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Equipment Response at the El Centro Steam Plant During the October 15, 1979 Imperial Valley Earthquake

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ABSTRACT

For the U.S. Nuclear Regulatory Commission (NRC), Lawrence Livermore National Laboratory (LLNL) performed a dynamic seismic analysis of Unit 4 of the El Centro Steam Plant in El Centro, Calif. Built in 1968, Unit 4 is an oil- or gas-fired, steam-driven turbine-generator that was designed to resist a static lateral force equivalent to 20% of the dead and live load. The unit's structural and mechanical systems sustained only minor damage during the October 15, 1979 Imperial Valley earthquake that produced an estimated 0.5 g peak horizontal ground acceleration (0.66 g vertical) at the site. LLNL's seismic analysis was done to analytically estimate the equipment response, which, when compared to actual observation, will indicate the levels of actual equipment capacity.

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PREFACE

Following the October 15, 1979 Imperial Valley earthquake, a reconnaissance team of U.S. Nuclear Regulatory Commission (NRC) staff members went to California's Imperial Valley to survey damage and collect seismological data.[1] During this visit, which took place from October 17 to 19, 1979, team members inspected the El Centro Steam Plant because the facility is similar in both design and types of equipment to older operating nuclear power plants. The NRC team observed only minor damage to the plant's structural and mechanical systems despite the estimated 0.5 g peak horizontal ground acceleration (0.66 g vertical) produced at the site.

This study resulted from the NRC reconnaissance team's visit. It is part of a continuing NRC effort to incorporate the lessons learned from operating experience during earthquakes into the licensing process. The NRC anticipates significant benefits to be derived from this study for both current licensing activities and for the ongoing Systematic Evaluation Program--a review of older operating nuclear facilities. This effort--along with other activities such as the NRC-sponsored Seismic Safety Margins Research Program being conducted at the Lawrence Livermore National Laboratory (LLNL)--are aimed at developing an improved understanding of the design and analysis of nuclear power facilities.

The NRC engaged LLNL to analyze Unit 4 of the steam plant and estimate equipment response. Work was done under Technical Assistance Contracts, NRC FIN Nos. A-0225 and A-0242. Technical contacts were J. A. Martore of the Office of Nuclear Reactor Regulation (NRR), Division of Licensing, and H. A. Levin of the NRR Division of Engineering.

This report reflects a collective effort on the part of the following persons:

- T. A. Nelson (LLNL, Task Leader) provided overall leadership for the analysis and report compilation.
- R. C. Murray (LLNL, Technical Assistance Project Manager) provided program management support and report review.
- J. A. Young (EG&G, Incorporated, San Ramon Operations) formulated the structural model of Unit 4, conducted the seismic analysis, and estimated equipment response.
- R. D. Campbell (Structural Mechanics Associates) estimated equipment frequencies and dampings and provided equipment photographs.

- J. A. Martore (NRC reconnaissance team member) was instrumental in establishing the study and providing input to the damage assessment and discussion sections of this report.
- H. A. Levin (NRC reconnaissance team member) was instrumental in establishing the study and providing input to the damage assessment and discussion sections of this report.
- L. Reiter (NRR Division of Engineering) provided input to the geology and seismology section and the strong motion data section.

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EXECUTIVE SUMMARY

On October 15, 1979, an earthquake shook the Imperial Valley of California. The earthquake had a Richter magnitude of 6.6; its epicenter was on the Imperial Fault, approximately 16 km east of Calexico. When the earthquake occurred, Units 3 and 4 of the four-unit nonnuclear El Centro Steam Plant in El Centro, Calif., were operating. The operating units tripped off line when station power was lost. Unit 3 was restored to service within 15 min after the main shock. Unit 4 was restored to service within 2 hr.

Two days after the earthquake, an earthquake reconnaissance team of U.S. Nuclear Regulatory Commission staff members surveyed the damage and collected seismological data.[1] Team members inspected the El Centro Steam Plant because the facility is similar in both design and types of equipment to older operating nuclear power plants. The NRC team observed only minor damage to the plant's structural and mechanical systems despite an estimated 0.5 g peak horizontal ground acceleration (0.66 g vertical) for the site. The original design criteria specified a static lateral load equivalent to 20% of the dead as well as the live load.

The NRC engaged LLNL to analyze Unit 4 of the steam plant and estimate equipment response. Laboratory representatives visited the site in March 1980 to become familiar with the plant layout and to gather information for modeling and analyzing the structure and equipment. A site visit was made in August 1980 to photograph equipment and gather additional information necessary for estimating equipment frequencies.

The seismic response of the turbine building and its associated structures was determined using a normal mode time history method of analysis for a linear elastic model composed of equivalent vertical beam members with masses lumped at floor levels. To account for soil-structure interaction, the soil was treated as an elastic half-space, and soil springs and damping were developed. The time history input was based on records of the earthquake for U.S. Geological Survey Station No. 5165. A ground motion duration of 20 s with a 0.01 s time step was used for this analysis.

Maximum accelerations and response for the main equipment floors were calculated. The maximum predicted base shear values were computed and found to agree closely with design base shears determined from design data.

To estimate equipment response, in-structure response spectra nodes were selected and considered to be rigidly attached to the center of rigidity at each elevation in the model. Acceleration time histories and response spectra were generated at each node. Then, for each direction, a fundamental frequency and damping ratio were estimated for each equipment item. The frequencies were either calculated, estimated by engineering judgment, or estimated by comparison to similar equipment for which frequencies were determined by analysis or test. Damping values were estimated for elastic response. Equipment acceleration responses were determined from the various in-structure response spectra. In general, the equipment acceleration responses varied from 0.5 to 1.8 g in the N-S direction, 0.4 to 1.2 g in the E-W direction, and 0.4 to 1.55 g in the vertical direction.

We concluded that the turbine building dynamic model with highly damped soil springs reasonably reflects the forces induced in the building during the earthquake. This conclusion is evidenced by:

- The low level of damage observed at the plant.
- The close relationship of the design to predicted base shears.
- The low displacement at the turbine pedestal.

In drawing conclusions about structural response, it is difficult to relate the static loading design criterion (a static lateral load equivalent to 20% of the dead as well as the live load) to an earthquake loading characterized by a particular peak ground acceleration. However, for this particular earthquake and structure the triaxial time history analysis gives base shear values in close agreement with the original design values.

The forces experienced by the plant equipment were on the order of 2 to 9 times greater than the 0.2 g specified design load. This would seem to imply a reserve seismic equipment capacity of about 200%. However, it is difficult to verify this value without knowing the actual lateral load the equipment could withstand. One can only conclude that nuclear power plant equipment similar to that in Unit 4 and anchored as well should perform equally well during a similar earthquake.

We recommend that:

- Additional work be done to estimate the ultimate capacity of the equipment in Unit 4 to quantify equipment reserve capacities.

- An analysis of stresses in the structure based on this study be conducted.
- The possible discrepancy between effective and peak instrumental acceleration be investigated further, perhaps using data from the differential array.
- Additional soil-structure interaction studies be conducted.
- Seismic recording instruments be installed in the El Centro Steam Plant.
- Unit 4 be analyzed to current NRC criteria to indicate the levels of conservatism inherent in these design-related procedures.

INTRODUCTION

On October 15, 1979, at 4:16:55 p.m. (PDT) an earthquake shook the Imperial Valley of California. The earthquake had a Richter magnitude M_L of 6.6. The epicenter was on the Imperial Fault, 16 km east of Calexico (Fig. 1). Three aftershocks that exceeded $M_L = 5$ occurred about 6.5, 7. and 7.5 hr after the main shock. There was no loss of life in this sparsely populated agricultural area, but there was some property damage and extensive damage to irrigation canals and subsurface drain tiles.[1]

When the earthquake occurred, Units 3 and 4 of the four-unit nonnuclear El Centro Steam Plant in El Centro, Calif., were operating, and Units 1 and 2 were shut down for maintenance. The operating units tripped off line when station power was lost because of a short circuit resulting from a broken insulator in a lightning rod in Unit 1. Unit 3 was restored to service within 15 min after the main shock. Unit 4 was restored to service within 2 hr. Much of the time was spent by plant personnel inspecting for damage.

There was some minor damage to the facility; however, the two operating units safely shut down with no known malfunctions of electrical control and instrumentation equipment during or after the earthquake. The most notable damage occurred at Units 3 and 4, which developed leaks in the cooling water piping for the hydrogen cooler and the exciter cooler. The leaks occurred in locations weakened by corrosion. The piping is made of 3- and 4-in. diameter welded carbon steel pipes with some threaded and unthreaded couplings. By expedient plugging of leaks, Unit 3 was kept in service. Because the load was abnormally low for several hours, Unit 4 was removed from service at 9:40 p.m. that evening to make repairs. More detailed information about other damage that did not impair normal operation can be found in the next section under the heading "Damage to the Plant."

SCOPE OF THE STUDY

Lawrence Livermore National Laboratory (LLNL) performed a dynamic seismic analysis of Unit 4 of the El Centro Steam Plant to analytically estimate structural and equipment response. Built in 1968, Unit 4 is an oil- or gas-fired, steam-driven turbine-generator that was designed to resist a static lateral force equivalent to 20% of the dead and live load. The unit's structural and mechanical systems sustained only minor damage during the October 15, 1979 Imperial Valley earthquake, which produced an estimated 0.5 g peak horizontal ground acceleration (0.66 g vertical) at the site.

Conservative assumptions usually made in the analysis of nuclear power plants were eliminated whenever possible in estimating equipment response. Estimated response indicates that the structure and equipment experienced accelerations higher than those originally specified in the design. This provides an estimate of the inherent reserve capacity that may be inferred for these types of equipment.

The LLNL analysis consisted of the following steps:

- Review and summarize available design and construction information such as structural geometry, equipment locations and characteristics, and design criteria.

- Visit the site to inspect the structures and damage caused by the October 15, 1979 Imperial Valley earthquake.

- Develop a lumped-mass model of Unit 4, including the mass effects of major equipment items and soil-structure interaction, and determine the fundamental mode shapes and modal frequencies.

- Perform a dynamic analysis of the model using the actual time histories of the October 15, 1979 Imperial Valley earthquake recorded near the plant.

- Estimate structural accelerations and displacements.

- Generate in-structure response spectra at locations of major equipment items.

- Estimate fundamental frequencies for selected equipment items.

- Estimate the accelerations that equipment experienced.

ORGANIZATION OF REPORT

This report documents LLNL's findings in providing the above technical assistance to the NRC. The next section discusses the October 15 earthquake, including the background geology and seismicity, the strong-motion monitoring equipment, and the damage to the plant. Then, a detailed facility description is presented. The following section discusses the model of Unit 4 and its use in the dynamic analysis of structures and the generation of in-structure response spectra for the subsequent evaluation of equipment. Analytical conclusions and recommendations are then presented, followed by a discussion written by the NRC technical contacts. Appendix A contains soil data for the site.

THE OCTOBER 15, 1979 EARTHQUAKE

At 4:16:55 p.m. (PDT) a moderate earthquake shook the Imperial Valley of California. It had a local magnitude of 6.6, a surface-wave magnitude (M_S) of 6.9, and a body-wave magnitude (m_b) of 5.7. The earthquake produced 30 km of predominately strike slip rupture on the Imperial Fault (Fig. 1). There were many aftershocks, three of which exceeded M_L = 5 within the first 8 hr. There was no loss of life, but there was property damage in Imperial County. The most severe damage occurred at the six-story Imperial County Services Building in El Centro. Lesser damage occurred at storage tanks, bridges, trailer parks, and buildings in the towns of El Centro and Brawley.[1] Minor damage at the El Centro Steam Plant, approximately 5 km from the fault, is described later in this section.

GEOLOGY AND SEISMOLOGY

The Imperial and related Brawley faults (Fig. 1) are part of the fault system that separates the Pacific and American Plates. Further north, this system becomes the San Andreas fault. To the south, it underlies the Gulf of California. Historically, the Imperial Valley has been the scene of repeated earthquakes of local magnitude between 6 and 7. In this century, such events occurred in 1906, 1915, and 1940.

In this, as in previous Imperial Valley earthquakes, the slip was right-lateral strike slip. Lateral displacements of up to 50 cm occurred at the surface during or within 24 hr of the main event. Some vertical displacements were observed along the northern half of the rupture, and a portion of the Brawley Fault east of the Imperial Valley Fault also broke.[2] Although the epicenter has been located several kilometers south of the international border in Mexico (Fig. 1), no surface rupture was observed there. The hypocentral depth is believed to have been 12 km, and apparently the rupture traveled several kilometers before it reached the surface.

The Imperial Valley itself is part of the Salton Trough and is filled with over 6 km sedimentary deposits overlain by alluvium (Fig. 2). It is an agricultural area, and much of the earthquake-induced damage was related to the intricate irrigation system used by the local farmers.

STRONG MOTION DATA

As a result of the 1940 and other smaller earthquakes, the Imperial Valley was extensively instrumented by a network of strong motion accelerographs. Most important was a 13-station linear array straddling the fault (Fig. 1). Included in this array is Station No. 9, the original instrument that recorded the classic 1940 accelerogram used by many engineers to describe strong ground motion. The peak horizontal acceleration (0.81 g uncorrected) was recorded at Bonds Corner, 3 km from the southern part of the fault, while the peak vertical acceleration (1.74 g uncorrected) was recorded at Station No. 6 between the Imperial and Brawley faults. The maximum duration greater than 0.1 g was 13.3 s, also measured at Bonds Corner.[3]

About two weeks before the earthquake, the U.S. Geological Survey finished installing a digital recording differential array in a large vacant lot near Unit 4 of the El Centro Steam Plant.[4] The purpose of this array was to detect differential ground motion over a relatively short distance caused by horizontally propagating seismic surface waves. The array is oriented south to north, and has six instruments placed at distances of 0, 60, 180, 420, 700, and 1000 ft along the array. The sensors are triaxial, downhole, force-balance accelerometers. They were placed in 5-in.-diam holes, 4-ft deep and tamped in with coarse sand, and connected by wiring laid in conduit to recorders in an air-conditioned building at the south end of the array.

Unfortunately, there were several malfunctions in the operation of the system. The recorder for the northern most instrument was out of magnetic tape because it had previously mistriggered. The time signal was not recorded; consequently, common time among the stations was lost. Fortunately, a standard Kinemetrics SMA-1 strong-motion accelerometer associated with the system was installed in the building at the south end of the array. This instrument, labeled No. 5165 El Centro Differential Array, functioned well and produced a timed record.

Seismic Input

The El Centro Steam Plant lies between Station No. 9 and the Differential Array (Fig. 1). Because of the age of Station No. 9 there has been some difficulty in obtaining a complete time history from its accelerograph. In addition, there has been some question as to the free-field nature of this station since it is situated on a large foundation.[5] However, Station 5165 at the south end of the differential array is less than one kilometer from the steam plant in a small building. A digitized corrected version of the time history from Station 5165 was used in the analysis presented in this report. lots of the ground motion and corresponding response spectra are shown in igs. 3, 4 and 5. Some question may be raised as to the adequacy of strong motion recorded 0.85 km from the site to be analyzed in an area where ground motion may vary rapidly. While it certainly would have been better to have a free-field record much closer to the plant, the adequacy of the record used can be supported by two observations:

- Except for differences in peak vertical acceleration, the differential array records appear very similar over the full 700-ft operating length of the array.[4] These differences (0.33 to 0.67 g) only affected an unusual high-frequency arrival early in the time history.

- A Wilmot Seismoscope with a period of 0.75 s situated at the bottom floor of the steam plant itself registered a maximum reponse displacement of 6.57 cm and a maximum response velocity of 55 cm/s.[6] These are approximately equivalent to the maximum responses at 0.75 s and 10% damping computed from the horizontal time histories at Station 5165. The seismoscope is believed to have a damping of 10%.

Similarities to 1940 Earthquake

Because the 1940 accelerogram is one of the earthquakes from which the NRC Regulatory Guide 1.60 response spectra were derived. .. 's useful to compare the 1940 and 1979 earthquakes. Although of lesser extent, the October 15, 1979 ground rupture followed the same t ace as that of the 1940 event. Both ruptures appeared to have maximum lateral displacement near the international border, and maximum vertical displacement near the Mesquite Depression east of Imperial. Aftershock activity shifted progressively to the north following both events, and each earthquake had damaging aftershocks near Brawley. Table 1 shows these and other comparative features for the two earthquakes.

Another useful comparison is made when R.G. 1.60 response spectra are plotted along with response spectra derived from the actual time histories. As shown in Figs. 3, 4, and 5, the R.G. 1.60 spectra are generally above the October 15 spectra at frequencies less than about 10 Hz for N-S, E-W, and vertical response.

TABLE 1. Comparison of 1940 and 1979 Imperial Valley earthquakes, after Ref. 7.

	May 18, 1940	October 15, 1979
Local magnitude (M_L)[a]	6.4±.2	6.6±.3
M_L at Pasadena	6.2±.1	6.7±.2
M_L at Tinemaha	6.4±.1	6.7±.1
Maximum accelerations at	0.22 g vertical	0.38 g vertical
El Centro Station No. 9	0.36 g horizontal	0.40 g horizontal
Duration of strong shaking (>0.1 g) at Station 9	≈20 s	≈7 s
Length of surface rupture	60 km	30 km
Rupture mode	Bilateral	Unilateral
Number of aftershock $M_L \geq 5.0$	4	3
Deaths	9	0
Estimated property loss[b]	\$5-6 million	\$30 million

[a]Best estimate based on amplitude of 0.1 to 1.0 s waves.

[b]Based on dollar value at time of the earthquake.

OBSERVATIONS AND IMPLICATIONS OF DAMAGE TO THE PLANT

An NRC earthquake reconnaissance team visited the site shortly after the earthquake struck and issued a report on its findings (Ref. 1). LLNL representatives visited the site on March 18, 1980 to become familiar with the plant layout and to gather information for modeling and analyzing the structure and equipment. At the request of LLNL, a representative of Structural Mechanics Associates (SMA) visited the site in August 1980 to gather additional equipment data.

NRC Site Visit

On October 18, 1979 the NRC reconnaissance team toured the steam plant facility to assess the degree of damage sustained as a result of the earthquake. The following observations are based on the report of that visit, which included inspection of the plant log book, firsthand observations, and discussions with plant personnel.

Structural

No significant structural damage was observed. Minor concrete cracking was generally apparent throughout the plant. More significant cracking, on the order of about 1 in., was observed at a junction of a floor diaphragm high in the structure and the turbine building shear wall. In addition, concrete cracks were observed at upper elevations between the various units, where larger deflections would be expected. However, in all cases this cracking was local in nature and overall structural integrity was maintained.

Structural steel, for the most part, was not permanently deformed as a result of the earthquake. Significant stressing was apparent on some members through observations of cracked paint on structural sections and the gouging of metal near connections. Slotted key-ways were provided in various locations to act as lateral seismic restraints.

One of the few areas where structural damage was observed was the Unit 4 boiler, which is hung in a pendulum fashion using rods supported by a braced frame. Lateral seismic restraints are mounted on the braced frame to minimize excessive motion of the freely supported boiler. Travel through the restraint gap was evidenced by paint chipping in the area and permanent deformation of the restraint (Fig. 6). In addition, three diagonal bracing members on the boiler frame buckled (Fig. 7), apparently due to excessive compressive loads. These diagonals were later replaced.

Other isolated structural damage which did not impair normal operating function was

Unit 4 Air Preheater Foundation - One of the concrete foundation support pads had partially crushed but was still functional, and had not been replaced (Fig. 8).

Stacks - The stacks on Units 1, 2, and 3 are bolted to concrete foundations with approximately 1.25-in.-diam bolts and 18 in. long. These bolts were stretched on all three units. They were retightened; however, none were replaced. The stacks are constructed of steel, and no anomalous observations were made.

Floor Joint - A steel plate which spanned the gap between Units 1 and 2 on the operating floor had buckled due to relative motion between buildings.

Chemical Lab - Small cracks were apparent in the masonry walls. A check of drawings indicated that the masonry walls were reinforced.

Piping

No high-temperature or high-pressure piping failed during the earthquake. However, a Victaulic coupling on a straight section of a 2-in.-diam cooling water line was damaged (Fig. 9). Additionally, 3- and 4-in.-diam water treatment and hydrogen cooling water lines in Units 3 and 4 failed in straight runs in areas which had been either weld repaired or excessively corroded. Circumferential cracks in these corroded lines, apparently caused by the earthquake, were observed; however, leakage was minimal since the cracks were later found to be essentially closed.

General observations indicate that the piping systems are hung in a more flexible manner than that which would be required by current NRC criteria. For example, the Unit 4 main steam line moved about 1.5 in., impacting a structural steel member and denting its insulation. In most cases, the piping is supported in a similar manner to older operating nuclear power plants, and it may be inferred that the seismic response would be similar. These observations are, on the surface, encouraging since in all cases the circumstances leading to failure are dissimilar to nuclear applications in that damage occurred at weld repaired areas of past corrosive attack or at nonwelded pipe joints.

The circulating water lines are buried and are concrete/bell and spigot with O-rings in diameters of 36 in., 40 in., 48 in., and 60 in. for Units 1-4, respectively. No evidence of failure or excessive leakage of these underground pipes was observed.

Mechanical Equipment

It was generally observed that mechanical equipment was supported in a consistent and adequate manner, and in most cases the supports appeared quite rigid. Attention was paid in the design to provide lateral load resisting mechanisms. Historically, anchorage specified for thermal-hydraulic reasons has tended to provide adequate seismic resistance for mechanical equipment.

Only two functional failures of mechanical components were observed. The first was the fracture of a yoke of an air-actuated valve operator on a 4 in. line supplying steam to an evaporator in Unit 4. This observation is consistent with preliminary NRC evaluations which have predicted potential problems associated with high bending and torsional stress in piping and yokes due to the seismic response amplification of the operator eccentric mass cantilevered off the piping.

The second failure, which was later repaired, resulted from an unbolted pumphouse filter in Units 1 and 2. Movement of the filter caused failure of a small threaded pipe.

In a few cases, yielding was observed in the steel supports of various horizontal heat exchangers and horizontal tanks. This was often due to weak direction bending on wide flanged structural sections. In no case did the yielding affect operability. For example, Unit 2 feedwater supports had undergone plastic deformation without loss of function, and had been replaced with new designs. In addition, the guide bracing of a Unit 2 mud drum was damaged and had to be repaired.

Two items in Unit 3 were not securely anchored and moved during the earthquake. The turbine oil cooler was not anchored to its support pedestal and moved about 1/4 in. The feedwater heater, located on the roof was anchored on one end only with rollers on the opposite end. The unanchored roller supports appeared to have slid laterally (E-W) about 3-5 in. (Fig. 10). Apparently these deflections were adequately tolerated by the attached piping since no evidence of cracking was observed at the nozzles or other discontinuities in the piping. A lateral support has since been added.

Some transformers with large height-to-width ratios were mounted on steel wheels. Relatively small-diameter hold-down bolts restrained vertical movement. In adition, siesmic stops were provided to restrict E-W lateral movement and overturning loads. The hold-down bolts stretched until the seismic stops picked up the additional overturning loads. There was no positive support in the N-S direction, and one transformer examined had moved about 2 in. north. However, no structural damage or loss of function was evident. Other transformers were more positively anchored, and no evidence of movement or yielding was present.

Other components viewed included horizontal pumps, vertical pumps, and column-supported vertical tanks. None of these experienced observable problems. Of particular interest are the vertical deep-draft condensate pumps with 20-ft shafts. The anchorage of these pumps was originally strengthened to adequately withstand startup hydraulic forces. The hydraulic provisions adequately provided seismic resistance capability.

Electrical Equipment

The base and lat al support details of electrical equipment, including the anchorage of smaller pieces of electrical equipment mounted on racks or housed in cabinets, appeared to have adequately responded to the earthquake. There was no evidence of permanent deformation or uplift of any anchorages or supports. All electrical equipment functioned properly except for the output transformers where broken ceramic insulators on lightning arrestors caused a short. These were nearly vertical cantilever columns about 9 ft long, made up of three sections. The base section failed in each case, i.e. at the fixed end of the cantilever. Large insulators typically have failed during earthquakes; however, these occurrences do not have safety impact for either nuclear or fossil plants.

The observed equipment was typical of electrical equipment at older operating nuclear facilities. Representative examples of the following were surveyed:

Motor control centers
Swithchgear
Instrument panels
Control room panels
Transformers
Cable trays
Transmitters.

Cable trays in the El Centro Steam Plant are ladder type, a single layer in height, vertically supported by channel struts, and somewhat lightly loaded compared to nuclear power plant configurations. Overall, the cable tray systems are more flexible than current criteria would require; however, they are similarly supported as those observed at older plants. The support integrity was maintained, and the cable trays appeared to h[illegible] [illegible]t their functional requirements.

There were no apparent malfunctions of electrical control and instrumentation equipment during or after the seismic event. The control room panels were judged to be rigid since they were constructed of 1/4-in.-thick steel plate with steel framing. It appears that care was taken during fabrication and installation of the cabinets and their supports to assure an adequate anchorage. It is not immediately evident whether the anchorages were engineered. However, indiscriminate tack welding or the use of anchors that rely on frictional clamping forces was not observed. Most of the equipment was anchored using embedded anchor bolts. In no cases were pulled out bolts observed.

Special Structures

Failures were observed for all of the large right circular oil storage tanks located in the yard, except for No. 4 (Fig. 11). In each, the tanks were damaged at the roof-wall interface and leaked oil from the top due to sloshing during the earthquake. No damage was evident at the unanchored bases. The larger tanks are mounted on a bed of about 4 ft of gravel topped with 6 in. of asphalt, and the smaller tanks are mounted on concrete foundations.

The 135-ft-diam x 45-ft-high tank (No. 6) was the most badly damaged (Fig. 12). The tank roof had buckled, and a 1 in. by 8 ft slit at the welded junction of the roof and the wall occurred. The tank has since been repaired.

Failures were not observed in the dikes of the circulating water make-up lagoons.

Asbestos panels in the cooling tower were knocked off during the earthquake, but these were repaired within the first day.

LLNL Site Visit

For the most part, damage noted while becoming familiar with the plant layout was limited to minor cracking of concrete block walls and loss of caulking at isolation joints between units. The only significant structural damage to Unit 4 was buckling of the upper cross braces in the boiler tower (Fig. 7). Equipment damage of consequence was limited to water line breakage and shifting of some equipment supports. The boiler itself suffered damage that was most likely caused by bumping against the rigid stops (Fig. 6).

SMA Site Visit

A site visit was made in August 1980 by R. D. Campbell to photograph equipment and gather additional information necessary for estimating equipment frequencies. During the visit, the plant log book for the day of the earthquake and the day following was examined to determine immediate effects of the earthquake related to plant function. The following description is based on the log book entries and firsthand observations:

> The earthquake hit at 4:20 p.m. on 15 October 1979. Units 3 and 4 were operating at the time of the event. A lightning arrestor in Unit 1 broke, causing a short which tripped breakers and resulted in loss of station power. Both Units 3 and 4 were tripped when station power was lost. Unit 3 was restored to 10 MW capacity within 15 min, and Unit 4 was restored to service within 2 hr. At 9:40 p.m. that evening, Unit 4 was removed from service to repair cooling water piping that was damaged during the earthquake. The following day, Unit 3 was shut down for repairs to the hydrogen cooling water line and to the piping on the control air compressor.

> Functional damage that occurred or was repaired within the first one and one-half days as noted in the log book was:

Unit 1	Lightning arrestor broke causing short and breaker trip and loss of station power.
Unit 3	Hydrogen cooling water line broke in spot that was badly corroded.

Unit 4	Damaged cooling water lines to and from exciter. The lines leaked at Victaulic couplings (Fig. 9). Hydrogen cooling water line broke in badly corroded location. Asbestos panels in cooling tower knocked off during earthquake.
Units 1 and 2	Pumphouse filter was not bolted. Movement caused failure of small threaded pipe.
Unit 2	Guide bracing on Unit 2 mud drum damaged.

Other damage noted later that did not impair normal operating function was:

Unit 4 Structure	Diagonal braces on boiler support framed structure buckled due to impact of boiler on structure (Figs. 6 and 7. Note: Diagonals have since been replaced.)
Oil Tanks	All tanks except No. 4 were damaged at the roof and leaked oil from top (Figs. 11 and 12). No apparent damage at bottom of tanks. Tanks were unanchored with the large tanks mounted on asphalt bases and the small tanks on concrete foundations.
	Tank No. 3 was drained and entry was made during the August visit (Fig. 13). Of the 5 columns supporting the roof, one was badly buckled, causing the roof to sag locally. At about 90° from the buckled column, the roof-to-tank wall weld joint had failed, opening up a crack several feet long where fuel oil sloshed out. On tank No. 5, the roof sagged on one side indicating loss of column support. No. 5 was not inspected in August because it still contained fuel oil. The 135-ft-diam tank (No. 6) was the most badly damaged, but it had been repaired by the time of the August visit.

Unit 4 Air Preheater Foundation	One of the concrete foundation support pads had partially crushed but was still functional and had not been replaced (Fig. 8).
Unit 3	An unanchored turbine oil cooler moved about 1/4 in. on its pedestal.
Unit 4	The main steam line moved about 1.5 in., hitting a structural member and denting insulation (Fig. 14). Lateral snubber, which was about 6 ft away, may not have been operable.
Unit 3	Feedwater heater on roof was anchored on one end only. The unanchored end moved laterally about 4 in. (Fig. 10). A lateral support has since been added.
Unit 2	Feedwater heater on roof was mounted on wide-flange beam sections with weight supported by the flanges. Seismic event caused flanges to bend. Supports have been replaced with a more positive load path design. Note that the Unit 1 feedwater heater was supported in the same manner, but the supports did not yield.
Transformers	The transformers with the greatest height-to-width ratio were mounted on steel wheels. Hold-down bolts restrained vertical movement, and seismic stops were present to secure the transformers in both the vertical and E-W directions. The hold-down bolts stretched until the seismic stops picked up the overturning load. There was no positive support in the N-S direction, and the one transformer examined had moved about 2 in. north.

Another type of transformer with height and width about equ . was secured by four metal tabs mounted over lugs at the base of the transformer. The tabs could resist vertical and E-W movement. No yielding of the tabs or N-S movement of the transformers was evident.

A third type of transformer (the largest unit) was securely bolted to its foundation with no evidence of movement or support yielding.

Ceramic Insulators	Two grounding insulators failed. These were nearly vertical cantilever columns about 9 ft long made up of 3 sections. The base section failed in each case, i.e., the fixed end of the cantilever.
Stacks	The stacks on Units 1, 2, and 3 are bolted to concrete foundations with approximately 1.25-in.-diam bolts about 18 in. long. The bolts were stretched on all three units. They were retightened, but none were replaced.
Floor Joint	A steel plate that spanned the gap between Units 1 and 2 on the operating floor had buckled from relative motion between buildings.
Chemical Lab	Small cracks were apparent in the masonry walls. A check of drawings indicated that masonry walls were reinforced.

Conversations with plant engineering personnel indicated that there were no malfunctions of electrical control and instrumentation equipment during or after the earthquake. This type of equipment is usually the most vulnerable to shock and vibratory loading. The control room panels were judged to be rigid since they were constructed of 1/4-in.-thick steel plate with steel framing. Consequently, control room instruments probably did not experience high acceleration levels.

In general, most of the equipment appeared to be securely mounted. Most equipment supports appeared to be significantly overdesigned for the modest seismic criterion specified.

FACILITY DESCRIPTION

The El Centro Power Plant is the principal electric power generating facility of the Imperial Irrigation District. The plant is about 3 mi from the Imperial Fault and about 15 mi from the epicenter of the October 15, 1979 earthquake (Fig. 1). The facility consists of four units that burn oil or natural gas. Units 1, 2, and 3 were designed by Gibbs and Hill, and were built in 1949, 1952, and 1957, respectively. Unit 4--an 80-MW facility designed by The Fluor Corporation, Ltd.--was built in 1968. Figure 15 is a plot plan of the four units, and Fig. 16 is a photograph of Unit 4 looking north.

When Unit 4 was added, the existing station building was extended southward by adding three 31-ft bays. There are no E-W walls between the units; thus, one can see all the turbine-generators in a row (Fig. 17). A 2.75-ft space between the last column line of Unit 3 and the first column line of Unit 4 was provided to allow for possible differential settlement. Thus, total southward extension was 95.75 ft. Figure 18 is an elevation that shows the three bays, while Fig. 19 is a floor plan of the ground floor, which shows the column lines referred to in the elevations.

In the E-W direction (Fig. 20), the width of each bay as well as the overall width of Unit 4 were made the same as those for the existing building. From east to west these widths are 30 ft (service bay), 55 ft (central or turbine bay), 25 ft (heater bay), and 18 ft (auxiliary bay), for a total of 128 ft. The boiler structure, from which the boiler is suspended, extends an additional 76.5 ft westward from the building proper, for an overall E-W width of 204.5 ft.

The heights of the service and turbine bays were the same as for the existing units. The operating floor remained at 20 ft above grade. The height of the heater and auxiliary bays was increased to suit the requirements of the new equipment. The existing control room for Unit 3 was extended 34 ft southward to serve Unit 4. A new air conditioning system was installed to serve the requirements of the larger control room and to provide make-up cooled air. The existing passenger elevator located at the southwest corner of Unit 3 provides access to the roof and the upper platforms of Boiler No. 4.

SOIL

The soil at the site consists of very deep alluvial deposits composed primarily of stiff to hard clay interlain with laminations of silty clay loam and sandy loam. Appendix A discusses the data used to estimate the shear wave velocity for calculating the soil stiffnesses.

FOUNDATION

The three main structures of Unit 4--the turbine building, the boiler support tower, and the turbine-generator pedestal--are founded on a single 12.2-ft-thick hollow honeycomb-like reinforced concrete foundation (See Figs. 18 and 20). The foundation of Unit 4 is structurally independent from the foundations for the other three units. The plan dimensions of the foundation are approximately 96 ft by 207 ft. The bottom slab is 18-in. thick and rests atop unexcavated ground at El. 936.8 ft. The top of the foundation is at El. 949 ft, which corresponds to the existing site grade level. The top slab consists of 7-in.-thick precast concrete panels supported on a girder system and interlocked by a 5-in.-thick topping slab that serves as the ground floor. The increased column spacing and heavier loads for Unit 4 as compared to Units 1, 2, and 3 required thicker shear walls (honeycomb web members) in the foundation.

PRINCIPAL STRUCTURES

The turbine building structure consists of eight moment-resisting structural steel frames and three reinforced concrete shear walls. The shear walls are cast monolithically with the exterior steel frames on the east, west, and south sides of the building. Within the turbine building, in addition to the ground level slab at El. 949 ft, there is an operating floor at El. 969 ft, a heater floor in the service bay at El. 983 ft, and a heater roof at El. 998.7 ft.

The boiler support tower has three braced frames and one combination braced and moment-resisting frame that is shared with and forms a contiguous connection to the turbine building at column line G between column lines 15 and 16 (See Figs. 19 and 20). The boiler support tower extends an additional 45 ft above the turbine building to El. 1045 ft. The boiler is suspended from the top of the support tower by steel rods. Lateral restraint for the boiler is provided by rigid bumper stops at various elevations throughout the support tower (See Fig. 6).

A reinforced concrete pedestal supports the turbine-generator. The pedestal's eight columns form three bays in the N-S direction and one bay in the E-W direction. The pedestal is isolated from the building at the operating floor level by a 1-in. gap.

OTHER STRUCTURES

Each of the first three units has a gunite-lined steel stack supported on an independent concrete foundation. The unlined Unit 4 stack is supported on a steel braced frame with lateral support provided by the boiler structure.

There are six unanchored, fixed-roof oil storage tanks at the site (See Figs. 11, 12, 13, and 15). Numbers 1, 2, and 3 have a capacity of 15,000 barrels (1 barrel equals 42 gal of petroleum) each; Nos. 4 and 5 can hold 45,000 barrrels, each; No. 6, the largest, can hold 115,000 barrels, and is 135 ft in diameter and about 45 ft high.

ORIGINAL SEISMIC DESIGN CRITERIA

According to the engineering-design and construction completion report by Fluor Corporation (Ref. 9), the original seismic design of Unit 4 was as follows:

> The architectural and structural treatment of the building, with minor modifications dictated by variations in size and arrangement of equipment, was carried out in general conformity with criteria established for the previous three units. The building frame was of the rigid frame structural steel type of design. The framing was designed to handle specified loads and to resist stresses from earthquake shocks equivalent to a horizontal force of 0.2 of live and dead weights supported. It was assumed that this force was applied at the center of gravity of the live and dead weights, and then transferred to the structure and foundation.

Item 14 of the equipment specifications reads:

> The area is also subject to seismic disturbances, and all equipment supplied shall be designed to resist seismic forces of 0.2 g magnitude.

No further details of the seismic design were found.

SEISMIC RESPONSE OF STRUCTURES

The seismic response of the turbine building and its associated structures was determined using a normal mode time history method of analysis for a linear elastic model. The actual analysis was done using the commercially available STARDYNE structural analysis program package maintained by Mechanics Research, Inc., and supplied by Control Data Corporation (Ref. 10).

STRUCTURAL MODEL

The model used for analysis is composed of equivalent vertical beam members with masses lumped at floor levels. A schematic representation of the analysis model is shown in Fig. 21, and the average areas and moments of inertia of the respective members are given in Table 2. Material properties are shown in Table 3. Each model mass point represents the mass of the floor, boiler, and equipment at the respective level and a proportionate share of the mass of the walls and columns between each level. All masses are lumped at the respective floor levels of the structure. In particular, the expansion gaps between the boiler and its lateral support points are not considered. Each mass point has six degrees of freedom. The translational and rotational mass components are given in Table 4 for each node. The turbine building and the boiler support tower structural steel are interconnected and monolithically cast with the concrete shear walls along column line G (Fig. 20). Hence, these two structures are assumed to act together, and a single three-dimensional model is used to represent their combined stiffness and mass.

The section properties of the turbine building and boiler tower model are derived from the stiffnesses of the concrete shear walls, moment-resisting frames, and cross-braced frames. The overall stiffness of the turbine building is primarily dominated by the concrete shear walls. Therefore, the stiffness of the steel frames in the boiler tower and turbine building is represented by an equivalent concrete shear wall area.

The flexural stiffness of the frames is determined using the structural analysis program STRUDL, which is maintained by the Georgia Institute of Technology and provided by the Boeing Computer Service Company (Ref. 11). Unit loads are independently applied to the frames at each of the elevations

TABLE 2. Analysis model member properties.

Beam No.	Axial area resisting vertical motion, ft^2	Shear area resisting E-W motion, ft^2	Shear area resisting N-S motion, ft^2	Beam torsional constant, ft^4	Moment of inertia resisting E-W motion, ft^4	Moment of inertia resisting N-S motion, ft^4
1	6.9	0.5	0.8	600	3,900	1,900
2	8.0	0.7	1.2	900	3,900	2,400
3	8.0	1.1	1.9	900	3,900	2,400
4	176.6	41.6	93.4	189,000	383,900	184,300
5	208.8	59.5	93.7	430,000	717,900	220,000
6	237.7	65.4	110.2	470,000	806,800	234,800
19	359.0	359.0	359.0	45,400	28,000	139,0[illegible]

TABLE 3. Material properties of concrete and steel.

Material	Modulus of elasticity, ksi	Material damping, %
Concrete	3,100	10
Steel	29,000	7

TABLE 4. Nodal weights and rotational moments of inertia of the analysis model.

Node No.	Weight, kip	Rotational inertia about each axis, kip-ft2		
		E-W	N-S	Vertical
1	112	9,000	24,300	33,300
2	524	42,000	113,600	155,600
3	524	42,000	113,600	155,600
106	1,153	512,000	1,149,000	1,661,000
109	1,270	474,000	1,752,000	2,226,000
112	1,952	1,100,000	5,862,000	6,962,000
118	16,706	8,552,000	44,305,000	52,857,000
20	1,925	877,000	99,000	976,000

in question, and the resulting flexural stiffness constants are transformed to an equivalent concrete shear wall area. This equivalent concrete shear wall area is then used in conjunction with the actual concrete shear walls to determine the model beam properties shown in Table 2.

In constructing the model, we assumed that a rigid diaphragm connects the boiler tower, boiler, and turbine building at each elevation. The actual conditions deviate from this assumption in two locations. The platforms within the boiler tower are primarily steel grating and are considerably less than rigid in the horizontal plane wh~ compared to the essentially rigid concrete slabs at Els. 969 and 998.7. Also, the concrete slab at El. 983 is not fully rigid in the N-S directicn because it is only supported by the steel frames of the turbine building and does not touch the N-S shear walls.

'~ mentioned above, the gap between the boiler and its lateral seismic restraints is not modeled. In a design analysis of the turbine or boiler structure this omission would generally underestimate the actual building accelerations. However, for the purposes of equipment capacity estimates, we expect it to be an assumption that will lead to a lower bounding estimate of the equipment capacity.

The turbine pedestal is structurally isolated from the turbine building at El. 969 by a 1-in. design gap; hence, a separate single-mass three dimensional stick is used to model the pedestal (Fig. 21). The model properties are determined in a manner similar to that of the turbine-boiler model in that both flexural and shear stiffnesses are considered. However, unlike the turbine-boiler model, both stiffnesses of the pedestal columns contribute significantly to the overall stiffness of the pedestal model.

The material damping of the pedestal model and the first three levels of the turbine-boiler model is taken as 10% of critical damping--the value recommended for highly stressed concrete in NUREG/CR-0098 (Ref. 12). The upper three levels of the turbine-boiler model use 7% material damping, which more nearly represents the characteristics of a steel structure. Note that the structure as a whole was not damaged; however, the use of these high damping values will lead to a low estimate of the equipment acceleration, and, by implication, its capacity.

Both the turbine-boiler model and the pedestal model join at the base slab level where these structures are supported on a common base slab. At this level the entire model is supported by three translational and three rotational soil springs (Fig. 21).

Soil Springs

To account for soil-structure interaction, the soil was treated as an elastic half-space, and soil springs and damping were developed using the formulations in Ref. 13. The complex shaped footing was transformed into an equivalent rectangle with the same area for calculational purposes. The resulting six springs were attached to the center of rigidity at the mid-plane of the base slab. The spring and damping values are shown in Table 5.

In performing soil-structure interaction analyses, the large damping ratios for the soil are frequently of concern. This is especially true when using modal superposition methods with frequency-independent soil parameters. As indicated in Ref. 14, when the soil damping gets very high, the response approaches that of the fixed-base case.

For this analysis, the responses of the centers of gravity of the base mat and operating floor are compared for the fixed-base case and the flexible-base case, which uses composite modal damping incorporating the high damping ratios shown in Table 5. It is recognized that the resulting composite modal damping values shown in Table 6 may be unrealistically high for use in a modal superposition analysis. However, use of these values is appropriate for this evaluation because overestimation of equipment response is undesirable.

Figures 22 through 25 show the response comparisons for the two horizontal directions. The peak floor accelerations of the base mat (Figs. 22 and 23) are slightly greater for the flexible-base case than for the fixed-base case. In general, the two responses are similar. Conversely, at the operating floor (Figs. 24 and 25) the peak floor accelerations are lower for the flexible-base case than for the fixed-base case. The soil springs and damping may account for this effect.

TABLE 5. Summary of soil springs and damping.

Direction	Stiffness	Damping ratio, % of critical
N-S translation	638,700 kip/ft	94.5
Rotation about E-W axis	1.93×10^9 kip-ft/radian	77.8
E-W translation	569,060 kip/ft	89.2
Rotation about N-S axis	4.55×10^9 kip-ft/radian	68.6
Vertical translation	753,500 kip/ft	152[a]
Rotation about vertical axis	2.21×10^{10} kip-ft/radian	48.8

[a] 100% was used in the dynamic analysis.

TABLE 6. Natural frequencies and composite modal dampings.

Mode No.	Natural frequencies, Hz	Composite modal damping,[a] %	Mode No.	Natural frequencies, Hz	Composite modal damping %
1	1.92	43.4	21	18.38	10.1
2	2.02	54.4	22	18.85	10.0
3	2.69	32.0	23	22.16	10.1
4	3.69	32.7	24	23.54	7.1
5	4.43	79.6	25	27.22	9.8
6	4.75	50.3	26	28.73	8.1
7	4.90	78.7	27	30.98	9.9
8	5.01	42.7	28	32.81	10.1
9	5.47	61.4	29	38.22	10.2
10	6.16	12.3	30	40.52	9.5
11	7.57	7.3	31	42.06	9.0
12	7.89	11.4	32	43.66	8.4
13	9.42	9.1	33	43.83	8.2
14	9.80	11.2	34	44.45	8.4
15	10.55	13.9	35	46.95	9.4
16	13.02	7.1	36	50.52	10.0
17	14.18	12.2	37	56.29	7.2
18	15.91	7.0	38	61.36	9.7
19	17.32	9.9	39	70.37	7.0
20	17.50	7.2	40	71.53	9.9

[a]Calculated using the strain energy approach.

Figures 26 and 27 show the comparison in vertical response for the two cases. At both elevations, the peak response is significantly less for the flexible-base case than for the fixed-base case. In fact, the peak responses are less than the peak ground acceleration. This happens because the peak ground acceleration is caused by a single spike which is filtered out by the soil flexibility and damping. The peaks of the vertical floor response spectra occur at about the same frequency for both cases, as was expected for such high damping. Logic constraints of the software necessitate a maximum input value of 100% damping for the vertical direction. Parameter studies were conducted using an arbitrary 40% cutoff on composite modal damping. These studies indicate that the 100% vertical damping limitation will reduce the equipment response, thereby leading to a low estimate of equipment reserve capacity.

SEISMIC INPUT

The time history input is based on records of the October 15, 1979 Imperial Valley earthquake for U.S. Geological Survey Station No. 5165 (Ref. 3). Plots of the ground motion time histories and response spectra are included as Figs. 3 through 5. The motion records for the N-S, E-W, and vertical directions are properly time phased. Hence, no special numerical summation technique is required when simultaneously subjecting the model to all three ground motions. The digitized records had a 20 s duration and a 0.01 s time step.

PEAK ACCELERATIONS AND RESPONSE

The maximum accelerations and response for the main equipment floors are shown in Tables 7 through 9. This response is based on the use of composite modal damping values (Table 6) as determined by typical strain energy techniques. Note that the floor accelerations in the turbine building decrease as elevation increases. This behavior results from the soil springs, which cause an increase in the participation of higher modes compared to the fixed-base analysis.

The maximum predicted base shear values are relatively close to the design base shears determined from Ref. 9. The two shears are compared in Table 10.

The maximum absolute differential displacement of the turbine building relative to the pedestal is 0.7 in., which is less than the design gap of 1 in. This result agrees with the fact that no damage was observed at the building-pedestal interface.

TABLE 7. Maximum E-W structural accelerations and displacements for a peak ground acceleration of 0.35 g.

Elevation, ft	Acceleration, g	Displacement, 10^{-2} ft
Turbine building		
998.7	0.37	2.41
983.0	0.38	1.96
969.0	0.39	1.58
949.0	0.40	1.10
Turbine pedestal		
969.0	0.38	1.37

TABLE 8. Maximum N-S structural accelerations and displacements for a peak ground acceleration of 0.49 g.

Elevation, ft	Acceleration, g	Displacement, 10^{-2} ft
Turbine building		
998.7	0.51	4.21
983.0	0.51	3.34
969.0	0.52	2.57
949.0	0.55	1.56
Turbine pedestal		
969.0	0.52	2.50

TABLE 9. Maximum vertical structural accelerations and displacements for a peak ground acceleration of 0.66 g.

Elevation, ft	Acceleration, g	Displacement, 10^{-2} ft
Turbine building		
998.7	0.36	0.69
983.0	0.37	0.69
969.0	0.39	0.68
949.0	0.40	0.66
Turbine pedestal		
969.0	0.39	0.69

TABLE 10. Base force comparison.

	Base force, kip	
Direction	Design	Predicted
E-W	1800[a]	1389
N-S	1800[a]	1752
Vertical	Unknown	1024

[a]Estimated shear (0.2 x weight) based on Ref. 9.

MODE SHAPES AND FREQUENCIES

The representative mode shapes of the primary contributing modes are shown in Figs. 28 through 38. Table 6 lists the frequencies of the first 40 vibrational modes.

Shown in Fig. 28, the dominant E-W translational modes are 1 and 5 (1.93 and 4.43 Hz, respectively). Note that the first mode shows the motion of the top portion of the boiler structure relative to the lower portion of the turbine building. This is in contrast to the 5th mode, for which the relative motion is between the soil springs and the upper tower while the lower turbine portion remains essentially rigid.

Shown in Fig. 29, the dominant N-S modes are 2, 4, 6, 8, and 9, with modes 6, 8 and 9 (4.76, 5.01 and 5.47 Hz, respectively) largely controlled by soil translation.

Mode 7 is the dominant vertical mode, and at 4.9 Hz indicates a significant amount of soil translation (Fig. 30). The 15th mode at 10.55 Hz primarily results from vertical vibration of the upper boiler tower.

The 18th and 39th modes (15.9 and 70.3 Hz, respectively) are the only torsional modes to occur in the first 40 modes. They characterize the torsional and E-W rocking modes.

IN-STRUCTURE RESPONSE SPECTRA

To generate in-structure response spectra, response spectra nodes were selected and considered to be rigidly attached to the center of rigidity at each elevation in the model (See Fig. 21). The STARDYNE code was then used to generate acceleration time histories and response spectra at each of the in-structure response spectrum nodes. These spectra (which are shown in Figs. 35 through 51) include the effects of structural torsion and rocking. They are used in the next section to predict equipment response. Note that directional coupling effects are automatically accounted for because actual records were used. The spectral peaks were not broadened for this study.

EQUIPMENT RESPONSE

In this section, the in-structure response spectra generated from the model discussed in the previous section are used to estimate the peak spectral accelerations of selected equipment items in Unit 4 of the El Centro Steam Plant. The equipment items are located by floor in Figs. 31 through 34 (Table 11 contains the equipment list key). The figures also show the locations on each floor at which the in-structure spectra were generated. The spectra are shown in Figs. 35 through 51. For ground-supported equipment, see Figs. 22, 23, and 26.

ESTIMATING EQUIPMENT CHARACTERISTICS

For each direction, a fundamental frequency and damping ratio were estimated for each equipment item (Table 11). The frequencies were either calculated, estimated by engineering judgment, or estimated by comparison to similar equipment for which frequencies were determined by analysis or test. A visit was made to the site on August 13, 1980, to examine each equipment item, take pertinent measurements of supports, and photograph the equipment (Note: These photographs follow the in-structure response spectra, and each item is named and identified by the key number in Table 11). General observations, measurements, and photographs taken during the August visit served as the basis for estimating damping and fundamental frequencies. Because the time span of the study was short, drawings of equipment and supports were not obtained; thus, pertinent dimensions, weights, etc., germane to equipment frequency had to be taken in the field or estimated.

It appeared that most of the equipment responded well within the elastic range of material behavior; thus, damping values were estimated for elastic response. Reference 15 summarizes equipment damping ranges for piping and equipment for both elastic response corresponding to an approximate OBE response level and for inelastic response corresponding to an SSE response level. The guidance of Ref. 15 was used in estimating equipment damping.

In estimating equipment frequencies, calculations were not conducted for several items that appeared by inspection to be rigid--that is, having a fundamental frequency greater than 33 Hz. In the case of equipment that appeared to be supported in a nonrigid manner, dimensions were taken at the

Table 11. Estimated frequencies and response accelerations for equipment. Note: equipment photographs follow Fig. 51.

Equipment No.	Name	In-structure response spectrum node No.	Median damping, % critical	Estimated frequency, Hz			Basis for estimate	Spectral acceleration, g		
				N-S	E-W	Vertical		N-S	E-W	Vertical
				GROUND FLOOR						
1	Plant air compressor	51	NA[a]	Rigid[b]	Rigid	Rigid	c	0.55	0.40	0.40
2	Air receiver tank	51	NA	Rigid	Rigid	Rigid	d	0.55	0.40	0.40
3	Lube oil conditioning equipment	51	NA	Rigid	Rigid	Rigid	e	0.55	0.40	0.40
4	Condenser	118	NA	Rigid	Rigid	Rigid	d	0.55	0.40	0.45
5	Condensate pumps	52, 53	3	3-12[f]	3-12	Rigid	d	1.80-0.65	0.90-0.60	0.45
								1.80-0.60	0.95-0.50	0.40
6	Turbine oil tank and coolers	52	NA	Rigid	Rigid	Rigid	c	0.50	0.40	0.45
7	Seal oil cooler system	53	3	14-28	14-28	Rigid	d	0.55	0.45-0.40	0.40
8	Excitation cubicle	53	5	5-10	5-10	10-20	e	1.25-0.70	1.10-0.60	1.20-0.60
9	Neutral transformer	53	5	5-10	5-10	10-20	e	1.25-0.70	1.10-0.60	1.20-0.60
10	Voltage transformer and surge protection cubicle	53	5	5-10	5-10	10-20	e	1.25-0.70	1.1-0.60	1.20-0.60
11	Gas dryer	53	3	Rigid	23[g]	Rigid	d	0.55	0.40	0.40
12	480-V motor control center No. 1	53	5	5-10	5-10	10-20	e	1.25-0.70	1.10-0.60	1.20-0.60
13	480-V motor control center No. 2	52	5	5-10	5-10	10-20	e	1.20-0.75	1.00-0.40	1.30-0.65
14	Service water return pumps	56	5	3-7	3-7	Rigid	d	1.30-1.20	0.80-1.20	0.40
15	Drainage sump pump	56	5	3-7	3-7	Rigid	d	1.30-1.20	0.80-1.20	0.40
16	Boiler feed pumps	55	NA	Rigid	Rigid	Rigid	c	0.55	0.40	0.45
17	480-V switchgear	54	5	5-10	5-10	10-20	e	1.10-0.70	1.00-0.65	1.55-0.80

[a] Not applicable because item is rigid.
[b] Rigid denotes fundamental frequency greater than 33 Hz.
[c] By engineering judgment.
[d] Calculated.
[e] By comparison to calculated or test values for similar equipment.
[f] Ranges of frequencies are considered approximately ± one standard deviation from mean.
[g] Single-value frequencies have approximately one standard deviation of 35% of value listed because of uncertainties in modeling details.

Table 11 (cont'd.)

Equipment No.	Name	In-structure response spectrum node No.	Median damping, % critical	Estimated frequency, Hz			Basis for estimate	Spectral acceleration, g		
				N-S	E-W	Vertical		N-S	E-W	Vertical
18	Gland steam condenser	118	NA	Rigid	Rigid	Rigid	c	0.55	0.40	0.45
19	Low-pressure feedwater heater	54	NA	Rigid	Rigid	Rigid	d	0.55	0.40	0.50
20	High-pressure fuel oil pumps	55	NA	Rigid	Rigid	Rigid	c	0.55	0.40	0.45
21	Fuel oil circulation pump	55	NA	Rigid	Rigid	Rigid	c	0.55	0.40	0.45
22	Pre-boiler chemical feed equipment	56	3	16	16	Rigid	d	0.45	0.55	0.40
23	Instrument air compressor	54	NA	Rigid	Rigid	Rigid	c	0.55	0.40	0.50
24	Instrument air dryer	54	3	Rigid	23	Rigid	d	0.55	0.40	0.50
25	Pump control panel	55	3	2-6	2-6	Rigid	d	1.15-1.10	0.80-1.90	0.45
				OPERATING FLOOR						
None	Turbine-generator unit with exciter	20	NA	Rigid	Rigid	Rigid		0.50	0.40	0.40
26	24-kV unit switchgear	57	5	5-10	5-10	10-20	e	1.15-0.60	0.90-0.50	1.10-0.60
27	Hogging jet ejector[g]	58	3	2-9	2-9	2-9	d,h	1.30-0.80	0.80-0.75	0.25-0.90
28	Steam jet air ejector	58	NA	Rigid	Rigid	Rigid	d	0.55	0.40	0.40
29	Deaerator[i]	61	3	9-16	9-16	Rigid		1.00-0.55	0.80-0.40	0.45
30	Condensate drain tank	61	NA	Rigid	Rigid	Rigid		0.50	0.40	0.45
31	Boiler control panel	59	NA	Rigid	Rigid	Rigid	c	0.50	0.40	0.45
32	Turbine-generator panel	59	NA	Rigid	Rigid	Rigid	c	0.50	0.40	0.45
33	Differential oil pump	60	NA	Rigid	Rigid	Rigid	d	0.50	0.35	0.40
34	Motor control center No. 4-3	60	5	5-10	5-10	10-20	e	0.80-0.70	0.95-0.50	1.30-0.65

[c] By engineering judgment.
[d] Calculated.
[e] By comparison to calculated or test values for similar equipment.
[h] Ejector is pipe-supported; thus, piping frequencies govern.
[i] Deaerator is supported from deaerator storage tank, not floor mounted.

Table 11 (cont'd.)

Equipment No.	Name	In-structure response spectrum node No.	Median damping, % critical	Estimated frequency, Hz N-S	E-W	Vertical	Basis for estimate	Spectral acceleration, g N-S	E-W	Vertical
				HEATER FLOOR						
35	High-pressure feedwater heater No. 4-3	62	3	15	2	Rigid	d	0.55	0.85	0.40
36	Evaporator	63	3	9	Rigid	Rigid	d	0.95	0.35	0.40
				ROOF						
37	High-pressure feedwater heaters No. 4-4 and 4-5	64	3	23	2	Rigid	d	0.50	0.90	0.40
				YARD						
None	Condensate surge tank	Ground	NA	Rigid	Rigid	Rigid	d	0.50	0.35	0.75
None	Transformers	Ground	NA	Rigid	Rigid	Rigid	c	0.50	0.35	0.75
None	Forced draft fans	Ground	NA	Rigid	Rigid	Rigid	c	0.50	0.35	0.75

[c] By engineering judgment.

[d] Calculated.

site and used in approximate hand calculations to determine fundamental equipment frequencies. In many cases, the calculation revealed that the equipment was, in fact, rigid.

Frequencies estimated for electrical cabinets, such as motor control centers and switchgear, were based on test data for similar equipment. In a few cases, such as battery racks and equipment that had little support in one or more directions, the frequencies were estimated to be below the amplified acceleration response range of most response spectra (< 1 Hz). Sloshing frequencies for tanks filled with fluid were not estimated. The tank contents were treated as rigid masses, and fundamental frequencies were computed or estimated for the tank/support structural system.

No attempt was made to compute relative magnitudes of stresses in equipment or supports. In almost all cases, it appeared visually that all responses were within the elastic range for equipment listed in Table 11.

The frequency values listed in Table 11 are approximate. As indicated by the notes in the table, where a single frequency is given the ± one standard deviation on that frequency is estimated to be ± 35% of the value listed. Where a range is given the range should be considered to be the ± one standard deviation bounds. Damping values listed are considered to be median values with the standard deviation equivalent to about 35 to 40% of the listed value.

The equipment accelerational response is summarized in Table 11. In general, the equipment acceleration response varied from 0.5 to 1.8 g in the N-S direction, 0.4 to 1.2 g in the E-W direction, and 0.4 to 1.55 g in the vertical direction.

ANALYTICAL CONCLUSIONS

The El Centro Steam Plant, Unit 4, turbine building dynamic model with highly damped soil springs reasonably reflects the forces induced in the building during the earthquake. This conclusion is supported by the evidence of a low level of damage observed at the plant, the close relationship of the design to predicted base shears, and the small relative displacement at the turbine pedestal expansion joint. Because of the open north wall, the turbine building exhibits a large degree of eccentricity which, though undesirable, is partially compensated for by the inclusion of a significant live load in the original lateral analysis.

The original structural design criterion specified a static lateral load equivalent to 20% of the dead as well as the live load. It is difficult to relate this type of static loading criterion to an earthquake loading characterized by a particular peak ground acceleration. However, for this particular earthquake and structure the triaxial time history analysis gives base shear values in close agreement with the original design values. One must be cautious in assuming that all members of this structure will be adequate when designed by the 20% criterion. Because of the time phasing characteristics of earthquakes, it is quite possible for individual members to be over stressed while the base shear is at or below the predicted values. This situation may have been demonstrated by the buckled boiler tower bracing.

Equipment performed well during the earthquake, most likely because most equipment items were well anchored. The forces experienced by the plant equipment were on the order of 2 to 9 times greater than the 0.2 g specified design load. This would seem to imply a reserve seismic equipment capacity of about 200%. However, it is difficult to verify this value without knowing the actual lateral load the equipment could withstand. For example, if a stock item in Unit 4 is able to withstand a lateral load greater than that specified in the facility design, then such equipment may merely have operated at or below its capacity. It is expected that nuclear power plant equipment similar to that in Unit 4 and anchored as well should perform equally well during a similar earthquake.

RECOMMENDATIONS

Our recommendations address the following topics:

- Ultimate capacity estimates of equipment
- Stress evaluation of structural elements
- Effective vs instrumental acceleration
- Additional soil-structure interaction analysis
- Seismic instrumentation
- Predicted acceleration levels using current NRC methodology.

Additional work is recommended to estimate the ultimate capacity of the equipment in Unit 4. Such information could be used to quantify equipment reserve capacities.

We recommend that an analysis of stresses in the structure be conducted to verify the assumption that response was elastic.

Because there was so little damage, the effective acceleration that the structure experienced may have been less than that recorded. The phenomenon of effective vs peak instrumental acceleration should be investigated further, perhaps using data from the differential array.

The soil-structure interaction parameters represent an important part of the analysis. Therefore, we recommend that additional studies be conducted with a more refined treatment of soil-structure interaction than that used in this study.

We recommend that seismic recording instruments be installed in the El Centro Steam Plant to help resolve the questions about effective vs peak instrumental acceleration and soil-structure interaction.

An analysis of Unit 4 to current NRC criteria would indicate the levels of conservatism inherent in these design-related procedures.

DISCUSSIONS

An expanding data base of observations at large industrial facilities, such as the El Centro Steam Plant, that have experienced strong ground motion suggests that these facilities possess significant seismic resistance capabilities. It is estimated that the October 15, 1979 earthquake produced a horizontal peak ground acceleration of 0.5 g (0.66 g vertical) as compared to the original design criteria which specified a static lateral load equivalent to 20% of the dead and live load. A very simplistic comparison of these acceleration levels suggests performance of plant structures and equipment up to approximately 2.5 to 3 times design; however, actual performance may vary significantly from this based upon some very important considerations.

Before hard conclusions can be made relative to the demonstration of reserve capacity of the plant, a detailed review of the original design considerations is necessary. As previously indicated, very little is known; therefore, we must rely upon our knowledge of the state-of-design in the period of the mid 1940's to the mid 1960's, the time frame when the four units were designed. For piping and equipment, the very simple analytical technique of applying the 20% lateral load through the center of gravity was most probably used. This technique neglects the probable structural amplification of motion through the building which may increase the seismic load many times. Sesimic stresses may have been computed for the pressure retaining boundaries and supports; however, it is certain that the functionability of active equipment was not rigorously addressed. Therefore, we may attempt to infer factors above design for these items and others such as structural steel and concrete which may tend to show, on a qualitative basis, higher inherent capability. However, caution is necessary.

The reported analyses show that plant equipment experienced forces as a minimum, 2 to 9 times greater than the 20% specified design load. These estimates are considered conservative in so far as their absolute values; however, it is usually meaningless to attempt an estimate of reserve capacity unless the actual design load is known. In general, for particular performance requirements, this design load is less than the specified design criteria because the off-the-shelf capacities should exceed the minimum design requirements. This is most applicable to items which perform structurally or must retain leak tight integrity. It is unclear how close specific pieces of active equipment would be to the 20% design requirement if evaluated for functional requirements. Based upon these considerations, it appears that a detailed evaluation considering current acceptance criteria would be required to compare this estimation of seismic capacity to the response predicted in this study. This information would lead to improved estimates of reserve capacity.

As further review is necessary before a quantitative evaluation is possible, this study in its current state of evolution takes on a very qualitative perspective. Noteworthy is that the two operating units safely shut down after having experienced a severe seismic environment which generally exceeds that used in the current design of nuclear power plants in the eastern United States and, in some cases, nuclear power plants in California. Most importantly, there were no known malfunctions of electrical control and instrumentation equipment which have been recognized as the most sensitive to seismic forces in nuclear power plants.

On the surface, the piping leaks are disturbing; however, it is clear that the circumstances leading to the failures are dissimilar to nuclear applications (e.g., use of threaded couplings, weld repaired areas in locations of past corrosive attack, etc.). All high pressure and high temperature seamless, welded piping performed satisfactorily. This piping was fabricated using A-106B material which is used in nuclear steam piping.

Except for buckling of a few members of the boiler support frame, significant structural damage was not observed. The buckled members are not that surprizing considering the fact that the frame saw both inertial and impact loads which were much in excess of design. This structural configuration is also dissimilar to nuclear applications. Although the predicted base shears were judged to be less than design, higher up in the building structural elements saw higher loads than considered in design.

From these data and others available in the literature, it can be concluded that the inherent seismic resistance of engineered structures, piping and equipment is greater than is assumed in both past and current analysis and design procedures. Even facilities designed with very nominal seismic consideration, such as the El Centro Steam Plant, withstand severe seismic environments. It can be concluded that when even the most modest attention is paid in design to providing lateral load carrying paths, significant capability is rendered. In contrast, nuclear power plants are designed to very rigorous techniques. Therefore, it is reasonable to expect even higher inherent margins than are implied in this evaluation. Many of the factors which contribute to our conservative prediction of seismic response during earthquakes can be quantified in light of current knowledge. Other factors are largely unquantified at this time; however, this study and others demonstrate that they do exist.

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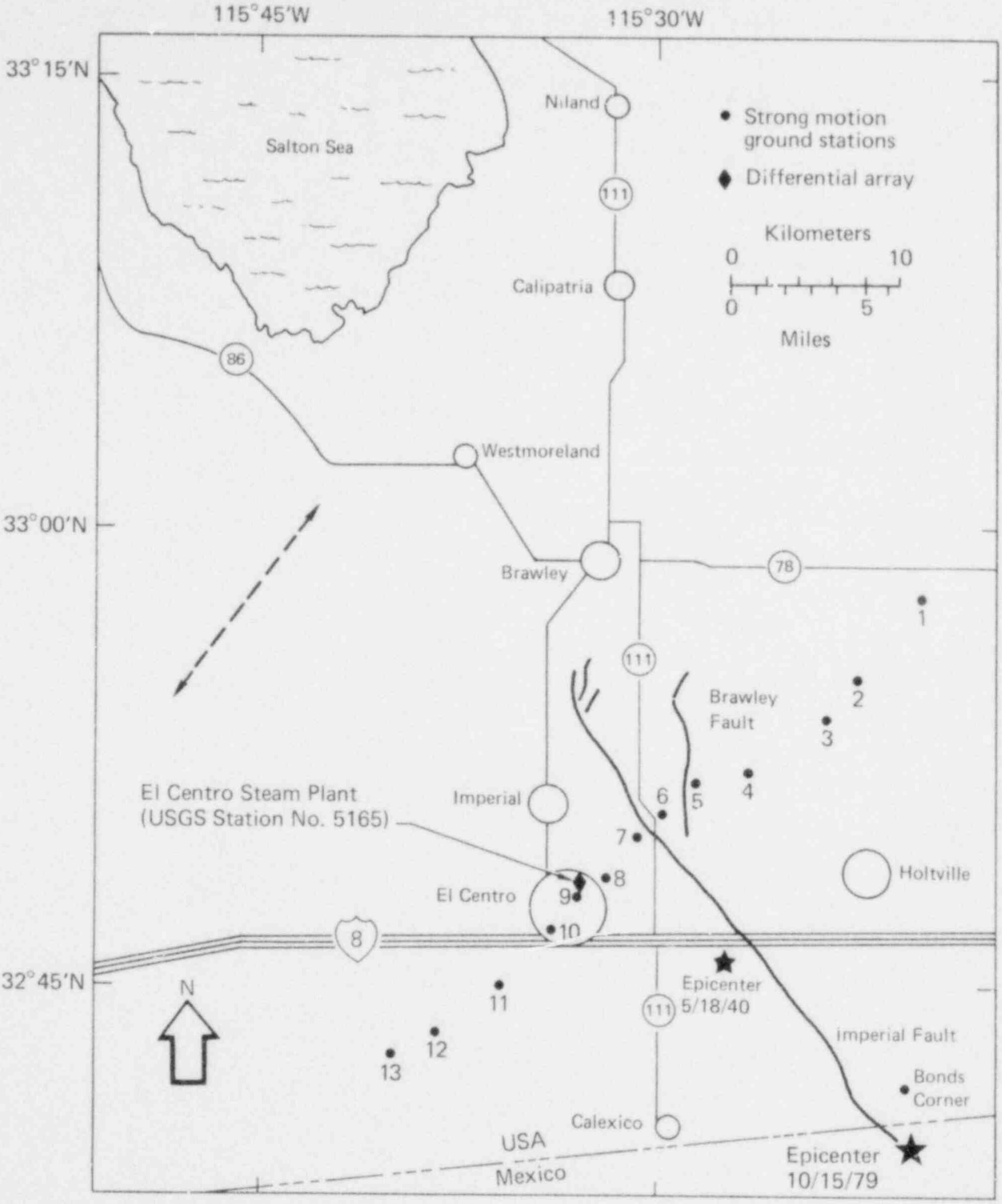
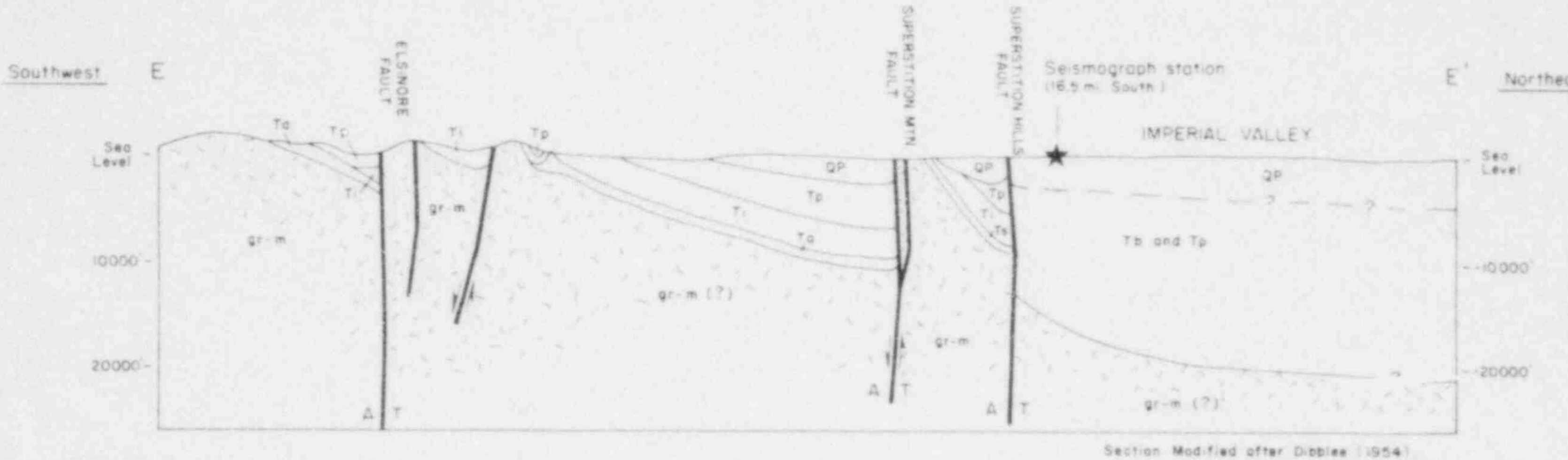


Fig. 1. Map of the Imperial Valley, Californi:, shows the Imperial Fault and the El Centro Steam Plant site location relative to the epicenter in Mexico of the October 15, 1979 earthquake, after Ref. 3. Also shown is the strong motion array that straddles the fault. Note: dashed line shows approximate location of section shown in Fig. 2.



EXPLANATION

0 6 12
Scale in miles

QP	Ocotillo Conglomerate and Brawley Formation, sedimentary rocks, Plio-Pleistocene in age
Tb	Borrego Formation, sedimentary rocks, Tertiary in age
Tp	Palm Spring Formation, sedimentary rocks, Tertiary in age
Ti	Imperial Formation, sedimentary rocks, Tertiary in age
Ts	Split Mountain Formation, sedimentary rocks, Tertiary in age
Ta	Alverson Andesite, volcanic rocks, Tertiary in age
gr-m	Igneous and metamorphic rocks, Pre-Cenozoic in age

———?— Contact between geologic units, dashed where approximate or inferred, queried where uncertain

———?— Fault, dashed where approximate or inferred, queried where uncertain

Fig. 2. Geologic cross section 16 mi northwest of El Centro, from Ref. 8.

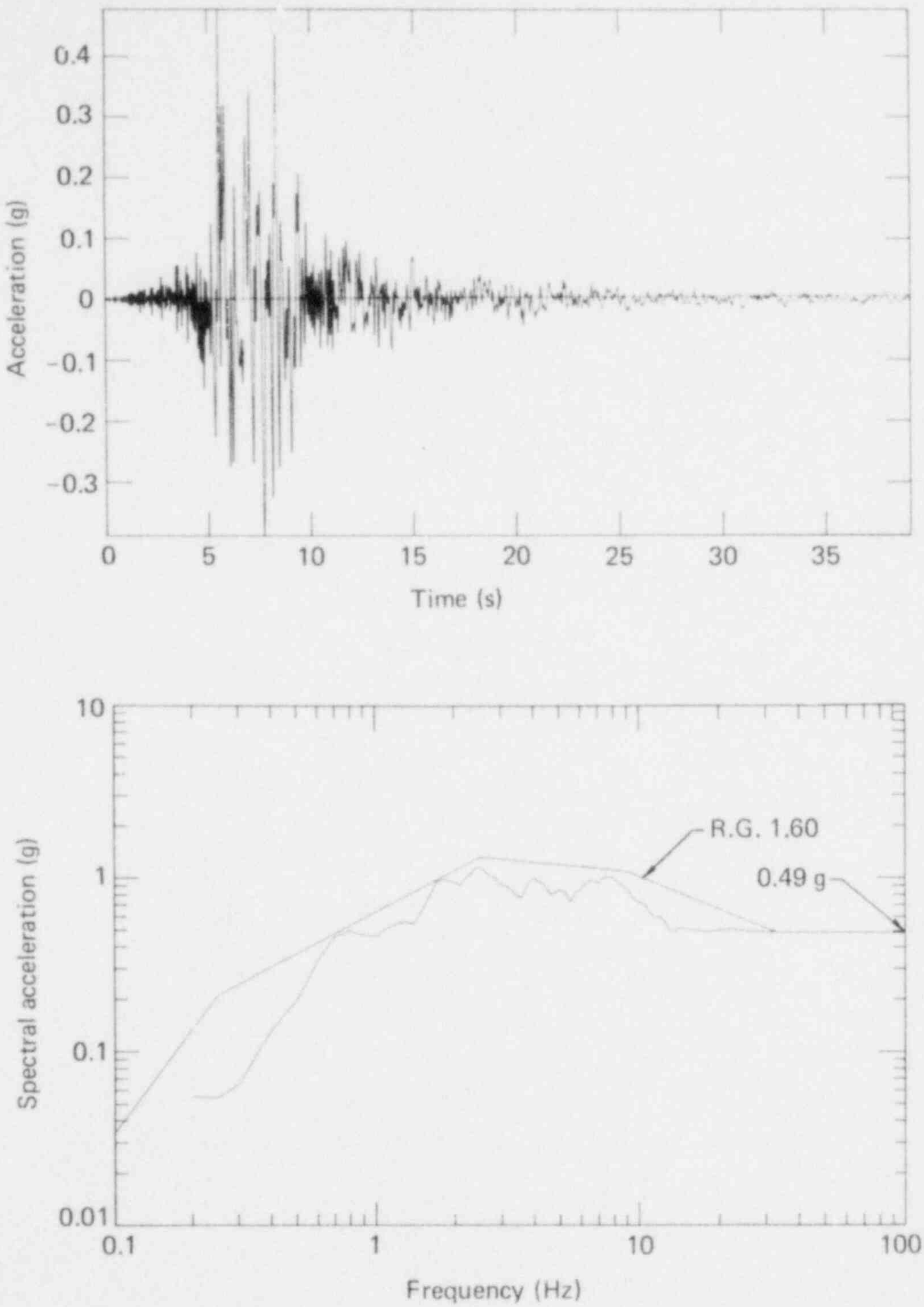


Fig. 3. North-south ground motion time history for the October 15, 1979 Imperial Valley earthquake recorded at U.S. Geological Survey Station No. 5165 (part of the differential array at the El Centro Steam Plant site) and used in the dynamic analysis. Also shown are the response spectrum developed from the time history of for 7% of critical damping and the NRC Regulatory Guide 1.60 spectrum for the same damping.

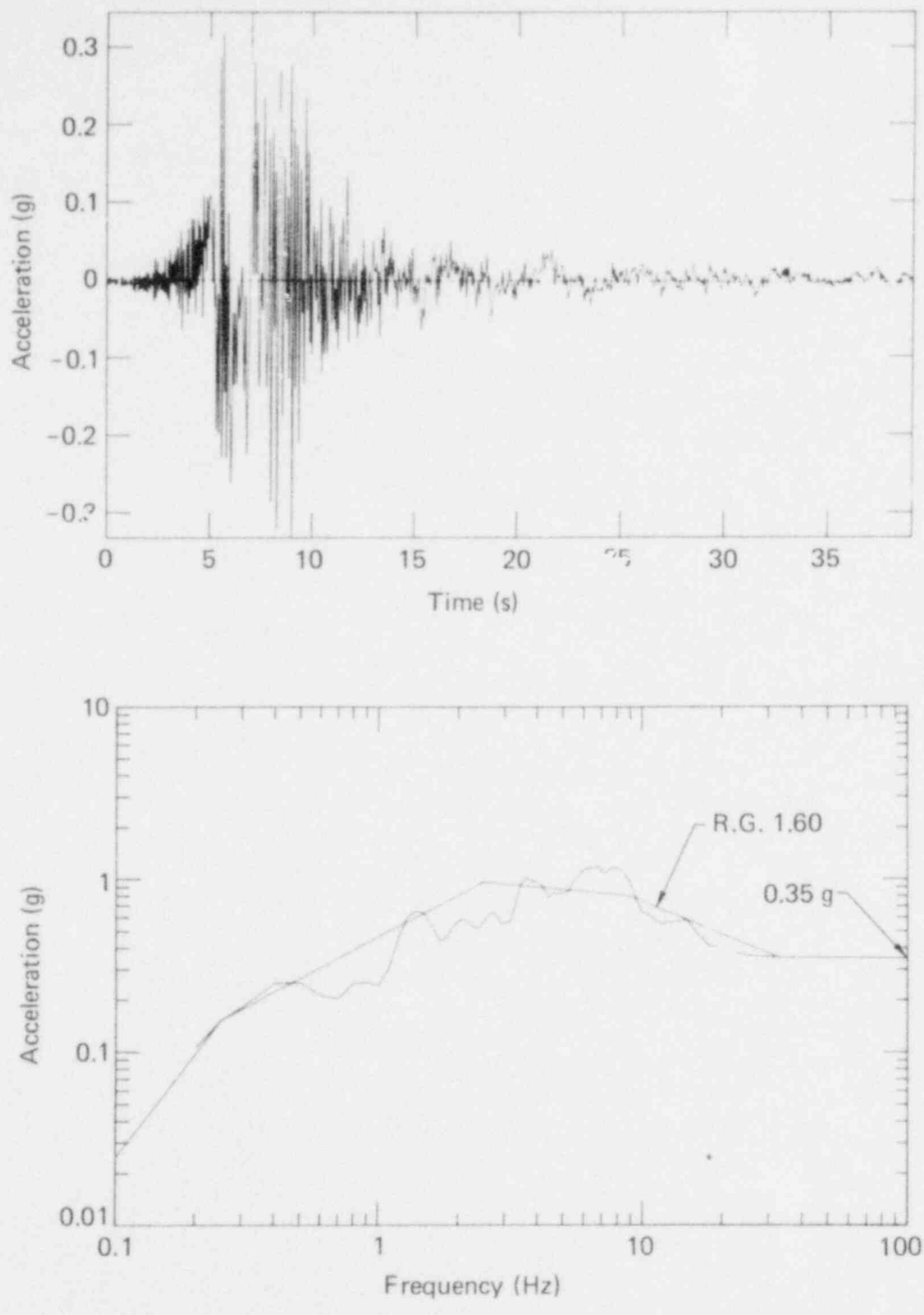
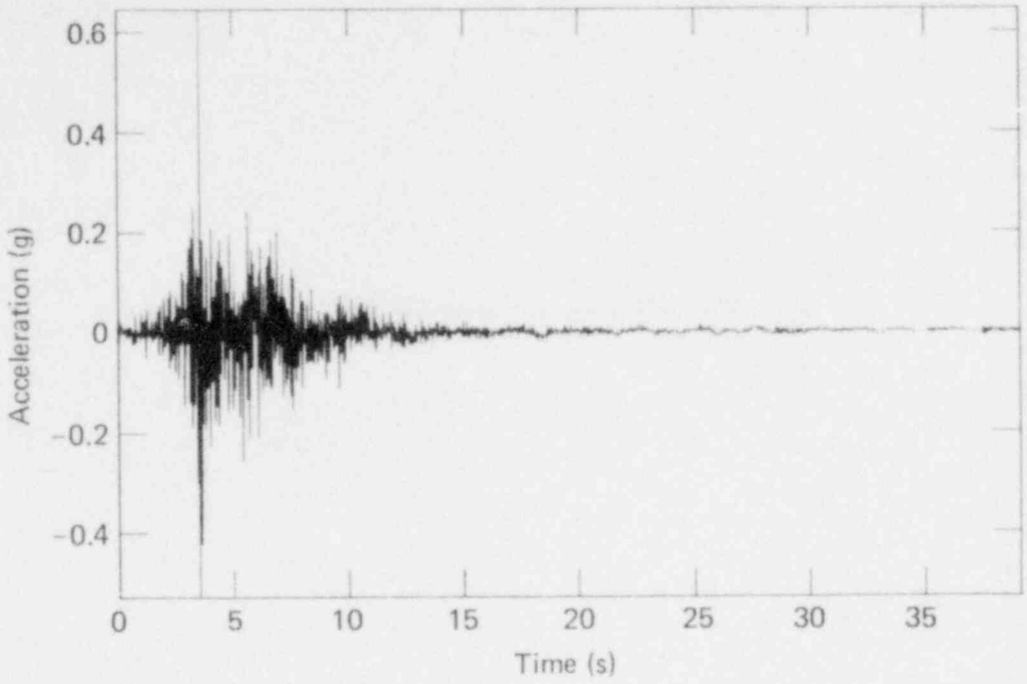


Fig. 4. East-west ground motion time history for the October 15, 1979 Imperial Valley earthquake recorded at U.S. Geological Survey Station No. 5165 (part of the differential array at the El Centro Steam Plant site) and used in the dynamic analysis. Also shown are the response spectrum developed from the time history of for 7% of critical damping and the NRC Regulatory Guide 1.60 spectrum for the same damping.



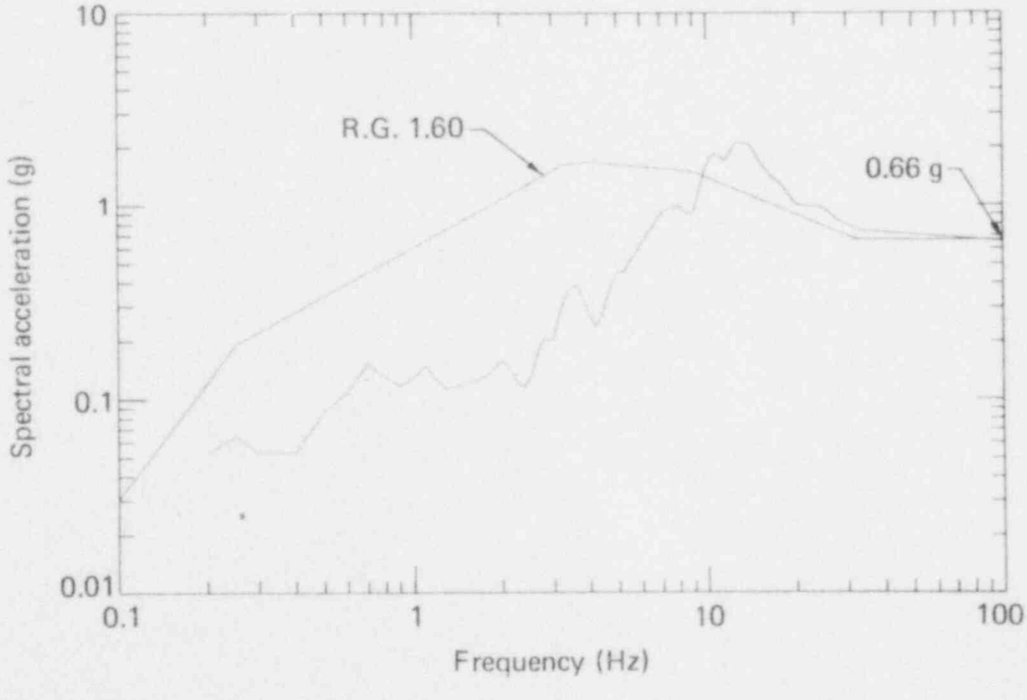


Fig. 5. Vertical ground motion time history for the October 15, 1979 Imperial Valley earthquake recorded at U.S. Geological Survey Station No. 5165 (part of the differential array at the El Centro Steam Plant site) and used in the dynamic analysis. Also shown are the response spectrum developed from the time history of for 7% of critical damping and the NRC Regulatory Guide 1.60 spectrum for the same damping.

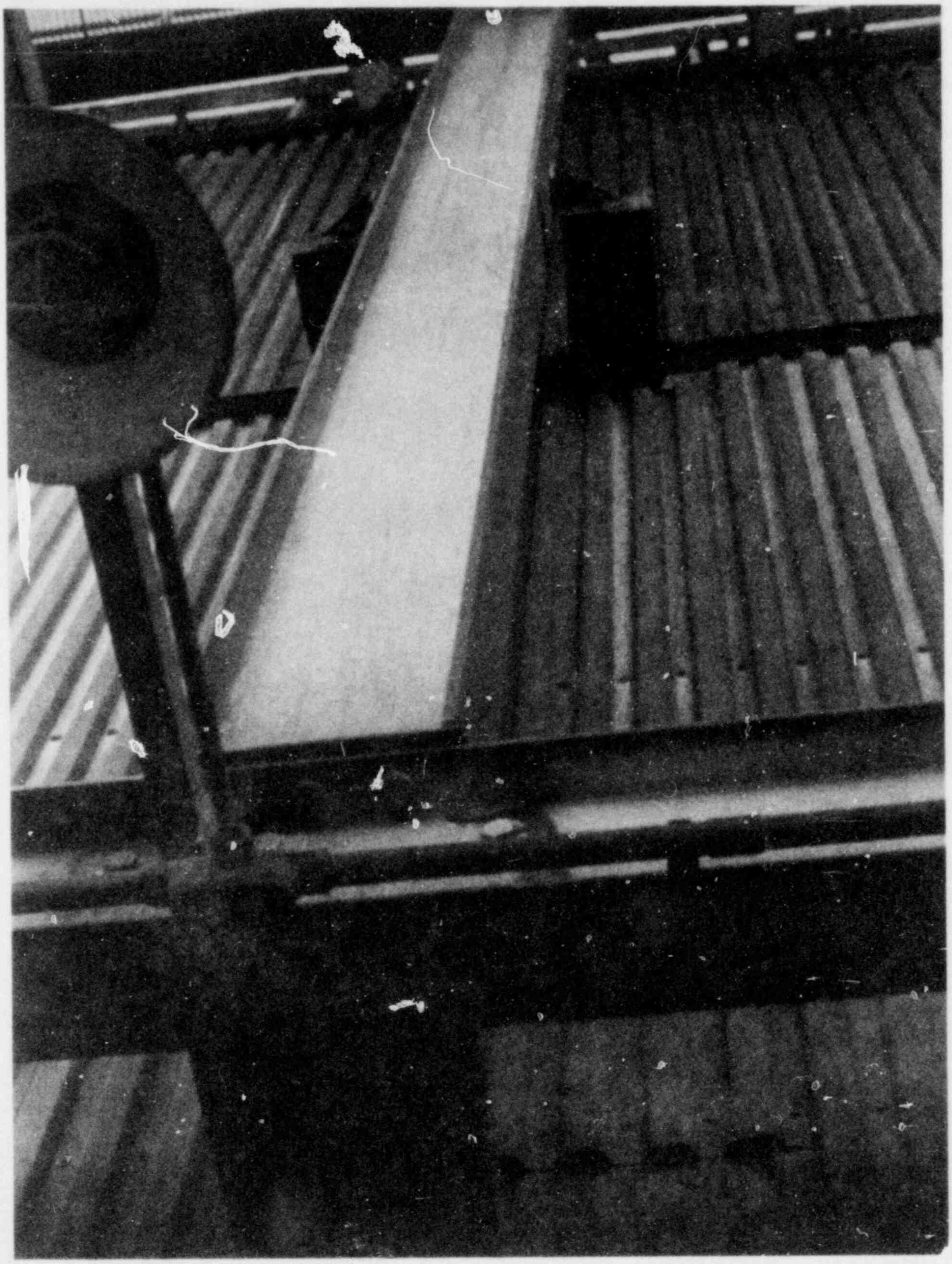


Fig. 6. Earthquake-induced motion of the Unit 4 boiler--which is suspended from a support structure--widened this seismic restraint on the east side approximately 2 in. by bending the cantilever beams (from Ref. 6).

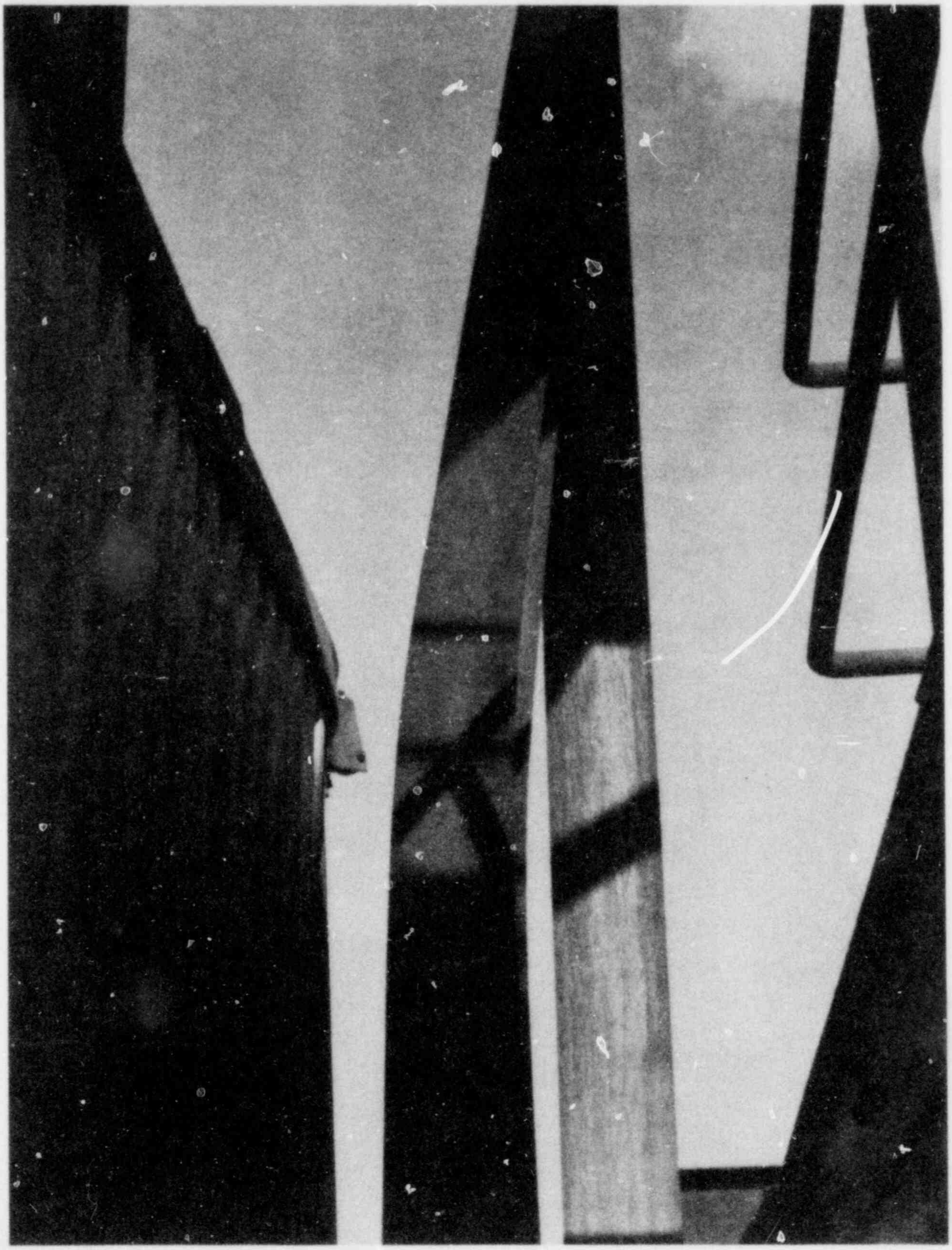


Fig. 7. Diagonal braces on the Unit 4 boiler support frame buckled and had to be replaced.

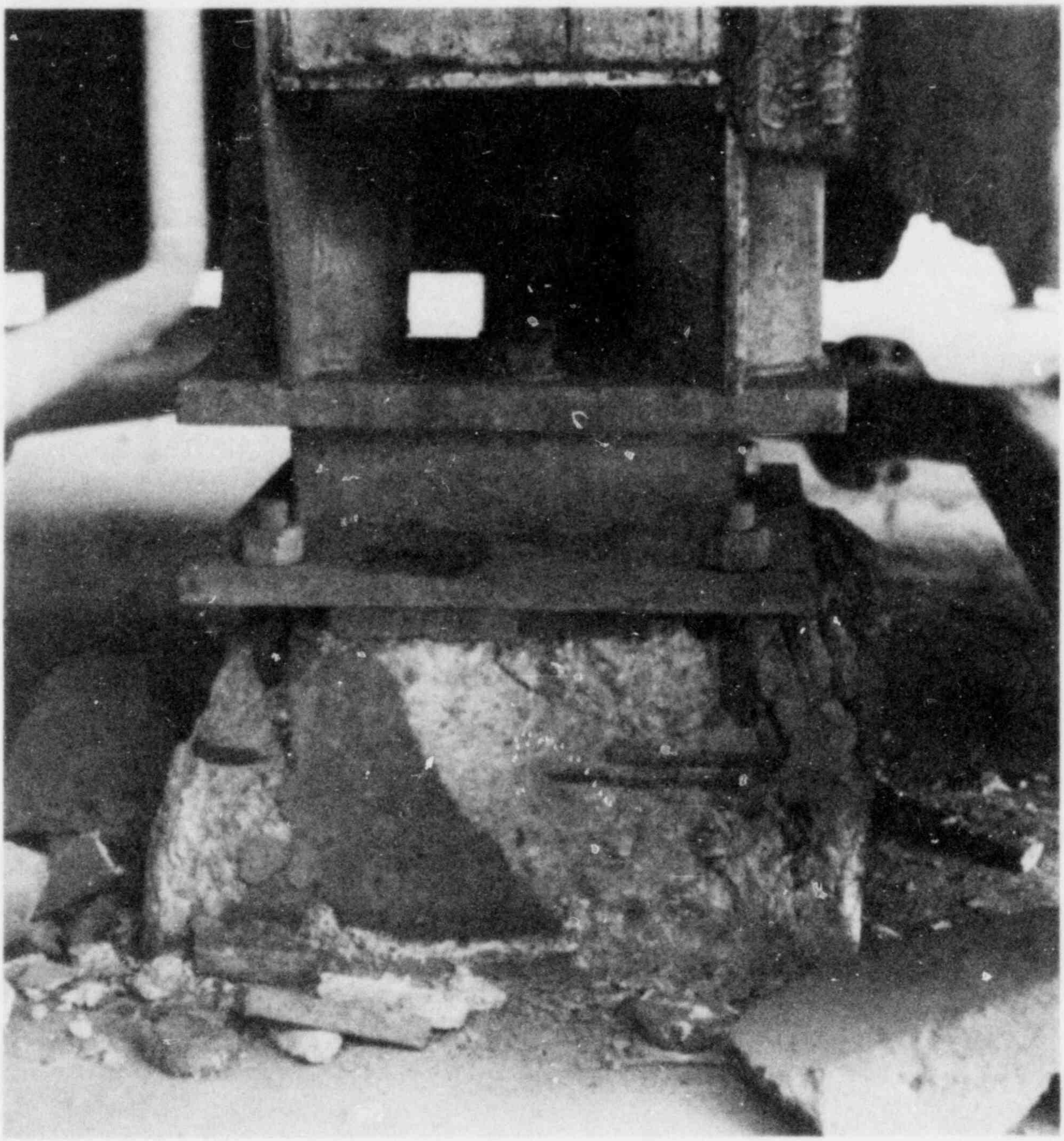


Fig. 8. A concrete foundation support pad on the Unit 4 air preheater foundation was partially crushed but remained functional.

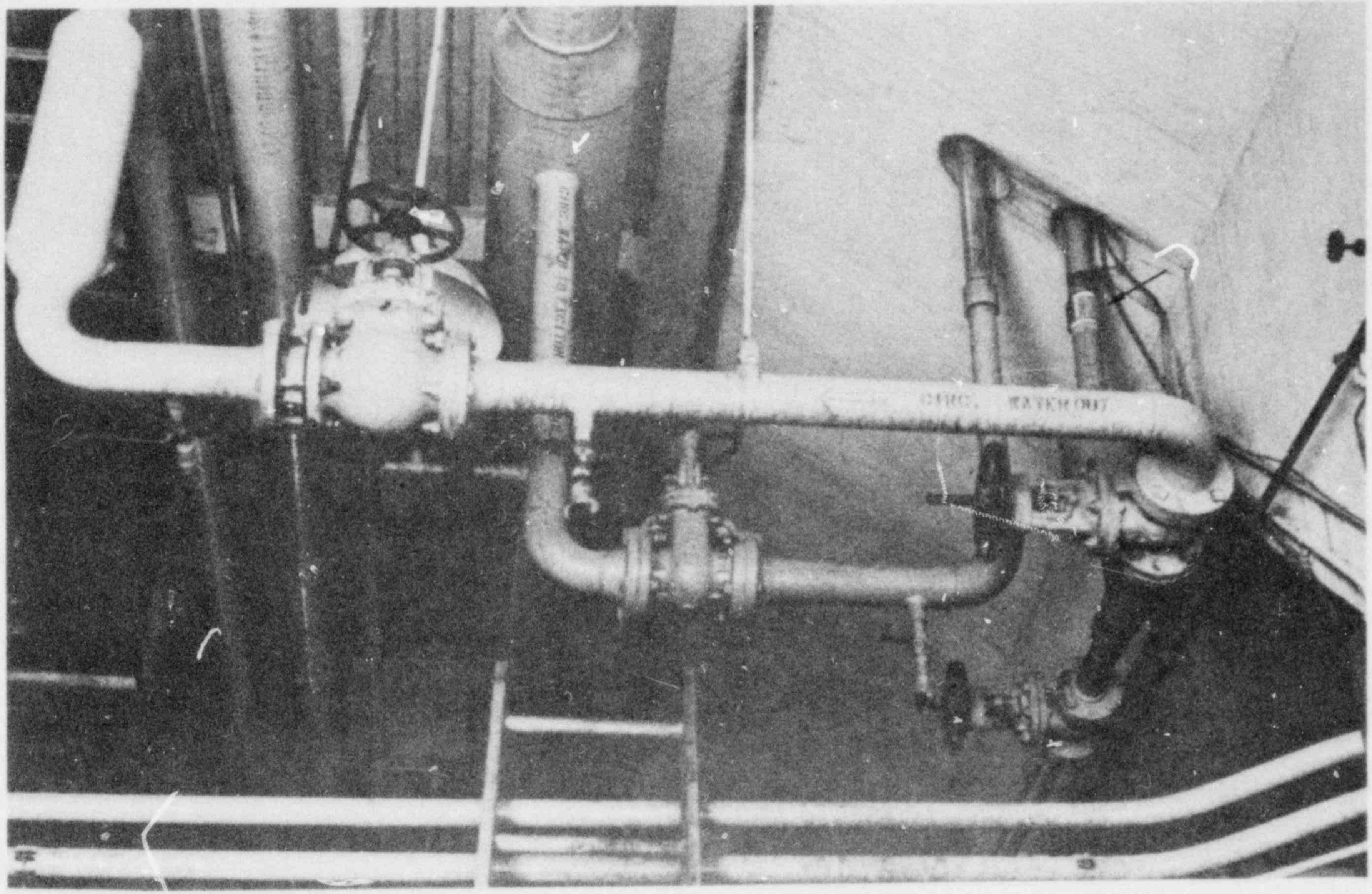


Fig. 9. Unit 4 developed leaks in the cooling water piping for the exciter cooler, shown here. Leaks occurred in locations weakened by corrosion. Arrow points to new Victaulic coupling.

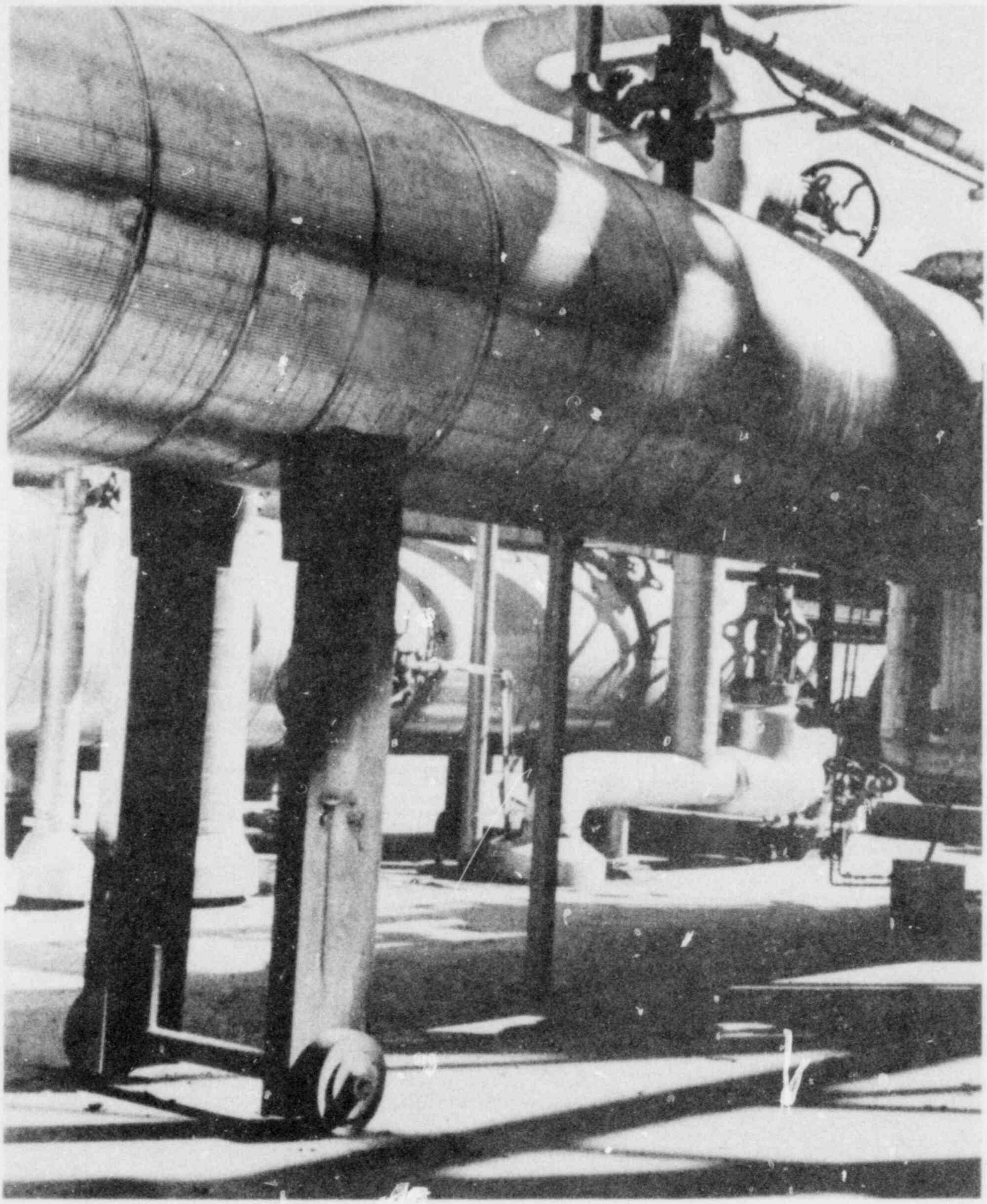


Fig. 10. A feedwater heater on the roof of Unit 3 was anchored on one end, but the unanchored end shown moved about 4 in. laterally. Note: wheels are about 6 to 8 in. in diameter.

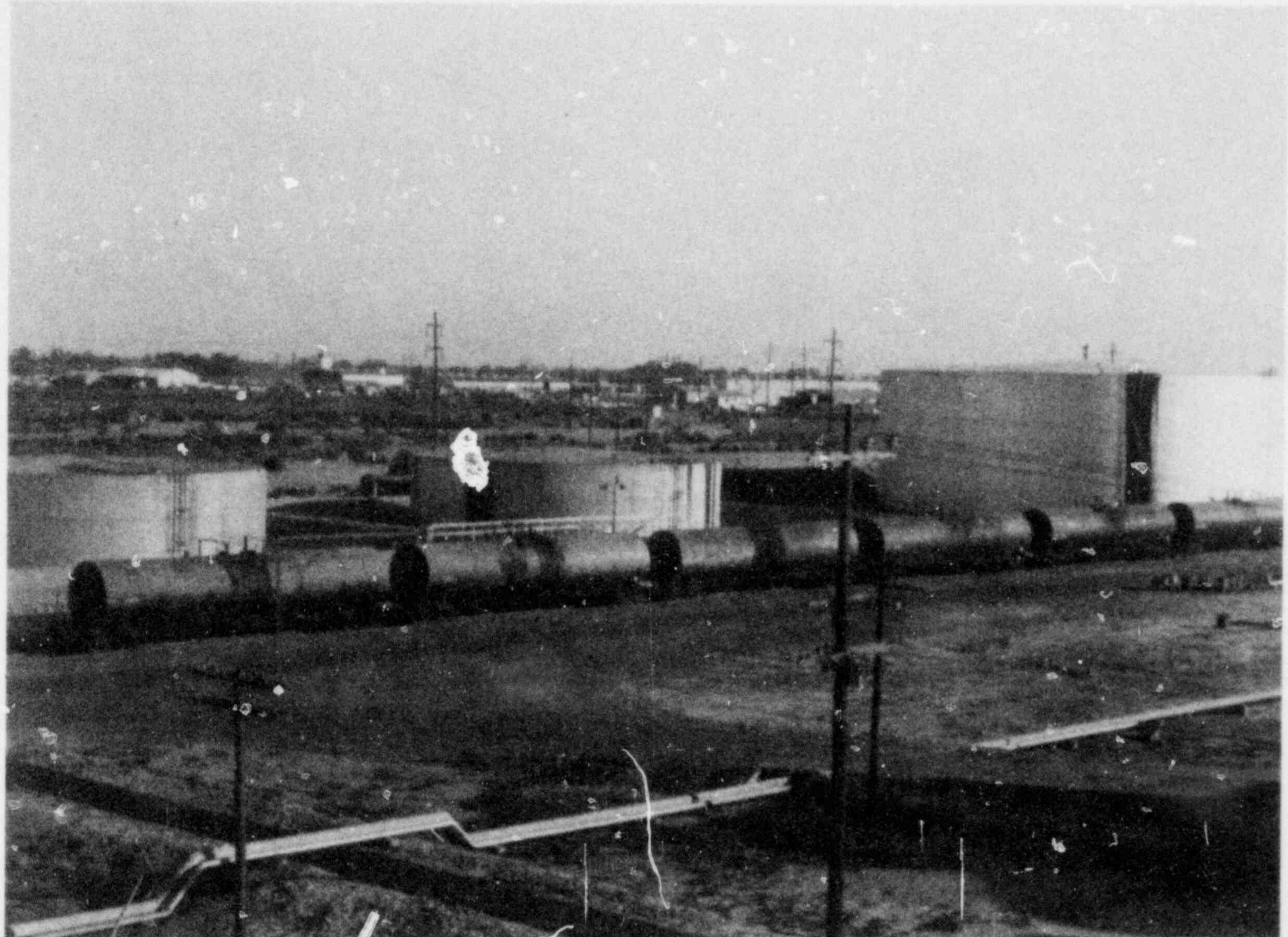


Fig.11. All oil storage tanks except No. 4 were damaged at the roof and leaked oil from the top (from Ref. 6).

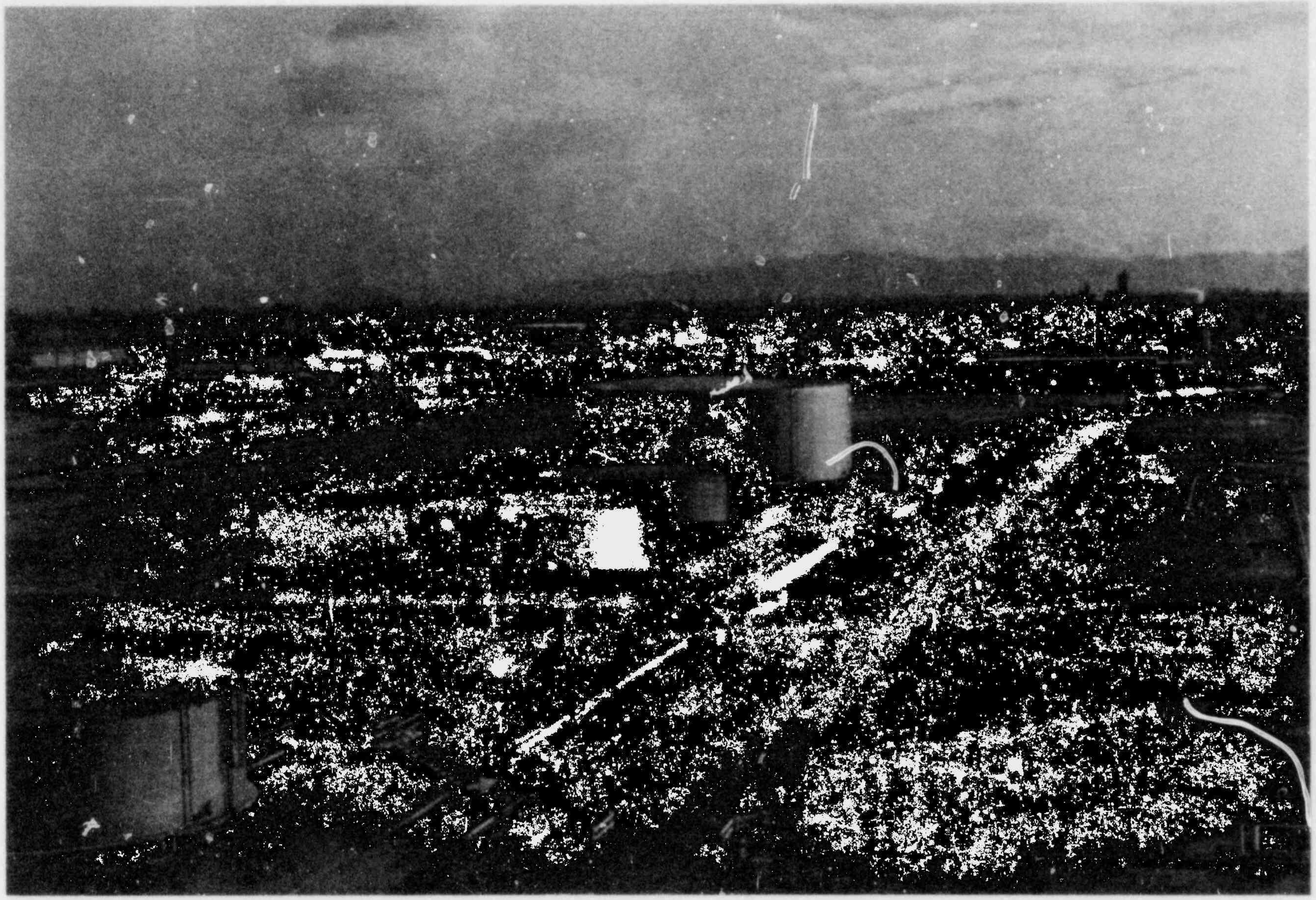


Fig. 12. The large 135-ft-diam tank No. 6 was the most badly damaged.

Fig. 13. Tank No. 3 was drained and entry was made during the August site visit. Of the 5 columns supporting the roof, one was badly buckled, causing the roof to sag locally. At about 90° from the buckled column, the roof-to-tank wall weld joint had failed, opening up a crack several feet long where fuel oil sloshed out.

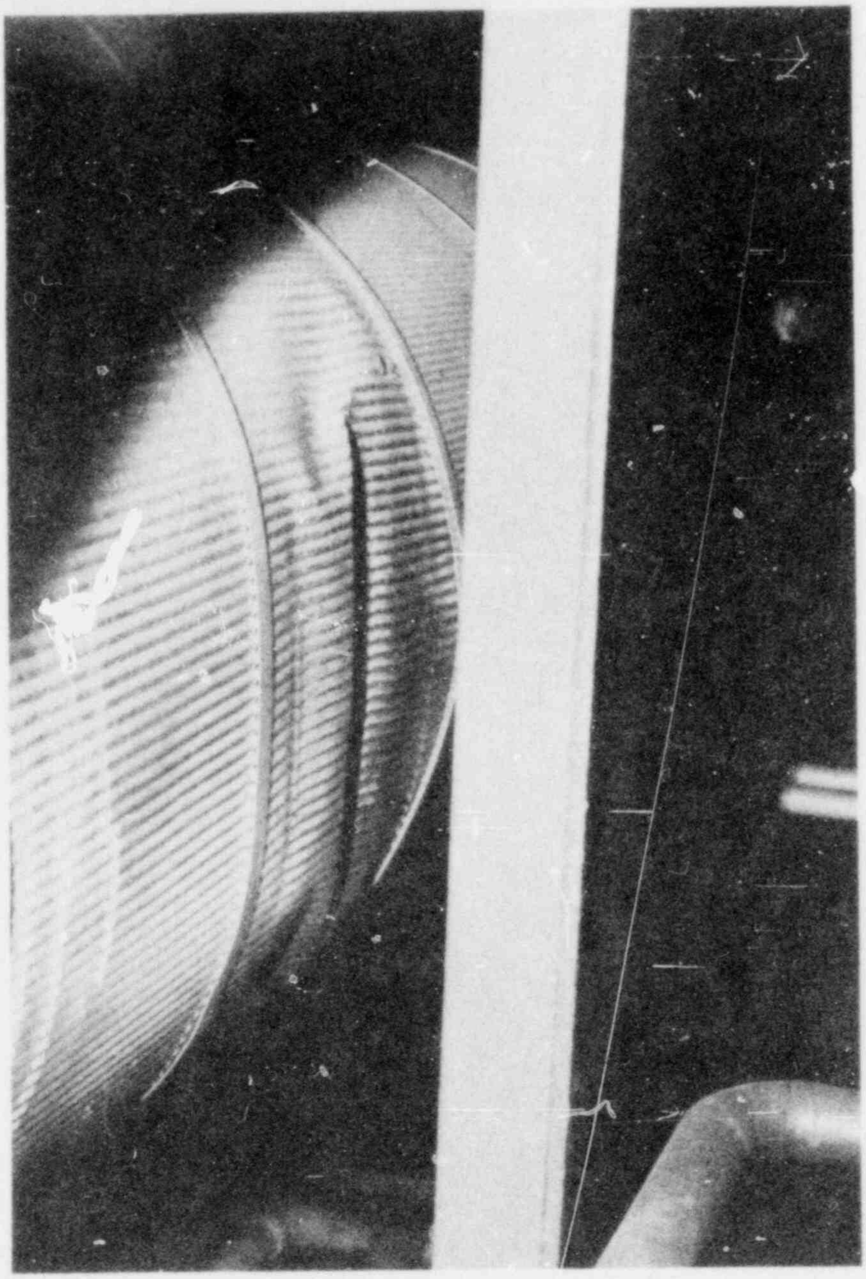


Fig. 14. The Unit 4 main steam line moved about 1.5 in., hitting a structural member and denting insulation.

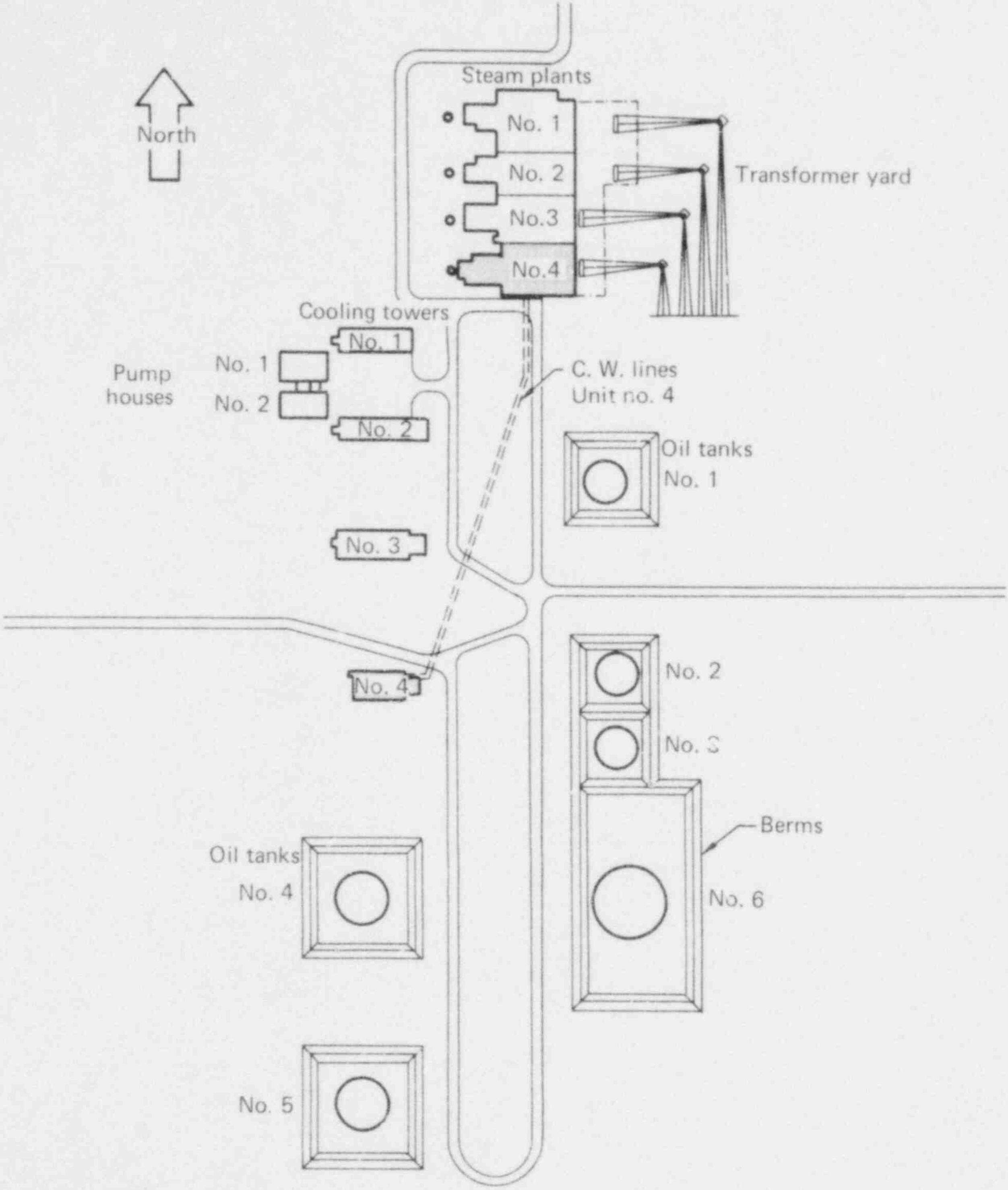


Fig. 15. Plot plan of the El Centro Steam Plant shows the location of Unit 4 relative to the smaller, older units.

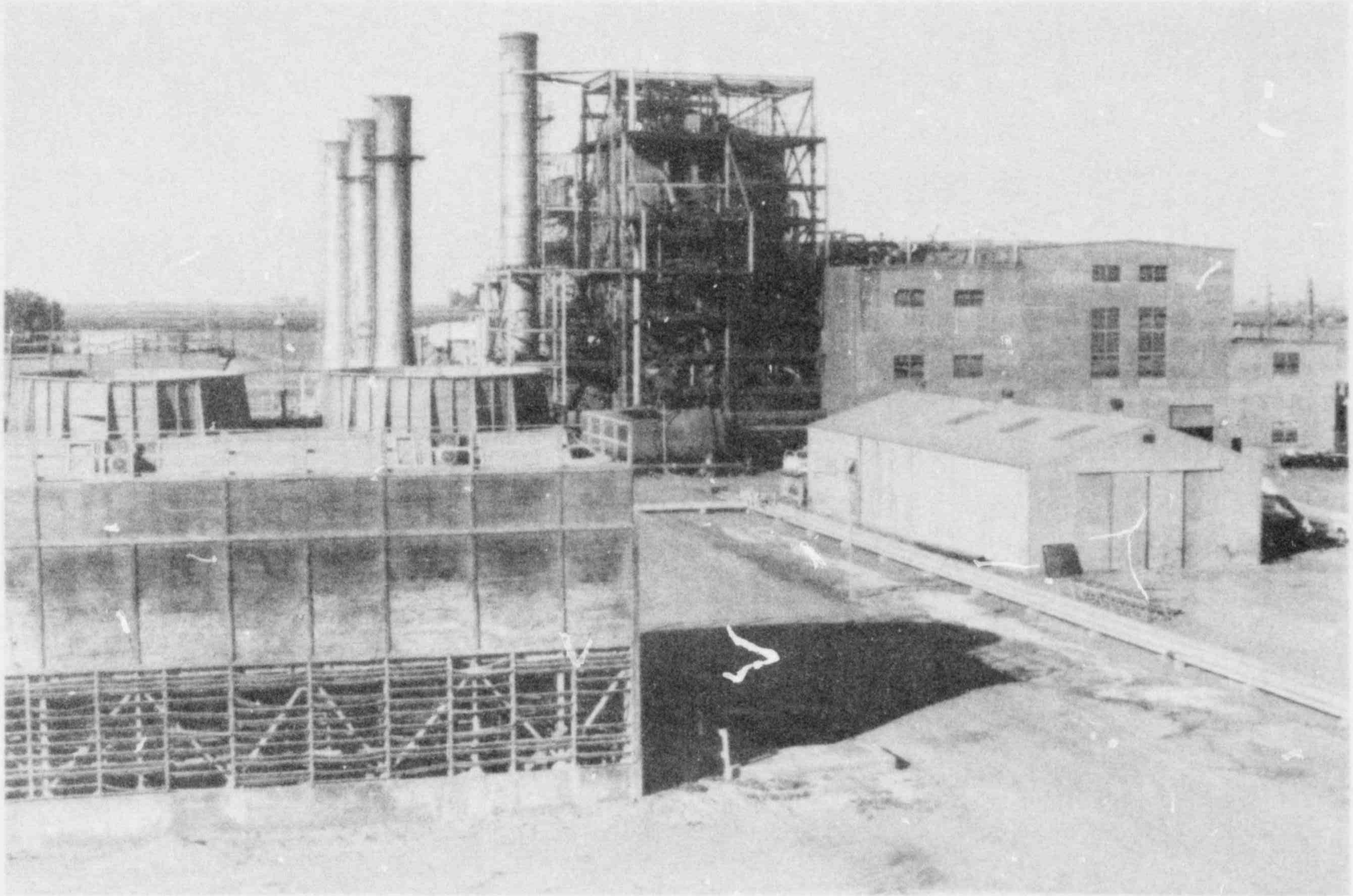


Fig. 16. View, looking north, of the El Centro Steam Plant shows (left to right) the free-standing stacks of Units 1, 2, and 3, the stack of Unit 4, the Unit 4 boiler support structure, and the south windowed walls of the Unit 4 bays. The cooling tower in the left foreground is for Unit 2. (Source: Ref. 6)

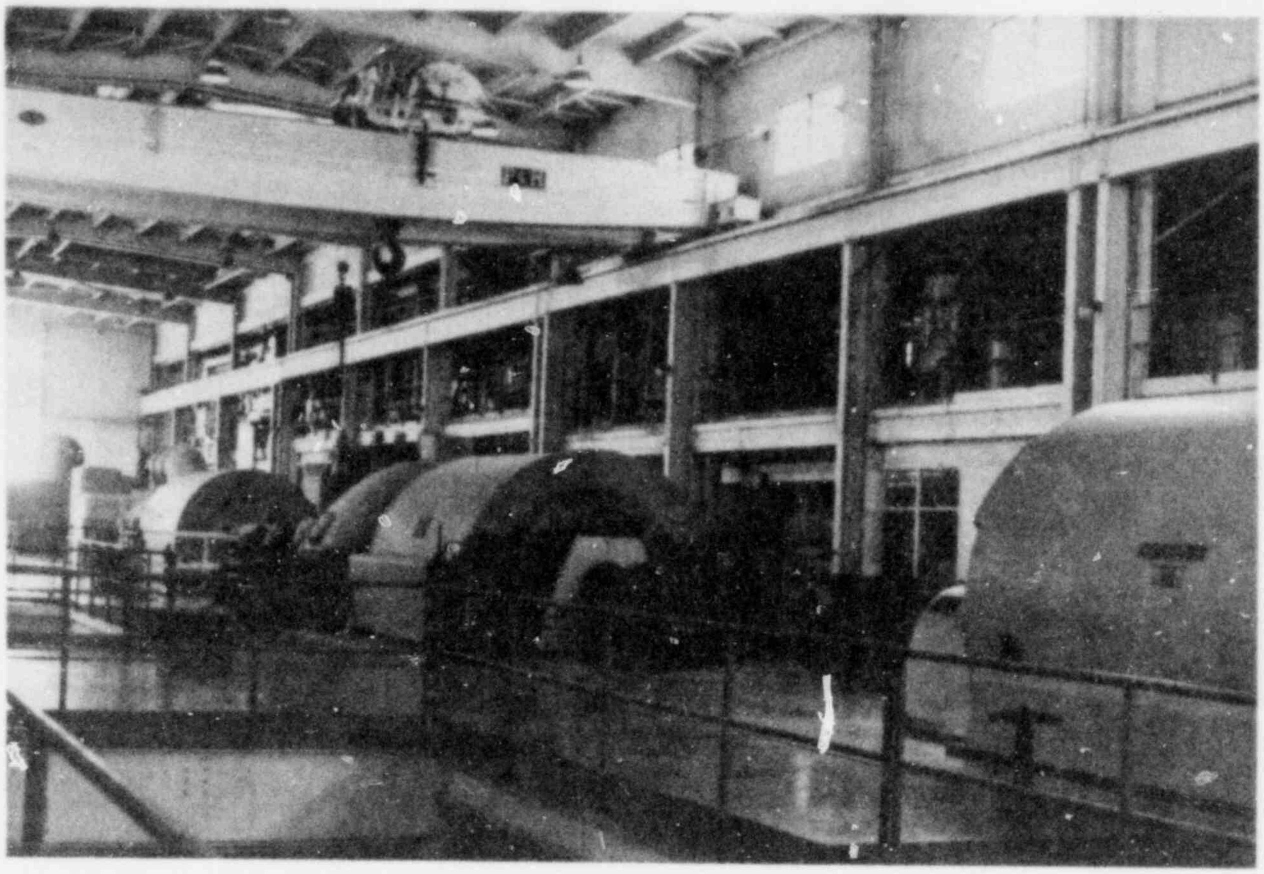


Fig. 17. View, looking south along the row of turbine-generators toward Unit 4, shows that there are no E-W partitions between the structurally separate units.

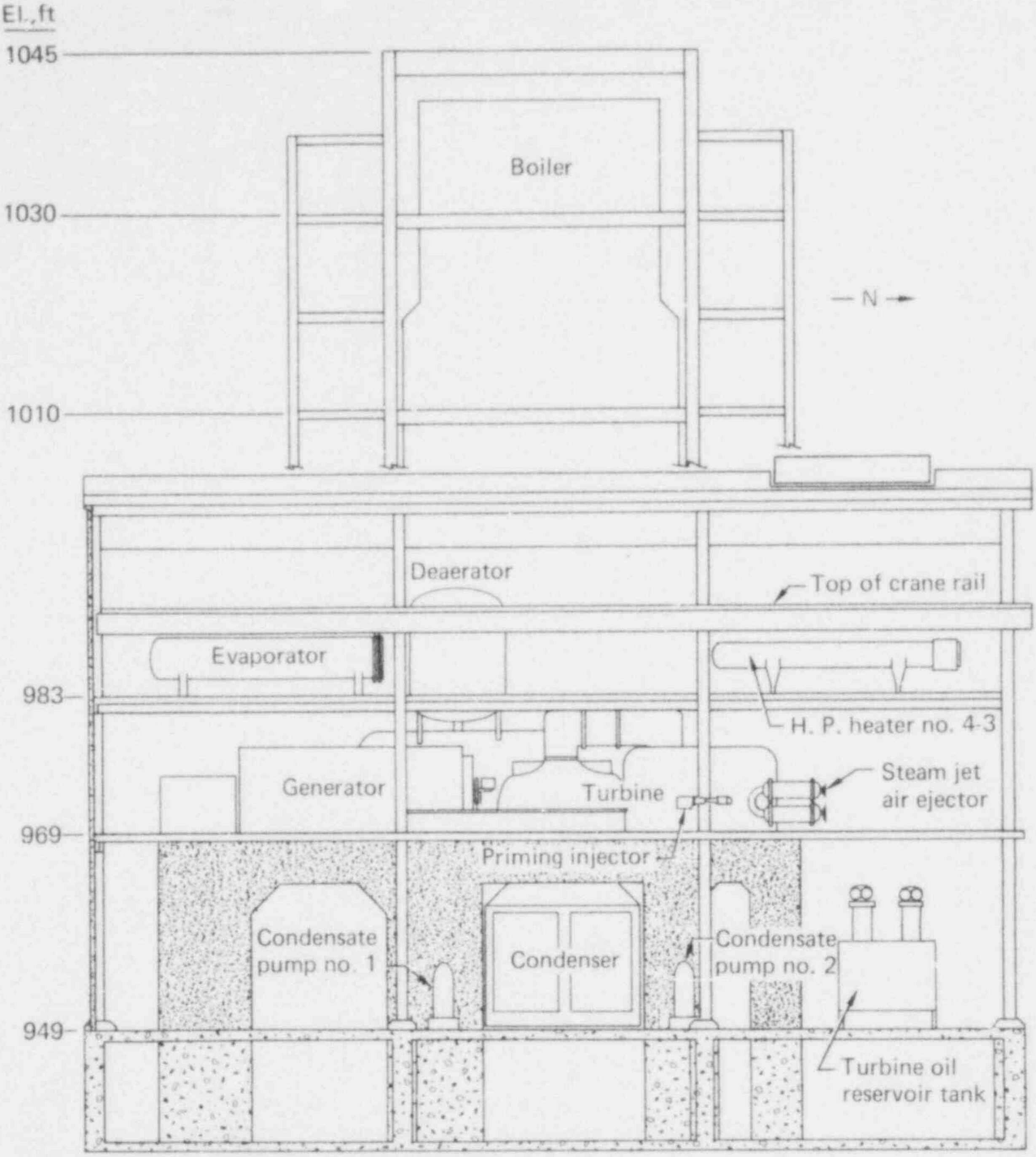


Fig. 18. Elevation drawing, looking west at column line B, of the El Centro Steam Plant, Unit 4. Note: boiler tower in background is an elevation at column line G. See Fig. 20 for column line locations.

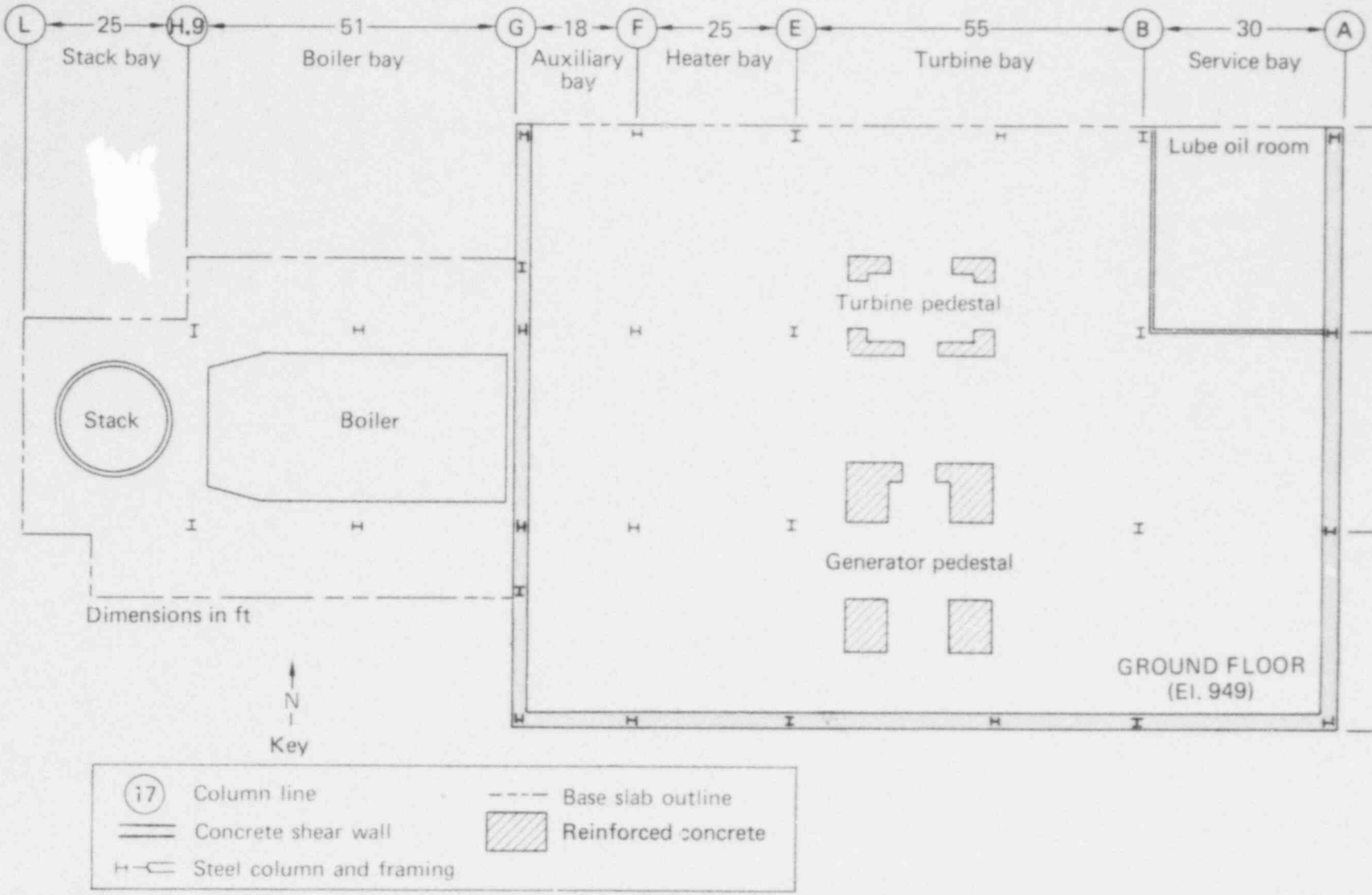


Fig. 19. Schematic plan view of the Unit 4 ground floor (El. 949 ft) shows shear walls, column lines, foundation outline, and turbine-generator supports.

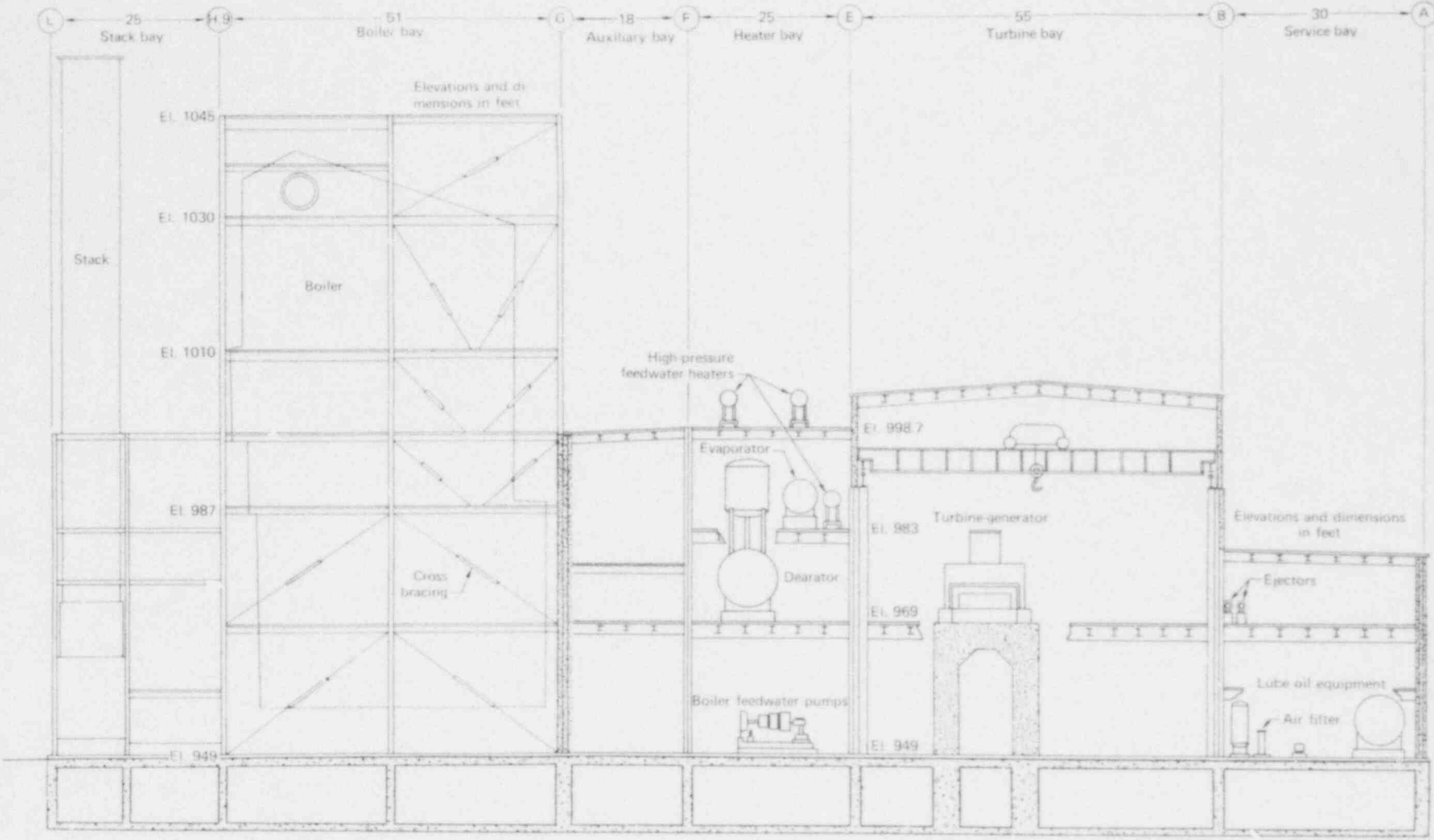


Fig. 20. Elevation drawing, looking north, of the El Centro Steam Plant, Unit 4. Note single honeycomb-like foundation.

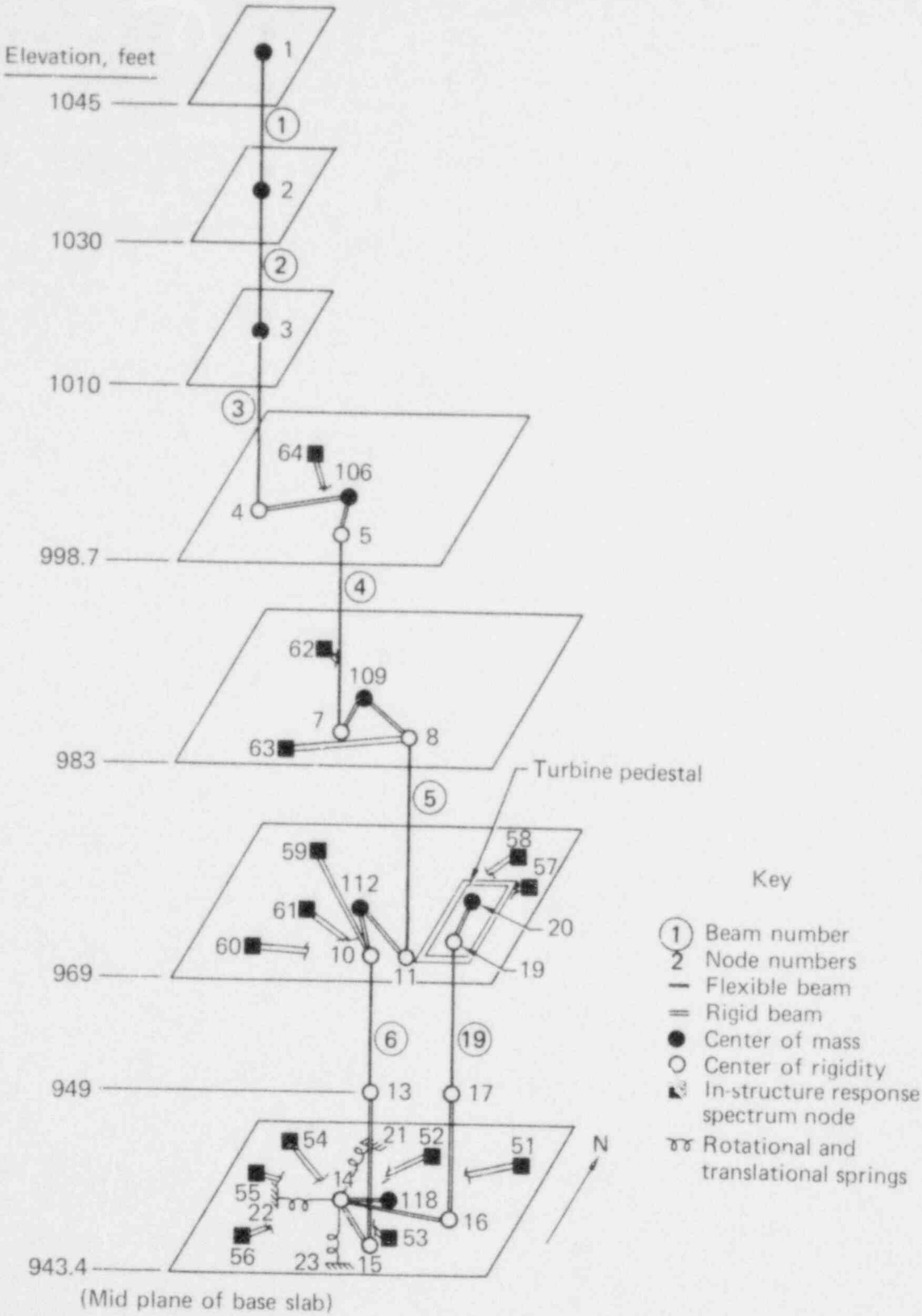


Fig. 21. Lumped-mass and beam model of the El Centro Steam Plant, Unit 4. Note the separate turbine pedestal beam, the translational and rotational soil springs, and the different locations at certain elevations of centers of gravity and rigidity.

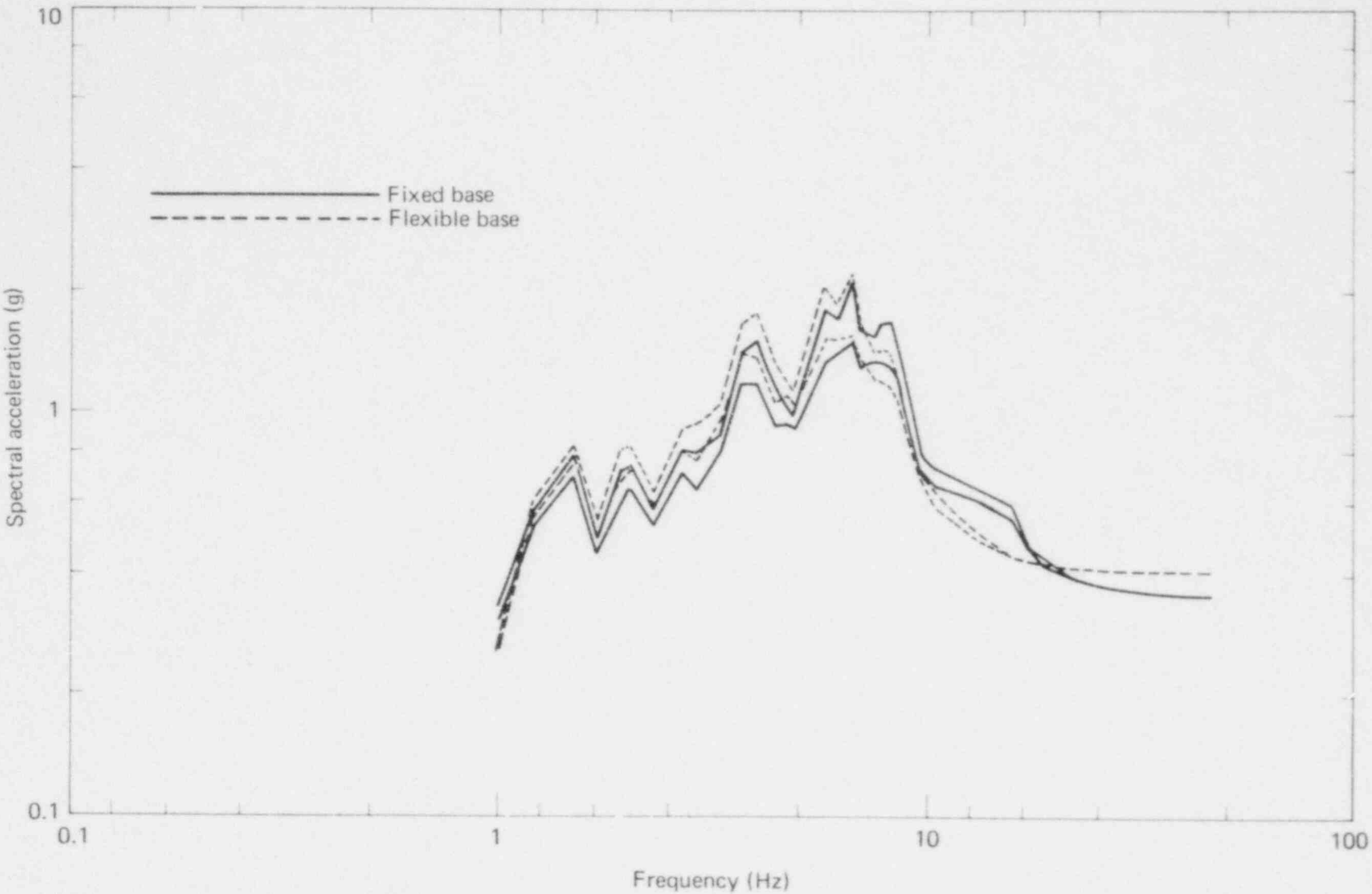


Fig. 22. Response spectra at 3 and 5% of critical damping for E-W motion at El. 949 for the fixed-base (solid) and flexible-base (dashed) analyses of the model in Fig. 21.

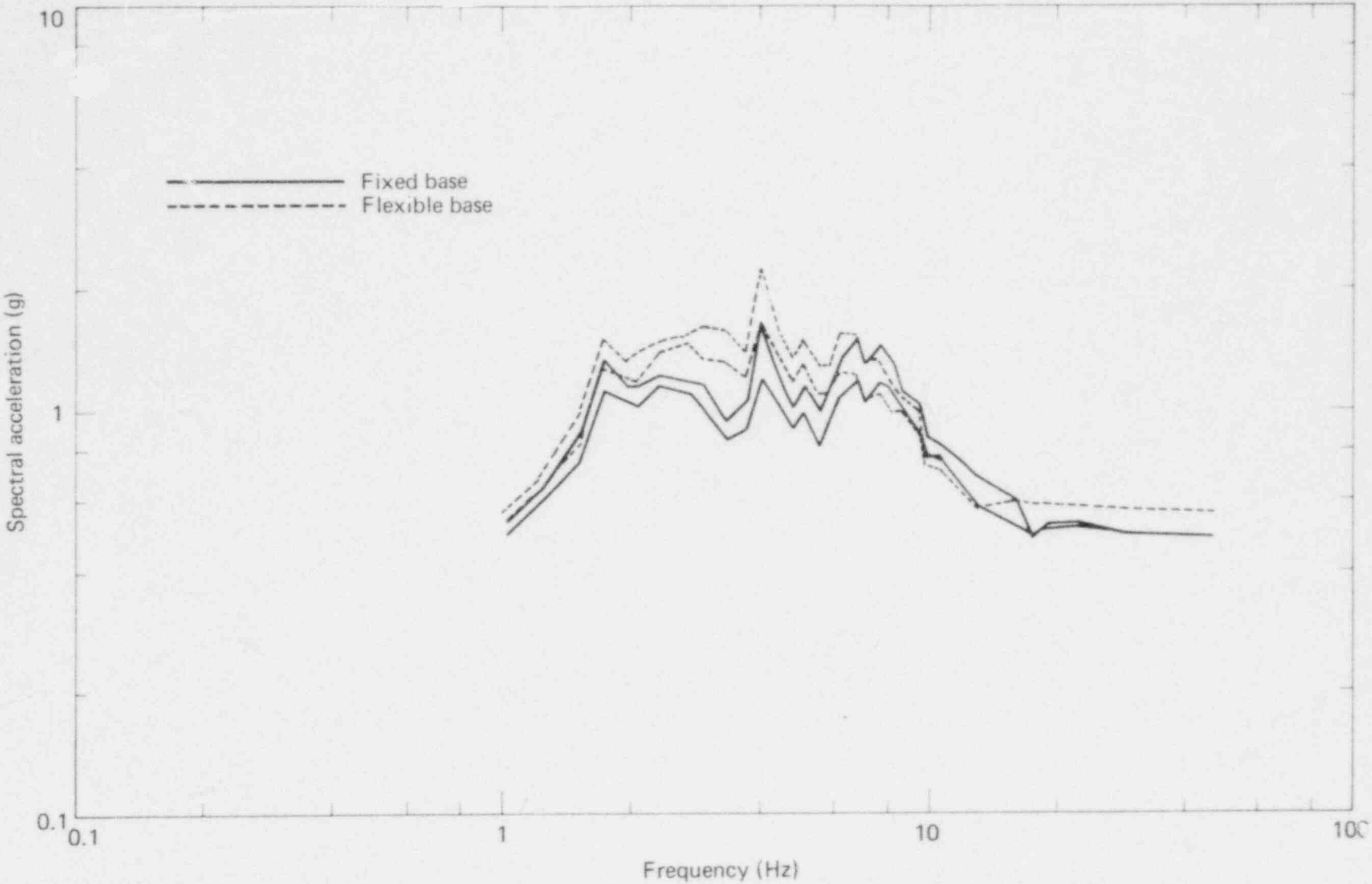


Fig. 23. Response spectra at 3 and 5% of critical damping for N-S motion at El. 949 for the fixed-base (solid) and flexible-base (dashed) analyses of the model in Fig. 21.

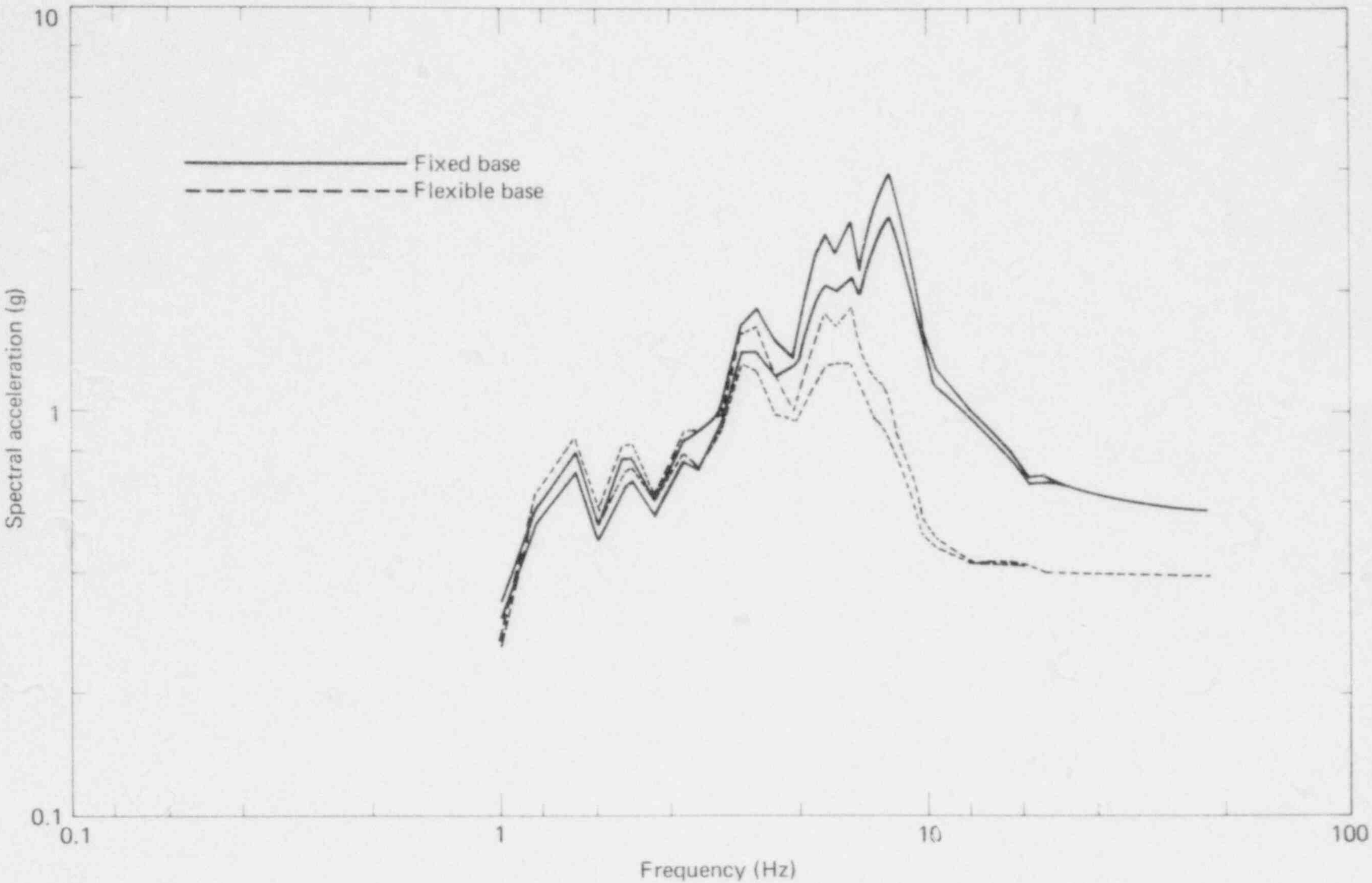


Fig. 24. Response spectra at 3 and 5% of critical damping for E-W motion at El. 969 for the fixed-base (solid) and flexible-base (dashed) analyses of the model in Fig. 21.

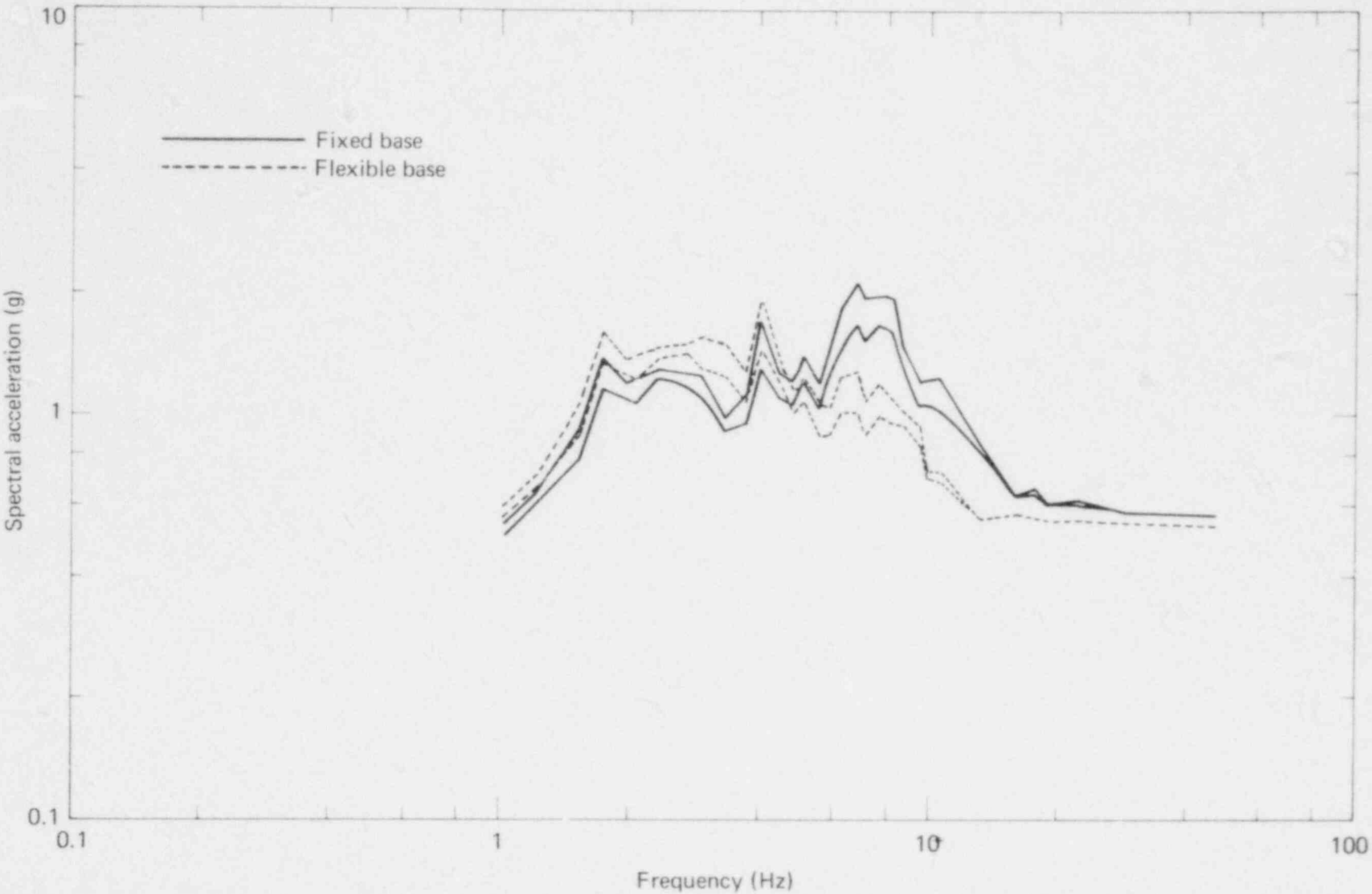


Fig. 25. Response spectra at 3 and 5% of critical damping for N-S motion at El. 969 for the fixed-base (solid) and flexible-base (dashed) analyses of the model in Fig. 21.

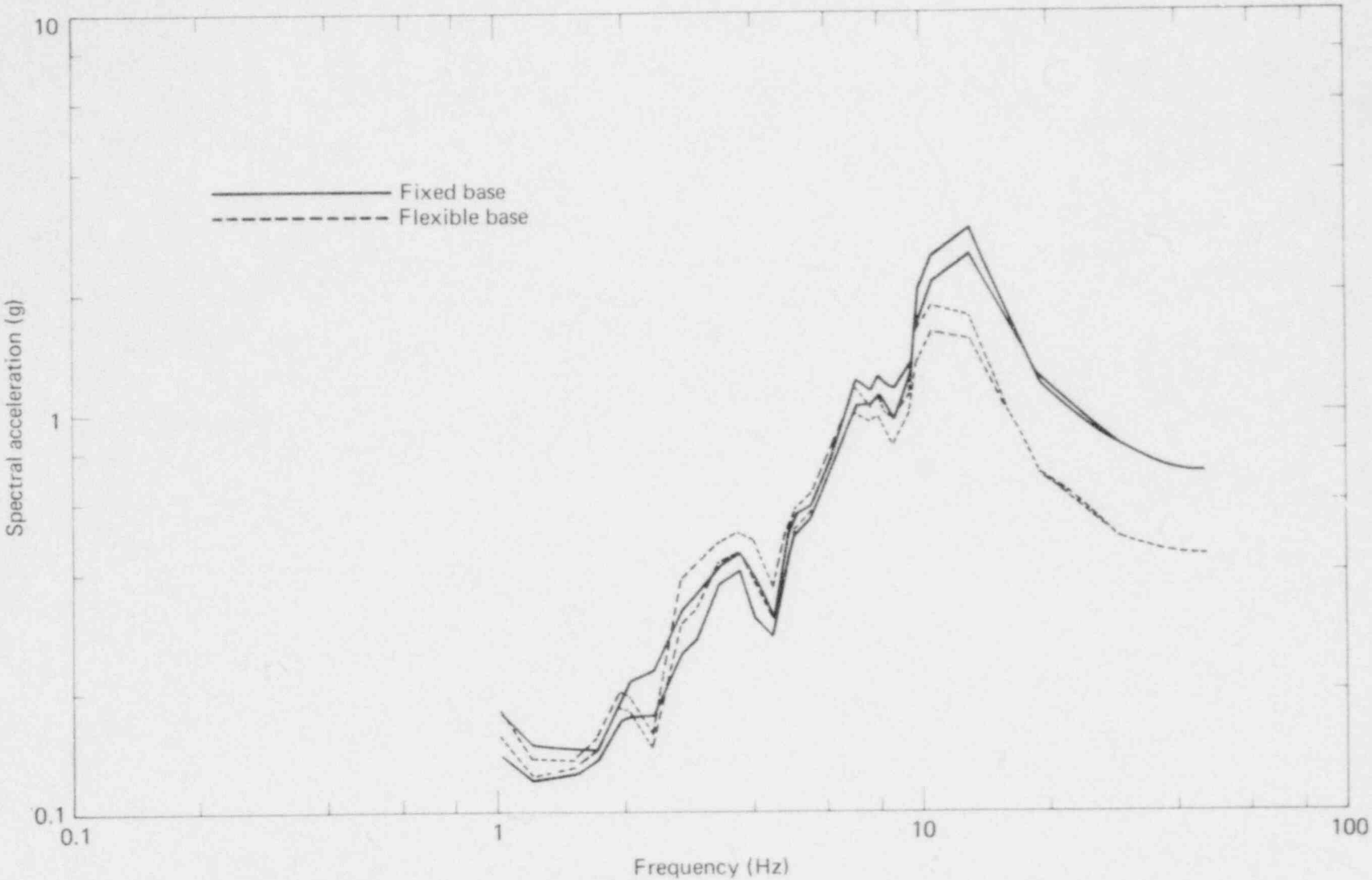


Fig. 26. Vertical response spectra at 3 and 5% of critical damping for El. 949 for the fixed-base (solid) and flexible-base (dashed) analyses of the model in Fig. 21.

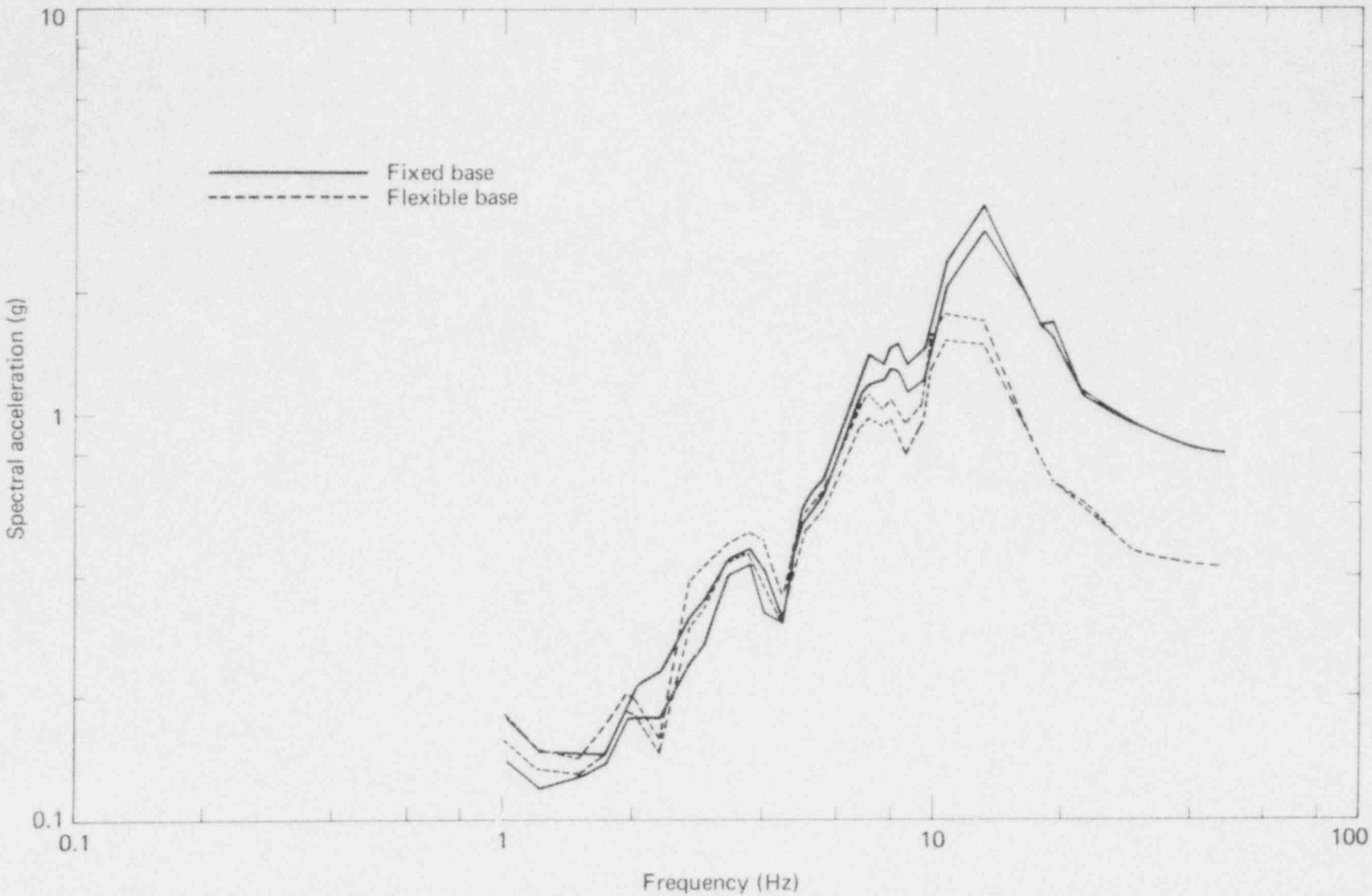


Fig. 27. Vertical response spectra at 3 and 5% of critical damping for El. 969 for the fixed-base (solid) and flexible-base (dashed) analyses of the model in Fig. 21.

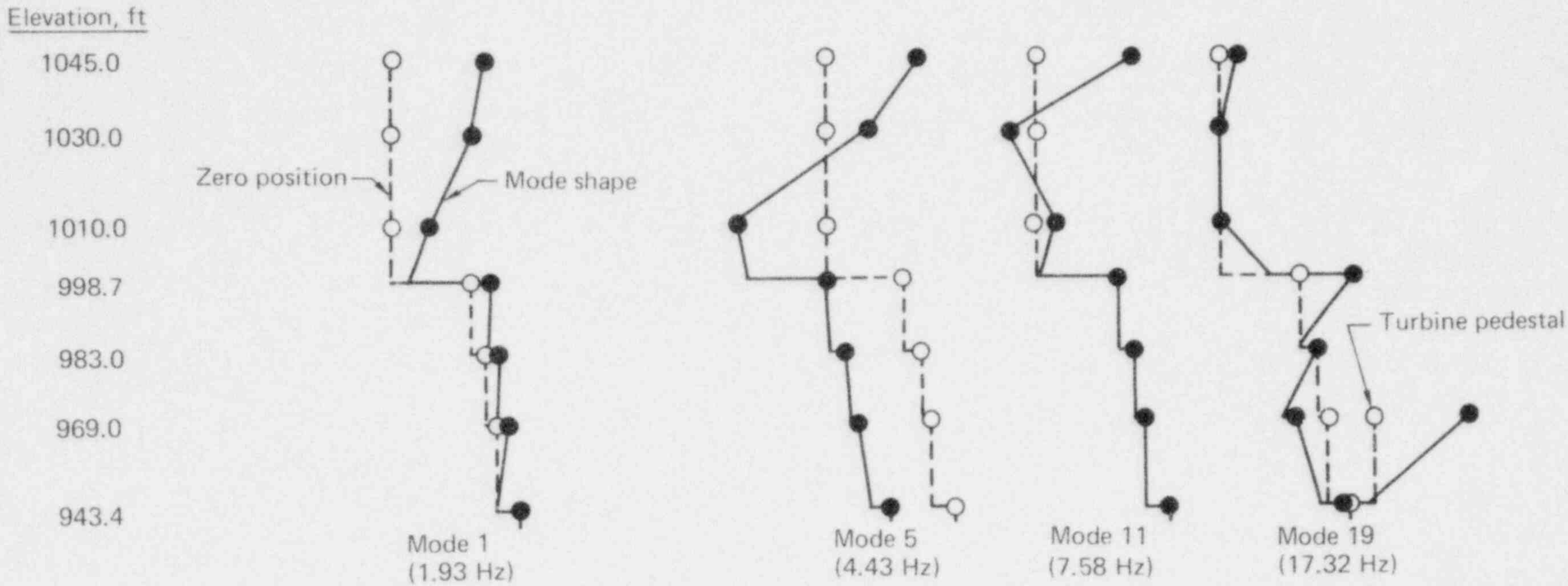


Fig. 28. Representative mode shapes and frequencies for E-W response of the model shown in Fig. 21. Note that the first mode shows the motion of the top portion of the boiler structure relative to the lower portion of the turbine building. This is in contrast to the 5th mode, for which the relative motion is between the soil springs and the upper tower while the lower turbine portion remains essentially rigid.

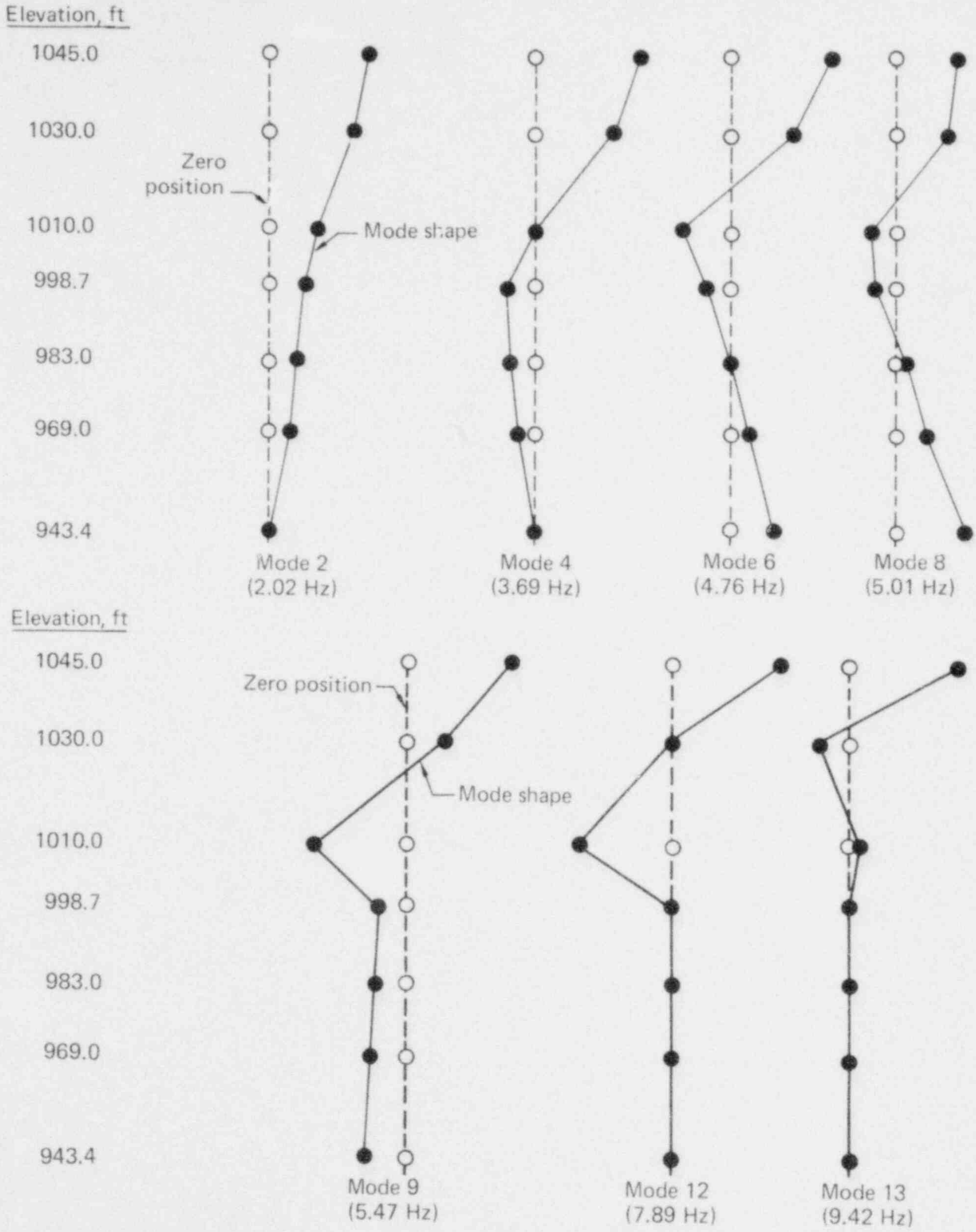


Fig. 29. Representative mode shapes and frequencies for N-S response of the model shown in Fig. 21. Note that modes 6, 8 and 9 are largely controlled by soil translation.

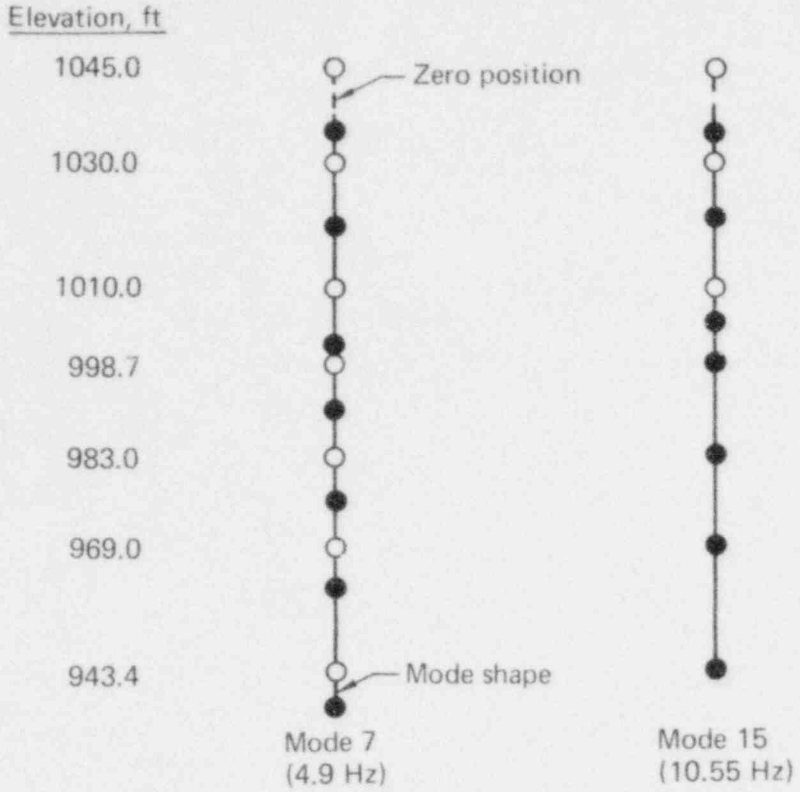


Fig. 30. Representative mode shapes and frequencies for vertical response of the model shown in Fig. 21. Mode 7 is the dominant vertical mode, and at 4.9 Hz indicates a significant amount of soil translation. The 15th mode primarily results from vertical vibration of the upper boiler tower.

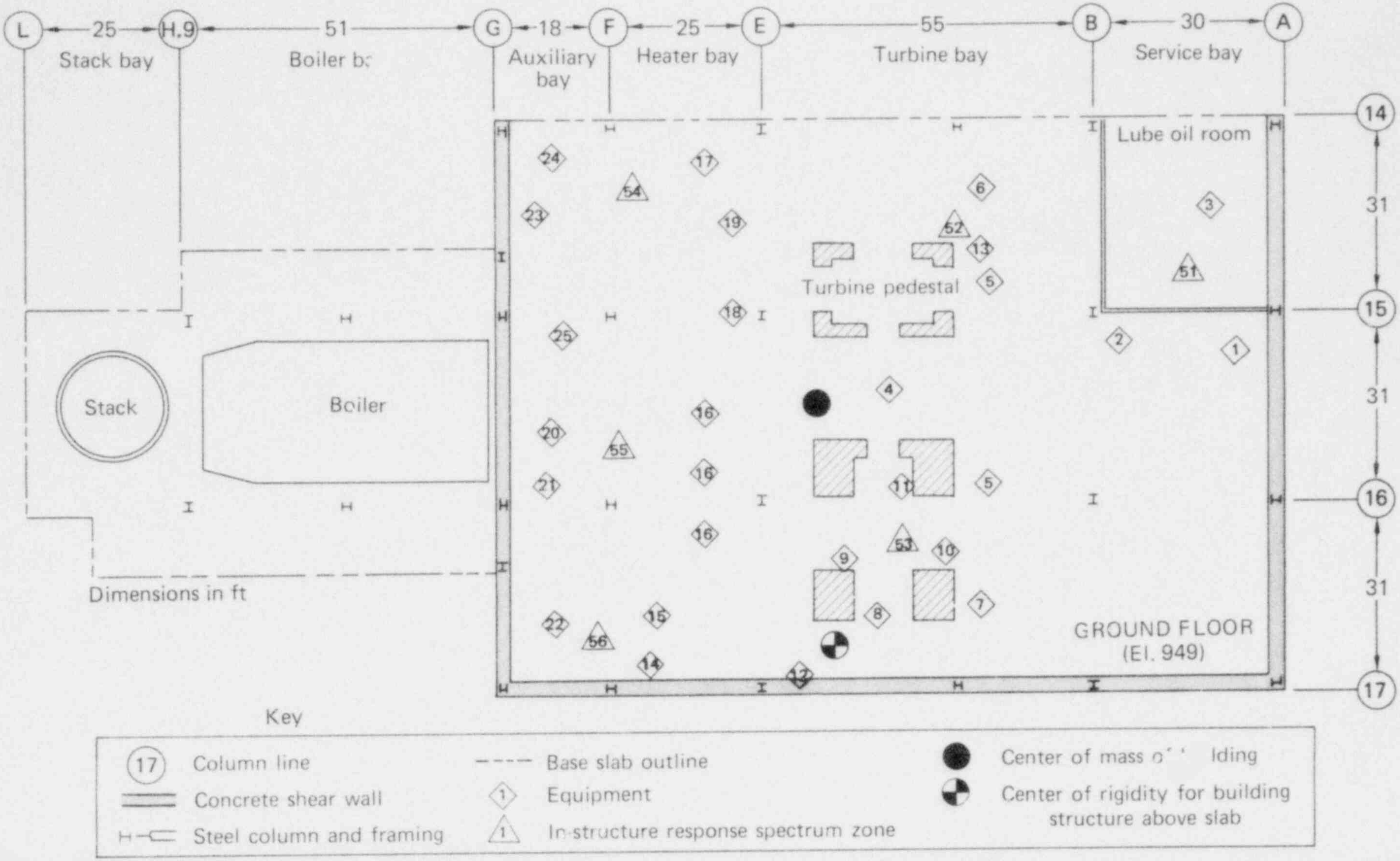


Fig. 31. Schematic plan view of the Unit 4 ground floor (El. 949 ft) shows shear walls, column lines, equipment locations, and locations where in-structure response spectra were generated. See Table 11 for the identity of each equipment item.

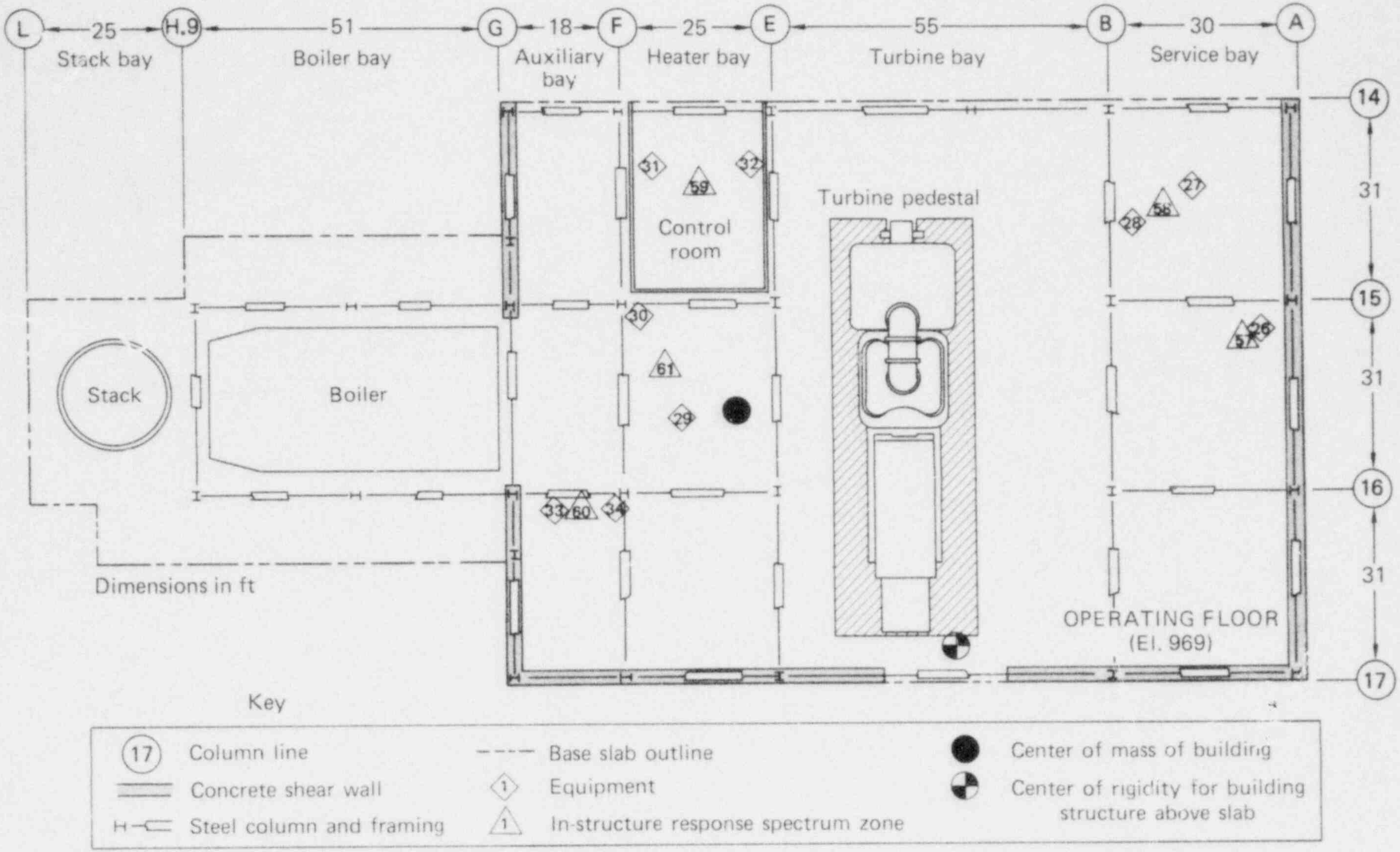


Fig. 32. Schematic plan view of the Unit 4 operating floor (El. 969 ft) shows shear walls, column lines, equipment locations, and locations where in-structure response spectra were generated. See Table 11 for the identity of each equipment item.

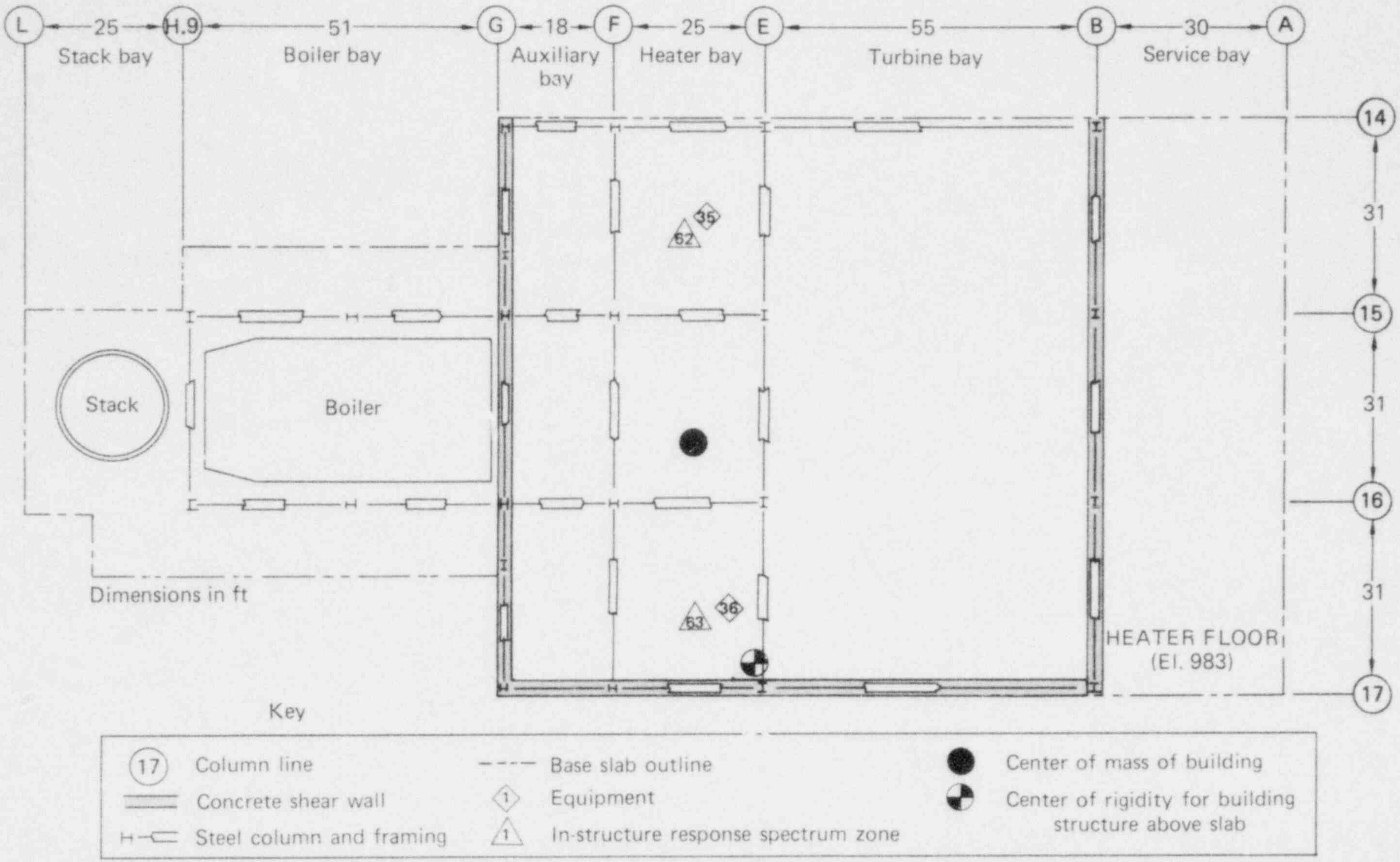


Fig. 33. Schematic plan view of the Unit 4 heater floor (El. 983 ft) shows shear walls, column lines, equipment locations, and locations where in-structure response spectra were generated. See Table 11 for the identity of each equipment item.

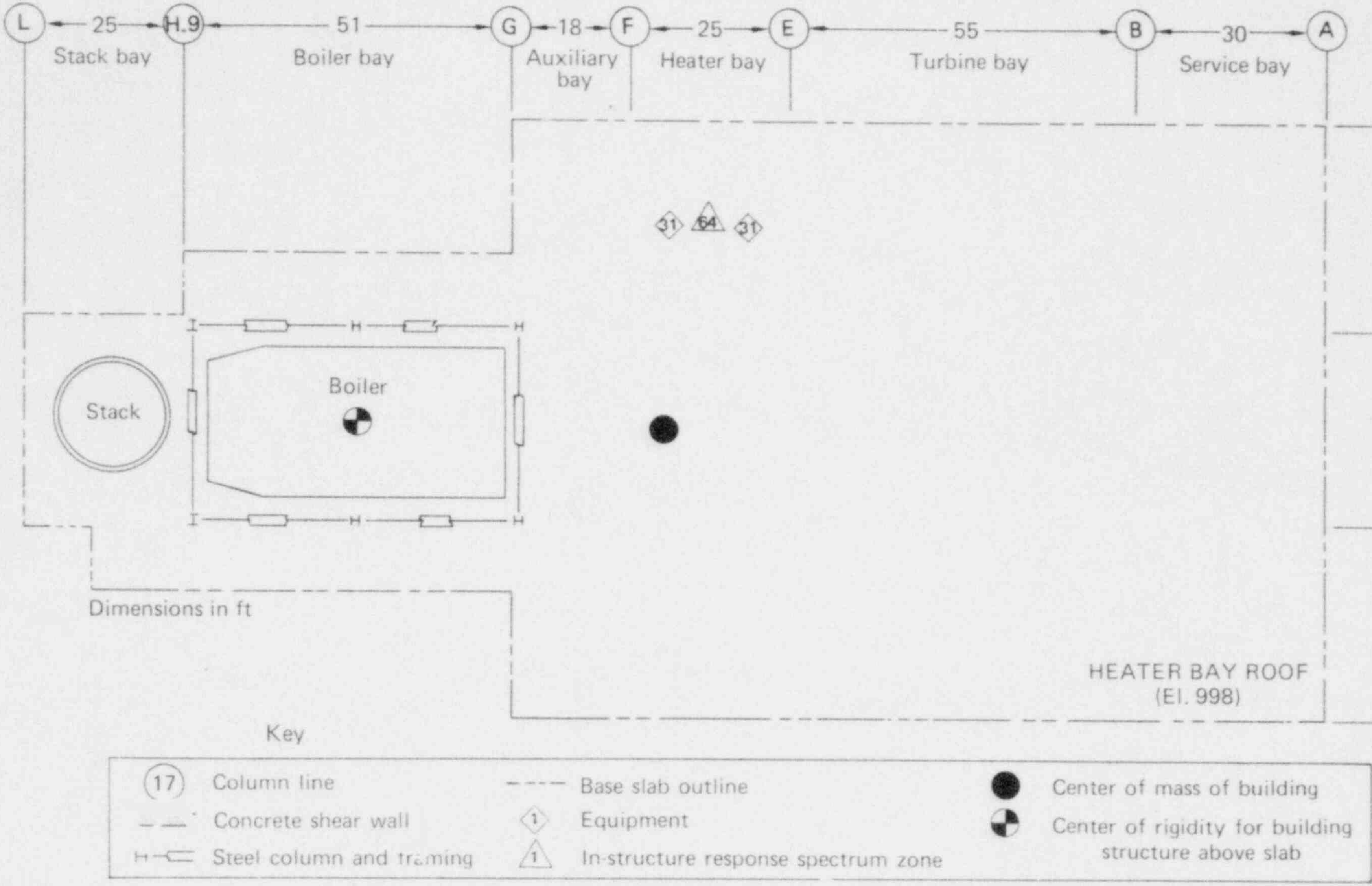


Fig. 34. Schematic plan view of the Unit 4 heater bay roof (El. 998 ft) shows shear walls, column lines, equipment locations, and locations where in-structure response spectra were generated. See Table 11 for the identity of each equipment item.

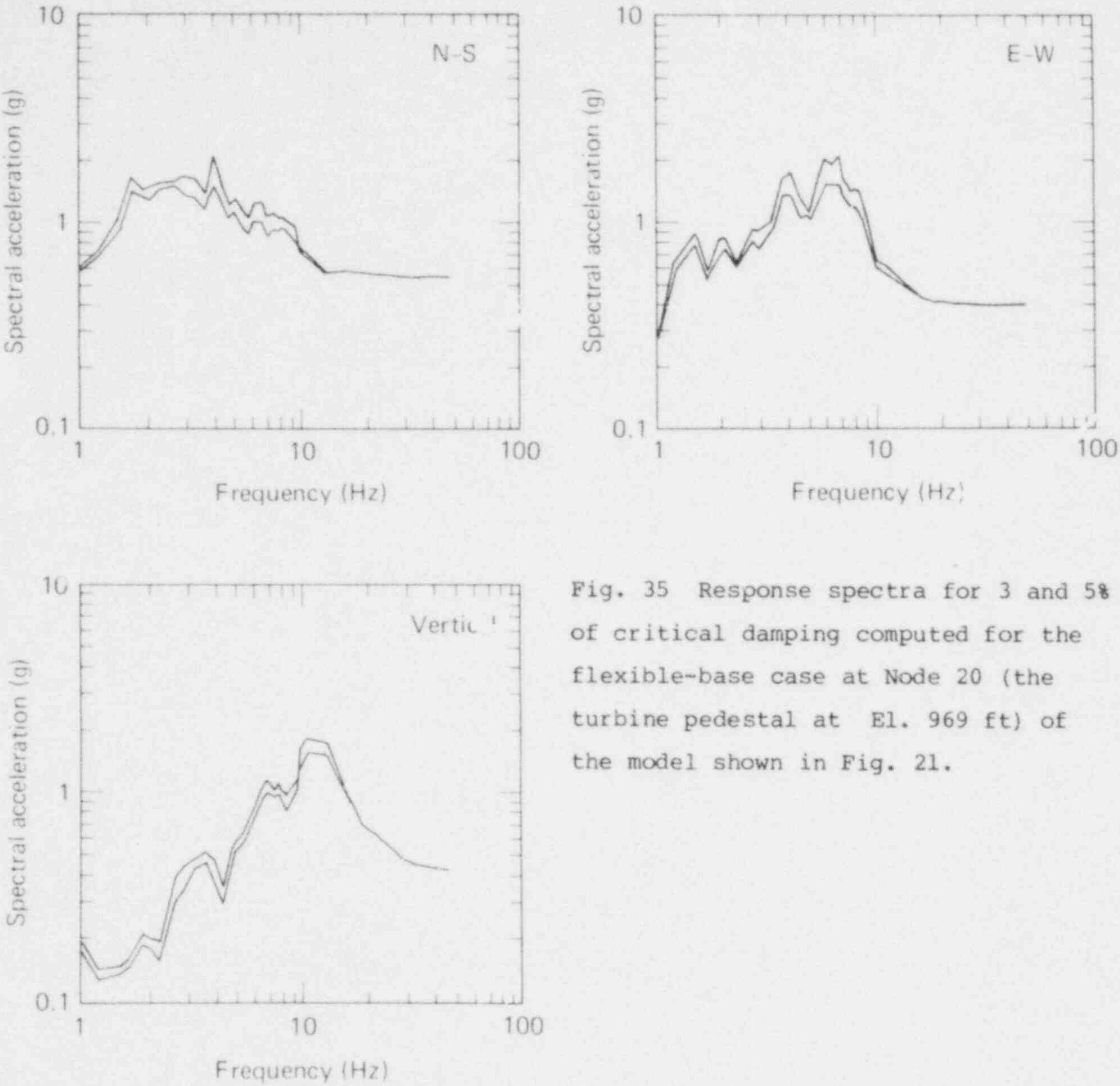
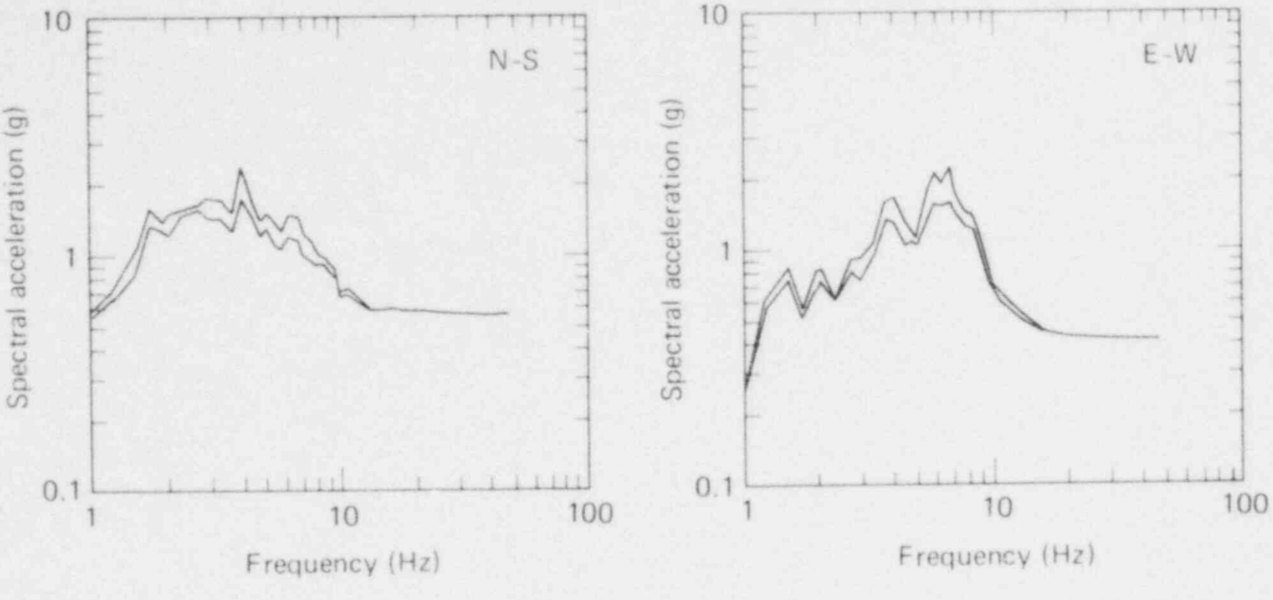


Fig. 35 Response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 20 (the turbine pedestal at El. 969 ft) of the model shown in Fig. 21.



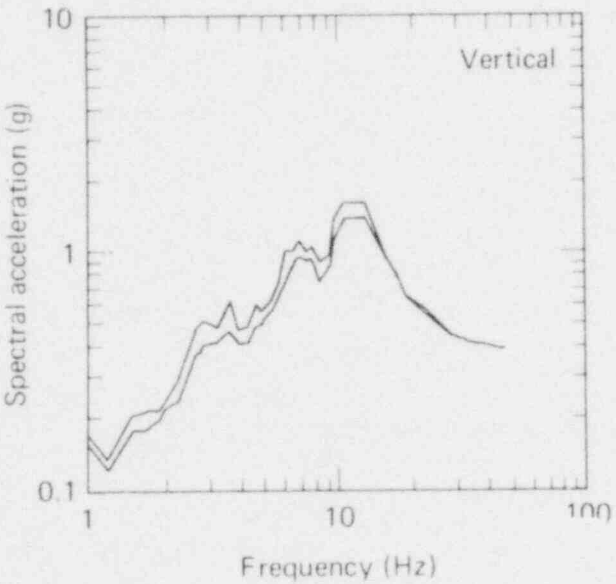
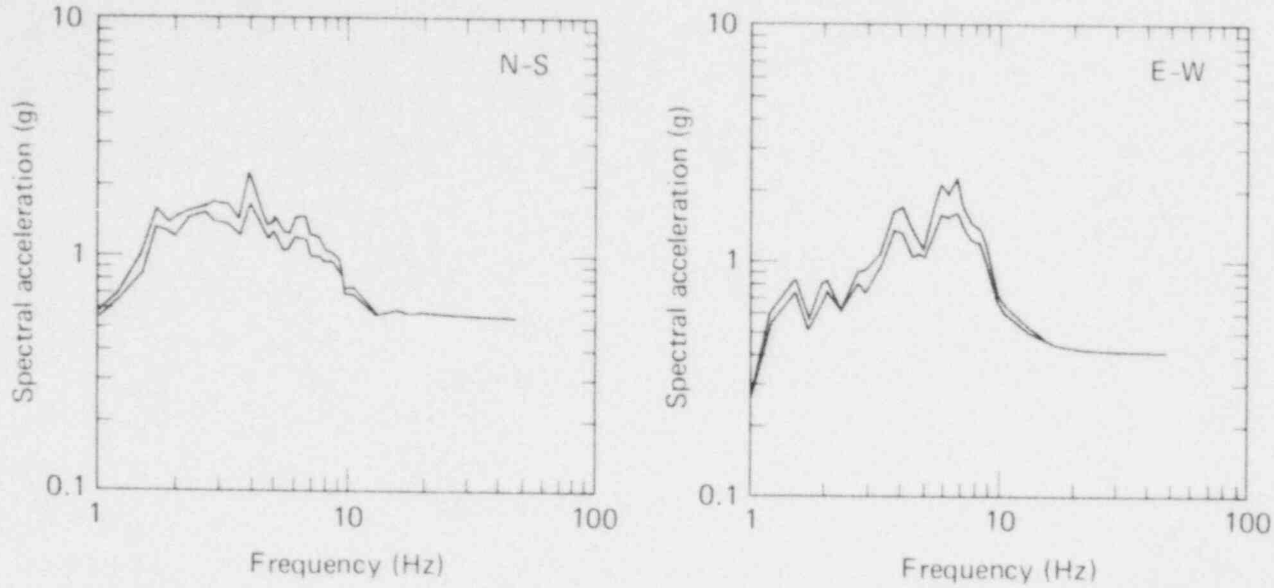


Fig. 36 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 51 (El. 949 ft) of the model shown in Fig. 21.



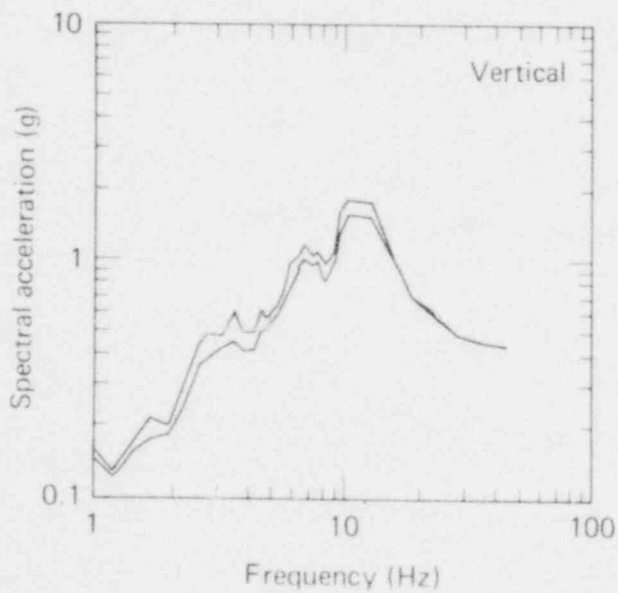


Fig. 37 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 52 (El. 949 ft) of the model shown in Fig. 21.

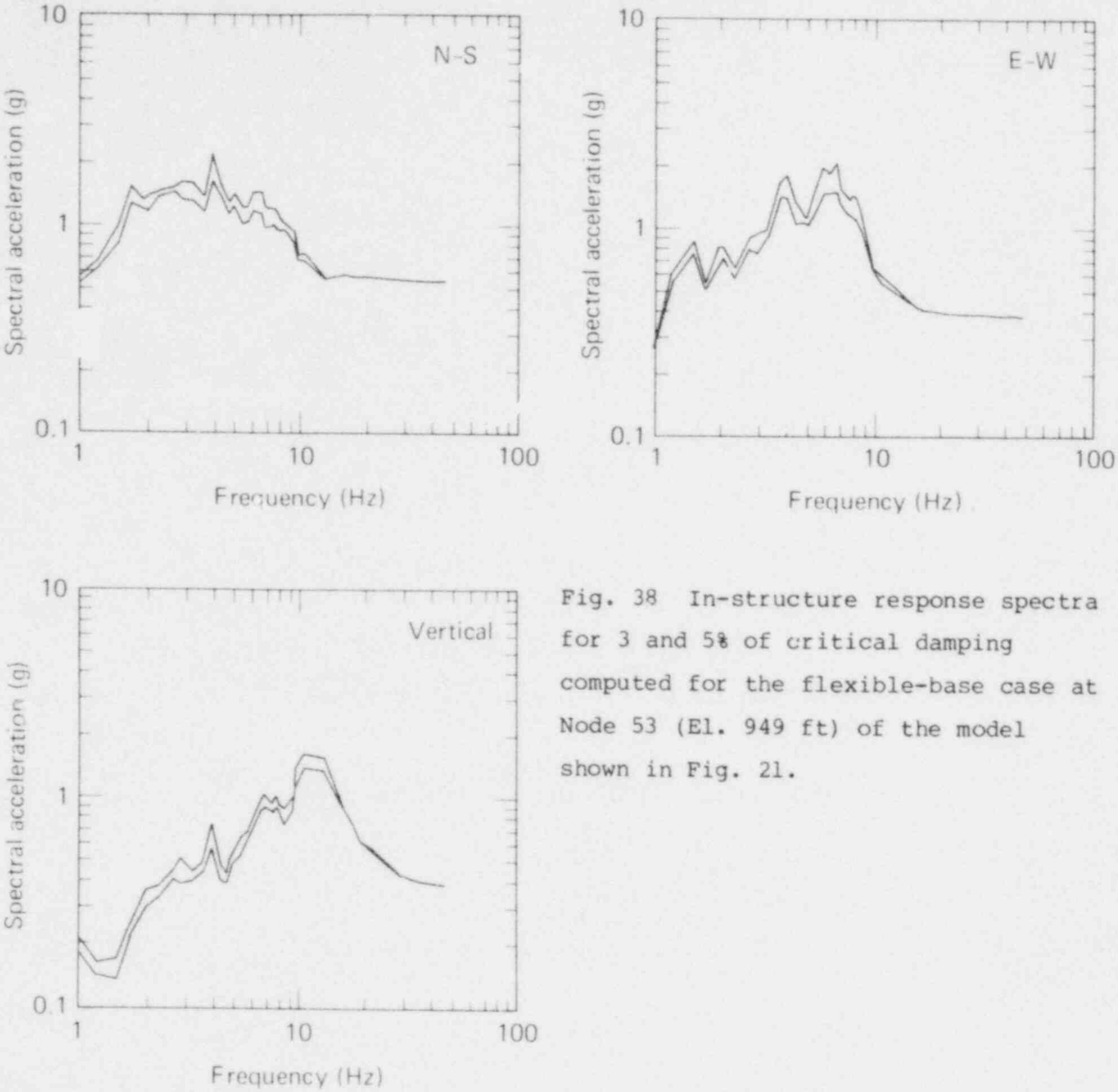


Fig. 38 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 53 (El. 949 ft) of the model shown in Fig. 21.

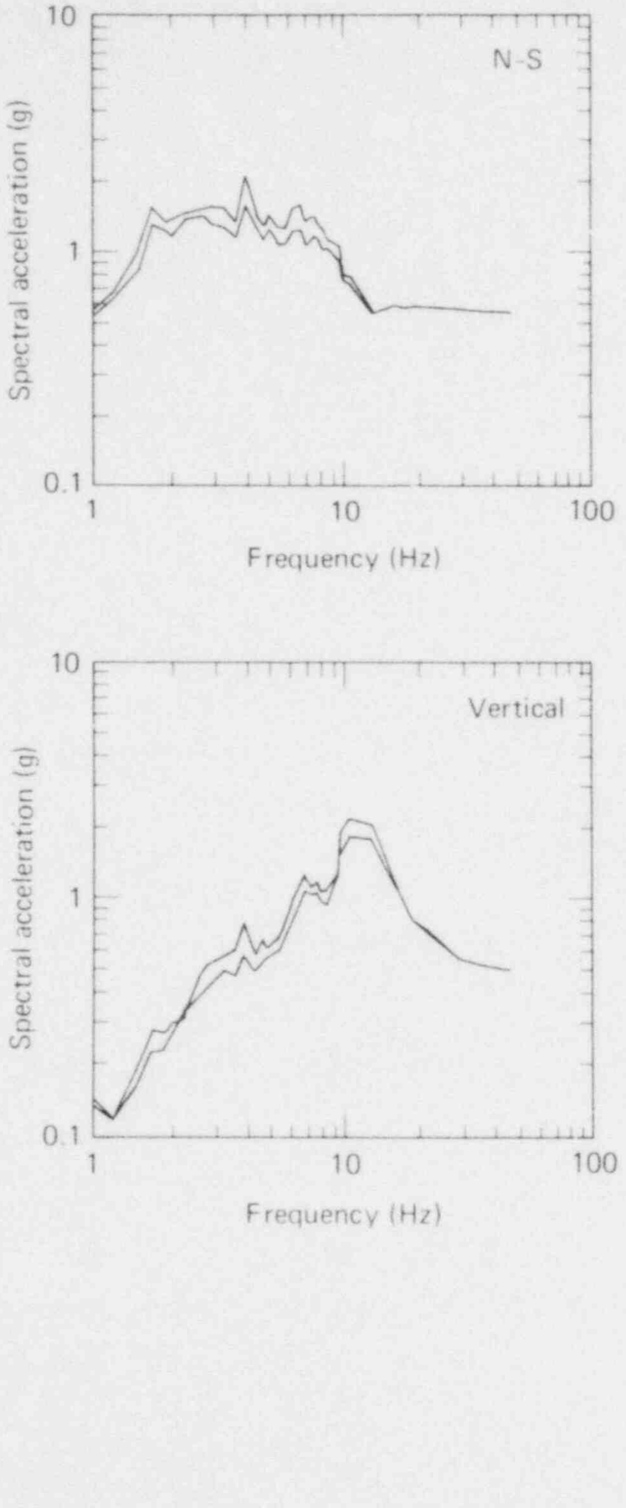
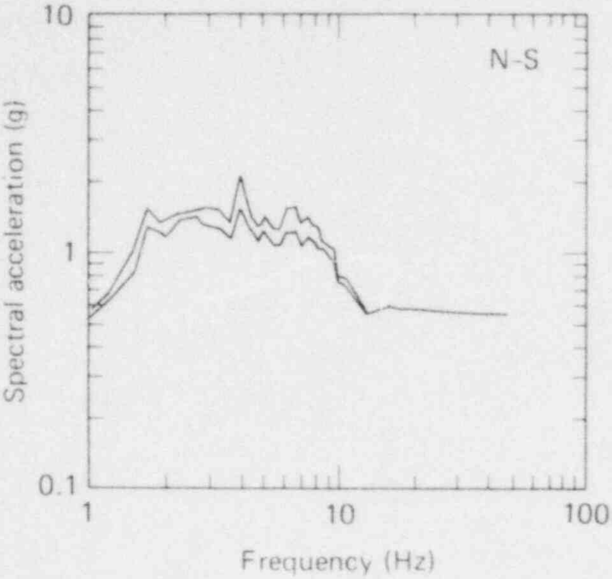
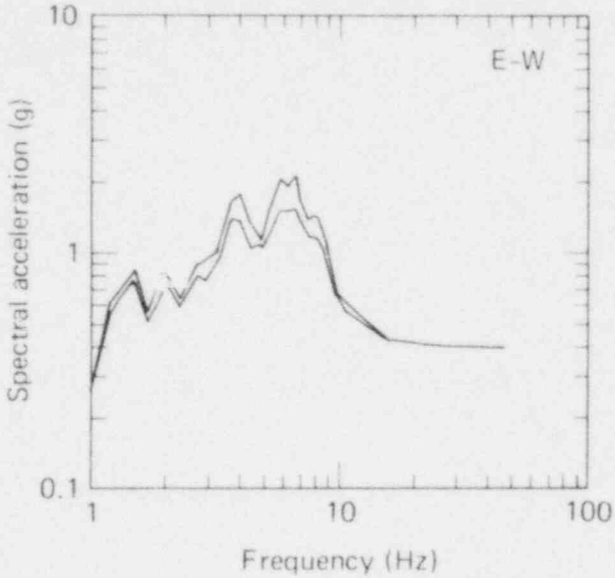


Fig. 39 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 54 (El. 949 ft) of the model shown in Fig. 21.





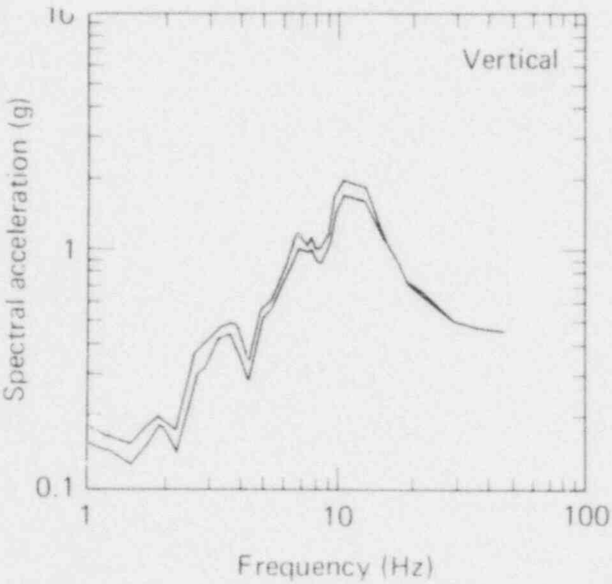
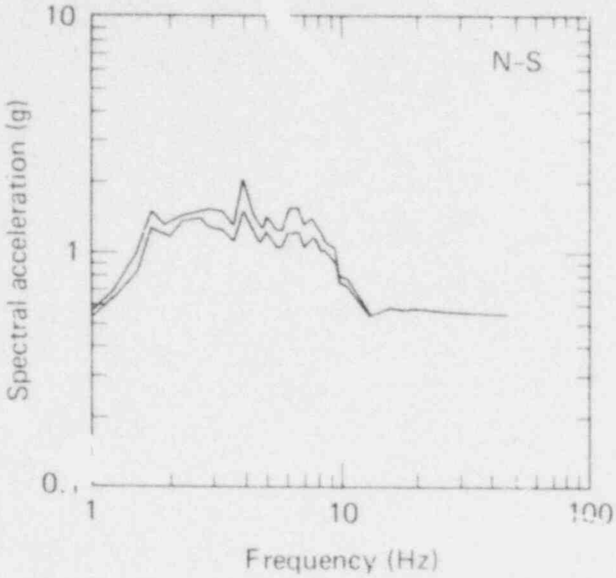
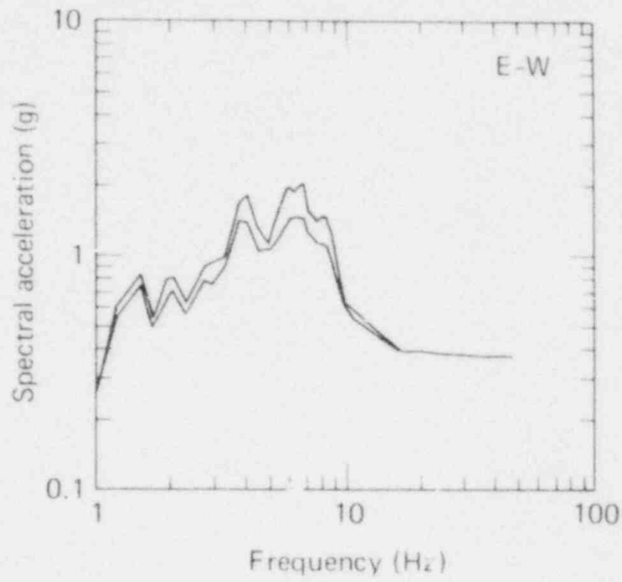


Fig. 40 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 55 (El. 949 ft) of the model shown in Fig. 21.





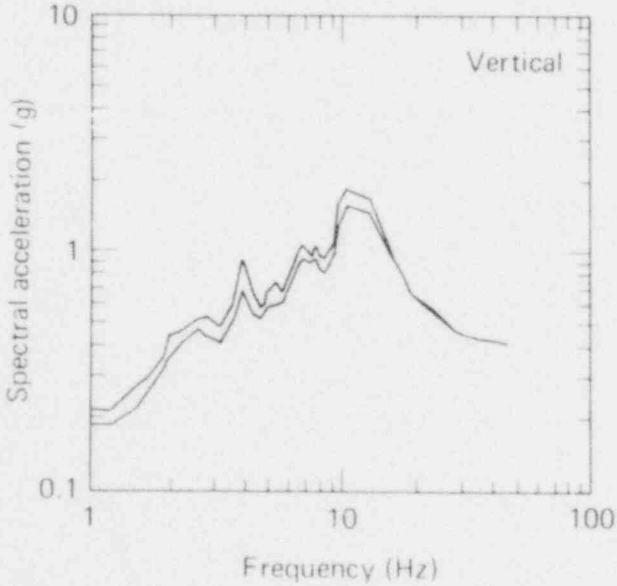


Fig. 41 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 56 (El. 949 ft) of the model shown in Fig. 21.

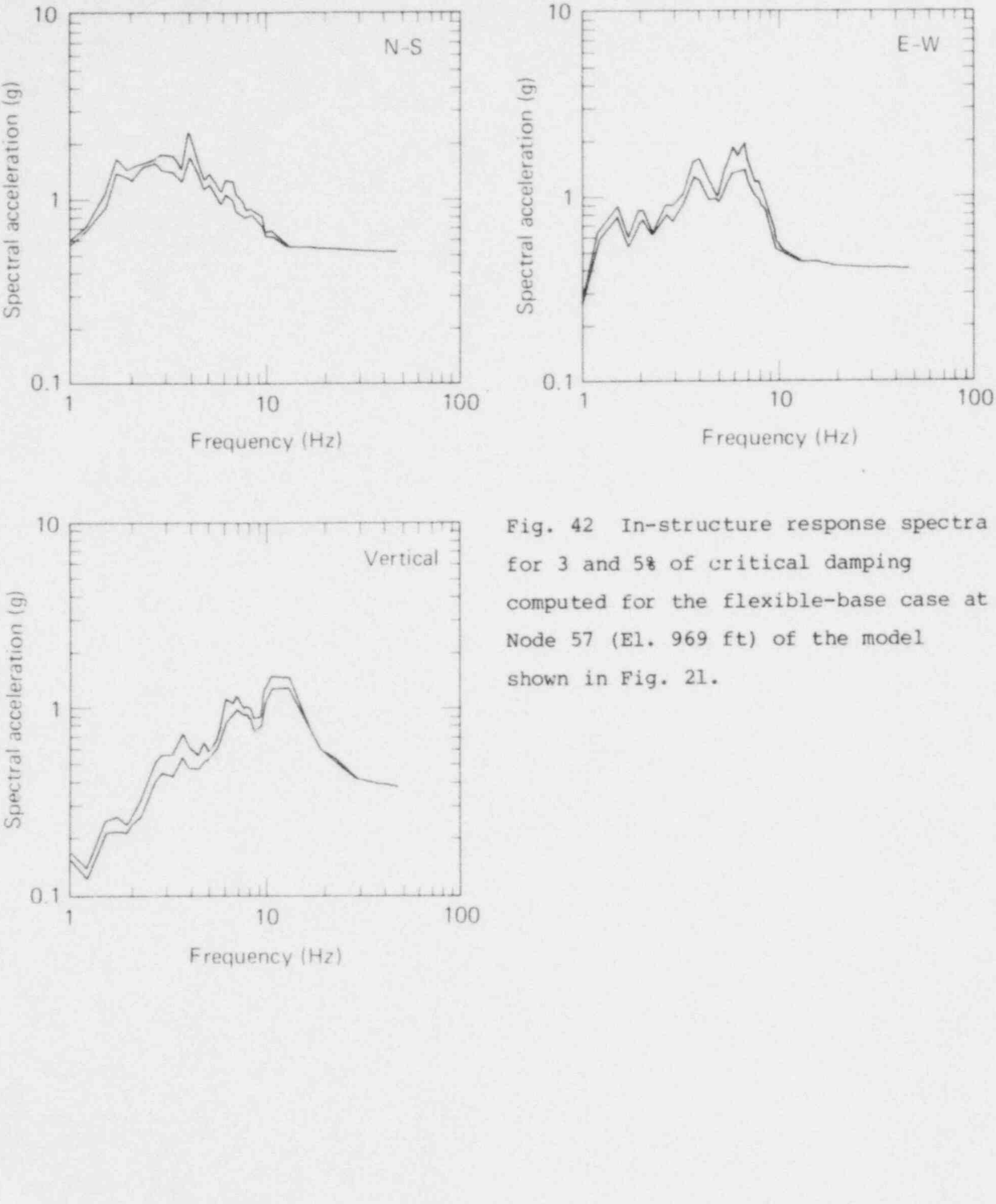
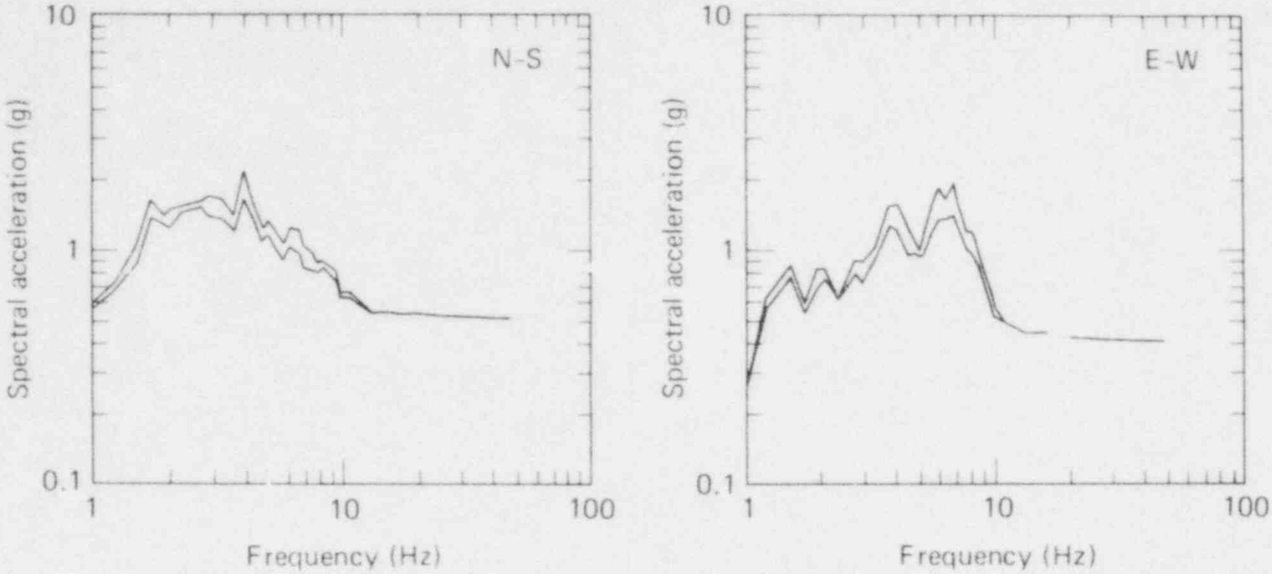


Fig. 42 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 57 (El. 969 ft) of the model shown in Fig. 21.



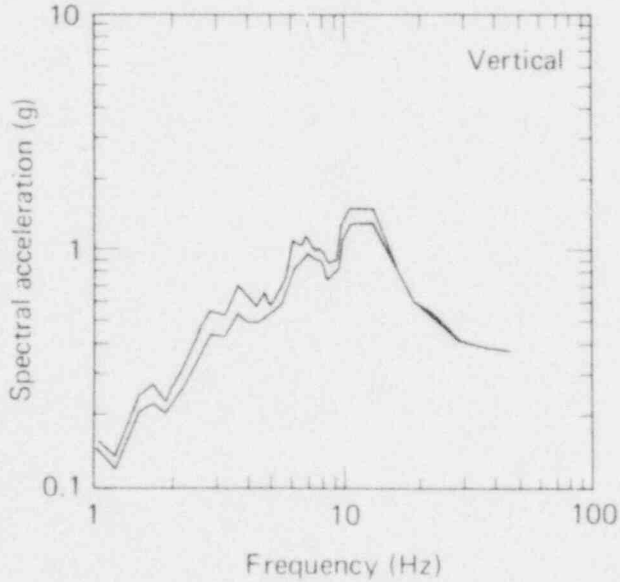
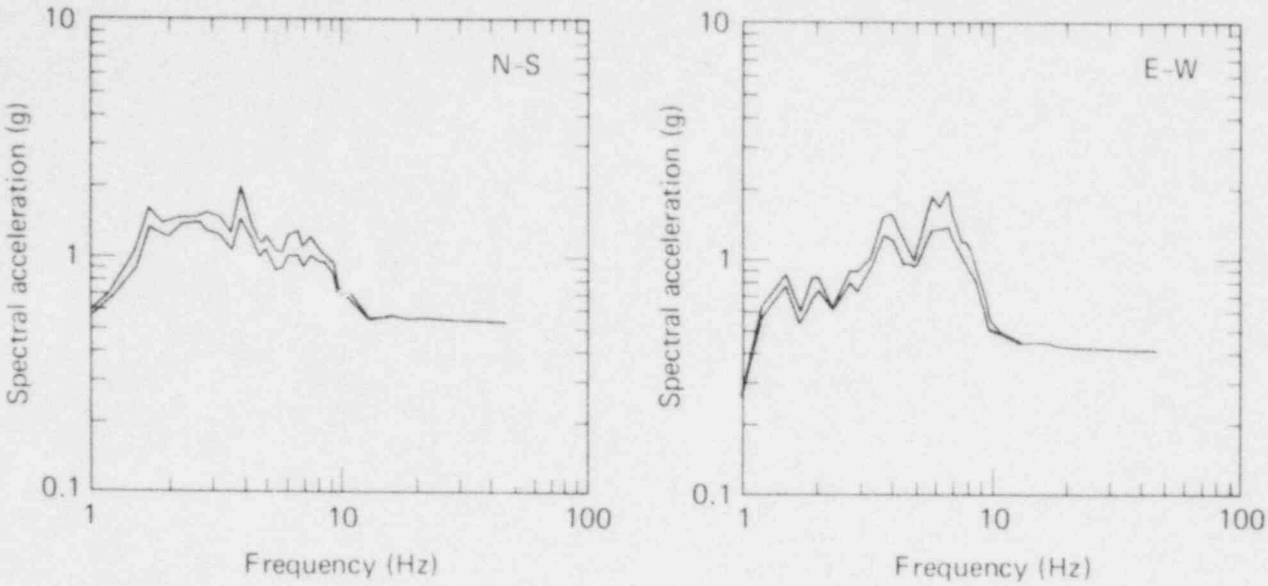


Fig. 43 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Ncde 58 (El. 969 ft) of the model shown in Fig. 21.



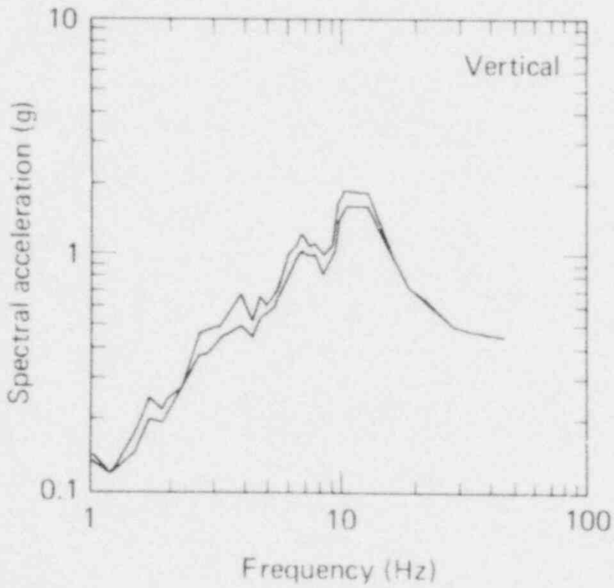
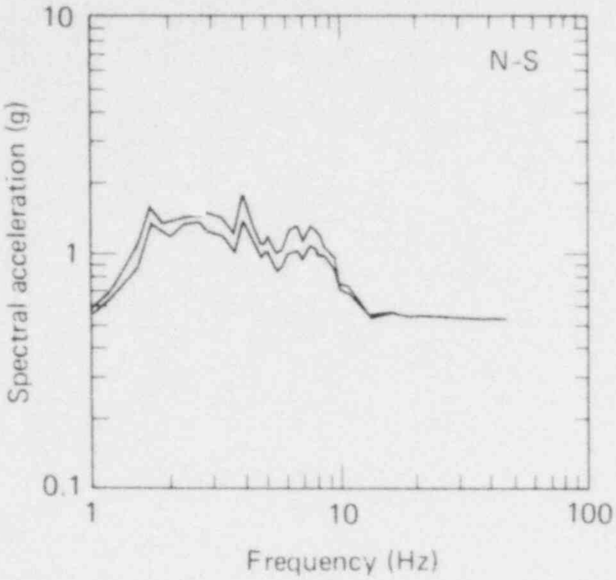
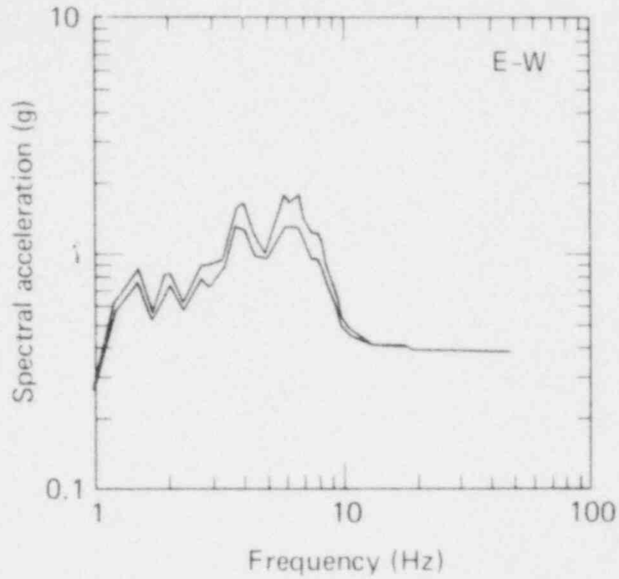


Fig. 44 In-structure response spectra for 3 and 5% of critical damping computed fcr the flexible-base case at Node 59 (El. 969 ft) of the model shown in Fig. 21.





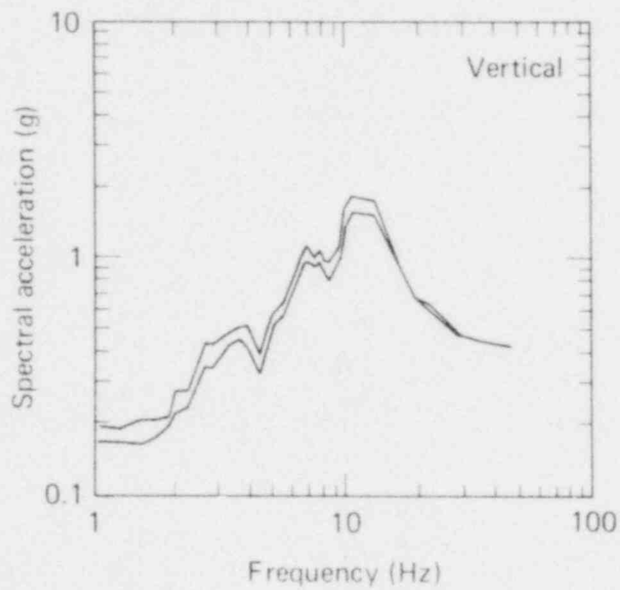


Fig. 45 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 60 (El. 969 ft) of the model shown in Fig. 21.

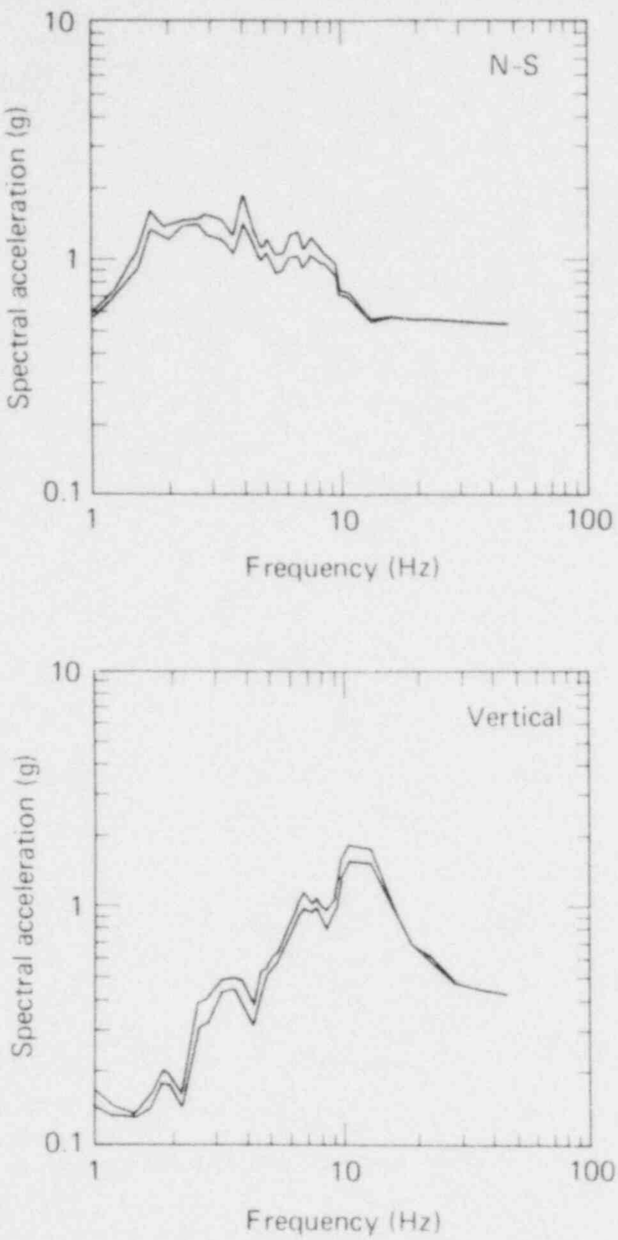
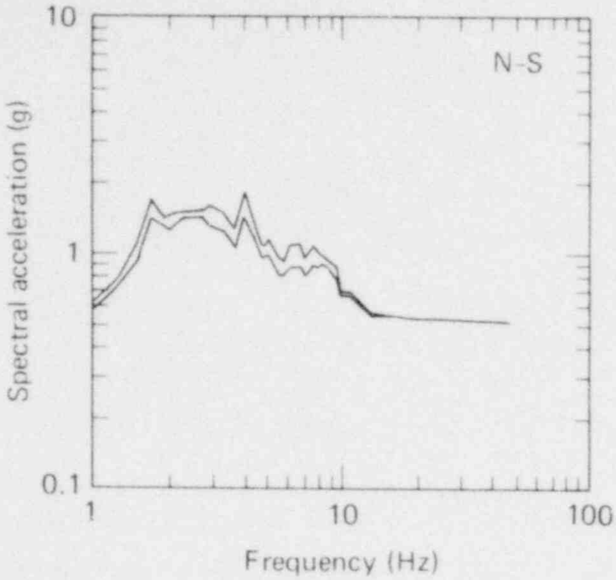
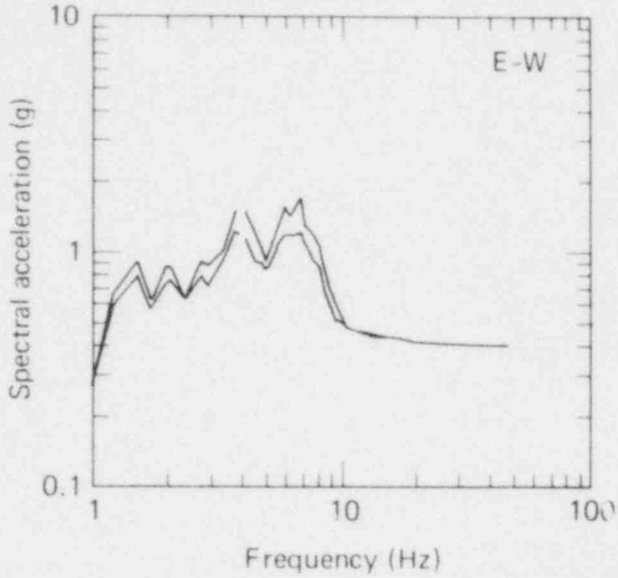


Fig. 46 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 61 (El. 969 ft) of the model shown in Fig. 21.





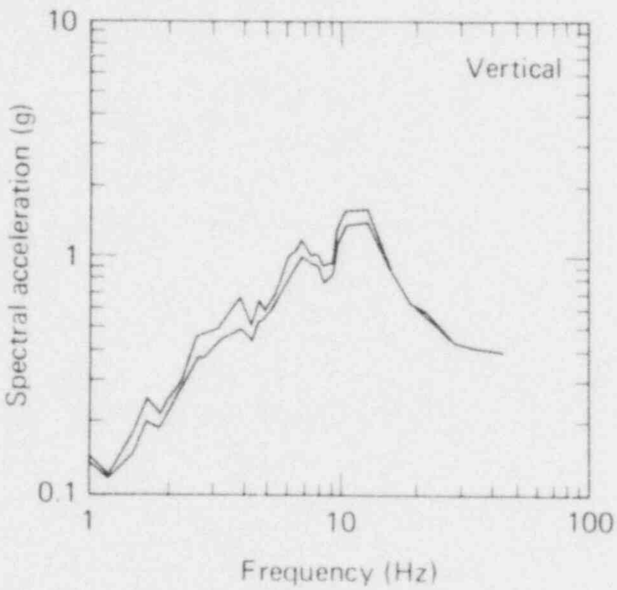


Fig. 47 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 62 (El. 983 ft) of the model shown in Fig. 21.

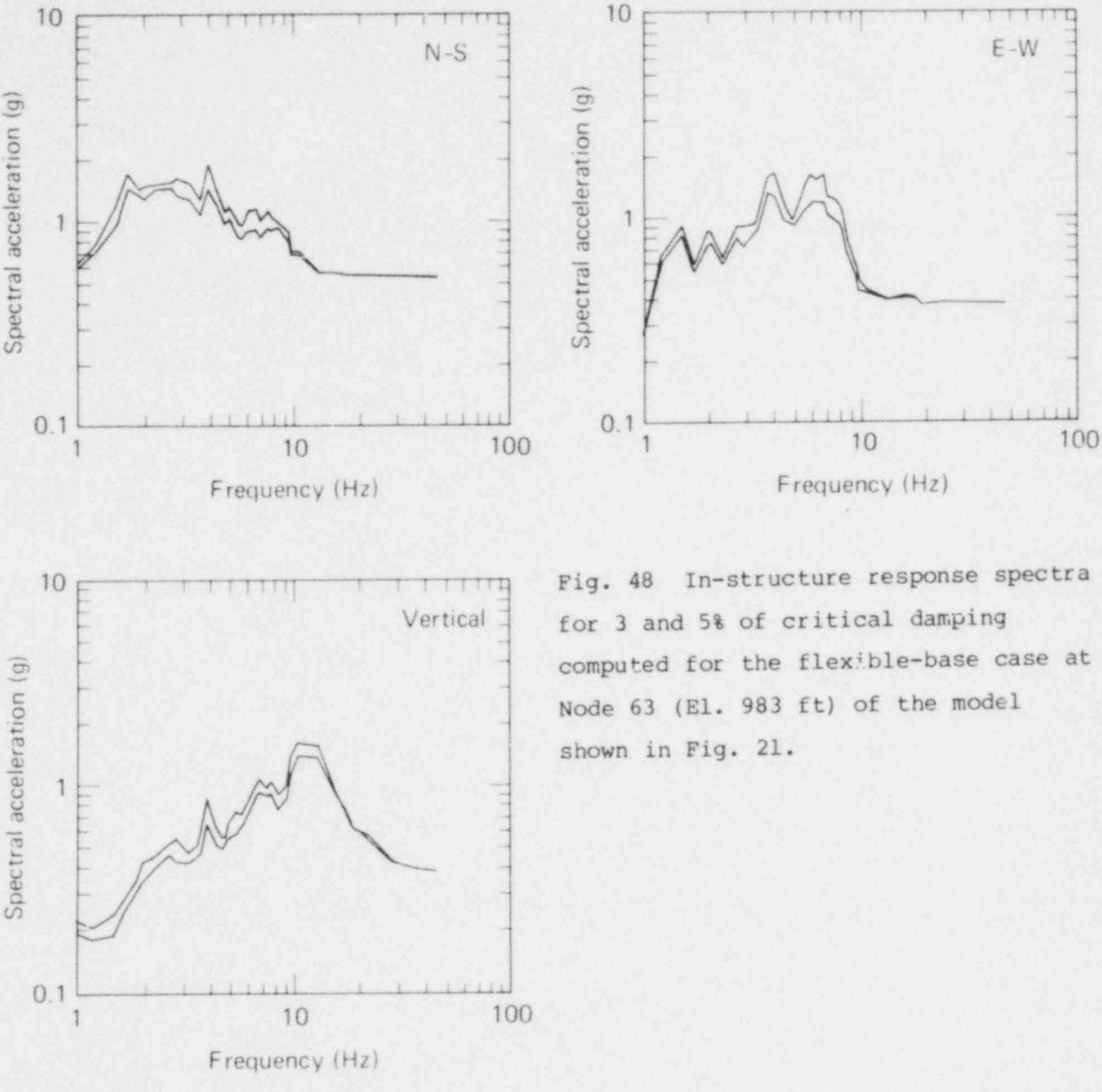


Fig. 48 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 63 (El. 983 ft) of the model shown in Fig. 21.

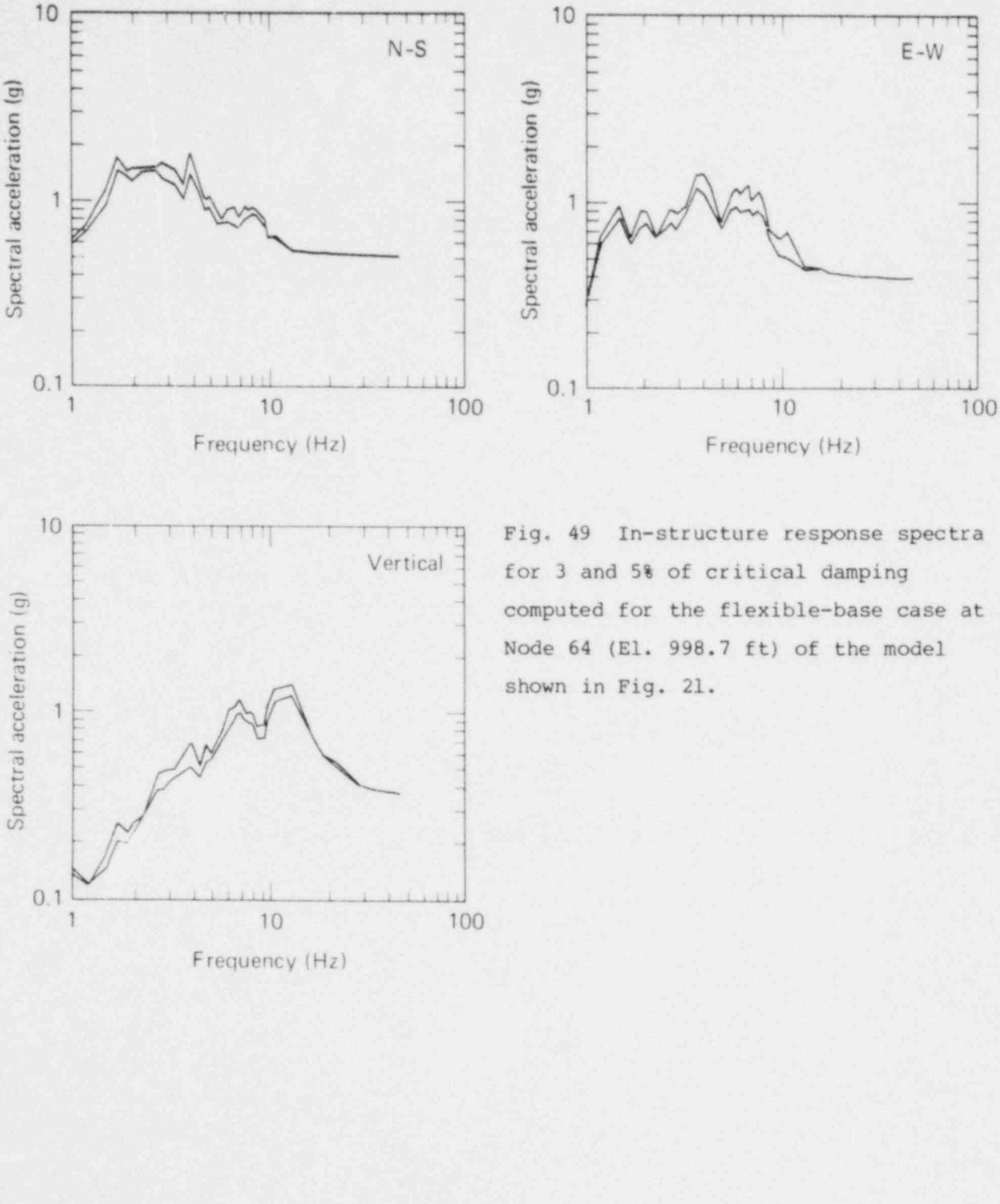


Fig. 49 In-structure response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 64 (El. 998.7 ft) of the model shown in Fig. 21.

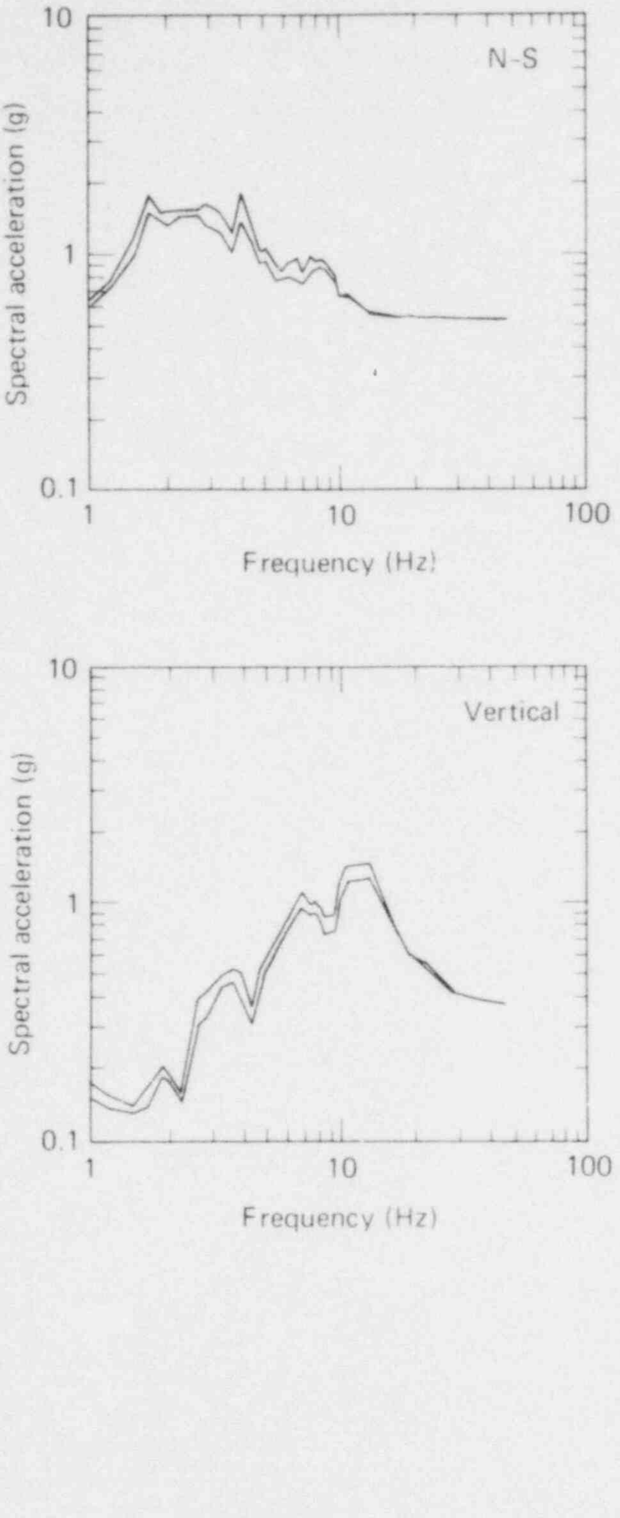
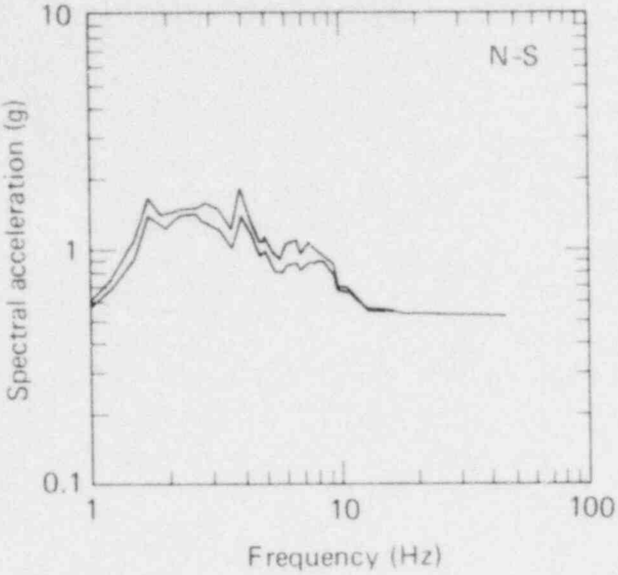
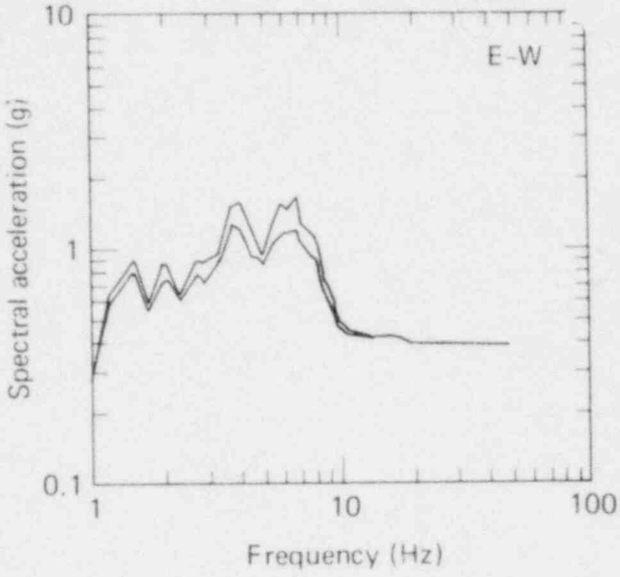


Fig. 50 Response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 106 (El. 998.7 ft) of the model shown in Fig. 21.





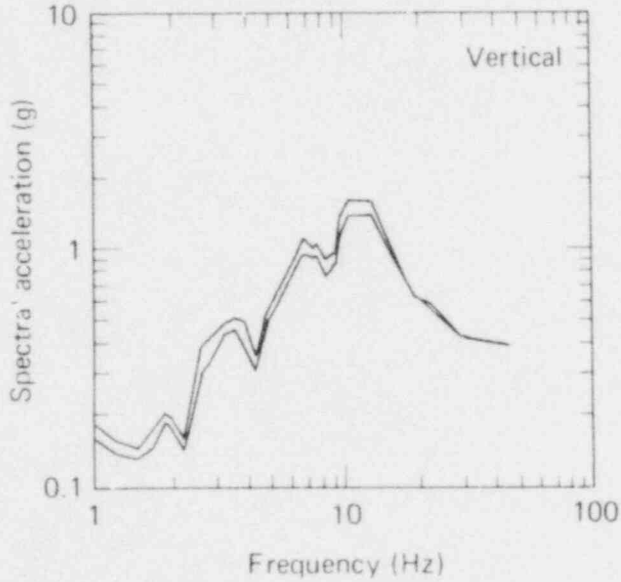
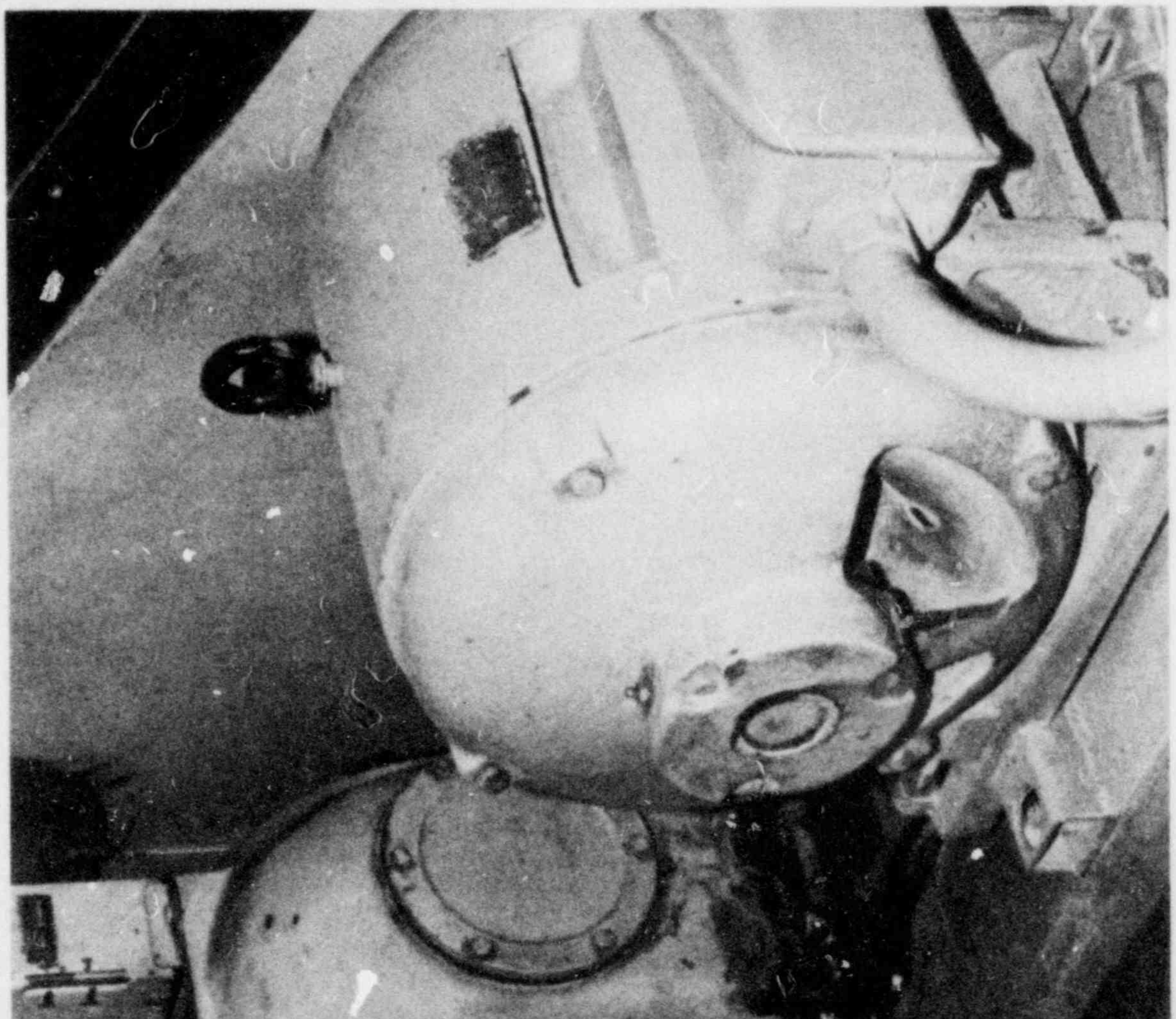
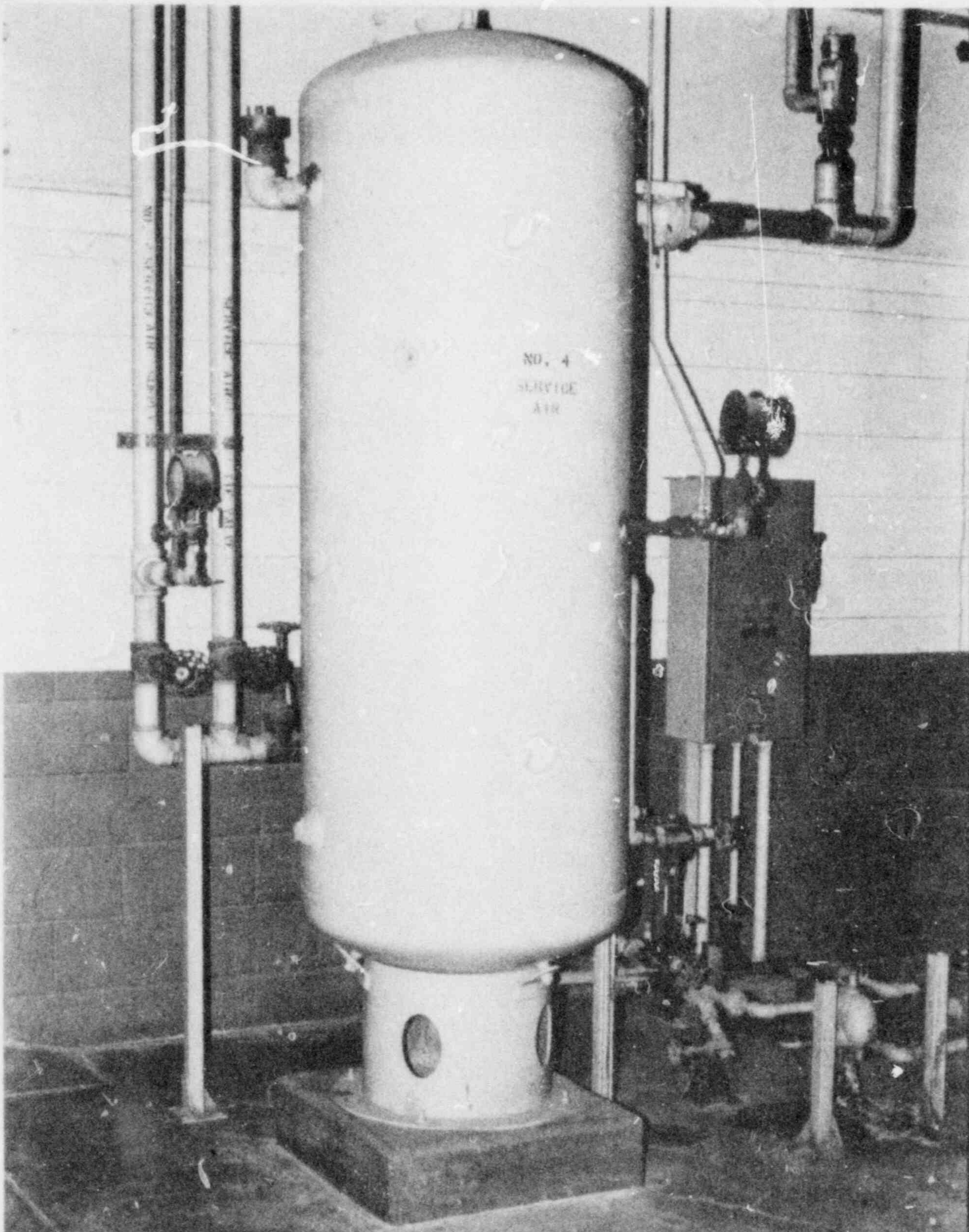


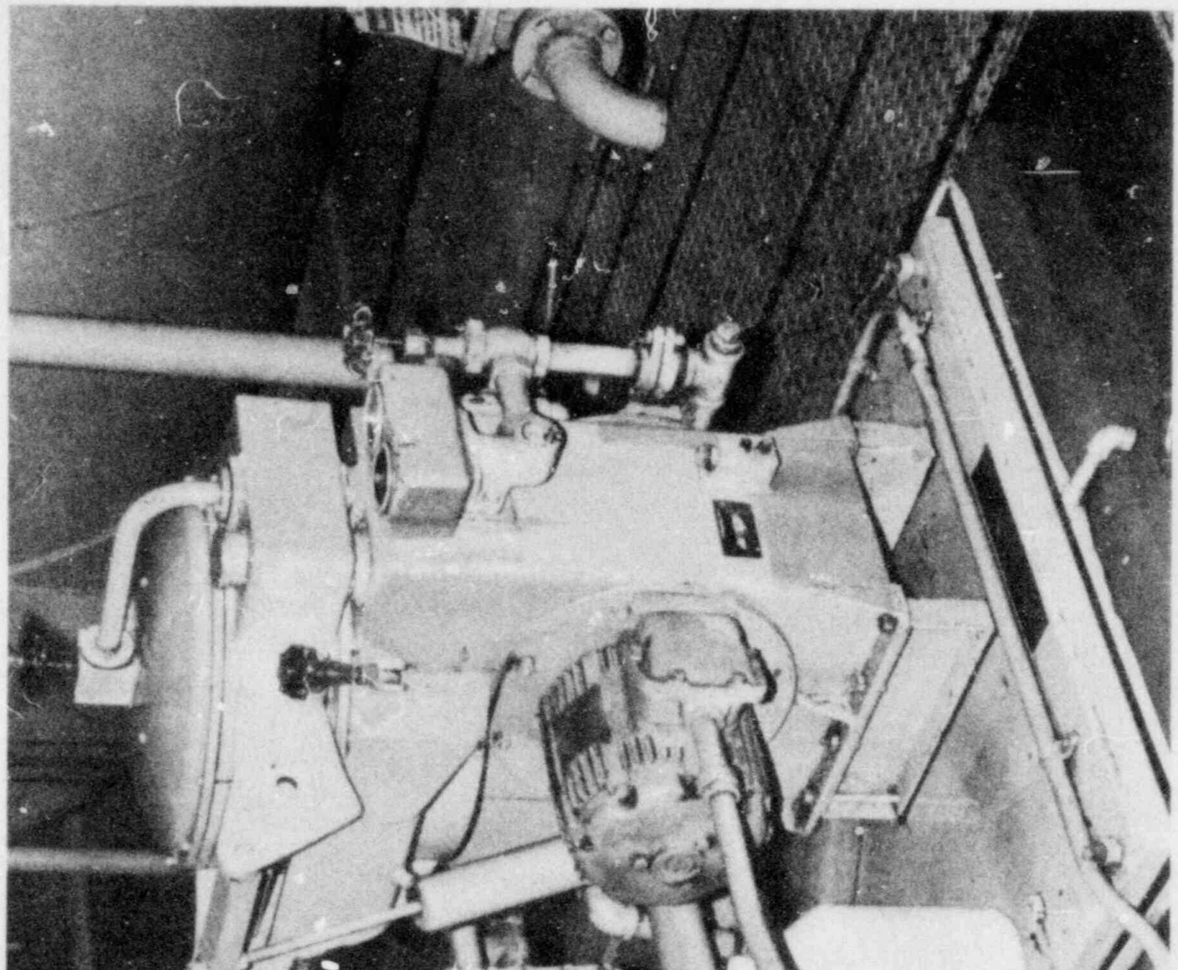
Fig. 51 Response spectra for 3 and 5% of critical damping computed for the flexible-base case at Node 109 (El. 983 ft) of the model shown in Fig. 21.



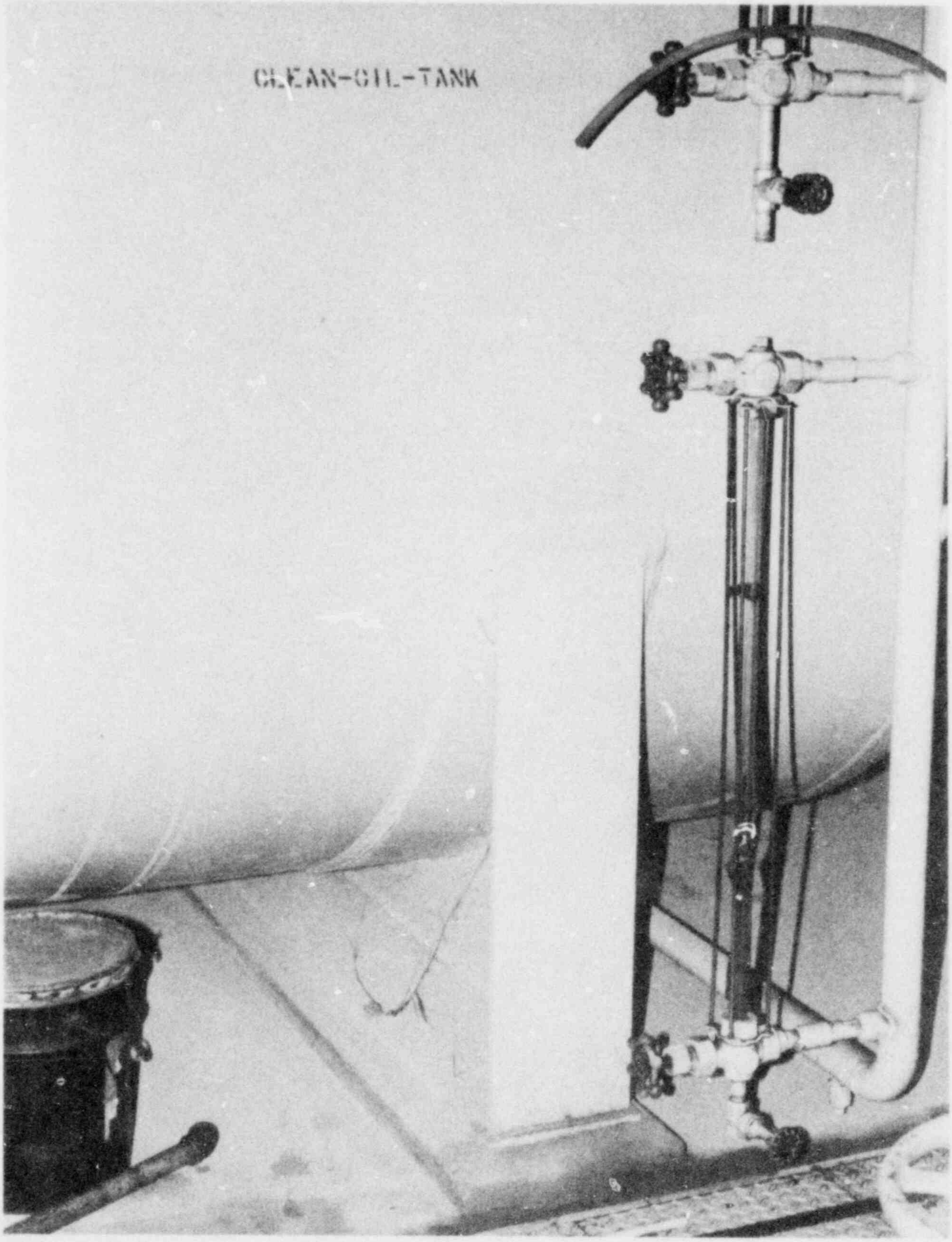
1 Plant air compressor



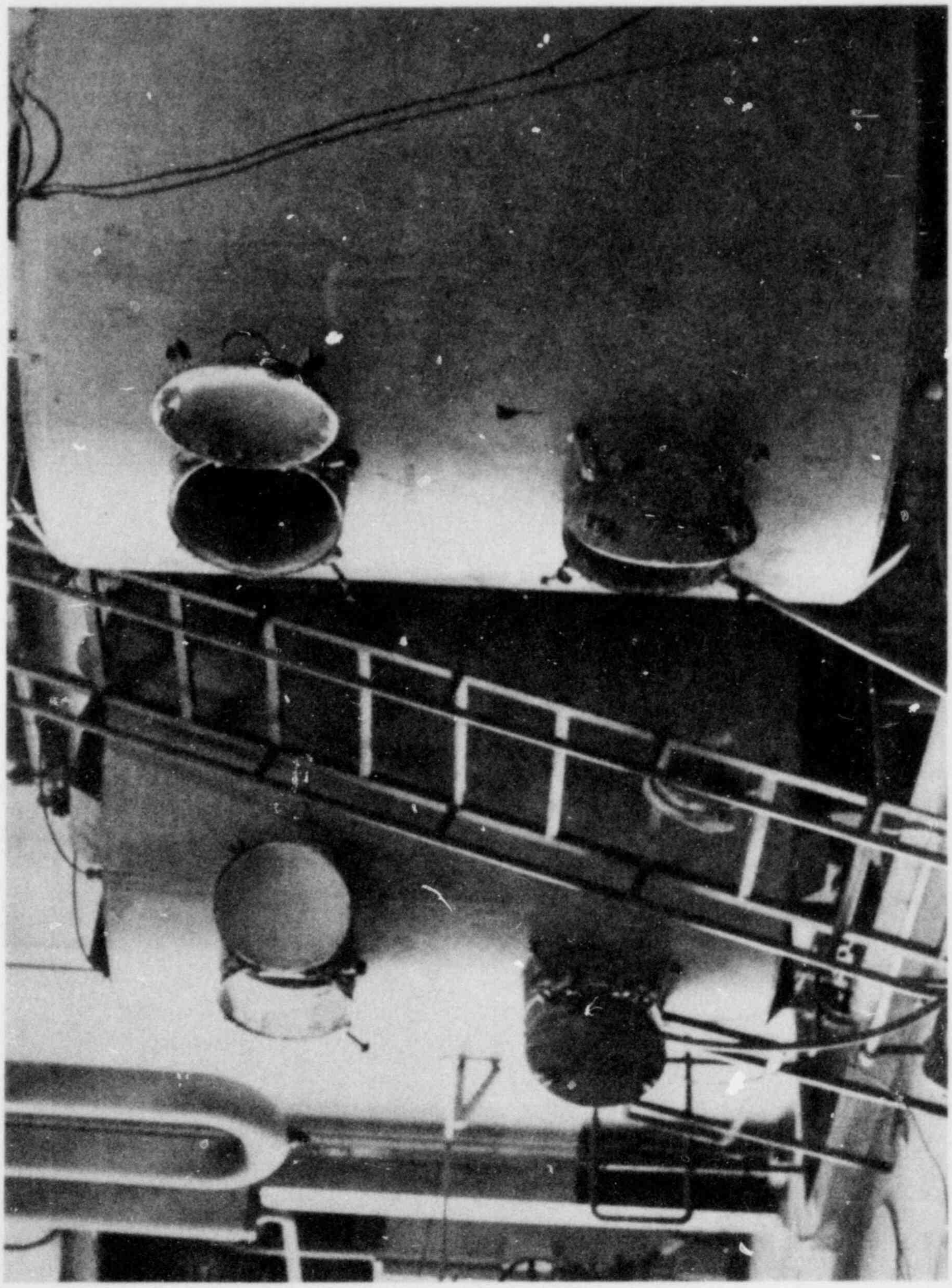
2 Air receiver tank



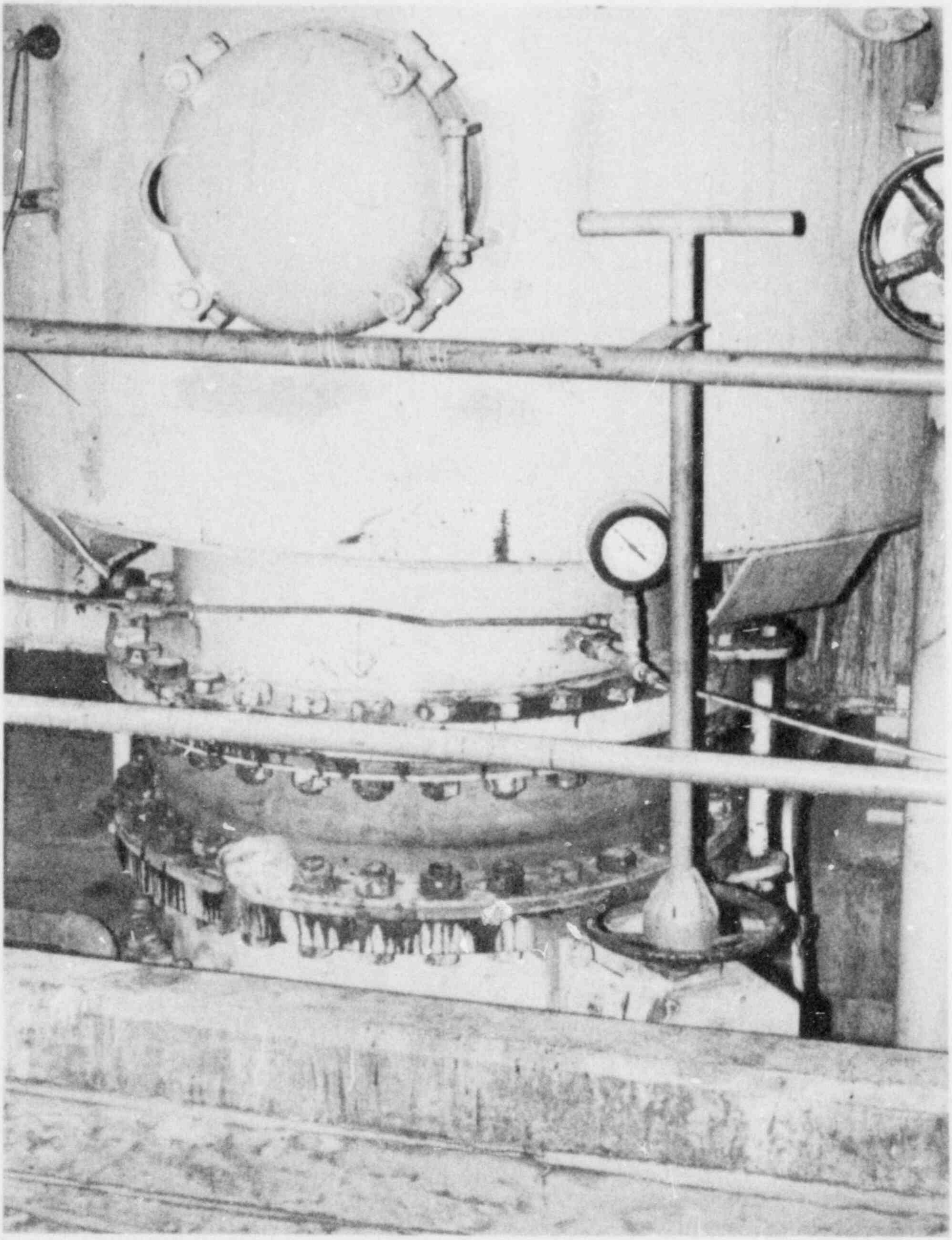
3 Lube oil conditioning equipment



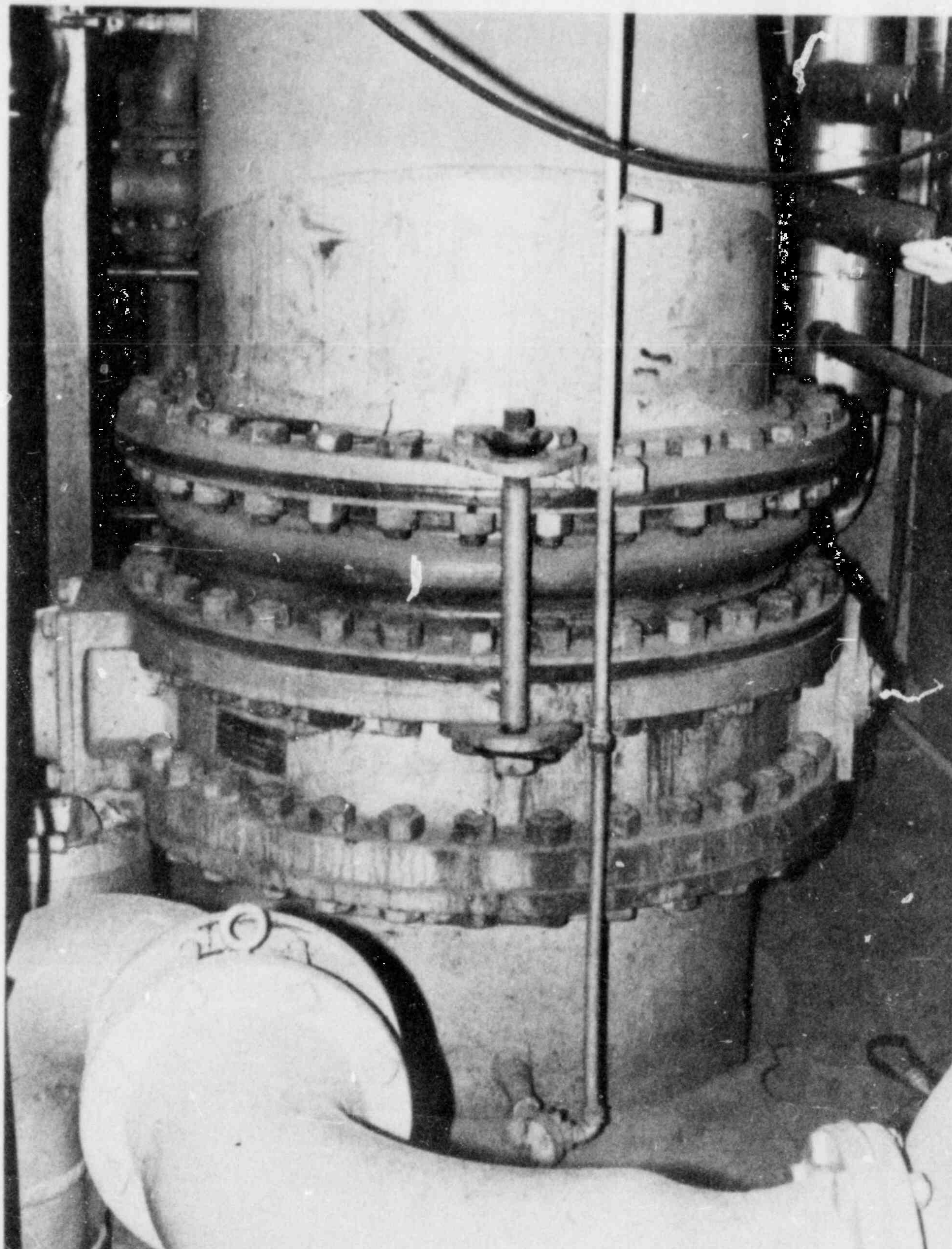
3 Lube oil conditioning tank



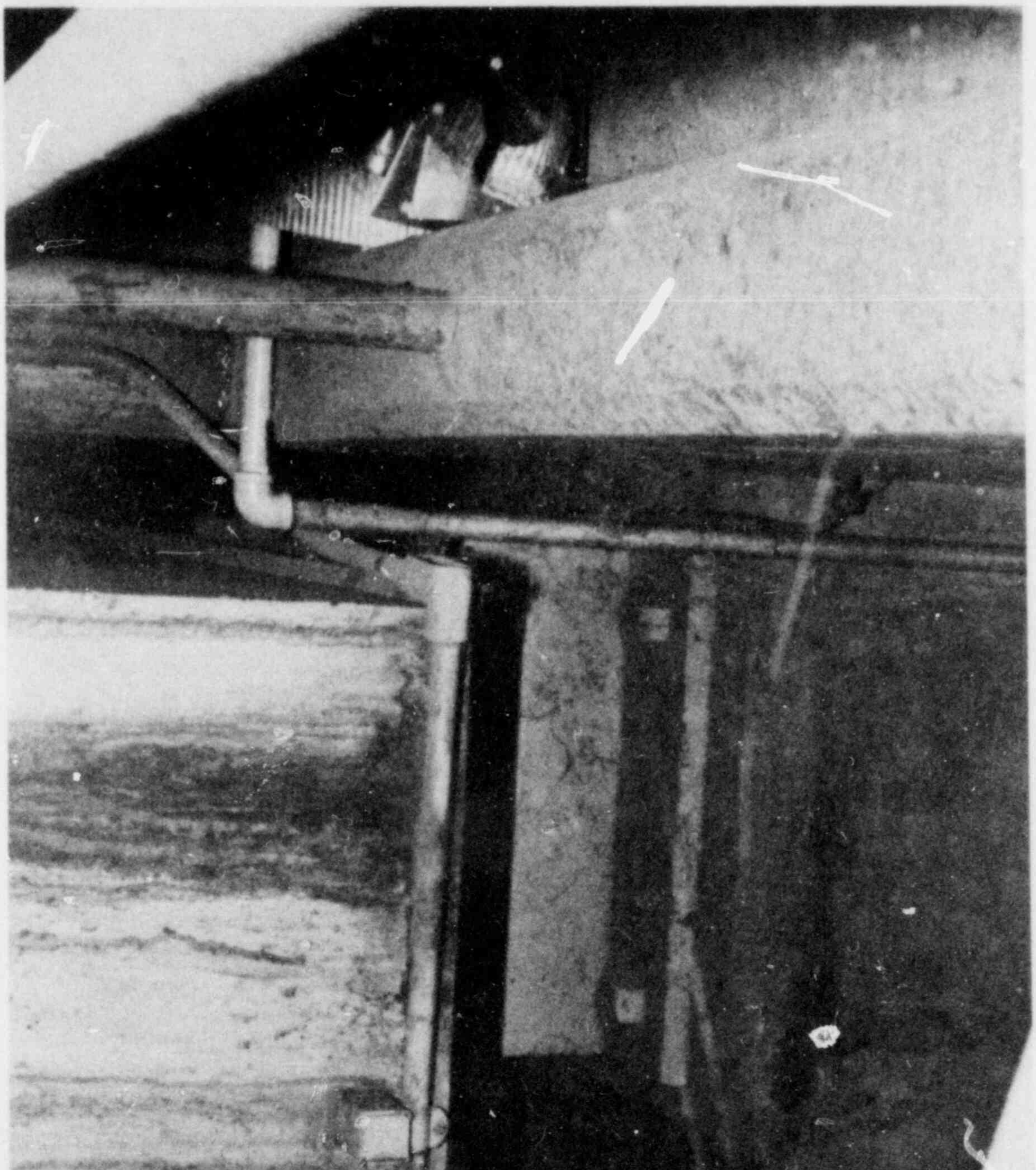
4 Condenser



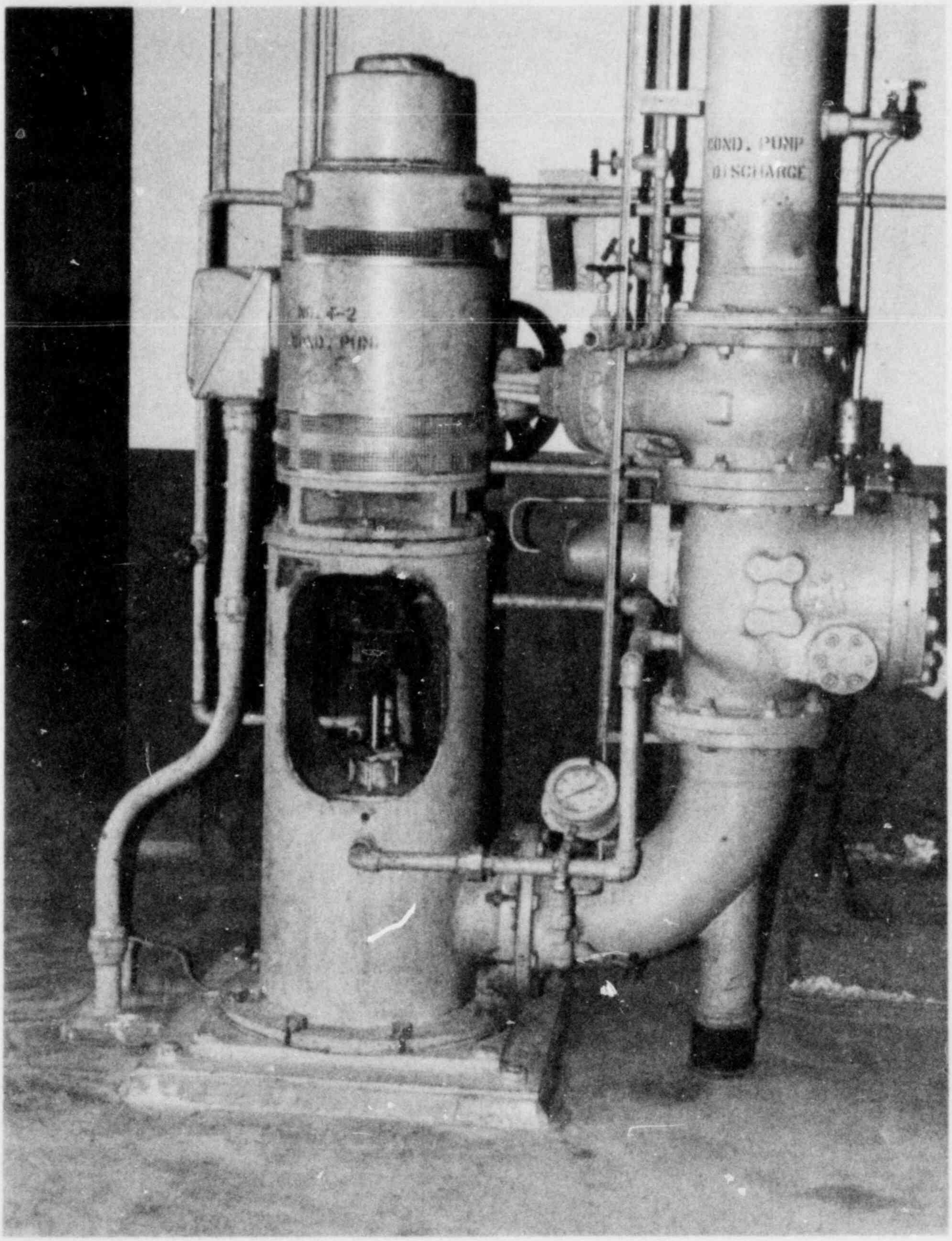
4 Condenser ou̇clet expansio̤ joint



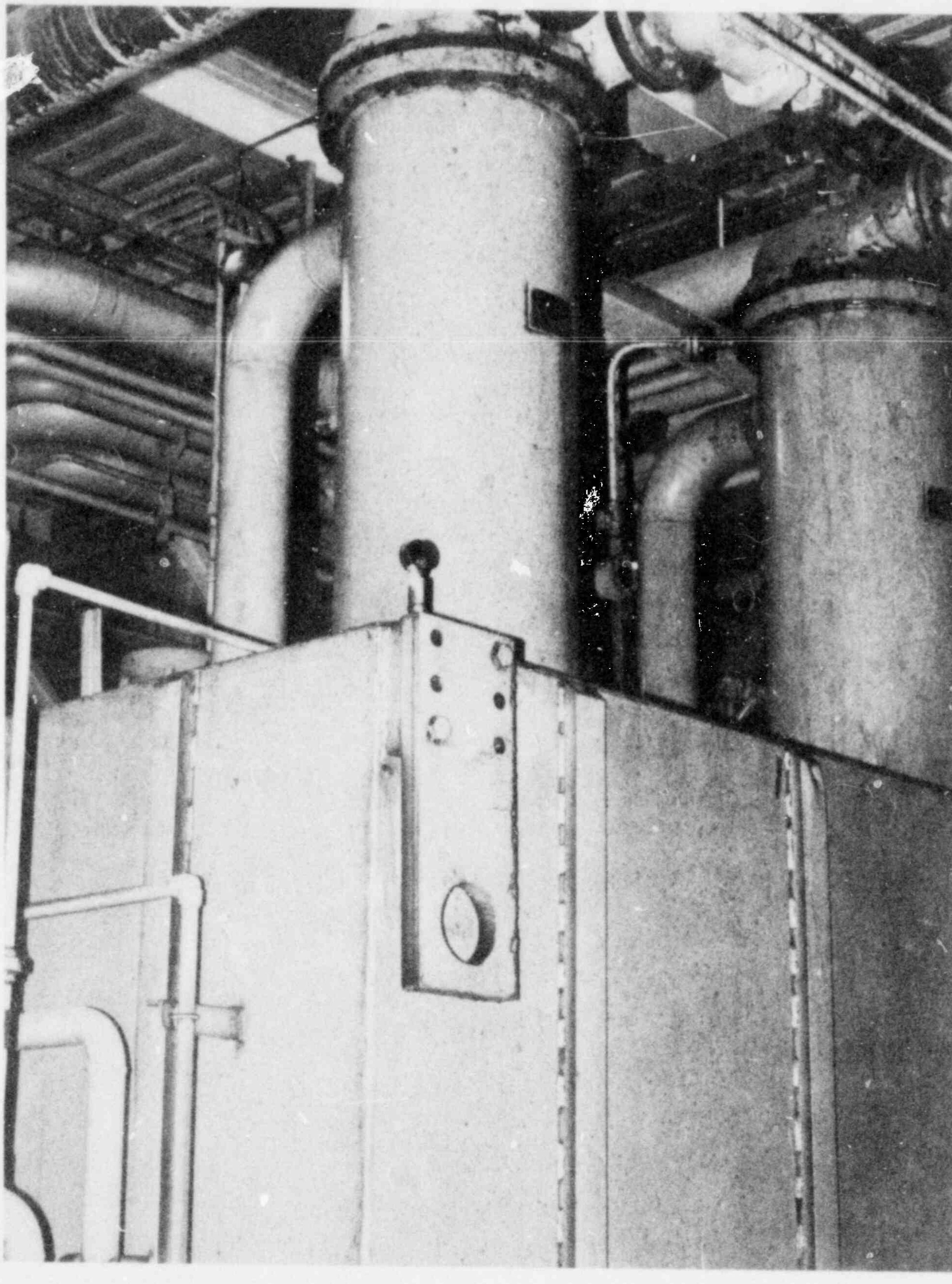
4 Condenser inlet expansion joint



4 Condenser support pedestal

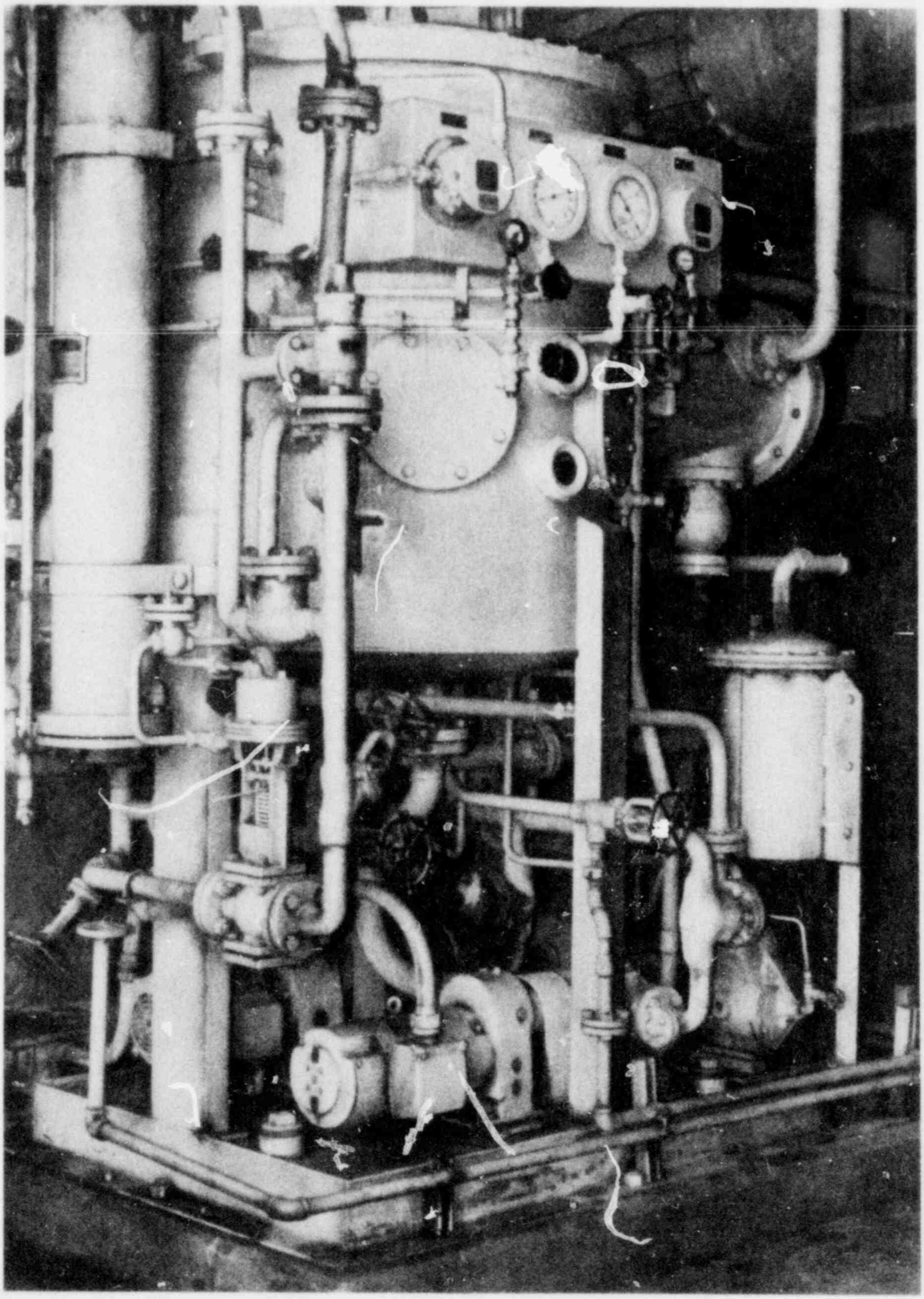


5 Condensate pump

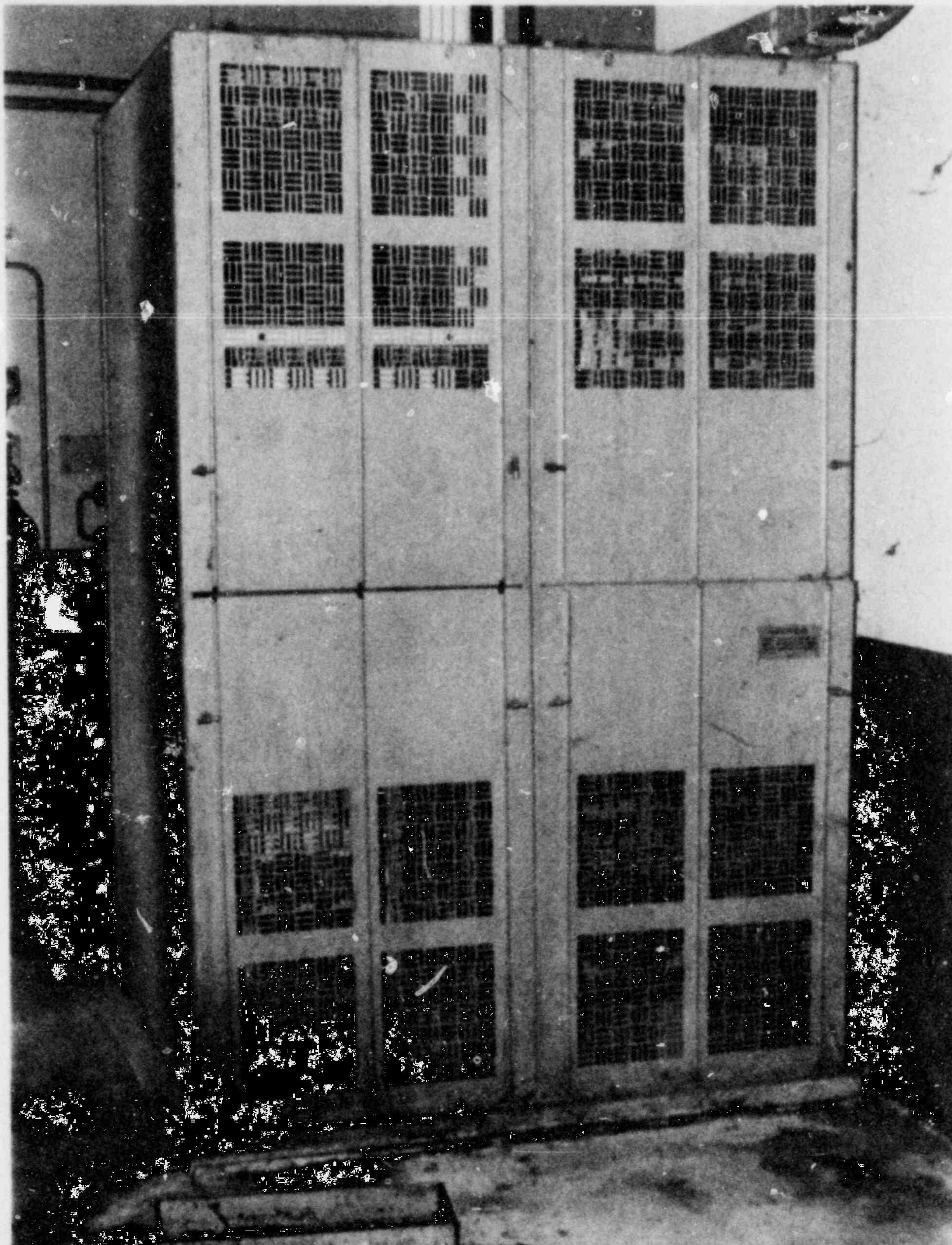


6 Turbine oil cooler tank

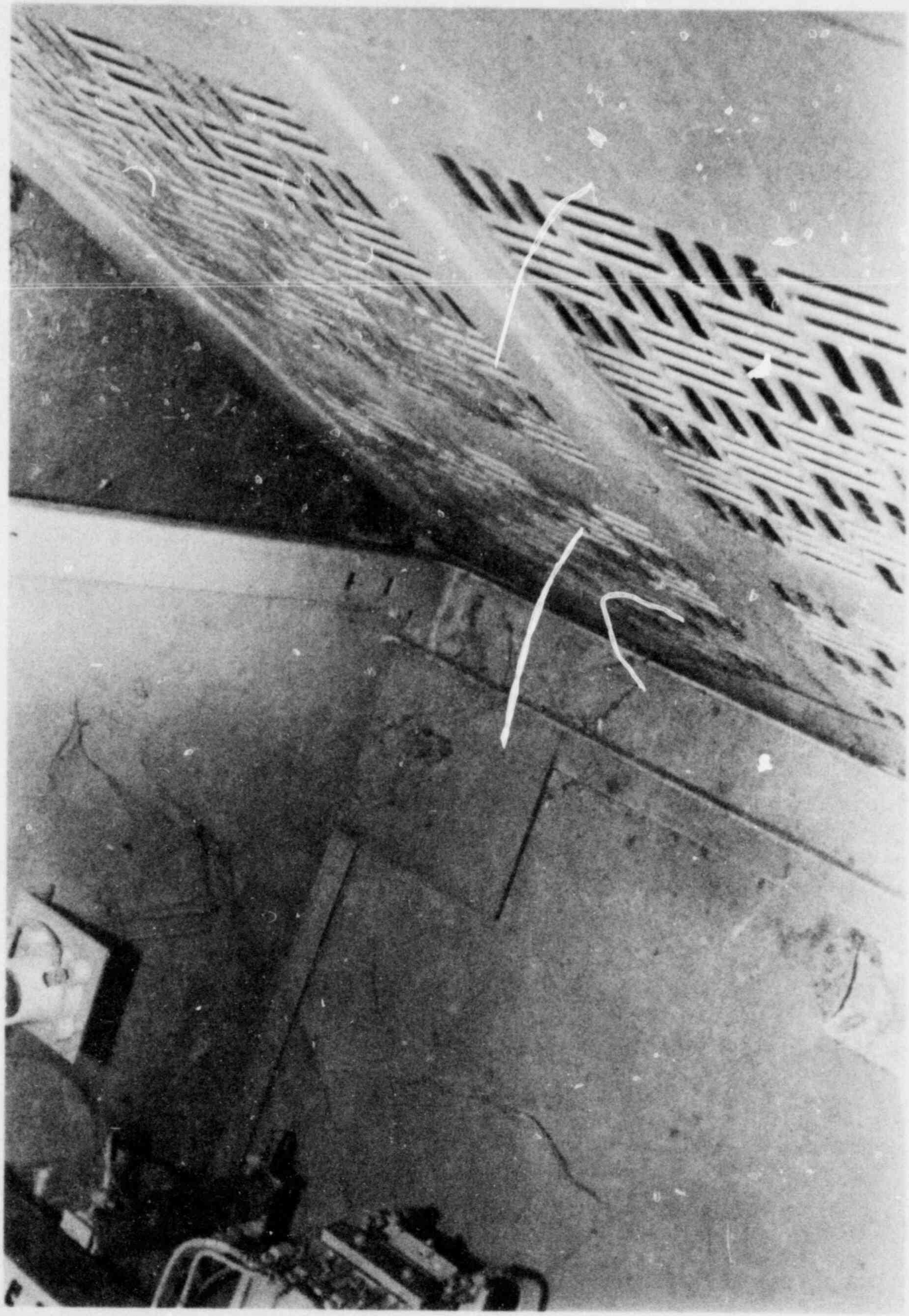
6 Turbine oil cooler tank anchorage



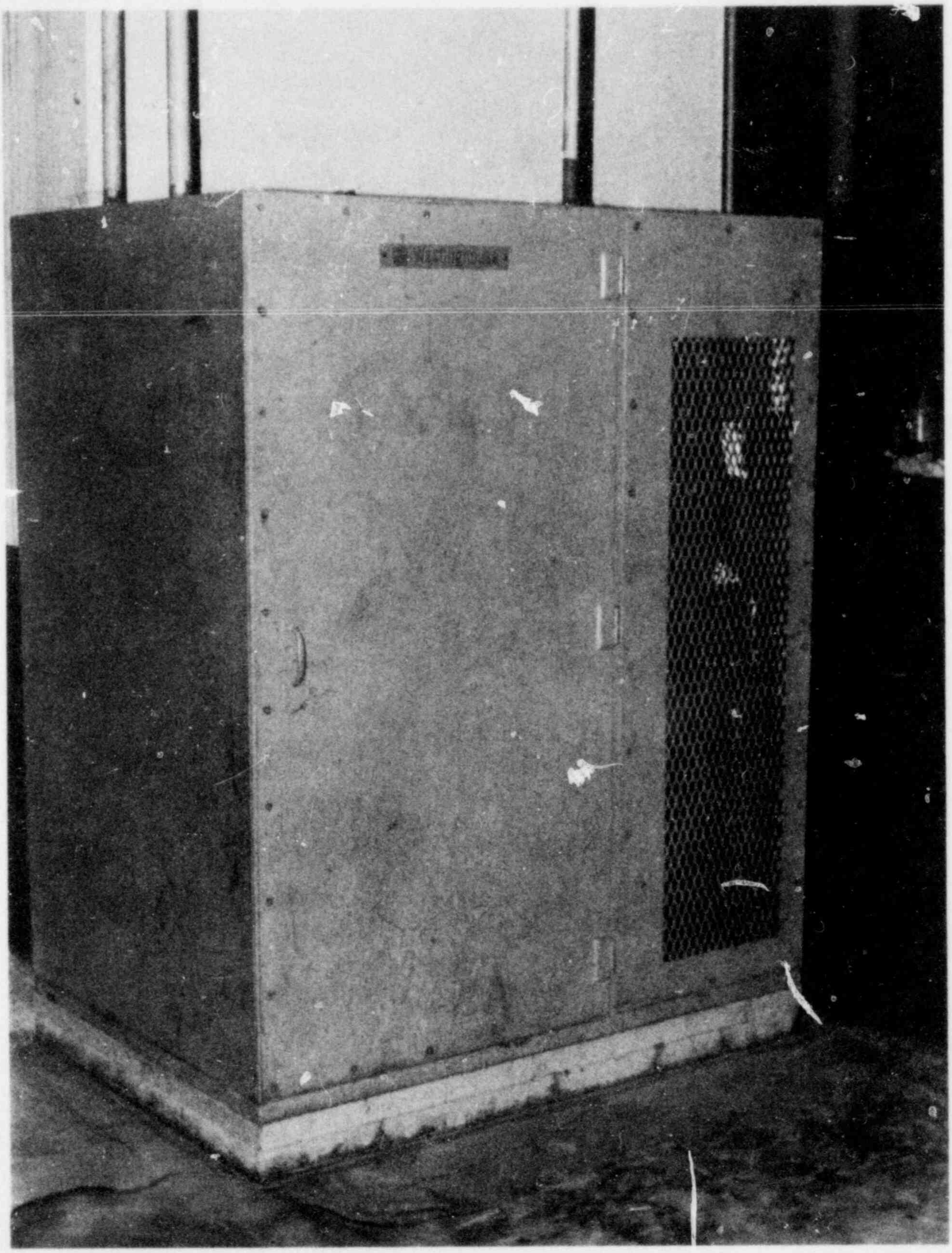
7 Seal oil conditioning system



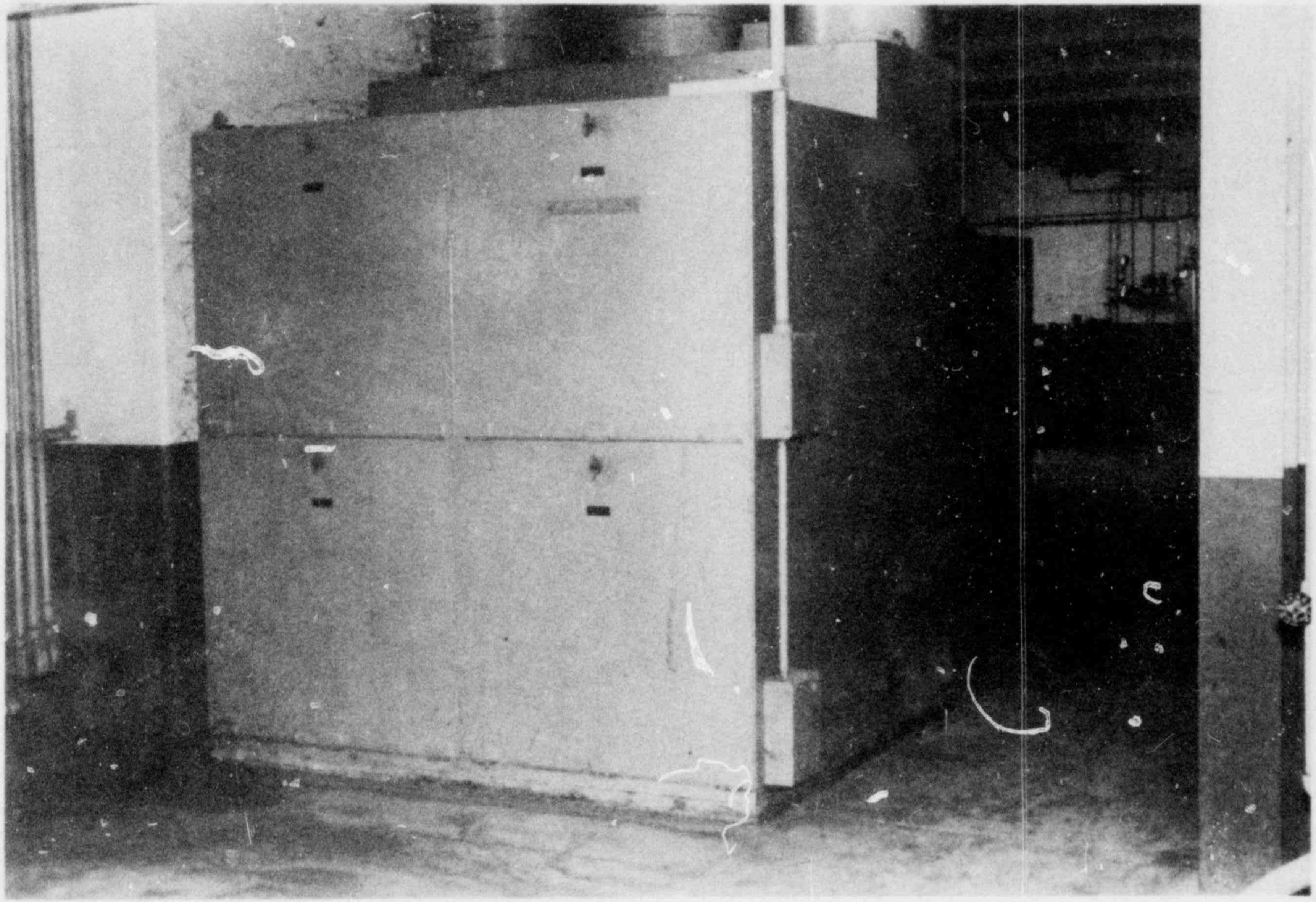
8 Exciter cubicle



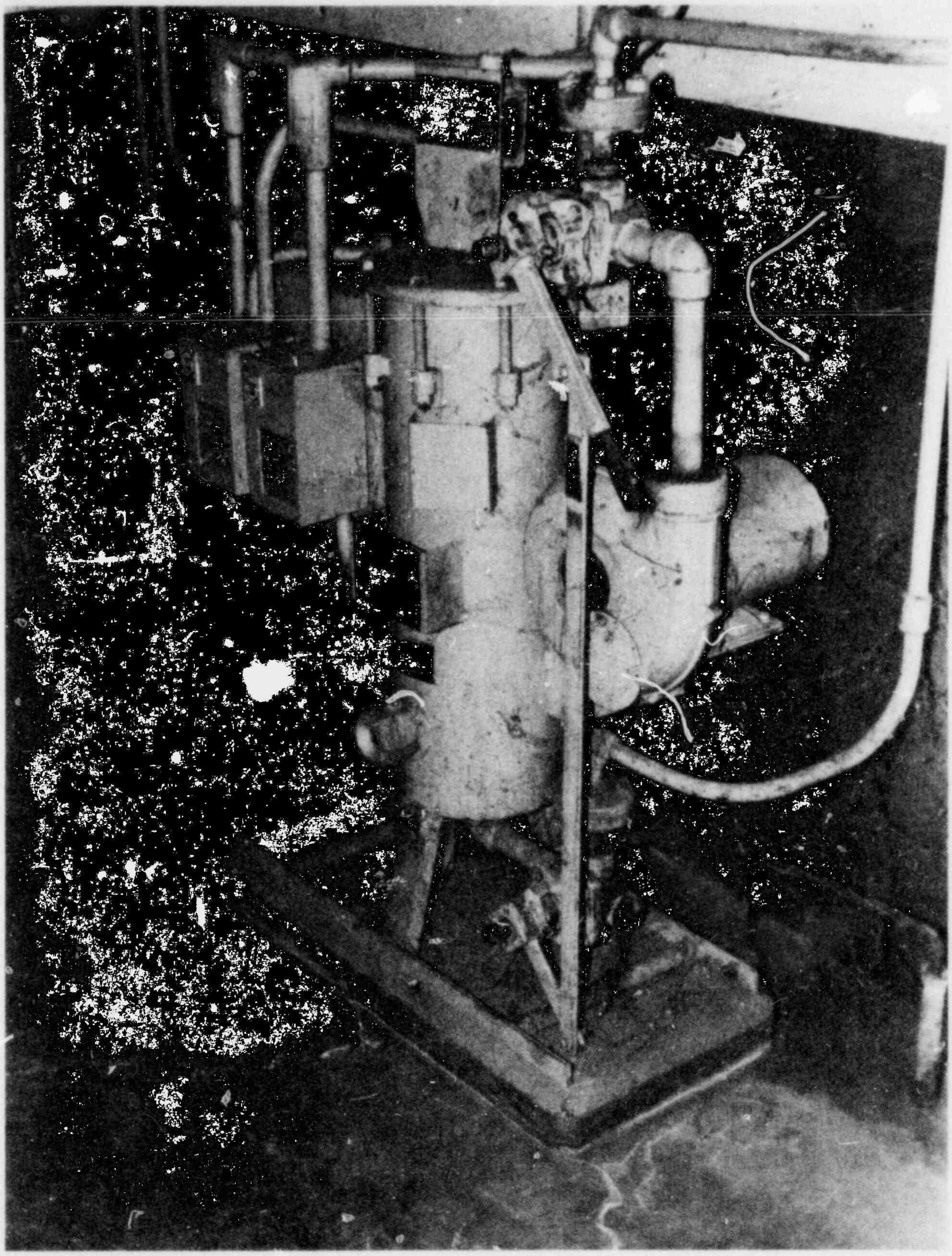
8 Exciter cubicle anchorage



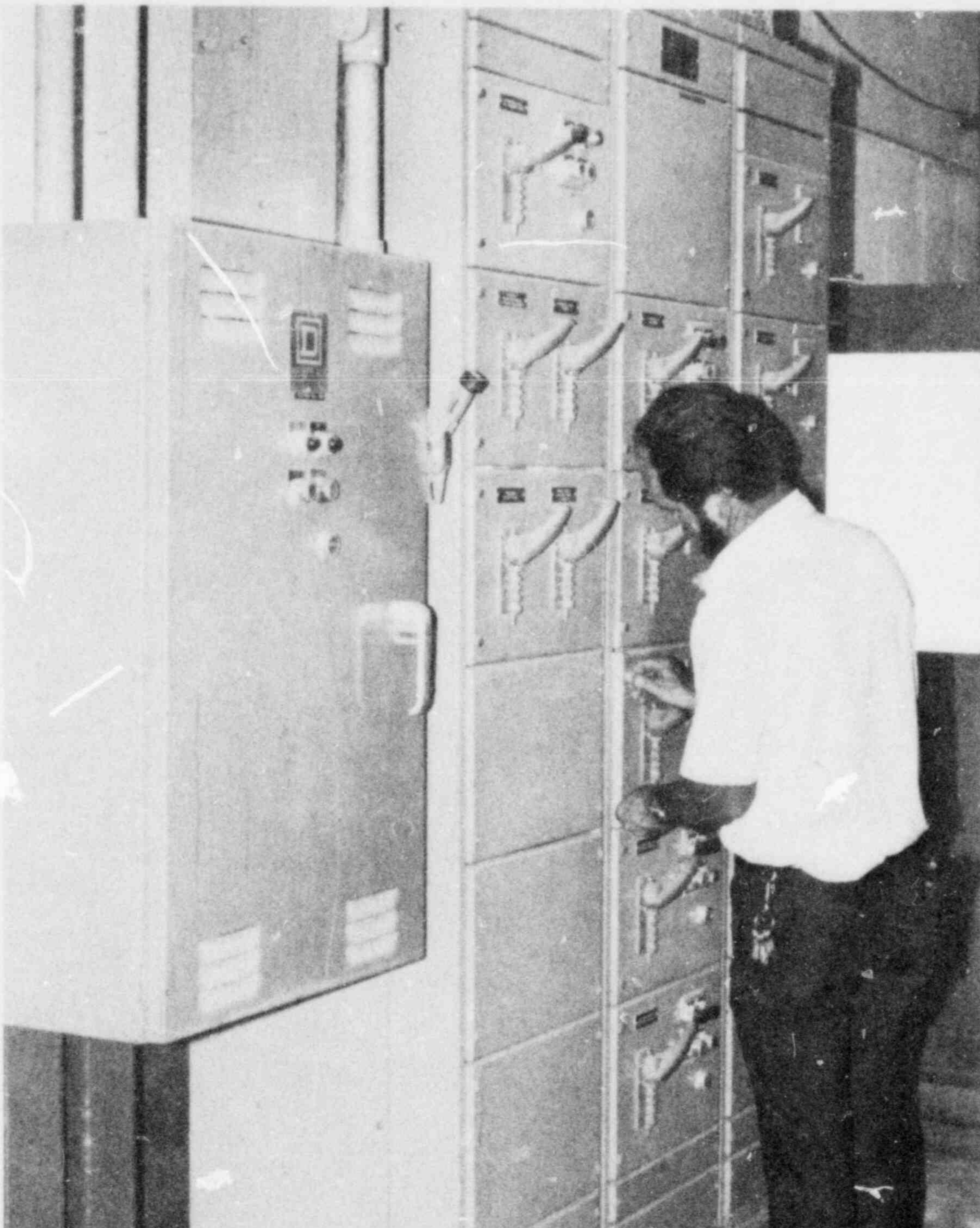
9 Neutral transformer



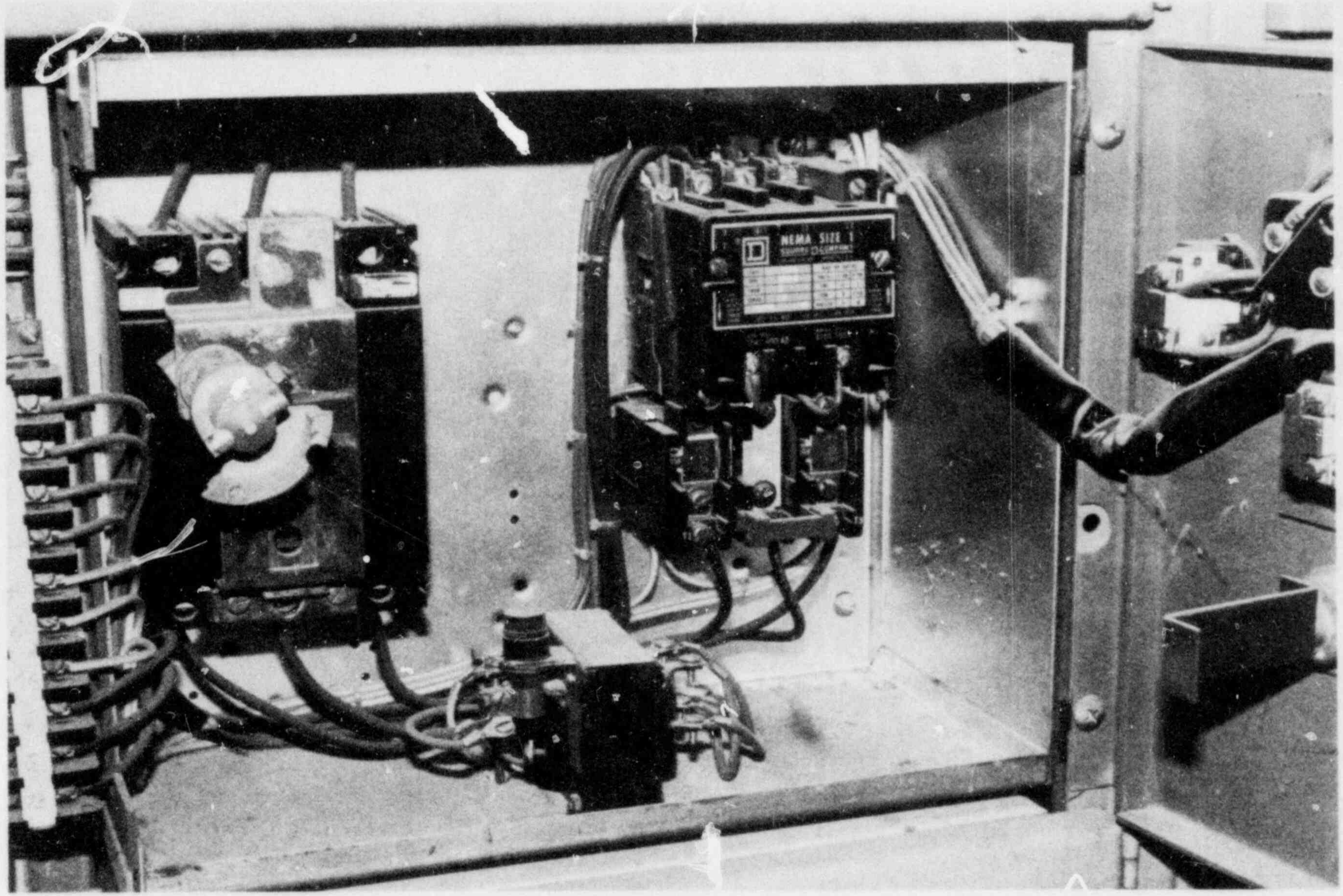
10 Voltage transformer and surge protection cubicle



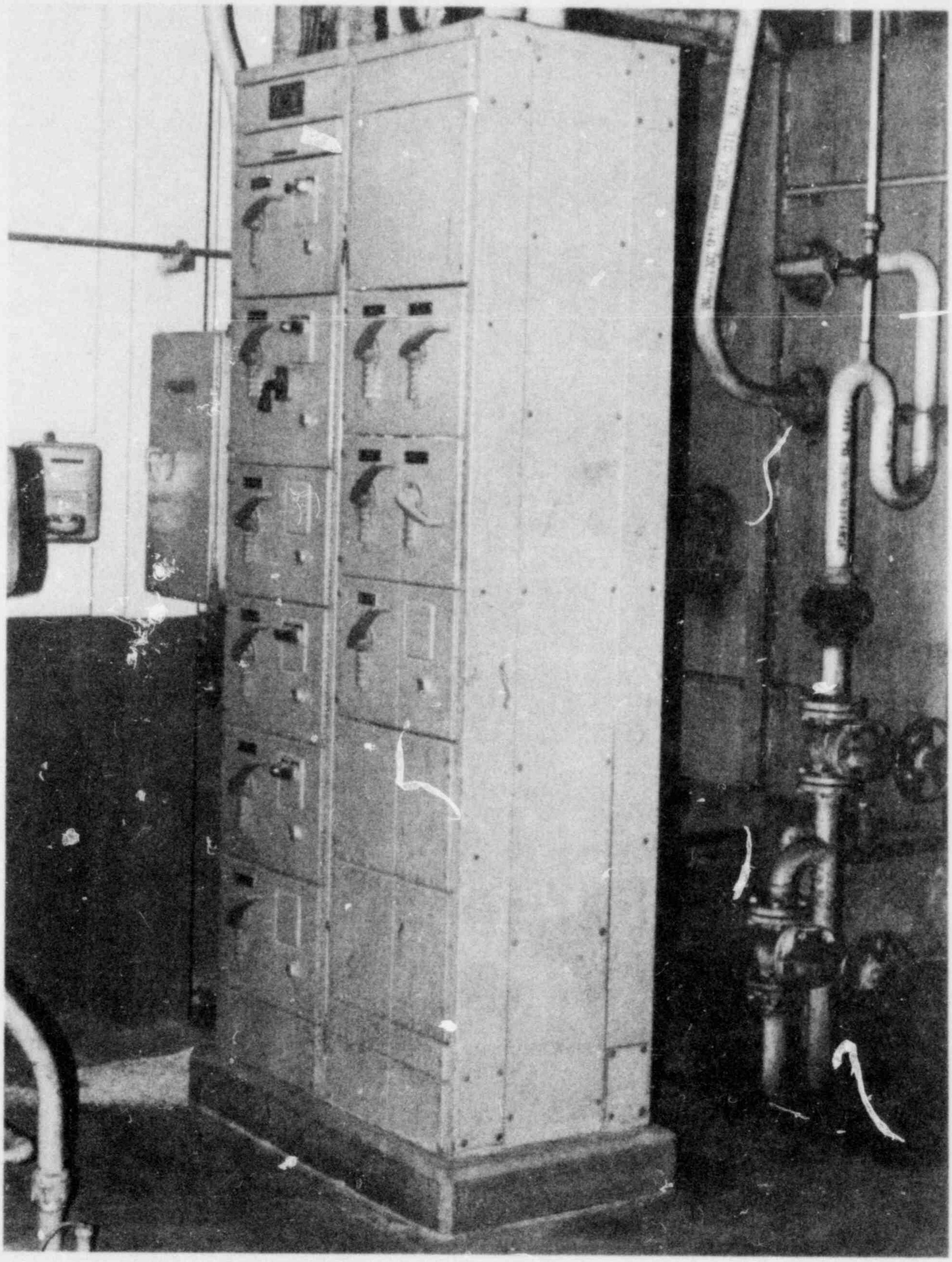
11 Gas dryer



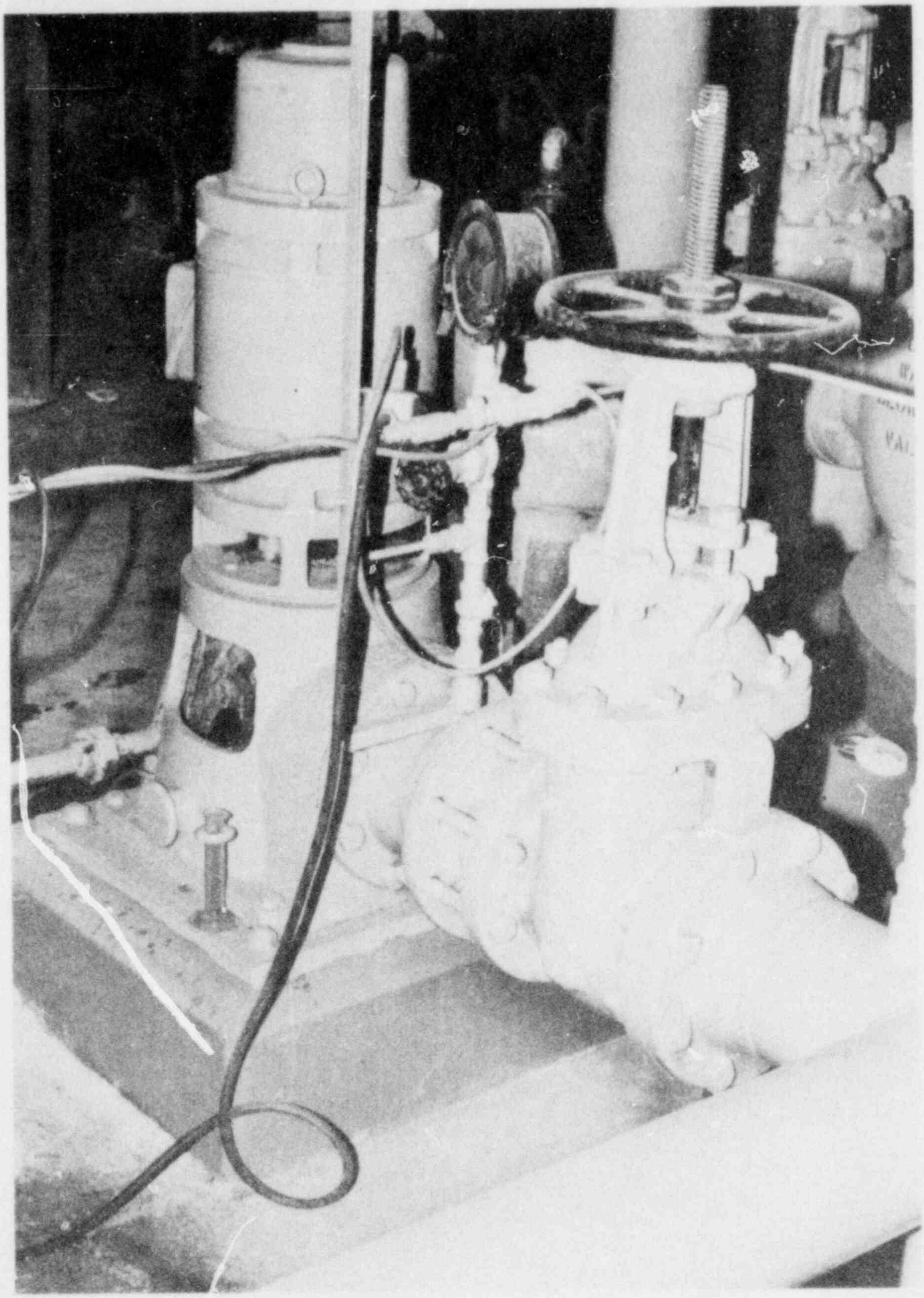
12 480-V motor control center No. 1



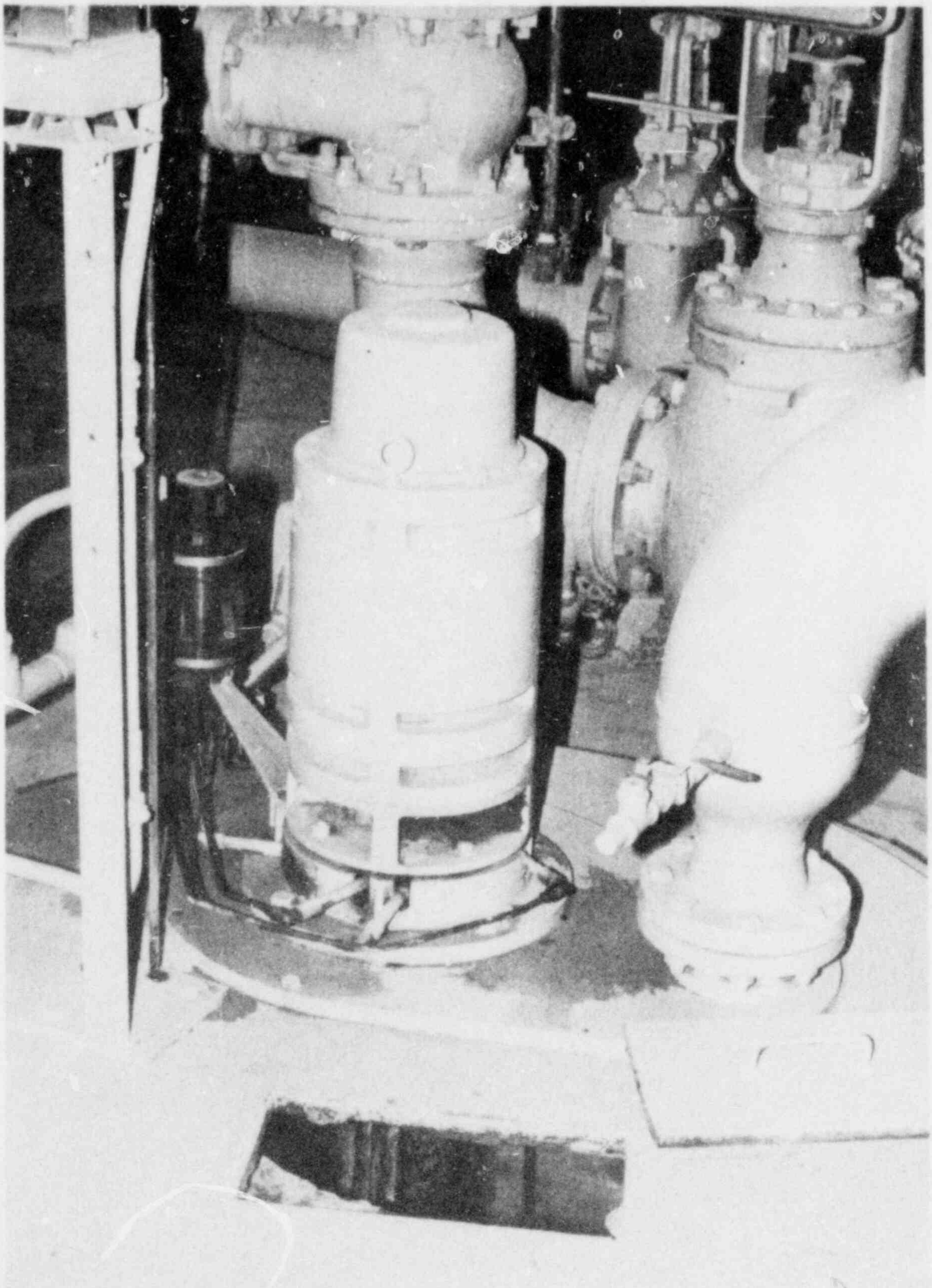
12 480-V motor control center No. 1--Breakers



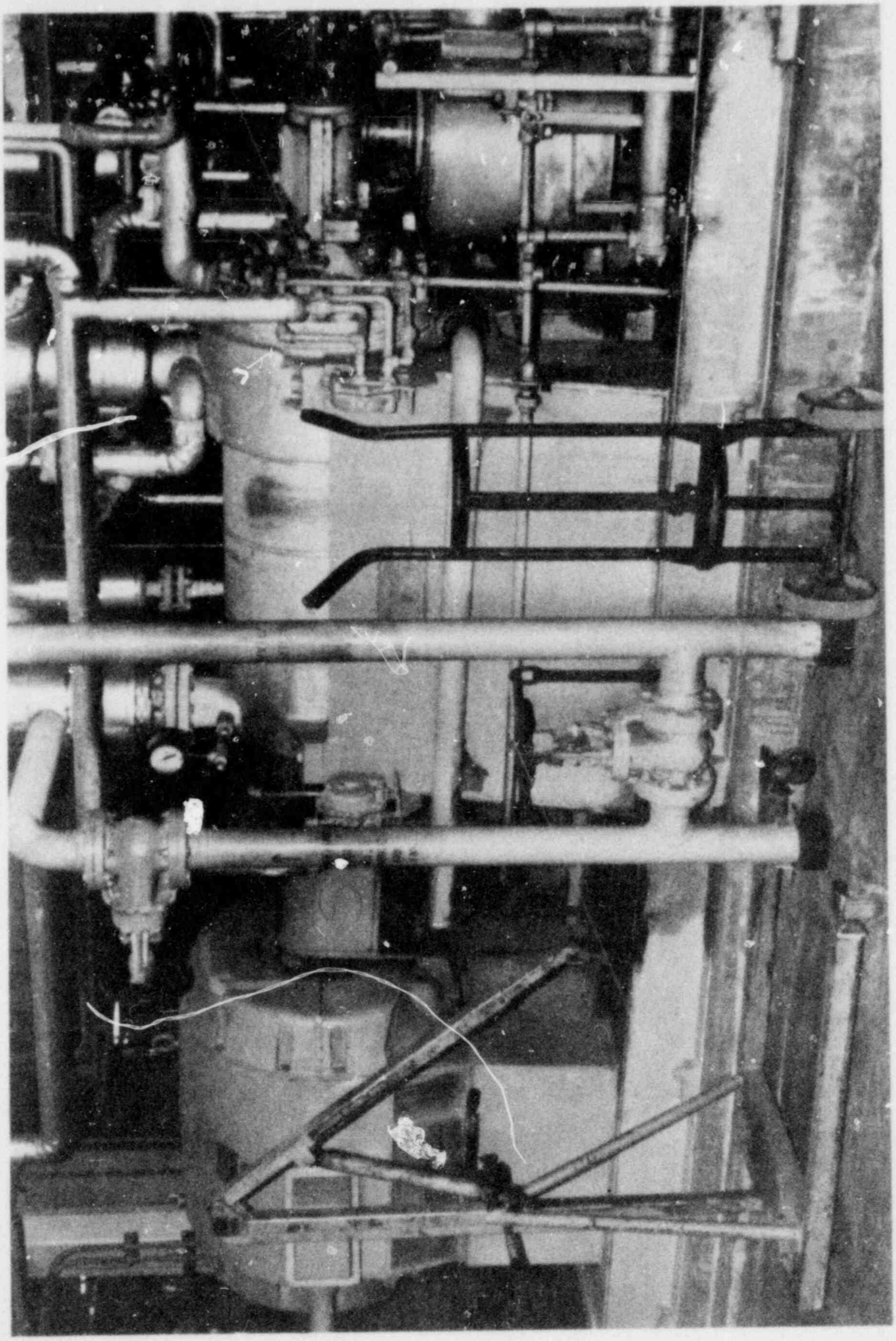
13 480-V motor control center No. 2



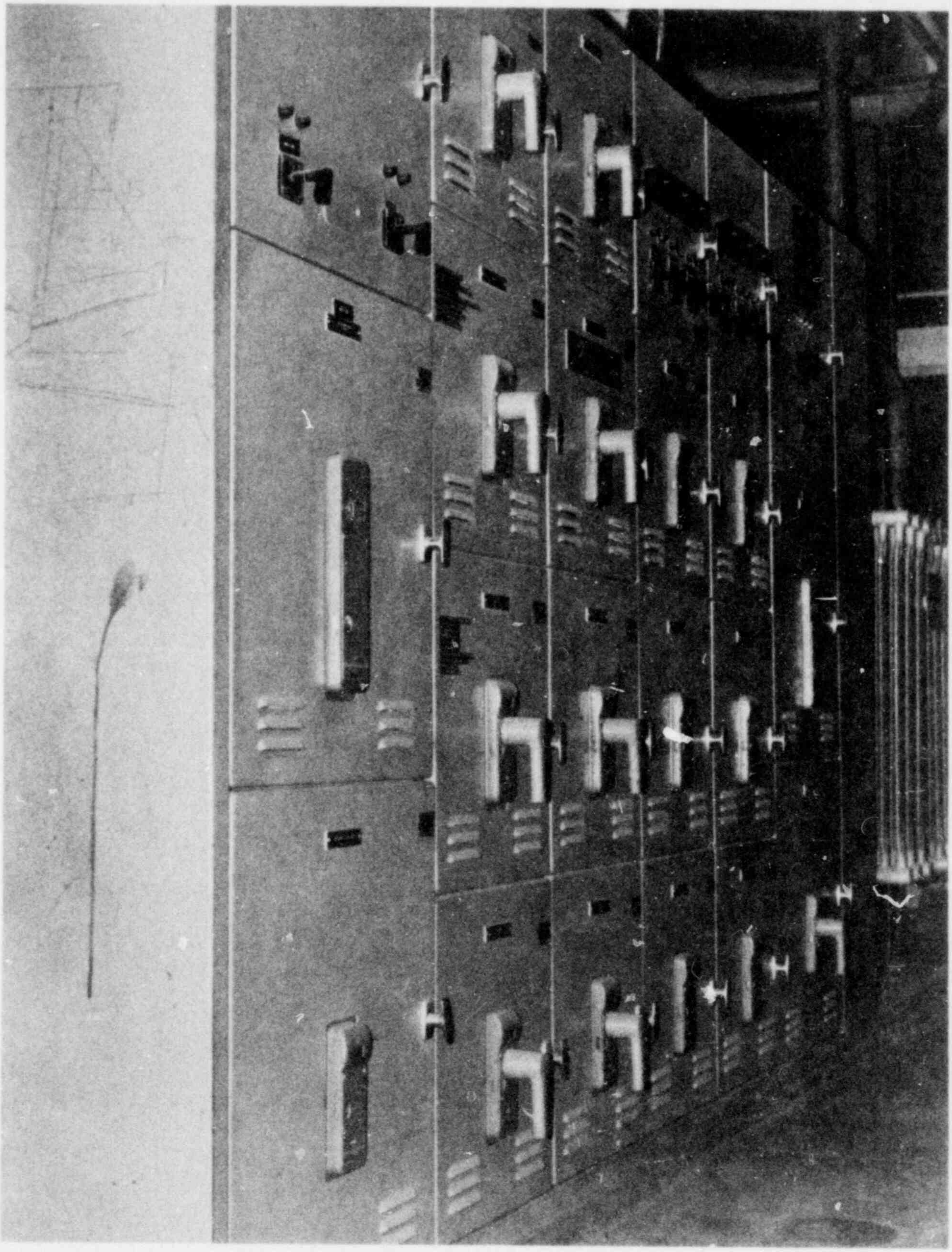
14 Service water return pump



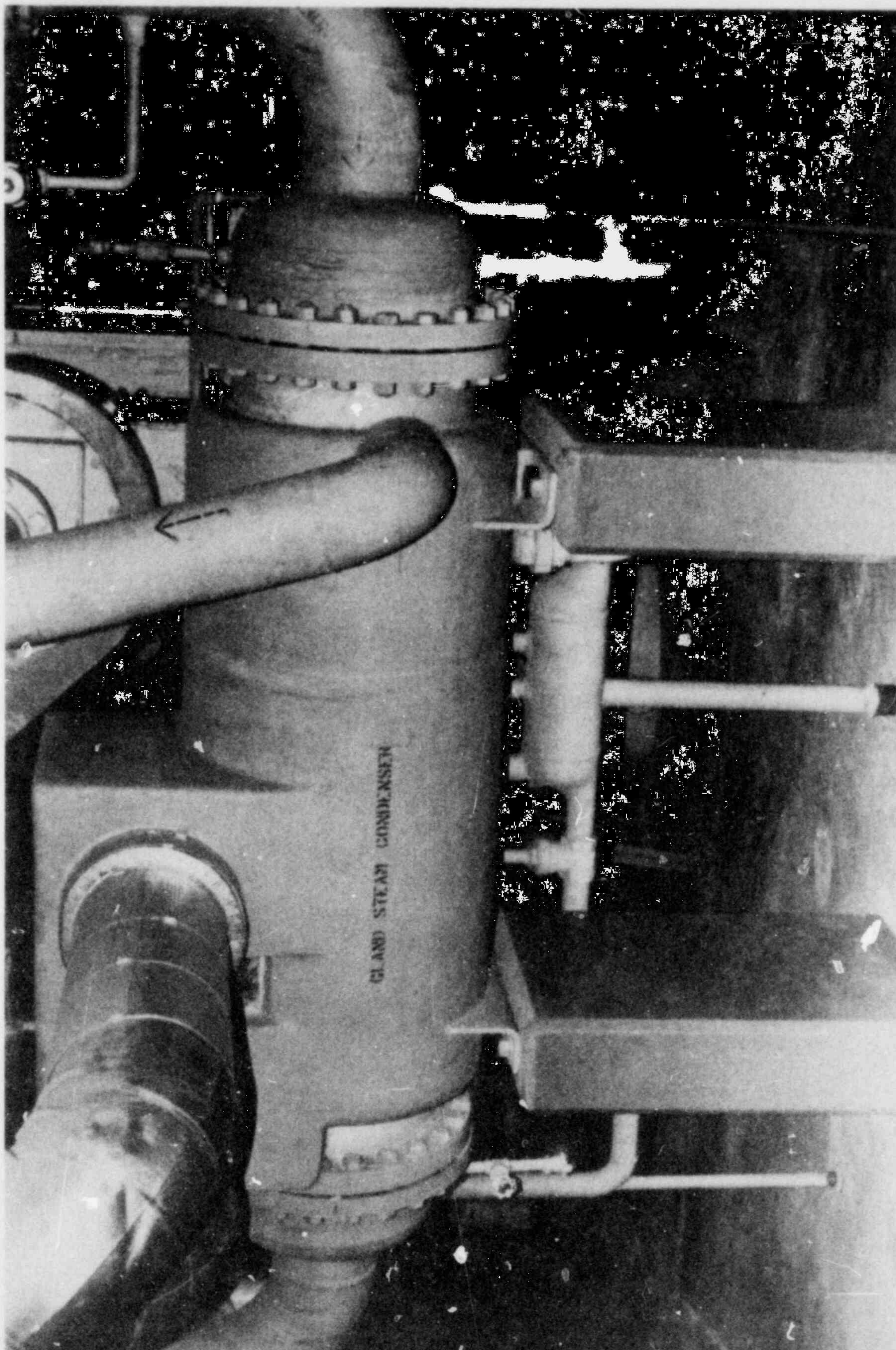
15 Drainage sump pump



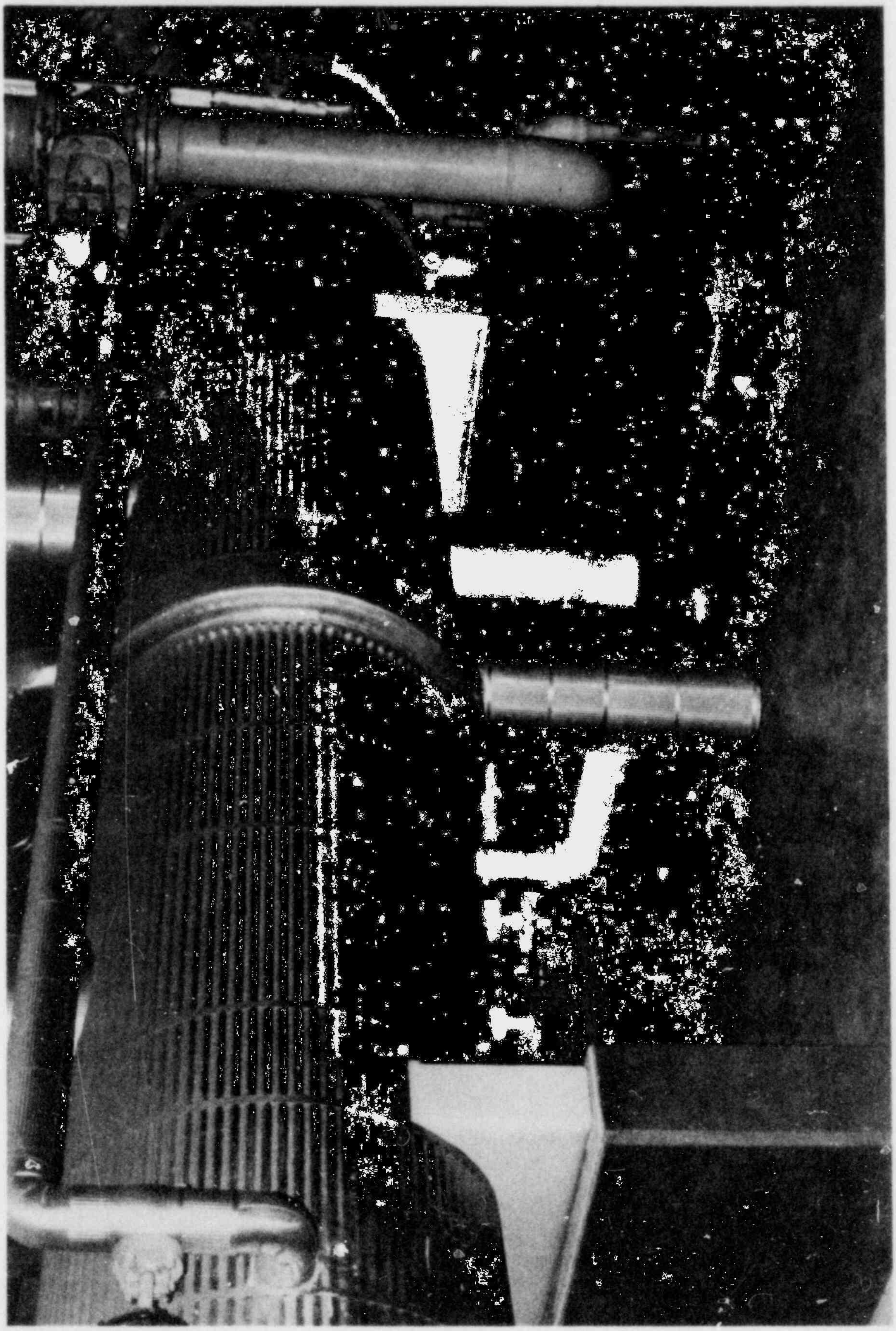
16 Boiler feed pump



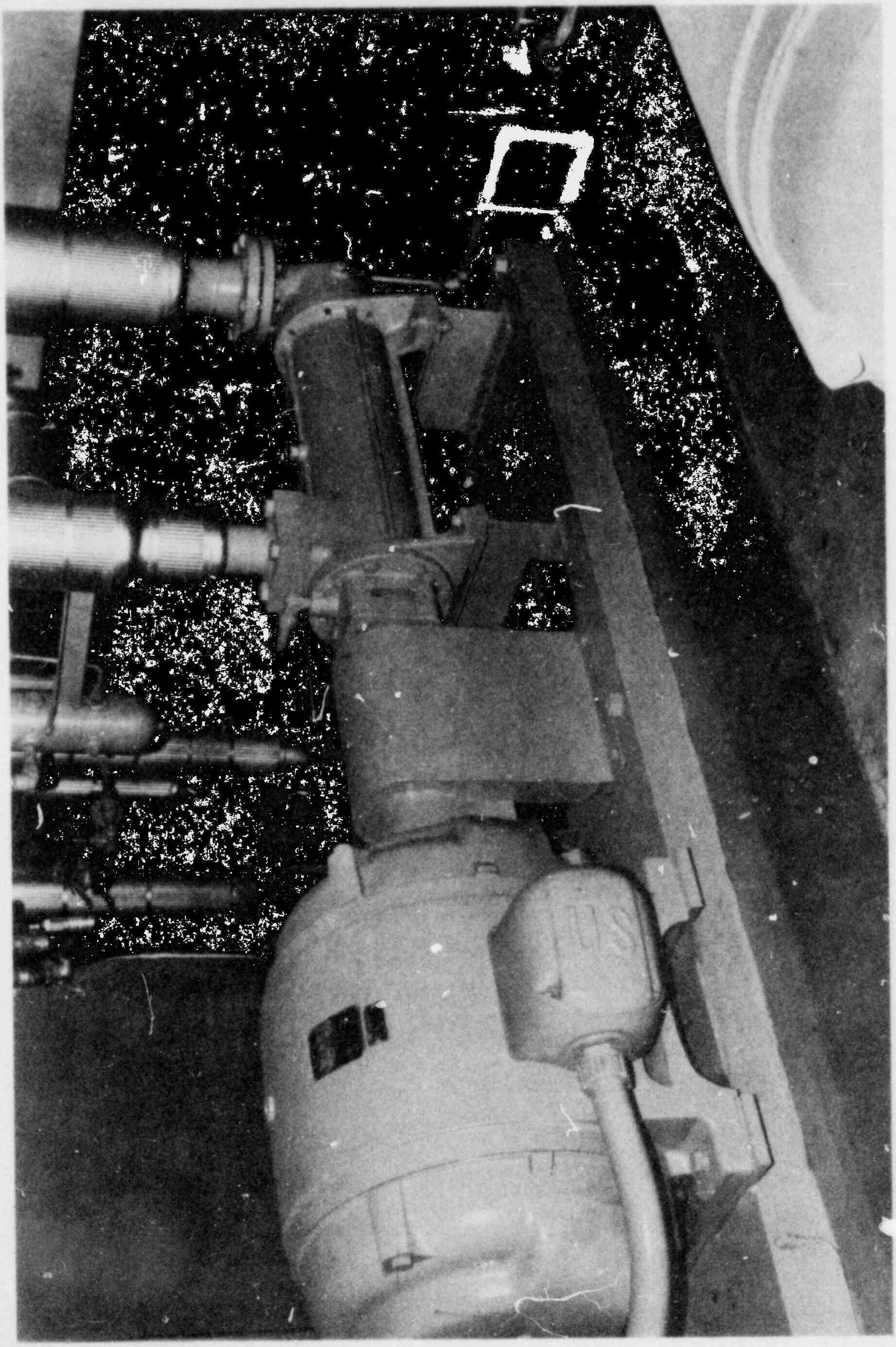
17 480-V switchgear



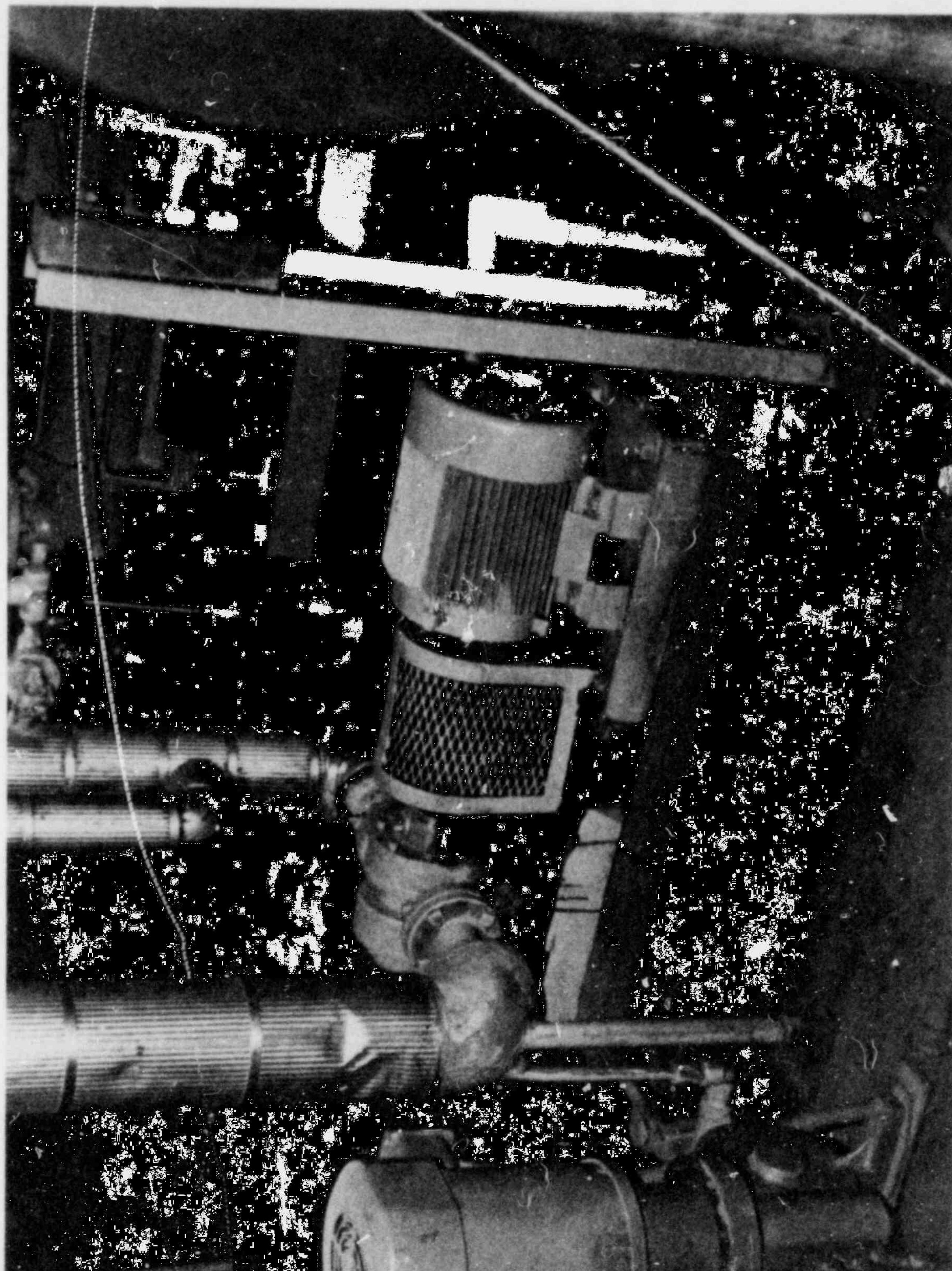
18 Gland steam condenser



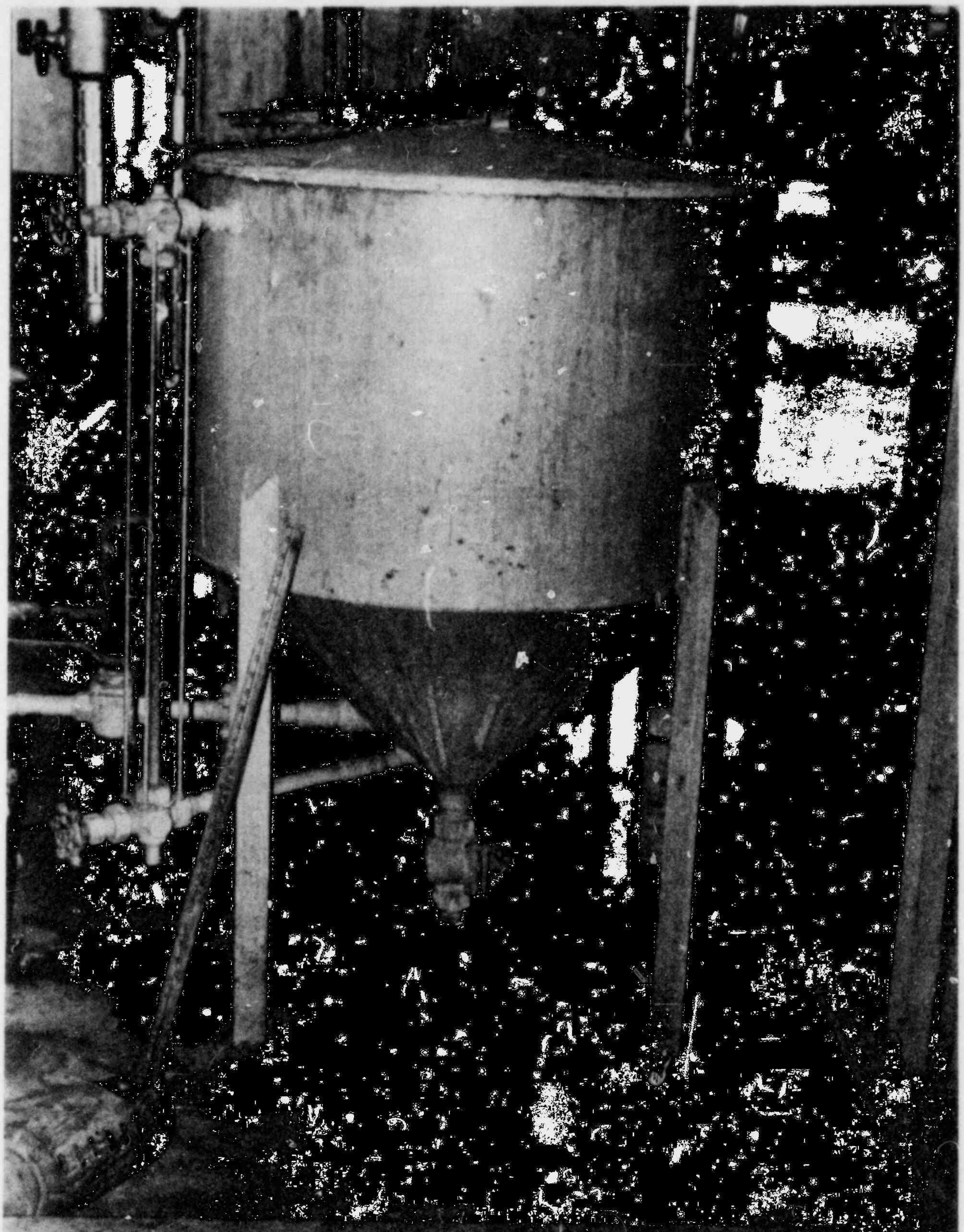
19 Low-pressure feedwater heater



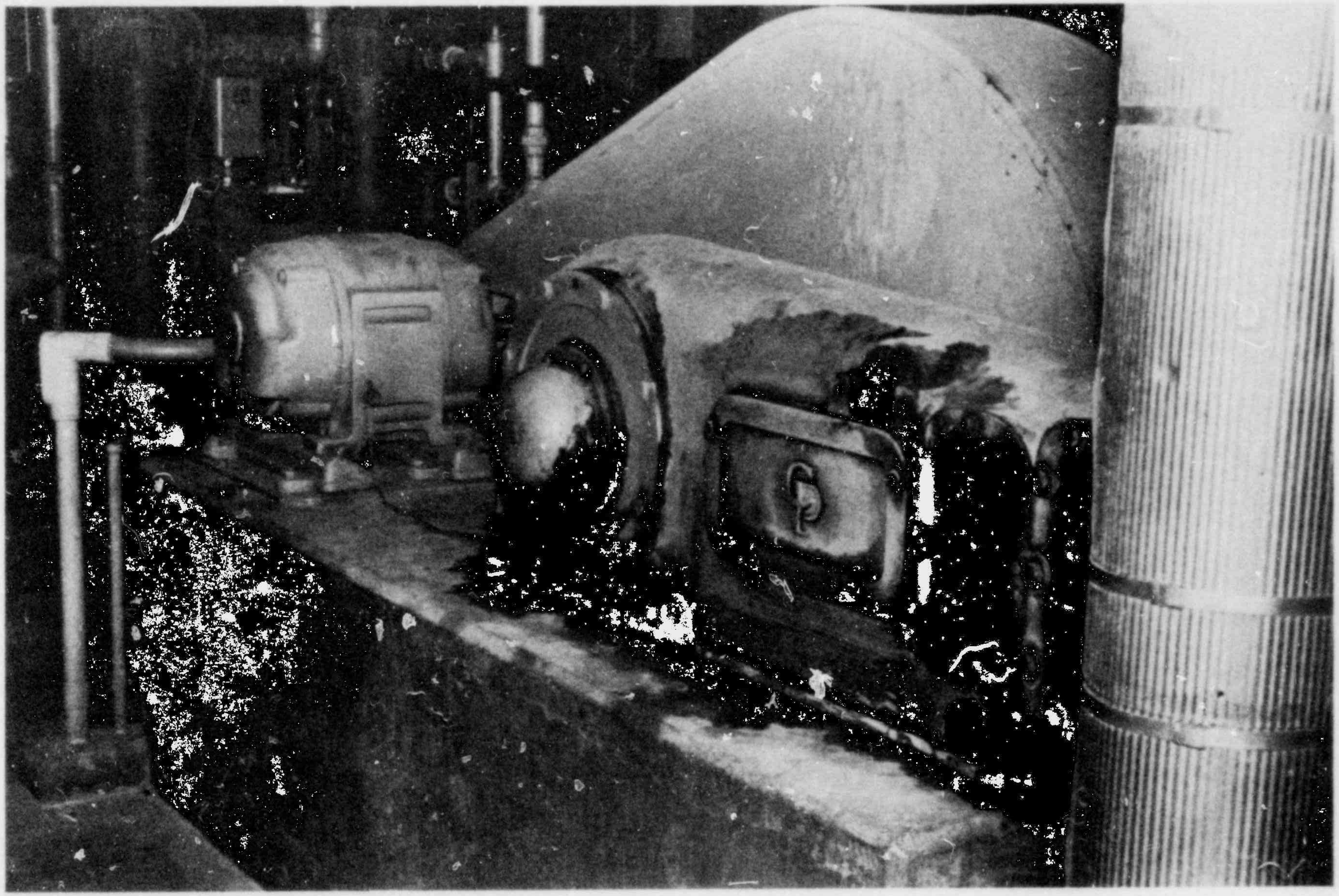
20 High-pressure fuel oil pump



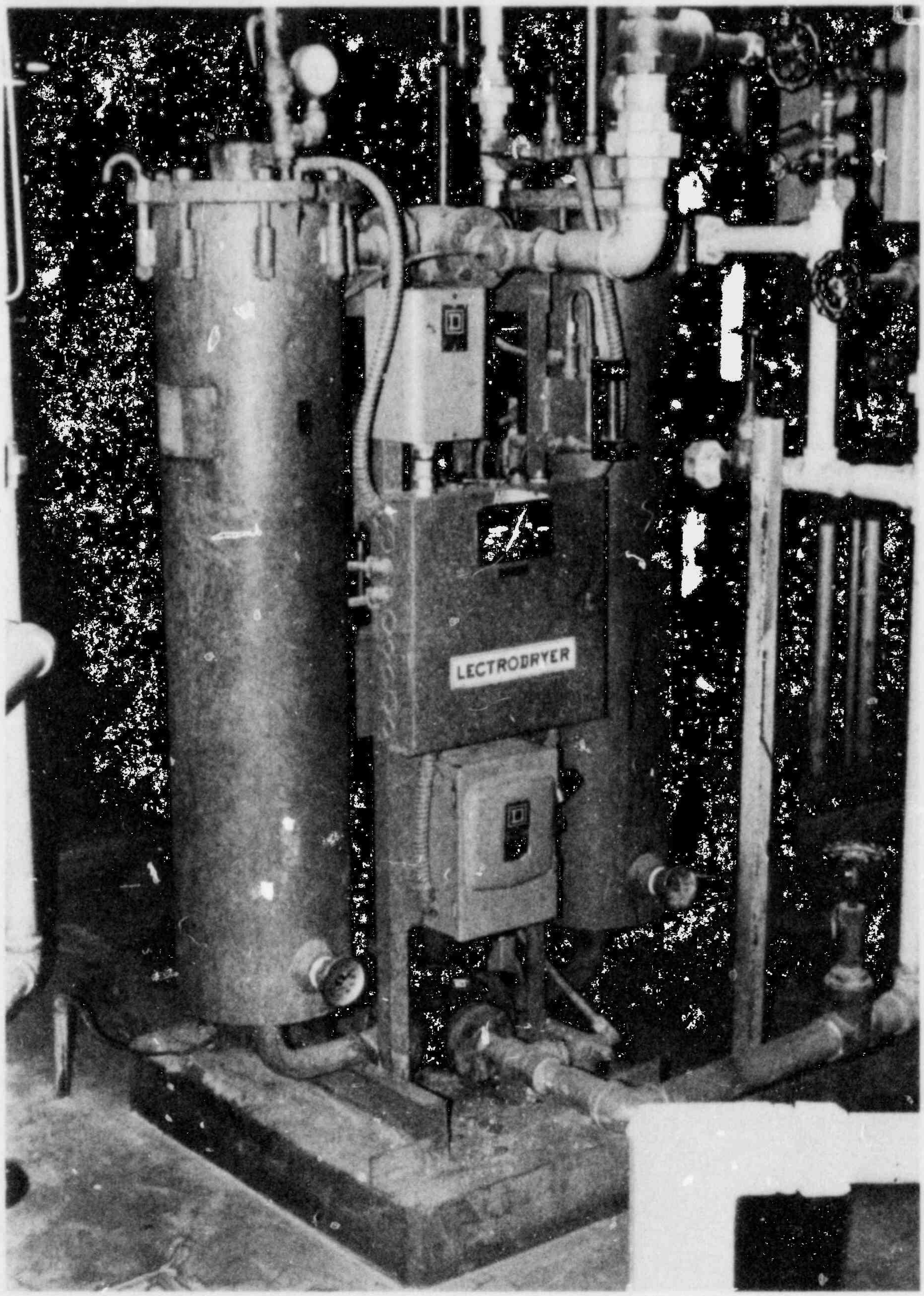
21 Fuel oil circulation pump



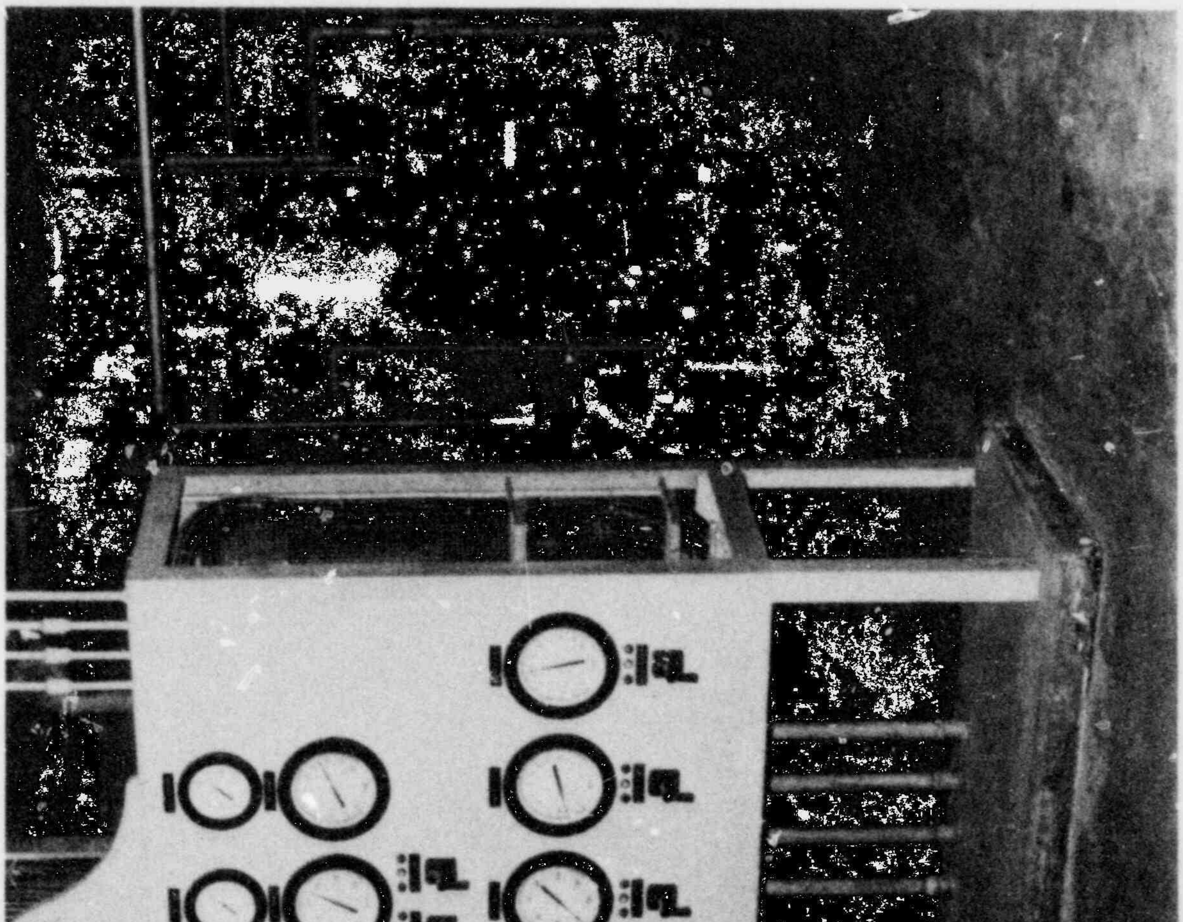
22 Pre-boiler chemical feed tank



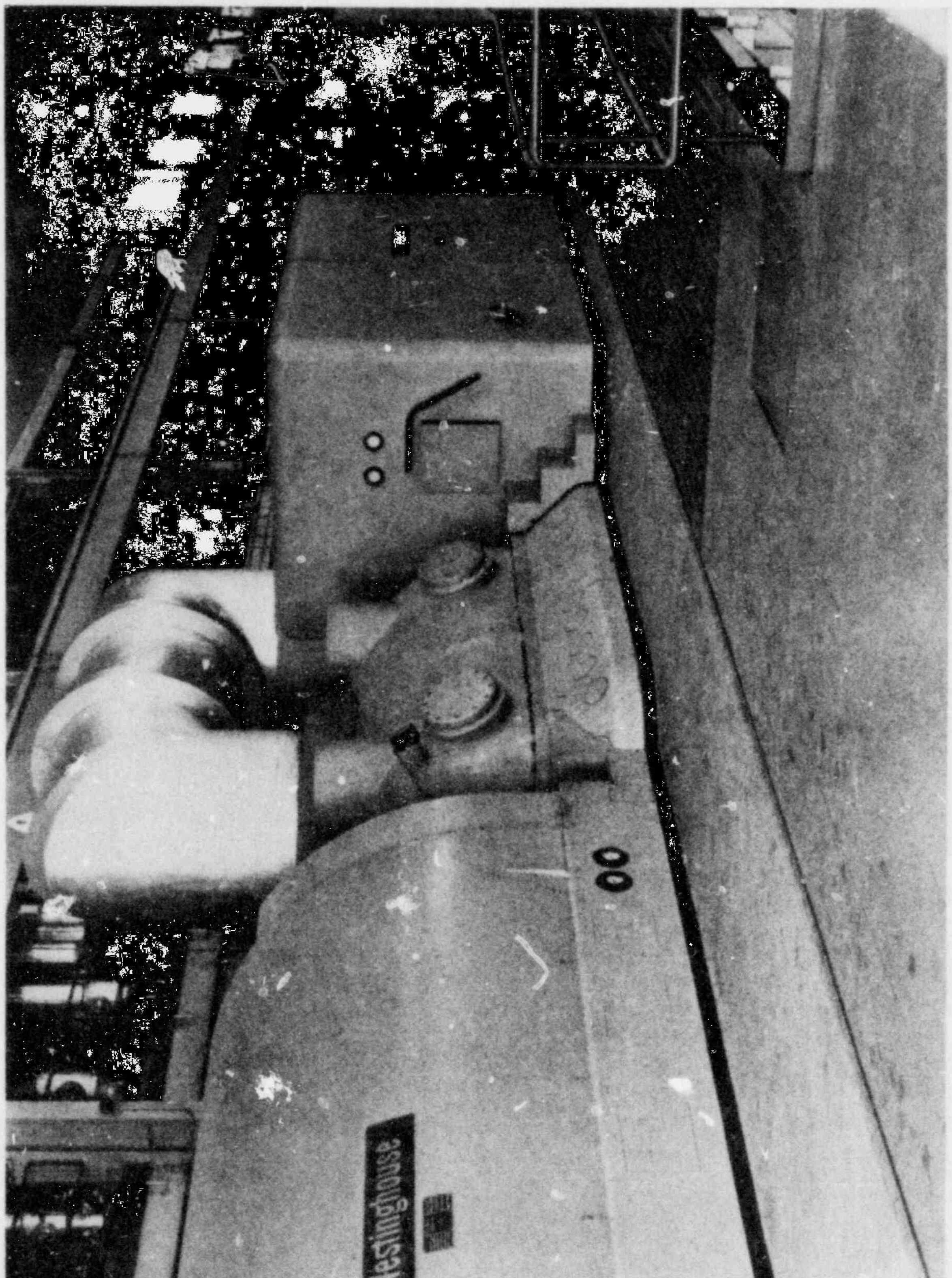
23 Instrument air compressor



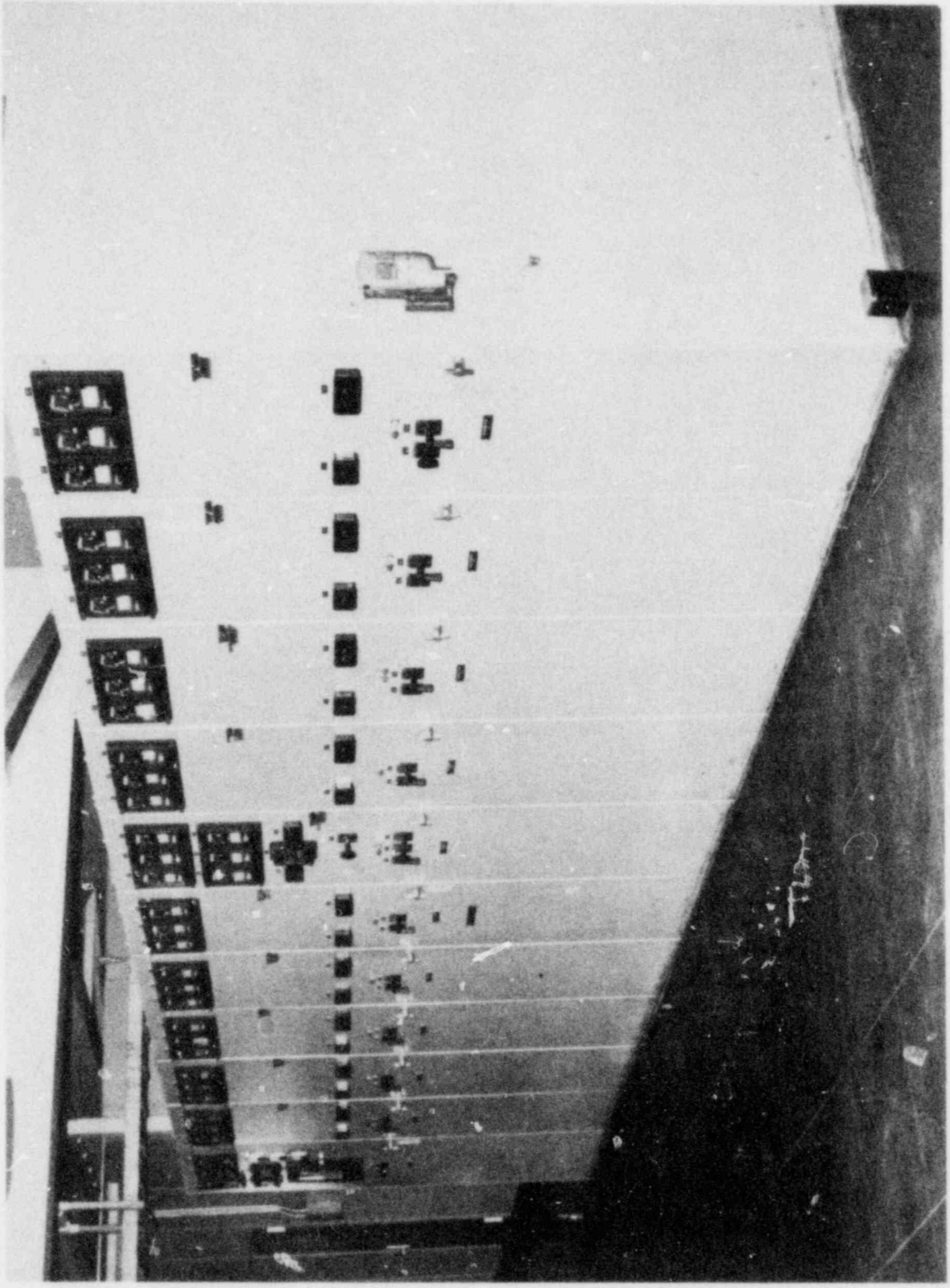
24 Instrument air dryer



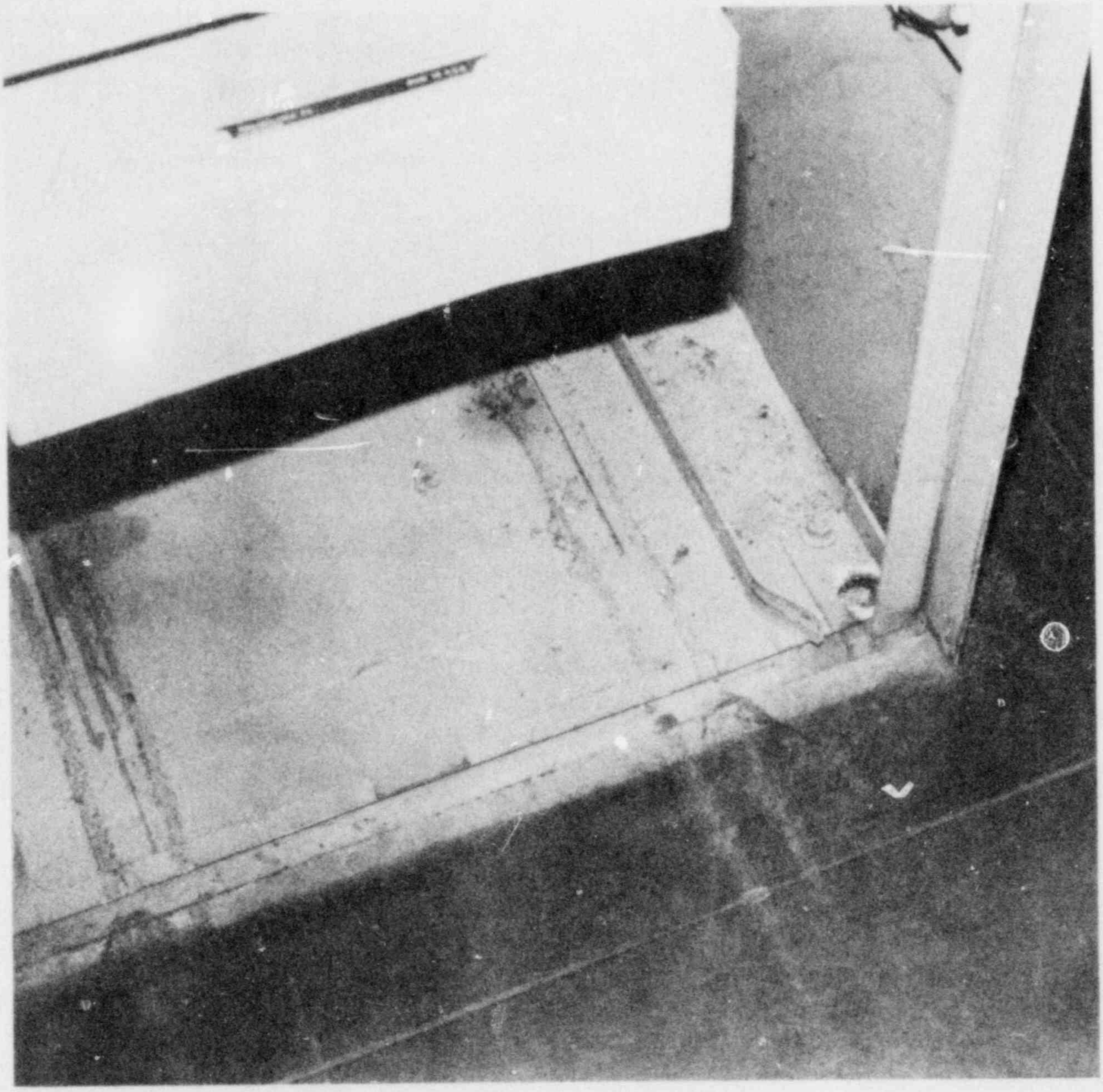
25 Pump control panel



Turbine-generator unit with exciter



26 24-kV unit switchgear



26 24-kV unit switchgear anchorage

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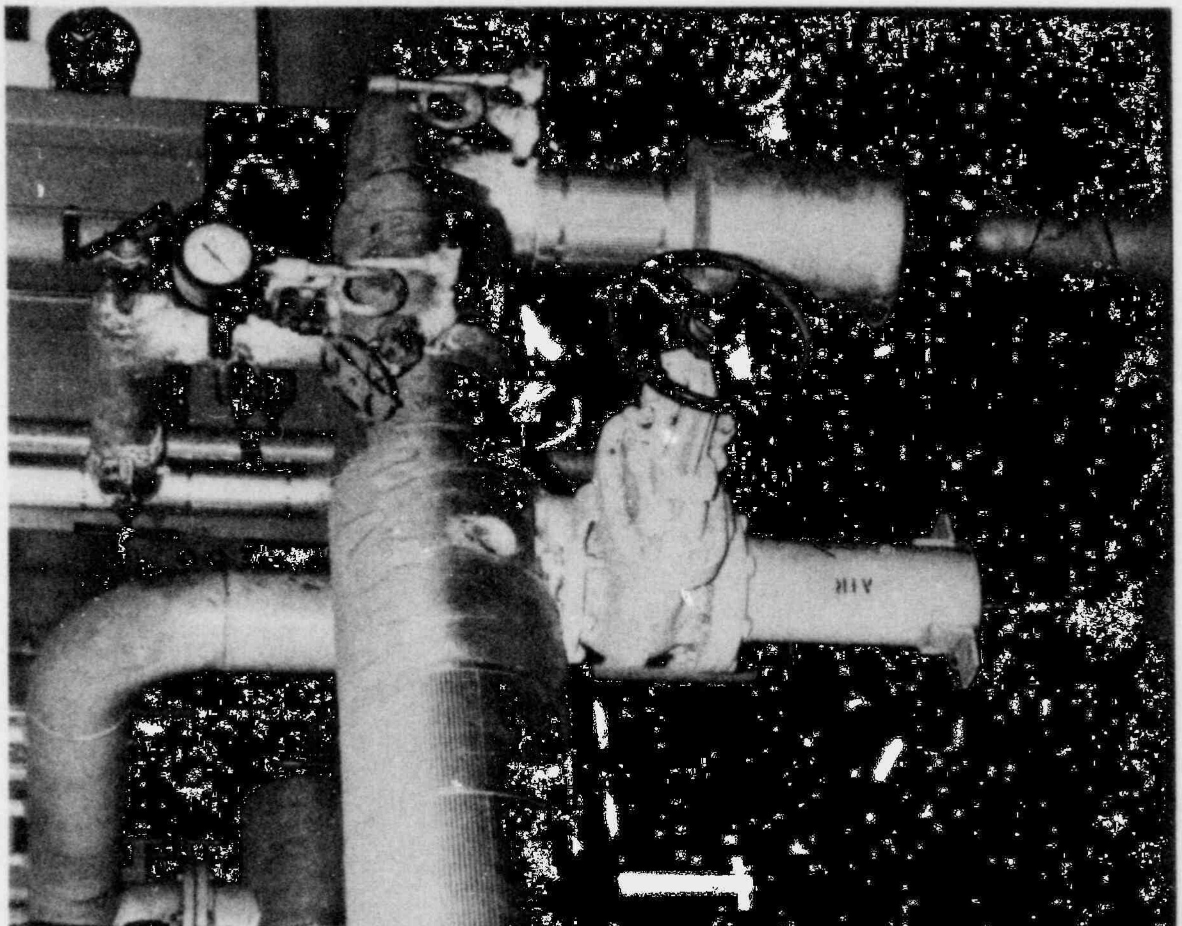
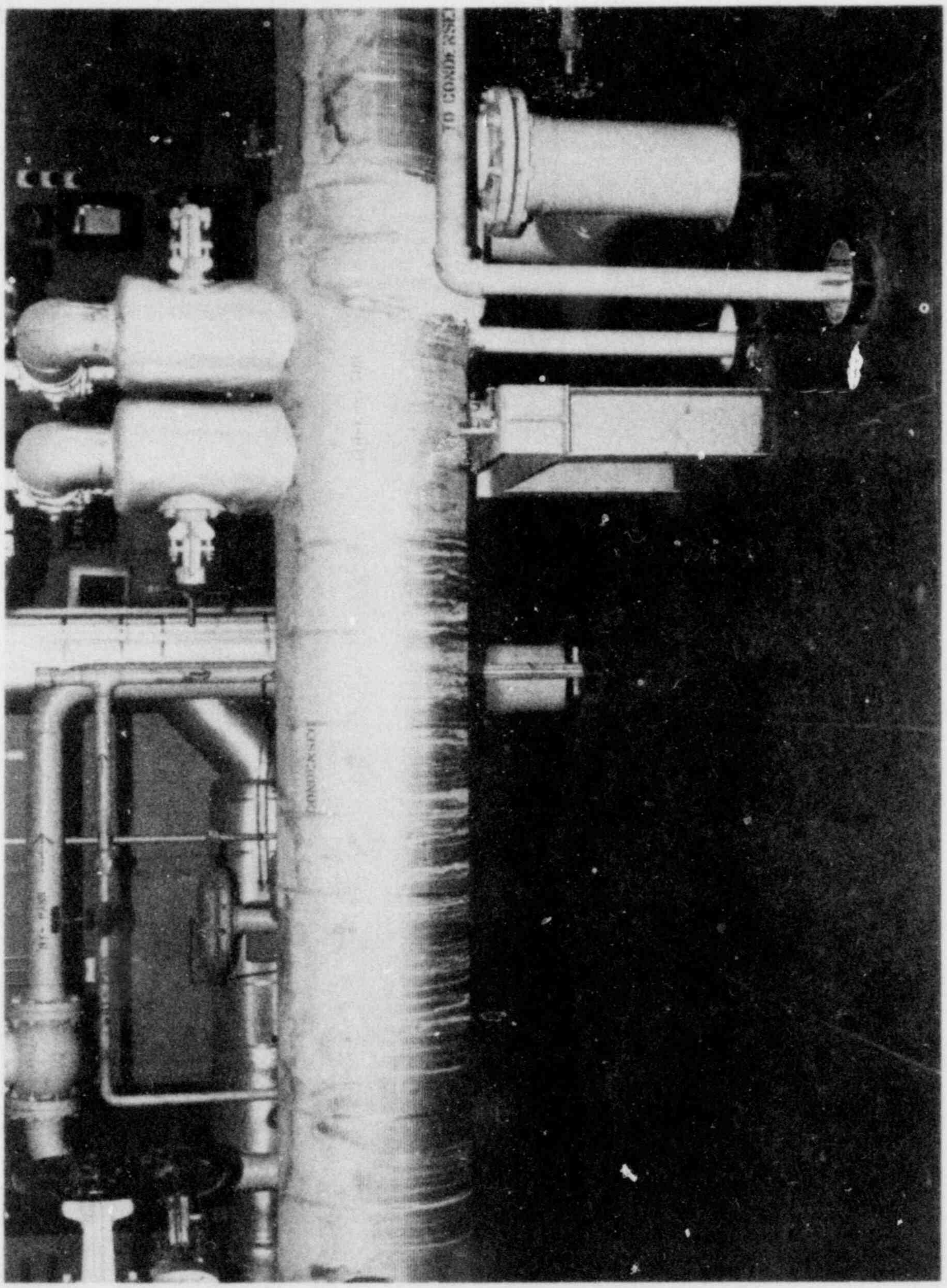


27 Hogging jet ejector



28 Steam jet air ejector

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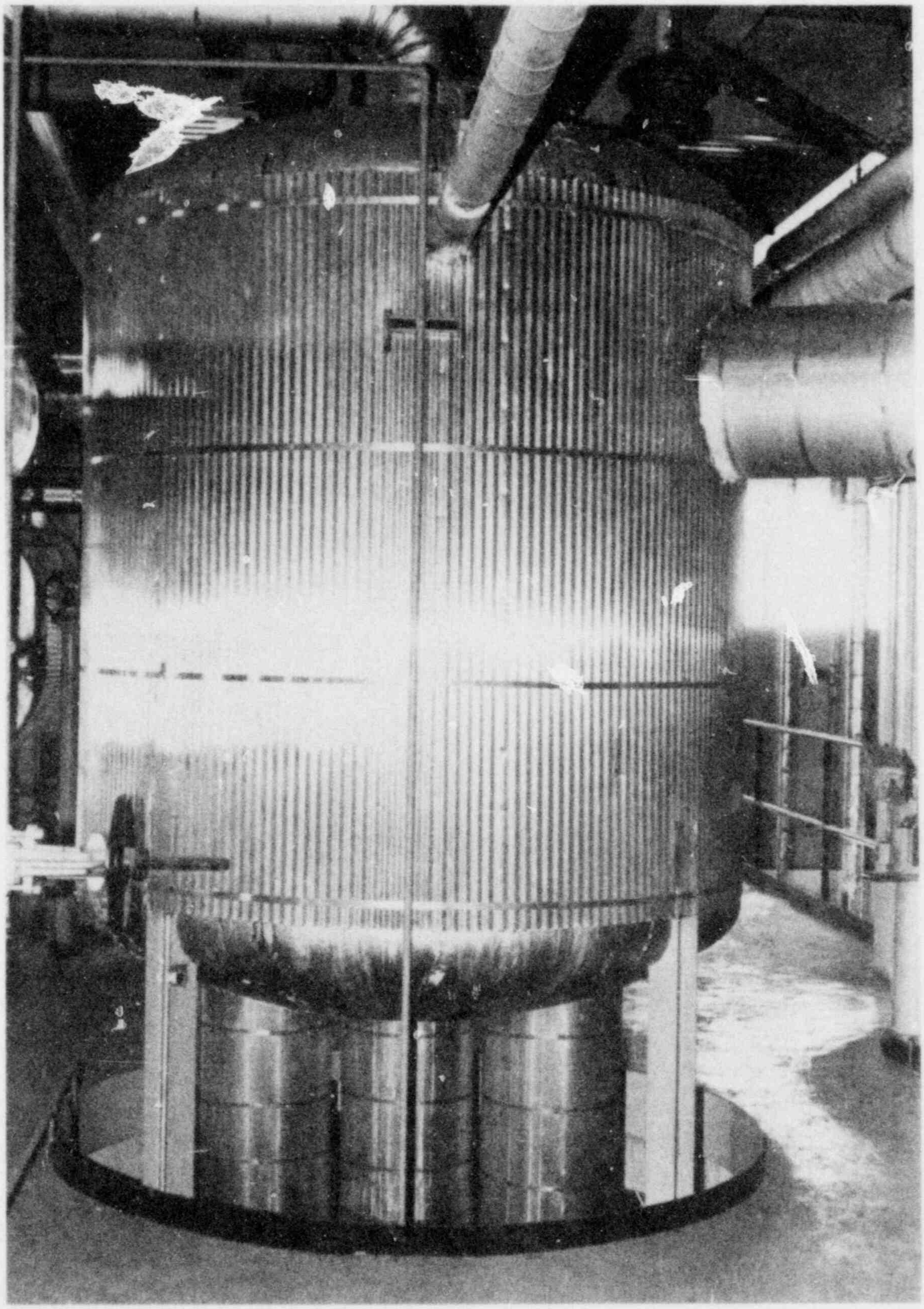
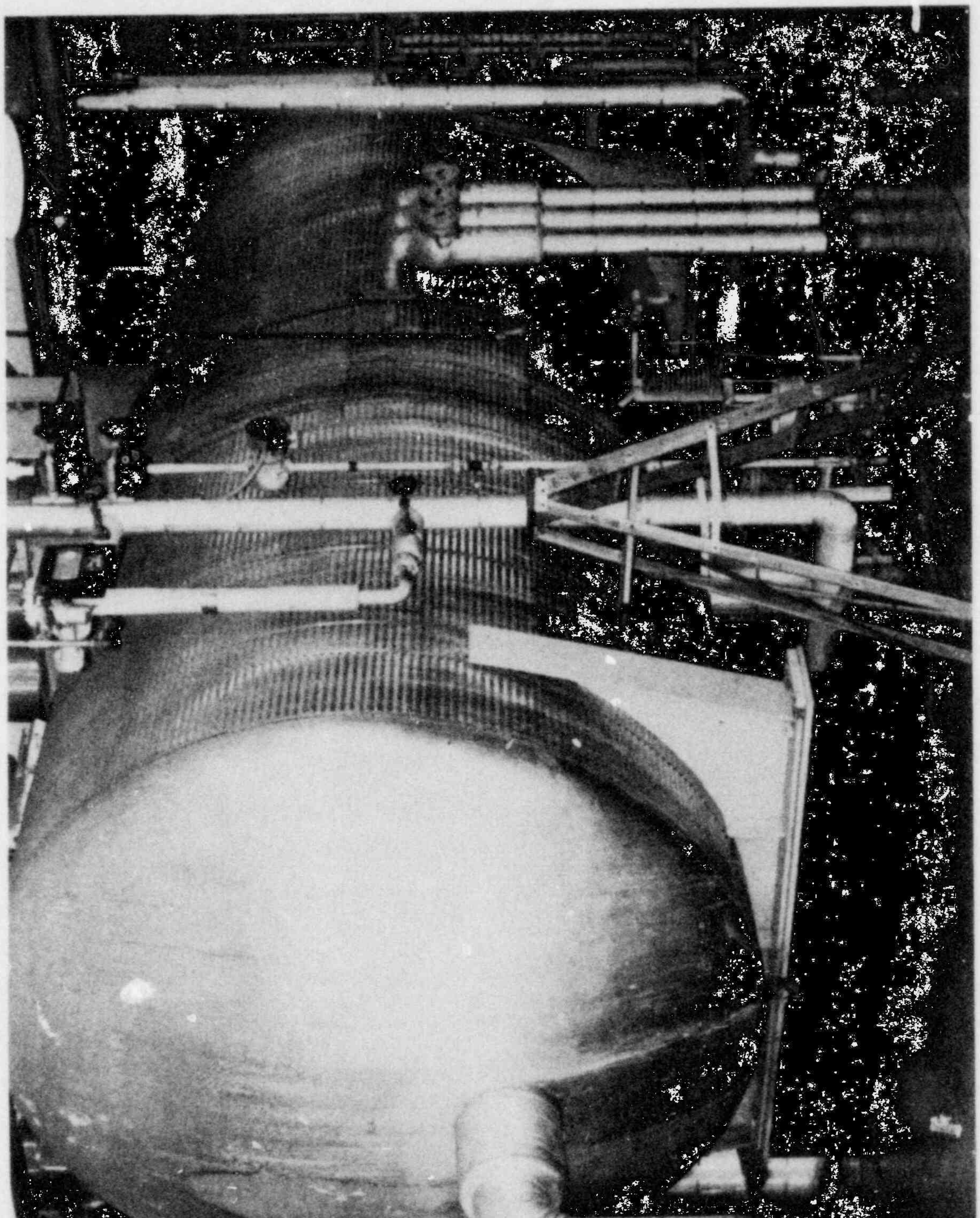
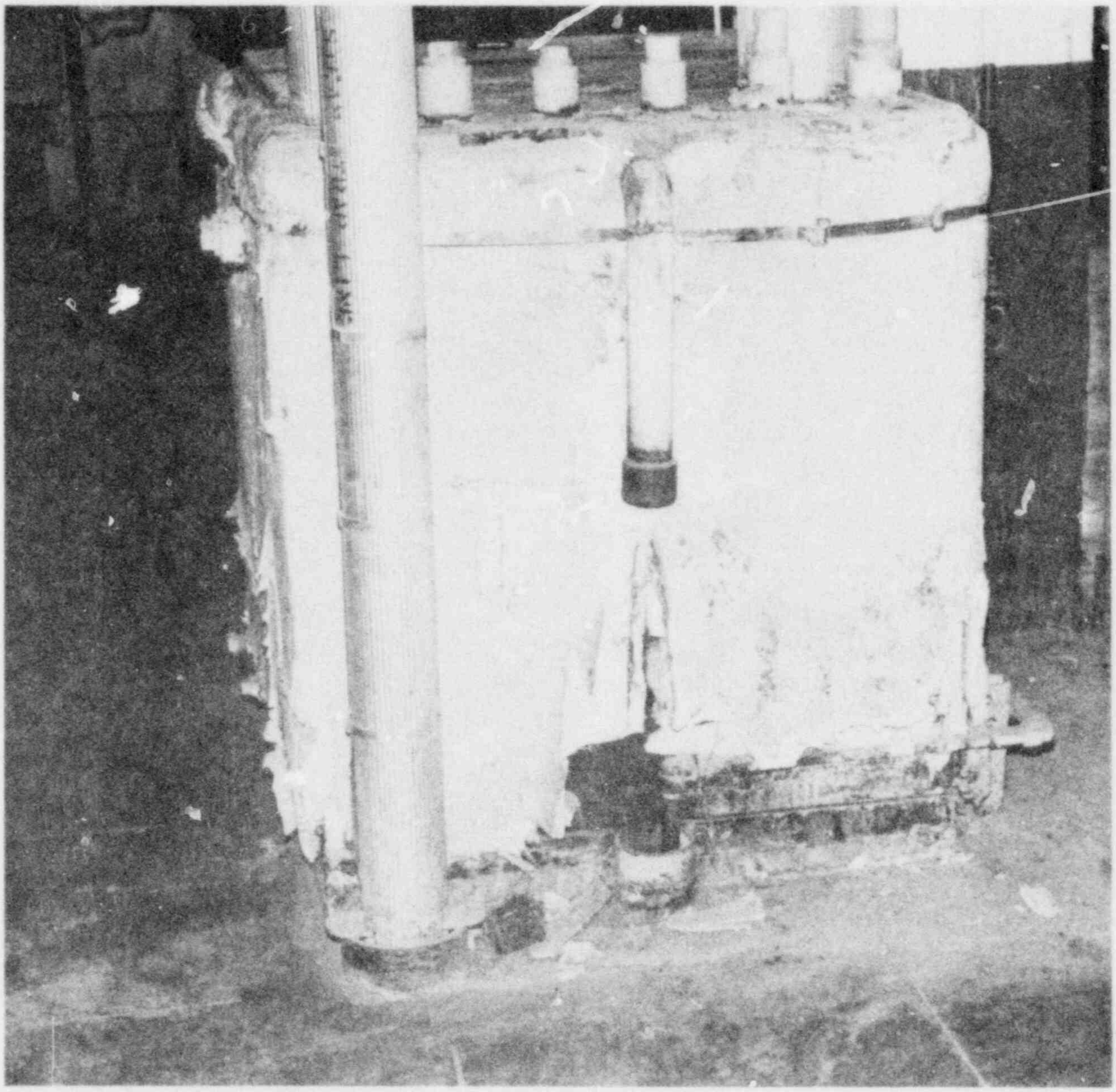


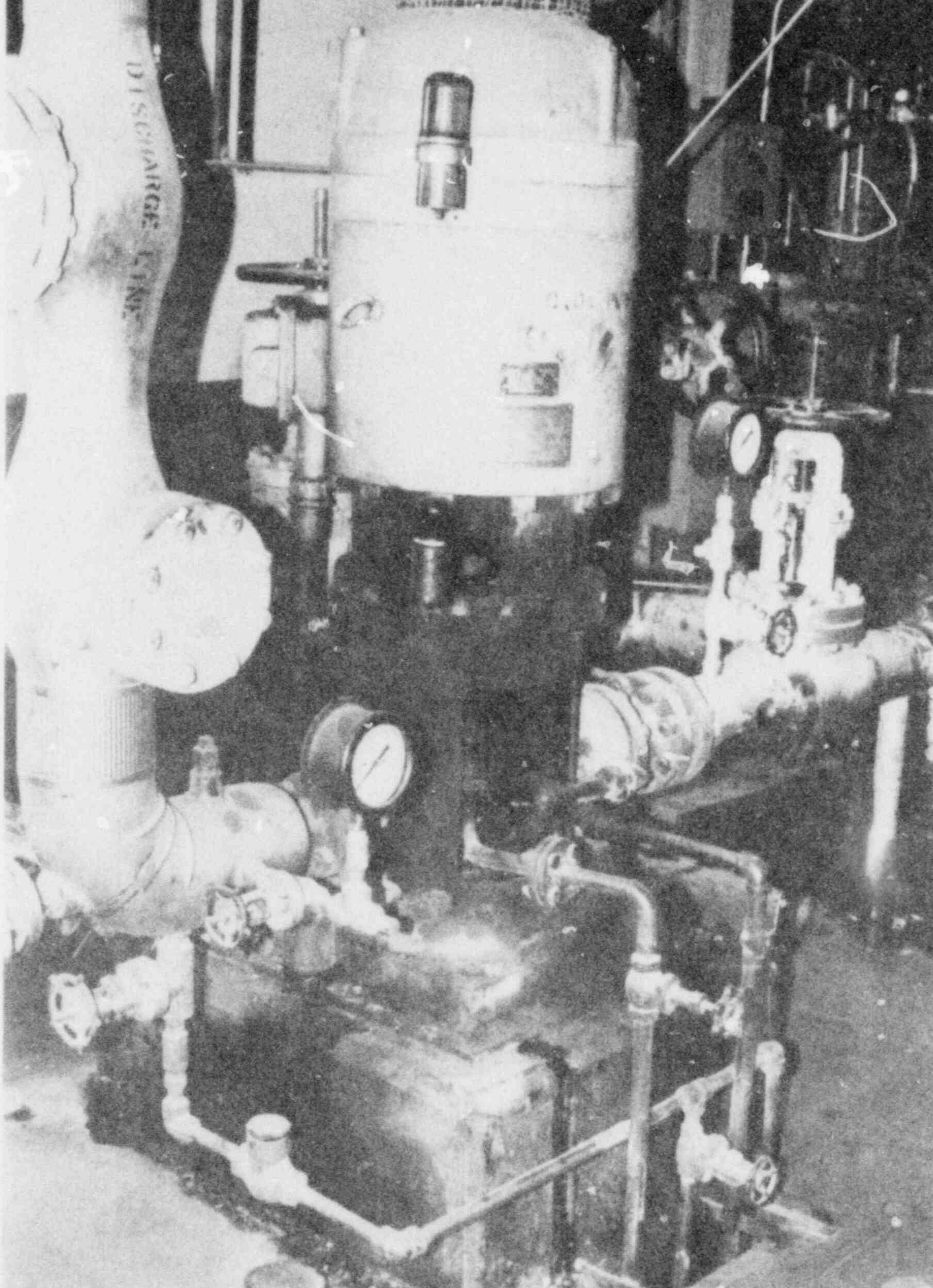
29 Deaerator



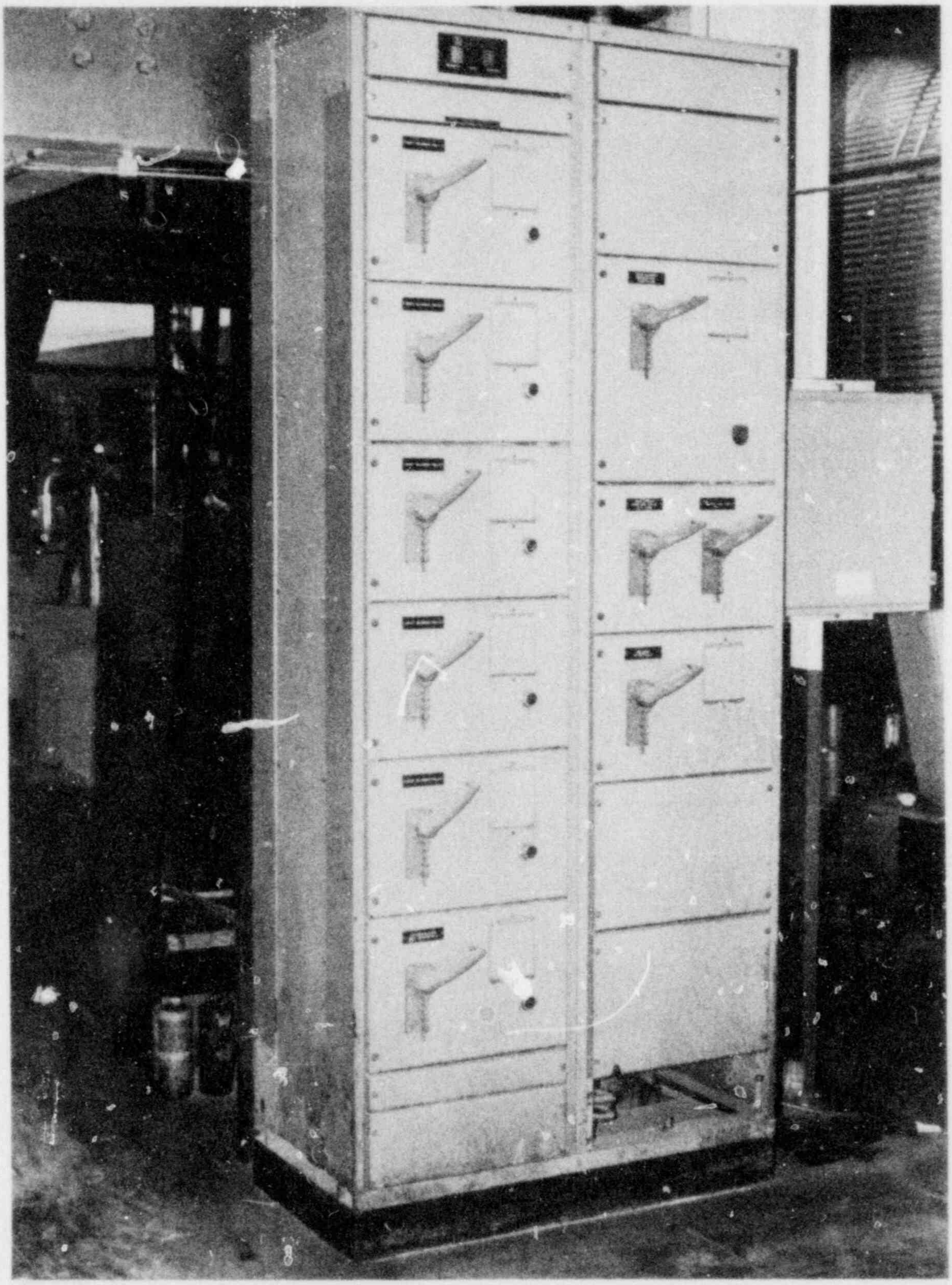
29 Deaerator storage tank



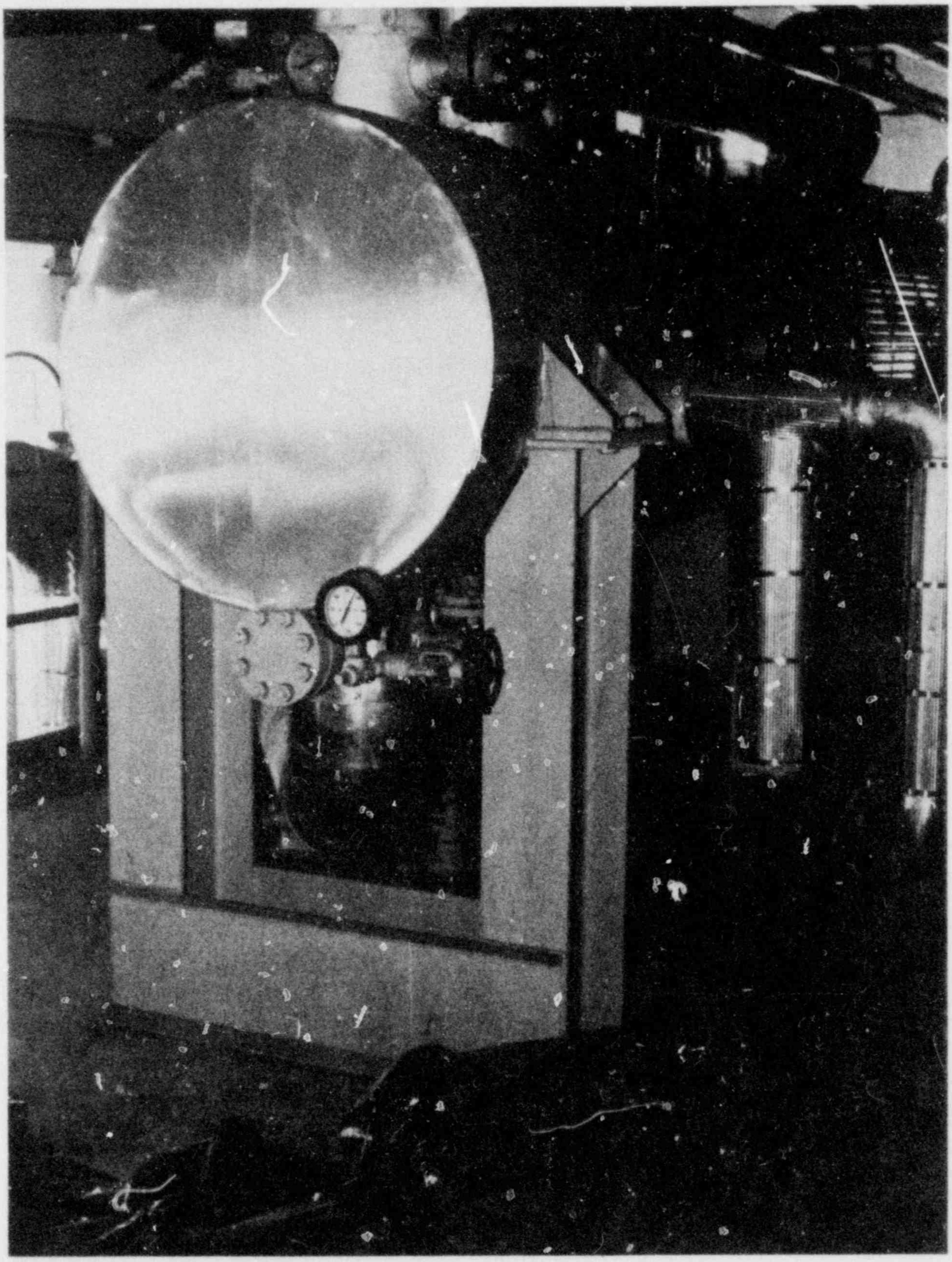
30 Condensate return tank



33 Differential oil pump

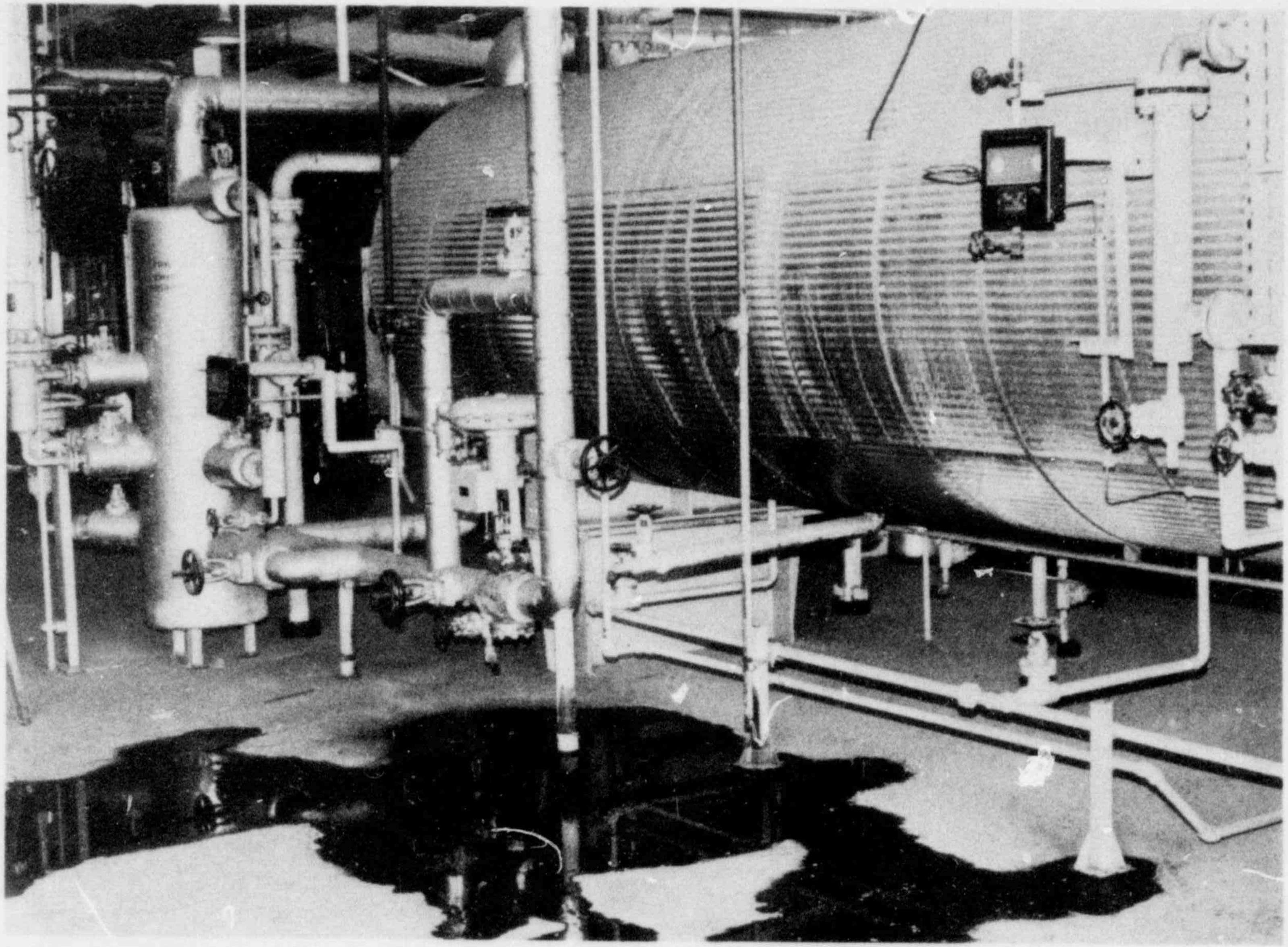


34 Motor control center No. 4-3

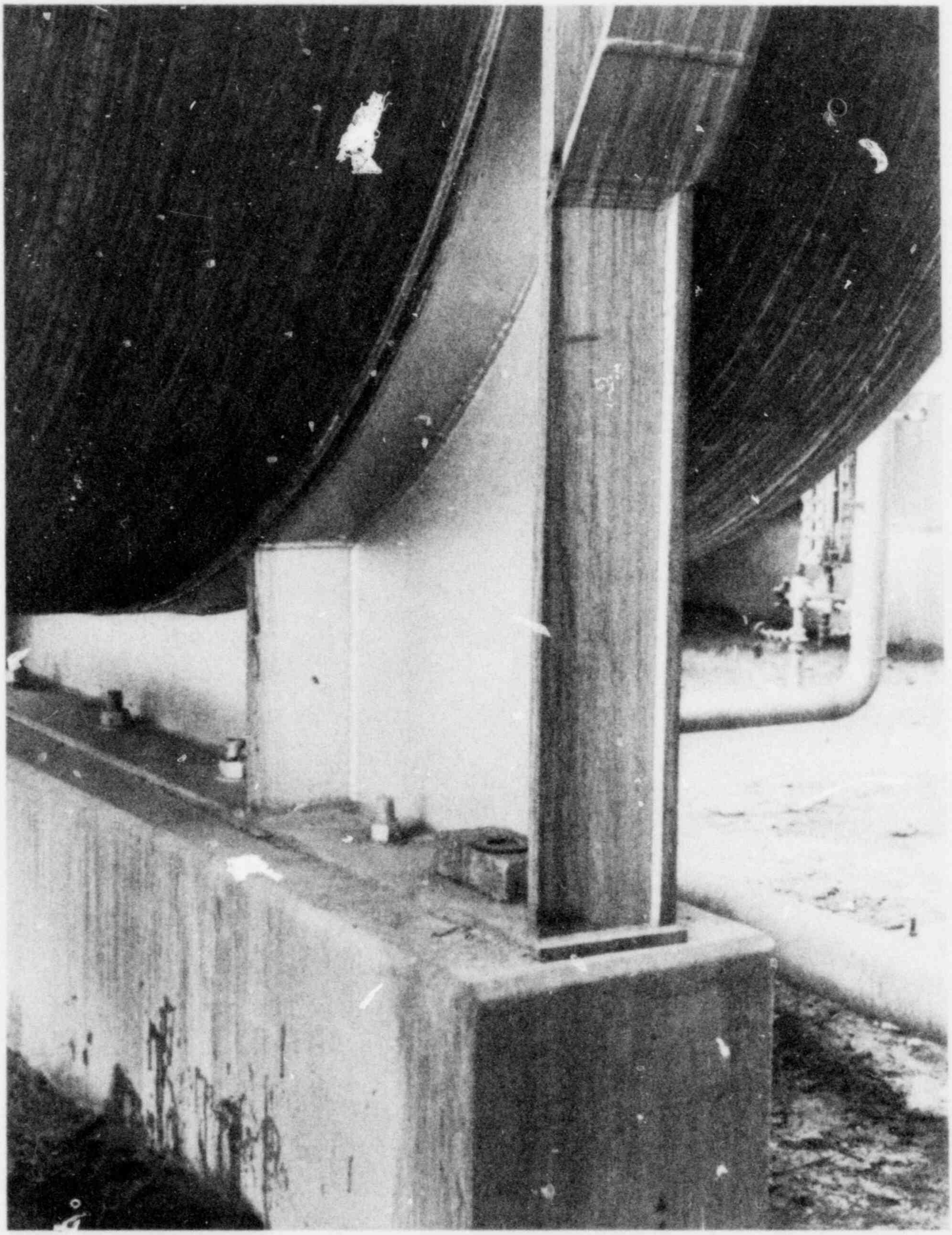


35 High-pressure feedwater heater

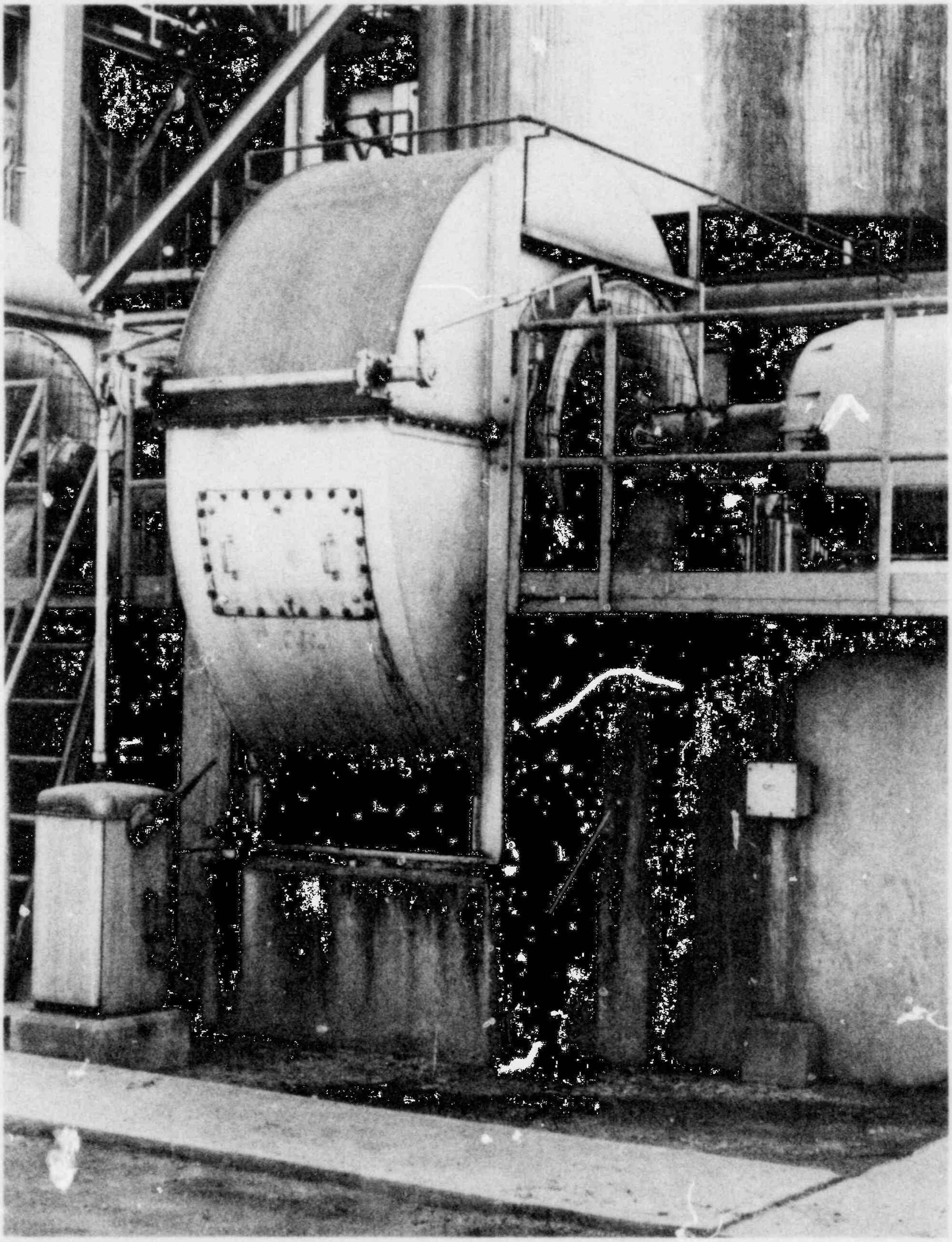
35 High-pressure feedwater heater anchorage



36 Evaporator



Condensate surge tank (ground-supported)



Forced draft fans (ground-supported)

APPENDIX A: SOIL CONDITIONS

The soil at the site consists of very deep alluvial deposits composed primarily of stiff to hard clay interlain with laminations of silty clay loam and sandy loam (Fig. A-1).[A-1] Several sources of data were used to estimate the shear wave velocity to be used in calculating the soil stiffnesses:

- Dames & Moore report (Ref. A-1) indicates a shear strength of around 1500 psf (Fig. A-2).
- USGS report (Ref. A-2) on the differential array (see Table A-1).
- NUREG-0029, Vol. 1 (Ref. A-3), which indicates a shear wave velocity of about 650 ft/s at 10^{-2}% shear strain. These tests were for the soil at the Imperial Irrigation District Terminal Station located at the corner of Commercial Avenue and Third Street in El Centro, approximately 1 km from the plant (see Figs. A-3, A-4, and A-5).

We decided that a shear wave velocity of 650 ft/s would be representative of the site. This value is based primarily on the in-situ impulse test in Fig. A-5 from soil conditions deemed representative of the plant. It represents an average value to a depth equal to the width of the foundation. This value results in a large strain shear modulus of 1600 ksf.

TABLE A-1. Low-strain shear wave velocities at El Centro, from Ref. A-2. Note: Shear wave velocities were obtained from field, downhole geophysical measurements at strain levels on the order of 10^{-4}%. Velocities were averaged for the indicated test intervals.

Test depth interval, ft	Average shear wave velocity, ft/s
0-16	400
16-32	550
32-72	700
72-116	850
116-225	1000
225-271	1150
271-344	1320
344-390	1450

APPENDIX A REFERENCES

A-1. Dames & Moore, Report of Soil and Foundation Investigation, El Centro Steam Station and Switching Station, El Centro, California (1946).

A-2. G. Noel Bycroft, El Centro California Differential Ground Motion Array, USGS Open-File Report No. 80-919 (1980).

A-3. Shannon & Wilson, Inc., and Agbabian Associates, Geotechnical and Strong Motion Earthquake Data from U.S. Accelerograph Stations, Ferndale, Cholame, and El Centro, California, NUREG-0029 (1976) Vol. 1.

A-4. Hall, Richart, and Woods, Vibrations of Soils and Foundations (Prentice Hall, Inc., 1970).

A-5. NCT Engineering, Inc., The Role of Radiation Damping in the Impedence Function Approach to Soil-Structure Interaction Analysis, Lawrence Livermore National Laboratory, Livermore CA, UCRL 15233 (1980).

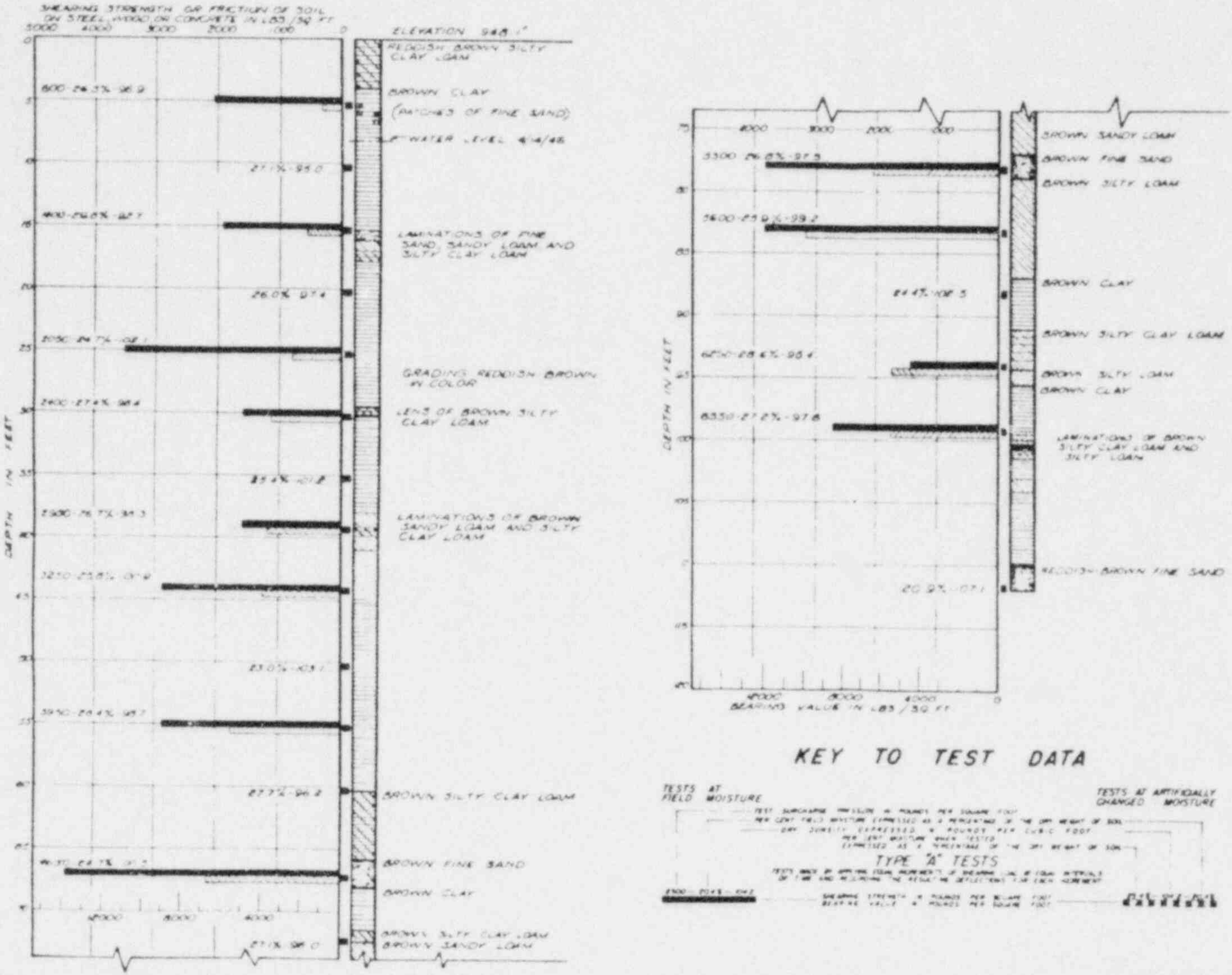


Fig. A-1. Log of test boring No. 3 at the site nearest Unit 4, from Ref. A-1.

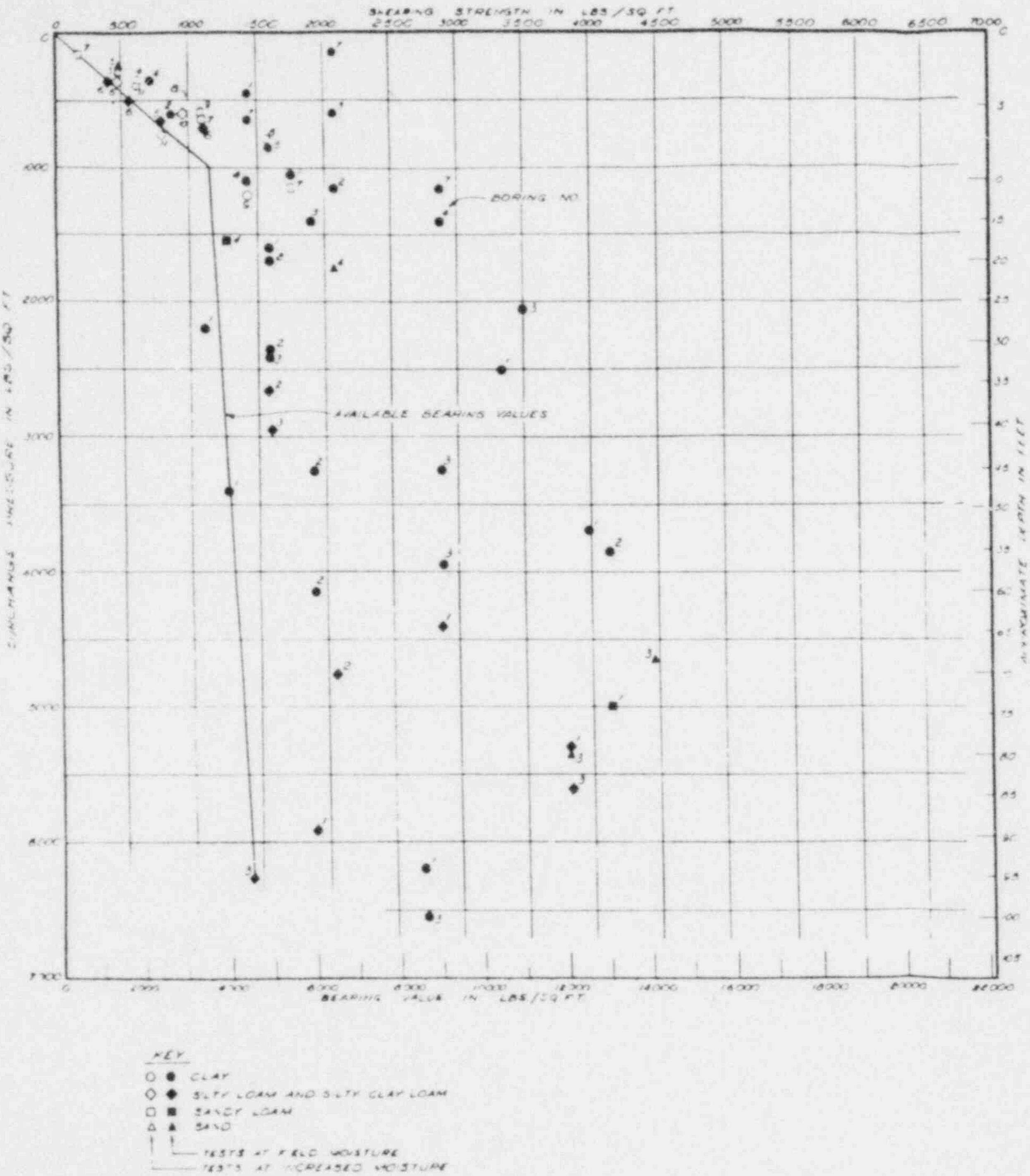


Fig. A-2. Shear strengths determined from laboratory tests of soil at the site, from Ref. A-1.

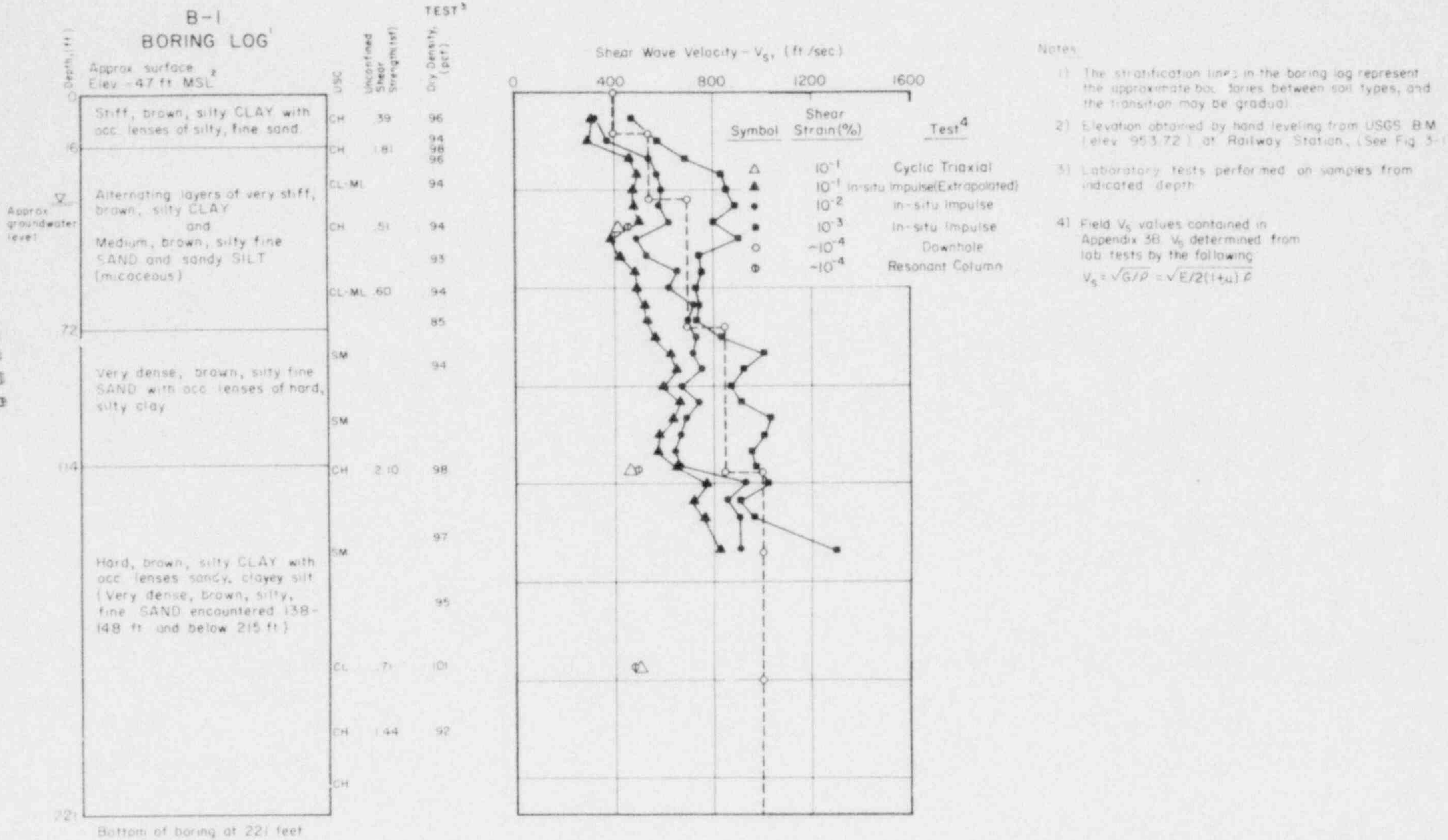


Fig. A-3. Log of test boring No. B-1 at the Imperial Irrigation District Terminal Station located at the corner of Commercial Avenue and Third Street in El Centro, from Ref. A-3.

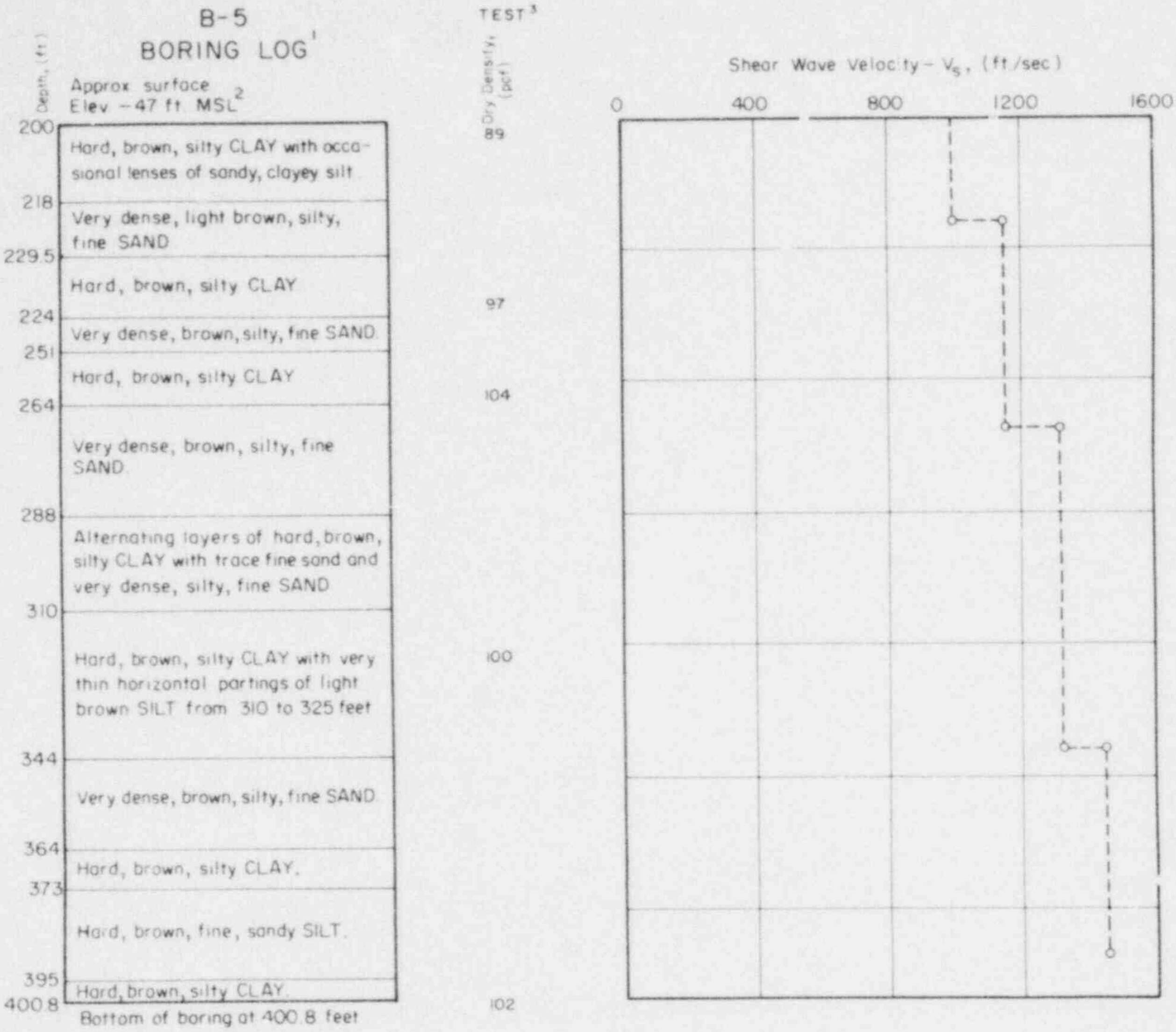
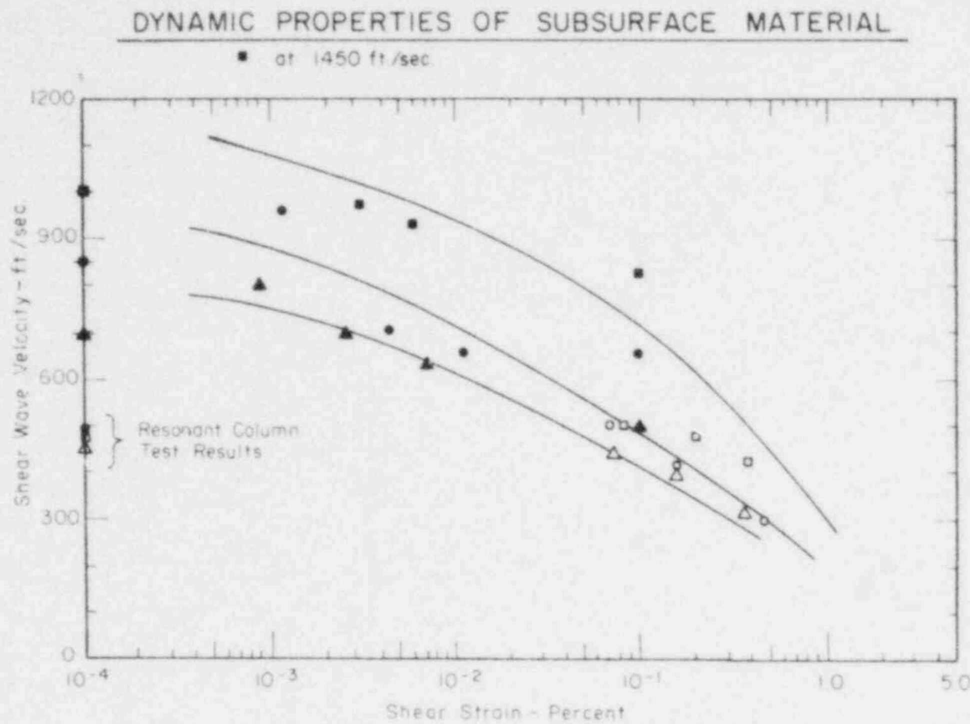


Fig. A-4. Log of test boring No. B-5 at the Imperial Irrigation District Terminal Station located at the corner of Commercial Avenue and Third Street in El Centro, from Ref. A-3.



Legend

Nominal Test Depth (feet)	Laboratory: Resonant Column 1)	Laboratory: Cyclic Triaxial 1)	Field: Down hole 2)	Field: In-situ Impulse
40	△	△	▲	▲
115	◉	○	●	●
175	▣	□	●	■

Notes:

1) Laboratory Resonant Column and Cyclic Triaxial Test results have not been corrected for possible sample disturbance. Shear wave velocity (V_s) determined for laboratory tests by the following:
$V_s = \sqrt{G/\rho} = \sqrt{E/2(1+\mu)\rho}$

2) Downhole Geophysical measurements made at about 10^{-4}% strain.

Fig. A-5. Recommended dynamic properties of subsurface material based on Figs. A-3 and A-4, from Ref. A-3.

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16. ABSTRACT (200 words or less)

For the U. S. Nuclear Regulatory Commission (NRC), Lawrence Livermore National Laboratory (LLNL) performed a dynamic seismic analysis of Unit 4 of the El Centro Steam Plant in El Centro, Calif. Built in 1968, Unit 4 is an oil-or-gas-fired, steam-driven turbine-generator that was designed to resist a static lateral force equivalent to 20% of the dead and live load applied laterally. However, the unit's structural and mechanical systems sustained only minor damage during the October 15, 1979 Imperial Valley earthquake that produced an estimated 0.5 g peak horizontal ground acceleration (0.6 g vertical) at the site. LLNL's seismic analysis was done to analytically estimate the equipment response, which, when compared to actual observation, will indicate the levels of actual equipment capacity.

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