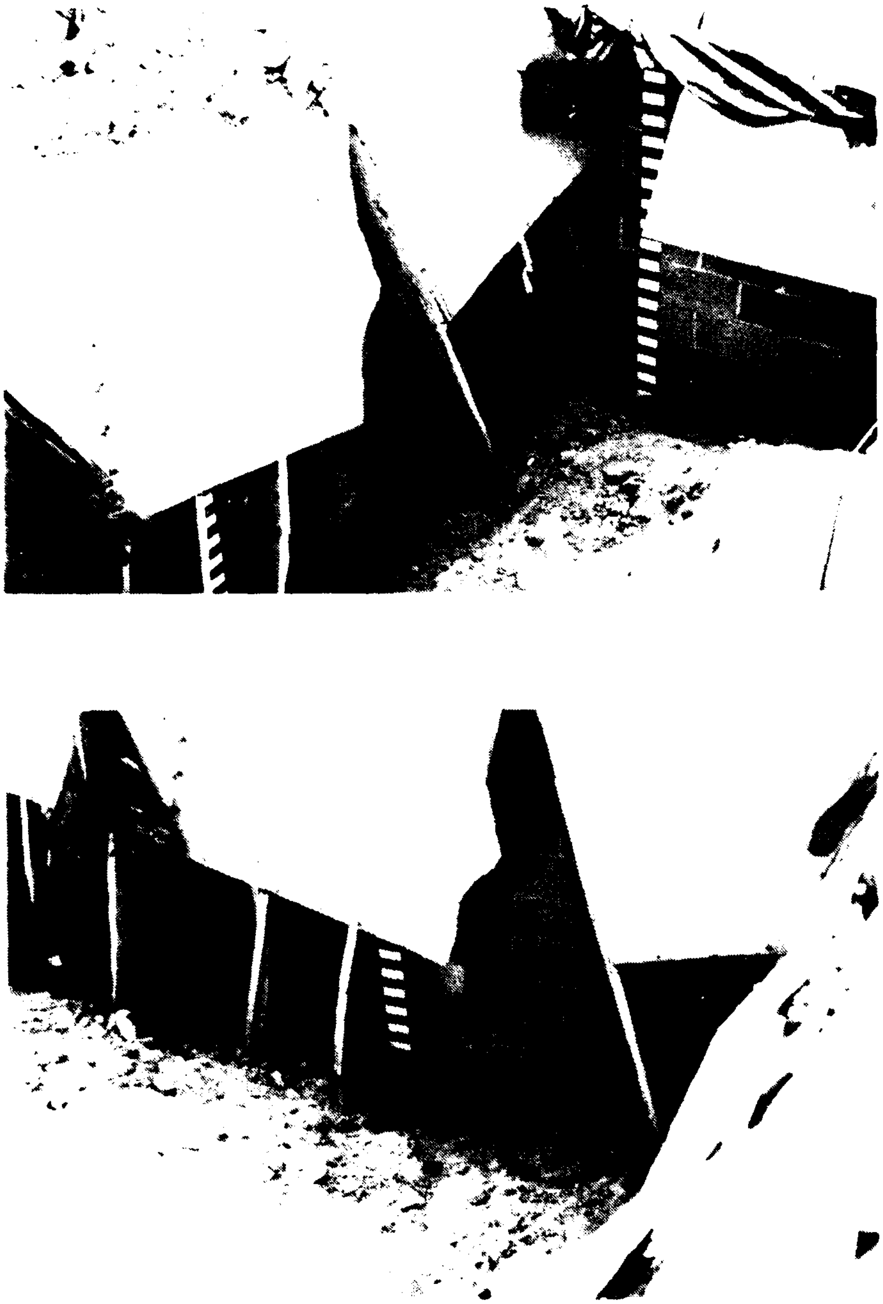


Fig. 4-16. Views of Failed Bay Looking Toward Exterior Wall.

DNA No. 5003

Fig. 4-17. Closeup of Anthropomorphic D . No. 23, Posttest.

The post and beam shore was completely intact and indicated no distress. The maximum permanent deflection noted for the timber beam shore was 1/8 in.

The interior of the uncollapsed shored bay was lightly covered with dust and dirt. Since there was no closure in the doorway between the shored and unshored bays, most of this material undoubtedly entered during the collapse of the adjacent bay.

The pressure gauge, which was located on the floor in the interior of the shored section of the building (see Figure 4-7 for location), recorded a positive pressure of approximately 0.7 psi with a rise time of 300 ms, immediately followed by a rapid decrease to a negative pressure of approximately 0.6 psi in 80 ms.

Owing to an equipment malfunction, no data were obtained from the load cell. Because the lighting system malfunctioned, no film data were recorded prior to and during collapse.

DISCUSSION AND CONCLUSIONS

The upgraded portion of the building performed well in the 2 psi range, and would likely be adequate at this overpressure with a larger weapon. The construction and installation of the post and beam shoring was practical and expedient. The design of this upgrading system was taken from the Shelter Upgrading Manual: Host Area Shelters (Ref. 2). The total time expended in upgrading this half of the building was 1 man-day (8 hours); it is estimated that the entire building could have been upgraded with this type of shoring system in 1½ man-days (12 hours). The heavy dimension timbers used as the posts and beams may not be a readily available resource during a crisis, and further investigation into alternative types of posts and beams, possibly using lighter and more easily installed members that are more available, is recommended.

An inspection of the non-upgraded portion of the building just prior to the test indicated that the floor system was barely able to support the load of the 18 in. of earth. Several of the floor joists above were cracked and all were badly deflected, The floor would have been judged unsafe by all criteria. The loading from the earth was approximately three times the design load of the floor.

The basement walls did not evidence any cracks or structural distress and, as mentioned previously, were not upgraded. The judgment not to shore the walls was based on conventional static analysis of earth pressure against basement walls. Unfortunately, the load applied to the walls during the test by the overpressure surcharge was not measured, and further investigation in the area of dynamic earth pressure on basement walls is recommended.

The closure over the hatch was not secured adequately to resist the positive interior pressure created by the failure of the adjacent bay. As in many of the experiments, this hatch was expedient and designed to permit ease of access immediately prior to the test. In order to perform as a viable shelter, it would be necessary to secure the hatch from the inside, as well as close off the doorway to the unshored bay.

Section 5

DNA NO. 5201

KEY WORKER SHELTER

INTRODUCTION

The structure for this experiment consisted of a building 151 ft long and varying in width from 16 ft to 18 ft. It was a concrete basement structure, and was located in the 40 psi environment, 580 ft from ground zero. The long dimension of the building was perpendicular to the radial. The building was divided into three 18-ft bays, and three sets of two 16-ft bays, each separated by a reinforced concrete wall. The floor above consisted of different construction types, and was upgraded with various shoring types and configurations. The exterior and interior walls, with the exception of a portion of one corner, which was used to evaluate three different basement wall constructions, were designed to be "non-failing" in order protect the integrity of the floor upgrading portion of the experiment. The exterior walls contained no openings. The floor above contained one hatch and one stairway for access. Figure 5-1 shows a plan and elevation of the structure.

OBJECTIVE

This structure simulated the basements of buildings used for heavy manufacturing. Basements in buildings designed for these uses would lend themselves to the sheltering of key workers. The specific objectives of this experiment were as follows:

(1) To evaluate various upgrading methods under a 40 psi environment and fallout radiation protection equal to a PF of 100 (18 in. of earth) for different types of concrete floors.

(2) To evaluate the performance of various types of typical basement walls under soil loadings created by blast.

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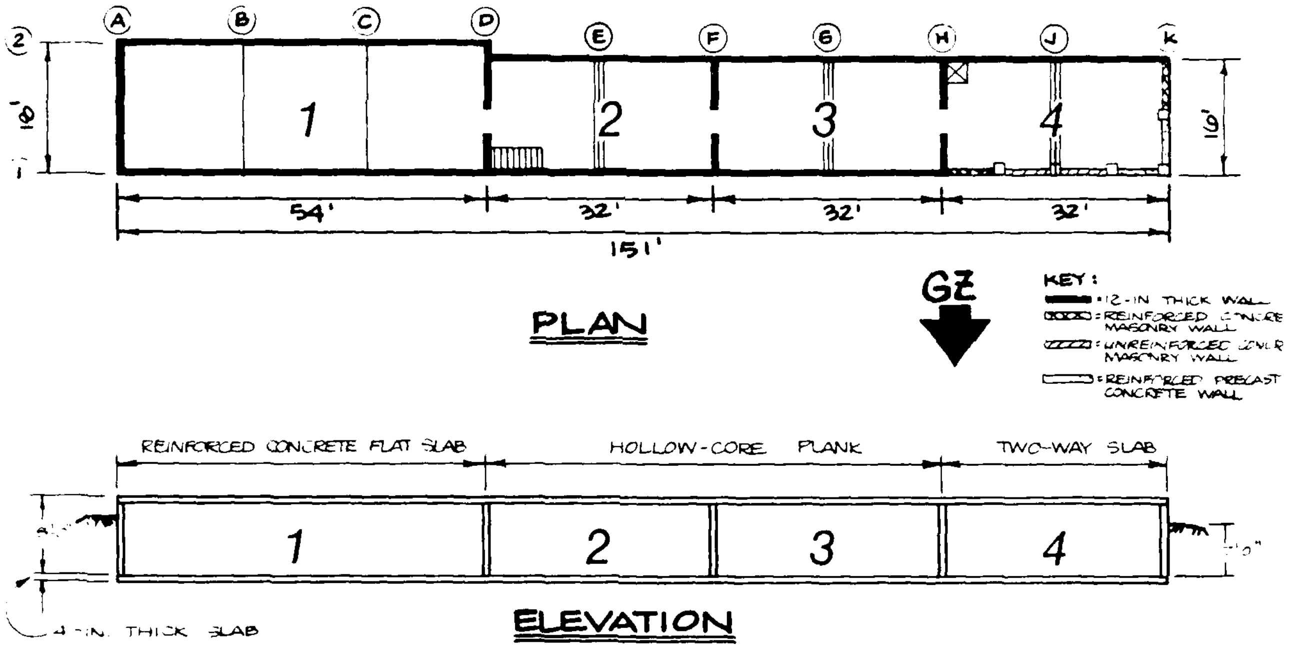

Fig. 5-1. Plan and Elevation o. Structure, DNA No. 5201.

(3) To obtain information on methods and materials required to upgrade basement walls.

(4) To observe and evaluate the performance and interaction of various building components, many of which have been individually tested, when they are tied together and made continuous in a typical structure.

DESIGN

The structure was designed in accordance with the 1979 edition of the Uniform Building Code (Ref. 9). The reinforced concrete slabs, walls, beams, and columns were designed in accordance with Chapter 26 and the 1978 edition of the CRSI Handbook (Ref. 11). The precast prestressed concrete slabs and beams were designed in accordance with Chapter 26 of the UBC and Chapter 18 of the Building Code Requirements for Reinforced Concrete (ACI 318-77), Ref. 10. The masonry test walls were based on the minimum requirements for reinforced and unreinforced hollow unit masonry from Chapter 24 of the UBC. The floor above was designed to support 125 psf live load in accordance with code recommendations.

CONSTRUCTION

As shown on Figure 5-1, the structure is divided lengthwise into bays, three are 18 ft by 18 ft (Area No. 1) and six are 16 ft by 16 ft (Areas No. 2, 3, and 4). The concrete footing, which was under all of the walls, was 2 ft wide and 12 in. deep, and was reinforced with two No. 6 bars longitudinally and No. 6 bars transversely at 10 in. on center. The top of the concrete slab was at an elevation of -7.00 ft and coincided with the top of the footing. The slab was 4 in. thick throughout and reinforced with 6x6-W1.4xW1.4 welded wire fabric located at mid-depth. A section of the slab on grade being finished is shown in Figure 5-2, with the vertical wall reinforcing steel protruding from the footings.

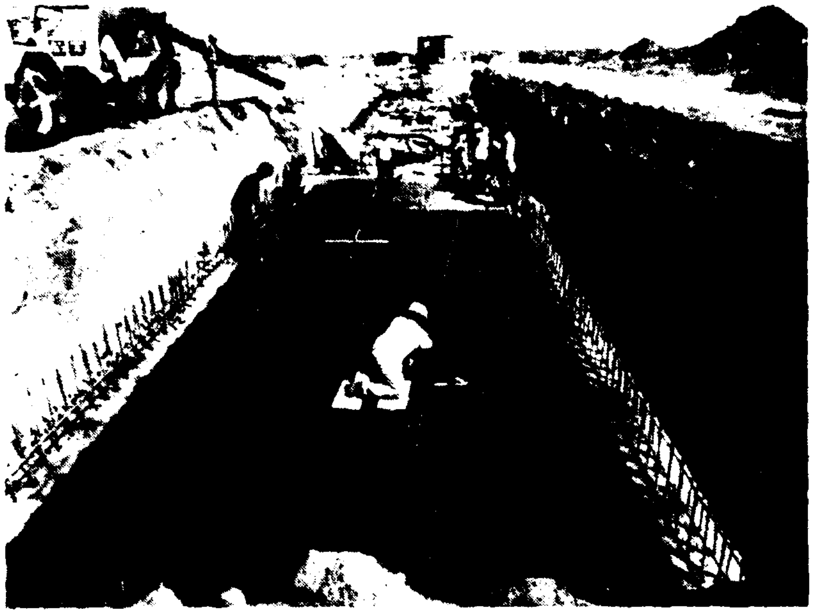

Fig. 5-2. Slab on Grade Being Finished.

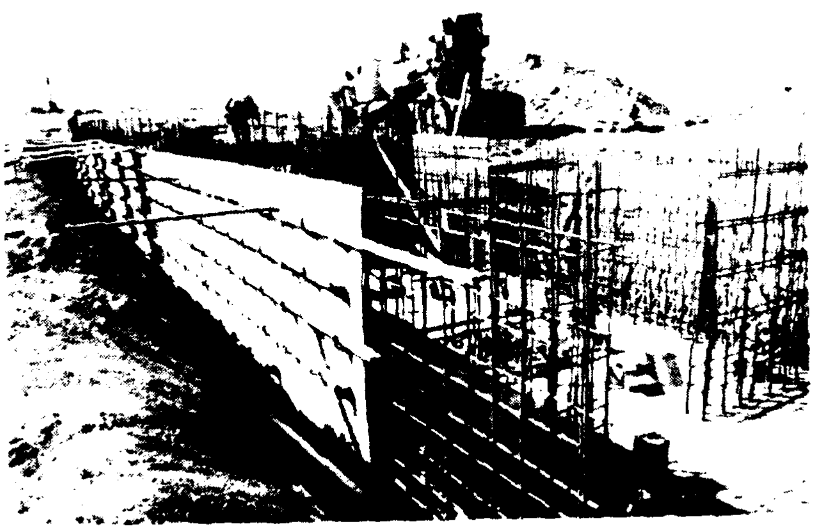

Fig. 5-3. Reinforcing Steel and Forms for Walls.

The walls, except the test walls (see below), were all 12-in. thick concrete, reinforced on both faces with No. 6 bars 10 in. on center vertically, and 18 in. on center horizontally. Figure 5-3 shows the reinforcing steel and forms for the wall being erected. These walls were tied to the footing with No. 6 bars from the footing lapping the vertical steel in the wall. The walls (see Figure 5-1) were located along line 1 from line A to H, along line 2 from line A to K, and from lines 1 to 2 on lines A, D, F, and H. Pilasters were constructed on line 1 at H.5, J, J.5 and K, and on line K at 1.5 in order to accept the test walls that were located in these areas. The pilasters were cast with six No. 6 vertical bars and No. 3 ties at 6 in. on center. Doors in the walls on lines D, F, and H were 3 ft wide and 6 ft 8 in. high.

Three different types of concrete construction were used for the floor above. **In Area No. 1** (the three bays from lines A to D in Figure 5-1), the floor above was a reinforced concrete flat slab, 8 in. thick with 2-in. drop panels, and was designed as interior bays (Figure 5-4). **In Areas No. 2 and No. 3** (between lines D and H), the floor area consisted of 6-in. thick, 48-in. wide precast prestressed hollow-core plank (Figure 5-5). Each of the planks was approximately 16 ft long, spanned one bay, and was supported at lines E and G, where no wall existed, by precast reinforced concrete beams spanning from lines 1 to 2 and supported by corbels cast monolithically with the walls. The precast beams were inverted tees with a 20-in. wide, 10-in. high base and a 12-in. wide, 6-in. high stem. The beams were reinforced with five No. 8 bars tied with No. 4 stirrups. **In Area No. 4** (between lines H and K), the floor area was a 6-in. thick reinforced concrete two-way slab, supported at line J by a monolithically cast beam, 12 in. deep and 16 in. wide. The beam was supported at midspan by a 16-in. square column (Figure 5-6). Figure 5-7 shows the pilasters and columns in Area No. 4 formed prior to concrete placement. Figure 5-8 shows the concrete deck in Area No. 1 being placed, and Figure 5-9 shows the prestressed precast concrete planks partially in place.

Six test walls were constructed in Area No. 4 — all 8 ft high and 6 ft 3 in. to 6 ft 9 in. in width, so as to fit between the wall/pilaster, pilaster/pilaster or corner column/pilaster areas as required. Three types of wall construction were used: unreinforced hollow unit masonry, reinforced hollow unit masonry, and precast concrete wall panels. Two of each type were constructed. The two unreinforced

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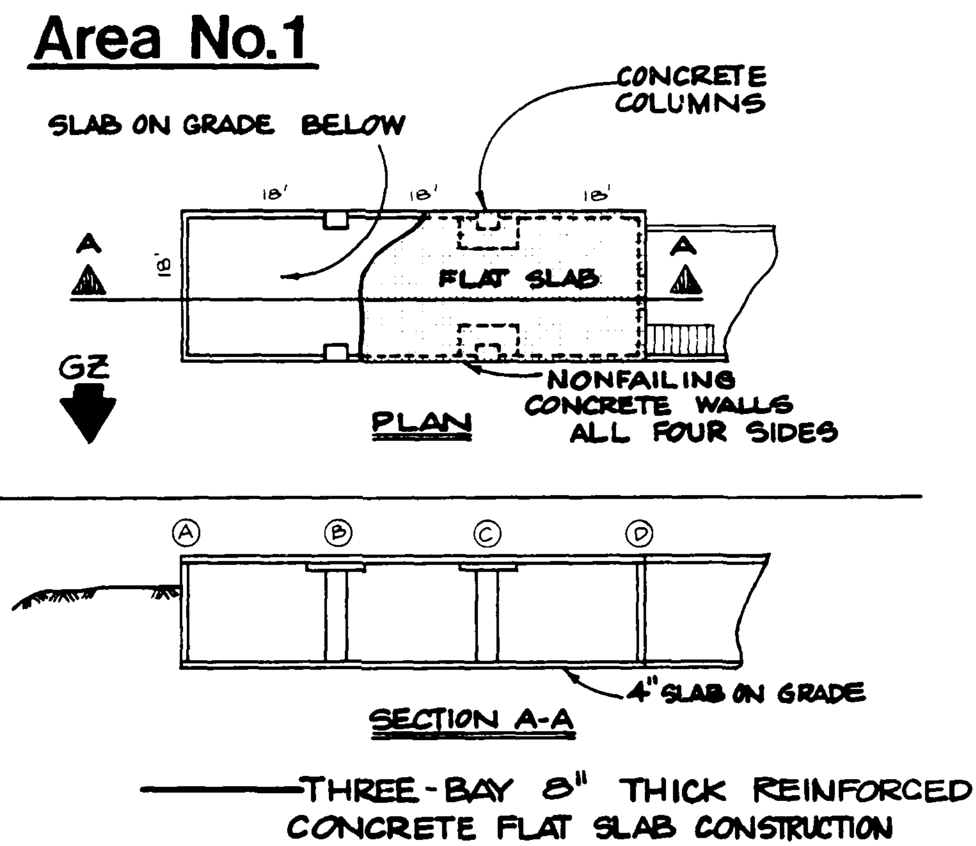

Fig. 5-4. Detail, Area No. 1.

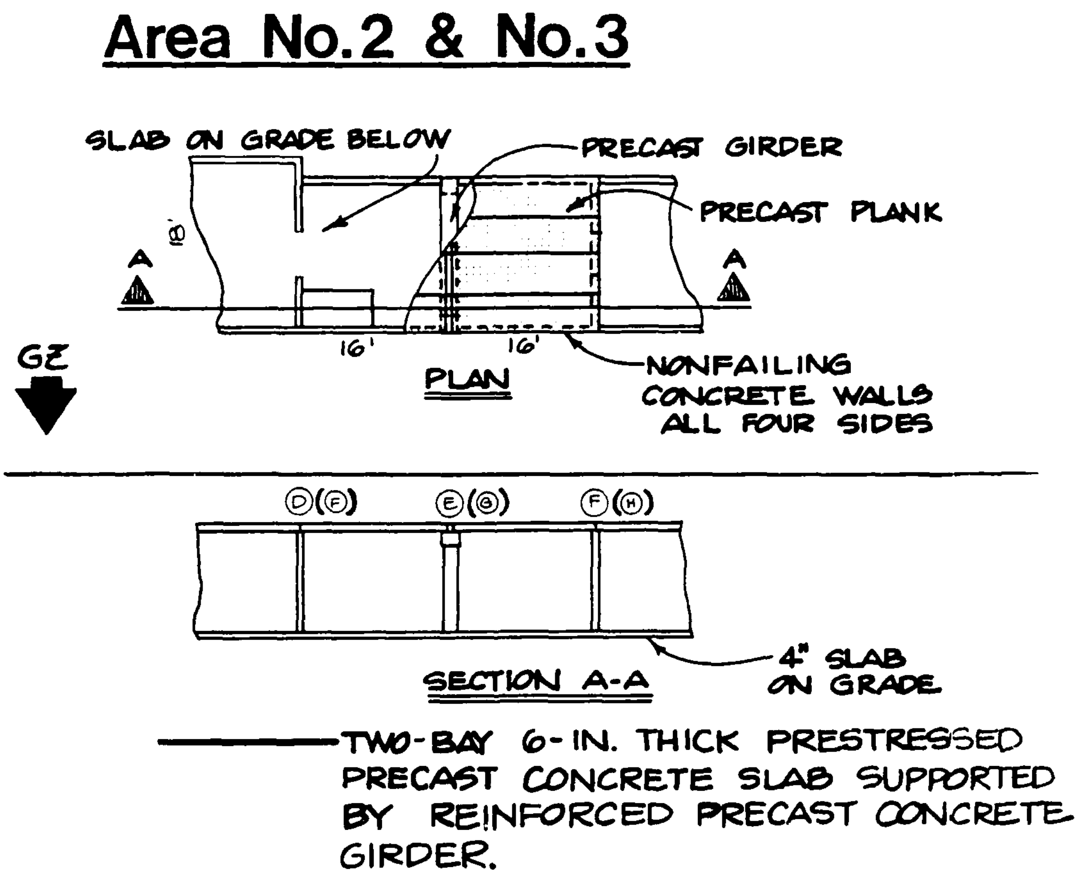

Fig. 5-5. Detail, Areas No. 2 and No. 3.

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Area No. 4

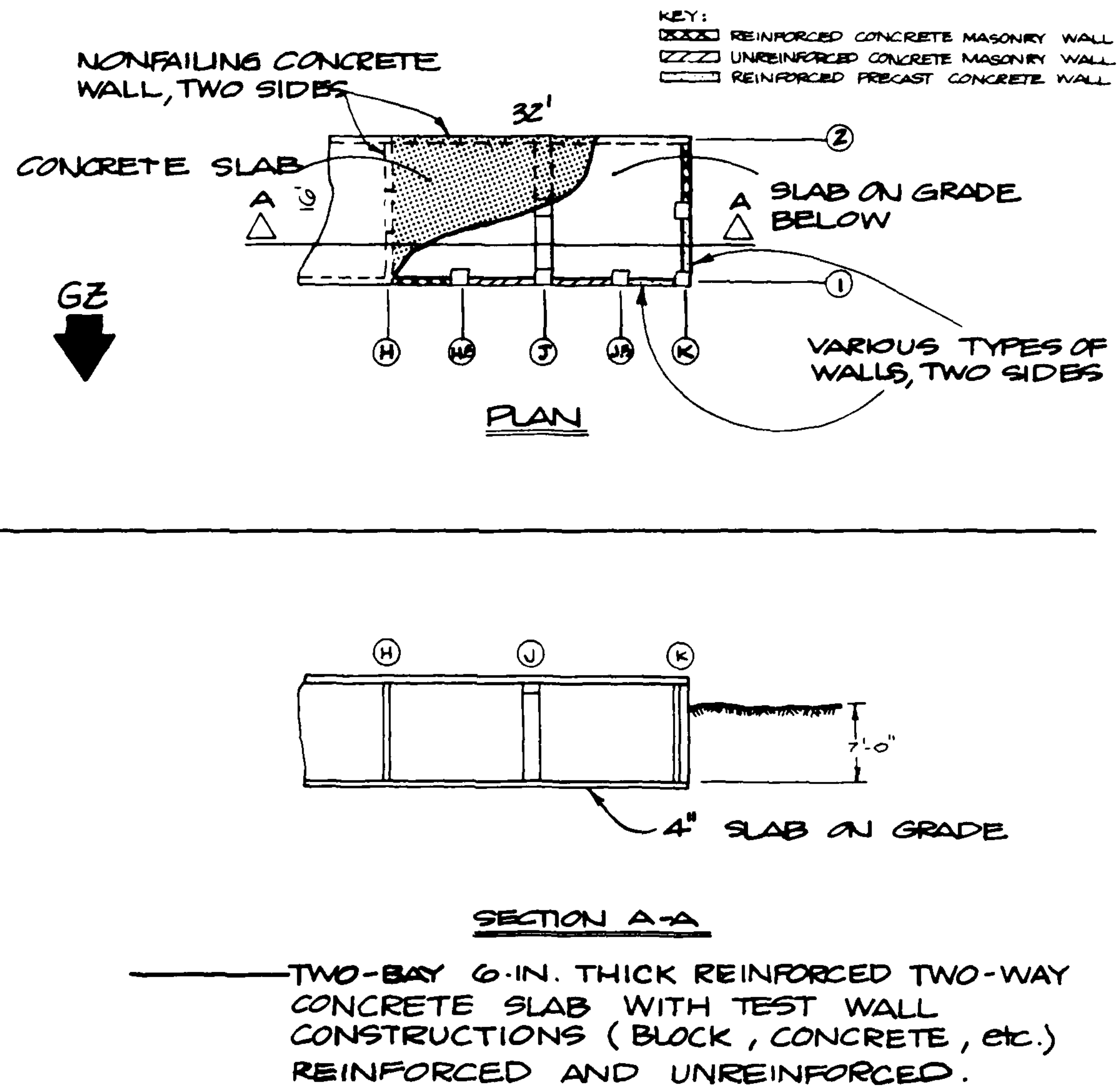

Fig. 5-6. Detail, Area No. 4.

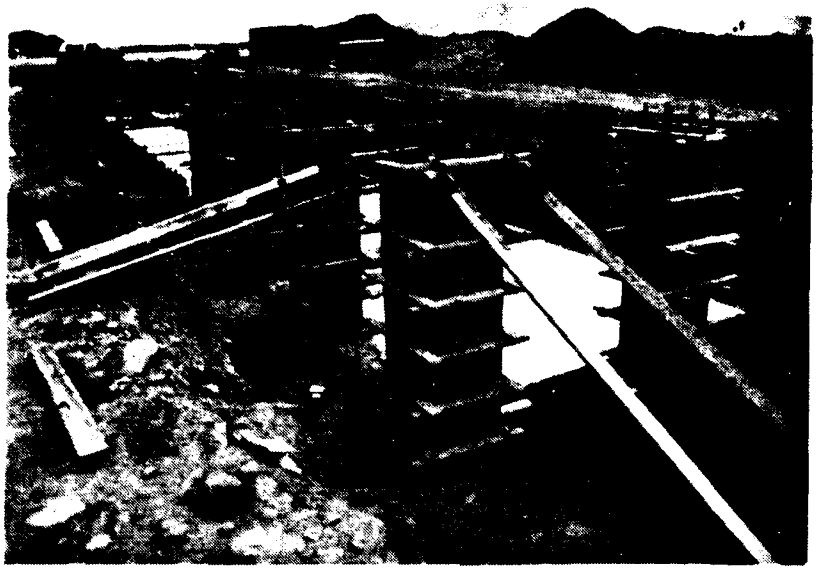

Fig. 5-7. Forms for Columns and Pilasters in Area No. 4.

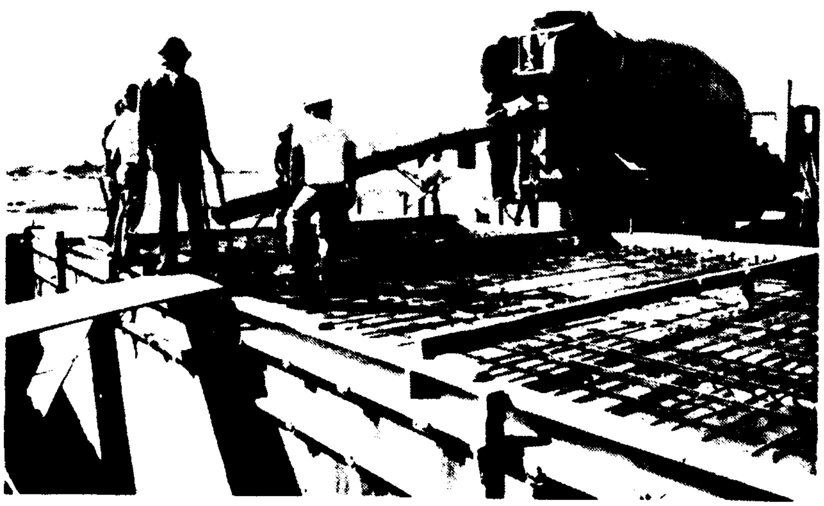

Fig. 5-8. Concrete Deck in Area No. 1 Being Placed.

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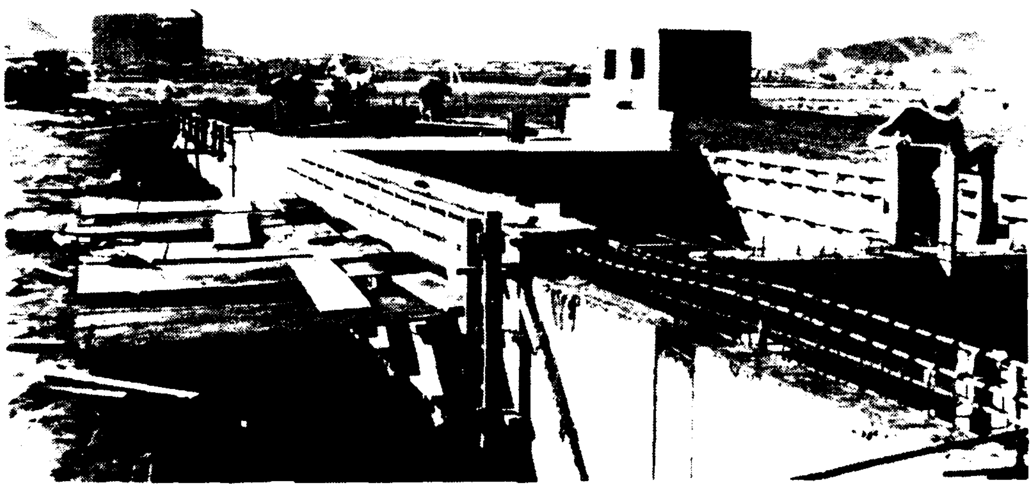

Fig. 5-9. Prestressed Precast Concrete Planks in Areas No. 2 and No. 3.

masonry walls were constructed of 8-in.-by-8-in.-by-16-in. block in a running bond pattern, ungrouted with only horizontal joint reinforcement 16 in. on center. The two reinforced masonry walls used the same size block and pattern, with all cells fully grouted. The reinforcing for these walls consisted of No. 4 bars vertical in every other cell (16 in. on center), and No. 4 bars horizontal in top and bottom bond beams and in the bond beams located every fourth course. The two precast wall panels were 8 in. thick and reinforced with No. 3 bars, 12 in. on center in each direction. The three types of test walls are illustrated in Figure 5-10.

All of the test walls were set into the openings so that the only lateral support was 4 in. of bearing at both the top and bottom, with an approximate 1/2-in. clear space provided the full height along both sides. The unreinforced masonry walls were installed on line 1, H.5 to J, and J to J.5. The reinforced masonry walls were located on line 1, H to H.5, and line K, 1.5 to 2. The precast panels were installed on line 1, J.5 to K, and line K, 1 to 1.5 (see Figure 5-6). Figure 5-11 shows the masonry walls under construction.

For access purposes a 4-ft-by-7½-ft stairwell was constructed in the floor in Area No. 2, adjacent to lines D and 1, and a 3-ft square hatch was located in Area No. 4 at lines H and 2.

All of the basement walls, including the test walls, were backfilled to grade with granular material. Figure 5-12 shows the completed structure partially backfilled, and Figure 5-13 the walls completely backfilled.

The design strength of the concrete in the poured-in-place walls, footings, basement slab, columns, and the flat slab and two-way floor slabs was 3,000 psi at 28 days. The precast elements, the prestressed floor planks, beams, and wall panels had a design concrete strength of 4,000 psi. Figure 5-14 shows the interior of Area No. 3 prior to upgrading.

The completed elevation of the top of the floor slab above was +1.67 ft in all bays.

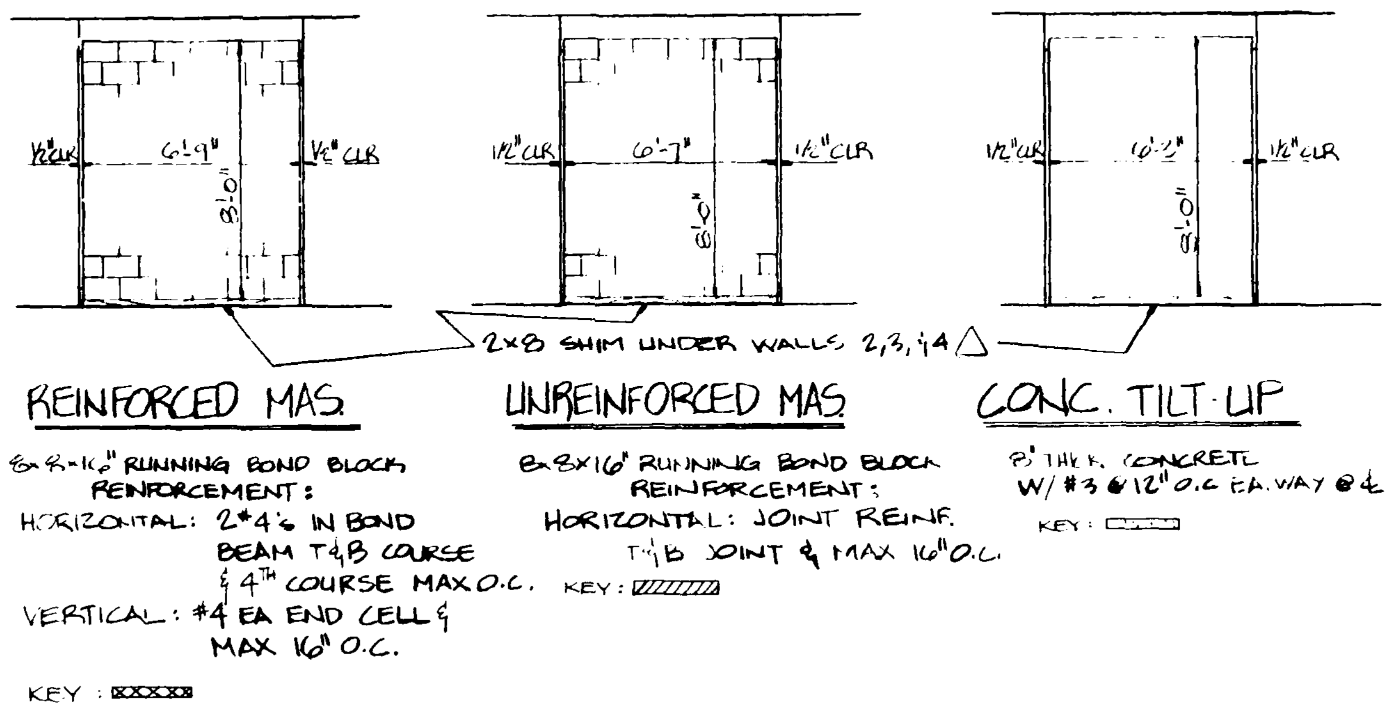

Fig. 5-10. Test Walls (see Figure 5-6 for test locations).

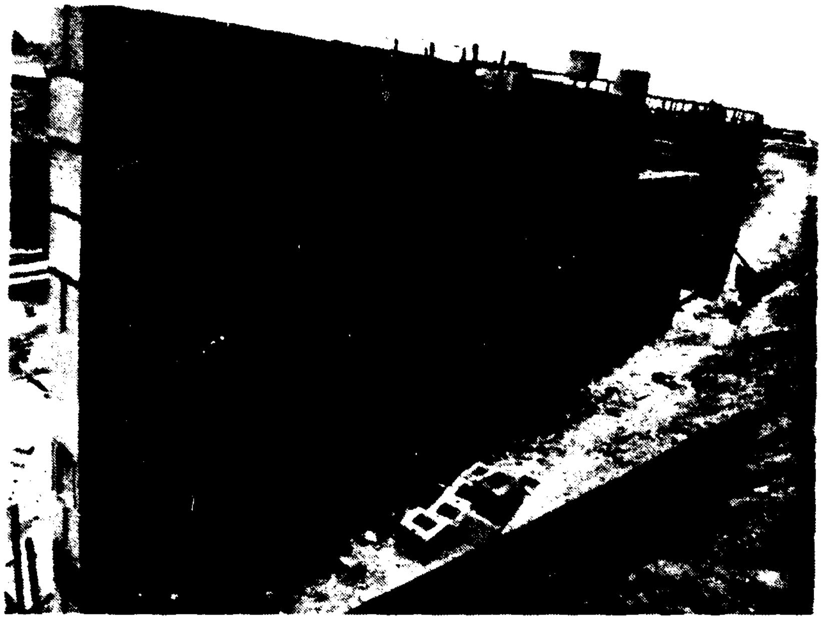

Fig. 5-11. Masonry Walls Under Construction.

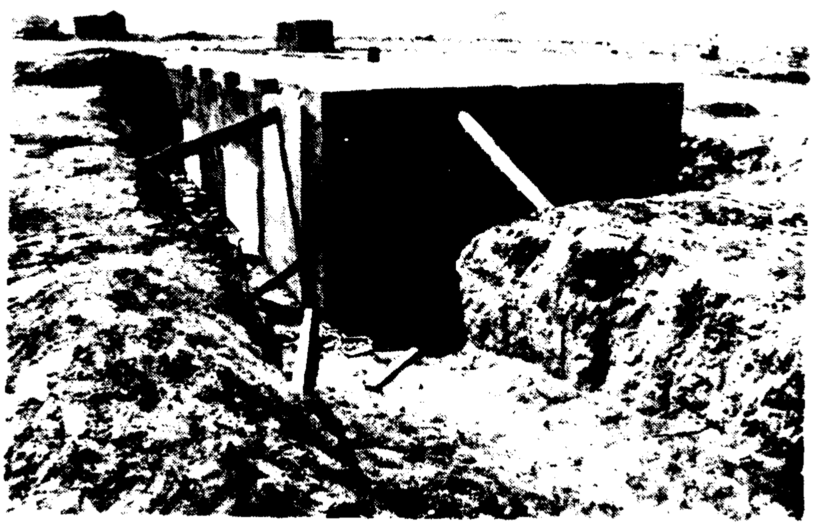

Fig. 5-12. Completed Structure Partially Backfilled.

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Fig. 5-13. Backfilling Completed.

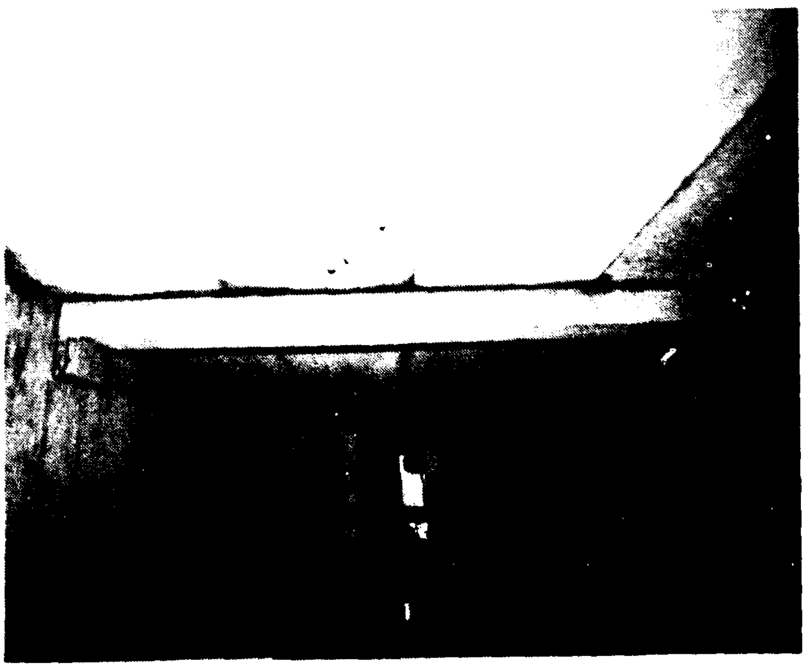

Fig. 5-14. Interior of Area No. 3 Prior to Upgrading.

UPGRADING

Area No. 1 - Flat Slab

The three 18-ft bays with the flat slab construction were shored with two different types of shores and in two different shoring configurations. In the first bay (from lines A to B) the shoring consisted of four sections of square structural steel tubing, TS 6x6x½ in. These four steel tubes were installed in a vertical position, 5 ft apart in both directions, symmetrically about the midpoint of this bay. This approximate one-third point spacing resulted in these shores being spaced 6 ft, 5 ft, and 6 ft in one direction, and 6 ft 3 in., 5 ft, and 6 ft 3 in. in the other. Each tube had 5/8-in. thick, 8-in. square, steel cap plates tack welded to each end. A piece of plywood, 1 in. thick and 12 in. square, was placed under the bottom cap plate, and wood shims (shingles) were driven between the cap plate and the plywood and/or the cap plate and the ceiling slab to secure the shore tightly between the floor and the ceiling.

The other two 18-ft bays (from lines B to D) were shored throughout at the quarter points with 7½-in. minimum diameter timber telephone poles. The spacing in both directions was 4 ft 3 in. on center. Plywood plates, 1 in. thick, were located beneath each pole, and wood shims were again driven in between the plywood and the pole or the pole and the ceiling slab to accomplish a tight fit. Figure 5-15 shows the poles being erected; Figure 5-16 shows the shims at the ceiling, Figure 5-17 the base of the poles and the plywood bearing plates.

Areas No. 2 and No. 3 - Precast Prestressed Concrete Plank

Lines of posts and beams running perpendicular to the direction of the planks were used to shore three of the bays (lines D to G) in these areas. Structural steel wide flange sections, cut to approximate 8-ft lengths for ease in handling, were used as the beams, and each beam was supported by two timber telephone poles. All of the timber pole shores were shimmed tightly from the floors to the beams as described above, and additional shims were added between the top of the pole and the steel beam, where required, to assure that the steel beam was level and true with respect to the floor above. Figure 5-18 shows these shores in place.

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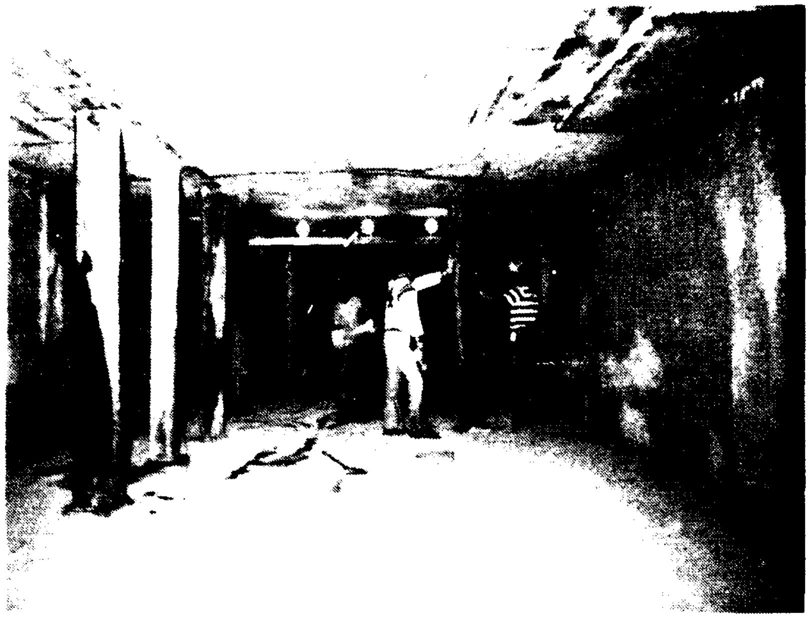

Fig. 5-15. Poles Being Erected, Area No. 1.

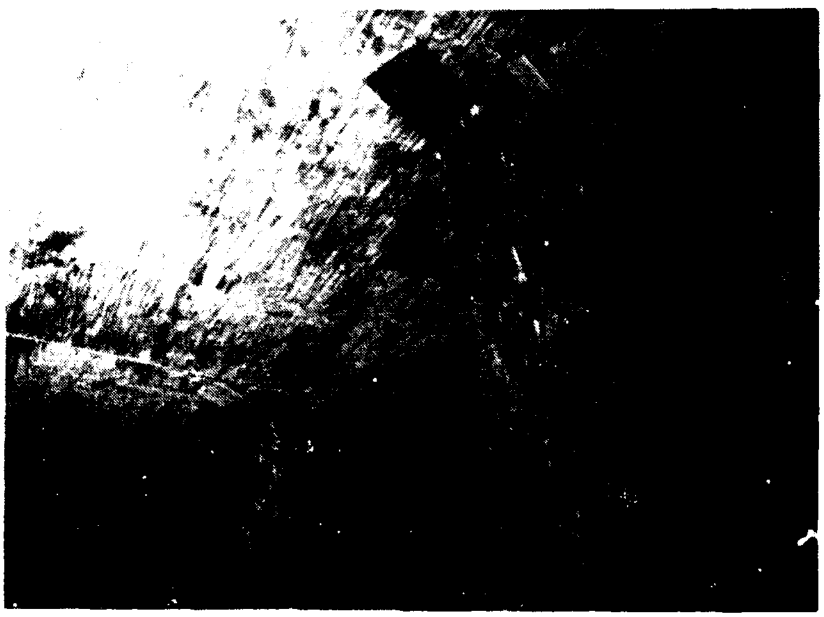

Fig. 5-16. Shims at Ceiling, Area No. 1.

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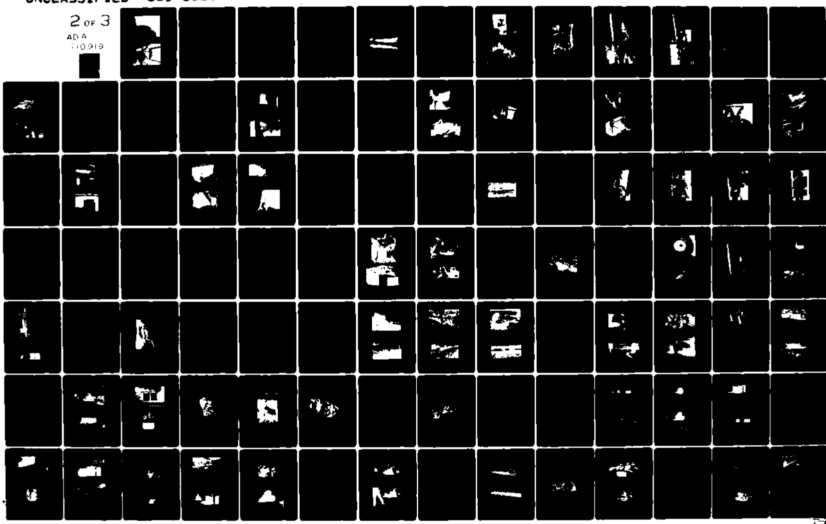

Fig. 5-17. Plywood Bearing Plates in Area No. 1.

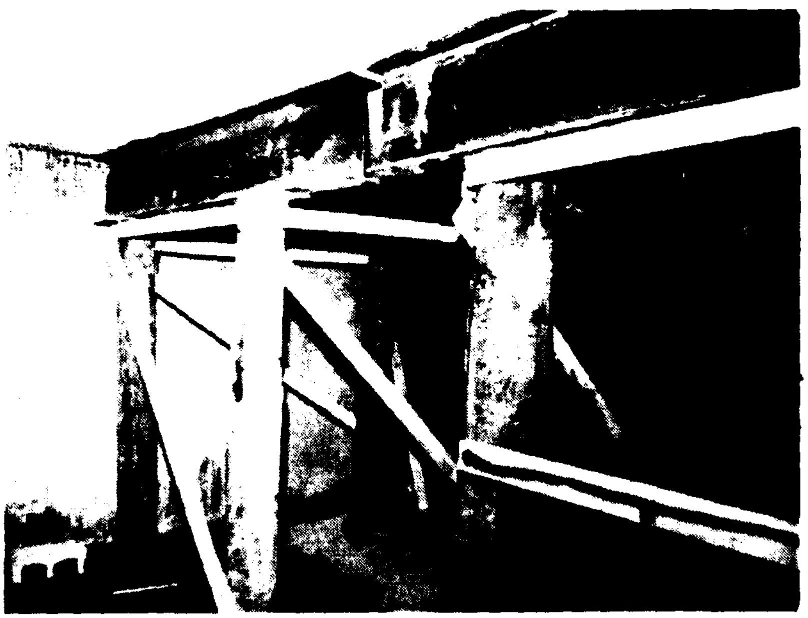

Fig. 5-18. Post and Beam Shores in Area No. 2.

The bay from lines D to E was shored with three lines of beams, symmetrically spaced 4 ft on center; the structural steel beam size was W14x30. The bay from lines E to F was shored with two lines of beams, spaced approximately 5 ft 4 in. on center, and the structural steel beam size was increased to W14x40. The bay from lines F to G was shored with only one line of beams; the size of these beams was again W14x30.

The precast beam on line E was shored with three timber posts symmetrically spaced along its length, and the precast beam on line G was shored with one timber pole positioned at midspan.

The bay between lines G and H was not shored or upgraded in any manner.

Area No. 4 - Two-Way Slab and Test Walls

The two-way slab construction in Area No. 4 was shored in a manner similar to the flat slab in the bays from lines B to D in Area No. 1. The shoring was again the timber telephone poles as described above, located 4 ft on center in both directions, and using the plywood plates and wood shims. The monolithically cast beam on line J was shored with two timber poles, one located to either side of the column, midway between the column and the exterior wall.

One of each of the three types of test walls was upgraded and one of each type remained as built. The upgrading consisted of the application of 2-in. thick sheets of expanded polystyrene to the exterior of each of the test walls to be upgraded. This material had a density of 1 lb per cubic foot, and was cut to the size required to completely cover the exterior face of the wall. The upgraded walls were located on line 1, H to H.5 (reinforced masonry) and J to J.5 (unreinforced masonry), and on line K, 1 to 1.5 (precast wall panel).

For all areas, the material, the sizes used, and the location of the shoring were obtained from the Shelter Upgrading Manual: Key Worker Shelters (Ref. 3). Figure 5-19 shows the basement floor plan and elevation and indicates the location and type of shoring used.

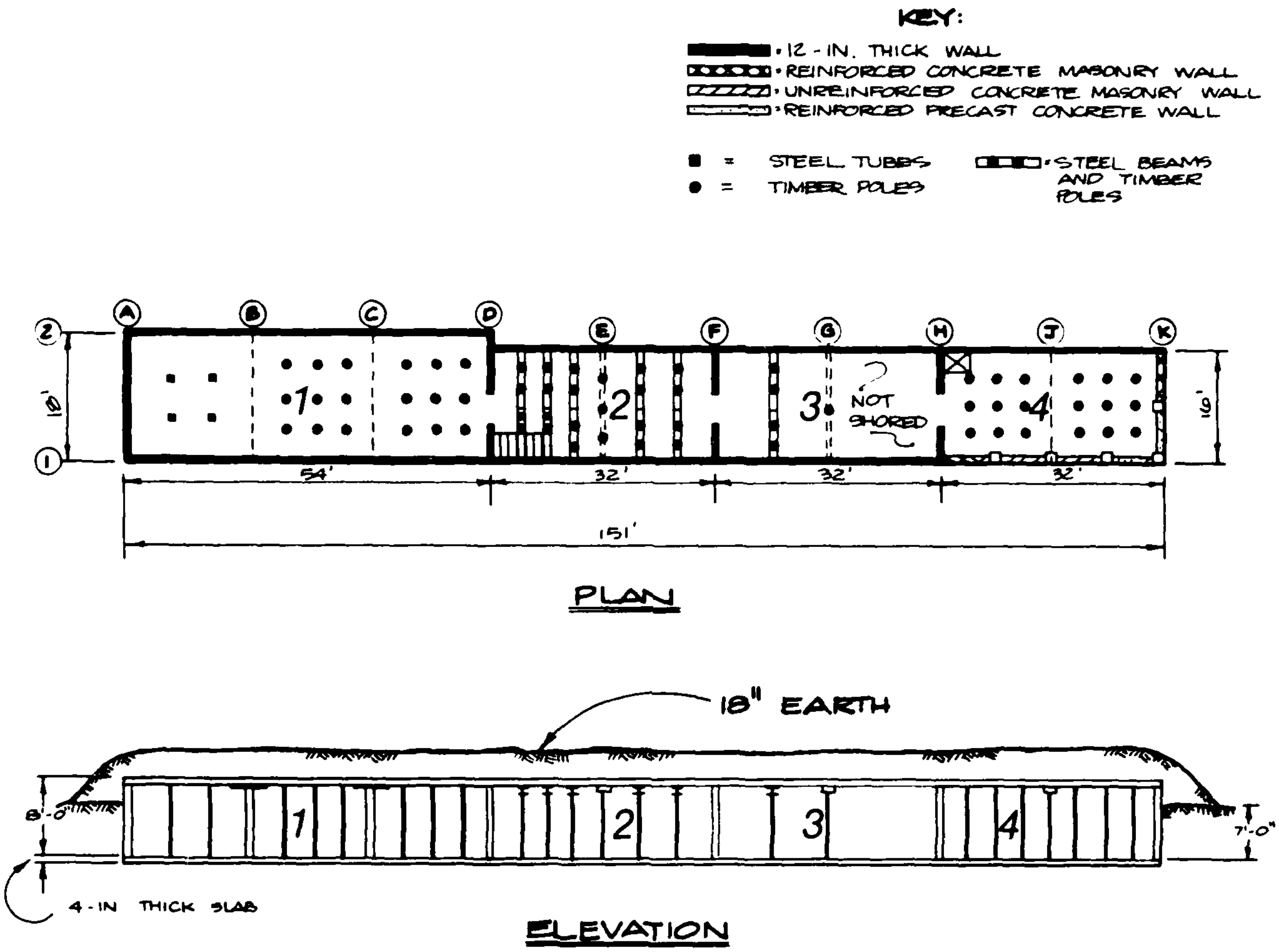

Fig. 5-19. Basement Floor Plan and Elevation Showing Location and Type of Shoring.

The stairwell opening and the hatch were closed off by cutting telephone poles to proper lengths, laying them over the openings side by side, and covering the poles with a layer of sandbags. The entire structure was then bermed and covered over with approximately 18 in. of earth. Figure 5-20 shows the structure completely bermed and covered with earth just prior to the test.

INSTRUMENTATION

Pressure Gauge

One pressure gauge was installed in the interior of the structure, 5 ft above the floor along line H in the bay from H to J, in order to measure and record the overpressure level. This gauge was a PCB Model 101A02 and was used in conjunction with a PCB Model 494A06 amplifier.

Load Cells

Four load cells were used in this experiment. One was located between the top cap plate on top of one of the structural steel tube shores and the ceiling slab in the bay from lines A to B. One was located between the top of the steel plate on top of the timber pole shore and the precast concrete beam on line G. Two were located on steel plates between the top of the timber pole shores and the ceiling slab in the bay from lines H to J. All the load cells (Model 3500-200, Houston Scientific Inc.) had a 200 kip capacity and were amplified by LM 741C op-amp.

Displacement Transducers

Two transducers (Bourns, Inc., Model 5108) were used at the interior of both of the unreinforced masonry test walls, one of which was upgraded.

Soil Pressure Gauges

Two soil pressure gauges were used in the exterior backfill material, approximately 12 in. away and at the midpoint of each of the two unreinforced masonry test walls. These gauges were Kulite Model LQU-080-8UL-200, and the amplifiers were Model LM 741C op-amp.

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Fig. 5-20. Structure Ready For Test.

Photography

Two cine cameras were used in this experiment. One was located 4 ft above the basement slab adjacent to the doorway in the wall on line H, and oriented so as to cover all of the test walls full height. This camera was 16 mm with a 5.7 mm lens. A second camera was positioned adjacent to the doorway in the wall on line F, and oriented to photograph both bays between lines H and F. These were the bays that were not shored, or only partially shored, and in which some degree of collapse and debris translation would be expected. This camera was a 16 mm with an 11 mm lens. Both cameras were installed and operated by personnel under the direction of the DNA, and operated at a speed of 400 frames per second with a coverage period of -5 to +12 seconds and a light period of -1 to +10 seconds. Figure 5-21 shows the cine camera located in the doorway on line H. The cine photography was supplemented with still photography.

Anthropomorphic Dummies

Six anthropomorphic dummies with self-recording accelerometers were located at points throughout the structure. Two were located in the bay from lines G to H, one (27) in supine position along line H near line 2, see Figure 5-22, and the other (26) in a sitting position against the wall on line H near line 1, see Figure 5-23. Two were located in the bay from lines H to J (33 and 28), both in sitting positions against the masonry test walls along line 1, see Figure 5-24. The remaining two were in the bay from lines J to K, one (25) in a sitting position against the test wall on line K near line 2, and the other (29) in a standing position against the test wall on line 1 between lines J and J.5, see Figure 5-25.

Vertical Displacement Measurements

Vertical floor to ceiling measurements were taken throughout the entire structure so that the permanent relative displacement could be determined after test.

The location of all the instrumentation is indicated on the building plan, shown in Figure 5-26.

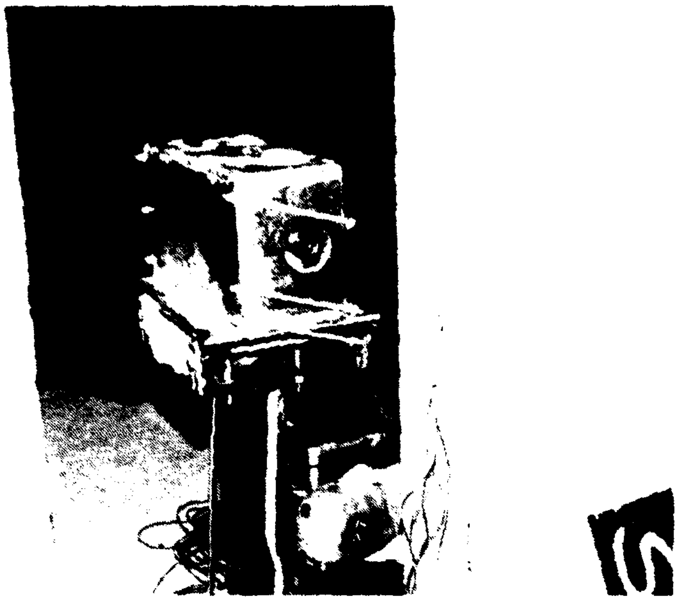

Fig. 5-21. Cine Camera Located Between Areas No. 3 and No. 4.

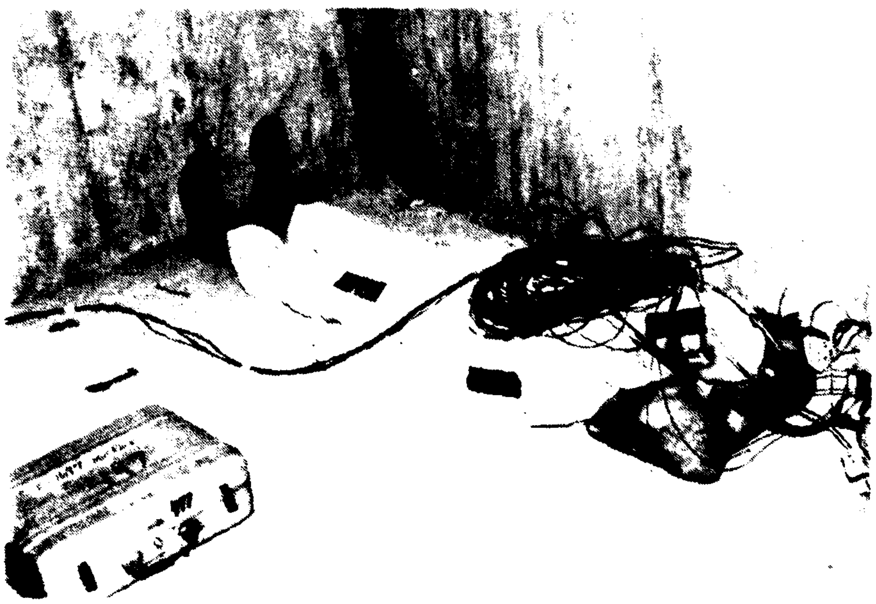

Fig. 5-22. Anthropomorphic Dummy No. 27 Along Line H in Area No. 3.

Fig. 5-23. Seated Dummy No. 26 on Line H in Area No. 3.

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Fig. 5-24. Seated Dummies No. 33 and No. 28 Against Masonry Test Walls, Area No. 4.

Fig. 5-25. Dummies No. 25 and No. 29 Against Test Walls, Area No. 4.

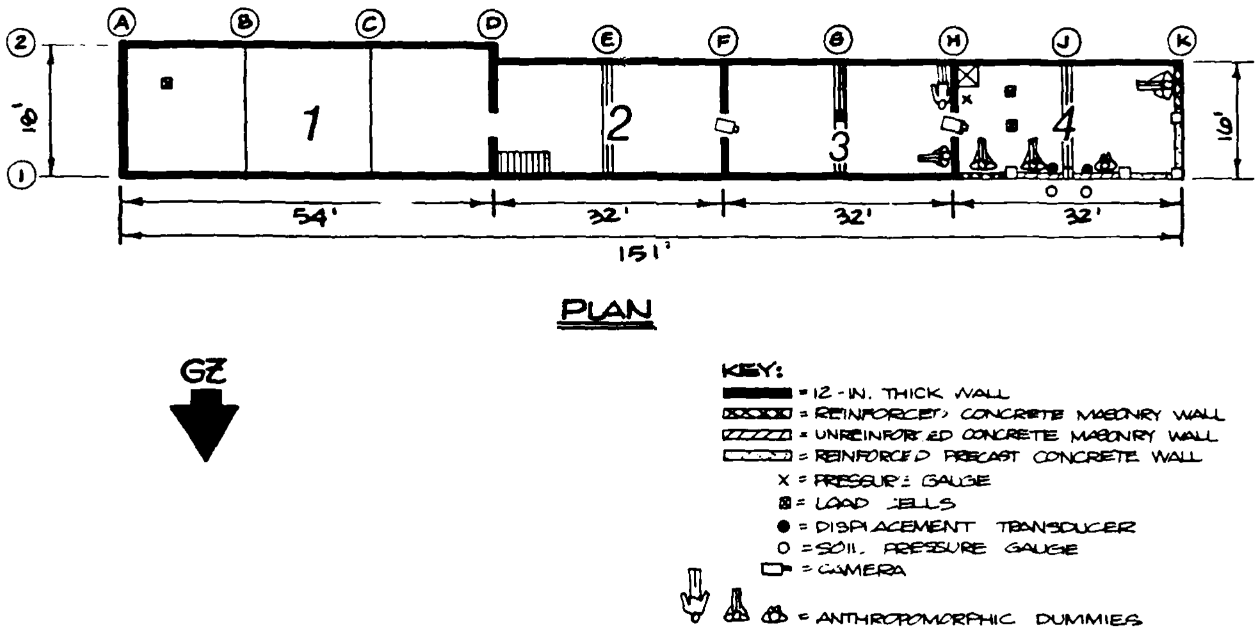

Fig. 5-26. Summary Sketch of Instrumentation Location.

TEST DATA

The peak free-field overpressure at this location, as recorded and furnished by DNA, was approximately 39 psi.

The initial observation of the structure after the test indicated complete or partial collapse of the four bays in Areas No. 2 and No. 3 (Figures 5-1 and 5-5). The closures over the stairwell and the hatch were both blown away. The telephone poles and sandbags that were used to cover the stairwell were scattered around the opening up to 15 ft from the periphery, and several of the poles had fallen down the stairwell. The poles and sandbags used over the hatch had been blown around and away from the hatch for a distance of 20 ft. Figure 5-27 presents views of the top of the structure immediately after test, and shows the collapsed bays and the scattered telephone poles and sandbags used for the closures.

Area No. 1

The 8-in. thick reinforced concrete flat slab construction located between lines A and D (Figures 5-1, 5-4) exhibited relatively little structural damage, as was anticipated. There appeared to be little difference in the distress noted in bay A to B, which was shored at the one-third points with steel tubing, and that in bays B to C and C to D, which were shored at the one-quarter points with timber poles. See Figure 5-19 for the shoring configuration. Light dust was observed, particularly in bay C to D; it obviously had passed through the doorway on line D, which was not closed. Also, one timber pole shore, located directly in front of this door, was knocked down by debris passing through the doorway.

Both the floor and ceiling slabs in all three bays exhibited cracking in both directions midway between the shoring. The maximum vertical deflection of the ceiling in these bays was 5/8 in. in a downward direction. Figures 5-28, 5-29, and 5-30 show the location of the cracks and the recorded ceiling deflections. Figure 5-31 is an interior view of these three bays looking toward line A from line C, and shows the timber pole shores in the foreground, and one of the steel tube shores in the background. Figure 5-32 is a closeup of the steel tube shores, and shows a load cell in location on top of the shore.

Fig. 5-27. Views of Top of Structure Immediately After Test.

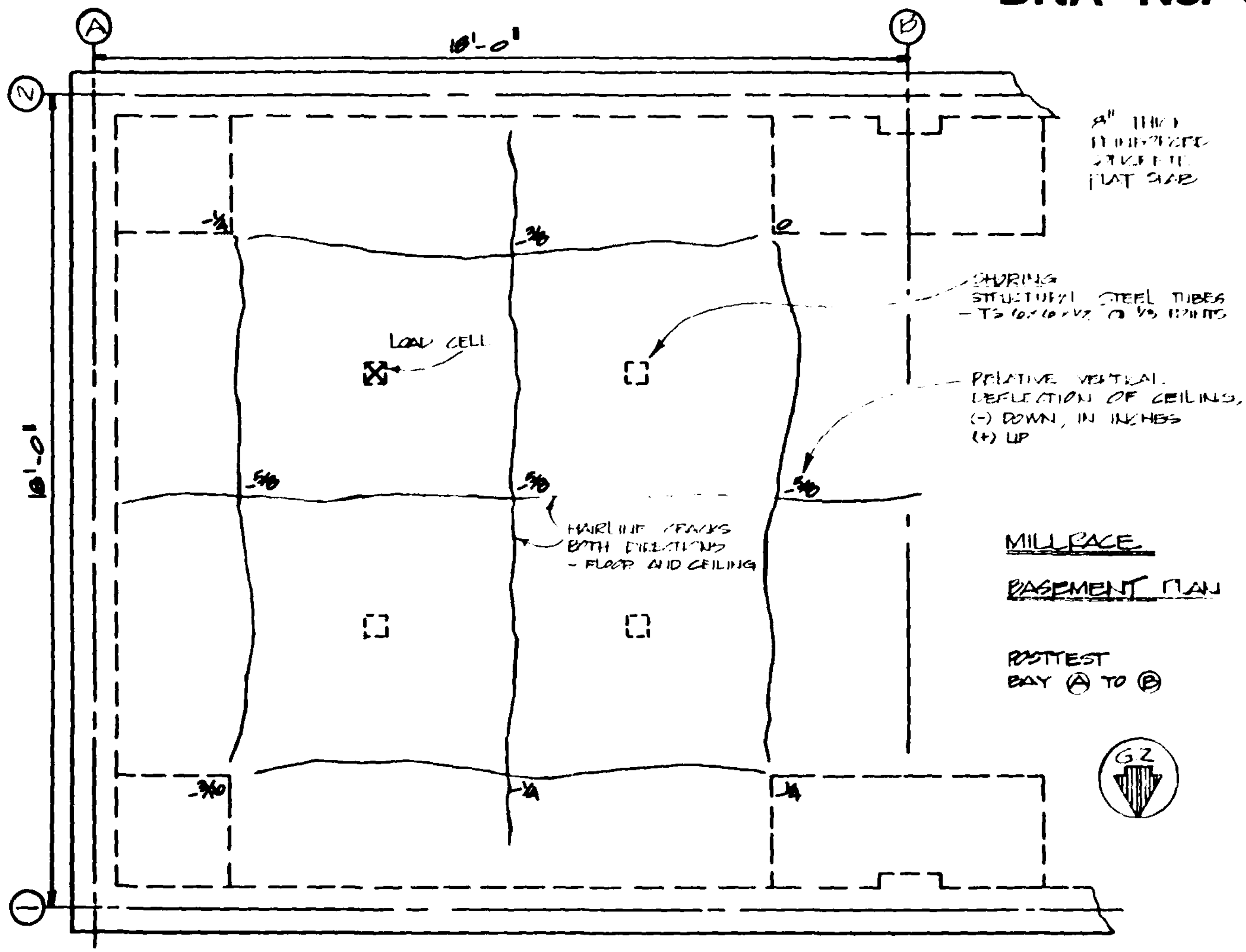

Fig. 5-28. Location of Shoring, Instrumentation, and Damage to Floor and Ceiling Slabs in Bay A to B, Area No. 1.

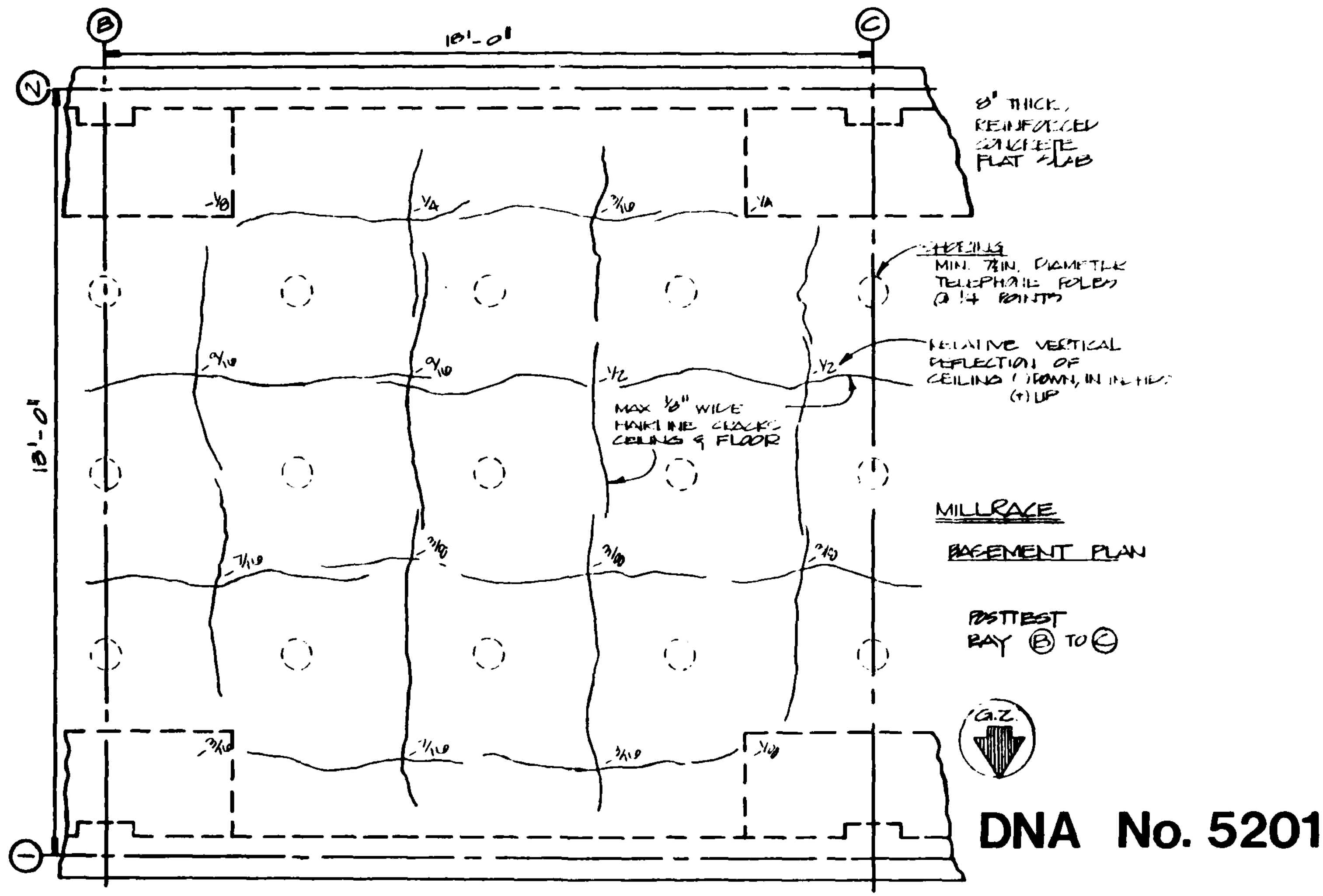

Fig. 5-29. Location of Shoring and Damage to Floor and Ceiling Slabs, Bay B to C, Area No. 1.

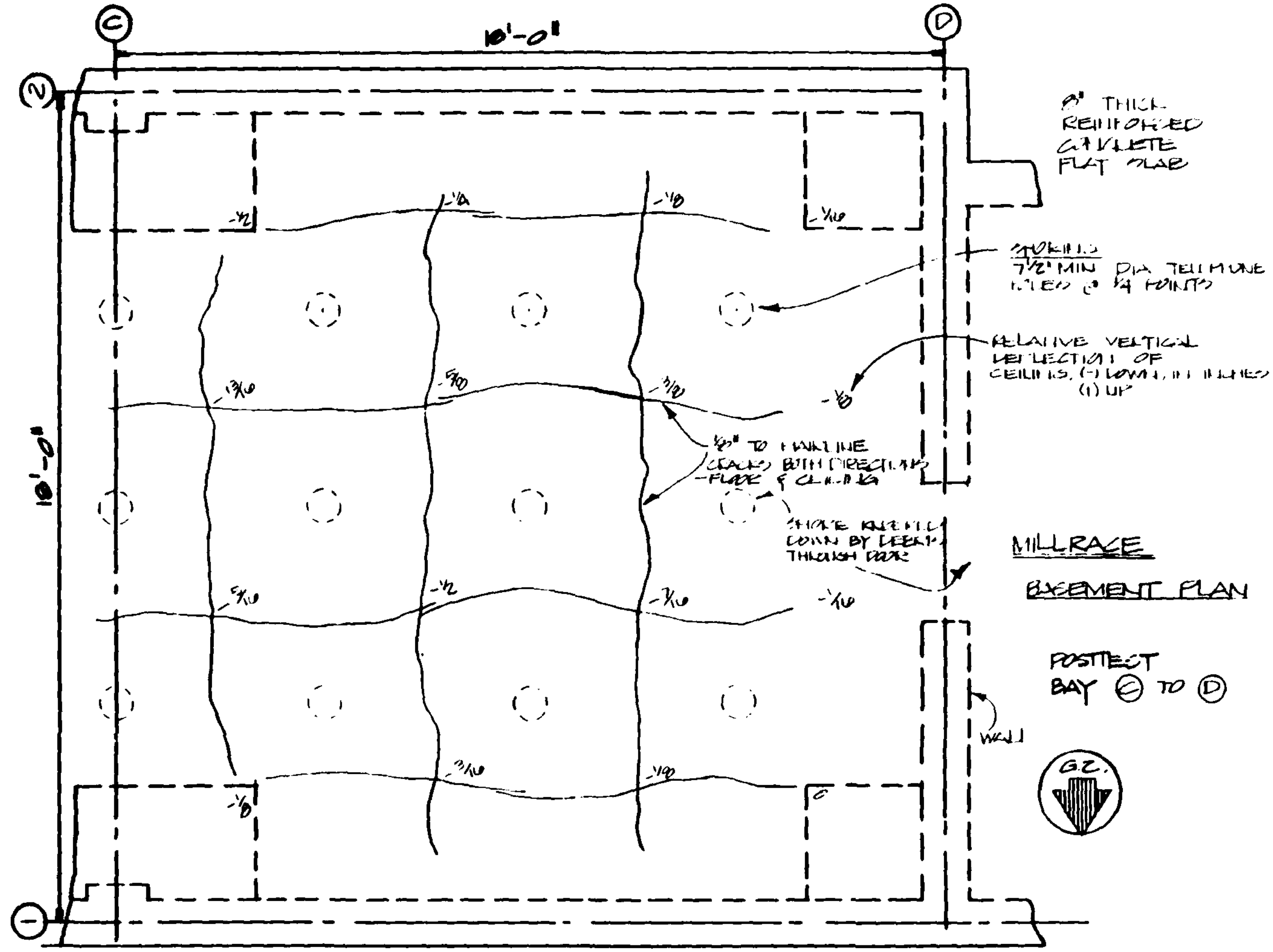

Fig. 5-30. Location of Shoring and Damage to Floor and Ceiling Slabs, Bay C to D, Area No. 1.

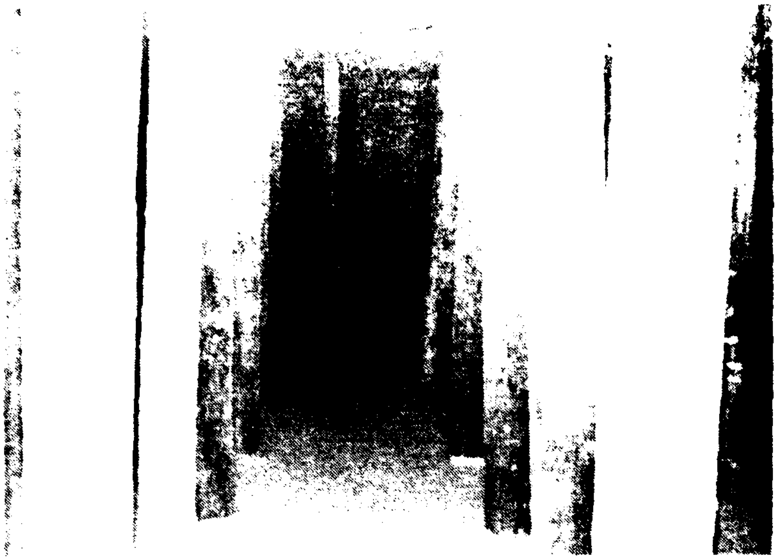

Fig. 5-31. Area No. 1, Interior Looking Toward Line A, Posttest.

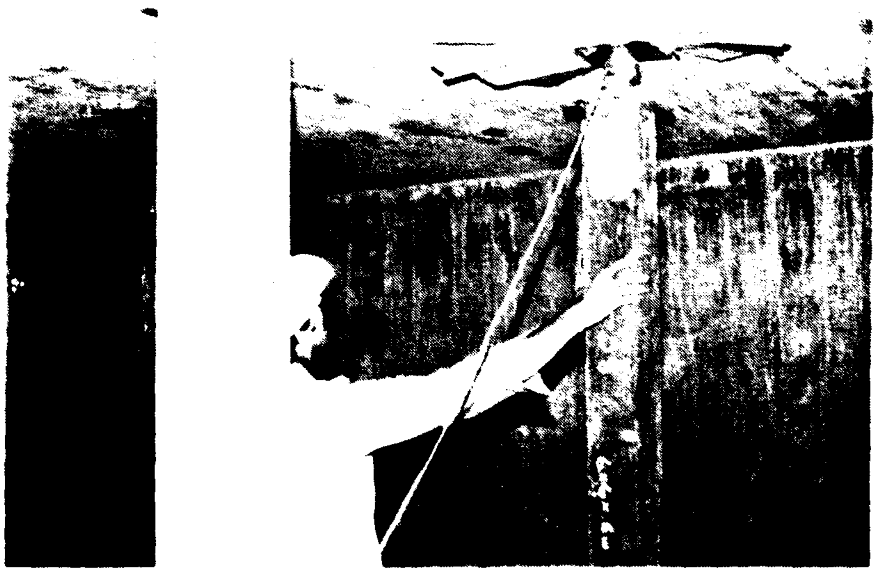

Fig. 5-32. Closeup of Steel Tube Shores, Area No. 1.

The load cell, located as shown in Figure 5-28, indicated a loading of 505 kips, with a loading duration of 50 ms.

Area No. 2

In the bay from lines D to E (Figures 5-1, 5-5, and 5-19), where the floor above was constructed of prestressed precast concrete plank and shored at the one-quarter points, approximately one quarter of the floor collapsed. This collapse occurred adjacent to the stairwell and along line D, as shown in Figure 5-33. The remainder of the planks forming the floor above were badly cracked in the transverse direction at locations of support, both at the shores and at the beam/wall supports, but did not collapse. Figure 5-34 is a view looking at the stairs, and Figure 5-35 is a view looking down the stairwell. Both of these views show a number of timber poles on and around the stairs; some of these are from the stairwell closure, and some are shores that have been knocked down. Figure 5-36 shows the doorway through the wall at line D. The stairs, which were constructed of heavy dimension lumber, were still usable after test.

The bay from lines E to F (Figures 5-1, 5-5, and 5-19), also constructed of prestressed concrete plank, but shored at the one-third points, had approximately one-half of the floor collapse, as shown in Figure 5-37. The remainder of the planks were again badly cracked transversely at their wall/beam and shore supports, but did not collapse. Figure 5-38 shows views looking down into the collapsed portion. The precast concrete beam and supporting corbels on line E did not indicate any structural damage, and the three shores supporting it were still in place.

Area No. 3

The bay from lines F to G (Figures 5-1, 5-5, and 5-19), of the same construction discussed above, completely collapsed, as shown in Figure 5-39. This bay was shored at the midspan, and the collapse completely knocked down the entire line of shores. Figure 5-40 shows this collapsed bay looking back toward the wall at line F. The cine camera can be seen in the doorway in this wall. Figure 5-41 is a view looking down into the bay toward line G, and shows the precast beam on line G.

Figure 5-42 again shows the precast beam on line G. This beam was upgraded

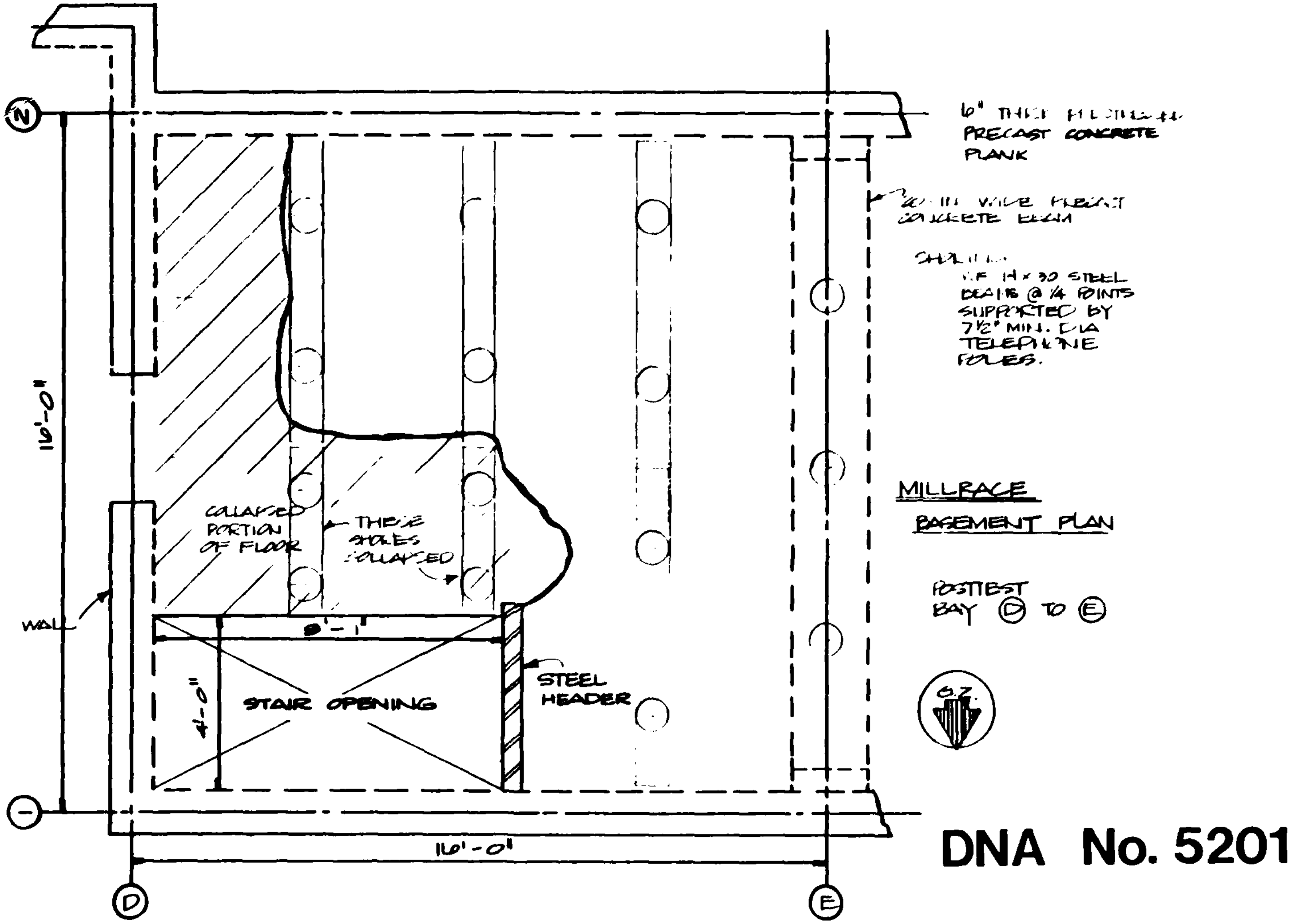

Fig. 5-33. Location of Shoring and Damage to Precast Plank Floor, Bay D to E, Area No. 2.

Fig. 5-34. View of Stairs, Posttest.

Fig. 5-35. View Down Stairwell, Posttest.

Fig. 5-36. Doorway Through Wall at Line D, Posttest.

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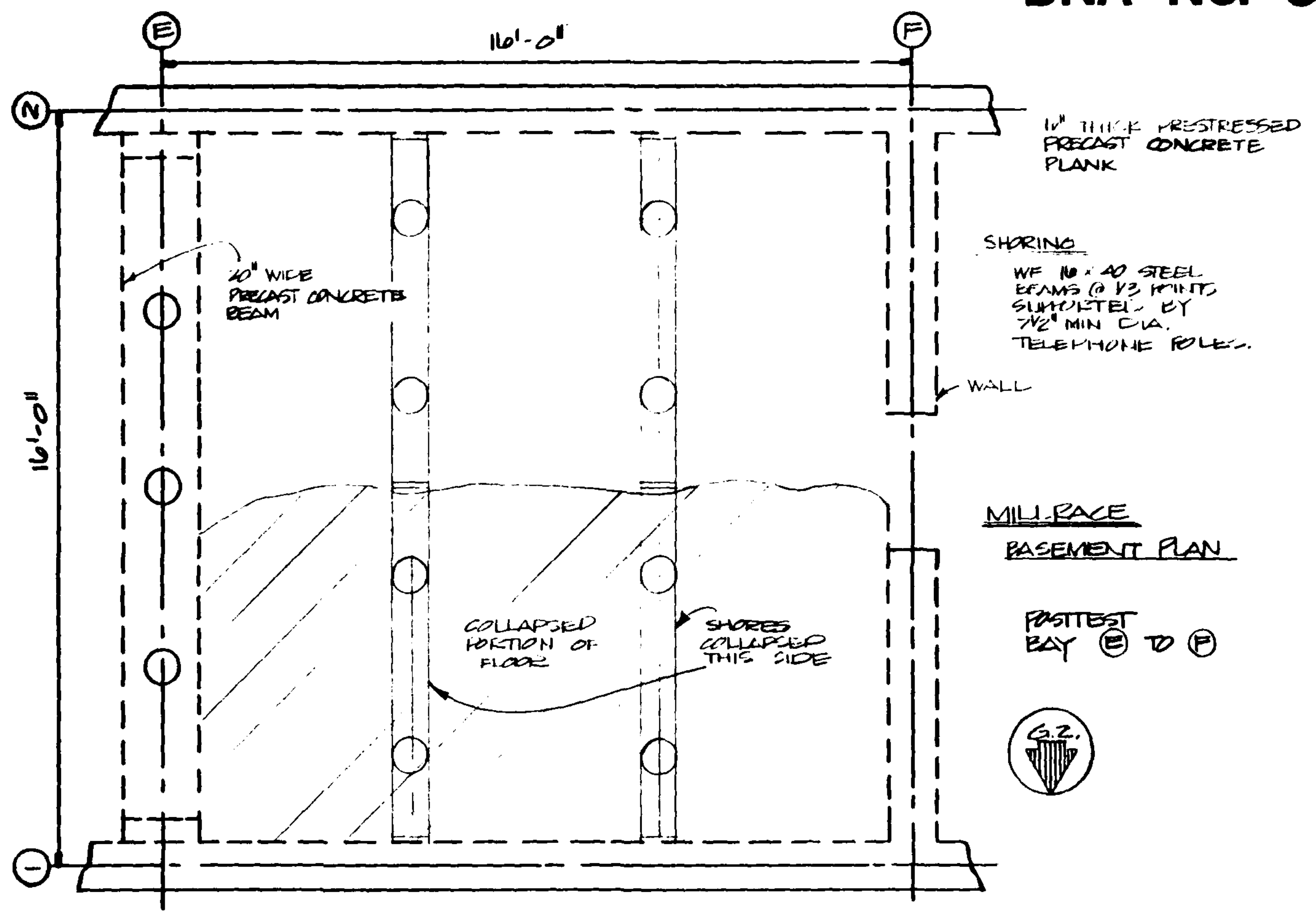

Fig. 5-37. Location of Shoring and Damage to Precast Plank Floor, Bay E to F, Area No, 2.

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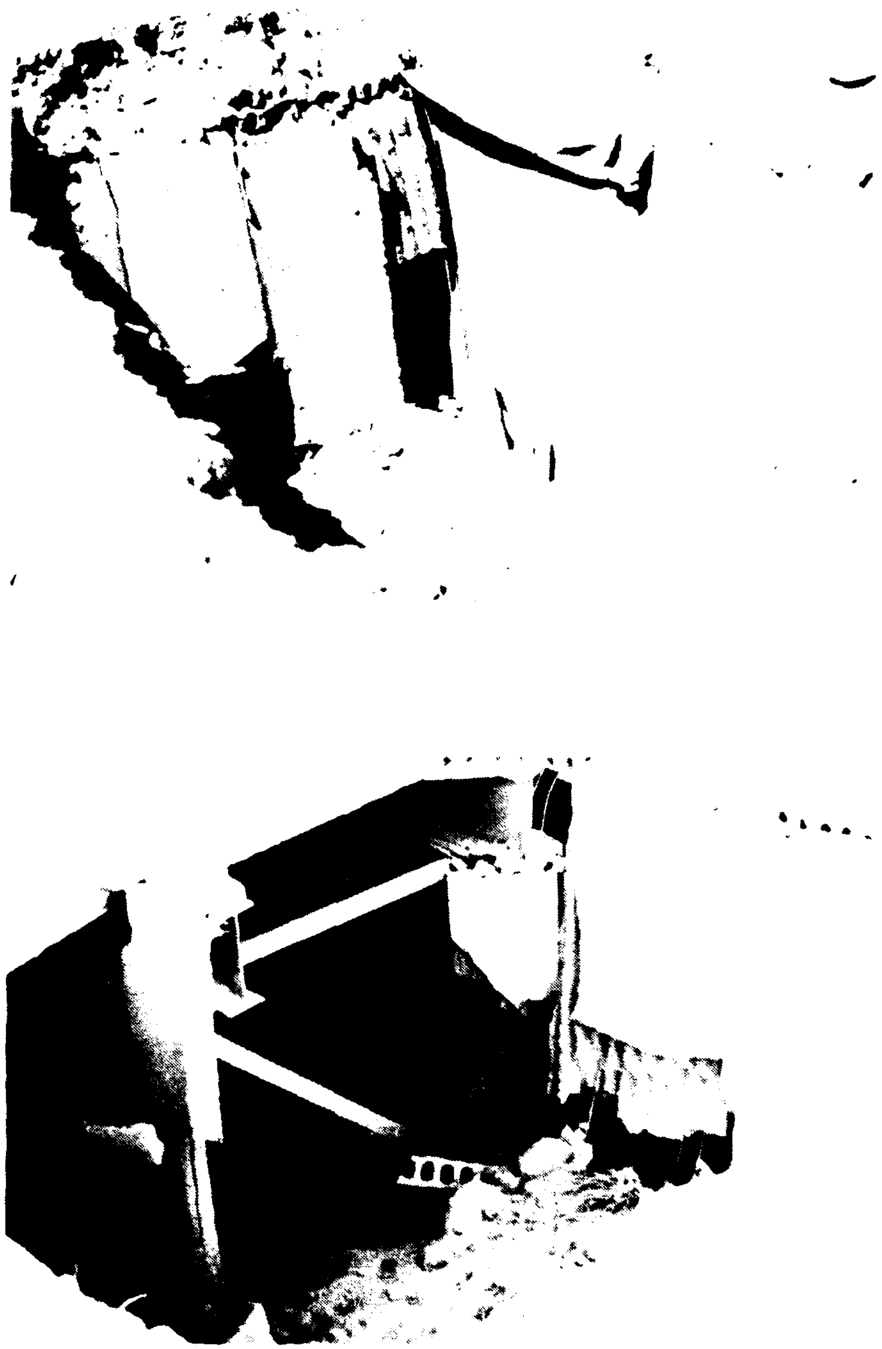

Fig. 5-38. Views of Collapsed Portion of Area No. 2, Bay E to F.

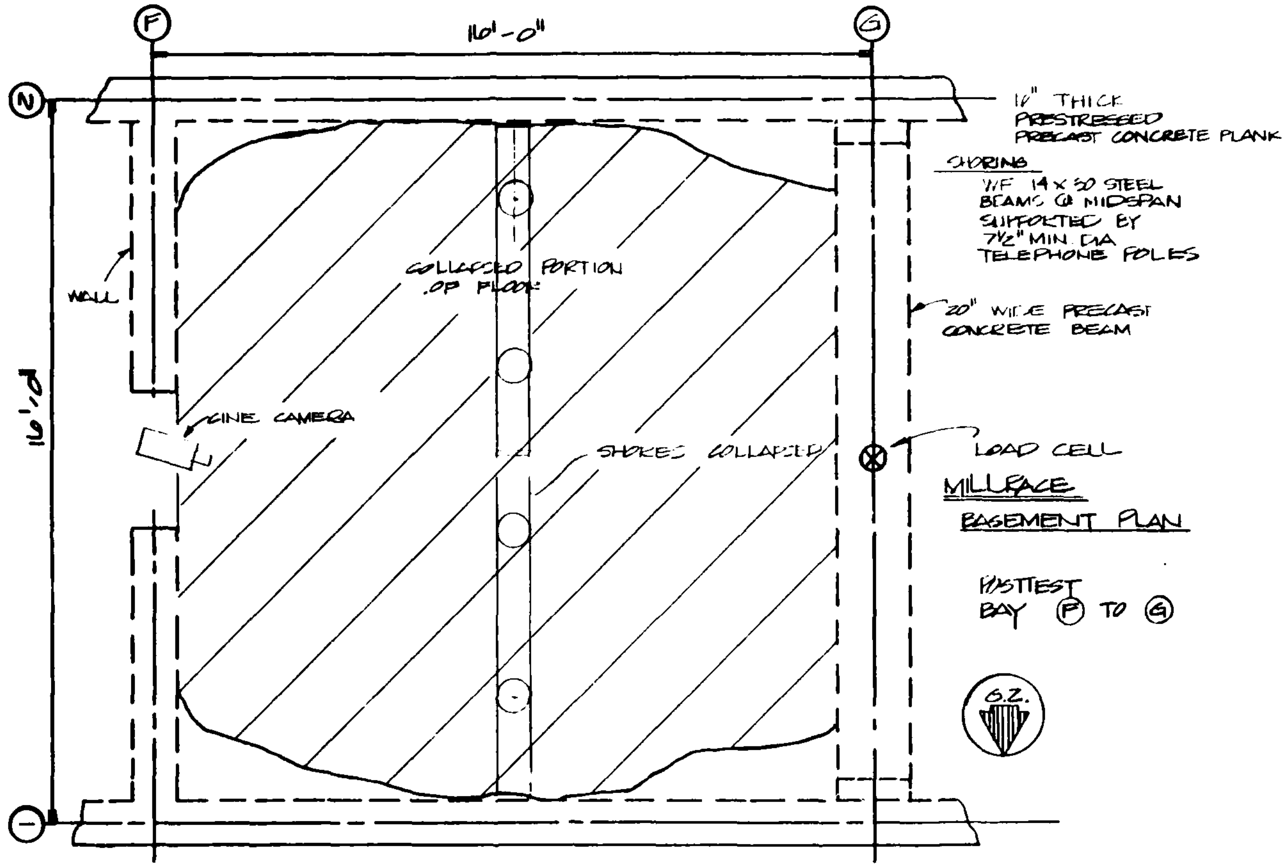

Fig. 5-39. Location of Shoring, Instrumentation, and Damage in Bay F to G, Area No. 3.

Fig. 5-40. Collapsed Bay of Area No. 3 Looking Toward Line F.

Fig. 5-41. View of Collapsed Bay in Area No. 3 Looking Toward Line G.

Fig. 5-42. View of Collapsed Area No. 3 Showing Precast Beam Along Line G.

with one timber post located at midspan, and as seen in this figure, this shore had fallen out. Other than some small negative moment cracks noted at midspan, the structural integrity of the beam appeared to be intact. The top bearing edges of the beam were slightly chipped as a result of the precast plank's rotating into the basement during collapse. The supporting corbels, which were monolithically cast with the wall, indicated shear cracking. Figure 5-43 shows these supporting corbels.

The load cell located on top of the shore under the beam on line G indicated a loading of 255 kips, with a loading duration of 50 ms.

The ceiling of the bay from lines G to H (Figures 5-1, 5-5, and 5-19), again constructed of precast prestressed concrete plank, was unshored and completely collapsed (see Figure 5-44). The collapse in this bay was total, filling all areas of the bay along the walls and in the corners with concrete debris and earth. Figures 5-45 and 5-46 are two different views looking down into the bay toward line H. The cine camera seen located in the doorway in these figures is oriented toward lines J and K.

Figure 5-45 shows the anthropomorphic dummy (26) located against the wall along line H, near line 1. This dummy was found rotated with the head moved toward ground zero 0.8 ft, and earth roof materials covering both legs to a depth of 1.5 ft. A 20-lb concrete fragment was found resting against the chest. The dummy was found to have sustained three tears in the leather of the right boot, multiple tears in clothing, and deep lacerations of the right shoulder, left hip and knee, right knee, calf, and foot. Figure 5-47 shows views of this dummy (26).

Figure 5-46 shows the posttest location of the dummy (27) that was originally located in the supine position along the wall on line H, and has been completely covered with 2 ft of earth and roof materials. Its location was the same as pretest, and a 500-lb concrete fragment was found in the area over the left thigh. The dummy sustained multiple tears in clothing, one each deep laceration on the head and face, left arm torn from shoulder and shoulder joint broken, and deep lacerations of thorax, left hip, and left thigh. There were no readings recorded on the accelerometers of either dummy (26 and 27).

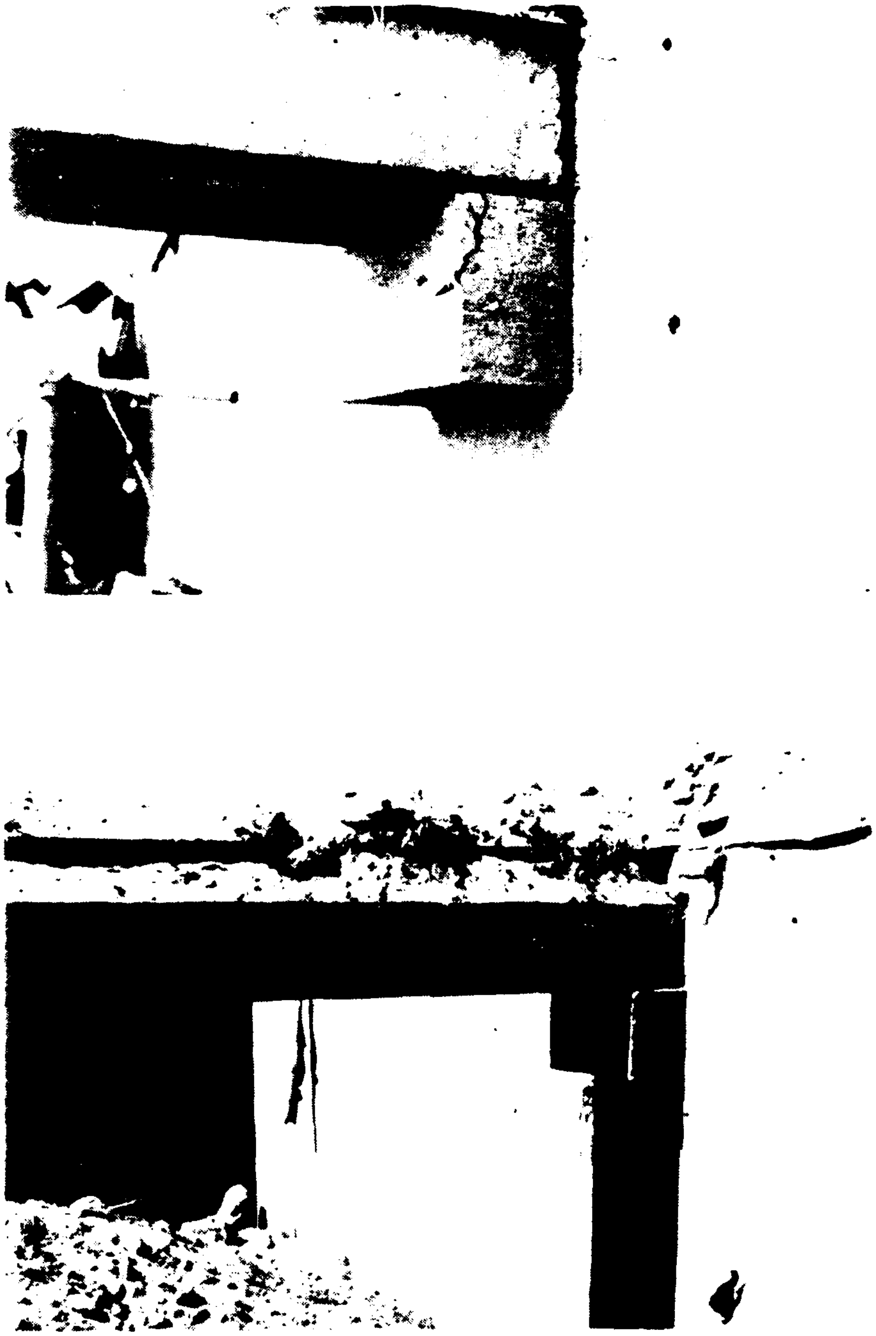

Fig. 5-43. Two Views of Supporting Corbels, Posttest.

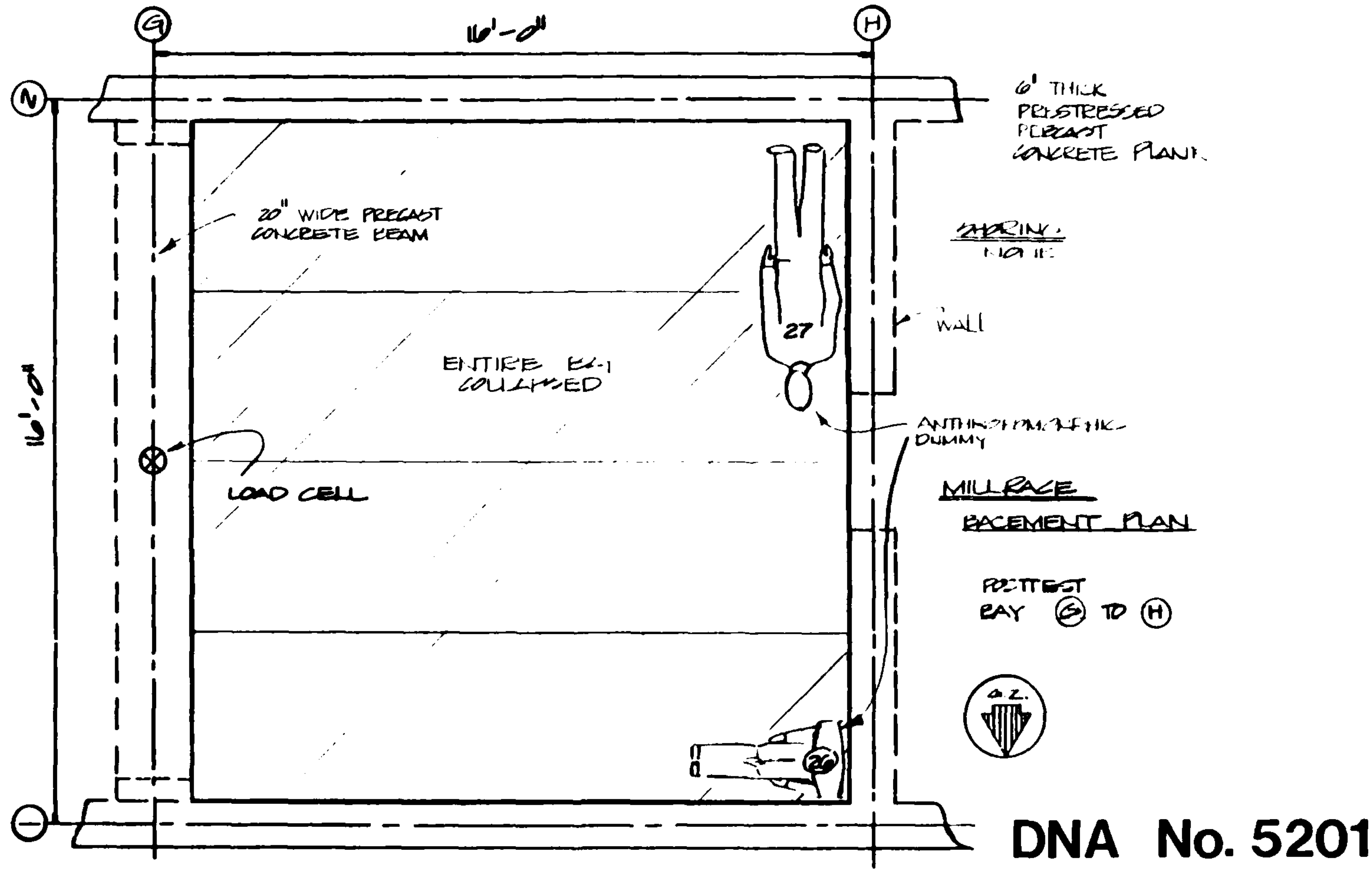

Fig. 5-44. Location of Instrumentation and Damage in Bay G to H, Area No. 3.

DNA No. 5201

Fig. 5-45. View of Bay G to H, Area No. 3, Posttest.

Fig. 5-46. View of Bay G to H, Area No. 3, Posttest.

Fig. 5-47. Views of Dummy No. 26, Posttest.

Area No. 4

The two bays between lines H and K (Figure 5-1, 5-6, and 5-19), of 6-in. thick reinforced concrete two-way slab construction, sustained very little damage. Both bays were shored at the one-quarter points with timber telephone poles, and were expected to perform well in the 40 psi range. No cracks were found in the floor slabs in either bay, and only relatively short hairline width cracks were noted in the ceiling. The ceiling cracks ran in both directions, midway between the shores. Figures 5-48 and 5-49 show the locations of these cracks. The maximum vertical deflection found in the ceiling was 15/16 in. The deflections recorded for both bays are shown in Figures 5-48 and 5-49.

No evidence of cracking or distress was observed in the four concrete masonry walls. Both of the precast concrete walls exhibited one horizontal crack each, approximately 1/8 in. wide for their full width, at about midheight of each wall. The maximum inward displacement of all of the walls was very slight and indicated no pattern. The transducer located on the interior of the upgraded unreinforced masonry wall recorded 0.25-in. inward displacement, while the one on the interior of the non-upgraded unreinforced masonry wall recorded 0.19-in. inward displacement. As measured from benchmarks, the maximum displacement of the two unreinforced masonry walls was 9/16 in. and occurred along the bottom edge, of the two reinforced masonry walls was 7/8 in. and occurred at midheight, and of the two precast concrete walls was 1/4 in. and occurred at midheight at the cracks. The soil gauge located on the exterior of the non-upgraded unreinforced masonry wall recorded a pressure of approximately 9 psi. The other gauge, located on the exterior of the upgraded unreinforced masonry wall, malfunctioned, and no data were recorded. Six days after the test the backfill material was partially removed from the exterior of the test walls, and Figure 5-50 shows the two walls at the end of the structure along line K after uncovering.

The pressure gauge located on the wall on line H, adjacent to the door in the bay from lines H to J (see Figure 5-48 for location), recorded a positive pressure of approximately 1.0 psi with a rise time of 125 ms, immediately followed by a rapid decrease to a negative pressure of approximately 2.5 psi in 150 ms.

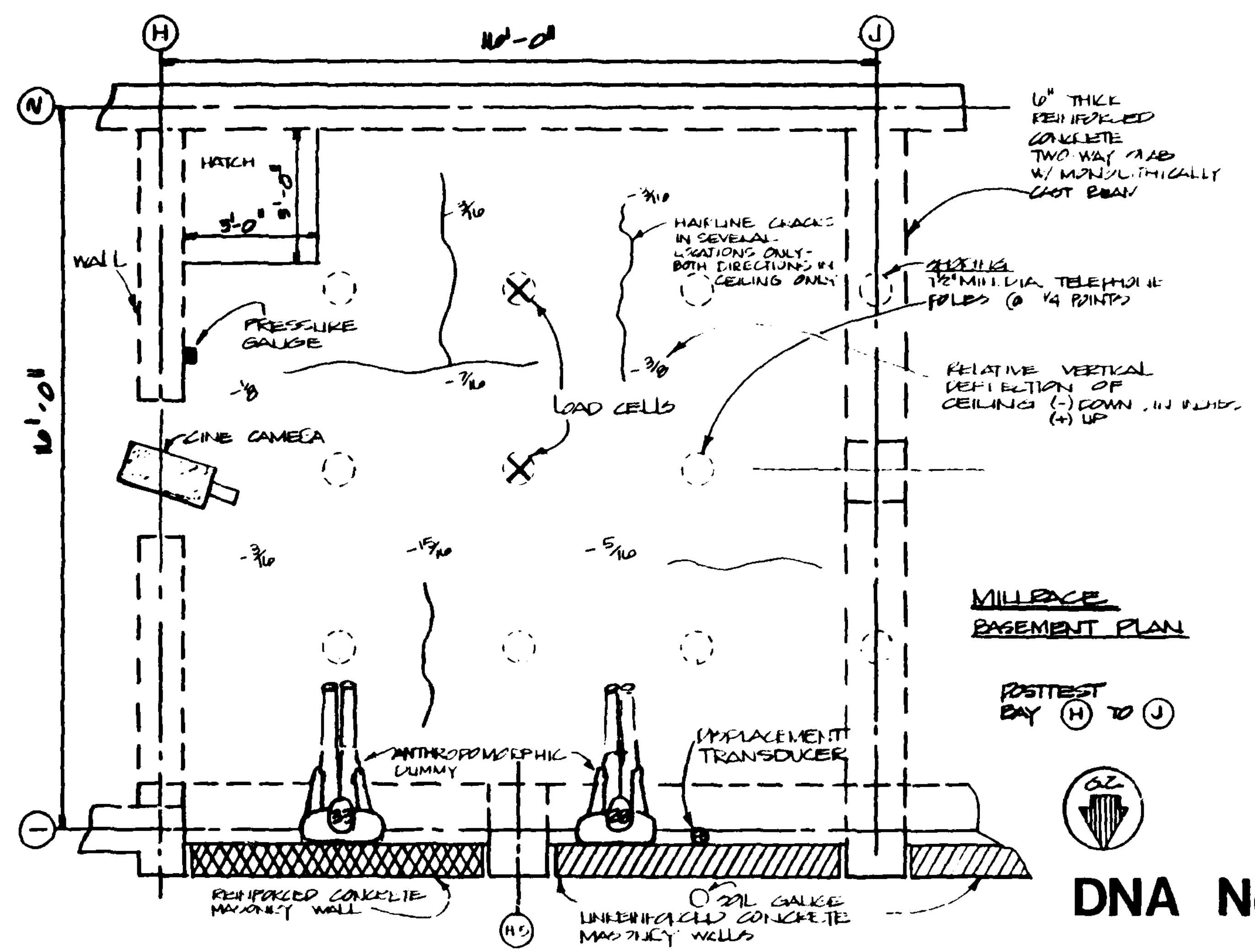

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Fig. 5-48. Location of Shoring, Instrumentation, and Damage to Two-Way Slab, Bay H to J, Area No. 4.

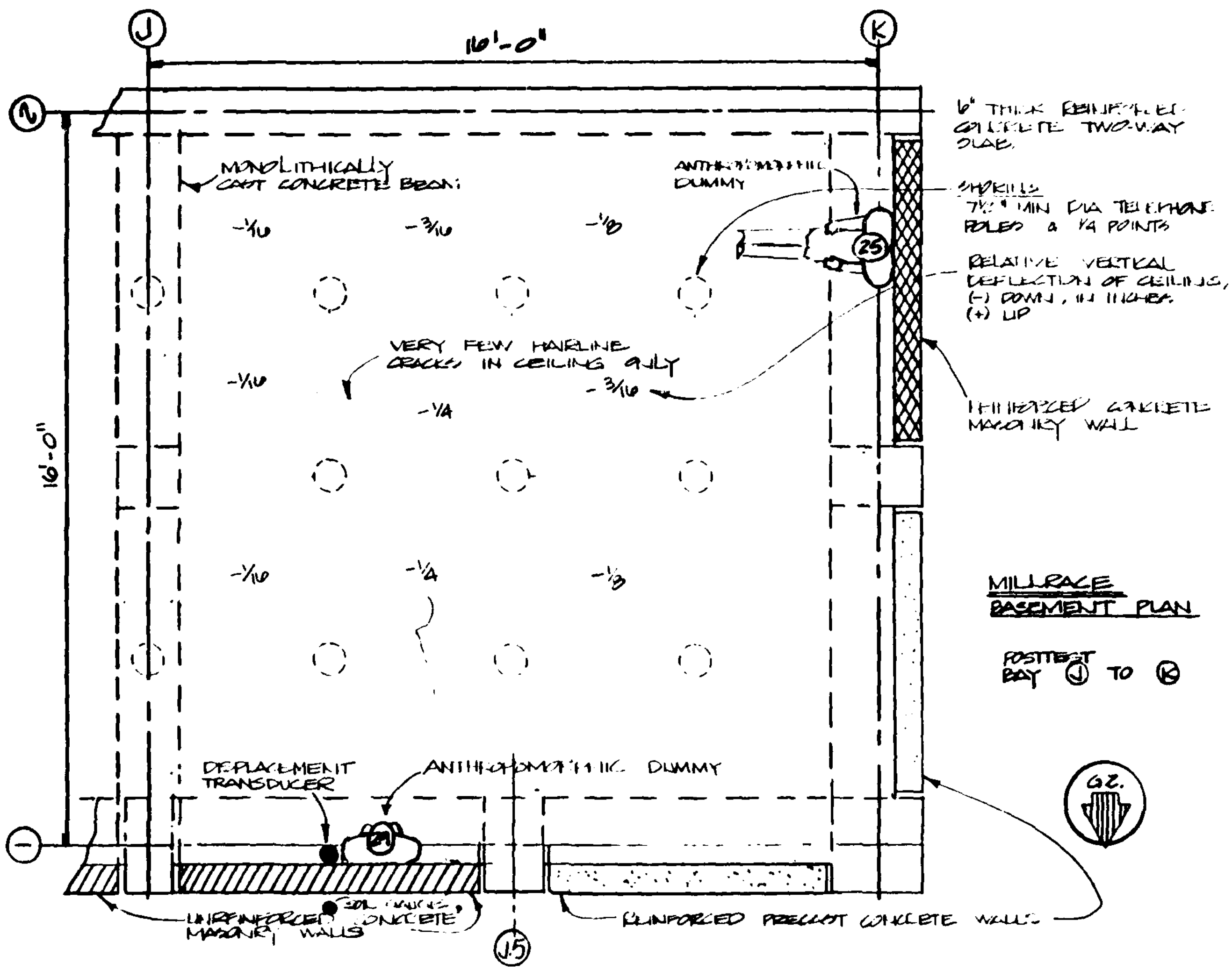

Fig. 5-49. Location of Shoring, Instrumentation, and Damage to Two-Way Slab, Bay J to K, Area No. 4.

Fig. 5-50. Two Test Walls at End of Structure Along Line K, After Test.

The load cell located nearest the wall on line 2 (see Figure 5-48 for location) in bay H to J recorded a loading of 180 kips and a loading duration of 50 ms, while the other load cell in the same bay recorded a loading of 205 kips with the same loading duration, 50 ms.

The three anthropomorphic dummies placed in the sitting position, two in bay H to J and one in bay J to K (33, 28, and 25), were all found in the pretest positions, sustained no damage, and no readings were recorded on their respective accelerometers. Figures 5-48 and 5-49 show the location of the dummies in each of the bays. All three of the dummies were covered moderately with dust that was assumed to have passed through the doorway, which was not closed, in the wall on line H during the failure of bay G to H. Figures 5-51, 5-52, and 5-53 show dummies (33) (28) and (25), respectively, after test.

The dummy that was standing against the wall along line 1 in bay J to K (see Figure 5-49 for location) is shown in Figure 5-54. The head on this dummy had moved 1 in. to the right and had fallen forward on the chest, the left foot had moved 0.5 in. to the left, and the right foot had moved 0.25 in. to the left. No damage had been sustained by the dummy. The reading on the accelerometer was 10 g.

Because of apparent problems with the lighting system at one location and malfunction of the camera at the other location, no usable films were obtained.

Fig. 5-51. Dummy No. 33 After Test.

DNA No. 5201

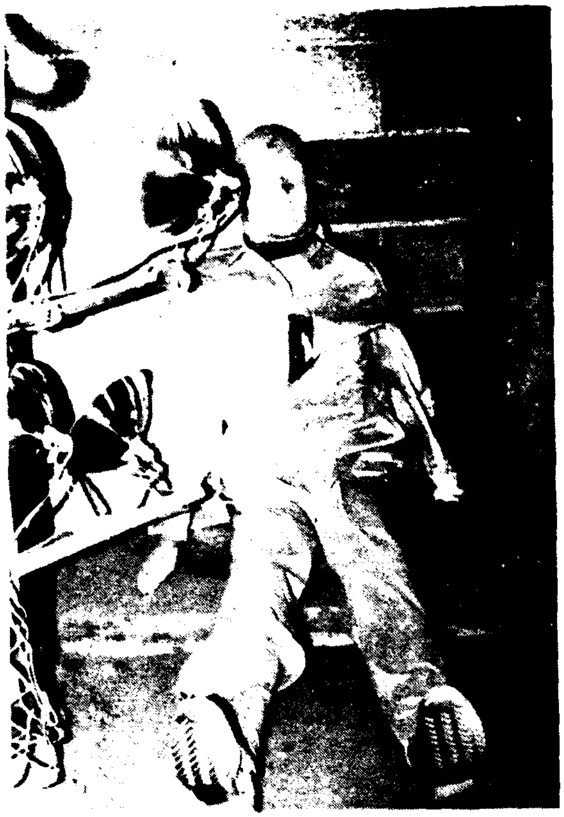

Fig. 5-52. Dummy No. 28 After Test.

Fig. 5-53. Dummy No. 25 After Test.

DNA No. 5201

Fig. 5-54. Dummy No. 29 After Test.

DISCUSSION AND CONCLUSIONS

The upper floors in the portions of the structure that had shoring designed to withstand 40 psi, with the exception of the bay from lines D to E, performed as anticipated. The cracks in evidence in the ceiling in the three bays from lines A to D were in the region of positive moment in both directions between the shores, as would be expected from classical yield line theory. The cracks in the ceiling in the bays between lines H and K, although less severe, were also positive moment cracks and represented yield line theory.

The floor in the bay between lines D and E was also shored to survive 40 psi; however, it sustained partial collapse in the area of the stairwell. It is believed that fundamentally the planks would have survived with this shoring configuration since a considerable portion of the floor, although severely damaged, remained in place. The partial failure appears to be partly the result of the closure over the stairwell failing and the poles from the closure falling into the basement and knocking out several of the pole shores supporting the damaged floor. Another factor that may have contributed to this partial collapse was the severe overloading of the prestressed plank adjacent and parallel to the long dimension of the opening. This plank was initially slightly overloaded by supporting one end of the header carrying the short planks headed off at the stairwell. Significant additional load was applied by construction of the closure in which sections of telephone poles were laid side by side, one end of each supported by this same plank, and sandbags placed on top of the poles. Possibly, additional shoring should have been used at this location to assist in supporting these increased loads. If such shoring had been added, this partial collapse may have been prevented.

The double line of shoring in bay E to F would be expected to upgrade the floor to withstand loading of approximately 25 psi, and the performance was as anticipated. The appearance of the failed planks was nearly identical to those tested in the laboratory and reported in Ref. 7. The partial collapse of this bay underscored the importance of laterally bracing the shores to each other and to adjacent walls. It appeared that a small portion of the planks collapsed first, swung down into adjacent shores and knocked them out, thus allowing additional portions of the floor to collapse.

The single line of shores in bay F to G would be expected to withstand loading of approximately 3 to 5 psi, and this bay was primarily tested to investigate debris translation and to verify modes of failure of this type of system. Some of the precast planks failed in the classical three-hinge, four-piece segments identical to those statically tested and reported in Ref. 7.

As a result of the performance in these four bays of precast prestressed concrete planks, it is probable that we would not recommend structures constructed of this material for selection as shelters without some method of connection upgrading. With the exception of areas of the country where seismic design is a prime consideration, such as parts of California, no structural concrete topping is used, and the end bearing connections are meager or non-existent. As witnessed in this test, disastrous types of collapse mechanisms can occur when the planks are untopped and connections are absent.

Although the bays on either side of the beam on line G failed completely, the beam was in surprisingly good condition and appeared to have maintained its structural integrity. The corbels supporting the beam were cracked, but not judged to be approaching failure. The obvious explanation for this beam's survival is that it did not receive the full overpressure loading. This is verified by the 255 kip load recorded by the load cell on top of the single pole shore. When taking in account the tributary load supported by this shore, this loading would translate to an applied load of approximately 18 psi. This reduced loading to the beam is probably the result of the failure of the planks rapidly under a relatively light loading, thus negating the possibility of a significant tributary loading to the beam.

The performance of the test walls was not as expected. Analytically, the unreinforced masonry walls would be of structural concern from the backfilling operation, let alone the lateral loading from the soil as a result of a 40 psi overpressure. Obviously, soil/structure interaction is an area where a great deal of emphasis must be placed in the future. An interesting observation is that the two walls that sustained cracking (the precast concrete walls) were located in areas where the excavation was ramped up to the surface for access. Accordingly, they were backfilled with considerably more material than the other four walls, which were only backfilled with approximately 3 ft of non-compacted granular material. These differences in the amount and configuration of the backfill and the degree of compaction are areas recommended for further investigation.

An additional area of interest is that of the punching of the pole shores through either the ceiling or floor slabs. A close inspection of this structure did not indicate any distress whatsoever as a result of shore punching. This may not be nearly as severe a problem as it first appeared when the results of the small static punching shear slab tests conducted in the laboratory (Ref. 7) were initially considered. It does, however, confirm the results observed during many full-scale static loading tests of various types of shored concrete floors, where even under quite high loading, there was very little tendency of the shores towards punching. One explanation of this absence of distress from punching in the test structure might be as follows.

As the structure is subjected to loading, it develops a hinge in the area of high negative moment; i.e., over the shore. This hinge development induces very high compressive forces at the slab face orthogonal to the punching direction. Once high compression forces have been introduced in the slab, the shear forces created by the punching may not permit the development of significant diagonal tension stresses. This phenomenon may be somewhat like that of "arching", in that high compressive forces combined with orthogonal shear forces result in low tensile or diagonal tension stresses. It may be that an analytical model could be developed where displacement constraints are introduced around the inflection points at the compression faces, and based on the resulting data, slabs designed and tested that would have the high punching resistance that was observed in the field. This is an area that certainly requires and merits further investigation and study.

As in the other experiments, the closures were not in position after the test and in fact, may have contributed to portions of the collapse. Both of the closures in this structure were blown off by the negative pressure phase and/or the positive pressure created by adjacent collapsing bays. In any case, further research and testing are required in this area in order to construct closures that provide reasonable access as well as the required protection.

In general, the performance of the various components of the experiment provided significant data. The shoring systems performed as designed, and the data developed with respect to debris translation will prove useful. This experiment, as expected, also clearly pointed up areas requiring further study: closures, punching shear, and the loading by soil on basement walls as a result of a dynamic surcharge from blast loading.

Section 6

DNA NO. 5101

INDUSTRIAL HARDENING - EXPEDIENT SHELTERS

INTRODUCTION

This experiment consisted of burying two concrete utility vaults, one at an expected 20 psi environment, 740 ft from ground zero, and the other at 40 psi, 580 ft from ground zero.

OBJECTIVE

These two vaults were standard production items of the type used by electrical and telephone utilities. In a previous study, vaults of this type were found to be ready made and available for use as shelters in many areas of the country. This basic study indicated that a vault of this type could be transported, placed in an excavation, backfilled, and covered with 3 ft of earth for radiation protection, by three men in approximately 8 hours. The object of this experiment was to test the viability of these vaults at 20 and 40 psi, covered with 3 ft of earth.

DESIGN

The vaults were supplied by the manufacturer as typical construction and in accordance with his specifications, which are as follows: "Vault designed for H-20 highway loading* plus 20% impact. May be set with top 2 feet below surface. Concrete to be minimum of 4,500 psi strength at 28 days. Reinforcing steel to be ASTM 615-60 (Grade 60)."

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* This is a typical highway loading attributed to a truck with a 32,000-lb rear axle load and an 8,000-lb front axle load.

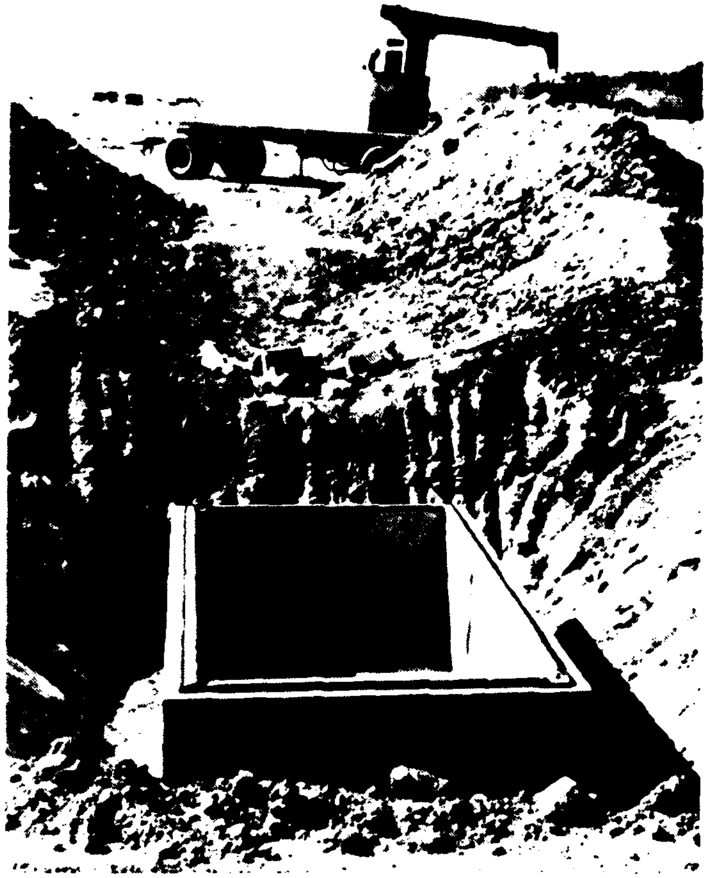
CONSTRUCTION

The two vaults were identical, with overall dimensions of 13 ft long, 7 ft wide, and 8 ft high. All of the walls were 6 in. thick and the inside clear dimensions were 12 ft by 6 ft by 7 ft. Each vault was cast in two identical sections, with the exception of the access opening, which was located in only one of the sections. During placement, one section was set into the excavation and the second section turned upside down and set on top to form complete closure. According to the manufacturer's shop drawings, all of the walls, the roof, and the floor were reinforced with Nos. 4 and 5 reinforcing steel in both directions, at the corners, and around all openings.

The vaults were set in place at the 20 and 40 psi locations in excavations that were 8 ft deep; accordingly, the top of the vault was at an elevation of 0.00 ft (Figure 6-1). Each vault contained an access opening 28 in. in diameter in the top section, and this opening was positioned on the side opposite ground zero. A piece of 24-in. diameter, 6-ft long reinforced concrete culvert or storm drain pipe was placed into each vault opening for personnel access after backfilling; a second piece of pipe was added to extend the access to the surface (Figure 6-2). The concrete pipe was Class III, the most typical manufactured, and was made in accordance with ASTM C76-76. (It should be noted that a 24-in. I.D. pipe does not provide the quick access that a 28-in. I.D. pipe does; it is recommended that 28-in. I.D. pipe be used.) The backfilling consisted of native soil around and over each vault to a depth of 3 ft. Figure 6-3 is an elevation schematic of a vault with the concrete pipe and the backfilling in place.

UPGRADING

The buried vault subjected to the 40 psi environment was upgraded by two methods. The roof of this vault was covered with 1-in. thick sheets of expanded polystyrene prior to backfilling (Figure 6-4), and the interior was shored, floor to ceiling, with two timber telephone poles shimmed tightly in the vertical position (see Figure 6-3). The two poles were located symmetrically with respect to the



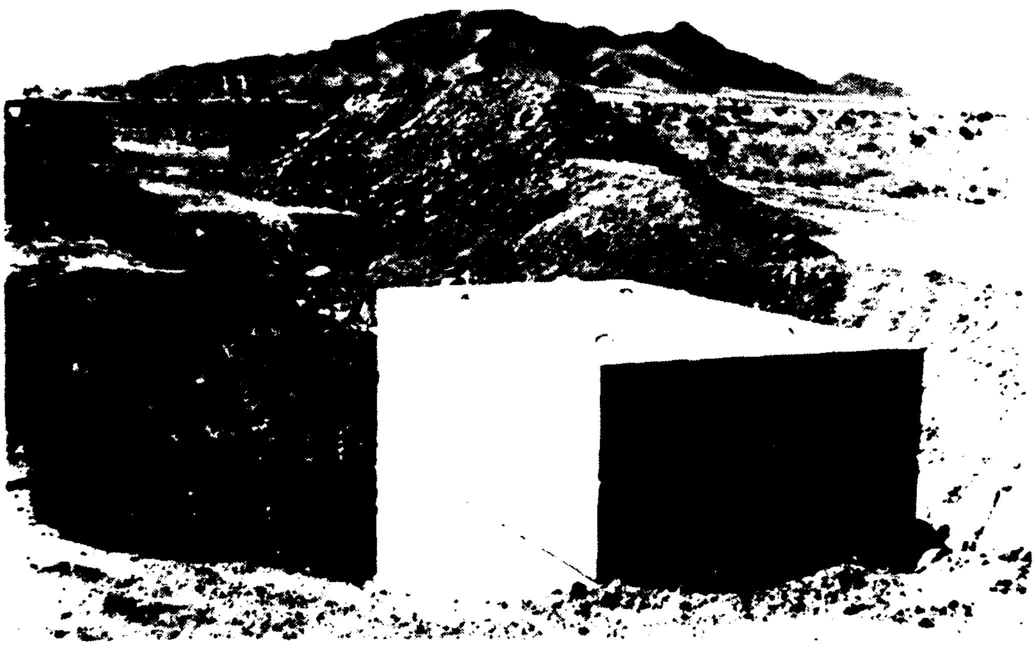


Fig. 6-1. Placement of Concrete Utility Vault Sections.

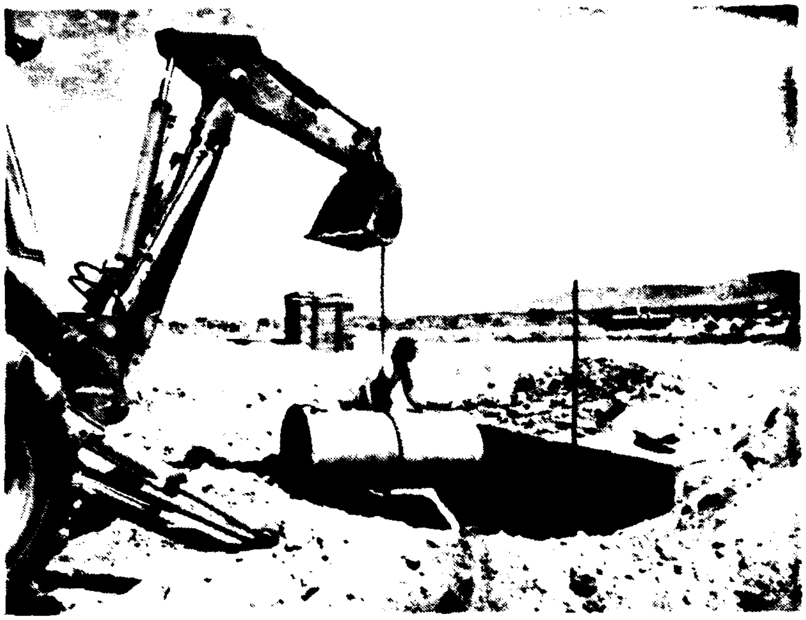


Fig. 6-2. Placement of Concrete Pipe Sections for Access Way.

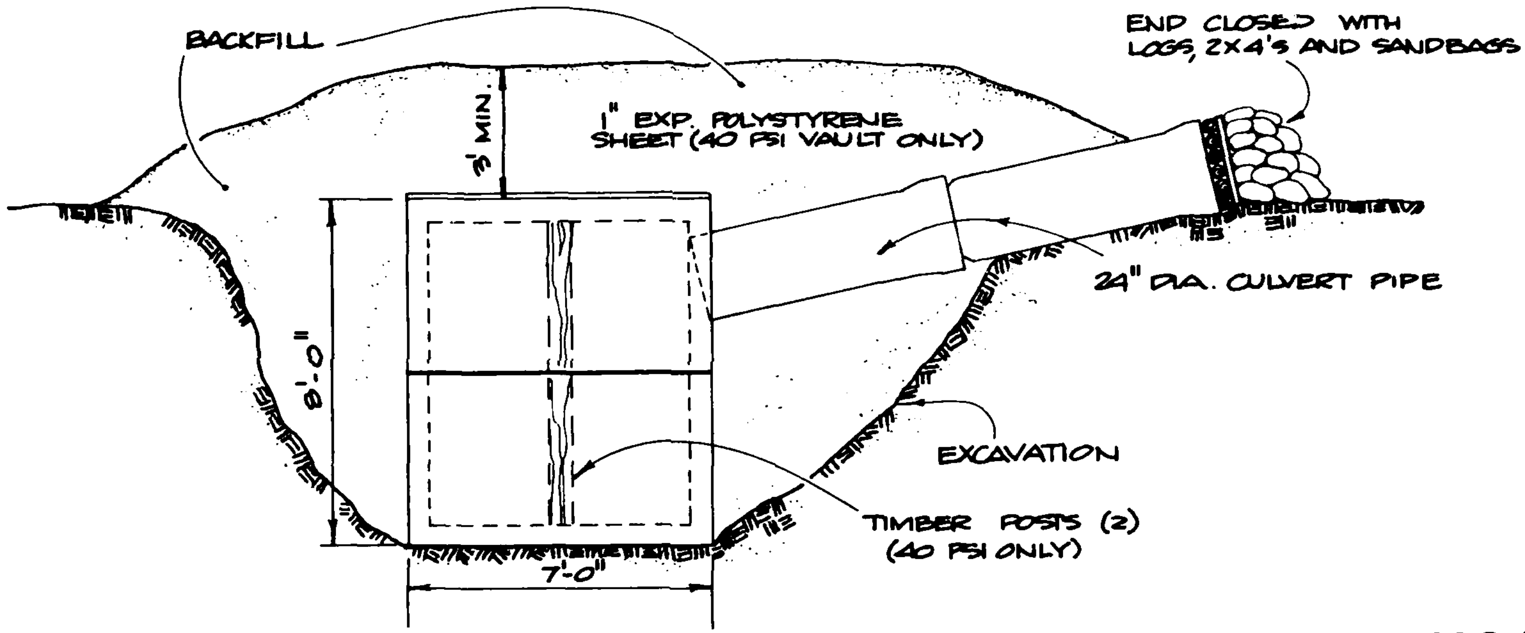


Fig. 6-3. Sketch of Buried Utility Vault With Access Pipe.

DNA No. 5101

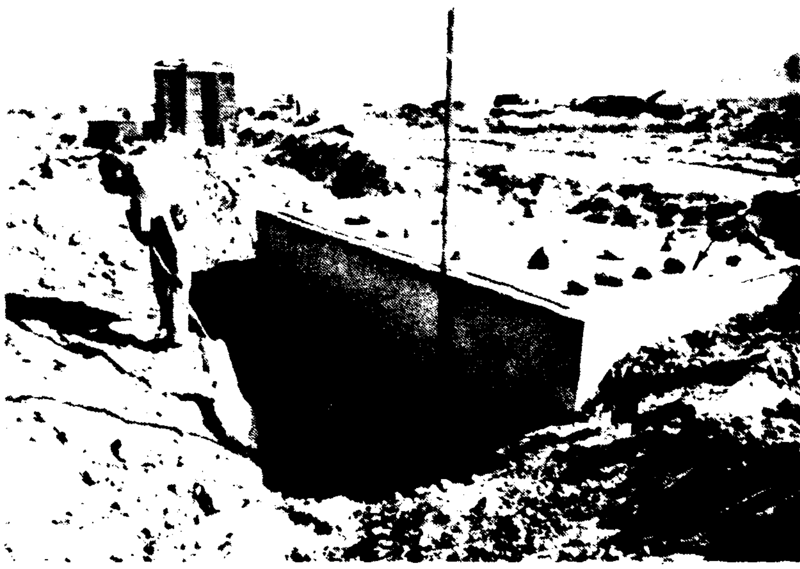


Fig. 6-4. Roof of Vault at 40 psi Environment.

floor/ceiling areas. In both vaults, the open ends of the 24-in. diameter culvert pipe (Figure 6-5) were closed with logs nailed together by means of 2x4's, then covered with sandbags (Figure 6-6).

INSTRUMENTATION

One anthropomorphic dummy with a self-recording accelerometer (Figure 6-7) was located in each of the vaults. For use as passive gauges, glass jars with contents were stored in each vault. The 40 psi vault contained an active pressure gauge and amplifier (Figure 6-8), the same model as previously described under DNA No. 5003.

TEST DATA, DISCUSSION, AND CONCLUSIONS

At the 40 psi range, the sandbags at the closure were scattered by the blast and the log barricade was shifted away from the entry access pipe (see Figure 6-9). However, the air pressure gauge in the vault indicated a peak positive pressure of 0.025 psi inside corresponding to arrival of the free-field positive phase, and a peak pressure of -0.20 psi inside, corresponding to the free-field negative phase. This indicates that integrity of the closure was maintained into the negative phase so that the differential load on the structure was the full 40 psi. (It should be noted that these closures were expedient for this test and not at all what would be required where access and egress are routinely needed for shelter inhabitants. Nevertheless, the observation clearly indicates that closures of this sort, unless modified, may not be adequate even for situations where access and egress aren't required, because closure integrity might have been lost too early in the pulse if it had been from a megaton weapon.)

The posttest environment inside the vault showed an accumulation of dust over all surfaces (e.g., Figure 6-10), but no real adverse conditions. One of the glass jars serving as a passive pressure gauge received an impulse sufficient to tilt it against an adjacent wall, where it remained undamaged (Figure 6-10). No cracks

Fig. 6-5. View From Interior of Vault Through Access Pipe.

Fig. 6-6. Sandbags Covering End of Access Pipe to Buried Vault.

Fig. 6-7. Instrumented Anthropomorphic Dummy Placed in Vaults.

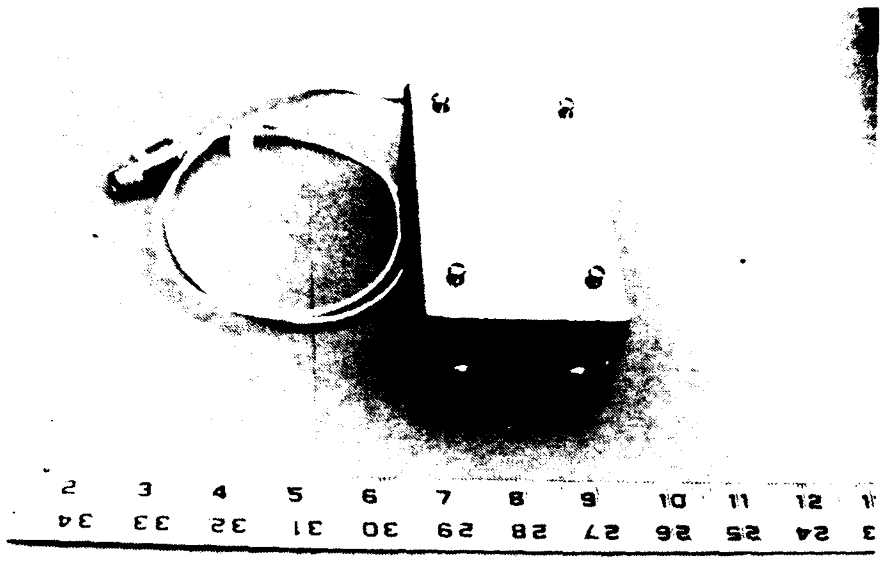


Fig. 6-8. Pressure Gauge and Amplifier Used in 40 psi Vault.

Fig. 6-9. Posttest Photograph of Entrance to 40 psi Vault.

Fig. 6-10. Posttest Photographs of Interior of 40 psi Vault.

were apparent in the vault, nor was there any evidence of relative motion between vault and access pipe; neither was there any apparent motion of the anthropomorphic dummy.

As there was no indication of even minor damage, it is concluded that this particular vault installation is likely to be adequate for overpressures of 40 psi (with appropriate closures), whatever the weapon size. Taking the most conservative possible point of view and applying the concepts in Ref. 12, it may be concluded that this vault installation would survive 20 psi from a multimegaton weapon in essentially the same condition as observed here at 40 psi from a 1-kiloton weapon. Moreover, as some vault damage could be sustained without seriously affecting contents, the failure rating under a multimegaton weapon loading is definitely something greater than 20 psi and quite possibly greater than 40 psi. Additional testing of vaults would be required to determine this, as well as to determine the effect of burial in different soil materials.

At the 20 psi ground range, the sandbags from the closure were displaced slightly, about two inches (though the displacement would have been greater for a larger weapon), and the logs were still in place. (There was no pressure gauge in this vault; the gauge originally scheduled was relocated between the two berms where one of the 5102 experiments was conducted.) The posttest environment inside the vault was similar to that in the vault at 40 psi, excepting there was a thinner layer of dust over all surfaces (Figure 6-11). No relative displacements appeared to have taken place, and there was no indication of motion of either the anthropomorphic dummy or the glass jar passive gauges. Neither were there signs of cracks or permanent deformations in the vault, despite the fact that no structural upgrading options (shores, or roof cover of expanded polystyrene) were installed.

Thus, it appears this vault installation without upgrading would be certain to survive at 10 psi in the same condition, even from a multimegaton weapon, and the failure rating without upgrading could be much higher. To establish the rating more exactly, or just what the performance might be in other soil materials, additional tests would be required, including one, or more, to failure.

Fig. 6-11. Posttest View of 20 psi Vault Interior.

Section 7
DNA NO. 5102 AND NO. 5103
INDUSTRIAL HARDENING

INTRODUCTION

These experiments were conducted at the 20 psi environment level, and consisted of testing various methods of protecting and securing industrial machinery, equipment, and storage containers. The methods used included berms, trenches, stabilization, tiedown anchors, and sandbags.

OBJECTIVE

One of the viable responses to crisis relocation with respect to industrial protection is a systematic reduction in the vulnerability of plant property and equipment. This response is referred to as "hardening". The objective of these experiments was to test and evaluate different methods of hardening (from the standpoint of the physical materials required - anchors, sandbags, etc.), the most appropriate methods for utilizing these hardening resources, and the practicality of personnel with little or no training and instruction performing hardening of this type within the time frame required.

CONSTRUCTION

The areas that were constructed for these experiments are described below:

Area 1 (5102)- A 12-ft long, 7-ft wide trench, 3 ft deep, with 45 degree sloping sides such as might readily be scooped out by a bulldozer.

Area 2 (5102)- Two 6-ft high berms, one on either side of a 5-ft wide level area. Both the berms and the level area were 12 ft long.

Area 3 (5102)- An approximate 12-ft-by-48-ft area of undisturbed natural soil.

Area 4 (5103)- A 20-ft-by-24-ft, 3-in. thick, unreinforced concrete slab on grade.

Area 5 (5103)- An approximate 12-ft-by-48-ft area of soil, which had been graded level as if prepared for surfacing.

The industrial equipment and materials that were hardened in these areas were:

1) Forty-eight 55-gallon steel drums, of the type that would be used for the storage of petroleum or chemical products. Each empty drum weighed 50 lb.

2) Four motorized 10-in. circular table saws, each weighing 115 lb.

3) Ten motorized cut-off band saws, each weighing 150 lb.

4) Twenty-two 1/10th-scale models of the 55-gallon drums, each of which was modeled to have the same ratio of drag to mass as the full-scale drums. These scaled drums were solid and modeled only the physical displacements. The initial assessment of these scale models was accomplished by Scientific Service, Inc., in a small-scale shock tube.

HARDENING

Area 1 (5102) - This area utilized a 7-ft wide, 3-ft deep trench to protect equipment. Four pieces of the industrial machinery, two band saws and two table saws were placed in the bottom of the trench unsecured, excepting that a 5-ft wide section of chain link fence was stretched across the trench to restrain one table saw. The chain link fence was secured by four 1-in. rebar stakes 24-in. long driven 18-in. into the ground at four corners located about 4 ft from the edges of the trench. Two 55-gallon drums, one sealed and empty, and one open and partly filled with water, were located standing upright in the trench and were left unsecured. This experimental arrangement is shown in Figure 7-1. The drum with water had a hand calculator (checkbook style) taped to it near the bottom on the outside. And an electronic instrument was submerged in alcohol in a plastic bag placed in a small hole dug in one of the back corners of the trench.

Area 2 (5102) - This area utilized two 6-ft high berms on each side of a 5-ft wide level area to protect equipment. Two band saws and one table saw were placed on the level area between the berms and left unsecured excepting for sandbags placed against the legs (Figure 7-2). Another hand calculator was taped to the underside of a 2x4 board fixed to a 4x4 stake, which had an air pressure gauge and amplifier attached to it (Figure 7-3).

Area 3 (5102) - This area utilized anchors and stabilization to protect items in a region of undisturbed natural soil. Eleven 55-gallon drums were located in this area arranged in groups of seven and three, with a single drum for a reference. Each of the multiple arrays of drums was strapped together tightly with seat belt webbing, but was not secured to the ground. All of these drums were completely filled with water (Figure 7-4), and the removable lids fastened back on. A rebar stake was driven into the ground at the leading edge of each array to mark its pretest position. Eleven of the 1/10th-scale model drums, strapped in identical configurations and not secured to the ground were also located in this area (see Figure 7-5).

Area 4 (5103) - This area utilized stabilization to protect items located on a 3-in. thick unreinforced concrete slab on grade. Both full-scale and 1/10th-scale model drums were located here. However, there was no single full-scale drum in this group because it had been found in shock tube tests that single drums on concrete surfaces are prone to overturning. Ten drums filled with water were

DNA No. 5102

Fig. 7-1. Area 1, DNA No. 5102.

Fig. 7-2. Area 2, DNA No. 5102.

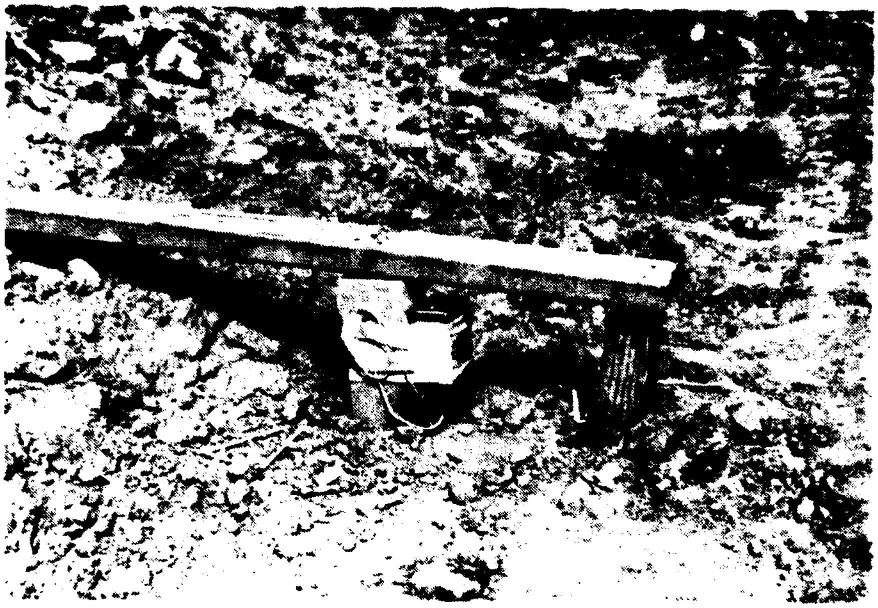


Fig. 7-3. Pressure Gauge, Area 2.

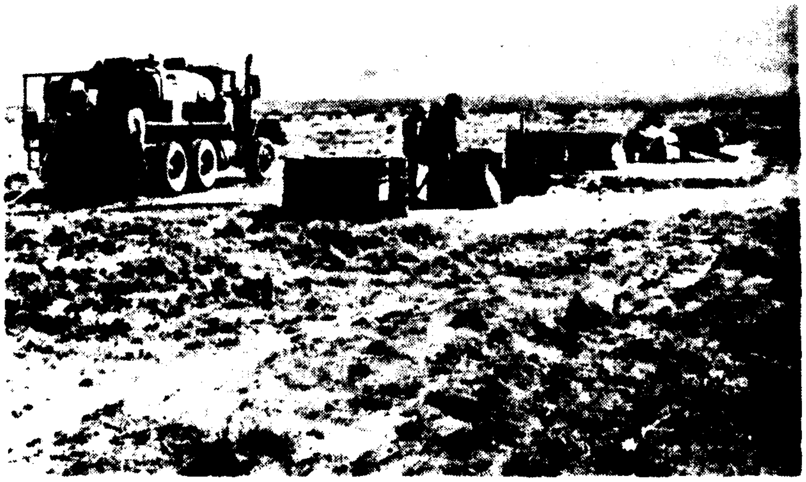


Fig. 7-4. Areas 3, 4, and 5, DNA No. 5102 and No. 5103.

DNA No. 5102 ,No. 5103

Fig. 7-5. Area 3, With Tenth-Scale Model Drums.

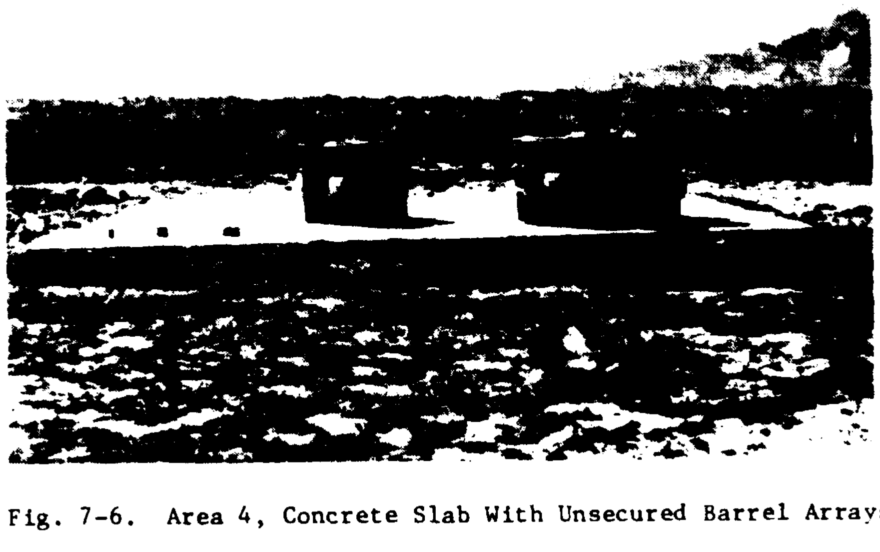


Fig. 7-6. Area 4, Concrete Slab With Unsecured Barrel Arrays.

strapped in arrays of seven and three, and not otherwise secured. Similarly, ten of the 1/10th-scale model drums were strapped in arrays of seven and three, with a single one for reference, and none of these was secured (Figure 7-6). Pretest positions of each of these arrays were marked with spray paint. The tops of the small-scale barrels were color coded so that if the drums overturned or broke free of the strapping in the arrays, the drum behavior could be identified (Figure 7-7).

Area 5 (5103) - This was another area of soil, graded level (the area beyond the slab visible in Figure 7-4) with a variety of experiments. A group of ten drums, each filled completely with water, was again strapped in arrays of seven and three; and another group of eleven drums, closed and sealed but one-third to one-half empty, was also strapped in arrays of seven and three, and placed together with a single reference drum on the graded area and left unsecured. All drums were coded so each could be identified as to the array from which it originated.

The industrial equipment used in this area consisted of: one table saw and two band saws completely exposed and unsecured other than for sandbags against the legs (Figure 7-8); two band saws protected by a berm of sandbags stacked nine high (the berm had the shape of a right triangle with the sloping side toward ground zero and required 21 bags per lineal foot, Figure 7-9); and two band saws protected front and back by stacks of lumber (triple rows of 2x4's) to the height of the saws, with the open areas between the lumber stacks filled with sandbags, stability against overturning provided on the downstream side, and the entire package anchored to the soil with expedient soil anchors fastened by webbing (Figure 7-10). One pair of anchors and a length of webbing were needed for each linear foot of package exposed to 20 psi overpressure.

A typical soil anchor is shown in Figure 7-11. One of these anchors was attached to each end of the seatbelt webbing as shown in Figure 7-12. The anchors were placed by pulling the webbing taut, setting the anchor on the soil, and then standing on the plate above each of the three ridged feet, in turn, and rocking side to side until the feet were buried and the plate was flush with the soil. Two 4-ft by 8-ft sheets of 3/4-in. plywood were then laid over the five anchors (Figure 7-13) on the ground zero side, and the plywood was covered with a few inches of soil to keep the overpressure from getting under the anchor plates (Figure 7-14). Thus, it was

Fig. 7-7. Small-Scale Barrels in Area 4.

Fig. 7-8. Area 5, Exposed Equipment.

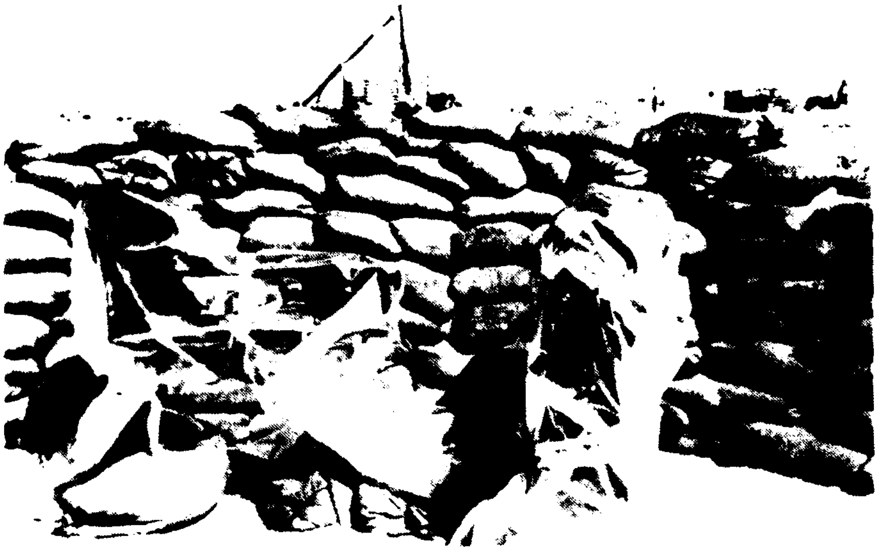


Fig. 7-9. Area 5, Berm of Sandbags Protecting Equipment.

Fig. 7-10. Area 5, Fully Protected Equipment Package.

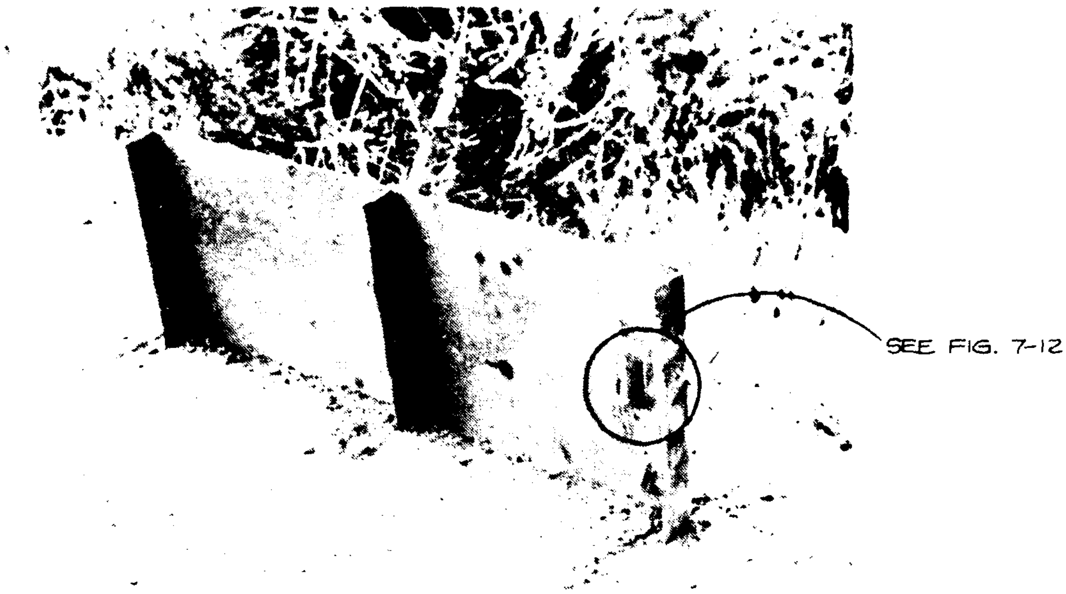


Fig. 7-11. Expedient Soil Anchor.

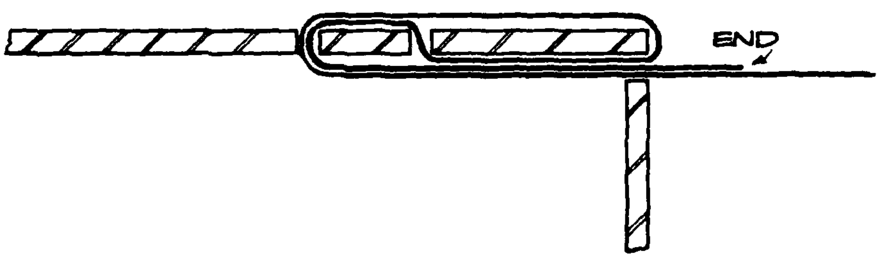


Fig. 7-12. Sketch of Soil Anchor/Webbing Arrangement.

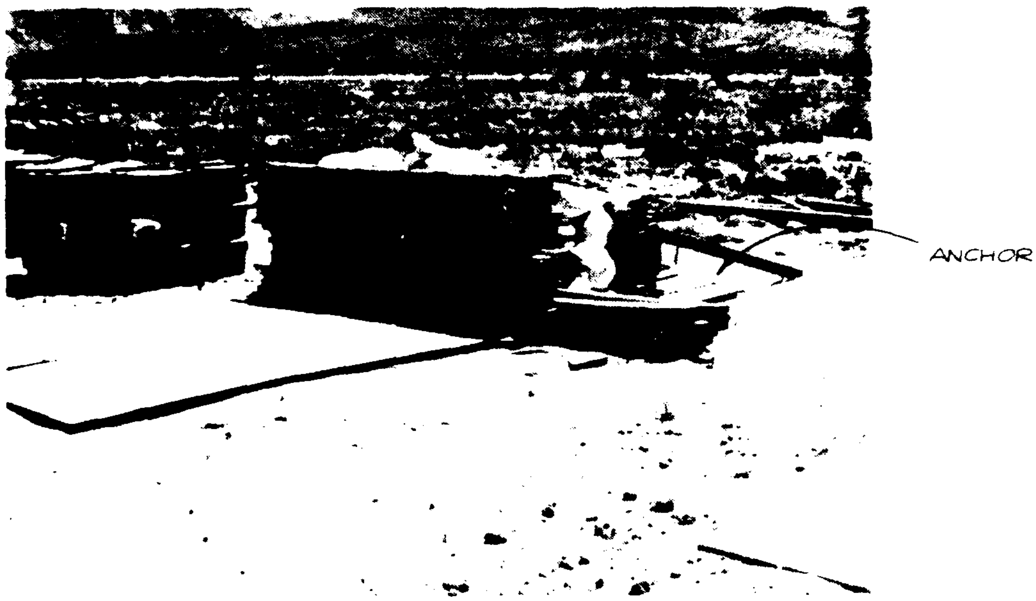


Fig. 7-13. Plywood Placed Over Soil Anchors.

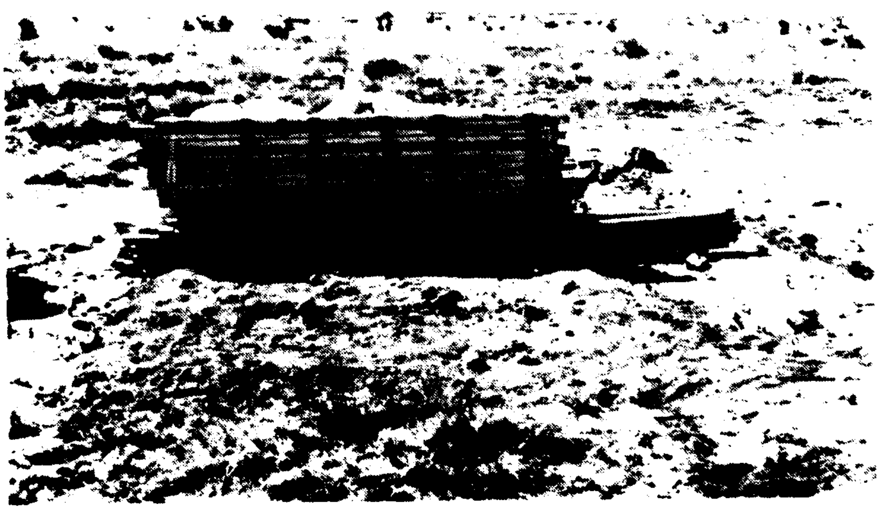


Fig. 7-14. Soil Covering Plywood Over Soil Anchors.

expected to demonstrate whether the 20 psi static overpressure (on the anchors) might be used as an aid to secure the package against the 7 psi dynamic pressure pulse.

A second package, made up of poles and anchored in a similar fashion but with the restraint at right angles to the blast, was used to test the expedient anchors without the plywood cover (Figure 7-15).

INSTRUMENTATION

The instrumentation for these experiments comprised still photography, displacement measurements using fixed benchmarks, and a pressure gauge (located between the two berms).

TEST DATA

Area 1 (5102) - In the trench (Figure 7-1), one table saw and two band saws survived with minor damage; one table saw received major damage; the empty, sealed drum was moderately crushed; the partially filled, open drum and its contents were apparently unaffected; the electronic equipment in this area (Figure 7-16) survived - the checkbook style hand calculator was scarred, but operable; and the digital ohmmeter, which had been submerged in isopropyl alcohol, was unmarked and operable. Posttest photographs of the trench area and artifacts are shown in Figures 7-17 and 7-18.

The table saw that was restrained by the chain link fence was still standing after the shot (Figure 7-17). This saw received minor damage; i.e., the table extension was broken off on impact of the other table saw, and one of the sawblade adjusting wheels came off, but the latter was readily replaced and the saw was immediately serviceable. The second table saw, which was originally on the same side of the chain link fence as the barrels (Figure 7-1), was lifted by the pressure pulse that reflected off the bottom of the trench and dropped, upside down, onto the chain link fence on the side with the band saws (Figures 7-17 and 7-19). On

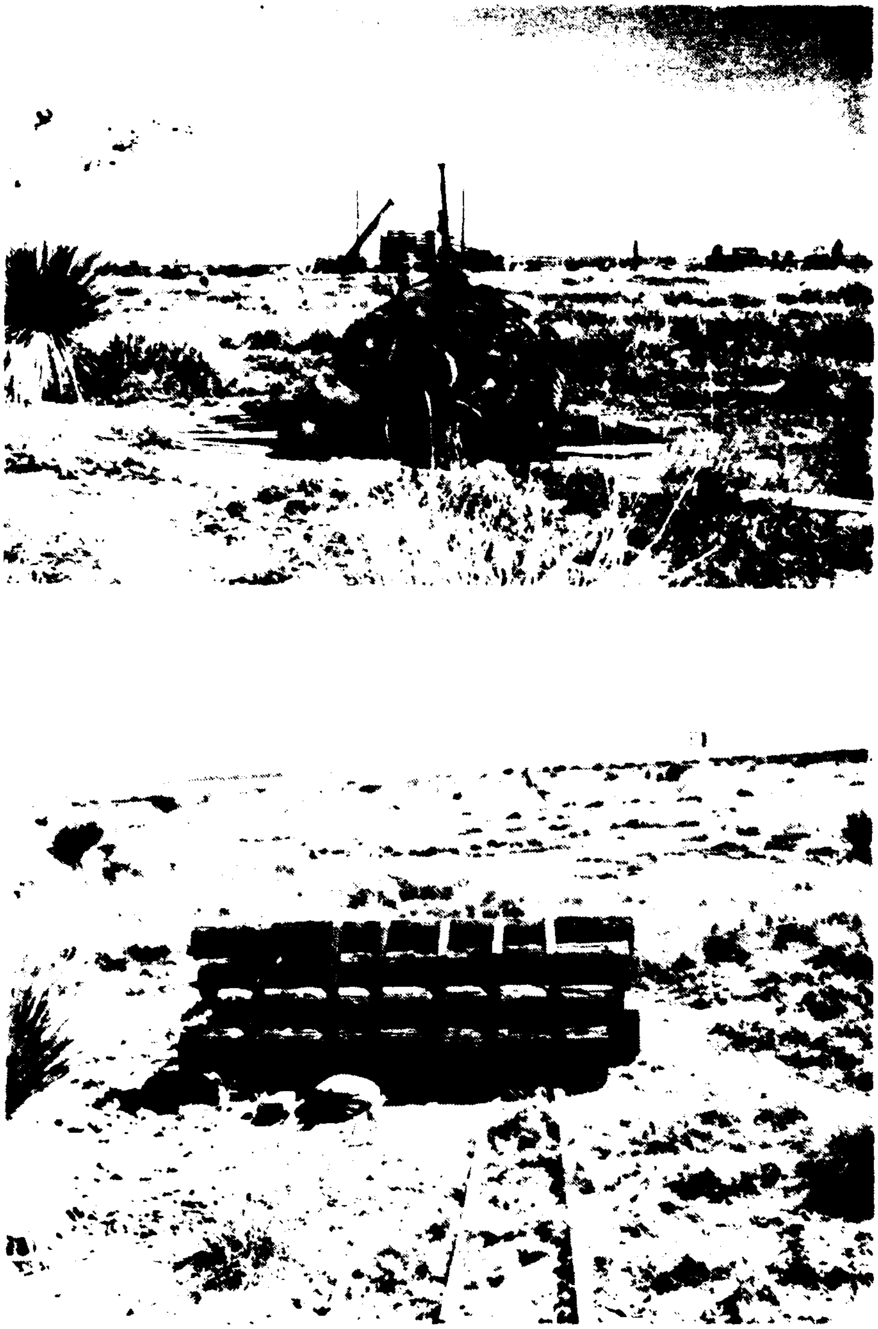


Fig. 7-15. Area 5, Test of Expedient Anchors Without Soil Cover.

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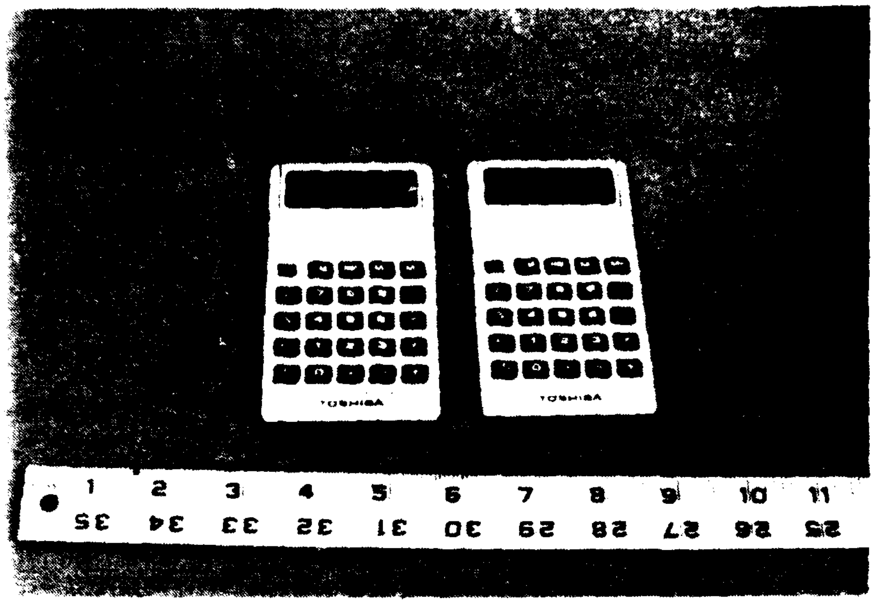

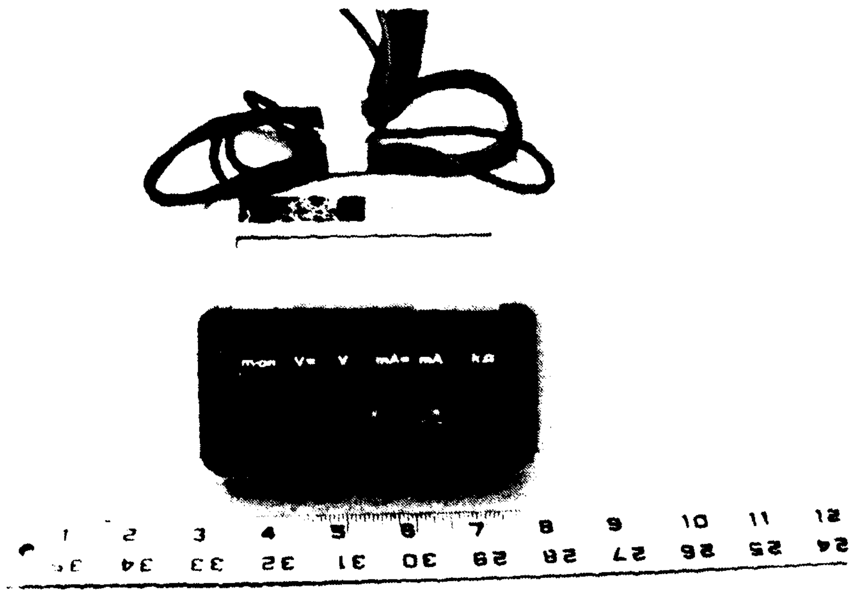

Fig. 7-16. Posttest Photographs of Electronic Equipment, Area I.

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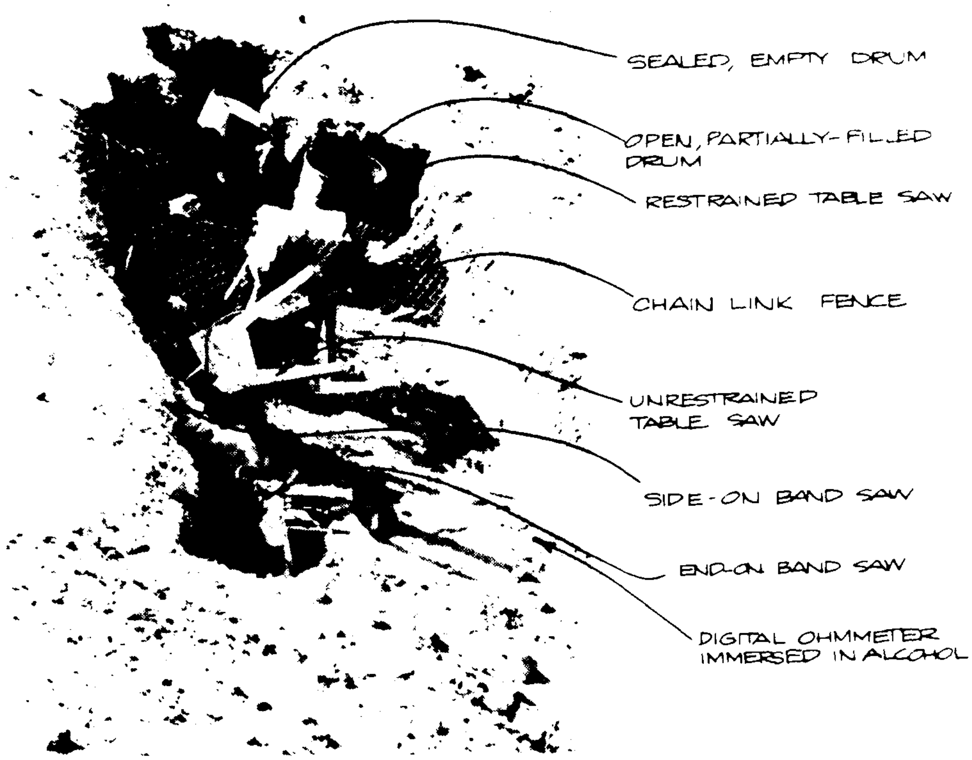

Fig. 7-17. Posttest Photograph of Area 1 Trench and Equipment.

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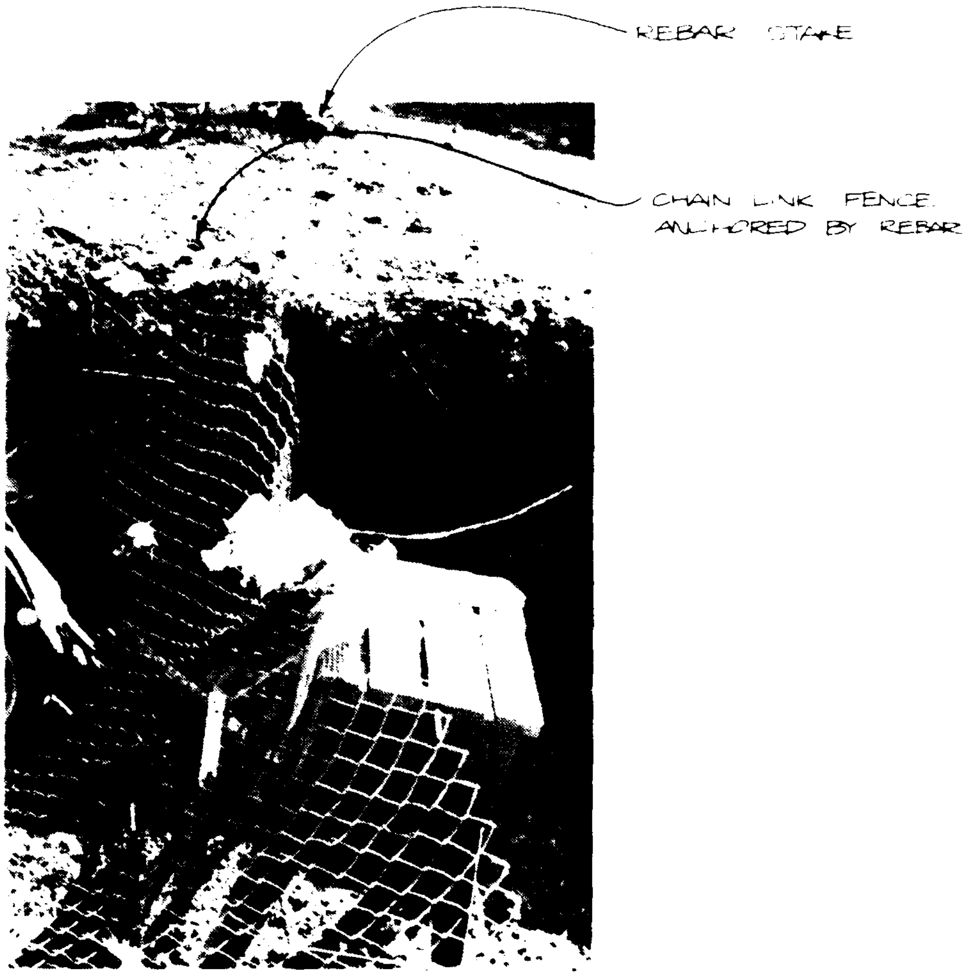

Fig. 7-18. Posttest Photograph of Area 1 Trench.

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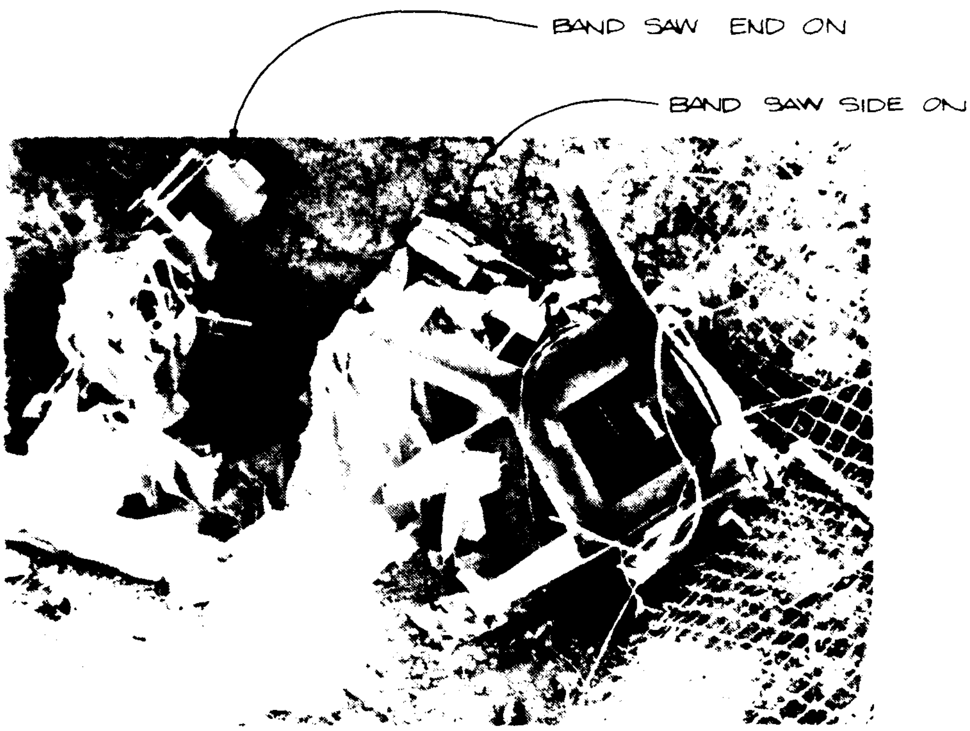

Fig. 7-19. Band Saws in Trench, Area 1, Posttest.

impact, this saw broke the table extension on the other table saw (Figure 7-17), broke its own table casting, and bent its motor and sawblade mounting so that it was no longer serviceable.

The band saw that was oriented end-on to the blast (Figure 7-19, and see Figure 7-1) lost the pulley cover but otherwise suffered little serious damage. The other band saw, oriented side-on in the trench, was tilted into the trench wall, but suffered little damage (Figure 7-19). Both band saws had the bottom plate that makes up the leg-assembly dished upward by the pressure pulse when the pulse reflected off the bottom of the trench.

Figure 7-16 showed photographs of the calculator and the digital ohmmeter, posttest. Neither was seriously damaged and both were immediately serviceable -- principally because both were protected from missile damage, and neither package could undergo much volume change when the pressure pulse hit.

Area 2 (5102) - Between the berms, only one of the band saws survived with minor damage. The other band saw, and the table saw, suffered major impact damage when they collided, and both were inoperable. The checkbook style calculator appeared undamaged and was immediately operable. Figure 7-20 is a posttest photograph of the area. It appears that the two outermost saws (see Figure 7-2) were blown inward and collided as the wave diffracted around the berm, while the center saw was blown toward the back berm. The forward berm apparently eroded, causing a small avalanche of loose soil to pile around the saws and shrink the spacing between berms. The pressure gauge registered a short pressure spike of about 38 psi when the diffractory waves collided.

The band saw that survived with minor damage had the on/off switch sheared off. These particular saws are designed with the band assembly hinged at one end so that the weight of the assembly provides the cutting force, and so that when the cut is completed this assembly descends to contact the on/off toggle switch and automatically shut off the saw. The result of this design is an unusual vulnerability of the switch to impact of the hinged assembly, but a relatively simple repair job.

Fig. 7-20. Posttest Photograph of Area 2.

The second band saw had a cracked casting, which would make it dangerous to use. The cause was most likely the impact of the table saw.

The table saw also suffered from a cracked main table casting. The cause of damage to table saws was the same in all cases: on these saws there is a shaft for mounting the safety fence attached to the table assembly; this shaft is vulnerable to impact damage because it protrudes six inches out the main body. A major impact on this lever arm is all that is needed to fracture the table assembly.

The peak overpressure recorded between the berms was a 1 to 2 ms spike, roughly twice the magnitude of the free-field overpressure (Figure 7-21). The spike was expected, the objective of the berms was not to reduce the overpressure (which is not the major problem), but rather the drag force.

Area 3 (5102) - In the open area where the natural soil was undisturbed, the three-barrel arrays and the single reference barrels, at both scales, were overturned; the stability of the seven-barrel arrays at both scales was sufficient to prevent overturning (Figures 7-22 and 7-23).

The bung had been left open on the single barrel, which was found nearly empty posttest and with its center section compressed somewhat (Figure 7-24). In addition, the barrel had been translated a total distance of 4 ft (two barrel diameters) where it was found overturned.

The three-barrel array also translated a total distance of 4 ft (one array diameter) and was overturned, but the array remained intact, all the lids remained firmly affixed and all the contents were still inside the barrels (Figure 7-25).

The seven-barrel array translated a total distance of 1 ft (0.16 array diameters). Some of the lids on these full-scale barrels did not seal well (the barrels were used/recycled) so that one lid was blown off entirely and two were blown partly off (Figure 7-26). About 15% of the contents of the barrel that lost the lid was also missing.

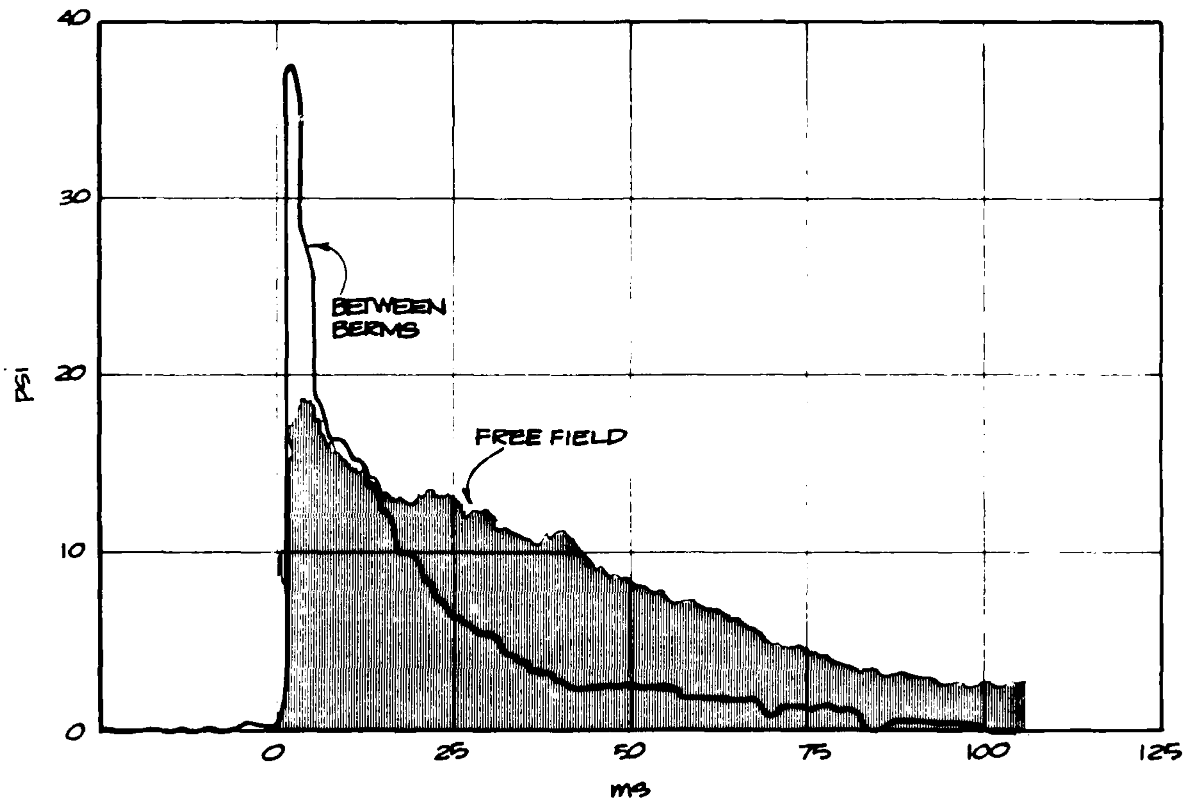

Fig. 7-21. Peak Cverpressure Between Berms Compared With Free-Field Overpressure.

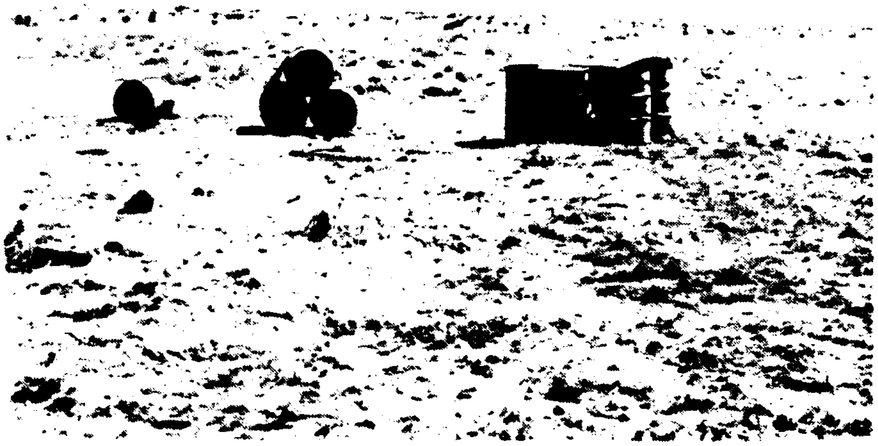

Fig. 7-22. Full-Scale Barrels, Area 3, After Test.

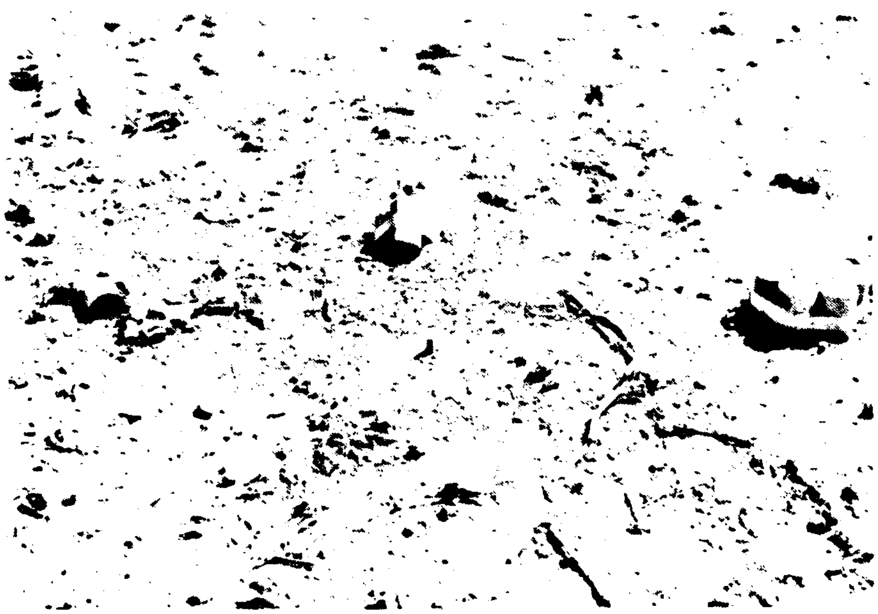

Fig. 7-23. Small-Scale Barrels, Area 3, After Test.

Fig. 7-24. Single Barrel, Area 3, After Test.

Fig. 7-25. Three-Barrel Array, Area 3, After Test.

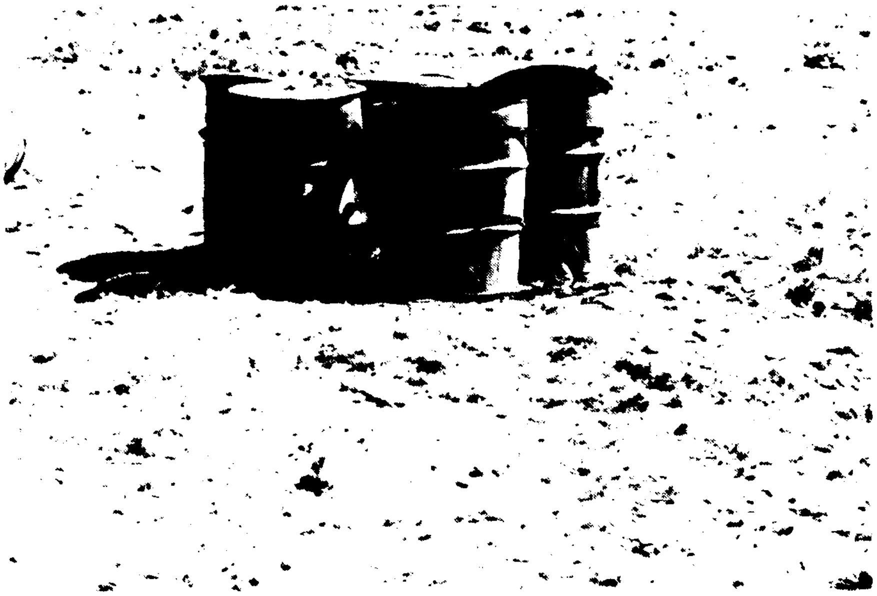

Fig. 7-26. Seven-Barrel Array, After Test.

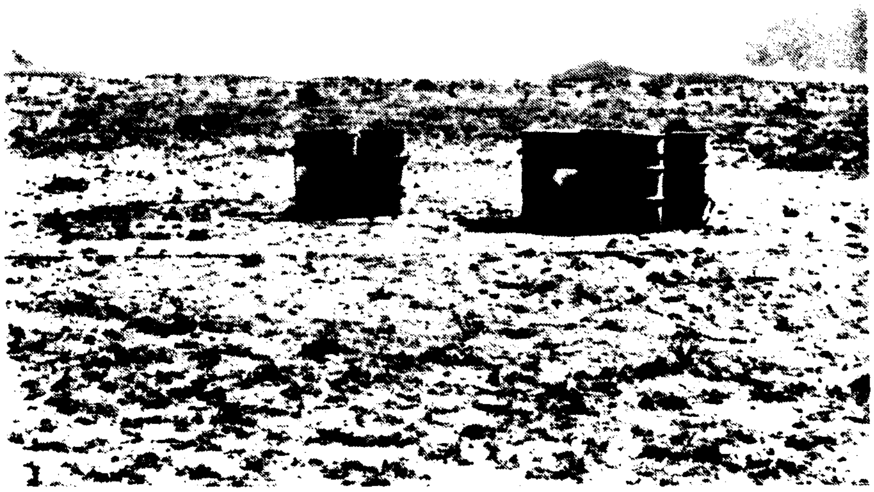

Fig. 7-27. Posttest View of Area 4.

The single 1/10th-scale barrel was translated three barrel diameters and overturned. The three-barrel array at 1/10th-scale was translated three array diameters and overturned. The seven-barrel array at the 1/10th-scale translated 0.33 array diameters and remained upright (Figure 7-23).

Area 4 (5103) - On the concrete slab, the two water-filled full-scale barrel arrays retained stability and merely slid, but at the 1/10th-scale, only the seven-barrel array retained stability (Figure 7-27).

There was no single full-scale barrel on the concrete slab. The three-barrel array slid 3/4ths of an array diameter, and rotated a foot off center (Figure 7-28), but did not overturn. All the lids were blown off (Figure 7-29) and one of the drums distorted out-of-round about 2½ inches. The other two drums in this array did not distort visibly, but all three drums in the array lost 15% to 20% of the contents.

The seven-barrel array slid 1/6th of an array diameter, and rotated a foot off center. One of the barrels lost a lid and 15% of its contents, and was distorted at the top end, but the others all maintained their integrity (Figure 7-30).

The single 1/10th-scale barrel overturned and translated nine barrel diameters, and may have gone end-over-end several times (see Figure 7-31).

The 1/10th-scale three barrel array translated three array diameters and flipped over onto its top (Figure 7-32).

The 1/10th-scale seven barrel array slid one array diameter (Figure 7-33).

Area 5 (5103) - In the area where the soil was graded level, the ten water-filled drums in the group that duplicated the three- and seven-barrel arrays in Area 3 also duplicated the response observed in Area 3 (Figure 7-34). Most of the eleven-barrel group that had been partially filled with water suffered from crushing, and the lids and half or more of the contents were missing (Figure 7-35). The two band saws protected by the lumber package were undamaged and immediately

Fig. 7-28. Posttest Photograph of Three-Barrel Array, Area 4.

Fig. 7-29. Posttest Photograph of Three-Barrel Array, Area 4.

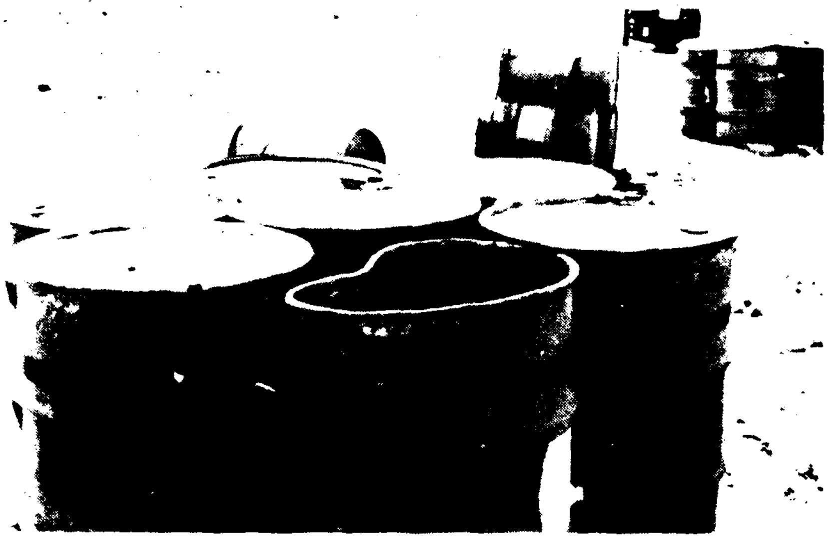

Fig. 7-30. Seven-Barrel Array on Slab, Area 4, After Test.

Fig. 7-31. Area 4, Tenth-Scale Barrels After Test.

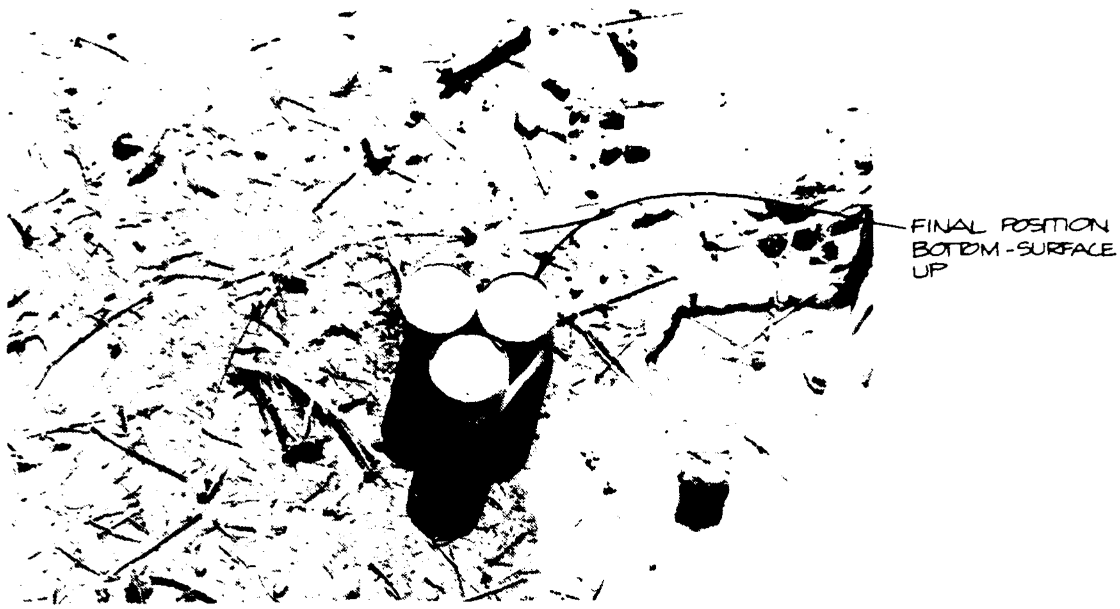

Fig. 7-32. Small-Scale Three-Barrel Array, Area 4.

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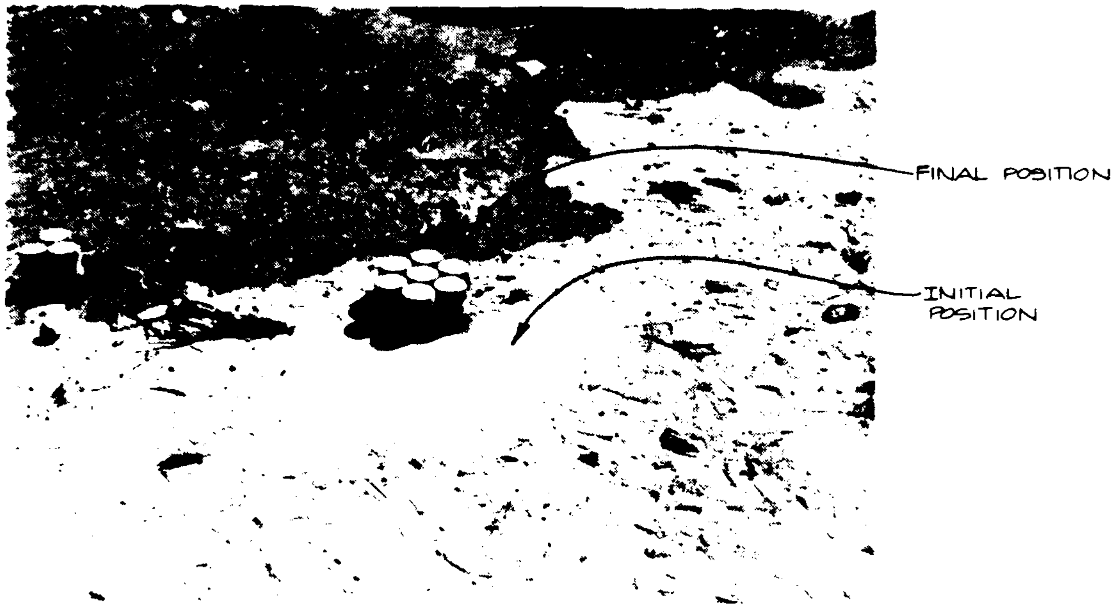

Fig. 7-33. Area 4, Tenth-Scale Seven-Barrel Array.

Fig. 7-34. Area 5, Three- and Seven-Barrel Arrays After Test.

EXPOSED TABLE SAW

Fig. 7-35. Posttest Photograph of Partially Filled Barrels, Area 5.

Fig. 7-36. Posttest Photograph of Three-Barrel Array, Area 5.

operable. One of the two band saws protected by the sandbag berm was undamaged and immediately operable, while the other suffered minor damage -- a sheared on/off switch. The band saw oriented end-on to the blast in the free field survived and was immediately operable, but the saw oriented side-on to the blast in the free field was severely damaged because of a broken casting. The table saw in the free field survived and was immediately operable, but the table extension was broken off. The log pile restrained by expedient anchors oriented at right angles to the blast remained anchored.

There was no single barrel in the ten-barrel group of water-filled drums. The three-barrel array translated a total distance of 4 ft (one array diameter) and overturned. The lid came off one of these drums, the one on the bottom right side of the array (Figure 7-36), and all the fluid spilled, but the other two drums maintained their integrity and contents. The results essentially duplicated those in Area 3.

The seven-barrel array visible in Figure 7-34 translated a total distance of one-third of a foot in the direction of the blast wave. Two lids came off altogether and one was partially lifted off, but there was little distortion in any of the drums. The lidless drum clearly visible in the figure lost 40% of its contents and the other lidless drum lost 25% of its contents, not unlike the events in Area 3.

Among the barrels in the group that was partially filled, the single barrel translated about the same total distance as the single barrels that were completely full, i.e., 4 ft, and it also overturned. In addition, it lost both its lid and contents and became slightly crushed in the middle, but not at the ends (the barrel marked **D'** in Figure 7-37).

The three-barrel array in this group translated a total distance of six feet (Figure 7-38 - which also shows the benchmark that indicates the array's initial position), and the array came apart. Two of the drums were overturned but maintained their integrity and contents, while one of the drums lost its lid and most of its contents and was severely distorted, but remained upright.

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Fig. 7-37. Area 5, Partially Filled Barrels After Test.

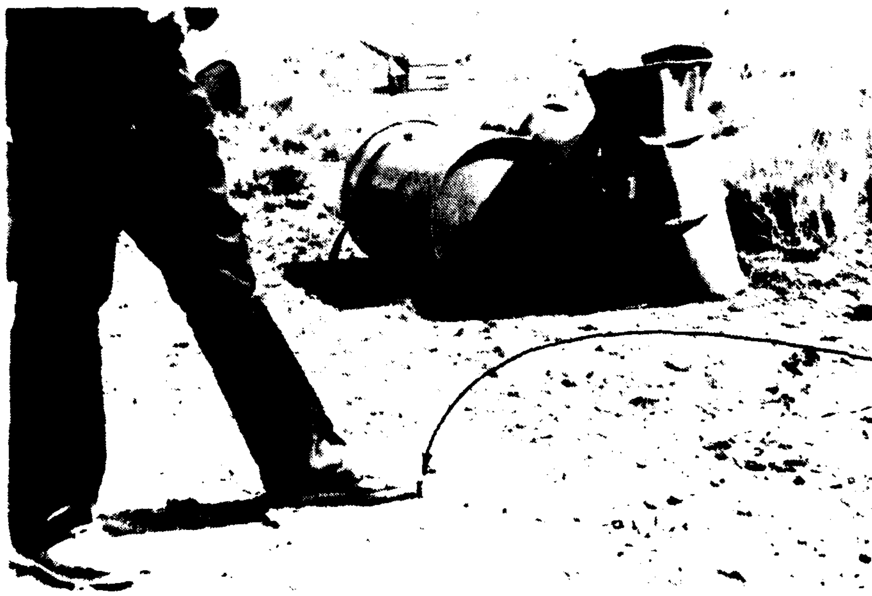

Fig. 7-38. Area 5, Three-Barrel Array After Test.

The seven-barrel array translated one half a foot. Five of the seven barrels lost their lids and half their contents (leaving them about 1/4th full) and these barrels were severely crushed. The center barrel, which retained its lid and contents, was sufficiently protected that it was not crushed. And the remaining barrel that retained its lid and contents was crushed at the top, by the tension in the webbing that crossed it there (Figure 7-35).

The lumber stack used to protect two band saws was partially overturned Figure 7-39), but maintained its integrity and remained anchored. Partial overturning occurred because only the bottom half of the package was stabilized by logs piled behind it. The total load on the package, which was oriented to provide the maximum drag, was slightly over 30,000 pounds. Thus, the load per anchor and the tension in the webbing corresponded to, roughly, 5,000 pounds. The effects of this tension can be seen in Figure 7-40 where the webbing cut into the lumber along the junction of the bottom and front faces of the package. The band saws recovered from inside the package suffered no apparent damage and were immediately operable.

The sandbag berm used to protect two band saws had the top two layers of sandbags blown off (Figure 7-41) and some of these were found 40 to 50 ft downstream. The band saw oriented side-on to the blast wave was tilted towards ground zero, and the one oriented end-on was tilted sideways about 30 degrees where it contacted some sandbags that had been knocked off the top of the berm. The pulley covers on both saws were opened by the blast but were not torn loose. The saw that was oriented end-on had the on/off switch sheared off, probably from the impact of a sandbag on the hinged arm, but otherwise neither saw suffered significant damage. One saw was immediately operable and the other required only minor repair.

The band saw located in the free field and oriented end-on to the blast wave translated about 2 ft downstream and overturned (Figure 7-42). This saw suffered no apparent damage and was immediately operable. The band saw located in the free field that was oriented side-on to the blast was found in five pieces, with the two major pieces (the heavy castings) and one leg ending up 24 ft downstream (Figure 7-43). The bottom plate from the leg assembly was found 74 feet

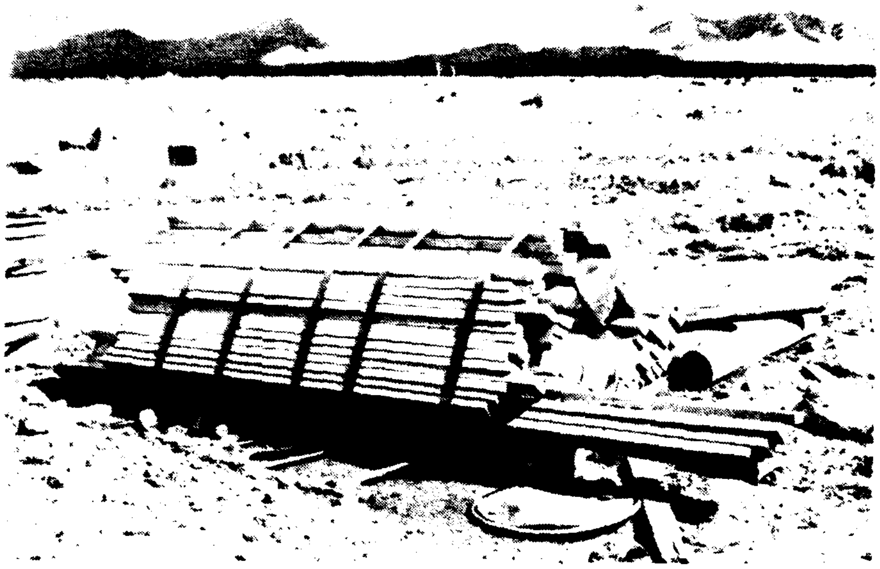

Fig. 7-39. Area 5, Lumber-Protected Equipment Package After Test.

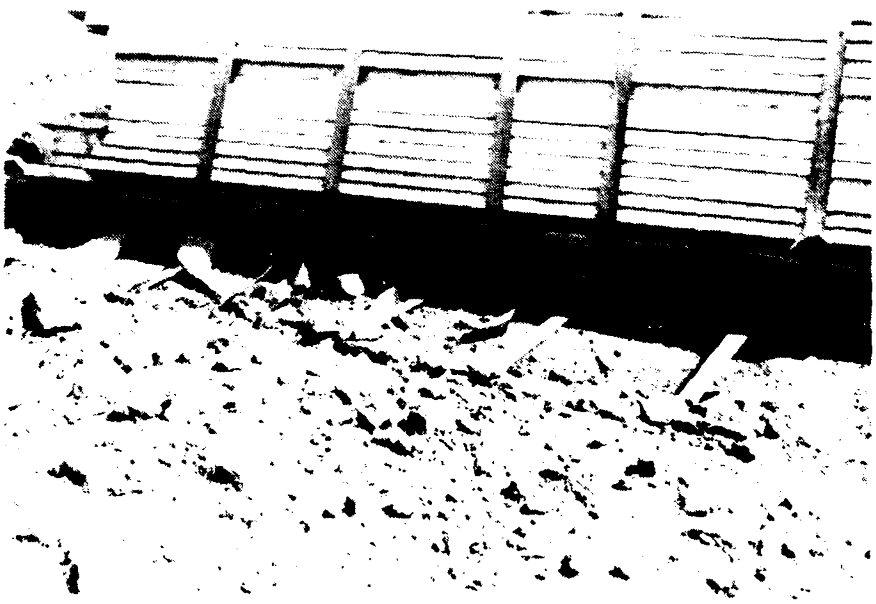

Fig. 7-40. Closeup Photograph of Lumber Stack Showing Cuts Caused by Webbing.

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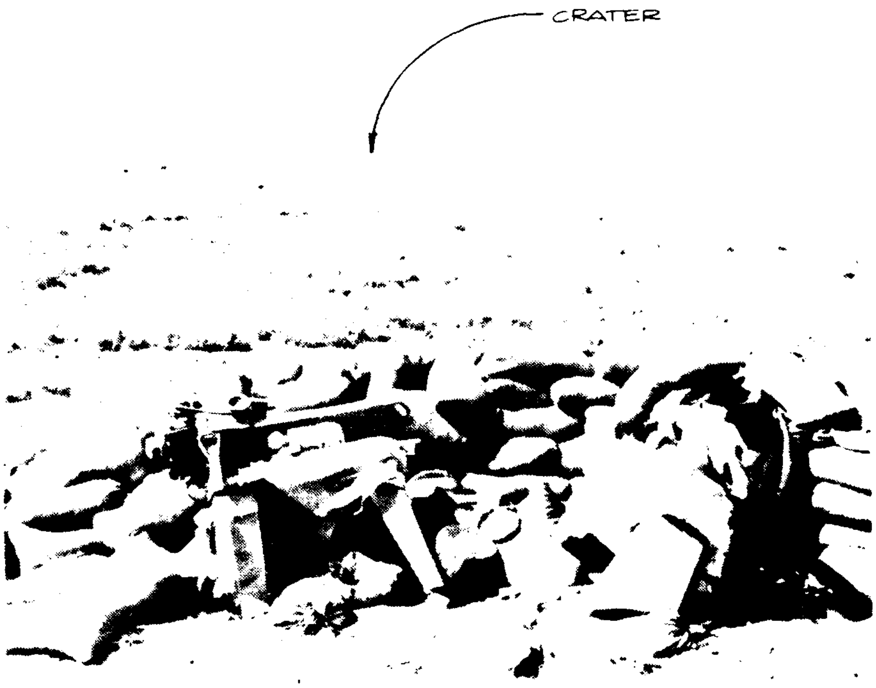

Fig. 7-41. Posttest Photograph of Sandbag Berm Protecting Band Saws.

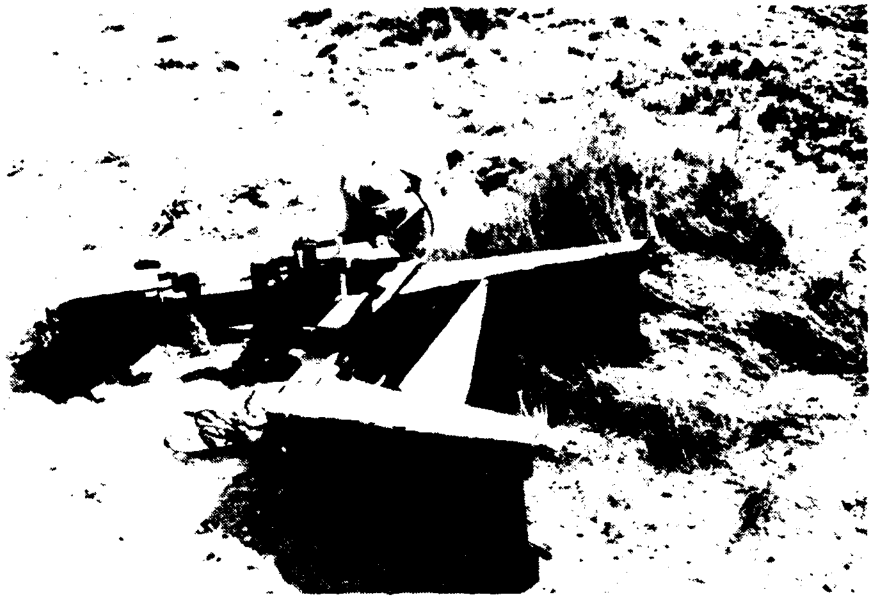

Fig. 7-42. Area 5, Band Saw (End-On Orientation) After Test.

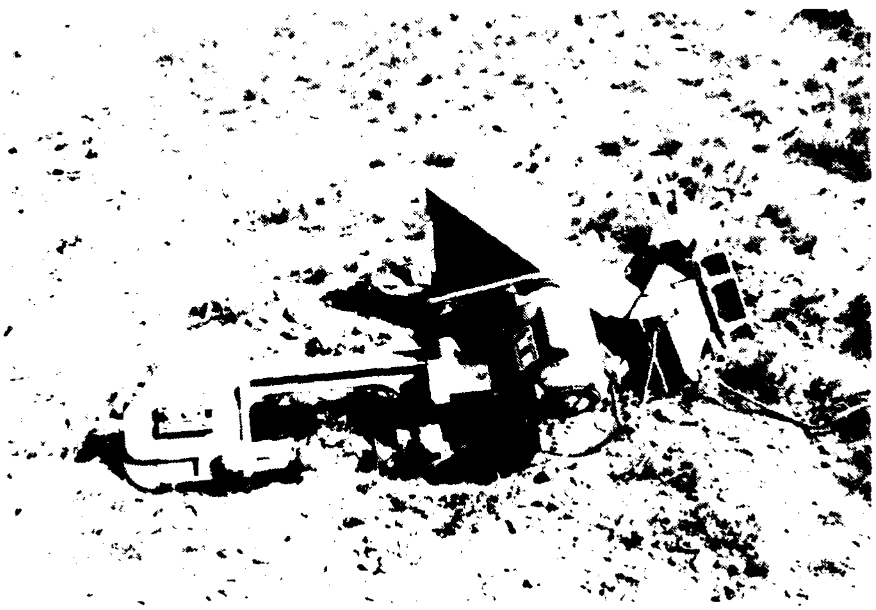

Fig. 7-43. Area 5, Band Saw (Side-On Orientation) After Test.

downstream, one of the legs was found 125 feet downstream, and the pulley cover was found 120 feet downstream. One of the two castings was broken so that the saw would have been inoperable if reassembled, and it could not have been easily repaired.

The table saw that was located in the free field was translated 25 ft downstream and overturned (see Figure 7-35), but only the on/off switch was damaged - apparently because this saw did not impact at any time on the vulnerable mounting appendage for the safety fence. The switch was readily repaired, and the saw was operable in a matter of minutes.

The stack of logs packaged and anchored so as to be exposed end-on to the blast with the anchoring force at right angles to it (Figure 7-15) remained anchored, but there was some minor rearrangement of the logs (Figure 7-44). It appears the lead expedient anchor was pulled loose (Figure 7-45, right-hand bottom corner, and center foreground in Figure 7-46).

DNA No. 5103

Fig. 7-44. Posttest Photographs of Anchored Log Package (compare Figure 7-15).

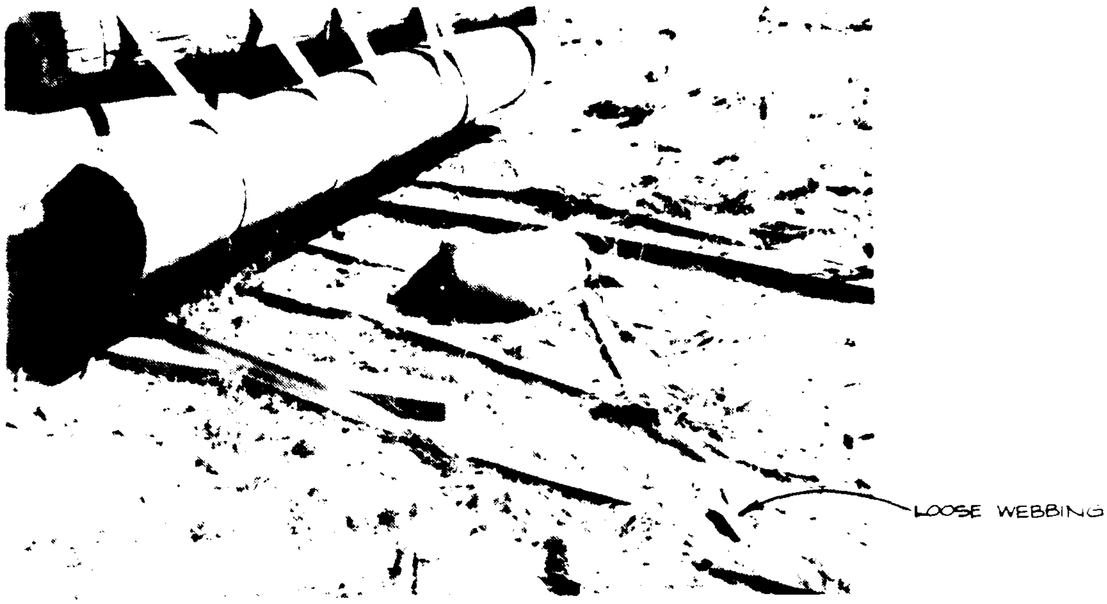

Fig. 7-45. Closeup Photograph of Log Package Anchoring.

Fig. 7-46. Closeup Photograph of Loosened Anchor.

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TESTING OF SHELTER DESIGN AND INDUSTRIAL HARDENING CONCEPTS AT --ETC(U)
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DISCUSSION AND CONCLUSIONS

Lightweight machine tools were subjected to blast waves at a ground range corresponding to 20 psi peak overpressure. These machine tools were selected to represent the more vulnerable types of industrial equipment in terms of blast loading. A portion of these machine tools were deliberately exposed both to static and to dynamic overpressure (hence, to drag forces and drag related phenomena), while others were protected to reduce exposure to the dynamic overpressure and its related effects without concern for protection against static overpressure effects. The conclusions are that the damage observed to lightweight machine tools at this overpressure level is principally a consequence of impact phenomena and not due to pressure, per se. Thus, eliminating the opportunity for overturning, sliding, or tumbling (and missile impacts) will eliminate most physical damage to a large part of industrial equipment. The nature of MILL RACE was such as to guarantee an essentially missile-free environment (unless a specific effort were made to change this circumstance) so that it was possible to concentrate on evaluating the difference between pressure and drag phenomena as damage mechanisms.

Notwithstanding survival in the free field (with relatively minor damage) of some of the test articles in the MILL RACE experiments, it is apparent that survival in the free field was principally a matter of "luck." That is, survival in the free field will be a random statistical event, with the probability that little or no damage will occur being indicated by a fraction that is considerably closer to zero than to one. The objective of hardening is to "stack the deck" so that this survival fraction is systematically moved closer to one (which represents certainty of survival.)

Hardening methods that use too many resources (materials or time) to implement are less desirable than those that are simply done with resources at hand. Moreover, methods that enable industries to remain in production will be much more desirable than those that require complete plant shutdown. For these reasons, some of the methods tested at MILL RACE will be more acceptable to industry decisionmakers than others.

For example, packaging lightweight machinery in stacks of lumber works very well but requires such machinery to cease production, whereas sandbag berms work almost as well but would enable production to continue until the last few minutes before an attack (assuming key worker shelters were available locally). The importance of evaluating different options is to enable plants without the space or resources to apply one option -- e.g., to erect sandbag (or dirt) berms -- to apply another, such as to use lumber, tires, or some other packaging materials for hardening.

Neither the trench nor the dirt berm appeared to work as well as the sandbag berm, but that is somewhat illusory. In all cases, the machine tools placed very close to vertical walls that were interposed between the test article and ground zero, whether above or below grade, did very well because they were less affected by the flow processes (viz, the test articles in the forward part of the trench and those immediately behind the vertical sandbag berm). Realistically, except in a test environment, it will be impossible to tell (with any certainty) where ground zero will be. Thus, real berms will have to be encircling -- which only the trench simulated. Even so, it is clear from the MILL RACE observations that lightweight pieces of equipment behind berms or in trenches should be anchored -- or they, themselves, will become damaging agents. Questions remain: such as whether drag alone would damage such lightweight equipment (which could be determined, experimentally, by anchoring them in a free-field environment that was missile free), and whether there is any density to drag ratio above which anchoring would be unnecessary.

The expedient anchors that were tested at MILL RACE showed great promise, insofar as adequacy to resist the design peak overpressure was concerned, but MILL RACE did not severely test them against the long durations characteristic of megaton weapons, nor is it yet known how well they would function in other soil materials.

Observations indicated that, as an alternative to anchoring, "stabilization" provides an effective means to reduce the probability of damage. The indication is that this works very well on drums strapped in seven barrel arrays; but the extent of effective application of the stabilization technique to other items of interest cannot be extrapolated from drum tests. Stabilization thus appears adequate as a method to deal with large quantities of hazardous materials, particularly those already in drums (or that can be transferred into drums from vulnerable tanks), provided the drums are full or can be consolidated and topped off quickly. Moreover, it is apparent that water-filled (or better yet, soil-filled) drums would make satisfactory barriers that are likely to be as good as some berms. Extrapolation from the drum tests indicates a seven-barrel array may be expected to slide about 2/3rds an array diameter on soil, when at the 20 psi overpressure level and subjected to a megaton weapon. The six thousand pound test seatbelt webbing provides a suitable strapping material, but it would be of interest to know whether other materials would perform suitably, as webbing might be in short supply when needed.

Another concept that indicated sufficient promise to warrant further testing was the immersion in liquid of pressure sensitive (in this case, electronic) equipment. A fluid that wetted and filled the interior compartments without the necessity to disassemble the instrument first was used. The fluid used (isopropyl alcohol) was one that would not cause deterioration through oxidation even in a week's time. Its beneficial effect was to make it impossible for a suddenly applied air overpressure to crush the plastic container (and smash and disrupt the delicate circuits). With this hardening scheme, pressures inside and outside the instruments rapidly achieved equilibrium with virtually no requirement for a significant volume change, unlike what happens when the volume is air filled.

In summary, the industrial hardening experiments conducted at MILL RACE provided valuable information that has immediate application, and these tests indicated promising paths that future exploration might take to provide industry with the tools to protect itself from nuclear attack.

Section 8
DNA NO. 5301
KEY WORKER EXPEDIENT SHELTER

INTRODUCTION

This experiment was a test of an expedient shelter constructed of dimensional lumber, buried with 30 in. of earth cover and subjected to a 40 psi environment.

OBJECTIVE

This shelter is of a basic design developed by Oak Ridge National Laboratory. A version of this shelter, constructed of wood poles and designed for 50 psi, was previously field tested, and the design of this pole shelter is described in the "Expedient Shelter Handbook", ORNL-4941 (Ref. 8). This experiment, however, was the first field test of the dimensional lumber version. Under a separate engineering support task, FEMA requested Scientific Service, Inc., to analyze this shelter and to re-design it to withstand a 40 psi environment.

DESIGN

A structural analysis of the previously designed ORNL dimensional lumber shelter indicated that the walls of the shelter were quite weak and probably could not withstand the backfilling process, and thus, would be subject to collapse at very low overpressure ranges, much less than 40 psi. Accordingly, the shelter was re-designed to withstand a 40 psi environment when buried and covered with 30 in. of earth. The re-design took into account a considerable amount of previous research in the areas of soil arching and in-plane loads generated by lateral earth pressure under blast loading. Of particular assistance in formulating this design was the work done at the U. S. Army Engineer Waterways Experiment Station by Kiger and Balsara, and presented in a paper entitled "Response of Shallow-Buried Structures to

Blast Loads" (Ref. 13). Using these available data, it was determined that the design ultimate capacity could be reduced by a factor of approximately 2.4, and the shelter would still maintain structural integrity when subjected to an overpressure of 40 psi; i.e., the maximum flexural capacity required for design purposes was based on an actual loading to the structure of approximately 17 psi. Using these reduced loadings, the shelter structure was designed with conventional design methods for timber construction. The overall dimensions and configuration of the original ORNL design were maintained, and standard dimensional lumber was used throughout.

CONSTRUCTION

The shelter structure was completely prefabricated at a location away from the test site, and transported to the site for assembly in sections. The overall outside dimensions of main shelter building were approximately 11 ft 5 in. long, 8 ft high, and 7 ft wide. A 5 ft 8 in. long by 3 ft 11 in. wide entryway with a vertical access hatch was located at one end of the main shelter. The basic construction consisted of 2 in. by 10 in. lumber, 6 in. on center in the walls, and 2-in.-by-12-in. lumber at 6 in. on center in the roof. Cross bracing of 4x4's at 12 in. on center was used in both the roof and the floor. The vertical access hatch was constructed of 2x10's nailed together to form solid 10-in. thick walls, and horizontally placed solid 4x4's. The hatch cover was made of solid 4x4's with 2x4 strongbacks. The walls and the roof were sheathed with 1-1/8-in. thick plywood. Figures 8-1 and 8-2 are drawings of the side and end elevations of the shelter. Figure 8-3 is the roof framing plan, and Figures 8-4 and 8-5 are longitudinal and cross sections through the shelter. The entire structure was set into an excavation of -6.00 ft. Figure 8-6 shows the excavation prior to erecting the shelter. Figures 8-7, 8-8, and 8-9 show the shelter in various stages of assembly. Figure 8-10 shows the completed shelter, and Figures 8-11 and 8-12 show the interior ceiling and floor prior to backfilling. When the assembly was completed, the entire structure was backfilled and covered over with 30 in. of native earth. The access shaft terminated at the earth cover level; when the hatch was in place, it was covered with sandbags. Figure 8-13 shows the shelter after backfilling and covered over with earth, and shows the sandbags on top of and around the hatch just prior to test.

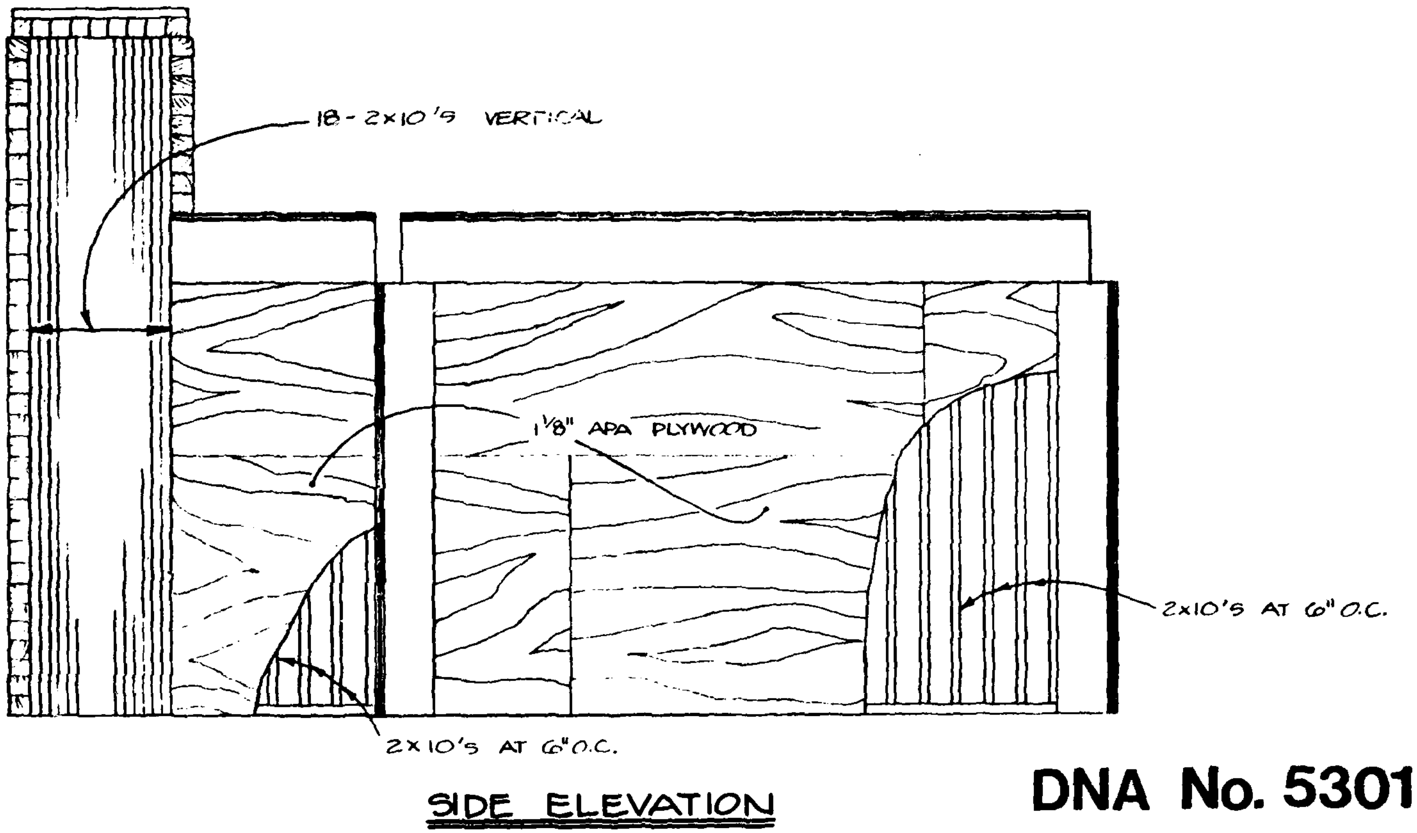
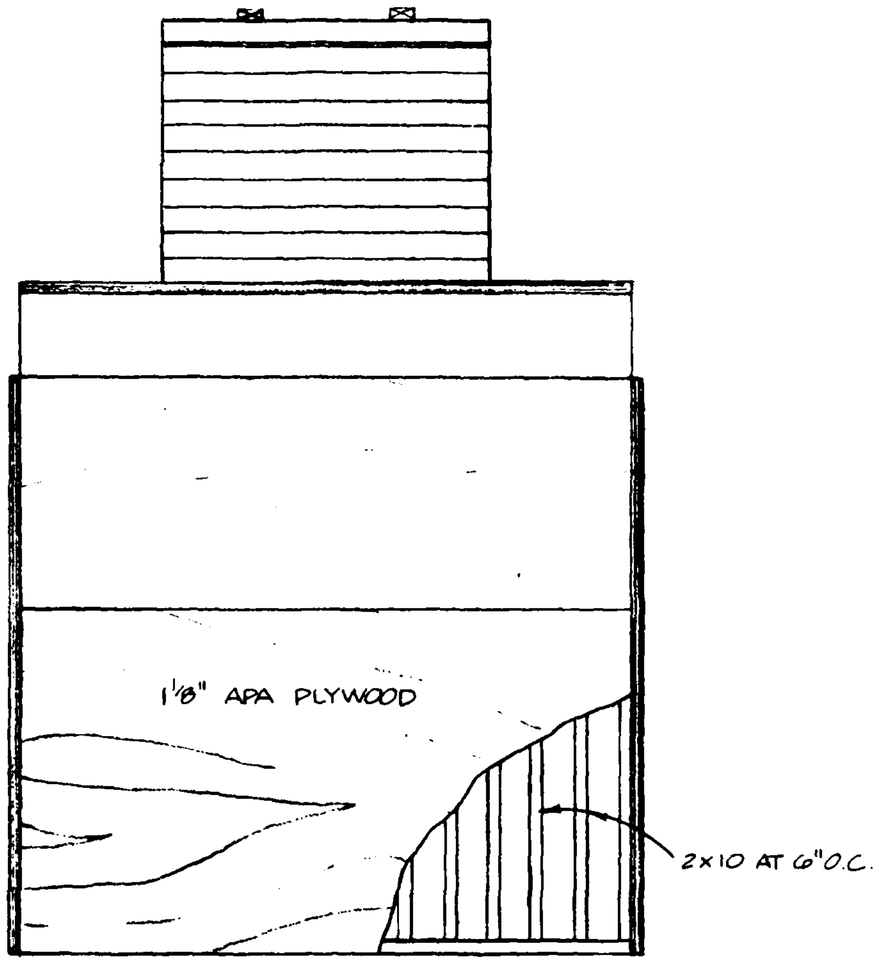


Fig. 8-1. Side Elevation, DNA No. 5301.

DNA No. 5301



END ELEVATION

Fig. 8-2. End Elevation, DNA No. 5301.

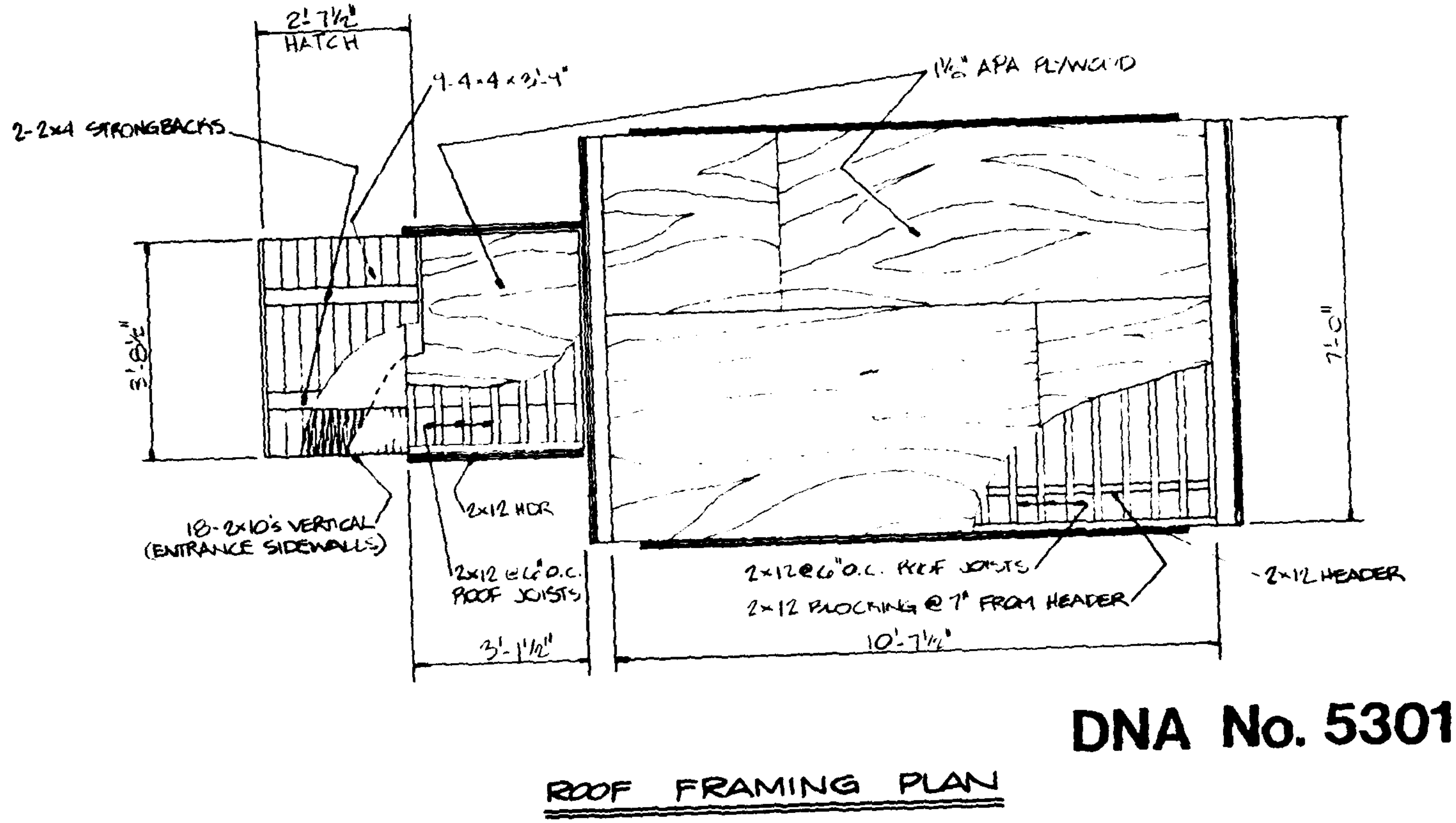


Fig. 8-3. Roof Framing Plan, DNA No. 5301.

DNA No. 5301

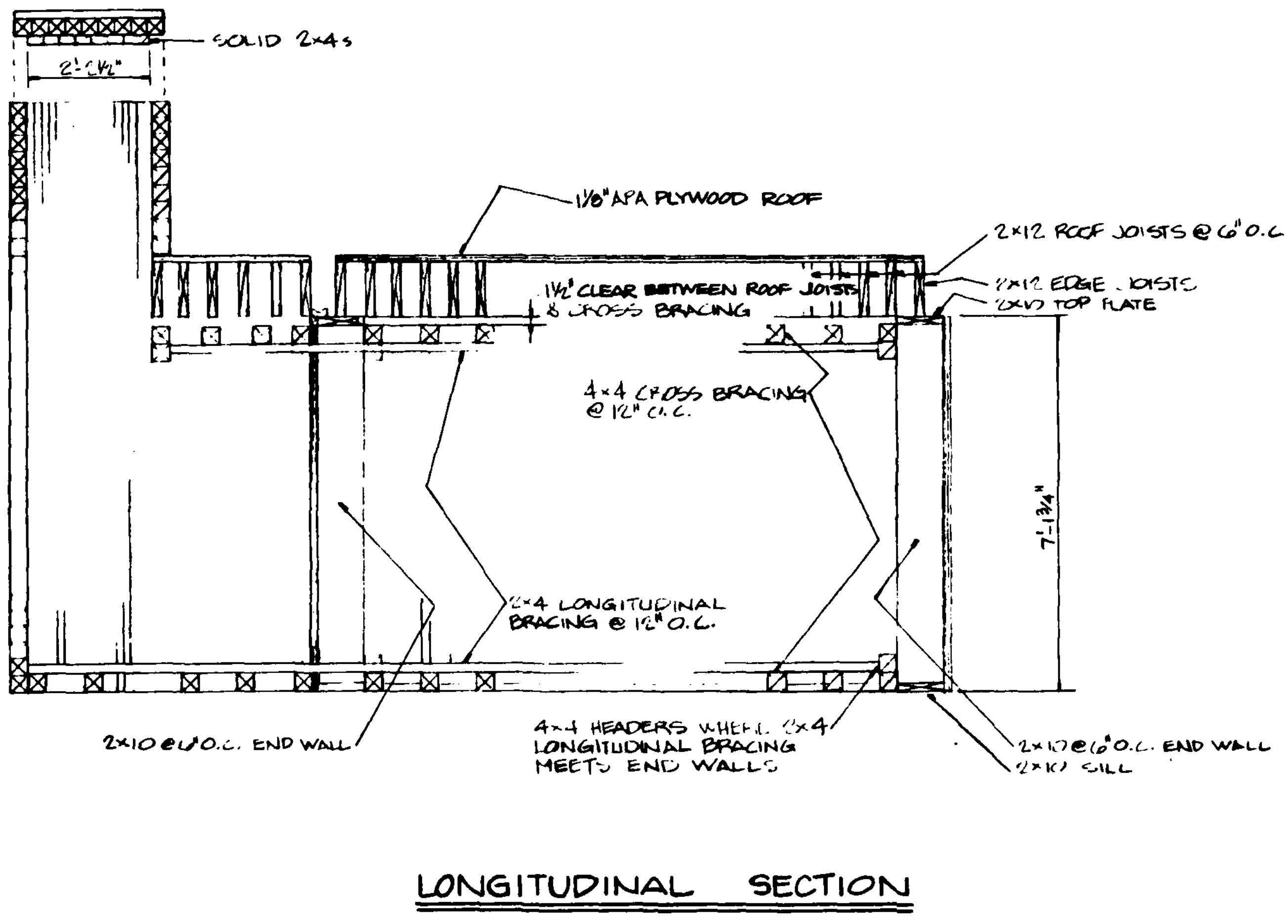


Fig. 8-4. Longitudinal Section Through Shelter.

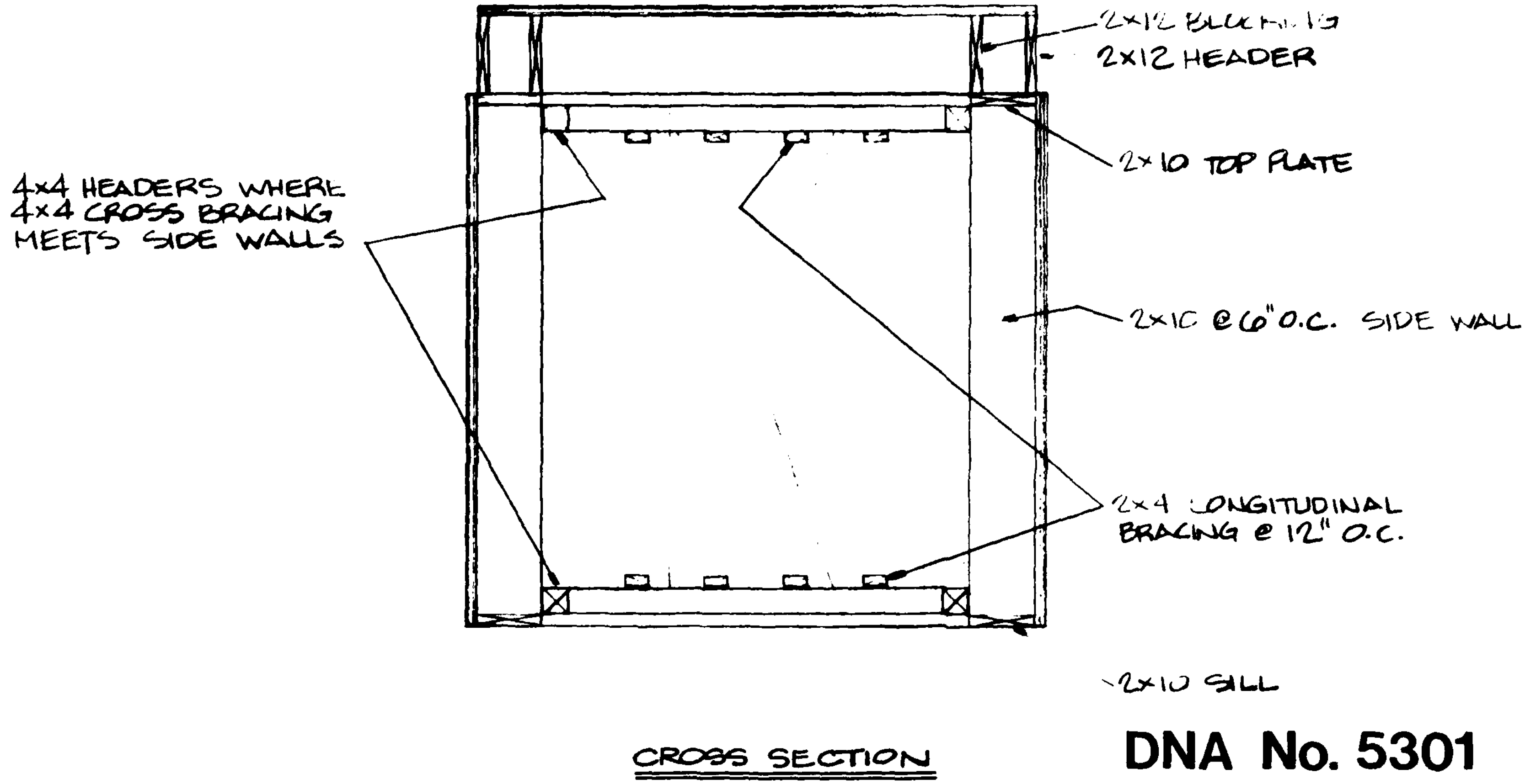


Fig. 8-5. Cross Section Through Shelter.

Fig. 8-6. Excavation for Key Worker Shelter.

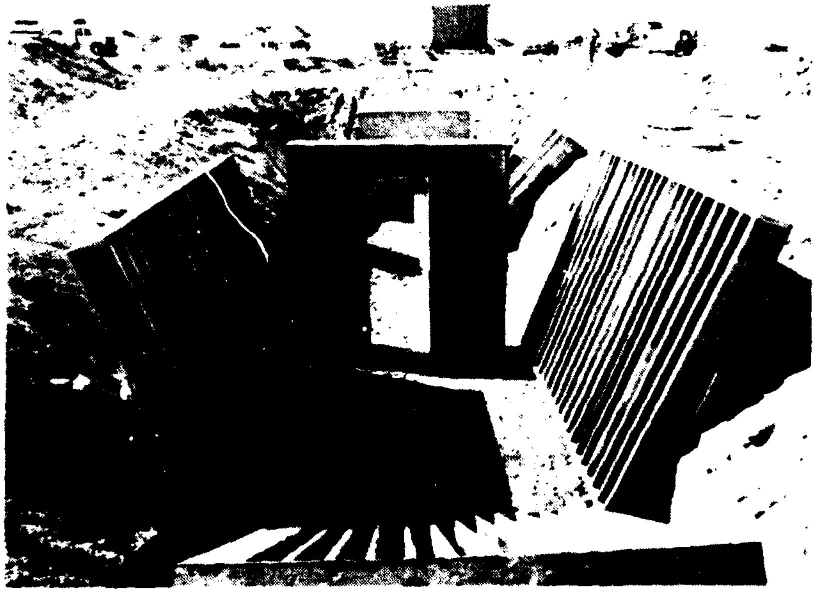


Fig. 8-7. First Stage of Assembly of Shelter.

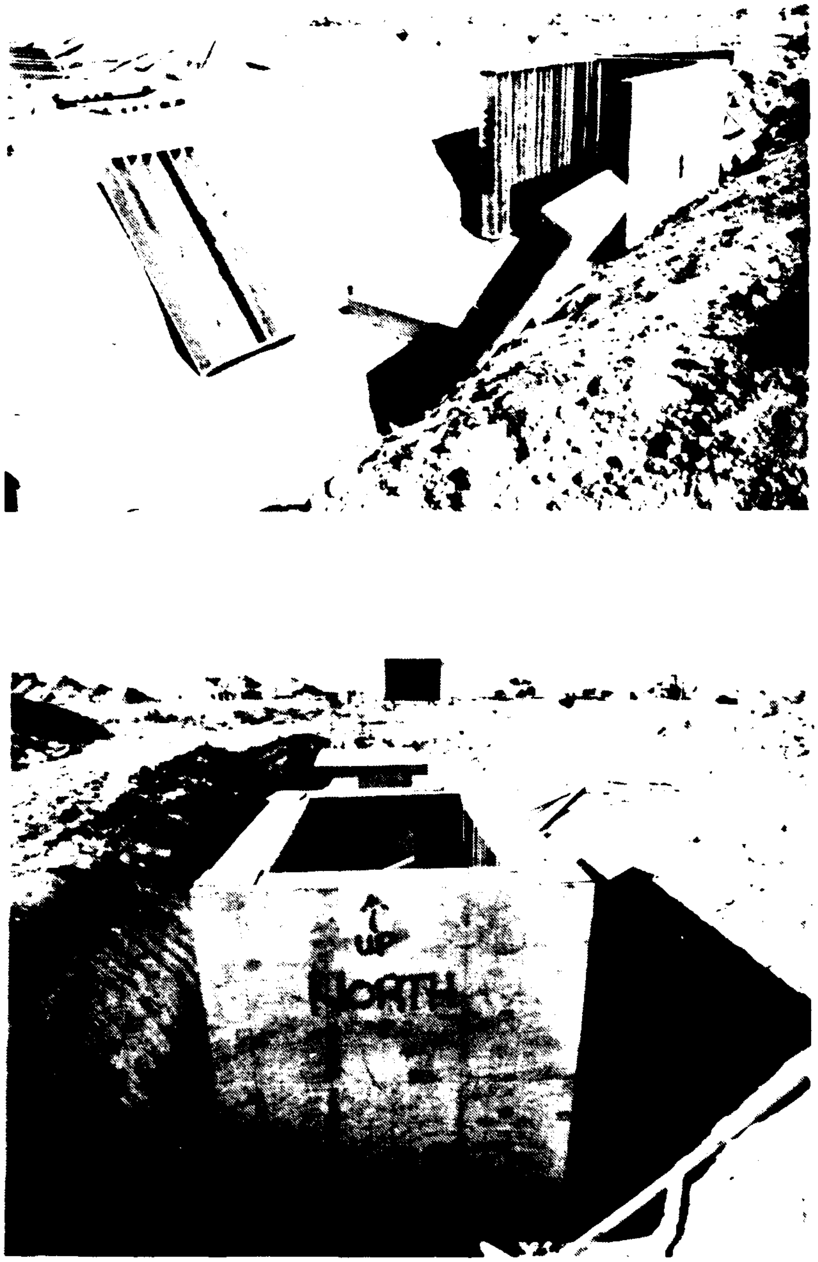


Fig. 8-8. Continuing Assembly of Shelter.

Fig. 8-9. Final Stages of Assembly.

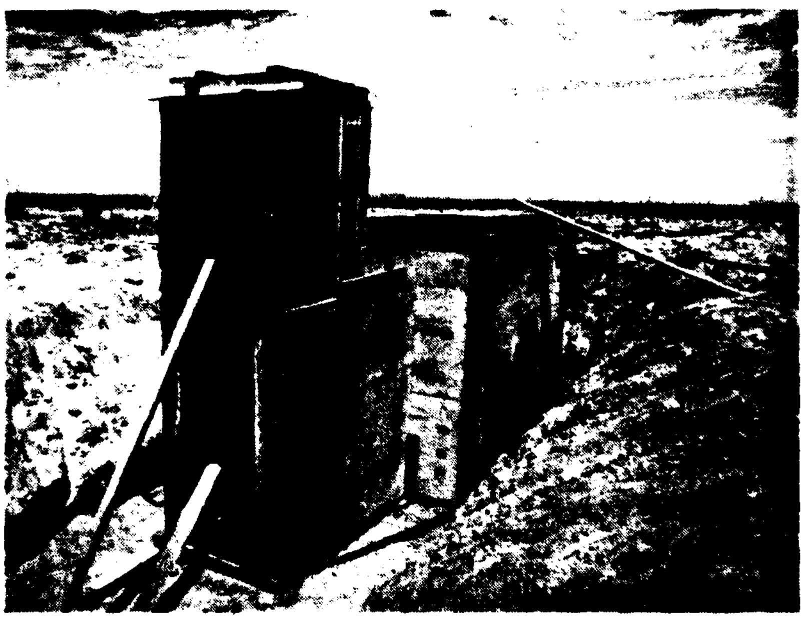


Fig. 8-10. Completed Shelter Prior to Backfilling.

Fig. 8-11. Interior Ceiling.

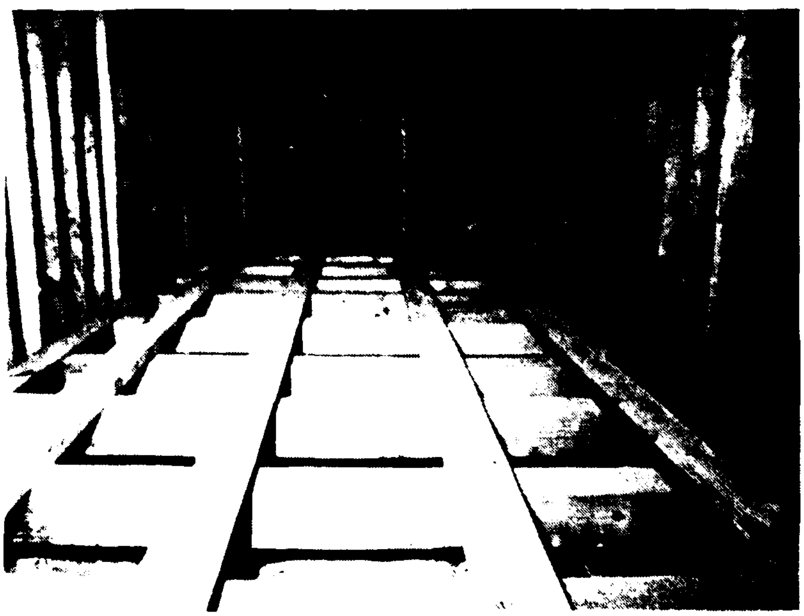


Fig. 8-12. Interior Floor.

Fig. 8-13. View of Shelter Just Prior to Test, With Sandbags Over Hatch.

INSTRUMENTATION

The shelter contained one active pressure gauge and amplifier of the same type and model used on the other experiments and as previously described. One anthropomorphic dummy (30) with a self-recording accelerometer was placed in a seated position in the center of the shelter. Still photography was also employed.

TEST DATA

The peak free-field overpressure at this location, as recorded and furnished by DNA, was approximately 39 psi.

An inspection of the site after test indicated that the hatch cover and the sandbags had been completely blown off. The hatch cover came to rest approximately 50 ft from the access shaft, and several of the sandbags were found 20 to 30 ft from their original location. Figure 8-14 shows the hatch and the hatch cover after test, and Figure 8-15 is a view of the hatch from the opposite side. A comparison of Figure 8-13 (before test) with Figures 8-14 and 8-15 (after test) show the depth of earth, approximately 2 ft, that was removed by the blast from all four sides of the hatch. Figure 8-16 shows a closeup of the top of the access hatch after test. The timber boards lying partially across the hatch opening in this figure are a portion of the hatch cover.

The pressure gauge, which was located on the interior wall approximately 5 ft above the floor, recorded a negative pressure phase of about 3.5 psi in 325 ms.

The inside of the shelter did not indicate any structural distress, but a significant amount of dust and earth was present. Figure 8-17 shows a view of the inside of the shelter, and shows the anthropomorphic dummy (30) in its original position. Figure 8-18 is a closeup of the dummy, and shows the amount of dust and earth present in the interior of the shelter. A posttest examination of the dummy indicated that it was in the same position as pretest, and the chair had not moved. It sustained no damage, and no reading was recorded on the accelerometer.

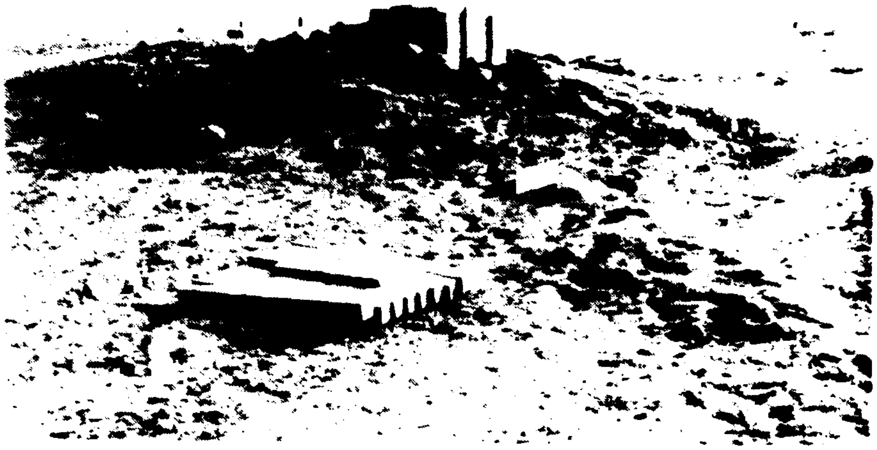


Fig. 8-14. Hatch and Hatch Cover After Test.

Fig. 8-15. View of Hatch From Opposite Side, After Test.

Fig. 8-16. Closeup of Top of Access Hatch After Test.

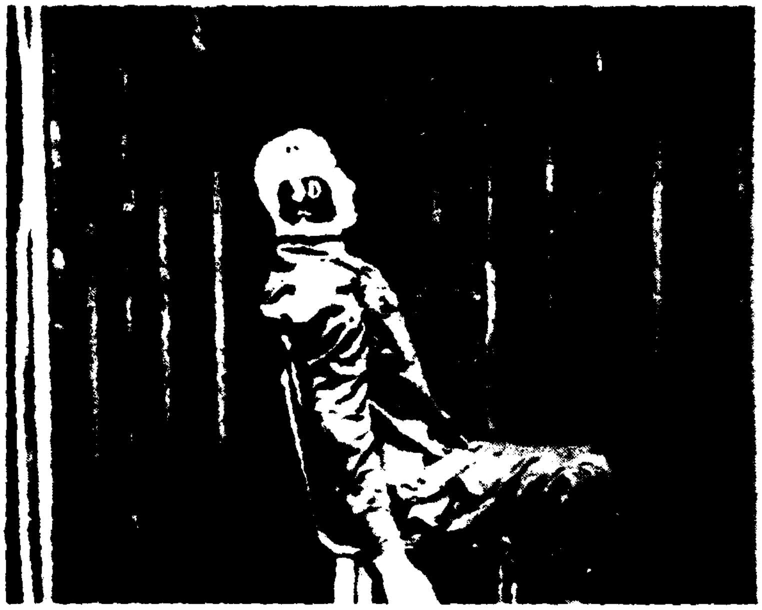


Fig. 8-17. Interior of Shelter After Test.

Fig. 8-18. Closeup of Dummy No. 30 After Test.

DISCUSSION AND CONCLUSIONS

This expedient shelter performed well in the 40 psi range and would likely perform adequately at 40 psi overpressures with increased weapon size. As mentioned in the Design portion of this section, the design analysis took into account a considerable amount of previous reseach in areas such as soil arching and timber failure methodology, and the shelter was actually designed for approximately 17 psi static loading. Even with these design considerations, the resulting shelter did not appear to be expedient from the standpoint of labor or resources.

The construction required 60 man-hours to complete, not including excavation and erection, using power table saws and power hand saws. The shelter contained nearly 3,000 board feet of dimension lumber, and provided 50 sq ft of floor area available for shelter space. This amounts to approximately 60 board feet of lumber per sq ft of floor area, or the amount of lumber used in constructing five to six 1,200-sq-ft single family homes.

When fully constructed, the shelter weighed 6,500 lb, which required building it in place in the excavation, or building it in small enough sections that it could constructed elsewhere, moved to the site, and erected without utilizing heavy equipment. The latter method was used in this constructing this experiment.

The structural performance of the shelter suggested a probable overdesign. As in several of the other experiments, lack of hard data on soil pressures resulting from dynamic or blast loading was the possible cause, as the structure apparently did not "feel" the loading anticipated in the design. In order that this type of shelter be more labor and material effective (more "expedient") it is recommended that this area of dynamic loading of soils be considered for additional research and testing.

Section 9
PROGRAM SUMMARY

This section of the report presents a summary of the results of FEMA-sponsored SSI experiments conducted at the MILL RACE high explosive test on September 16, 1981 at White Sands Missile Range, New Mexico. This test was conducted by DNA and consisted of the detonation of 600 tons of ANFO, which provided a simulated nuclear weapon airblast and ground motion environment for these and other varied experiments. The interpretation of the results of the experiments recognized the limitations of using data from an equivalent one kiloton nuclear blast to reach viable conclusions with respect to the performances resulting from megaton range weapons.

Scientific Service, Inc., under the sponsorship of FEMA, designed and conducted experiments at MILL RACE in the areas of upgrading existing structures for use as both host area and key worker shelters, industrial hardening, and expedient shelter development. The following is a brief description of the results and a summary of the conclusions for these experiments.

HOST AREA SHELTERS

The two 24-ft-by-16-ft buildings on grade at 2 psi, one wood framed and the other masonry, performed well and indicated no structural distress. The roofs of both buildings were shored with stud wall shoring and the walls were braced. Both buildings were bermed on all four sides and covered with earth. Although the upgrading performed adequately, the labor and materials required for construction and installation of the shoring and bracing appeared to be excessive.

The 24-ft-by-16-ft basement structure at 2 psi performed as expected. One half of the building's floor above was shored with post and beam shoring and the other half remained unshored. The concrete masonry walls were not braced. The shored portion of the building indicated little structural distress — only two cracked joists — and performed well. The unshored portion collapsed completely into the basement area under the blast loading. It is of interest to note that failure of the timber joists in the unshored area occurred prior to the test and was caused by the earth loading. The post and beam shoring was easily and quickly constructed and installed, and appeared to be practical and expedient.

KEY WORKER SHELTER

The reinforced concrete flat slab and two-way slab floors in the 151-ft long basement structure at 40 psi performed well, as expected. These floors were shored with both timber telephone poles and structural steel tubing. They showed evidence of yield line cracking, but did not deflect significantly nor exhibit any other damage. Neither the floor slab above nor the slab on grade showed any evidence of distress as a result of punching shear from the shores.

Of the four bays with the precast prestressed concrete plank floors, three were shored with various shoring configurations and one remained unshored. The shoring system in all cases was post and beams; one bay had three lines of shoring, one bay had two lines, and one was shored with only one line. The unshored bay and those supported with one and two lines of shores were tested primarily to investigate debris translation and to verify modes of failure for these types of systems, and as expected, these three bays collapsed to varying degrees. It was expected that the bay shored with three lines of shores would survive 40 psi; however, this bay also sustained partial collapse near the stairwell. It was concluded that this collapse was substantially caused by the failure of the stairwell closure combined with the inadequate shoring of the planks framing the stairwell. In general, it was concluded that because of the disastrous types of collapse mechanisms that may occur, structures constructed of this or similar material, and under consideration as shelter candidates, should probably include additional structural considerations in their evaluation, such as structural concrete topping and end connections.

This structure also contained six basement test walls, two unreinforced masonry, two reinforced masonry, and two precast concrete. The masonry walls did not sustain any damage and no cracks were found; the two precast walls cracked horizontally at midheight. Analytically, the unreinforced masonry walls would be of structural concern from the backfilling operation, let alone the lateral loading from the soil as a result of a 40 psi overpressure. It is obvious that these walls did not receive any significant lateral loading, and that soil/structure interaction requires further investigation.

INDUSTRIAL HARDENING

The industrial hardening experiments investigated two areas: the use of buried concrete utility vaults as expedient shelters, and the protection of industrial equipment and machinery, secured and hardened by various methods. The two concrete utility vaults, one located at 20 psi and the other at 40 psi, sustained no damage. The posttest interior environment of both vaults indicated no motion of the anthropomorphic dummies, or relative motion between the vaults and the access pipes. It was concluded that the upgraded vault at 40 psi would likely be adequate for 40 psi whatever the weapon size, and conservatively, would survive essentially undamaged at 20 psi from a multimegaton weapon. The vault at 20 psi, as tested without upgrading, would be expected to survive at 10 psi from a multimegaton weapon in the same undamaged condition. The closures that covered the ends of the access pipes were expedient and not what would be required for routine access and egress for shelter inhabitants. Additionally, even if access and egress are not prime considerations, design modifications would be required to maintain closure integrity for megaton size weapons. To more clearly define ratings on vaults of this type, additional tests, some to failure, of vaults buried in other soil materials would be required.

The other area of investigation associated with industrial hardening, the stabilization, securing, and protection of lightweight machine tools and steel drums, provided valuable information that has immediate application. These experiments were all conducted at 20 psi. The machine tools were selected to represent the more vulnerable types in terms of blast loading. Some were exposed to both static overpressure and dynamic pressure, while others were protected to reduce exposure to dynamic overpressure without protection against static overpressure. Various protection methods were used, including trenches, earth berms, sandbags, and lumber stacks secured with expedient soil anchors. The conclusions are that the damage to even such lightweight tools as these was primarily the result of impact, not pressure, and that elimination of the opportunity for these tools to overturn, slide, or tumble would eliminate most damage.

The investigation of the stabilization of steel drums, some of which were full, some empty, and some partially full, and arranged in varying arrays of one, three, and seven, provided significant data towards an alternative to anchoring. In particular, the seven-drum array, with the drums full and strapped together, performed well and indicated an effective means of reducing the probability of damage. The test results indicated that stabilization is an adequate method of dealing with large quantities of hazardous materials, particularly if the material is already in drums. It was also apparent that filled drums would make satisfactory protective barriers that would likely be as good as some berms. The extent of effective application of this stabilization technique to other items of interest, however, can not be extrapolated from these tests, but will require further investigation and testing.

Another concept preliminarily investigated was that of the protection of pressure-sensitive equipment by immersion in a non-deteriorating fluid (isopropyl alcohol was used) so that suddenly applied overpressures would not crush or disrupt the delicate circuits. The data developed as a result of this investigation indicated sufficient promise to suggest further testing.

The industrial hardening experiments conducted at MILL RACE provided valuable information that has immediate application, and indicated areas requiring additional research, for the protection of industry from nuclear attack.

KEY WORKER EXPEDIENT SHELTER

The dimension lumber version of the expedient pole shelter performed well at 40 psi, and would likely perform adequately at 40 psi with increased weapon size.

No structural distress was observed, and the anthropomorphic dummy did not indicate damage or movement. The labor and resources required to construct this shelter, however, did not appear to be expedient. A considerable amount of further research on these types of shelters is required and recommended, both in the area of dynamic loading of soils — thus leading to a more accurate and sophisticated design methodology -- and in the practical expenditures of resources.

SUMMATION

In general, all of the experiments conducted by SSI at MILL RACE provided excellent data. Areas that need additional investigation and/or require a more definitive or an expanded test program in the future, were clearly pointed out. The upgrading of both host and key worker shelters verified the data presented in the Shelter Upgrading Manuals (Refs. 2 and 3) for floors and roofs, but also underscored the requirement for further investigation of walls, closures, resource requirements (labor and materials), structural connections, and soil/structure interaction. The results of the experiments on expedient shelters showed, however limited, the favorable performance of structures or structural components (concrete utility vaults) in common use and, to some degree, available "off the shelf". These results also showed the resources required for a particular type of key worker expedient shelter (dimension lumber), and indicated that investigation and testing in this area are needed. A substantial amount of valuable information was acquired from the industrial hardening experiments, much of it having immediate application, but all of it indicating direction for promising future exploration.

As mentioned in the INTRODUCTION section of this report, the objectives of these experiments were to demostrate the validity and practicality of a number of shelter upgrading and industrial hardening concepts in support of crisis relocation planning. To this end, it is our conclusion that the experiments sponsored by FEMA and conducted by Scientific Service, Inc., in conjunction with the MILL RACE event, were eminently successful.

Section 10

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TESTING OF SHELTER DESIGN AND INDUSTRIAL HARDENING CONCEPTS AT THE MILL RACE EVENT

Unclassified

January 1982

Scientific Service, Inc., Redwood City, CA
Contract EMW-C-0611, Work Unit 1128D

216 pages

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