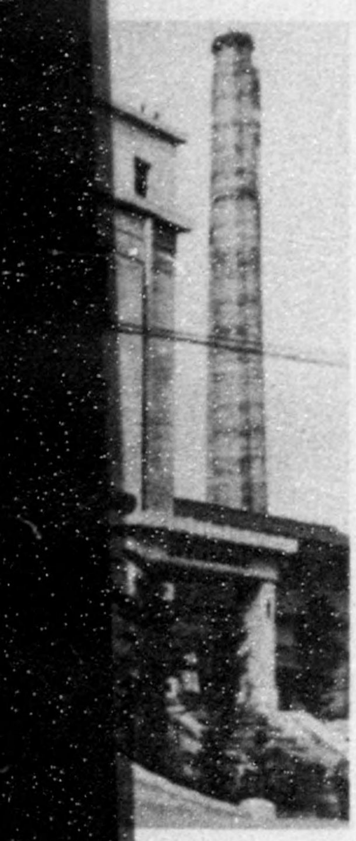
construction faults were low strength concrete, improper placement of reinforcing steel, and the formation of laitance and aggregate segregation.

The inspection of many of the buildings which suffered little or no structural damage showed that the designer had taken into account the effect of lateral forces and that the construction was conscientiously done. Approved and tested methods exist which, if used in designing, will result in a design offering high resistance to earthquake forces. The greatest difficulty seems to be in getting engineers to use them.

C. STEEL CONSTRUCTION

1. GENERAL - For consideration in this report, steel structures in the damaged area were those buildings entirely of steel, those which were steel framed, and those having only steel roof trusses.



2. ALL-STEEL STRUCTURES

a. Fukui Railroad Freight Station - Buildings north of and adjacent to the Fukui railroad passenger station are freight sheds. A portion of this area is covered by a roof of corrugated steel sheets supported on a steel frame made of discarded lightweight rails. Due to its lightness, and its having been designed with some consideration for lateral forces, the whole structure showed no signs of distress. The concrete slab covered platform under the shed and the nearby pavement were subjected to considerable ground movement, as evidenced

by numerous fissures and upheaved and subsided areas.

b. Municipal Gas Plant - The municipal gas plant located in the eastern part of Fukui City had one portion which was a steel frame structure with walls and roof covered with corrugated steel sheets. This section of the plant housed part of a battery of ovens.

Although the brick work of the ovens was damaged to some extent, the steel building was undamaged, except for the loosening of some of the corrugated sheets. The other section of the plant housing the rest of the ovens, scrubbers, etc., was of brick walls with wood roof trusses and failed almost completely.

The gas plant was equipped with two low pressure gas holders which were constructed of steel and were the wet seal type. Both suffered some cracking of foundation concrete. The tank shown on the left in Fig. 199 is the older of the two tanks and has not been used for some time. The top of this holder collapsed during the earthquake. It appeared from inspection that corrosion was the primary reason for its collapse. The holder to the right suffered no damage except that mentioned to the foundation. The steel building, housing the ovens, can be seen as the right hand building in Fig. 199.

c. Antenna Towers - Fukui Broadcasting Station - The two transmission antenna towers for the Fukui Broadcasting Station are shown in Fig. 200. These towers are each 203 feet

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high. They are located near the broadcasting station, which is a short distance northeast of the prefectural office buildings. The only noticeable damage to these towers was snapping of the antenna. There was evidence of considerable ground movement around the tower bases and considerable subsidence resulted. The reinforced concrete tie beams connecting the tower leg piers held them firmly in alignment. Fig. 201 shows the west tower alignment.

d. Elevated Water Tank - A small elevated water tank, located in the U. S. Army housing compound, was undamaged. The tank, shown in Fig 202, has a capacity of approximately 850 gallons and is approximately 25 feet high. The tank was about one-half full at the time of the earthquake. It is located atop a section of an old moat wall. The ground around the base of the tank was fissured and two of the tower piers were slightly cracked.

e. Miscellaneous - Fig. 203 shows a coal hopper and a traveling gantry crane. These are located close by the roundhouse and repair shops in south Fukui City. The coal hopper shown on the right of the tender was located, before the earthquake, in a position which placed it over the top of the upright tender. It was supported on six columns, each composed of two channel sections latticed together. The hopper perhaps was not sufficiently braced against lateral movement and, being approximately two-thirds full of coal,

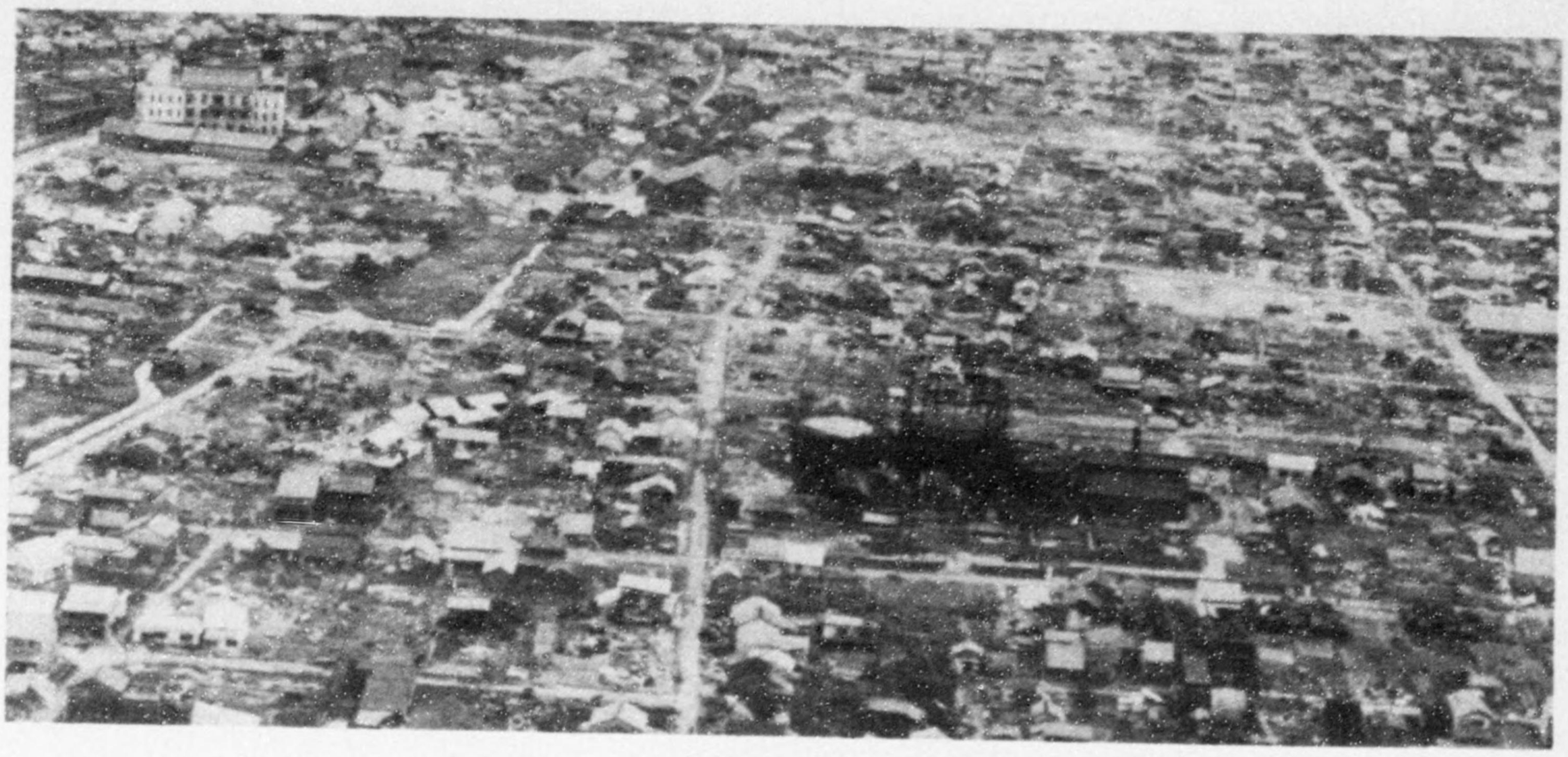

Fig. 199

Aerial View - Fukui Gas Plant
Fukui City

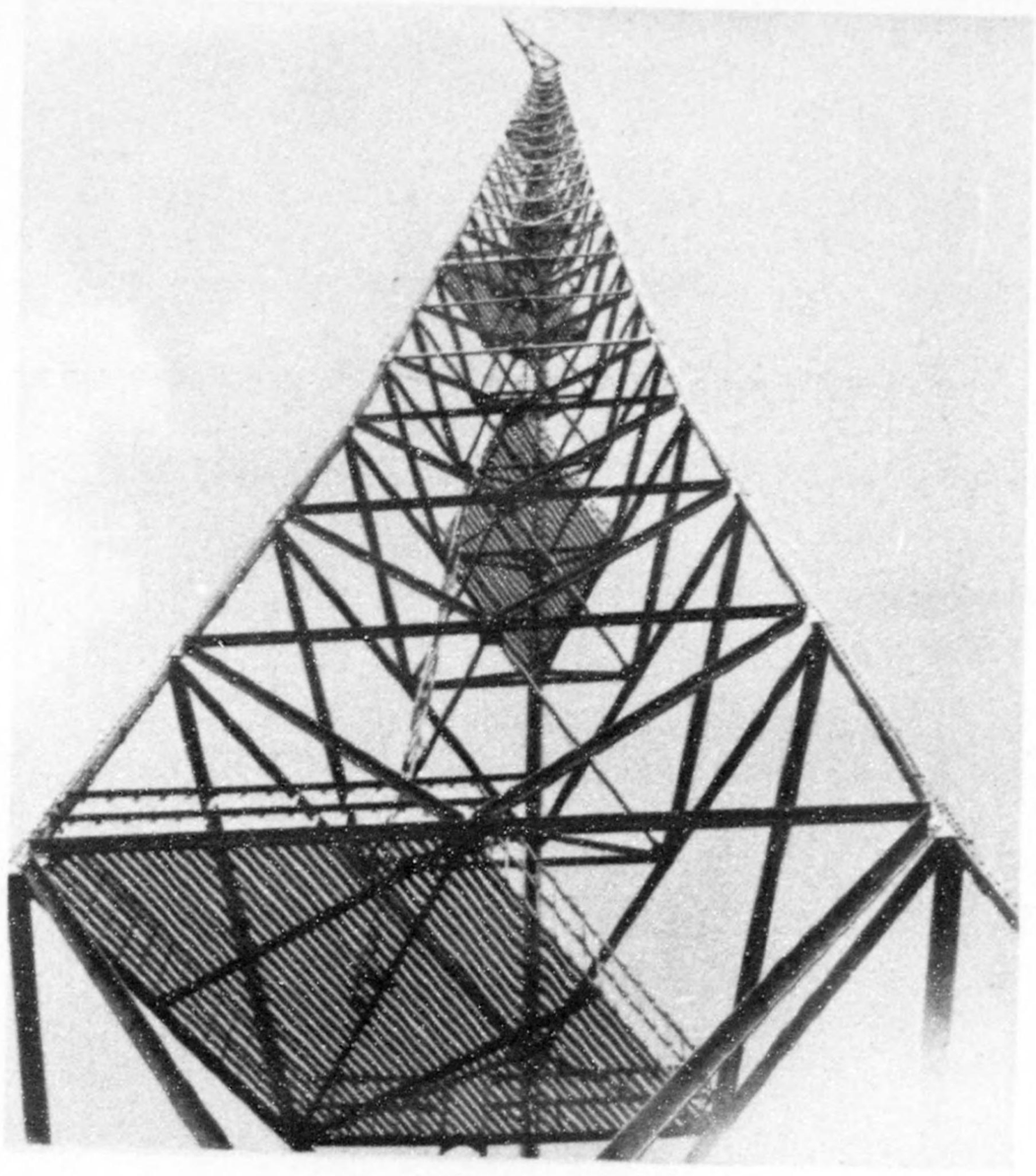

Fig. 201

West Tower - Vertical Alignment
Fukui City

Fig. 202

Small Elevated Wate
Fukui City

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Fig. 200

Radio Antenna Towers
Fukui City

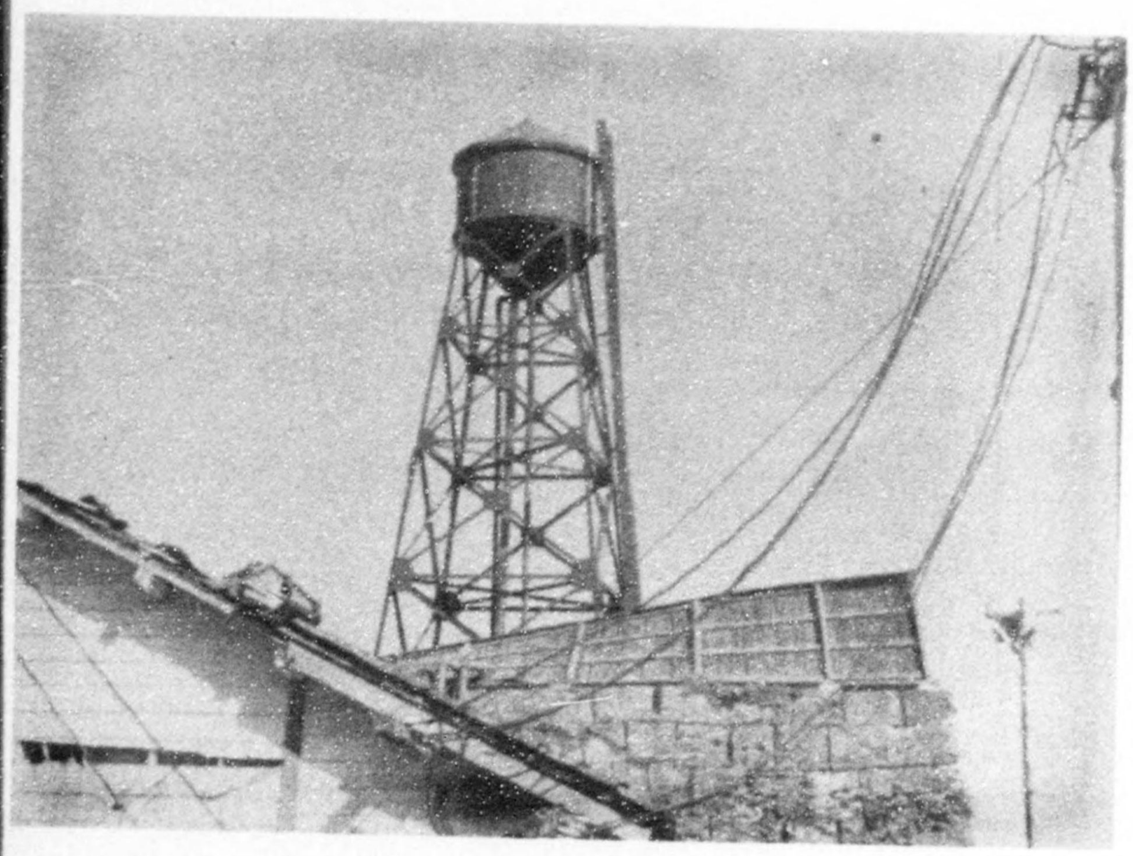

Fig. 202

Small Elevated Water Tank
Fukui City

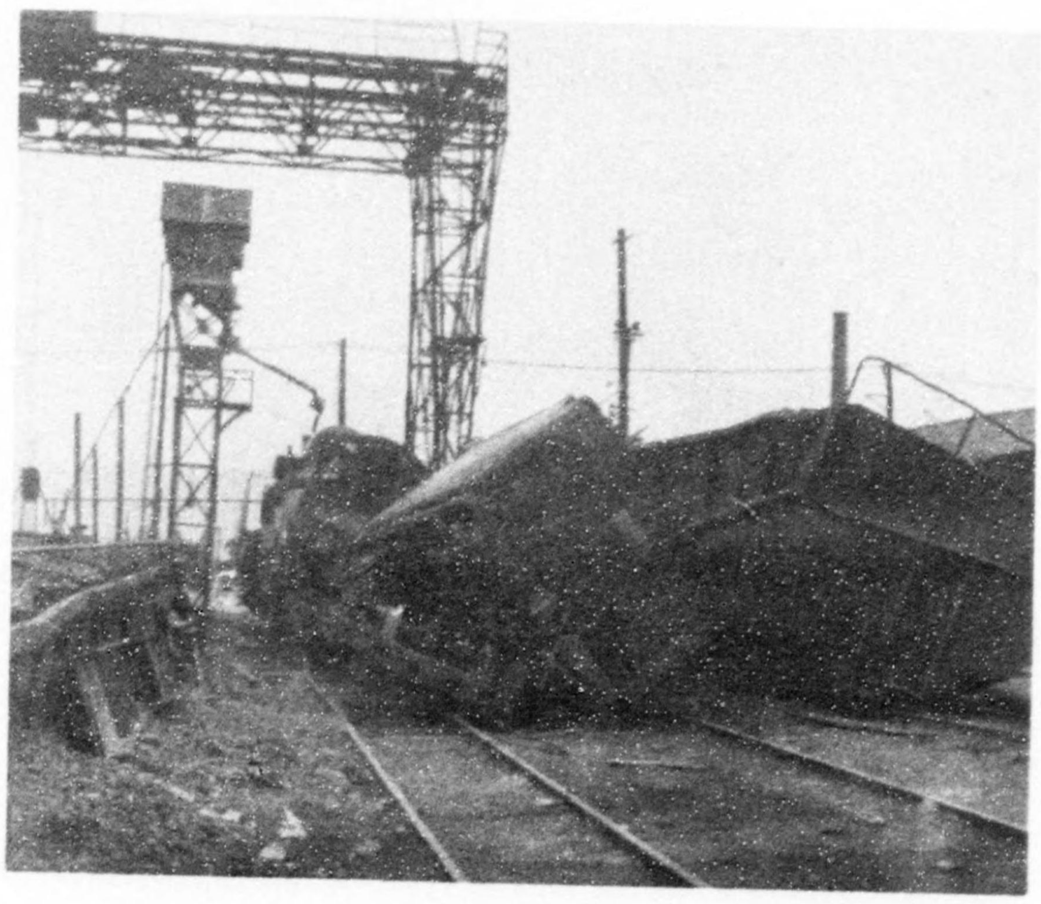

Fig. 203

Fallen Coal Hopper - Railroad Yard
Fukui City

fell on the right edge of the tender tipping it off its trucks. There is also the probability that the tender was bumping the columns and aided in the collapse.

Fig. 204 shows insulator and bus supporting steel framework in a large substation near the town of Matsuoka. It is of note that, although this substation suffered considerable damage to transformers and transformer foundations, the steel received none. There were numerous towers supporting high voltage electrical transmission lines which criss-crossed the heavily damaged area. These towers were, without known exception, singularly undamaged. Fig. 27 shows an aerial view of one of these towers. Fig. 205 is a view of a tower, typical of the 115 foot towers used in this area.

3. STEEL-FRAMED BUILDINGS

a. Steam Generation Plants - There were two steam generating plants in the Fukui area that were notable examples of steel frame construction successfully withstanding the recent earthquake.

The steam plant shown in Fig. 206 is located at the Fukui Seiren factory. The plant building and boiler foundation mat was 3 feet 3 inches thick resting on pine piles 18 feet long. Latticed steel columns supported fink-type, steel trusses with monitor, braces, and purlins. The roof and outside walls were covered with cement-asbestos corrugated sheets. No residual distortion of building steel resulted. The

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photograph shows no distortion in the steel framework supporting the overhead coal bunkers. It should be remembered that in this factory a high percentage of damage was inflicted to wood buildings and reinforced concrete chimneys.

The steam plant at the Daiwa Boseki factory was of similar construction. The three boilers were resting on concrete mat foundations, 2 feet thick, which were supported by pine piles fifteen feet long. The column footings were also supported by similar piles. The building frame consisted of solid shape, steel columns; fink-type, steel trusses, bracing and purlins; and corrugated cement-asbestos sheet roofing and brick filler walls. No structural damage was evident except for slight cracking in the brick filler walls.

b. Other - Three other buildings examined which had steel roof trusses, were the Shochikuya Theater in downtown Fukui City, the Ashikaga municipal waterworks pumping station, and the Hokuriku Light and Power Company office building.

The Shochikuya Theater had steel roof trusses which resisted the earthquake forces without deformation, but failed completely when they were subjected to the fire which consumed the building after the earthquake. Fig. 184 shows the effect of this fire on these trusses and the steel purlins.

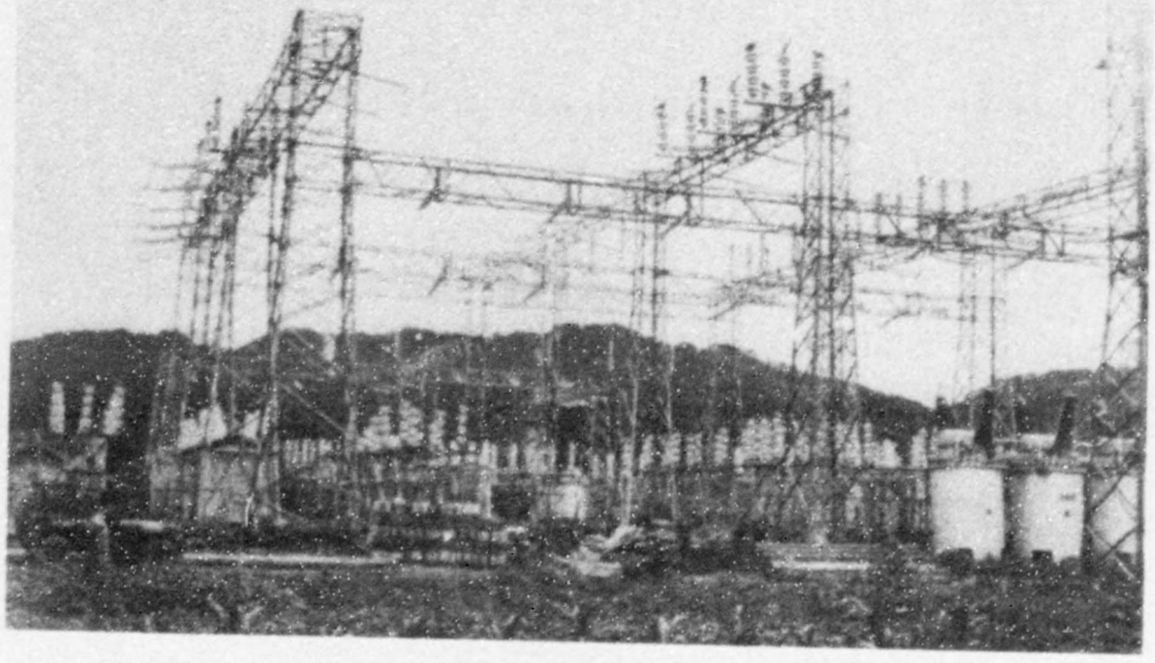

Fig. 204

Bus Supporting Framework
Matsuoka Substation

Fig. 207

Fukui City Gas Plant

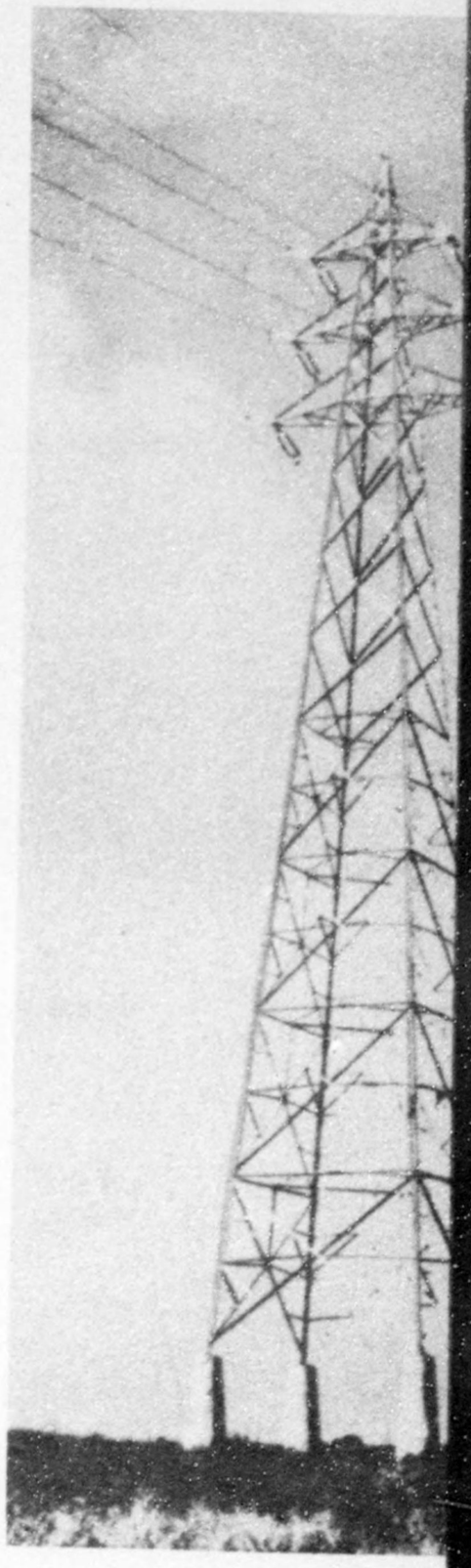

Fig. 205
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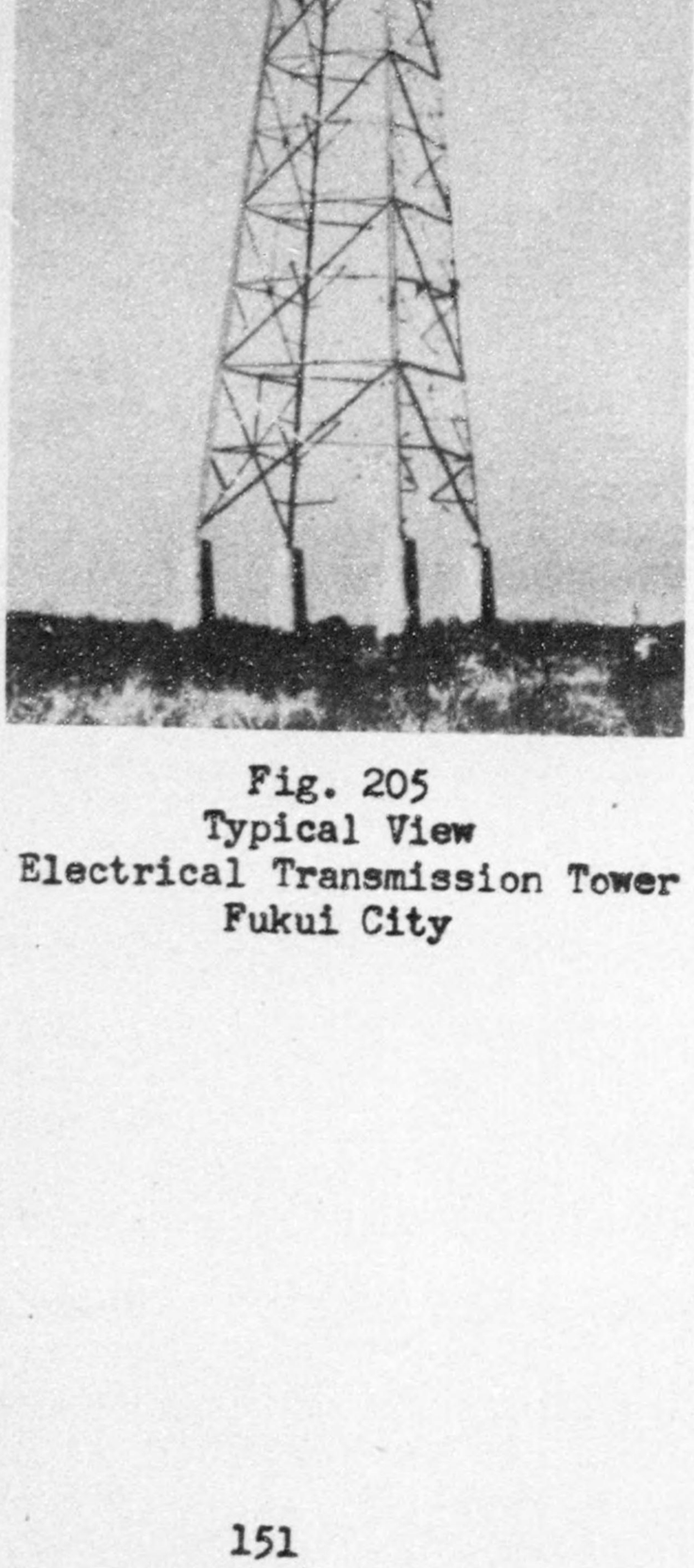

Fig. 205
Typical View
Electrical Transmission Tower
Fukui City

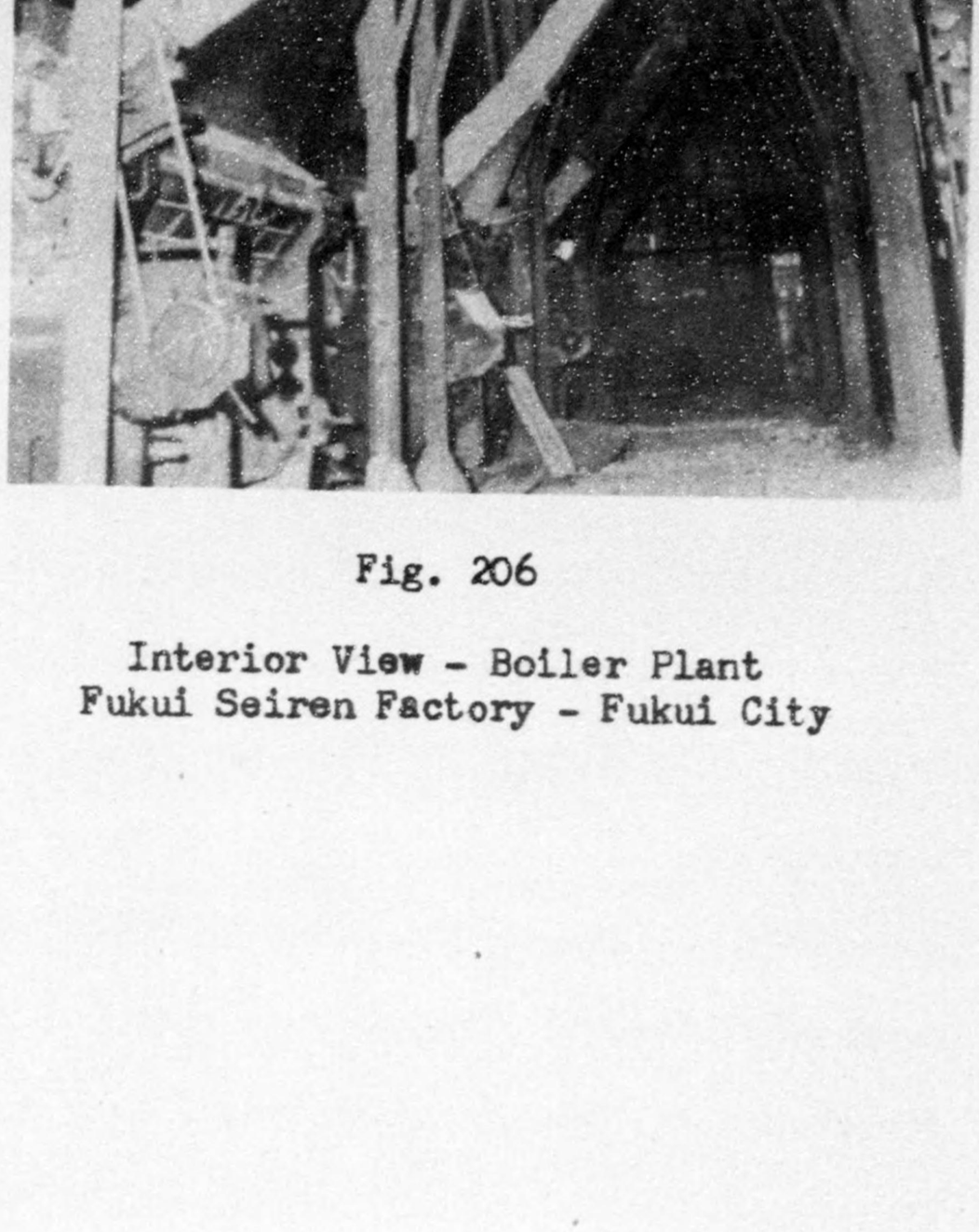

Fig. 206

Interior View - Boiler Plant
Fukui Seiren Factory - Fukui City

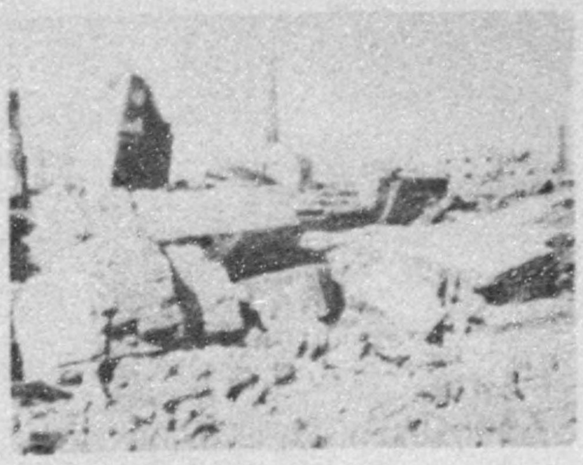

Fig. 208
Failure of Brick Building

The steel roof trusses in the Hokuriku Light and Power Building and the Ashikaga water pumping station withstood the earthquake without distortion. Other portions of the buildings and their equipment were damaged severely. Notably severe column failure in the basement of the Hokuriku Light and Power building occurred.

4. CONCLUSION - Steel construction of all types withstood the earthquake successfully. The one exception of an all-steel structure, the elevated coal hopper at the railroad coaling yard, not withstanding the earthquake is a case primarily of material failure or one of faulty construction. Rather, it could be attributed to poor judgment as to location and design. This success of all-steel construction's withstanding seismic forces only confirms the findings in past earthquakes. The instance of the failure of steel trusses due to fire, after successfully withstanding seismic forces, shows again how necessary it is to provide fireproof casement or protection for structural steel members where they are exposed to possible fire.

D. BRICK BUILDINGS - There were very few brick buildings of considerable size in the area. Within Fukui City itself the court house, a portion of the gas plant, and a bank building

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were the only ones that were noticed. Some of the buildings seen were so completely collapsed that no possibility existed of deducing any primary causes of failure. It was noticed in all of these building failures that the brick walls were the weakest in first, the transverse strengths of the bricks and secondly, the bond and the mortar bedding. No instances of the bricks having been crushed were noted, even though they were generally all of the classification, soft-burned clay.

The Fukui City gas generating plant main building consisted of an old portion constructed of brick and a new portion of steel frame construction with corrugated steel sheet roofing and siding. The brick walled portion of the plant was divided into two sections, one housing a battery of ovens, and the other housing scrubbers, etc. The section housing the scrubbers had a wood trussed roof and the other a steel trussed roof. It appeared, as can be seen in Fig. 207, that what failure of the walls occurred did so mainly because of the unseating of the roof trusses. The failure which occurred here could probably have been prevented by the use of a reinforced concrete head band at the top of the wall securely anchored to the wall and to which the trusses were in turn securely anchored.

Fig. 208 shows part of the brick walls of a downtown building which failed. It consisted in the main of large sections as seen in the figure. This building had large window openings, at the corners of which failure evidently commenced.

A large number of buildings in this area, both reinforced concrete and brick, were tile veneered. This tile veneering evidently generally resisted spalling during the deformations of the buildings in the earthquake but was consistently spalled by the heat of the subsequent fire. This was noticeable by the fact that the tile was generally spalled the most on the sides of buildings that had been exposed to the highest temperatures. Two important things seem apparent from the examination of the tile work, namely; that the joints should be raked out to below the facing line to prevent the chipping off of the face of the tile as a result of compression during building deformation, and that the tile, or brick if used, would probably be better if it were true enameled rather than glazed over a color slip.

Brick work should be done using select, hard burned clay bricks with cement mortar of not less than 1:3 (sand). Lime mortar never should be used, and never cement lime mortar unless the lime content is for better bonding.

The buildings examined in the Fukui area were usually open and had wooden floors. Numerous brick buildings in the Tokyo area are still standing, perfectly intact. that have survived many earthquake shocks, including the severe 1923 Tokyo (Kanto) earthquake. These buildings have numerous interior walls and monolithic concrete floors which give to the buildings a high degree of rigidity.

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E. FACTORIES

1. GENERAL - The Fukui area, being one of the main textile centers of Japan, is spotted with numerous mills, and factories, both large and small. There were four large factories in the immediate Fukui area, one belonging to the Fukui Seiren Company, one to the Daiwa Boseki Company, and two to the Sakai Seni Company. These factories were investigated in detail.

2. FUKUI SEIREN FACTORY

a. General - This factory is located at Keya-Cho, Fukui City. The factory site occupies an area of 432,000 square feet. On this site were located 21 buildings, producing silk and rayon cloth, with a total floor area of 180,000 square feet. Fig 209 shows the factory site layout.

This factory was destroyed by fire during the war and later reconstructed so that the majority of the buildings were only about two years old. With the exception of the administration building of reinforced concrete and the steam generation plant of steel construction, all buildings were of wood.

The damage to structures, according to factory officials was as follows:

Total collapse 50%

Partial damage 30%

Undamaged 20%

Although the damage to the buildings was extensive, as indicated above, the damage to machinery and equipment was

slight, amounting to approximately 10 per cent.

The top 10 to 15 foot layer of soil consists, in the factory area, of a mixture of sand, gravel and organic material. This layer rests on a layer of hard clay. The top 6 feet of the soil was mechanically compacted before any foundation construction was begun. The builders assumed that it was economical to provide wood piled foundations only for buildings having more than 3600 square feet of floor area.

b. Dye Plant - This was the largest building in the factory. It was provided with continuous foundation walls extending about three feet above the ground floor level upon which the structural frame of wood was bolted. It had wood siding and a very strong a tire building a result of e structure whi not fail and

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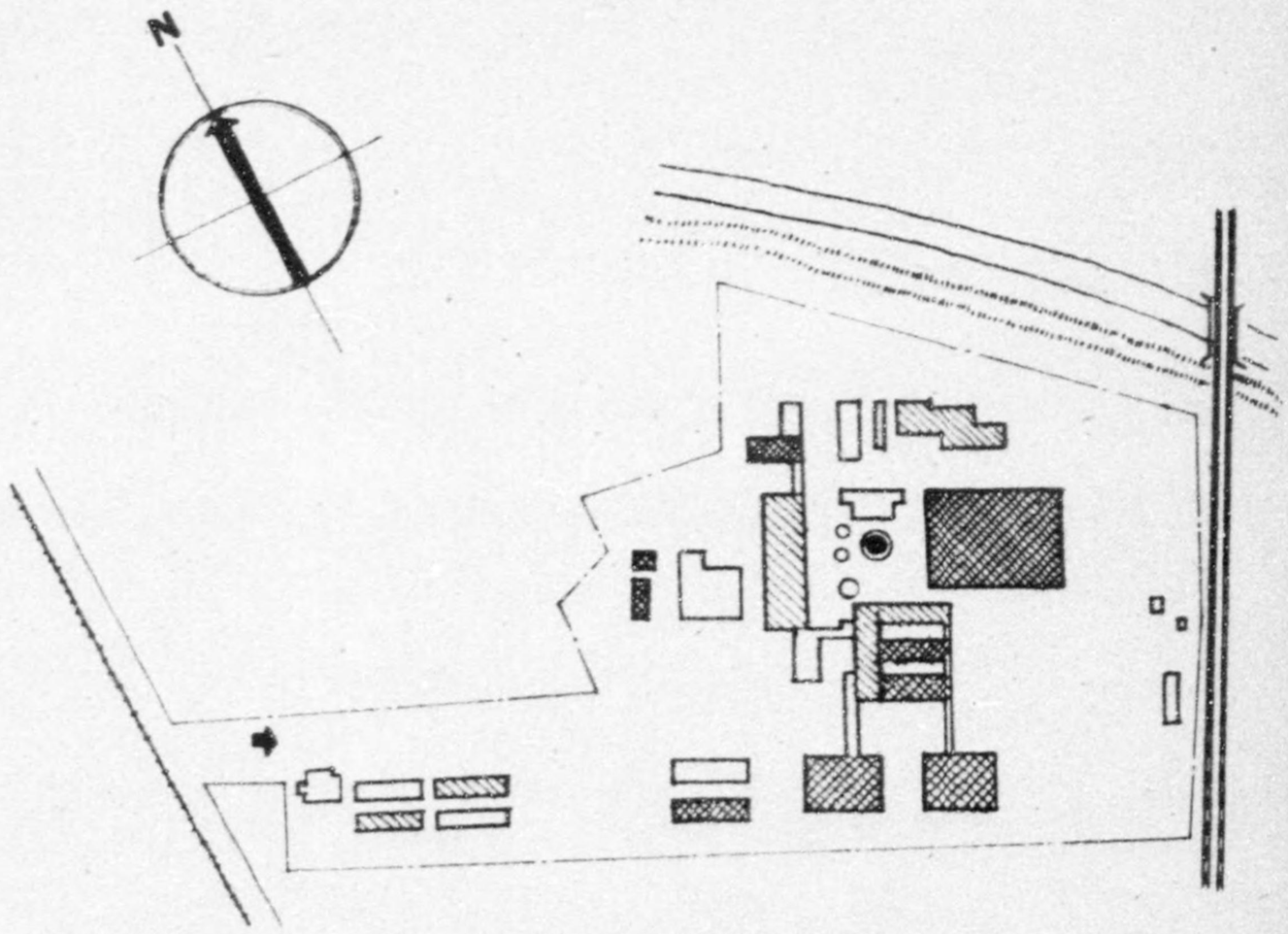

Fig. 209

Site Plan Layout - Fukui Seiren Factory
Fukui City

siding and a wood shingle roof. The wood roof trusses were very strong and provided a rigid top to the building. The entire building was deformed laterally in a manner often seen as a result of earthquakes. The building was a continuous structure which, together with the fact that the columns did not fail and the bracing held, kept it from total collapse.

c. Printing Plant - This building housed the printing machines used to print the silk or rayon cloth. The typical cross section of the framing of this building is shown in Fig. 210. The roof trusses were grounded due to the failing of the supporting columns at the junction point of the knee braces. Due to their heavy construction, the roof trusses were found to

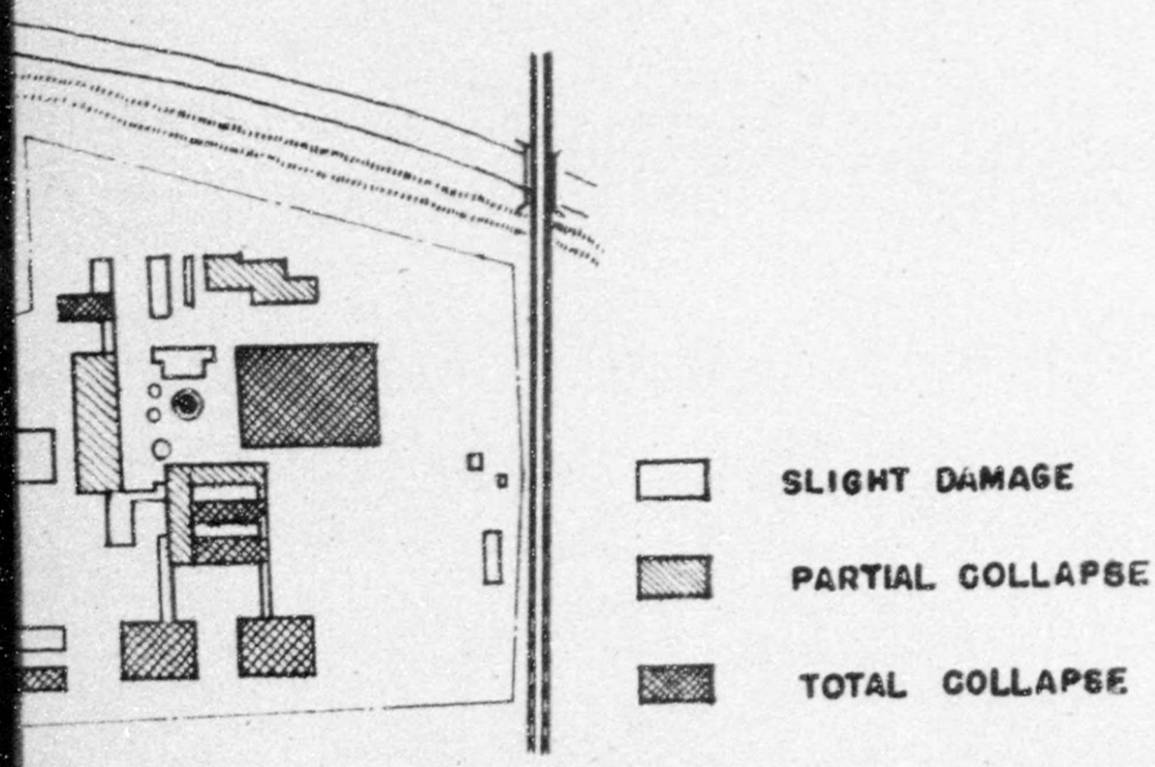

Fig. 209

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Fukui City

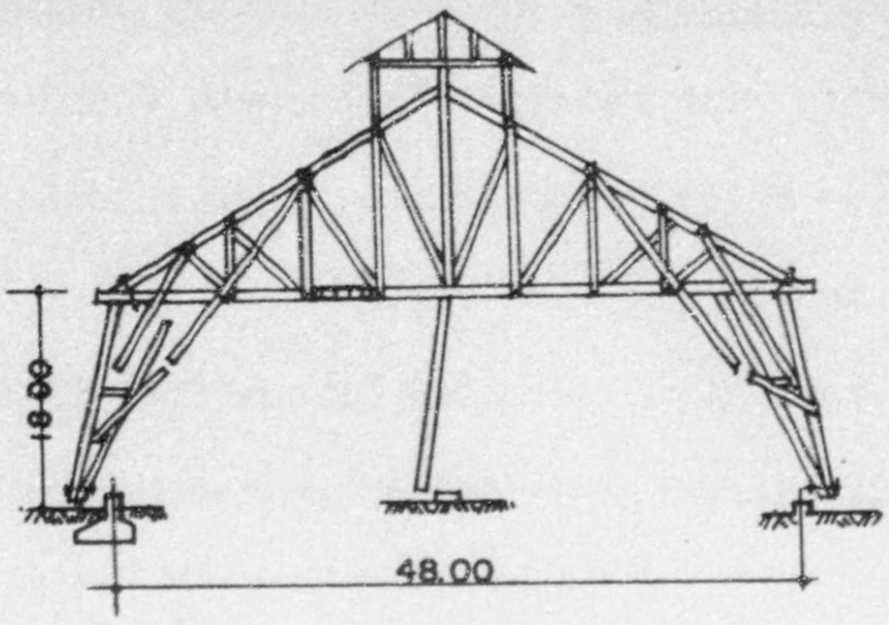

Fig. 210

Sectional Elevation - Printing Plant
Fukui Seiren Factory - Fukui City

Fig. 211
View of Collapsed Printing Plant Fukui Seiren Factory Fukui City

have retained original structural shape. The falling trusses inflicted damage to the machinery below and, when not obstructed by machinery, usually pierced the concrete floor. At the time of the first visit of the investigation party, the machinery had been crated in place to protect it in the event hanging trusses should fall in the after-shocks.

A view of the collapsed building from the outside is shown in Fig. 211.

d. Administration Building - This four-story reinforced concrete building, suffering only slight damage, is

described in Section B. REINFORCED CONCRETE BUILDINGS, of this chapter.

e. Steam Generation Plant - An all-steel framed structure, with cement-asbestos corrugated sheet roof and walls, suffering practically no damage, is described in Section C, STEEL STRUCTURES, of this chapter.

f. Other Buildings - A typical vertical cross-section is shown in Fig. 210. Some buildings had strut supports under the center of the roof trusses. Such buildings generally escaped total collapse whereas those not having this marginal support failed completely.

This factory had a reinforced concrete chimney which broke at about the midpoint. The chimney is described in Chapter V. A visit was made to this factory on 12 October, approximately three months after the first visit, and this chimney was found repaired and the reconstruction of damaged buildings well under way.

3. DAIWA BOSEKI FACTORY

a. General - This factory, which manufactures only cotton cloth, is located within the Fukui City limits about one mile south of the Asuha River. The factory site occupies an area of 2,160,00 square feet with a pre-earthquake building floor area of 340,000 square feet; 100,800 square feet consisting mainly of women's dormitories, was completely lost, while another 57,600 square feet was considerably damaged.

A site plan layout of this factory is shown by Fig. 212.

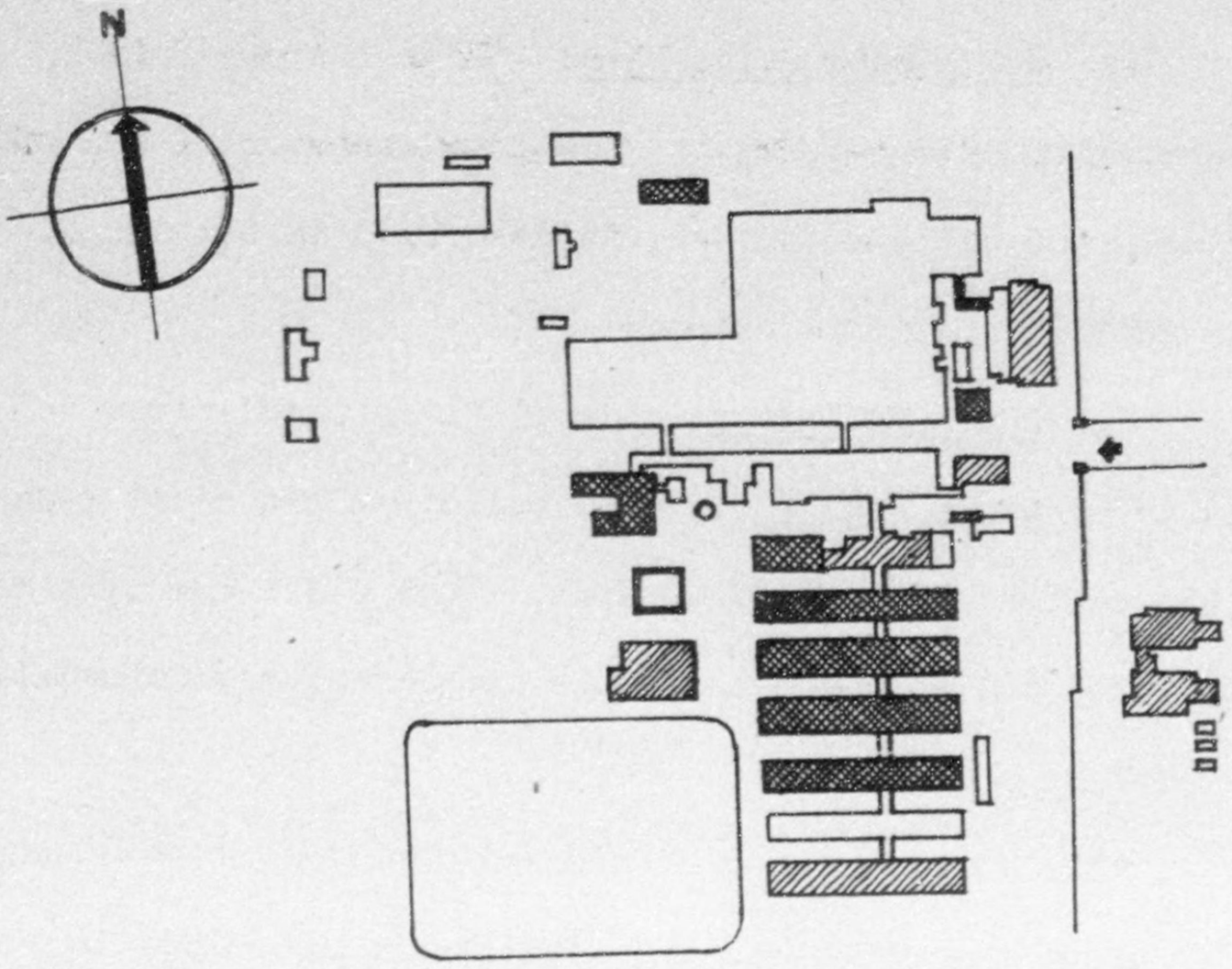

Fig. 212

Site Plan Layout - Daiwa Boseki Factory
Fukui City

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b. Main Building - The main factory building in this plant is constructed with reinforced-concrete foundations, floors, and outside walls and columns; steel interior columns, trusses, purlins and bracing and cement-asbestos corrugated sheet roof. The concrete floor slab of one foot thickness was constructed in 1928 directly on filled ground. This building is in places three sections wide, each section being five spans wide, saw-tooth in section, with a span width of 23 feet. The building was subjected to a wartime fire raid and in the resulting fire the structural steel was badly deformed. In reconstruction of this building, the frame was straightened just enough to make it usable. The columns, girders. and trusses which were utilized in this deformed state showed no signs of additional distress as a result of the violent shaking. The steel frame was braced both horizontally and vertically by light steel sections. Fig 213 shows an interior view of this factory building. The only damage was the heaving and rupture of the wood covered concrete floor in one of the units. In this same unit several connecting concrete walls pulled apart due to wall footing subsidence.

It will be noted in Fig. 213 that the damage to cast-iron machine frames was considerable. The frames were sheared off near the floor level due to the violent horizontal ground movements. The machines had a considerable unit mass which was enough to shear the frame when accelerated by the ground

movements. It would be well to remember, in the installation of machines in factories located in active seismic zones, that the frame is liable to shearing action and adequate strengthening would be a wise precaution. The damage to equipment occurred mainly to the bobbin winding machines.

c. Auditorium - The auditorium is just across the road from the main building described above. The floor and foundation walls are of reinforced concrete with wood columns, wood walls and siding, and wood roofing. The roof trusses with monitor are steel, Fink-type, 48-foot span, and supported by two wooden column sections, 5" x 5", at each end support. Fig. 214 shows the steel knee braces and the diagonal and longitudinal bracing members. The knee brace thrust is evenly distributed to the column by a heavy angle plate. Although subjected to violent ground movements, this building stood due to use of good materials, good construction, and a design which included consideration of lateral forces.

d. Steam Generation Plant - This building is described in Section C, STEEL CONSTRUCTION.

e. Other Buildings - To the south of, and parallel to, the auditorium were seven two-story wood buildings which were used as women's dormitories. These buildings, with the exception of one, all suffered total or partial collapse. These buildings were an example of the type of construction utilizing heavy roof framing and light, poorly braced columns.

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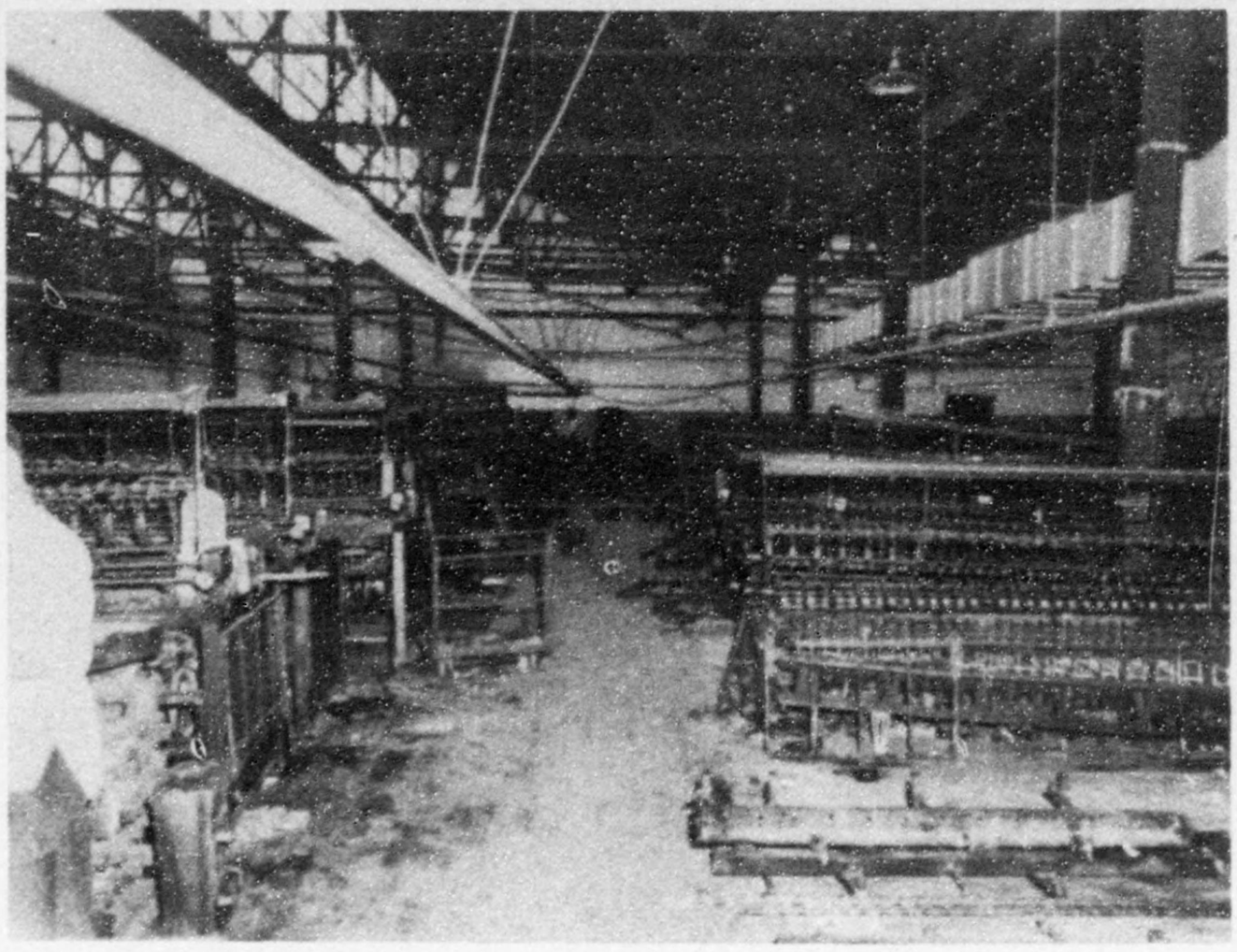


Fig. 213

Interior View - Main Building
Daiwa Boseki Factory - Fukui City

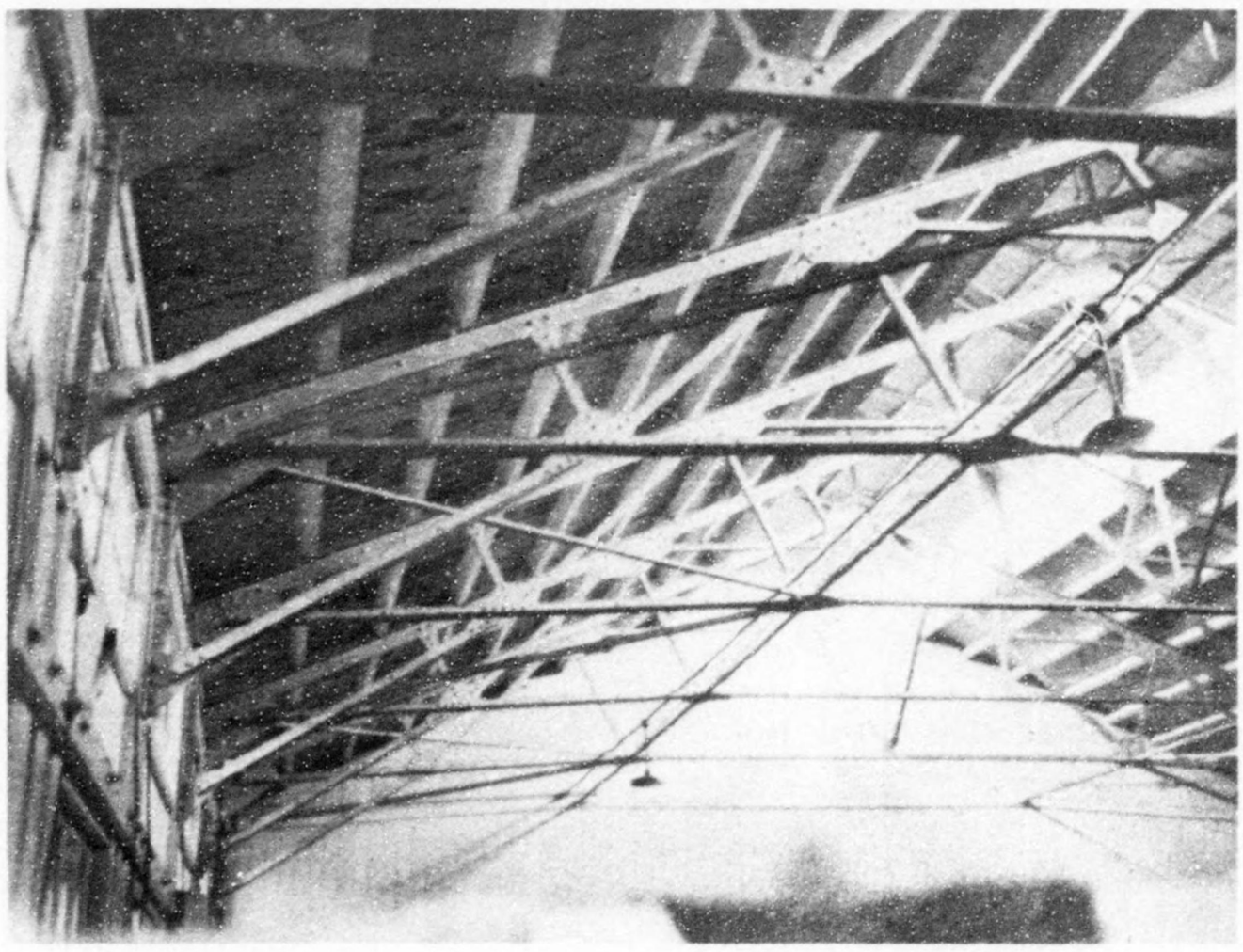


Fig. 214

Steel Roof Trusses - Auditorium
Daiwa Boseki Factory - Fukui City

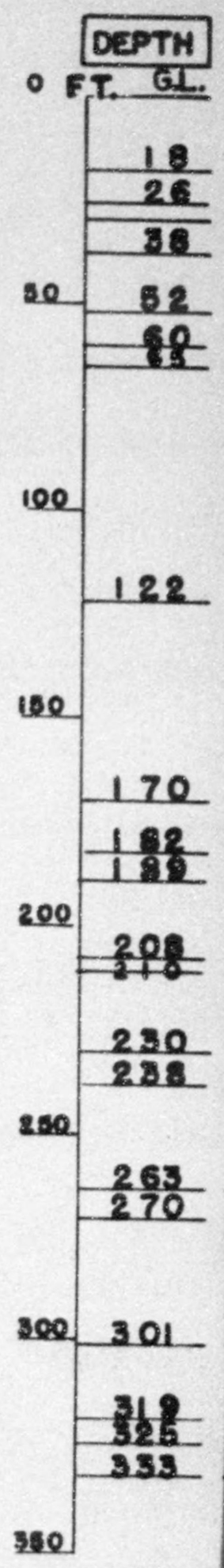
The company built a group of 42 houses which were located just to the south of the factory area. These houses were all wood, lightly framed, and fairly well constructed. As a result they suffered very slight damage.

This factory is an example of the efficacy of adequate bracing in protecting buildings against seismic forces. It is notable that the materials and workmanship used in the construction of the buildings which withstood the earthquake were of a more superior quality than found in the collapsed buildings.

4. SAKAI SENI FACTORY

a. General - Sakai Seni factory consists of the two separate plants located at Hanando-cho, just south of Fukui City limits. The plants are the Seiren plant and the Honsha plant located about two-thirds of a mile apart.

The soil conditions at the sites of these two plants are generally unsatisfactory, consisting for the most part of filled ground on top of alluvial layers of sand, gravel, clay, and organic matter. A well boring log at the Honsha plant site is shown in Fig. 215. This can be considered a general indication of the soil conditions in this area. In the case of the Honsha plant, the upper layer was filled in 1928 and the factory was built during the years 1934 and 1935.



BORING RECORD AT SAKAI SEN-I HONSHA FACTORY

HANADO-MACHI FUKUI

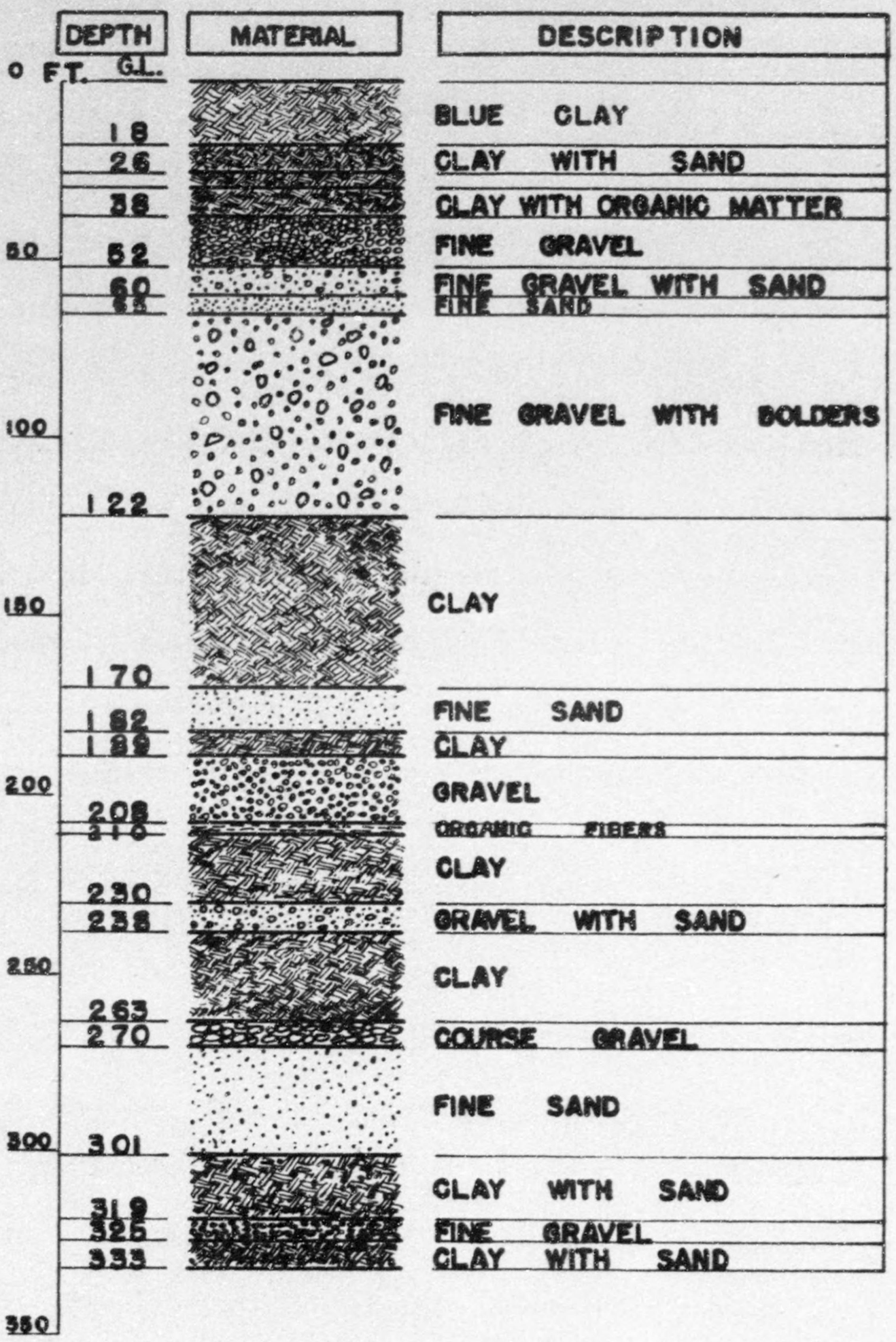


Fig. 215

Fig. 216

Central Section - Sakai Seni Factory
Seiren Plant - Fukui City

Fig. 217

Japanese Houses - North of Sakai Seni Factory
Seiren Plant - Fukui City

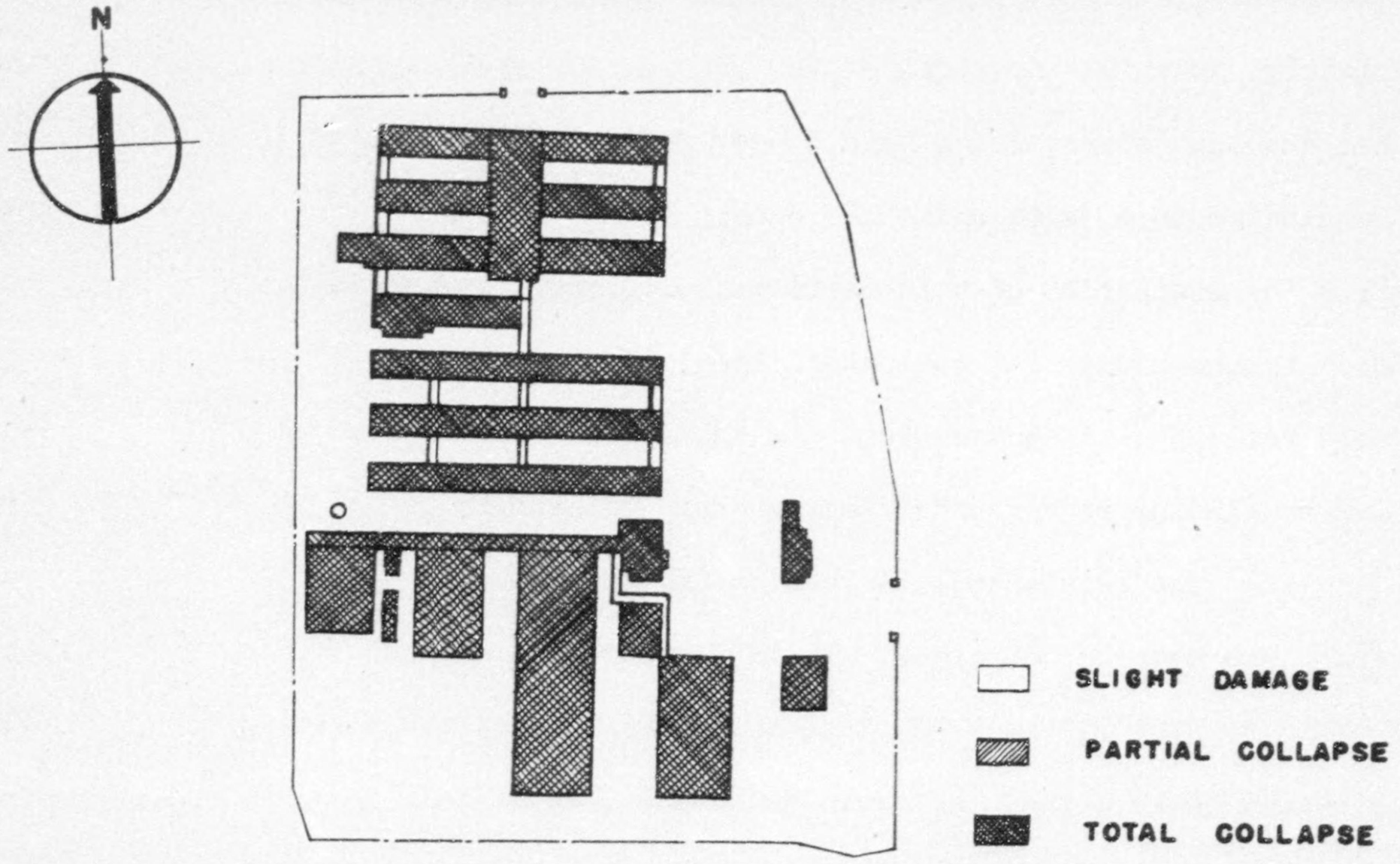


Fig. 218

Site Plan - Sakai Seni Factory
Honsha Plant - Fukui City

b. Seiren Plant - The area occupied by this plant site is 360,000 square feet. The total floor area of the fourteen buildings of the factory amounted to 180,000 square feet. The earthquake shattered all but about 5% of the buildings on a floor area basis. These few buildings were distorted but were better braced in the resultant direction than the other buildings. Fig. 216 shows a view looking from south to north down the middle of the plant site.

It will be noted that the few buildings which survived were very small, hardly deserving to be called buildings. The concrete chimney is part of a small steam generation plant. The steam plant building of wood construction collapsed completely, smashing the steam piping and pumping equipment. The boilers themselves, which were in two batteries of four each, Lancashire type, were undamaged except for cracked brickwork. From the inspection of this small boiler plant, and the damage done to the piping and equipment therein by the collapse of the wood building, it appears that the additional cost of a steel frame building is warranted for such an installation.

The reinforced-concrete chimney shown in this photograph was somewhat strained. It is described in Chapter V.

Fig. 217 is taken from the north side of the plant site and shows a group of Japanese houses located near the slope of the hill composed of tuffaceous soil. These houses are of substantial prewar construction and were undamaged

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structures are buil

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wood framing and mu

Fig. 219
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bolts were subjecte
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buildings, of whom

although the factory just described, less than one hundred yards away, was reduced practically to heaps of debris. According to the Japanese, some of these houses had the foundations placed on short wooden piles. This is one of many indications that earthquake damage is intimately related to the thickness of the alluvial soil and to the type of soil on which structures are built.

c. Honsha Plant - The site area of this plant approximates 900,000 square feet. The site plan is shown in Fig. 218. On this site stood seventeen buildings of wooden construction with a total floor area roughly 345,600 square feet. As in the sister plant, 95 per cent of the facilities were totally collapsed, 3 per cent half-damaged, and only 2 per cent, on a floor-area basis, escaped damage. Only one building of wood framing and mud masonry construction stood intact.

Fig. 219 shows a view of a collapsed building that is typical of all of this factories' buildings. In come cases the wooden columns were pulled out from the sills and displaced from six to ten feet. In this process, foundation plate anchor bolts were subjected to sufficient stress to fracture them in tension. A majority of the buildings fell in the north-south direction, which might indicate ground movement in this direction at this site.

One hundred and fifty persons were caught under fallen buildings, of whom fifteen were killed. According to eye-

Fig. 219

Collapsed Buildings - Sakai Seni Factory
Honsha Plant - Fukui City

witnesses, the shocks were of such violence that the buildings were brought down within twenty seconds after the initial shocks. Damage to the mechanical equipment was estimated at 40 per cent, being in this case, mainly caused by falling buildings.

It was evident that the columns were too slender and weak for the heavily framed tiled roofs they supported. This was a continual fault in this area. The roof trusses on the other hand, are more than sufficient, even considering the heavy snowfall. Fig. 220 shows the roof trusses in one of the buildings. The trusses suffered no damage or deformation even in the collapse of the buildings. Such combination of top-heavy roofs supported by slender columns is a flagrant violation of the fundamental principles of earthquake resistant

Fig. 220

Roof Trusses - Collapsed Building
Sakai Seni Factory - Honsha Plant
Fukui City

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construction. Another point requiring consideration and thought is the truss knee braces which are supposedly provided to give rigidity and strength to the building. In numerous instances, the columns were found fractured at the junction of the lower end of the knee brace. This is a measure of the effective working of the knee-brace, but in such cases it gives the impression that they were there for the purpose of fracturing the columns. The remedy in the final analysis is to provide columns of sufficient cross section and adequate slenderness ratio so that the columns can withstand the impact thrusts transmitted to them by the knee brace action.

5. OTHER FACTORIES - Among the other factories inspected in the disaster area, the Jomo Neshi plant at Kanazu, which embodied the best in factory design in every respect with the

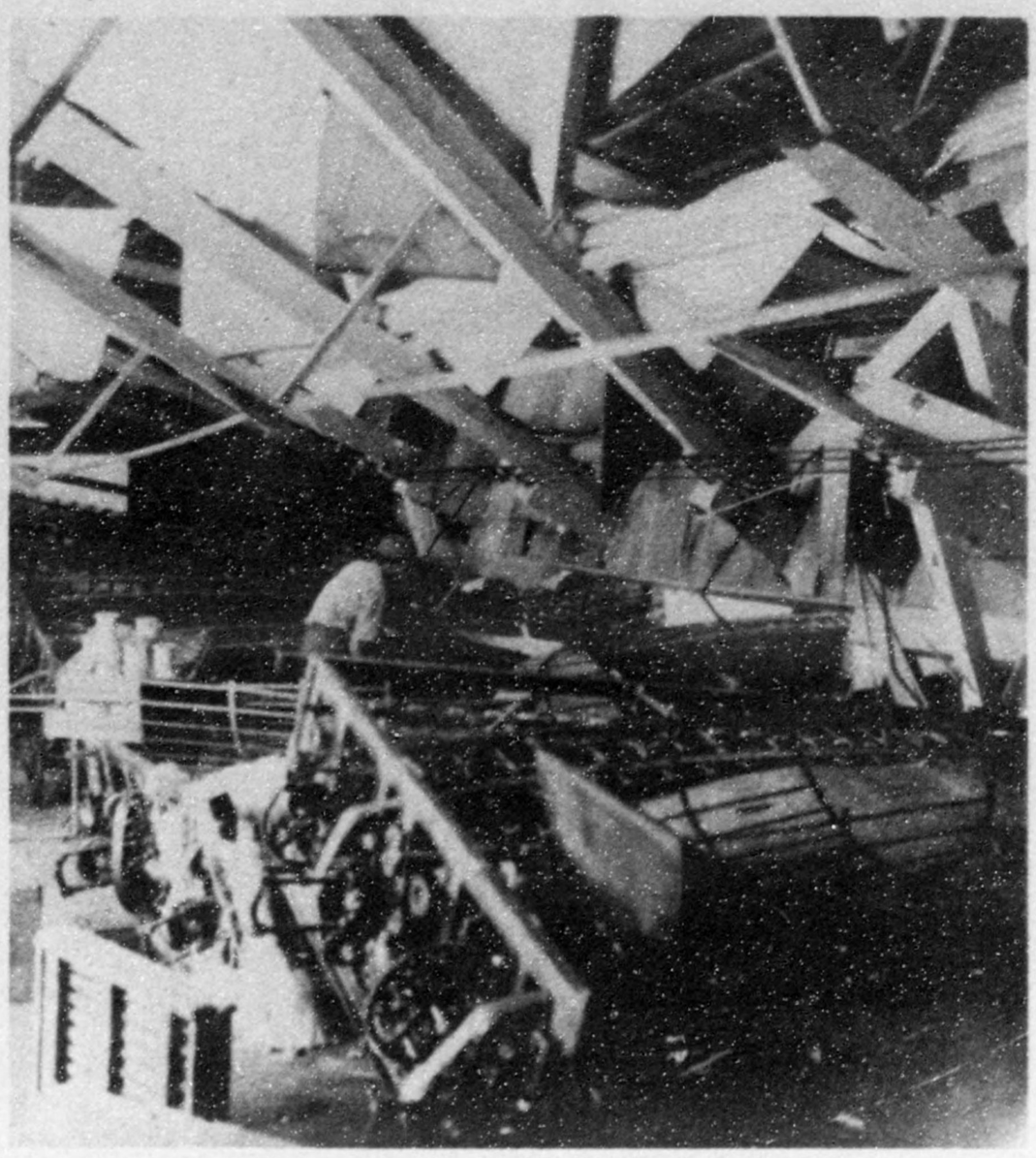

Fig. 221

Interior View - Winding Room
Jomo Nenshi Factory - Kanazu

notable exception of earthquake resistant construction, was a total loss beyond repair. Fig. 221 shows an interior view of one of the winding machine rooms. Column failure was the pri-

mary cause of c
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Fig. 222 s
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buildings were

Fig. 223
north of Fukui
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6. CONCL
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mary cause of collapse. An expenditure of perhaps 5 per cent of the initial plant cost to make it resistant to seismic forces might well have saved 95 per cent of the loss suffered here.

Fig. 222 shows an oblique aerial view of the Maruni Sangyo factory located about a mile north of Fukui City. Damage here was 90 per cent of the floor area. It will be noted that most buildings were displaced to the south.

Fig. 223 shows an oblique aerial view of another factory north of Fukui City, name unknown. This factory was almost 100 per cent destroyed. Investigation proved that the usual failure, that of columns, occurred.

6. CONCLUSION - The failures noted in wooden factory construction were caused mainly by lack of column strength and adequate bracing. Steel and reinforced-concrete construction used in factory construction withstood the earthquake very satisfactorily. The extra initial cost involved in the provision of adequately designed and constructed factory buildings was more than offset by the protection to expensive equipment and the allowing of continued uninterrupted production.

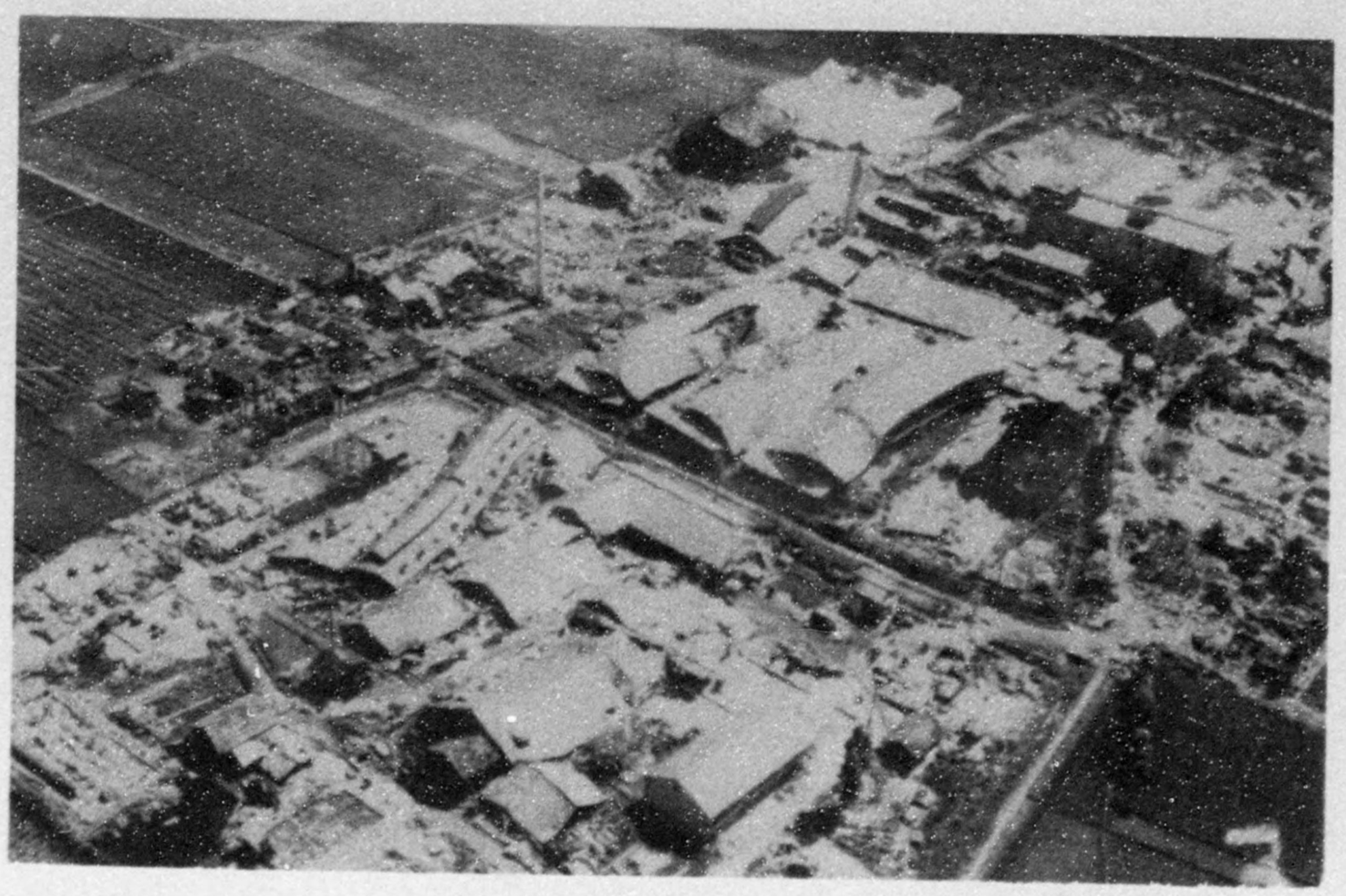

Fig. 222

Aerial View - Maruni Sangyo Factory
North of Fukui City

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Fig. 223

Aerial View - Collapsed Factory Buildings
North of Fukui City

WATER TANK FAILURE

CHAPTER V
CHIMNEYS AND TANKS

A. REINFORCED CONCRETE CHIMNEYS

1. GENERAL - There were numerous chimneys of reinforced concrete construction in the disaster area. Many of them failed completely, some developed serious cracks, some were bent and inclined, and not one came out of the earthquake undamaged.

Reinforced concrete chimneys come under the regulations of the building codes and in their structural design a horizontal seismic acceleration greater than 0.15 must be considered. The allowable steel and concrete stresses are 20,000 psi and 630 psi respectively. The fact that the chimneys showed such widespread distress is a source of concern, especially to the engineers who designed them. In the past earthquake a certain number of chimneys have fallen but never such a high percentage. Fig. 225 is a table which gives information on damages to brick, steel, and reinforced concrete chimneys as a result of the severe earthquake in the Tokyo (Kanto) area in 1923.

The results shown in this table indicate the superiority of reinforced-concrete chimneys over brick chimneys in ability to withstand seismic forces. Chimneys in Japan after 1923 were nearly all constructed of reinforced concrete. However, chimneys have continued to suffer in earthquakes and typhoons. Engineers concerned about these independent structures have,

from time to time, pointed out weaknesses in chimney design methods. Confronted with extensive damage, nearing total damage, it is not possible to ignore the apparent inadequacy of chimneys as earthquake-resistant structures. We will therefore consider them from various viewpoints.

The chimneys surveyed by the investigation party were only 13 in number. Japanese investigation parties have reported 41 chimneys in all. Discussion will cover all of them.

2. DAMAGES - The chimneys in the disaster area, and the damages sustained, are listed in Fig. 224. With the exception of four steel sectional guyed chimneys, the independent chimneys listed are all of reinforced concrete construction.

Figures 226 through 250, by means of photographs and drawings, show the exact damage sustained by some of the chimneys. Fig. 226 and Fig. 227 show the damage to the chimney located near the Fukui prefectural office buildings. The upper portion of the chimney fell on the wooden building located north of the chimney.

Figures 228 and 229 show the damage to the 170 foot chimney located at the Fukui Seiren factory in south Fukui City. This chimney had an integral economizer tank built around it above the breeching connection. The chimney section which broke off fell against a concrete water tank below, inflicting minor damage, Fig. 230. Another chimney with a height of 130 feet,

NO	LOCATION	CON-STRUC-TION	HEIGHT (FT.)	SIZE (FT.)		DESCRIPTION OF DA
				TOP	BOTTOM	
1	FUKUI-SEIREN-CO. LTD	R.C.	170	9.84	15.06	BROKEN AT APPROXIMATELY THREE FIFTHS
2	" " " "	"	130	—	10.50	CRACKED AT THE JUNCTURE WITH THE SMO
3	SAKAI-SENI IPPONGI FAC-TORY (MITSUBISHI-DENKO)	"	130	7.87	9.84	BROKEN AT THREE FIFTHS OF THE HEIGHT F
4	FUKUI-SEIREN-KAKO KATSUMI FACTORY	"	120	—	9.68	DEVELOPED A FEW CRACKS BETWEEN THE B OF THE HEIGHT, DEVELOPED LARGE CRA ONE HALF THE HEIGHT FROM THE GROUN
5	SAKAI-SENI CO. LTD REFINING FACTORY	"	120	—	10.37	DEVELOPED A FEW CRACKS AT THE BOTTOM HEIGHT FROM THE GROUND
6	DAIWA-BOSEKI CO. LTD	"	120	(4.59)	9.18	CONCRETE SPALLED OFF AND REINFORCING BOTH NORTH AND SOUTH SIDE AT THE BO AT ONE HALF THE HEIGHT FROM THE GROU
7	NAKAJIMA-ORIMONO KOGYO CO. LTD	"	120	5.58	9.18	BROKEN AT APPROXIMATELY ONE THIRD OF TH PORTION FELL, STUCK INTO THE GROUND U
8	FUKUSEN KOGYO CO.LTD	"	120	3.94	6.40	DEVELOPED A FEW CRACKS NEAR THE MIDDL BELOW THE MIDDLE.
9	SHINKOSHA (FUKUI KOGAKU)	"	117	—	—	INCLINED AT APPROXIMATELY ONE HALF
10	SAKAI-SENI CO. LTD REFINING FACTORY	*	110	—	5.41	BROKEN AT ONE FOURTH OF THE HEIGHT FR
11	MARUSAN-SENSHOKU CO.	R.C.	110	(8.00)	11.15	BROKEN AT ONE THIRD OF THE HEIGHT FROM
12	NEAR ASUHA YAMA	"	110	(4.26)	8.53	BROKEN AT APPROXIMATELY TWO THIRDS OF T
13	TOWA-BOSEKI	"	110	5.40	9.68	CRACKED ON THE WEST SIDE AT THE BOTTO
14	JOMO-NENSHI CO. LTD	"	110	3.94	6.56	INCLINED AT ONE THIRD OF THE HT. & BROK
15	DARUYAMA	"	100	3.28	5.58	BROKEN AT APPROXIMATELY ONE THIRD OF T
16	MARUNI-SANGYO CO. LTD	"	100	4.10	6.53	BROKEN AT APPROXIMATELY ONE HALF OF TH THE REMAINING PORTION OF THE SHAFT C INCLINED AT ONE FOURTH OF THE HEIGHT F
17	ASAHI KAGAKU CO. LTD	"	100	3.28	5.70	CRACKED AT APPROXIMATELY ONE HALF OF T A FEW CRACKS APPEARED AT THE GROUND
18	FUKUI PREFECTURAL OFF.	"	95	3.61	5.90	BROKEN AT TWO THIRDS THE HEIGHT FROM
19	SAKAI-SENI MAIN FACTORY	"	95	—	7.22	BROKEN AT ONE THIRD OF THE HEIGHT FROM
20	FUKUI TECHNICAL COLLEGE	"	90	(2.62)	(5.74)	DEVELOPED A FEW CRACKS AT APPROXIMA HEIGHT FROM THE GROUND.
21	FUKUI RED CROSS HOSPIT'L	"	80	3.77	6.23	BROKEN AT APPROXIMATELY ONE FIFTH OF
22	FUKUI-RAILWAY YARD	"	80	3.94	5.90	CRACKED BETWEEN ONE THIRD AND TWO T GROUND AND INCLINED. CRACKS APPEARE
23	FUKUI RED CROSS HOSPITAL	"	76	2.95	4.79	BROKEN AT TWO FIFTHS OF HEIGHT FROM T
24	MARUNI-SANGYO CO. LTD	"	75	4.26	6.77	DEVELOPED BIG CRACKS AROUND THE BOTT THE SOUTH SIDE OF THE UPPER PORTION
25	KOKUSAI KOKU FACTORY	"	70	—	8.20	BROKEN AT APPROXIMATELY ONE HALF OF
26	UNKNOWN	"	70	2.95	4.59	CRACKED ABOVE THE JUNCTURE OF THE SM SHAFT ; INCLINED.
27	TSUBOKANE-ORIMONO	"	70	3.28	4.92	BROKEN AT APPROXIMATELY ONE FIFTH OF
28	FORMERLY OKARI ORIMONO	"	70	3.28	4.92	BROKEN AT THE BOTTOM OF THE SHAFT
29	KASUMI-BOSHOKU CO. LTD	"	70	2.46	2.46	" " "
30	SAKAI BUILDING	"	67	—	3.94	DEVELOPED A FEW CRACKS WITHIN 10 TO I THE BUILDING (20 FT. ABOVE THE GROUND

* STEEL WITH STAY FOR SUPPORT

Fig. 224

Description of Chimney Damages
Fukui Earthquake, 1948

(FT.) BOTTOM	DESCRIPTION OF DAMAGE
5.06	BROKEN AT APPROXIMATELY THREE FIFTHS THE HT. FROM THE GROUND.
0.50	CRACKED AT THE JUNCTURE WITH THE SMOKE BREECHING.
9.84	BROKEN AT THREE FIFTHS OF THE HEIGHT FROM THE GROUND.
9.68	DEVELOPED A FEW CRACKS BETWEEN THE BOTTOM AND ONE FOURTH OF THE HEIGHT, DEVELOPED LARGE CRACKS AND INCLINED AT ONE HALF THE HEIGHT FROM THE GROUND.
0.37	DEVELOPED A FEW CRACKS AT THE BOTTOM AND AT TWO FIFTHS THE HEIGHT FROM THE GROUND
9.18	CONCRETE SPALLED OFF AND REINFORCING STEEL BARS BUCKLED AT BOTH NORTH AND SOUTH SIDE AT THE BOTTOM. CRACKS OCCURRED AT ONE HALF THE HEIGHT FROM THE GROUND & SHEARED IN TWO PIECES.
9.18	BROKEN AT APPROXIMATELY ONE THIRD OF THE HEIGHT. THE BROKEN UPPER PORTION FELL, STUCK INTO THE GROUND UPSIDE DOWN, & SHEAR'D INTO 2 P'CES
6.40	DEVELOPED A FEW CRACKS NEAR THE MIDDLE OF THE SHAFT AND A LITTLE BELOW THE MIDDLE.
—	INCLINED AT APPROXIMATELY ONE HALF THE HEIGHT FROM THE GROUND
6.41	BROKEN AT ONE FOURTH OF THE HEIGHT FROM THE GROUND.
1.15	BROKEN AT ONE THIRD OF THE HEIGHT FROM THE GROUND.
8.53	BROKEN AT APPROXIMATELY TWO THIRDS OF THE HEIGHT FROM THE GROUND.
9.68	CRACKED ON THE WEST SIDE AT THE BOTTOM.
6.56	INCLINED AT ONE THIRD OF THE HT. & BROKE AT 3/5 OF THE HT. FROM GR'ND.
6.58	BROKEN AT APPROXIMATELY ONE THIRD OF THE HEIGHT FROM THE GROUND.
6.53	BROKEN AT APPROXIMATELY ONE HALF OF THE HEIGHT FROM THE GROUND. THE REMAINING PORTION OF THE SHAFT CRACKED AT THE BOTTOM AND INCLINED AT ONE FOURTH OF THE HEIGHT FROM THE GROUND.
5.70	CRACKED AT APPROXIMATELY ONE HALF OF THE HEIGHT FROM THE GROUND. A FEW CRACKS APPEARED AT THE GROUND AROUND THE SHAFT BASE.
1.90	BROKEN AT TWO THIRDS THE HEIGHT FROM THE GROUND.
1.22	BROKEN AT ONE THIRD OF THE HEIGHT FROM THE GROUND.
1.74)	DEVELOPED A FEW CRACKS AT APPROXIMATELY ONE FOURTH OF THE HEIGHT FROM THE GROUND.
1.23	BROKEN AT APPROXIMATELY ONE FIFTH OF THE HEIGHT FROM THE GROUND.
1.90	CRACKED BETWEEN ONE THIRD AND TWO THIRDS OF THE HEIGHT FROM THE GROUND AND INCLINED. CRACKS APPEARED AT THE LIFT POINTS.
1.79	BROKEN AT TWO FIFTHS OF HEIGHT FROM THE GROUND.
1.77	DEVELOPED BIG CRACKS AROUND THE BOTTOM AND A FEW CRACKS ON THE SOUTH SIDE OF THE UPPER PORTION.
1.20	BROKEN AT APPROXIMATELY ONE HALF OF THE HEIGHT FROM THE GROUND.
1.59	CRACKED ABOVE THE JUNCTURE OF THE SMOKE BREECHING AND THE SHAFT; INCLINED.
1.92	BROKEN AT APPROXIMATELY ONE FIFTH OF THE HEIGHT FROM THE GROUND.
1.92	BROKEN AT THE BOTTOM OF THE SHAFT
1.46	" " "
1.94	DEVELOPED A FEW CRACKS WITHIN 10 TO 15 FT. OF THE JUNCTURE WITH THE BUILDING (20 FT. ABOVE THE GROUND)

Fig. 224

cription of Chimney Damages
Fukui Earthquake, 1948

NO	LOCATION	CON-STRUC-TION	HEIGHT (FT.)	SIZE (FT.)		DE
				TOP	BOTTOM	
31	FUKUI CITY HALL	R.C.	66	2.95	5.12	CRACKED THE SHA
32	TELEPHONE OFFICE	"	65	—	3.74	INCLINED (30FT. A
33	KWANGYO BANK	"	60	—	3.87	SLIGHTLY JUNCTURE
34	KOKUSAI – KOKU	"	60	3.12	4.92	BROKEN A GROUND.
35	NAKAI-JINKEN-ORIMONO CO. LTD.	"	60	2.80	3.97	BROKEN A
36	KAKUMASA - ORIMONO SAISEI FACTORY	"	50	(2.79)	4.00	BROKEN A
37	BATH HOUSE	*	40	—	2.62	THE REM COLLAPSE
38	IKUSHISA CREMATORY	R.C.	40	—	3.94	CONCRETE EXPOSE
39	KURASHIKI KINU ORIMONO CO. LTD.	"	30	—	—	INCLINED
40	U.S. ARMY HOUSING	"	35			BROKEN
41	KUBO – ORIMONO		—	—	—	BROKEN

Fig. 224

Description of Chimney Damage
Fukui Earthquake, 194

CONSTRUCTION		TOTALLY COLLAPSED	INCLINED	UND
BRICK	NUMBER	51	18	
	PERCENTAGE	64	23	
REINF. CONC.	NUMBER	25	17	
	PERCENTAGE	15.7	10	
STEEL	NUMBER	72	88	1
	PERCENTAGE	4	5	

Fig. 225

Tabulation of Chimney Da
Kanto Earthquake, 192

N-RUC-ON	HEIGHT (FT.)	SIZE (FT.) TOP	SIZE (FT.) BOTTOM	DESCRIPTION OF DAMAGE
C.	66	2.95	5.12	CRACKED AT THE JUNCTURE OF THE SMOKE DUCT AND THE SHAFT, INCLINED.
	65	—	3.74	INCLINED AND CRACKED AT THE JUNCTURE OF THE BUILDING. (30FT. ABOVE THE GROUND)
	60	—	3.87	SLIGHTLY DAMAGED. THE FINISH FELL OFF ABOVE THE JUNCTURE OF THE BUILDING. (30 FT. ABOVE THE GROUND)
	60	3.12	4.92	BROKEN A LITTLE BELOW ONE HALF OF THE HEIGHT FROM THE GROUND.
	60	2.80	3.97	BROKEN AT ONE SEVENTH OF THE HEIGHT FROM THE GROUND.
	50	(2.79)	4.00	BROKEN AT ONE HALF OF THE HEIGHT FROM THE GROUND AND THE REMAINDER OF THE SHAFT INCLINED ABOUT 5.7°.
	40	—	2.62	COLLAPSED AT THE BOTTOM OF THE RC BASE.
C.	40	—	3.94	CONCRETE AT THE TOP SPALLED OFF AND REINFORCING STEEL EXPOSED. (DAMAGED BEFORE RECENT EARTHQUAKE)
	30	—	—	INCLINED SLIGHTLY NORTHWARD.
	35			BROKEN TWO FT. ABOVE GROUND LINE.
	—	—	—	BROKEN AT A POINT 5 FT. FROM THE GROUND.

Fig. 224

Description of Chimney Damages - Cont.
Fukui Earthquake, 1948

	TOTALLY COLLAPSED	INCLINED	UNDAMAGED	TOTAL	FIRE DAMAGE
MBER	51	18	10	79	
CENTAGE	64	23	13	100	
MBER	25	17	128	170	
ENTAGE	15.7	10	75.3	100	
MBER	72	88	1312	1900	428
ENTAGE	4	5	69	100	22

Fig. 225

Tabulation of Chimney Damages
Kanto Earthquake, 1923

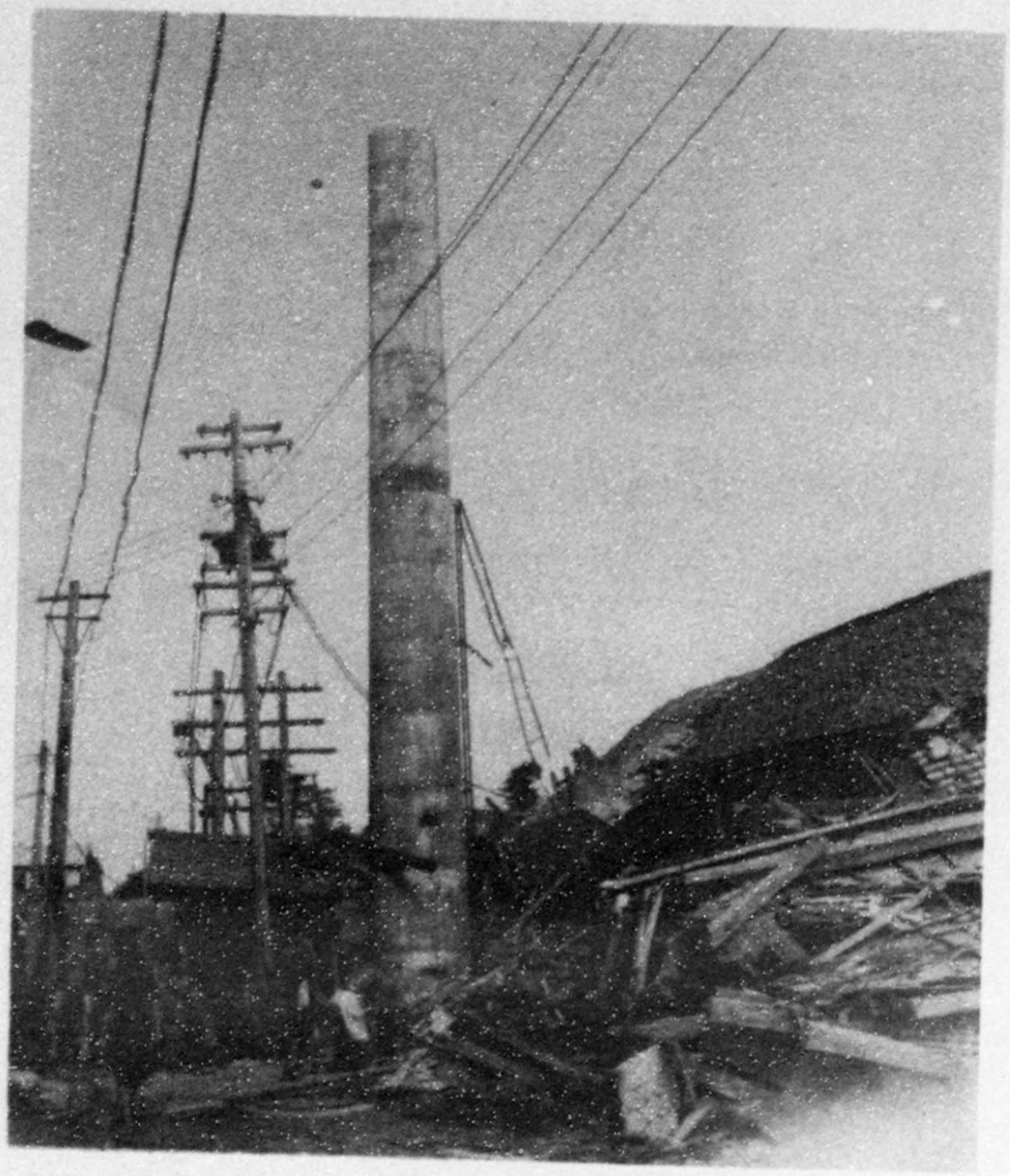

Fig. 226

Chimney - Prefectural Office Building Area
Fukui City

Fig. 228, stood nearby, undamaged except for some cracking around the smoke breeching opening at the base.

Figs. 231 and 232 show the damage to the independent chimney at the former Nakajima factory at Hinode-machi, which was razed by a wartime fire. The chimney broke at a position approximately three-fifths of its height and the broken off section pierced the ground. In the process of piercing the ground this section broke into two pieces, the thicker of the two pieces entering the ground in a vertical position at a second location. This gave, in the preliminary observation of aerial photographs, the impression of two stacks.

Fig. 233 and Fig. 234 show the damage to the independent chimney, 130 feet high, located at the Mitsubishi factory. The upper portion fell in a northwest direction. The remaining shaft was inclined in the opposite direction.

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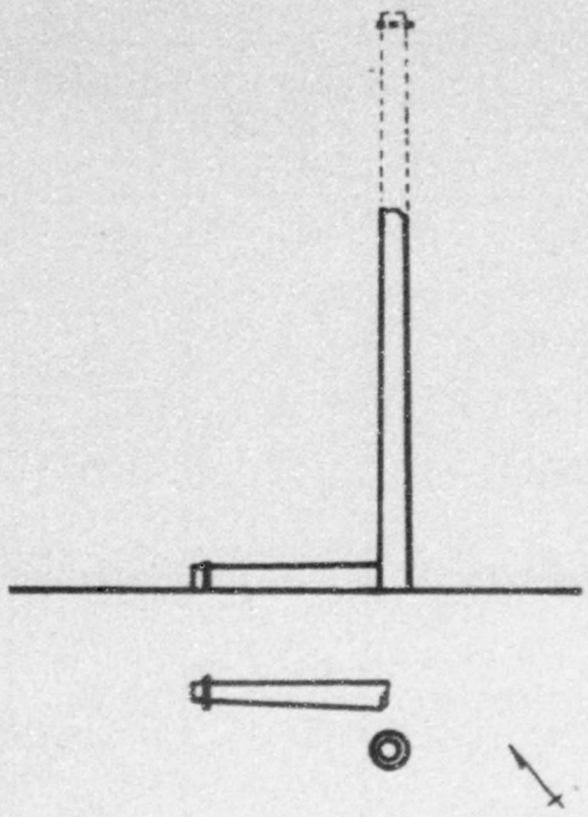


Fig. 227

Plan and Elevation - Chimney
Prefectural Office Building Area
Fukui City

Figures 235 and 236 show an independent chimney, 80 feet high, located at the Fukui railroad coaling and roundhouse area. It developed cracks at points 25 feet and 45 feet above the ground line. The cribbing was put up for temporary support.

Figures 237 and 238 show an independent chimney, 110 feet high, located at the Jomo Nenshi factory at Kanazu. The upper section, breaking off at a point 65 feet above the ground, fell in a southeasterly direction. Serious cracks also developed 40 feet above the ground and the shaft above these cracks inclined in the southwest direction.

Figures 239 and 240 show an independent chimney, 95 feet high, located at the Honsha plant of the Sakai Seni factory in Fukui City. The broken section fell in the northwest direction.

Figures 241 and 242 show a steel stack, 100 feet high, which was located at the Seiren plant of the Sakai Seni factory in south Fukui City. The sections were bolted together by utilizing small headed bolts and nuts with the flanges provided having no drilled holes but merely cutouts. The top sections toppled in the southeast direction and pierced the roof of an

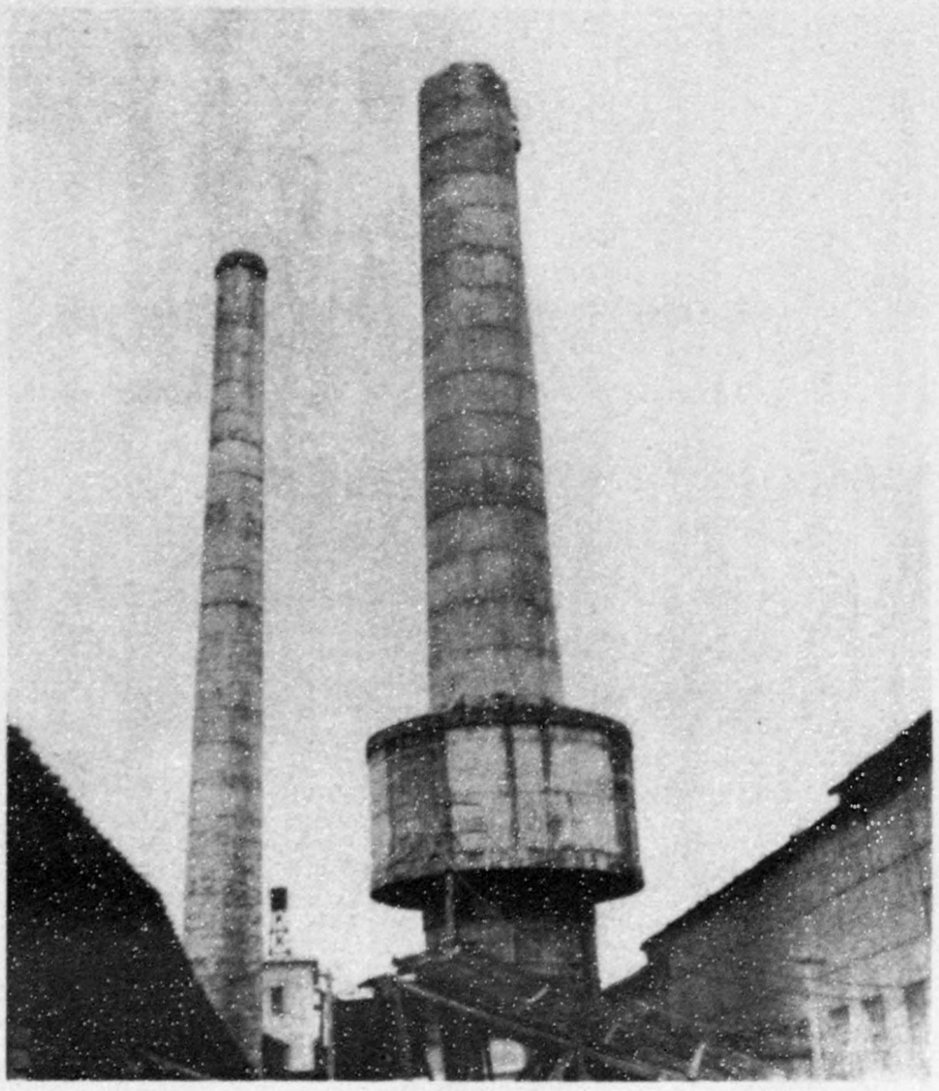


Fig. 228

Chimney Damage - Fukui Seiren Factory
Fukui City

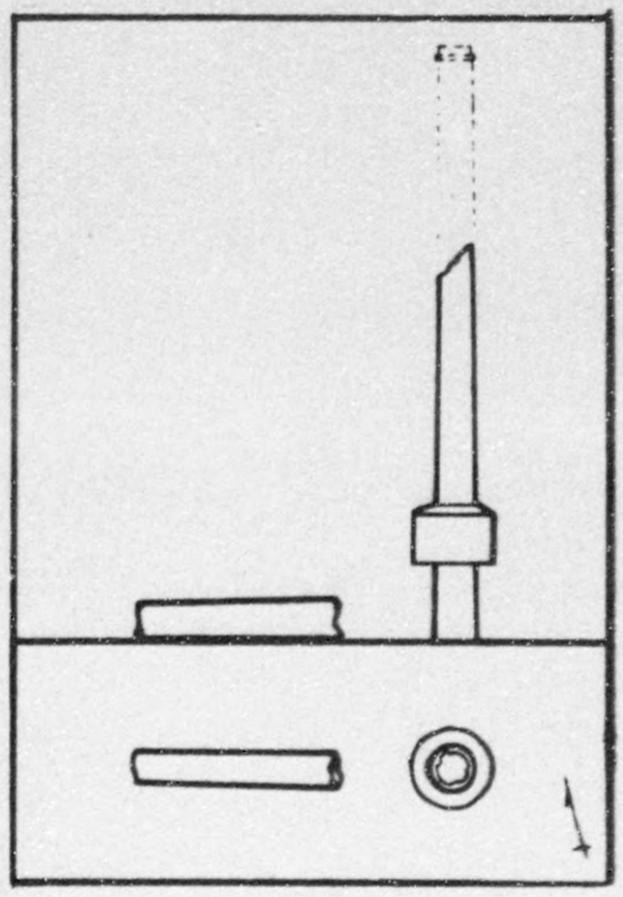


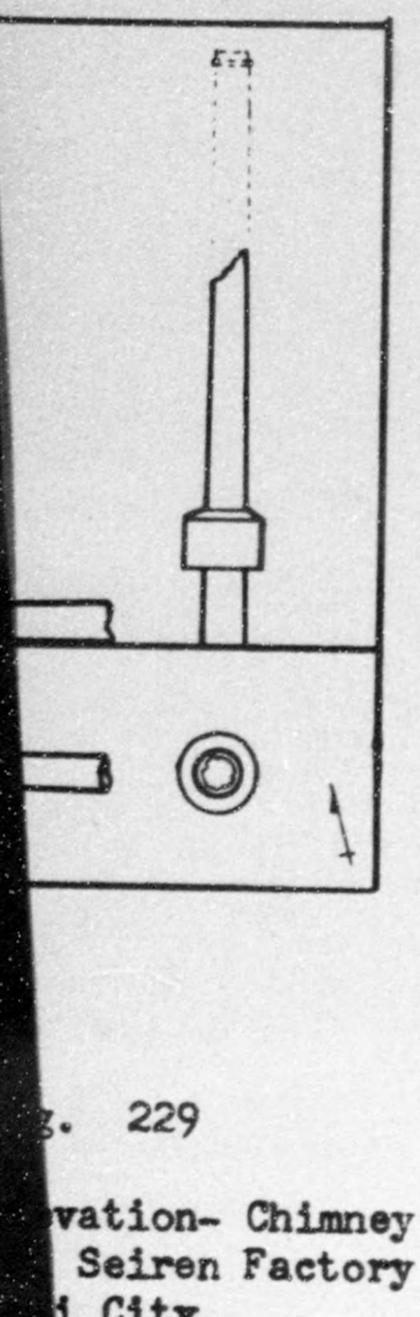
Fig. 229

Plan and elevation- Chimney
Damage Fukui Seiren Factory
Fukui City

adjoining building.

Figures 243 and 244 show an independent chimney, 120 feet high, located at the Seiren plant of the Sakai Seni factory in south Fukui City. The chimney cracked at about one-third of the height above the ground and inclined in a northwesterly direction. The photograph above the shaded portion represents the incline of the chimney off the vertical.

Figure 245 shows an independent chimney, 120 feet high, located at the Daiwa Boseki factory in south Fukui City. The stack cracked on the east and west sides at about the midpoint.



g. 229

vation- Chimney
Seiren Factory
i City

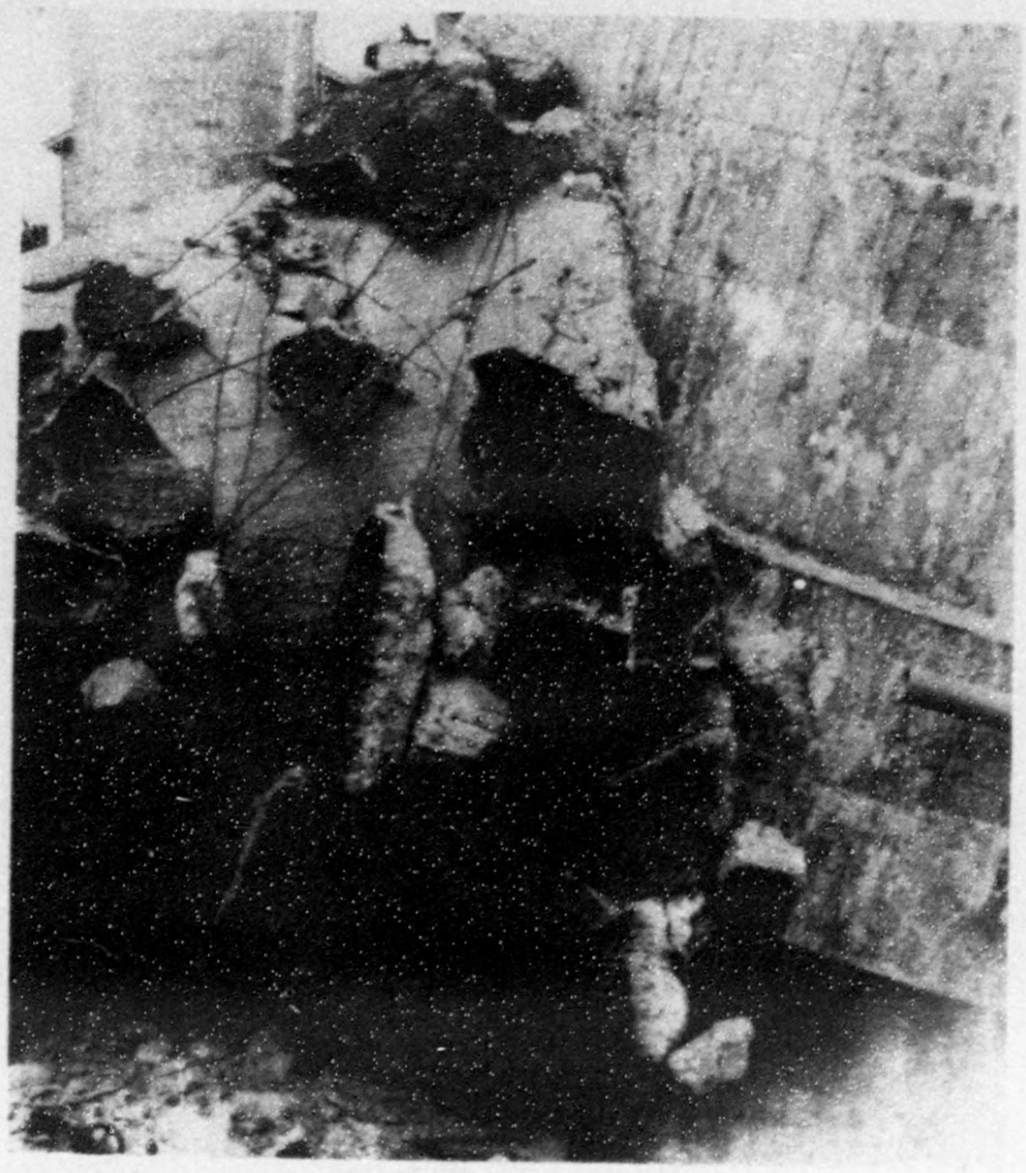


Fig. 230

Water Tank - Tipped by Chimney Section
Fukui Seiren Factory - Fukui City

Figs. 246 and 247 show the concrete spalled off and the cracks that developed around the smoke breeching opening and the clean-out opening.

Figure 248 shows a small independent chimney, 66 feet high, located at the Fukui City Hall. This chimney developed a crack just above the smoke breeching opening and inclined about 5 degrees in the southwest direction.

Figure 249 shows an independent chimney, 90 feet high, located at the Fukui Technical College. The chimney was badly cracked at about 18 feet above the ground line and inclined 8-9 degrees to the south of east. The chimney also developed minor cracks above this point. Some of these are visible in the photograph.

Figure 250 shows a brick chimney located at the municipal gas works. The chimney was 60 feet high and was integral with a brick walled building housing a battery of ovens. The chimney

ey, 120 feet
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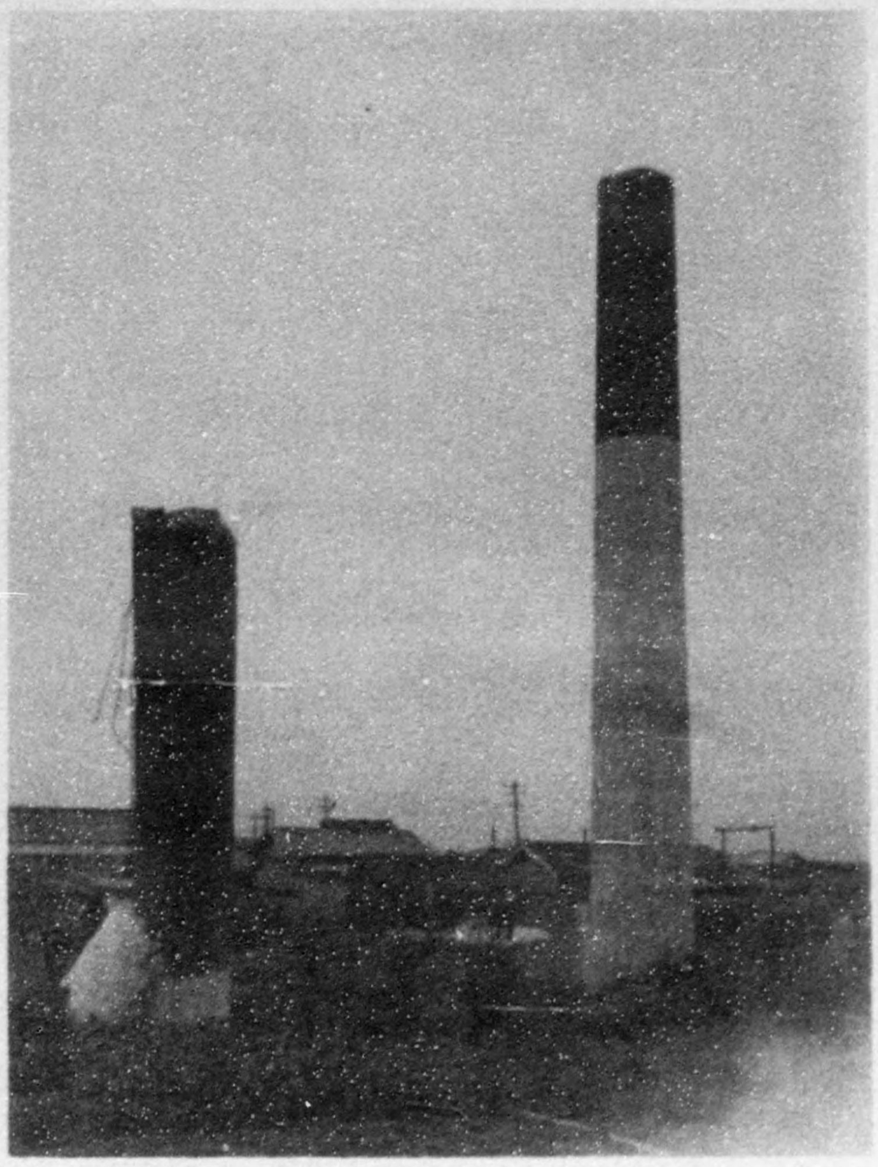

Fig. 231

Chimney Damage - Nakajima Factory
Fukui City

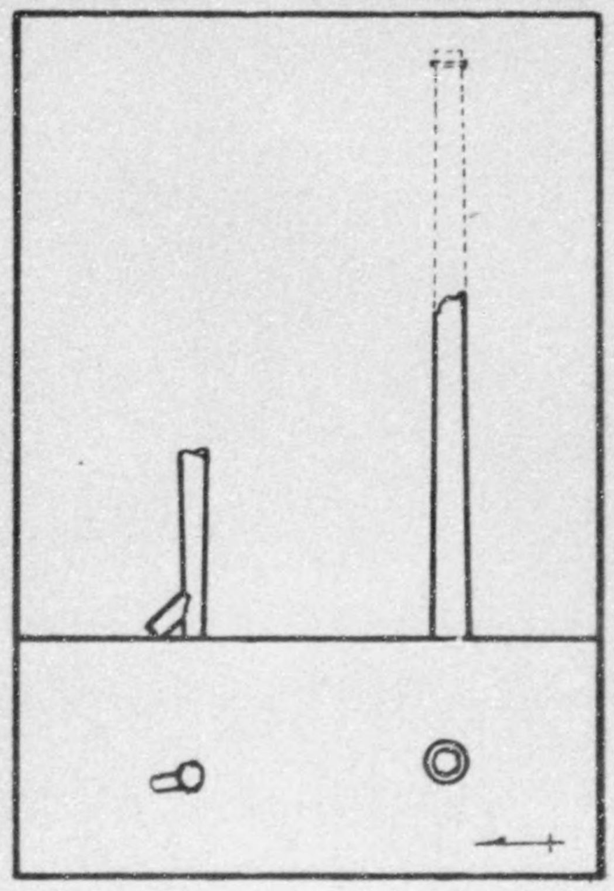

Fig. 231
Plan and Elevation - Chimney
Damage Nakajima Factory
Fukui City

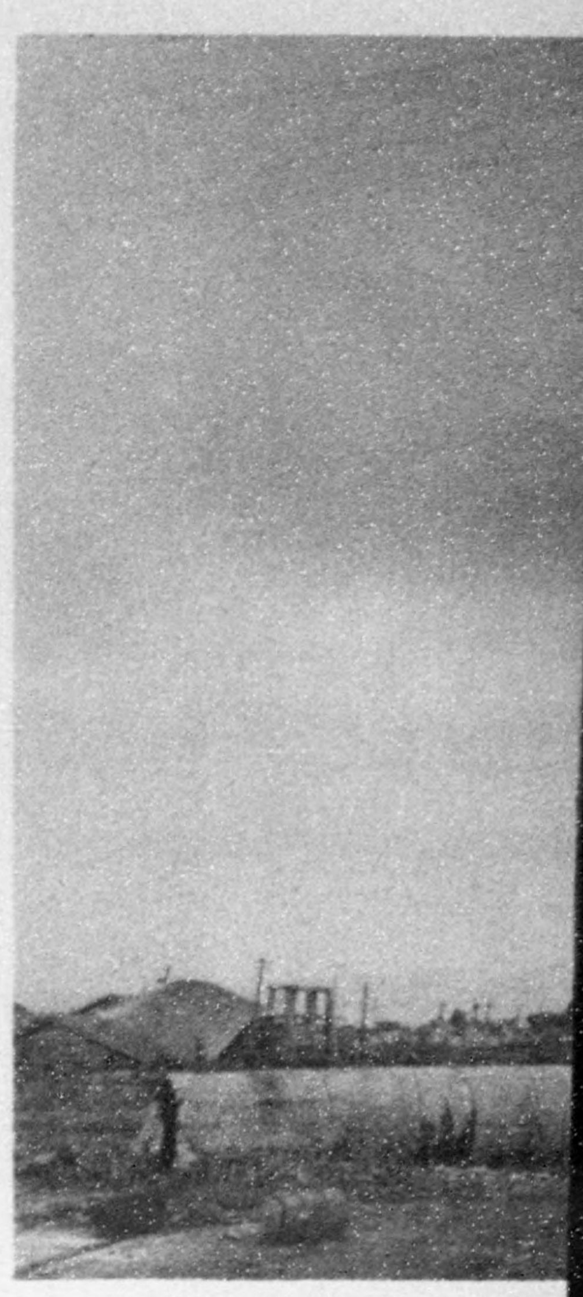

Fig. 2

Chimney Damage - Mi
Fukui C

was square in section and reinforced with steel angle corner plates and adjustable horizontal rod braces five feet apart. It is very probable that this chimney, as an integral part of the building structure, was adversely influenced by the movement of the building unit. The workmanship in the chimney construction was good as evidenced by a large majority of the breaks being through the bricks instead of along mortar joints.

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3. ANALYSIS OF DAMAGE

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a. Location of the Rupture - Prior to the great Kanto earthquake in 1923, it was found that in Japan most brick chimneys that failed did so at approximately two-thirds of the

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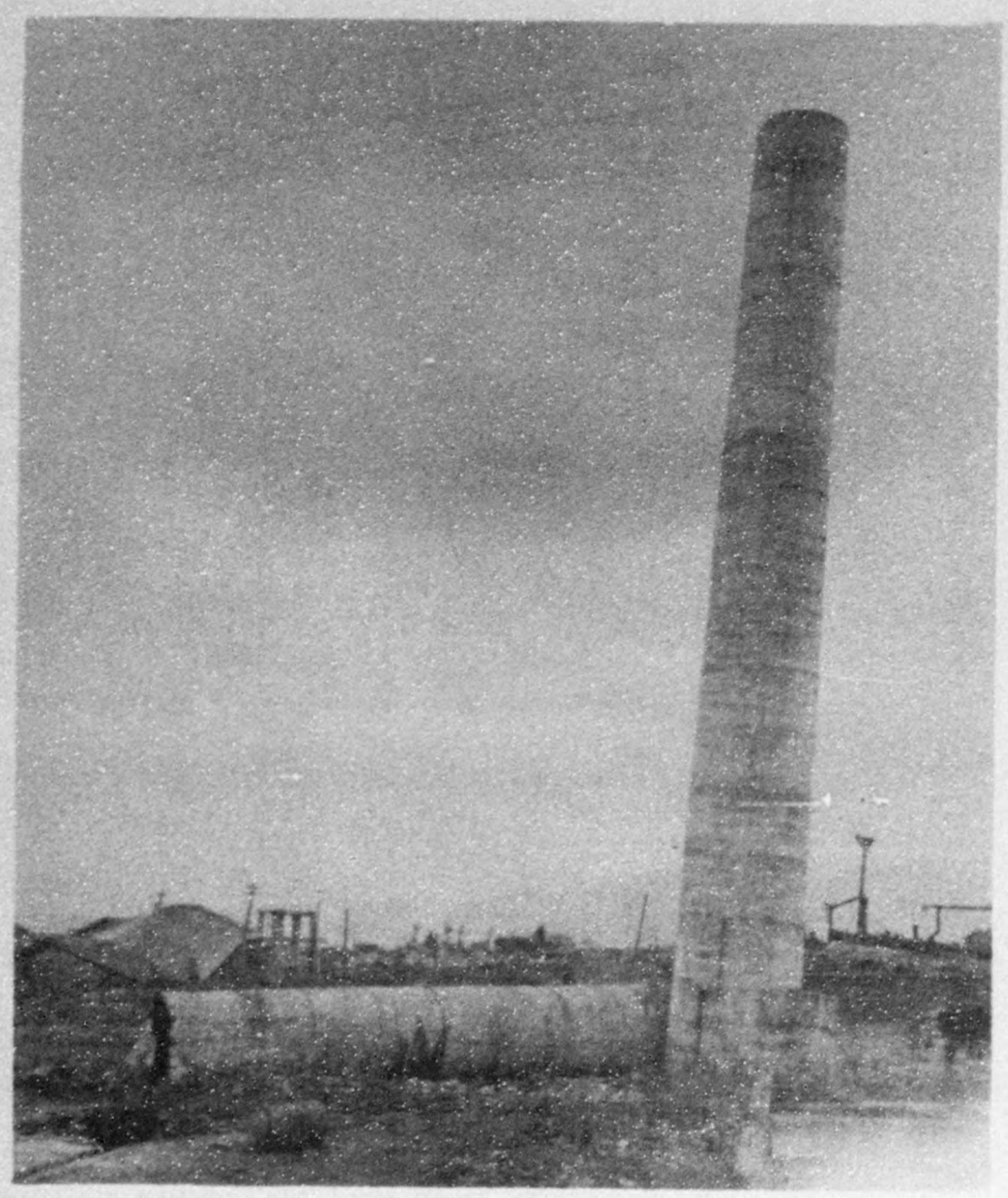

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Fig. 233

Chimney Damage - Mitsubishi Factory
Fukui City

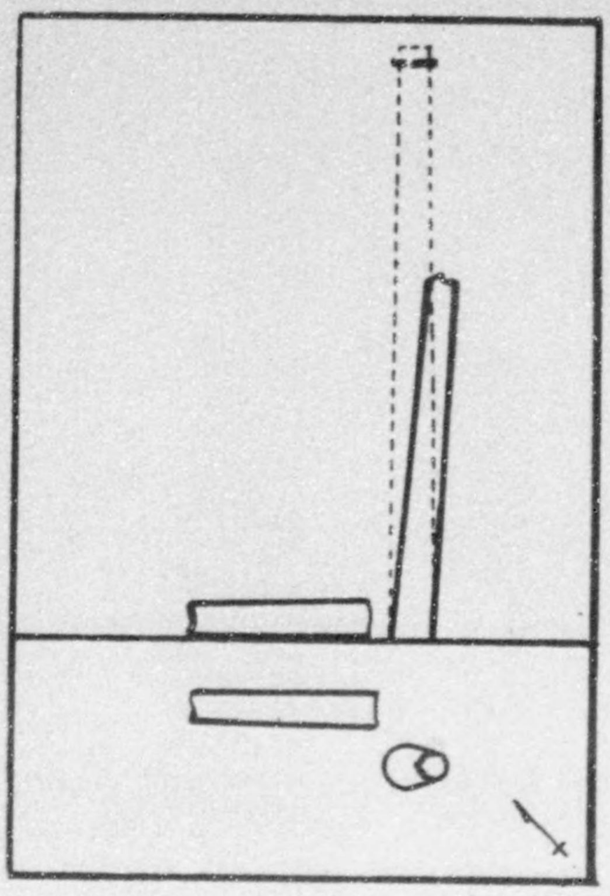


Fig. 234
Plan and elevation- Chimney
Damage Mitsubishi Factory
Fukui City

height from the ground. The belief that this relationship was true of chimneys in general has been shown to hold in certain cases only. Previously, there were only a few chimneys that suffered damage at the bottom of the shaft with failures usually occurring somewhere in the middle three fourths of the chimney height.

In the recent Hokuriku earthquake, a majority of the chimneys broke off, or were bent, in the center section. There were some that failed or developed cracks between the ground and one-fourth to one-third of their heights, and others did the same at three-fifths to two-thirds of their heights.

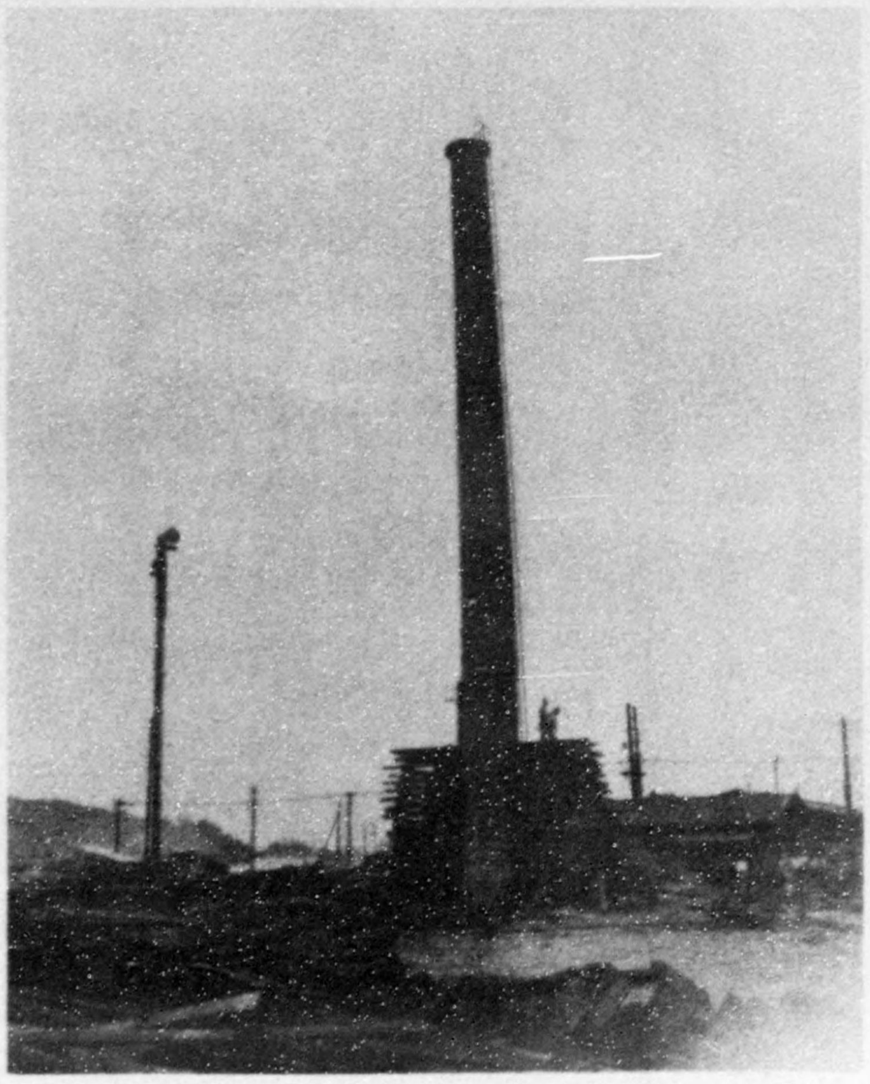


Fig. 235

Chimney Damage - Fukui Railroad Yard
Fukui City

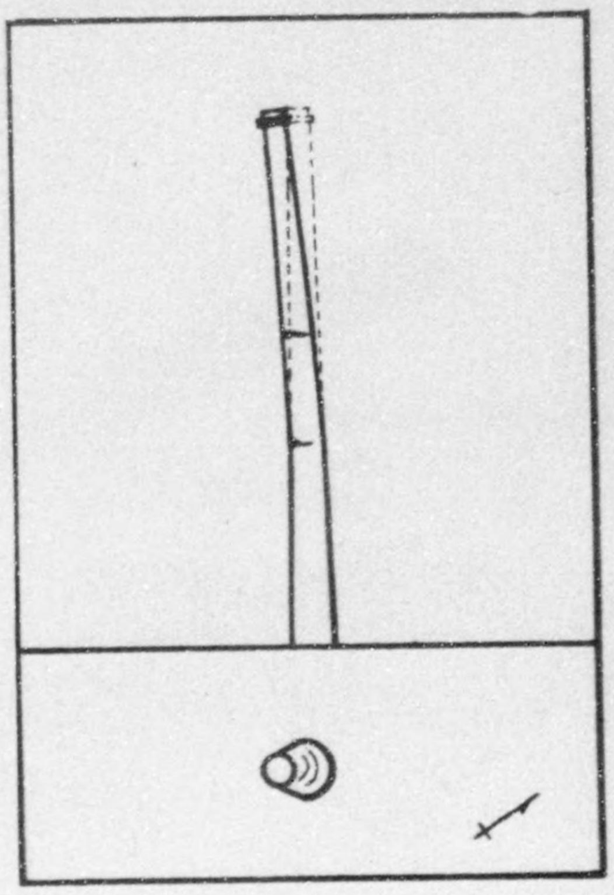
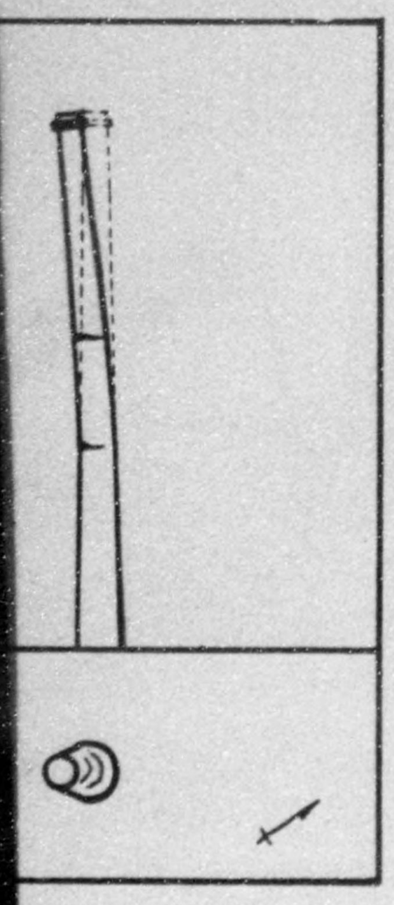


Fig. 236
Plan and Elevation- Chimney
Damage Fukui Railroad Yard
Fukui City

Observation of these failures proves that it would be difficult to say, or anticipate, at what height chimney failure occurs. In compilation of the information and calculations concerning these chimneys, Mr. T. Takahashi, a structural engineer in the Design Branch of the Engineer Section, discovered the interesting relationship noted in Fig. 251 between the points of failure of the chimneys and their heights. The point of failure apparently approaches a relatively constant distance from the top of the chimney.

b. Strength of Chimney Section at Level of Failure - The chimneys reasonably are assumed to possess rigidity that would make their natural periods of vibration less than that of



g. 236
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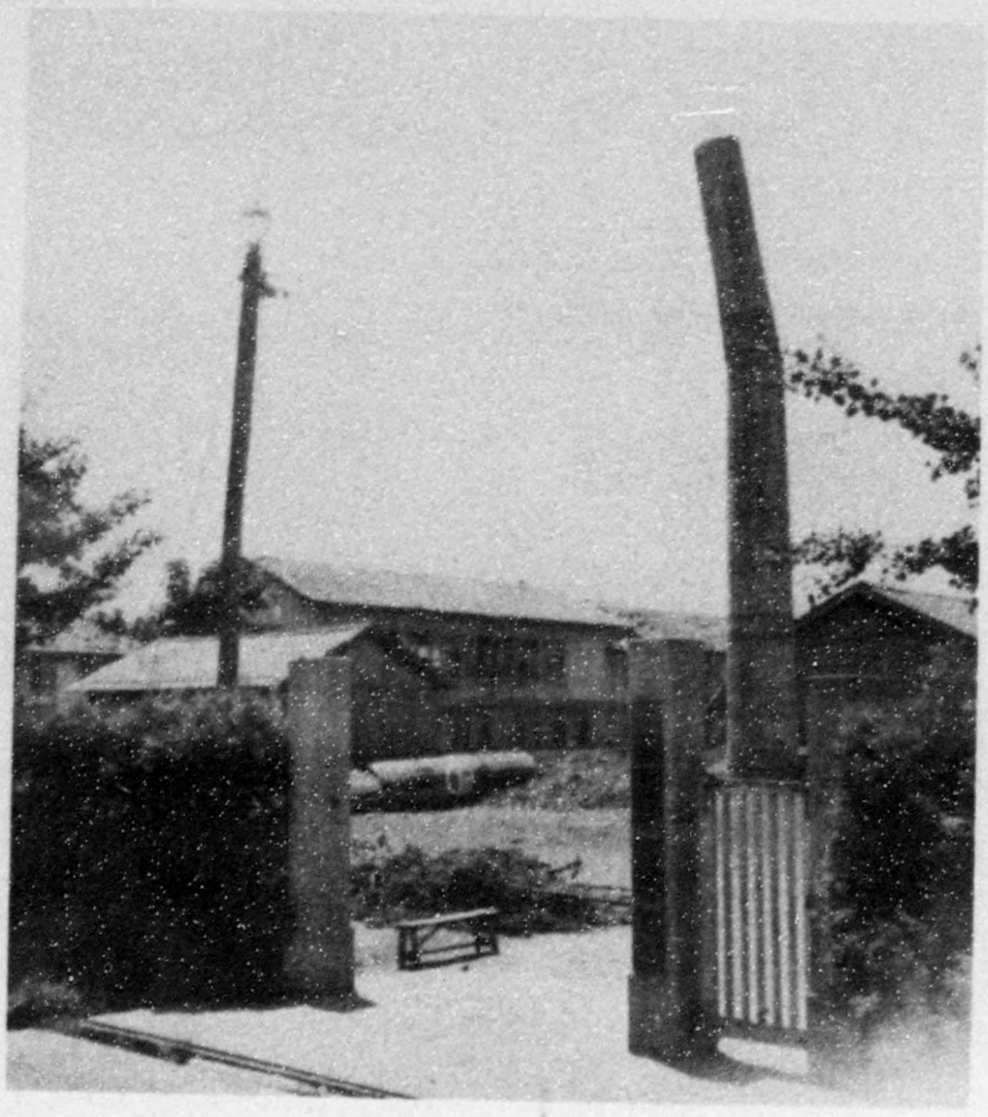

Fig. 237

Chimney Damage - Jomo Nenshi Factory
Kanazu

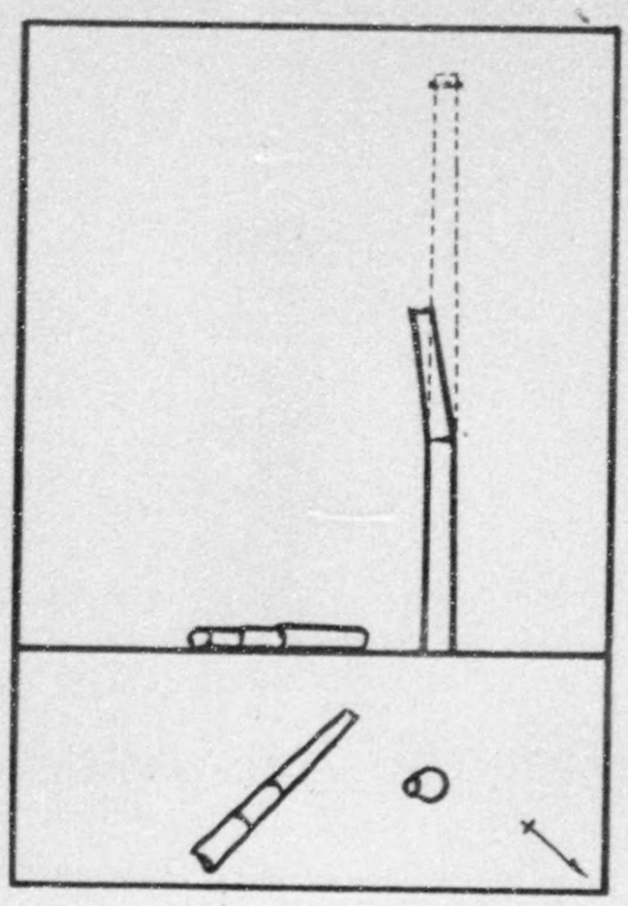

Fig. 238
Plan and Elevation - Chimney
Damage Jomo Nenshi Factory
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the earthquake. A certain horizontal component of the seismic acceleration, therefore, would act on the chimneys.

In practical design, a statical method of stress analysis is collowed, assuming a reasonable value for k, the seismic coefficient. The strength of the chimney section at the level of fracture will be calculated on the basis of this method together with the maximum theoretical seismic coefficient which these sections should have been capable of resisting. In the calculations, the usual assumptions in reinforced concrete design were used together with the code requirement, supposedly effective in the chimney's design. A modular ratio of steel to

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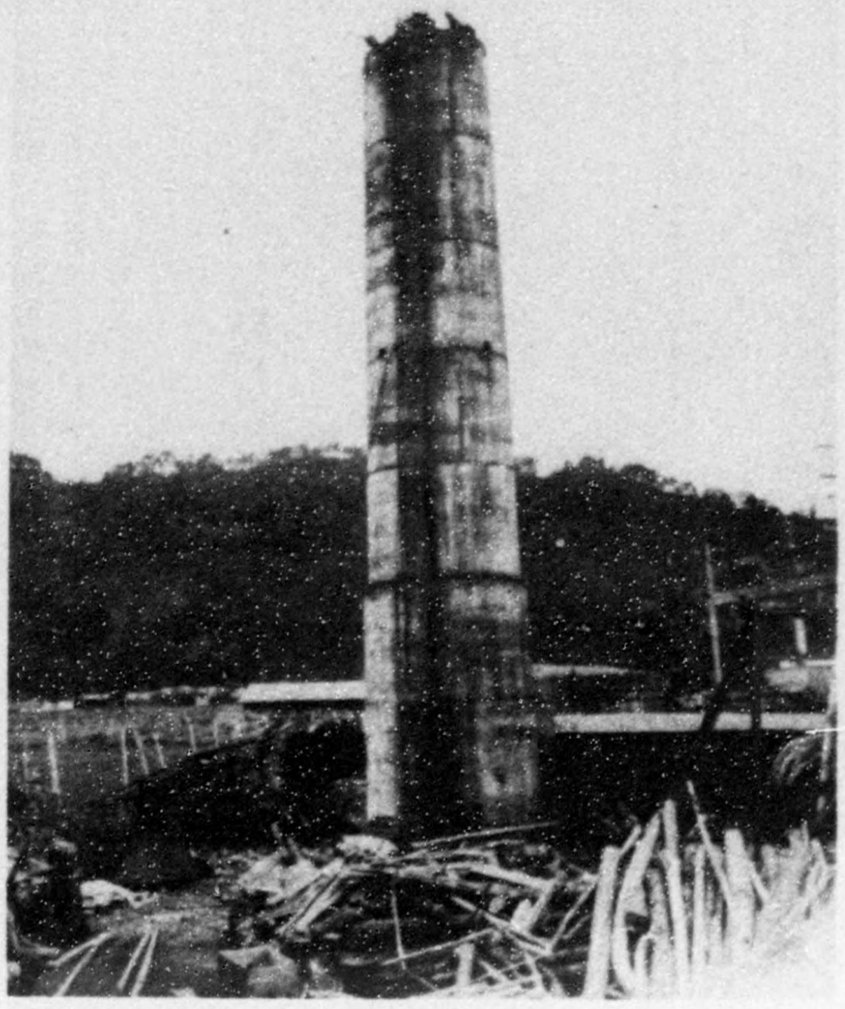


Fig. 239

Chimney Damage - Sakai Seni Factory
Honsha Plant - Fukui City

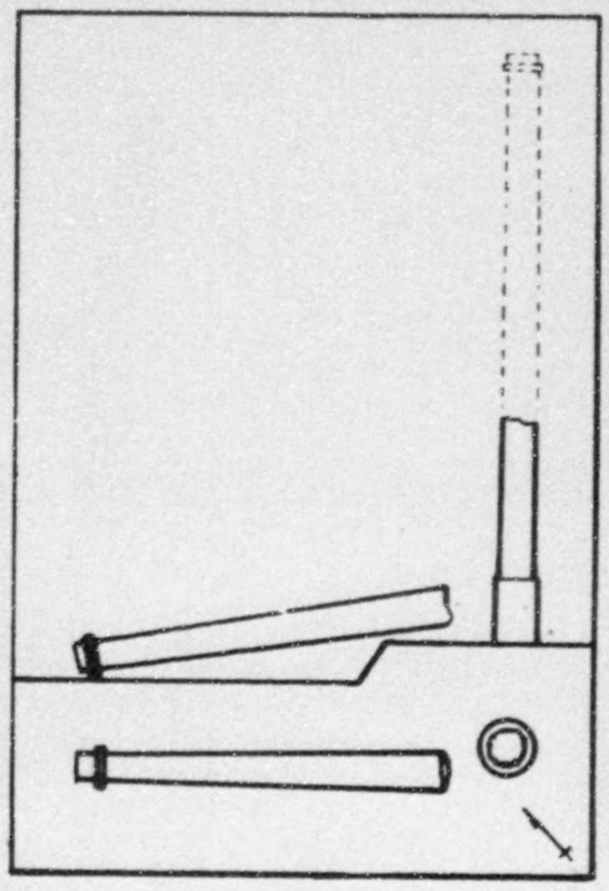


Fig. 240
Plan and Elevation - Chimney
Damage Sakai Seni Factory
Honsha Plant - Fukui City

Fig. 241

Chimney Damage - Saka
Seiren Plant - F

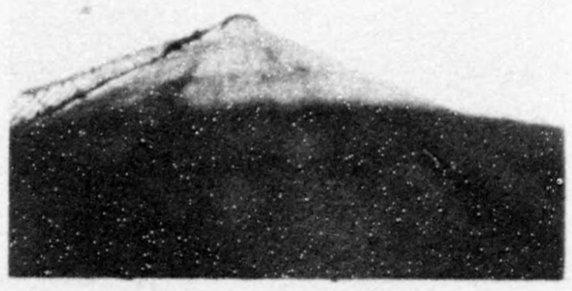


Fig. 243

Chimney Damage - Sakai Seni Factory
Seiren Plant - Fukui City

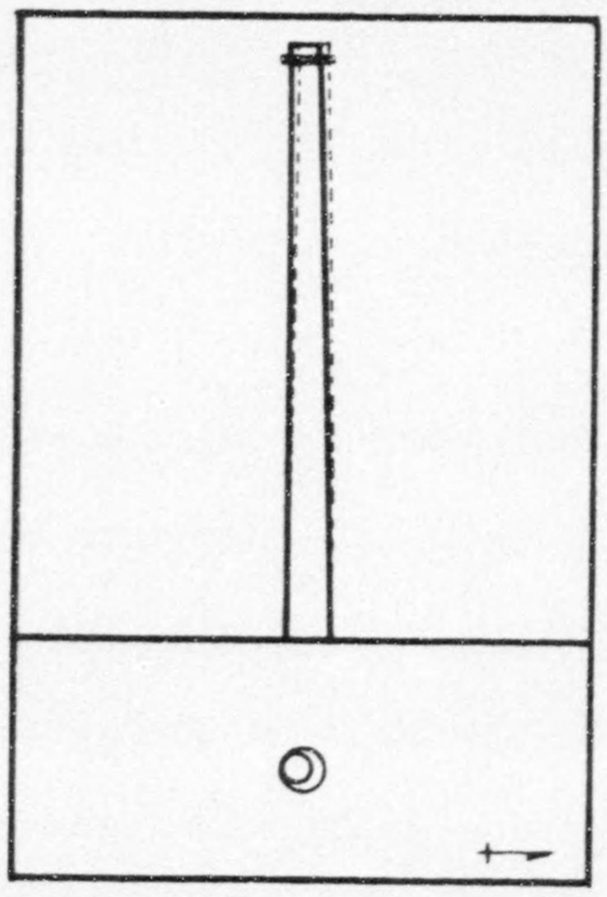


Fig. 244
Plan and Elevation - Chimney
Damage Sakai Seni Factory
Seiren Plant - Fukui City

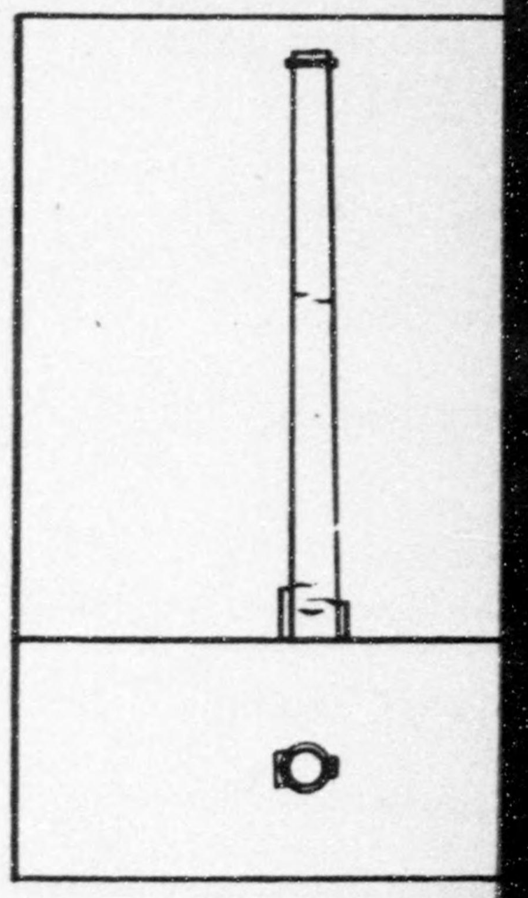


Fig. 245
Plan and Elevation - C
Damage Daiwa Boseki Fa
Fukui City

Fig. 241

Chimney Damage - Sakai Seni Factory
Seiren Plant - Fukui City

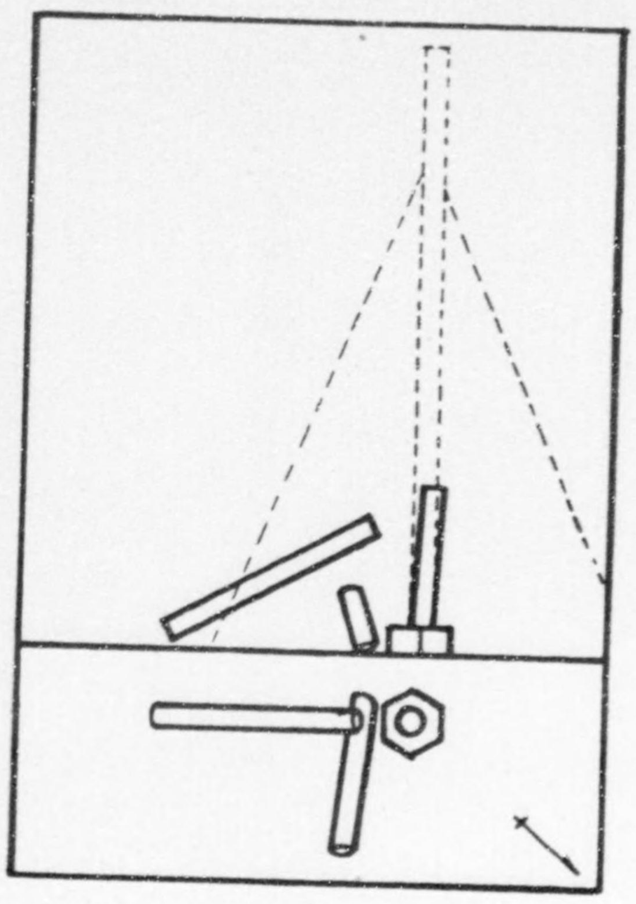


Fig. 242
Plan and Elevation - Chimney
Damage Sakai Seni Factory
Seiren Plant - Fukui City

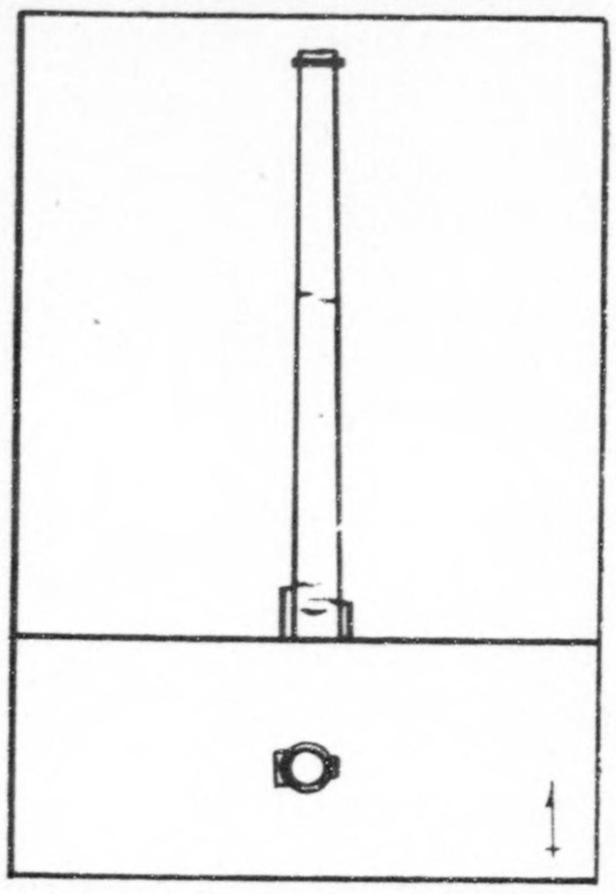
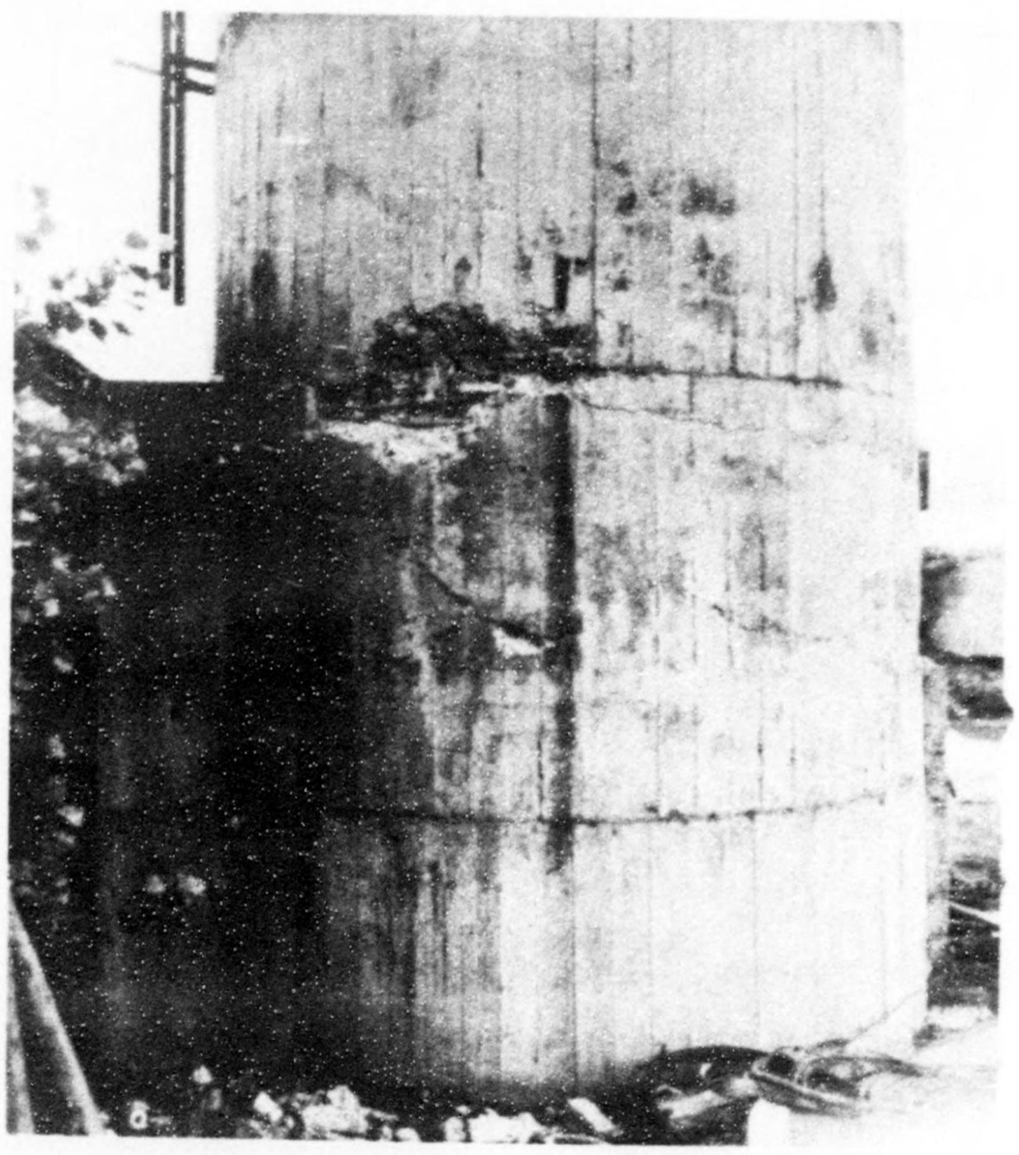


Fig. 245
Plan and Elevation - Chimney
Damage Daiwa Boseki Factory
Fukui City



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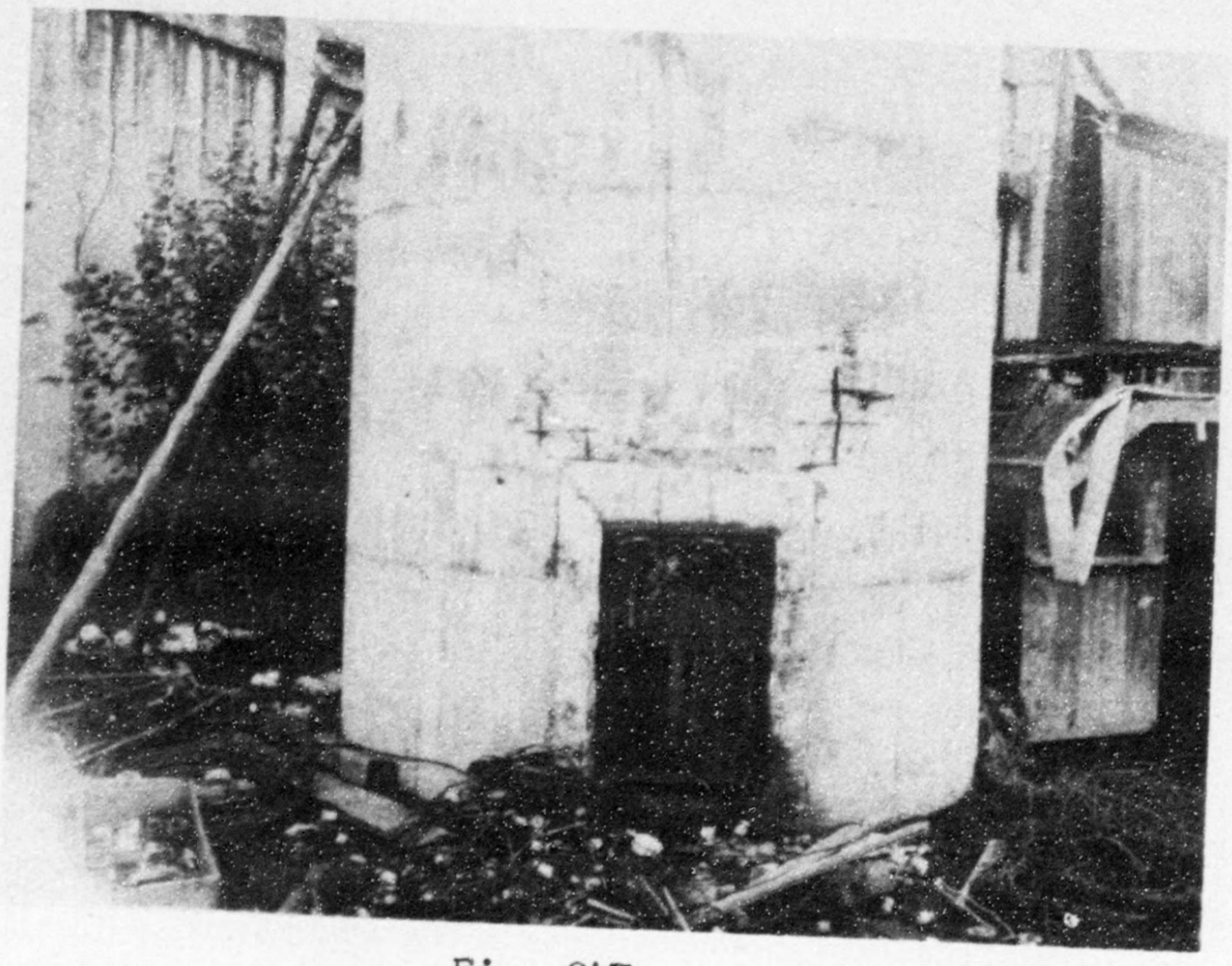

Fig. 247

Chimney Damage - Daiwa Boseki Factory
Fukui City

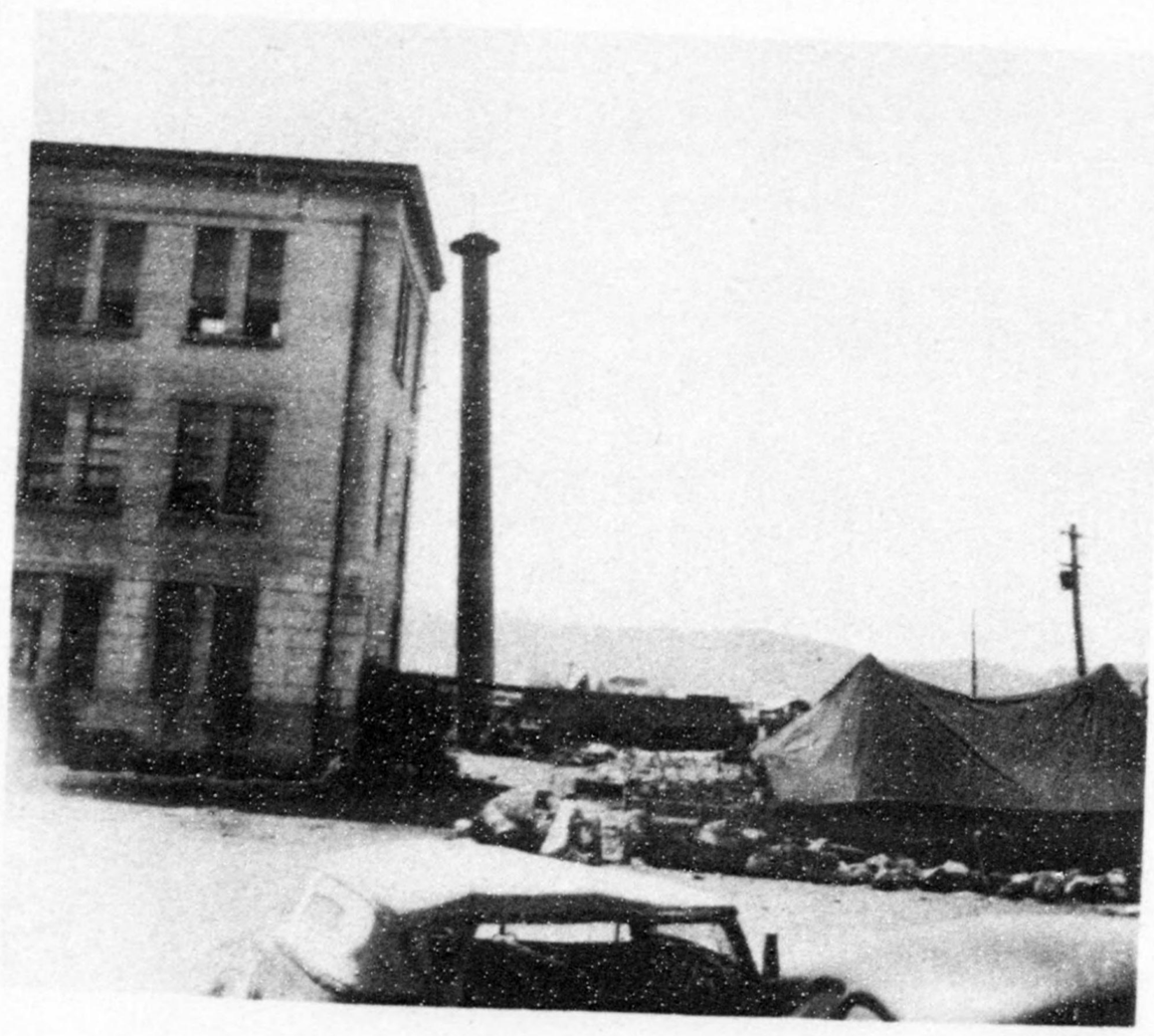

Fig. 248

Chimney Damage - Fukui City Hall

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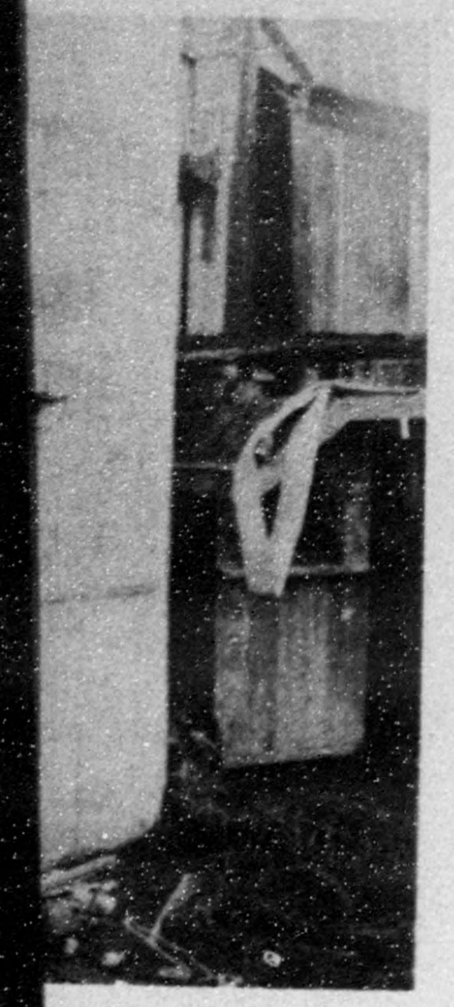

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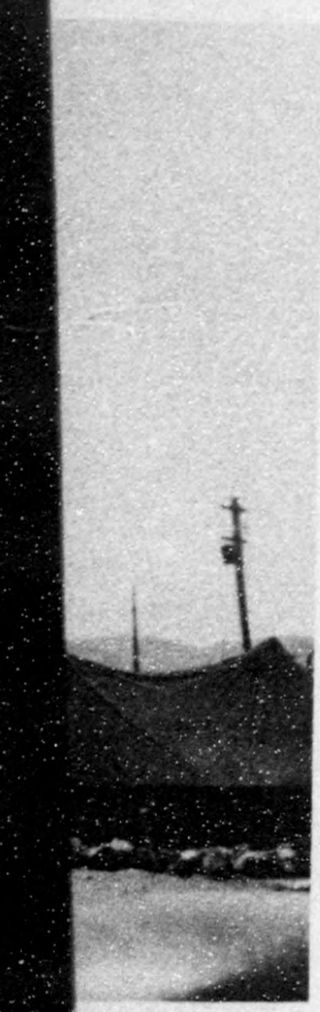

concrete of 15 is used together with the ultimate strength of concrete as 1900 psi and the yield point of steel as 34,000 psi. The vertical load and the vertical component of the seismic acceleration are neglected.

Stresses in the concrete and steel are first computed for a seismic coefficient of 0.15g. According to the provisions of the previous code, the limiting allowable stresses were 600 psi and 20,000 psi (17,000 before June 1937) for concrete and steel, respectively, and the computed stresses are compared to these values. The value of the seismic coefficient which would create a steel stress intensity of 34,000 psi was then computed, Figs. 252 and 253. The results of these calculations are shown in Fig. 254.

c. Results - The calculations show that:

(1) For a seismic coefficient of 0.15g, 10 out of 18 chimneys (55 per cent) did not satisfy the provisions and the requirements of the old code. The calculations were made considering the middle sections of the chimneys. If similar calculations were to be made for the bottom sections, it is expected that the percentage of chimneys failing to meet requirements would be increased.

(2) The ultimate seismic coefficients are shown to vary between the range of 0.07g. to 1.0g.

It is to be expected that chimneys which were lacking in strength should fail, but it is difficult to explain why

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chimneys seemingly possessing high strength should also fail. It is clear, after analyzing the results closely, that other points also need clarification. For instance, chimneys which had ultimate seismic coefficients ranging from 0.07 to 0.15 developed cracks at the bottom of the shaft while chimneys with seismic coefficients ranging from 0.4 to 1.0 were broken in the upper sections. It is difficult to explain why the chimneys should fail in the upper sections when those sections usually have a higher residual strength than the lower sections. We have attempted in the remainder of this Section to find some of the answers to the peculiar behavior of these chimneys.

4. DYNAMICAL PROPERTIES OF CHIMNEYS

a. Periods of Principal Vibration - The periods of vibration will be calculated by two methods. The first will be done by utilizing a formula derived by Dr. N. Mononobe and the second by the so-called gravity formula.

(1) Mononobe's Formula - This formula provides a solution for the principal vibration of conical cylinders, fixed at the base, and wherein the wall thickness is proportional to the diameter. The formula is as follows:

$$To = C \frac{h^2}{r} \sqrt{\frac{P}{E}}$$ where

h = height

r = radius of gyration at the base

P = density

E =

H_1 &

C =

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T =

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E = Young's Modulus

H_1 & H = As shown by Fig. 255

$$C = 0.719 + 0.1068 \frac{H_1}{H} + \left[0.113 - 0.452\left(0.5 - \frac{H_1}{H_2}\right)^2\right]$$

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The results of the calculations using this formula are given in column 4 of Fig. 256.

(2) Gravity Formula - The period of principal vibration, T, for the case when the footing develops elastic rotation with values of $\theta = 1/1{,}000$ to $1/20$, and the seismic coefficient is k = 1.0g, has been computed from the following gravity formula:

$$T = \sqrt{\frac{y_o + \theta h}{6.2}} \quad \text{where}$$

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$\sqrt{y_o} = 6.2\ To$

h = height

θ = angle of rotation as shown by Fig. 257.

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The results of the calculations by this formula are shown in columns 5, 6, 7, and 8 of Fig. 256.

d. Study of Stresses - The period of vibration for chimneys, assumed to have fixed bases, is about 0.41 seconds for low chimneys and 0.89 seconds for high chimneys. The period of vibration for chimneys considered to undergo elastic rotation at the base by an amount equal to $\theta = 1/100$ becomes about 0.83 seconds for short chimneys and 1.5 seconds for tall chimneys. The lengthening of the period is not great, as noted above.

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The principal or resultant period of vibration of the alluvial plains area around Fukui during the earthquake is believed to be about 1.5 seconds. Considering the earthquake motions to be smooth, simple harmonic motions, it is not possible to imagine that especially unfavorable moments developed in the chimney shafts. It does not appear possible that one or two wave cycles would create greater bending moments in the upper section than in the base section. In regard to the possibility of secondary vibrations causing failure, this also does not appear possible as the periods of these vibrations are much smaller than those of the main vibrations.

5. FACTORS INFLUENCING FAILURE - There were several factors which came to the notice of the investigating party that perhaps can be listed as contributory causes of failure.

a. Construction - Many cases were noted where the reinforcing steel had slipped due to insufficient anchorage. It is very probable that the steel did not, in these cases, develop its full strength. During construction of a chimney, the top sections are necessarily subjected to swaying which prevents full development of the bond strength. This factor may be responsible for damage suffered in earthquakes.

b. Heat Effect - Inspection of the fractured chimneys usually did not disclose any fire-brick linings. The absence would indicate that the inside surface was subjected to the heat of the gases. It is to be noted that stack gas temperatures

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below 350°F are seldom encountered except when an economizer installation is used. Such heat could cause cracking of the concrete on the outside surface of the shafts. Considered as a plain concrete chimney, the shaft elongation of the outside surface becomes approximately 0.002 of the shaft length, for a change in temperature of 392°F. Such strains are enough to create fine cracks (visible on inspection) on the outside surface. Such cracking would lessen the strength of the concrete and probably would affect the bond strength of the concrete and steel.

6. CONCLUSION - The search for possible inherent causes of the failure of chimneys in the disaster area disclosed that many chimneys did not satisfy code requirements. The fact that many of them failed, not at the base but in upper sections, is a phenomena difficult to explain by use of existing theory. It is possible that razing of the damaged chimneys will give more information concerning the quality of the concrete, the failure of reinforcing steel, and the bond strength. It is believed that experimental investigation is in order on chimney vibration periods when the earthquake vibration is of extremely large amplitudes.

B. REINFORCED CONCRETE ELEVATED TANKS

1. General - In the disaster area, there were a number of elevated tanks of relatively small capacities and low heights. These tanks were generally of all steel construction and were

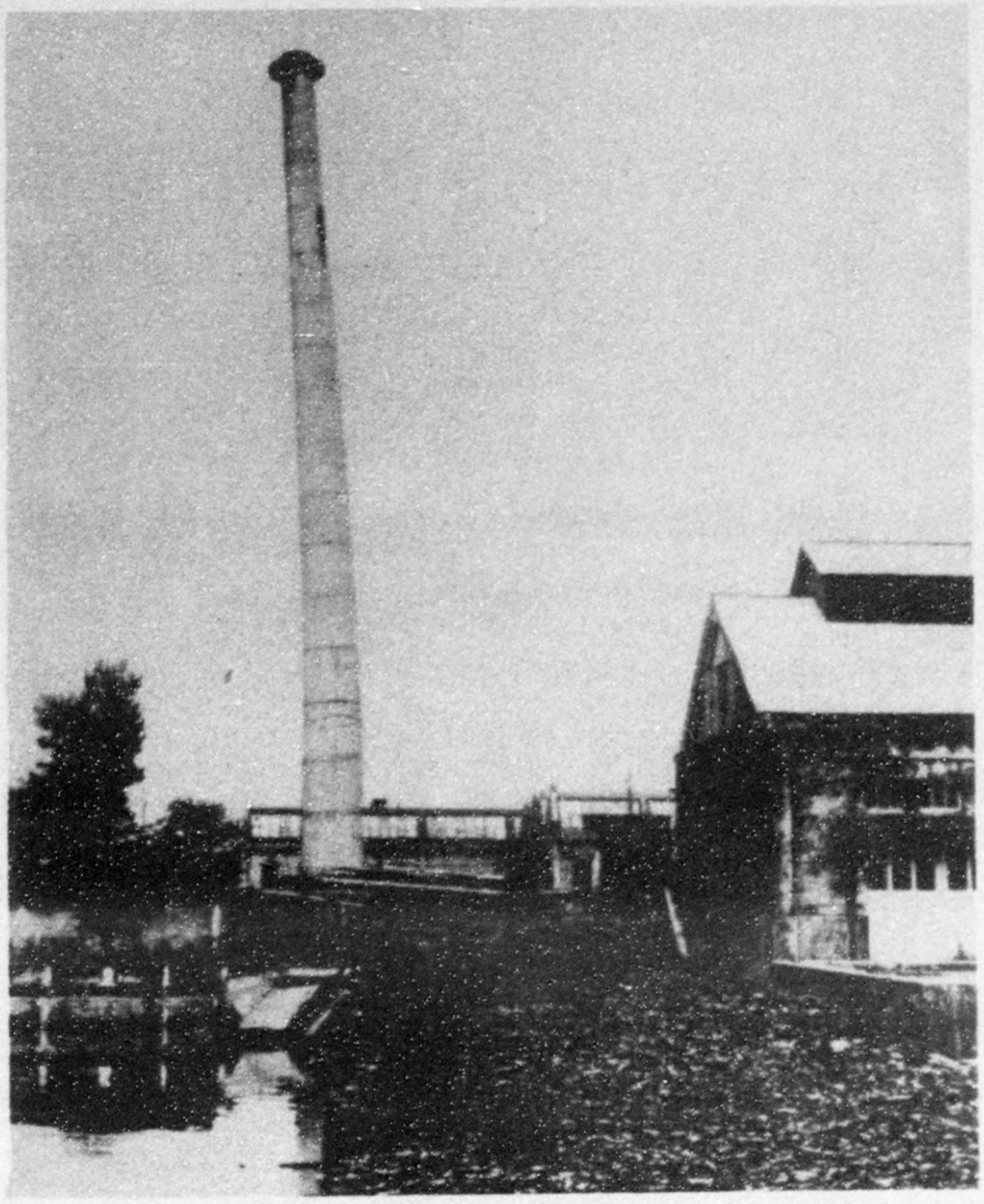

Fig. 249

Chimney Damage - Fukui Technical College
Fukui City

notably undamaged by the earthquake. A few small tanks of all reinforced concrete construction existed. The only one which failed or suffered damage, excepting minor cracking, was the tank located at the Fukui City railroad coaling yard and round-house area.

2. FUKUI CITY RAILROAD YARD WATER TANK

a. General - This tank had a capacity of approxi-

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Fig. 250

Chimney Damage - Municipal Gas Plant
Fukui City

imately 39,000 gallons. The tank was 20 feet in diameter and 16 feet, 4 inches high. It was supported by 6 columns and provided with a strong foundation. The supporting column length was about 40 feet and subdivided into 3 panels at which points the columns were tied together with beams.

b. Investigation of Damage - Fig. 258 is an elevation and cut-away section of the tank. It is a copy of the working drawings from which the tank was supposed to have been constructed. The tank was built during the war (about 1944) and, as will be shown later, did not possess adequate resistance to seismic forces. It will also be noted that the beam joints at columns

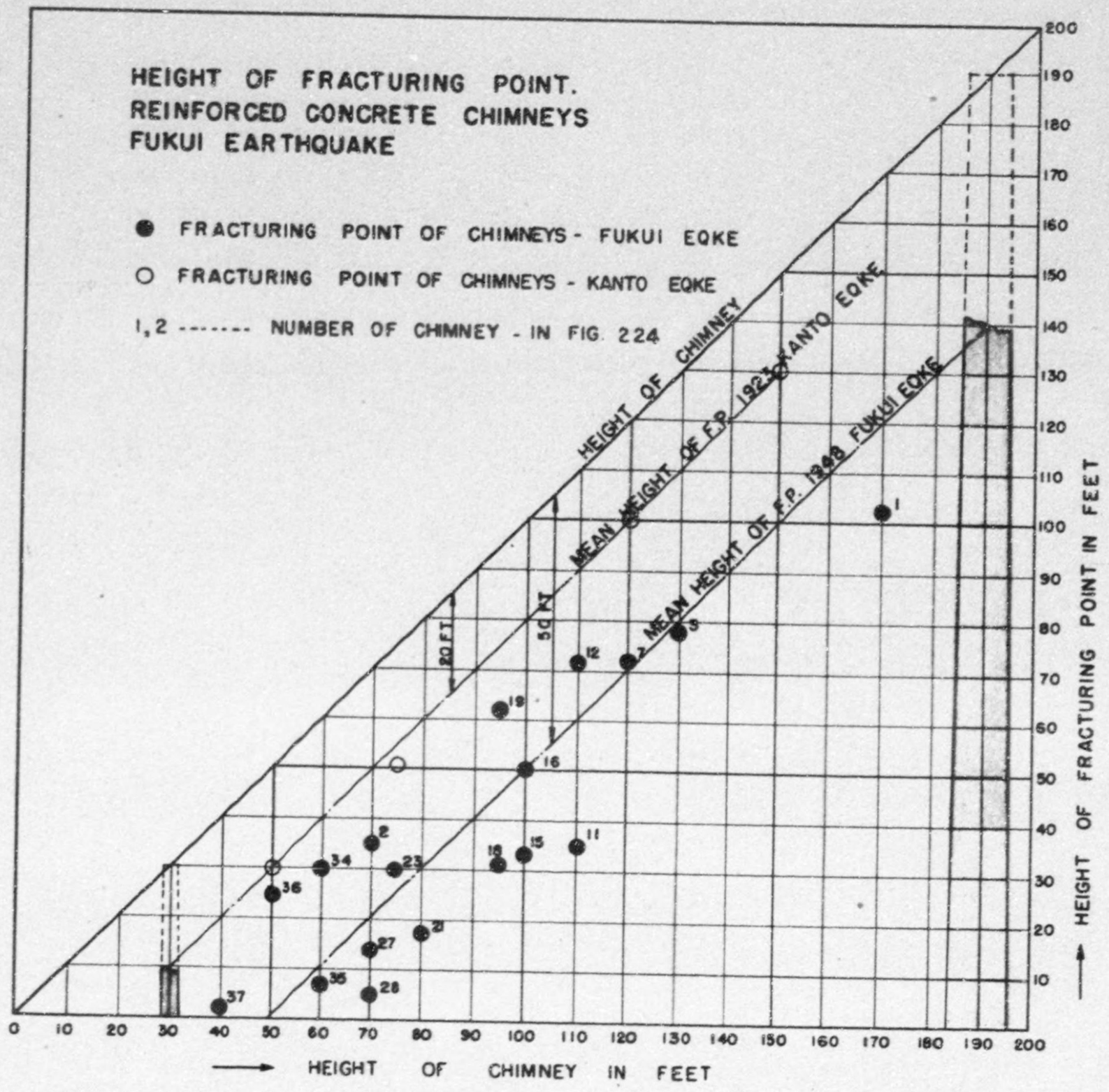
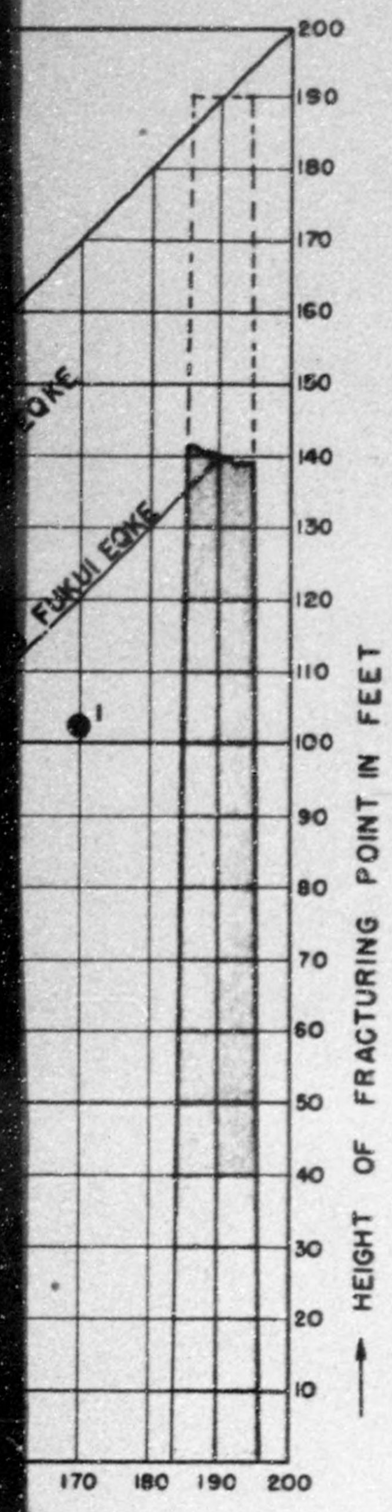
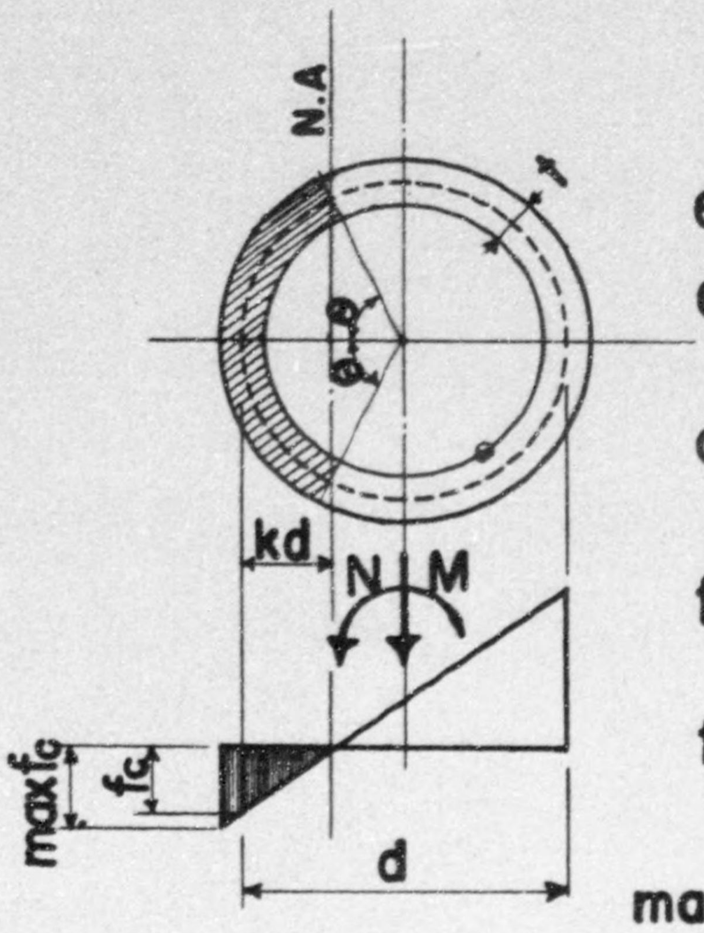


Fig. 251

were changed somewhat during the course of construction.

In order to ascertain the cause of failure, the ability of the tank structure to resist seismic forces, as well as the effect of soil conditions and period of vibration, was investigated. The solution for stresses in solid frames, such as this tower, is theoretically complicated and the results are open to doubt. Such an analysis should take into consideration lateral





$$e = \frac{M}{N}$$

$$e_1 = \frac{e}{d} = \frac{\theta - \frac{1}{2}\sin 2\theta + np}{\sin\theta - \theta\cos\theta - np\pi\cos\theta}$$

$$Q = \frac{(1-\cos\theta)\pi}{\sin\theta - \theta\cos\theta - np\pi\cos\theta}$$

$$f_c = Q\,\frac{N}{\pi d t}$$

$$f_s = nf_c\,\frac{1-k}{k}$$

$$\max f_c = f_c\left(1 + \frac{t}{2kd}\right)$$

LEGEND

N = WEIGHT OF THE CHIMNEY ABOVE THE SECTION UNDER CONSIDERATION

M = BENDING MOMENT ABOUT THAT SECTION DUE TO EQK.

f_c = MAX. COMPRESSIVE CONCRETE UNIT STRESS

f_s = MAX. UNIT STEEL TENSILE STRESS

d = MEAN DIAMETER OF SHELL

t = TOTAL THICKNESS OF SHELL

K = RATIO OF DISTANCE OF NEUTRAL AXIS, FROM CENTERLINE OF SECTION ON COMPRESSION SIDE, TO DIAMETER d

n = RATIO OF MODULUS OF ELASTICITY OF STEEL TO THAT OF CONCRETE = 15

e = ECCENTRICITY, THAT IS, THE DISTANCE FROM VERTICAL AXIS OF CENTER OF GRAVITY OF THE SECTION TO THE POINT OF APPLICATION OF THE THRUST

e_1 = RATIO OF ECCENTRICITY e TO DIAMETER d

p = RATIO OF STEEL TO TOTAL AREA OF SECTION

Fig. 252

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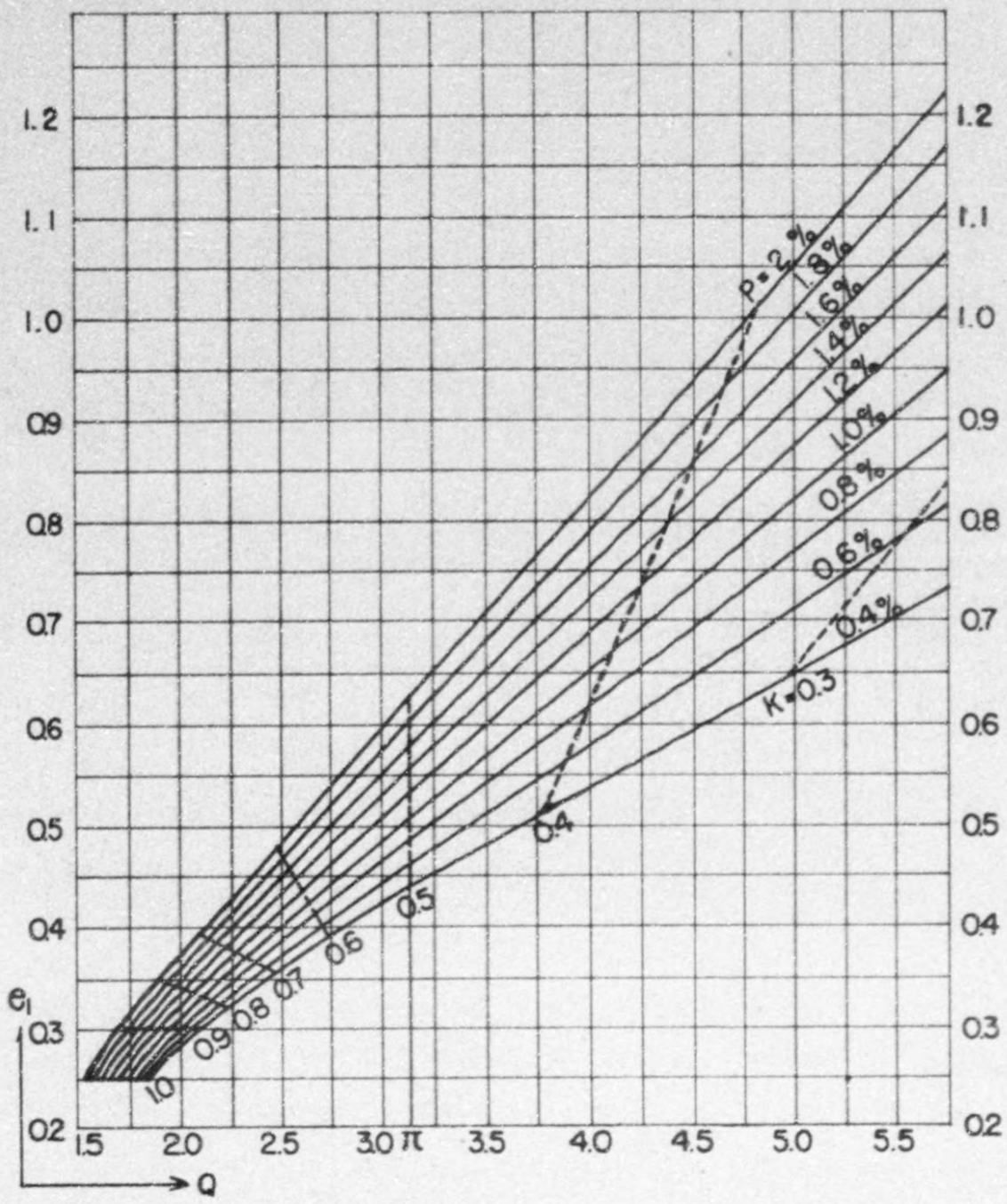

Fig. 253

buckling of columns which would make the calculations even more intricate. For the purposes of this presentation, a simplified approximate method is therefore used.

Tank weight.....,..126 kips

Water.....290 kips

Total 416 kips

Earthquake Force = k x 416

Wherein k = Seismic coefficient

ANALYSIS OF CHIMNEY SECTIONS AT LEVEL OF FRACTURE

	NAME OF CHIMNEY	1 H (FT)	2 OUTER DIAMETER TOP (FT)	2 OUTER DIAMETER BOTTOM (FT)	3 t (IN)	4 BREAKING POINT FROM BOTTOM (FT)	5 d (FT)	6 N (KIPS)	7 M (FT. KIPS)	8 p	9 $e=\frac{M}{N}$ (FT)	10 $e_1=\frac{e}{d}$	11 max f_c (PSI)	12 f_s (PSI)	13 ULTIMATE SEISMIC COEFFI-CIENT
1	FUKUI SEIREN	170	10.0	15.0	6.0	3/5 H	10.4	162	850	0.0033	5.2	0.50	287	6,600	0.77
3	SAKAI SENI	130	8.0	10.0	6.7	3/5 H	7.6	101	405	0.0023	4.0	0.53	250	6,400	0.79
7	NAKAJIMA ORIMONO	120	5.6	9.2	5.1	3/5 H	5.9	54	193	0.0028	3.6	0.61	270	9,200	0.55
11	MARUSAN SENSHOKU	110	8.0	11.2	7.0	1/3 H	7.9	154	860	0.0028	5.6	0.71	480	19,000	0.26

ANALYSIS OF CHIMNEY SECTIONS AT LEVEL OF FRACTURE

	NAME OF CHIMNEY	1	2		3	4	5	6	7	8	9	10	11	12	13
		H (FT)	OUTER DIAMETER TOP (FT)	OUTER DIAMETER BOTTOM (FT)	t (IN)	BREAKING POINT FROM BOTTOM (FT)	d (FT)	N (KIPS)	M (FT. KIPS)	p	$e=\frac{M}{N}$ (FT)	$e_1=\frac{e}{d}$	max f_c (PSI)	f_s (PSI)	ULTIMATE SEISMIC COEFFICIENT
1	FUKUI SEIREN	170	10.0	15.0	6.0	3/5 H	10.4	162	850	0.0033	5.2	0.50	287	6,600	0.77
3	SAKAI SENI	130	8.0	10.0	6.7	3/5 H	7.6	101	405	0.0023	4.0	0.53	250	6,400	0.79
7	NAKAJIMA ORIMONO	120	5.6	9.2	5.1	3/5 H	5.9	54	193	0.0028	3.6	0.61	270	9,200	0.55
11	MARUSAN SENSHOKU	110	8.0	11.2	7.0	1/3 H	7.9	154	860	0.0028	5.6	0.71	480	19,000	0.26
12	NEAR ASUKA MOUNTAIN	110	4.3	8.6	4.7	2/3 H	4.6	30	86	0.0029	2.9	0.62	210	7,100	0.71
15	DARUMAYA	100	3.3	5.5	6.0	1/3 H	3.6	52	255	0.0053	4.9	1.37	1,030	△50,000	0.10
16	MARUNI SANGHO NO.1	100	4.1	6.5	6.7	1/2 H	4.2	52	200	0.0065	3.8	0.92	380	11,700	0.43
18	FUKUI PREFECTURAL OFFICE	95	3.6	5.9	6.0	2/3 H	3.5	25	60	0.0054	2.4	0.68	264	7,000	0.72
19	SAKAI SENI MAIN FACTORY	95	4.6	7.2	6.0	1/3 H	5.0	73	362	0.0045	5.0	1.00	630	△27,000	0.19
21	FUKUI RED CROSS NO2	80	3.8	5.5	6.7	1/5 H	4.0	63	305	0.0054	4.8	1.20	760	△33,000	0.15
23	" " NO.1	76	3.0	4.8	6.7	2/5 H	3.0	34	116	0.0047	3.4	1.15	600	△24,000	0.21
24	MARUNI SANGHO NO.2	75	4.3	6.2	ASSUME 8	CRACK AT BOTTOM	4.7	74	455	0.0030	6.2	1.32	830	△55,000	0.09
27	TUBOGANE ORIMONO	70	3.3	4.9	6	1/5 H	3.5	43	185	0.0054	4.3	1.22	665	△28,500	0.18
28	FORMERLY OKARI ORIMONO	70	3.3	4.9	6	BOTTOM	3.6	56	300	0.0069	5.4	1.50	950	△40,500	0.13
31	FUKUI CITY HALL	66	3.0	5.1	8	CRACK AT BOTTOM	3.3	66	335	0.0136	5.1	1.50	950	△24,000	0.21
34	KOKUSAI - KOKU	60	3.1	4.9	6	1/2 H	3.1	23	57	0.0040	2.5	0.81	313	11,800	0.43
35	NANKAI RAYON	60	2.8	4.0	6	1/7 H	2.8	33	131	0.0050	4.0	1.43	840	△39,000	0.13
36	KAKUMASA SOMEMONO	50	2.8	4.0	4	1/2 H	2.7	10	20	0.0067	2.0	0.74	230	3,750	1.34

△ INDICATES STRESSES ABOVE CODE ALLOWABLE.
THE ALLOWABLE STRESSES OF THE OLD CODE f_c = 630 PSI.
f_s = 20,000 PSI.

THE STRESSES IN COLUMN 11.12 HAVE BEEN CALCULATED FOR AN EARTHQUAKE COEFFICIENT. k = 0.15

Fig. 254

Neglecting the dead weight of the supporting structures, the compression in the columns would be under normal conditions $\frac{416}{6}$ = 70 kips.

The forces acting on the tower (Fig. 259) will be considered as P on the tower and P_2 acting at an assumed point (apex of the triangle formed by extending the columns). P_2 creates uniform direct stress in the columns and as the tank is considered a rigid structure, the columns may be treated by the equations for uniform bending. In this case:

P = 416 x k (kips) acting 8 feet above P_1.

P_1= 0.94 P = 390 k (kips)

P_2= 0.06 P = 26 k (kips)

The moment at the bottom of the tower columns due to P_2 = 4200 k (ft. kips) and the direct stress in the columns can be calculated from the equation (using Fig. 260):

$$N = \frac{M}{J} y$$

where y = perpendicular distance to columns from N-N

$$J = \sum y^2 = 4.0 \times 6.6^2 + 2 \times 13.0^2 = 514 \text{ ft.}^2$$

The stress in the outer columns N_1 and N_2 is

$$N_1 = \frac{4200 \text{ K}}{514} \times 13 = 104 \text{ k (kips)}$$

$$N_2 = 0.5N_1 = 52 \text{ k (kips)}.$$

When a horizontal force acts on the top of the solid frame, A_1 A_4 D_4 D_1, as shown in Fig. 261, a standard method of solution may be used when the frames are parallel, but when there are

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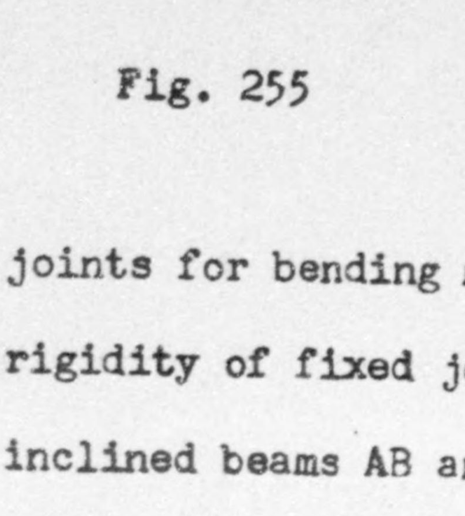

Fig. 255

inclined sides as AB and CD the solution becomes complicated. It is assumed that: (1) The column bases A_1, B_1, C_1, and D_1 are fixed. (2) The column tops A_4, B_4, C_4, and D_4 do not rotate. (3) For frame BC, only stresses occurring in a plane frame will be considered. (4) The oblique beams AB and CD will undergo deformation in the direction of the lateral force and will be resisted by frame BC. It is assumed that the joints along row A are nearly pin joints for bending moments and that they possess half the rigidity of fixed joints in resisting torsion. Stresses in the inclined beams AB and CD are assumed to be created by rotation of joints along rows B and C. (5) The outside columns A and D resist the horizontal shears, neglecting the presence of tie beams. Based on these assumptions, the resulting stresses are shown by Fig. 262.

The maximum stresses are found to occur at the base of the inside columns where M = 720 k (ft. kips), N = + 500 k (kips), and the moment in the beams of 800 k which, when reduced for the effect of the haunch, becomes 640 k (ft. kips).

In considering the earthquake resisting power of the tower, the combined maximum stresses, utilizing the vertical as well as horizontal forces, become as follows:

NO.	NAME	HEIGHT ft	PERIOD[1] sec	PERIOD WHEN BAS	
				$\theta=\frac{1}{1000}$[2]	$\frac{1}{200}$
1	CITY HALL	66	0,41	0,48	0,64
2	PREF OFFICE	95	0,74	0,79	0,96
3	NAKAJIMA	120	0,75	0,81	1,0
4	MITSUBISHI	130	0,85	0,91	1,1
5	FUKUI SEIREN	170	0,89	0,96	1,2

PERIOD OF PRINCIPAL VIBRATION

Fig. 256

Column N = 70 500 k (kips)

M = 720 k (ft. kips)

Beam M = 640 k (ft. Kips)

The column section was 17.5 inches by 17.5 inches, reinforced with 6 1-inch bars, of which 4 bars are considered effective. This gives one per cent of reinforcing steel. Assuming a seismic coefficient of k = 0.15, the direct strees N becomes 150 kips from which the actual stress in the concrete is found to be 1840 psi., a value near the ultimate strength (perhaps over that of the actual concrete constructed).

The beam sectio
forced with 3 5/8-in
percentage of 0.31.
steel area gives a r
the ultimate seismic
resisting, therefore
columns, the beams a
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The vibration a
of the tank structur
horizontally has bee

EIGHT ft	PERIOD[1] sec	PERIOD WHEN BASE NOT FIXED sec			
		$\theta=\frac{1}{1000}$[2]	$\frac{1}{200}$	$\frac{1}{100}$	$\frac{1}{20}$
66	0,41	0,48	0,64	0,83	1,7
95	0,74	0,79	0,96	1,1	2,1
120	0,75	0,81	1,0	1,2	2,3
130	0,85	0,91	1,1	1,3	2,4
170	0,89	0,96	1,2	1,5	2,8

D OF PRINCIPAL VIBRATION

Fig. 256

The beam section is 17.5 inches by 17.5 inches and is reinforced with 3 5/8-inch round bars. This gives a reinforcement percentage of 0.31. Calculating the resisting moment in the steel area gives a moment of about 40 foot kips. The value of the ultimate seismic coefficient that the beam is capable of resisting, therefore, is only 0.06g. In comparison with the columns, the beams are much weaker, so that the critical value of the seismic coefficient of the whole structure was only 0.06g.

The vibration analysis was considered. The displacement of the tank structure when the force of gravity is applied horizontally has been computed and from that the period of the

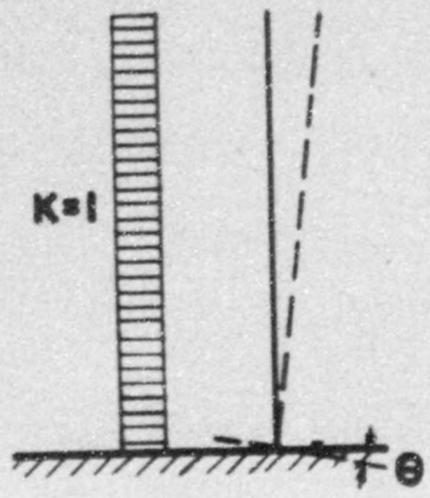

Fig. 257

tank has been determined by the gravity formula. In computing the stresses in the structure due to horizontal forces, the slope-deflection method was used. In this process it was found that the displacement of the tank could be expressed by the following equation:

$$\delta = \frac{6.7 \times 390\ k}{6\ EK}\ l\ \text{(ft.)}$$

wherein E = Young's modulus for concrete = 4.3×10^5 (kips/sq. ft.)

K = stiffnes of columns = 0.028 ft.^{3}

l = length of column = 13 ft.

k = 1.0g

Using the above values, the displacement was found to be 5.5 inches. By assuming the water as a solid mass, the period of vibration of the tank computed by the gravity formula becomes: $T = \frac{\sqrt{\delta}}{3} = \frac{\sqrt{5.5}}{3} = 0.78$ sec.
For a structure of the rigidity represented by this elevated tank, it does not seem necessary to consider the unfavorable condition of resonance.

Study of photographs of the failure of this structure shows that the joints of columns and beams, a usual weak point, did not fail, see Figs. 263 and 264. On-the-spot investigation of the failure seemed to indicate good workmanship in the

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ns = 0.028 ft.^{3}
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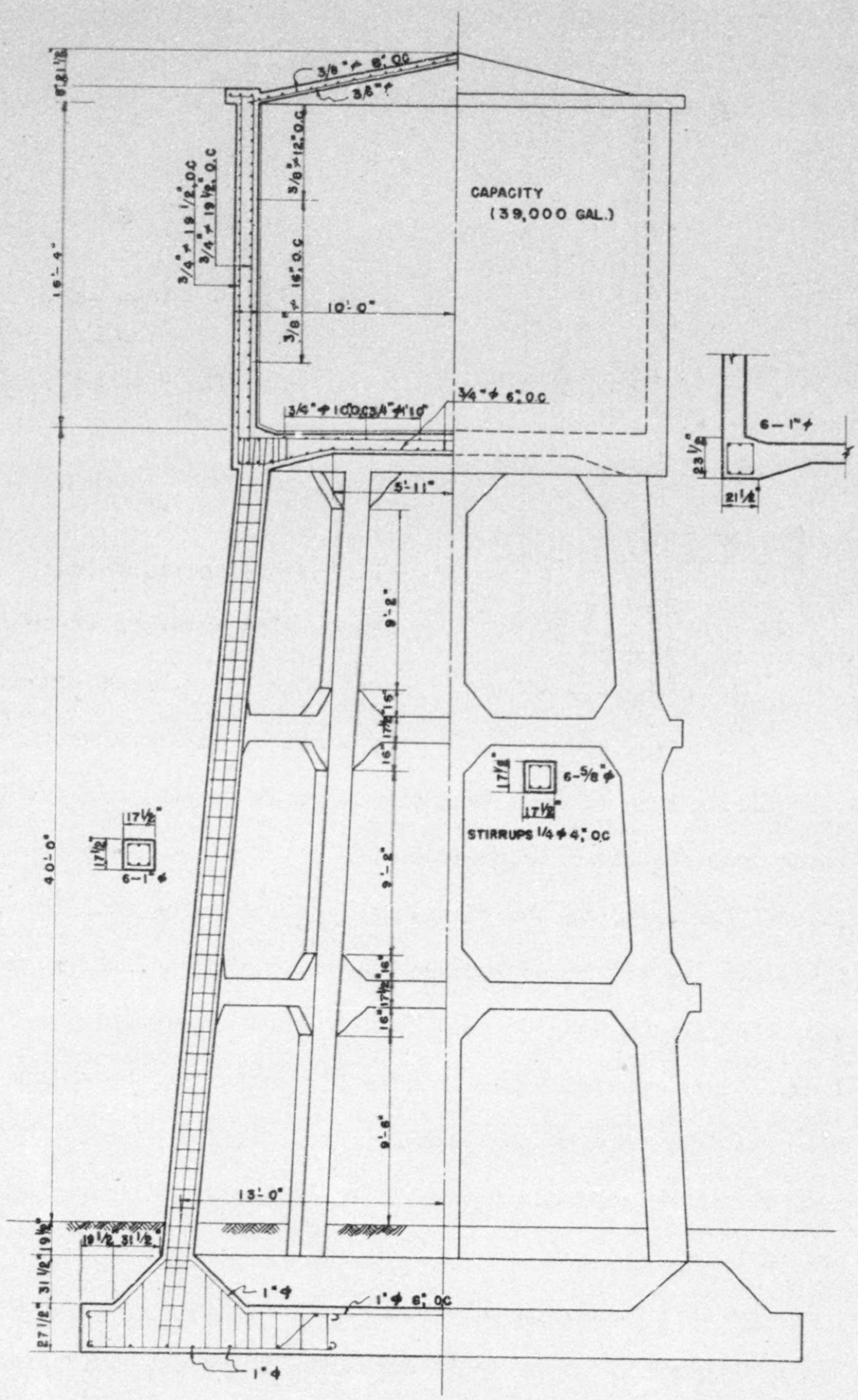


FUKUI RAILROAD YARD ELEVATED WATER TANK

Fig. 258

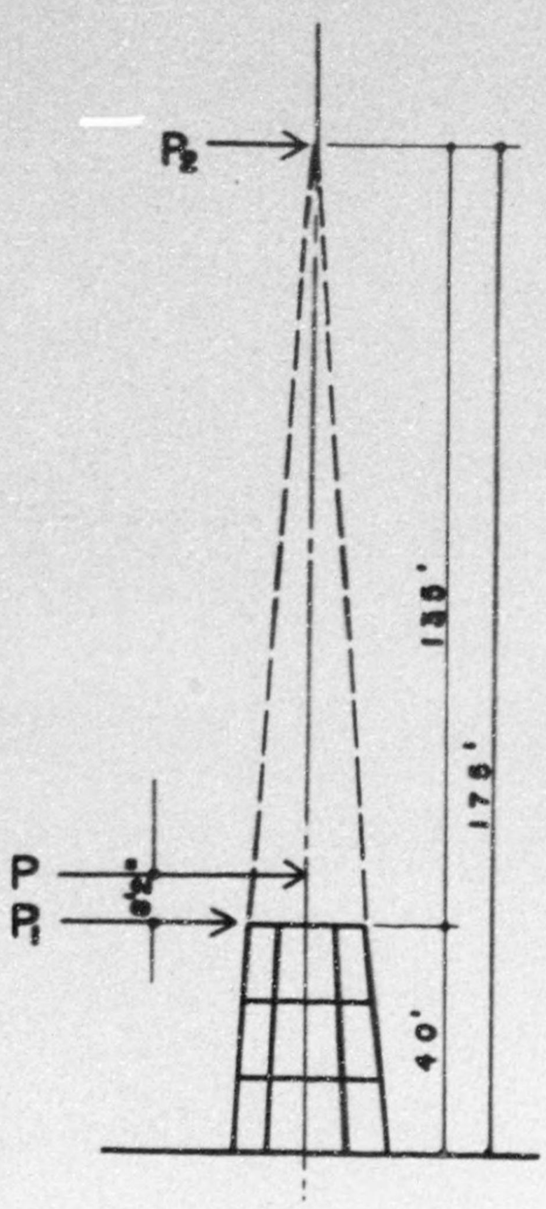


Fig. 259

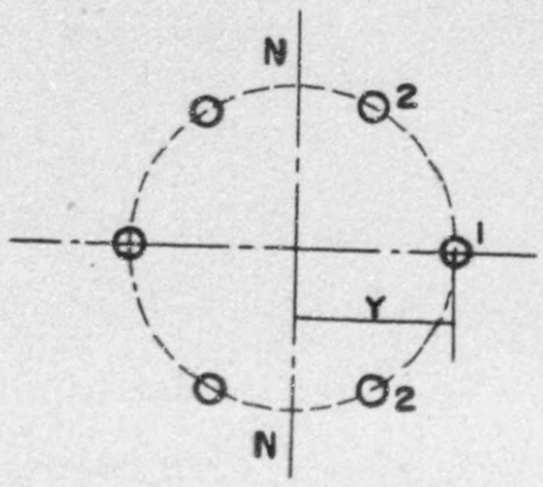


Fig. 260

placing of the reinforcing steel. The strength of the concrete has not been determined by tests and is unknown.

The failure of this tank structure is unusual and the foregoing analysis seems to indicate the lack of consideration for lateral forces to be the outstanding defect. The failure of the beams due to the earthquake shocks eliminated intermediate supports for the long columns, making them susceptible to buckling. This especially was true as the moment at the column base was increased by the beam failure. It is believed that the tank structure failed progressively, as described above, starting with beam failure.

There are numerous water tanks of a similar nature in Japan and a thorough check of their strength and resistance to earthquakes is desireable. Structures not meeting the strength requirements should be sufficiently reinforced.

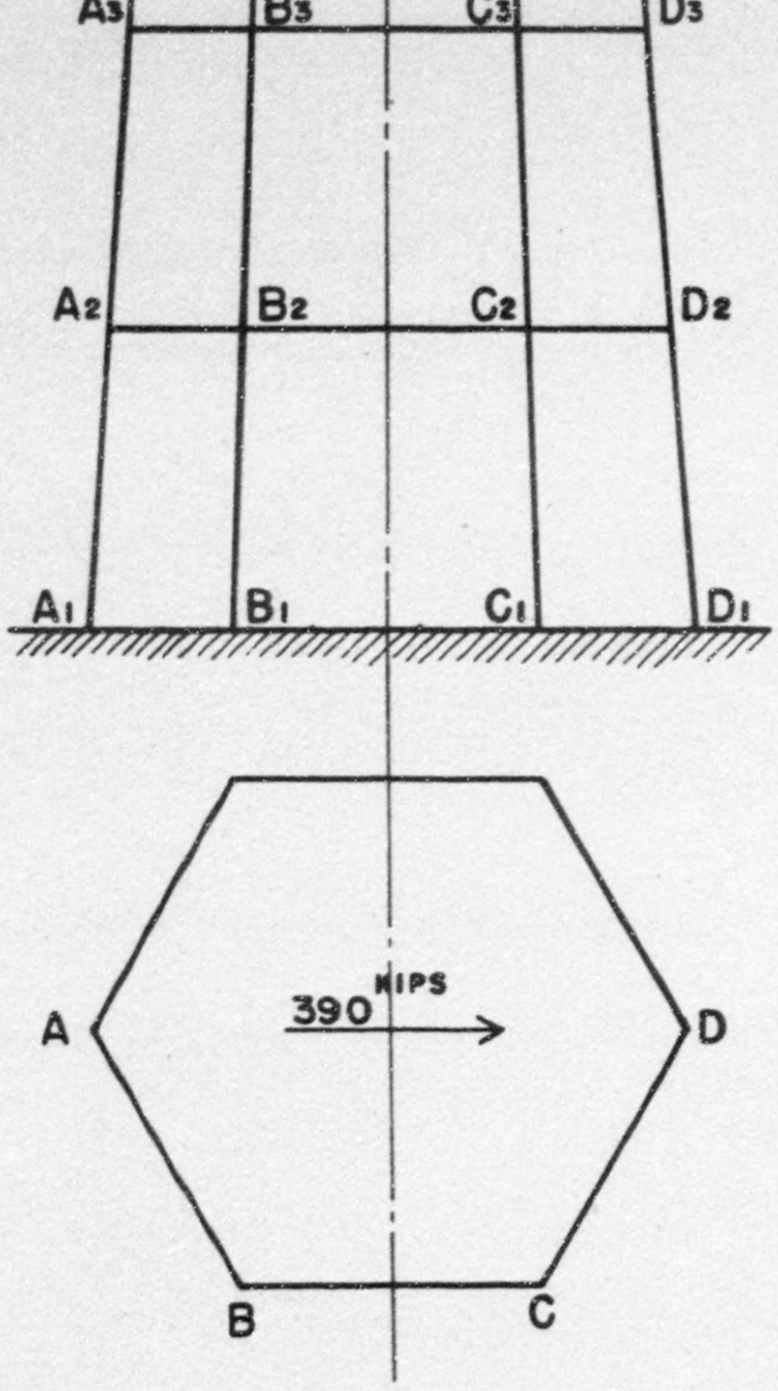
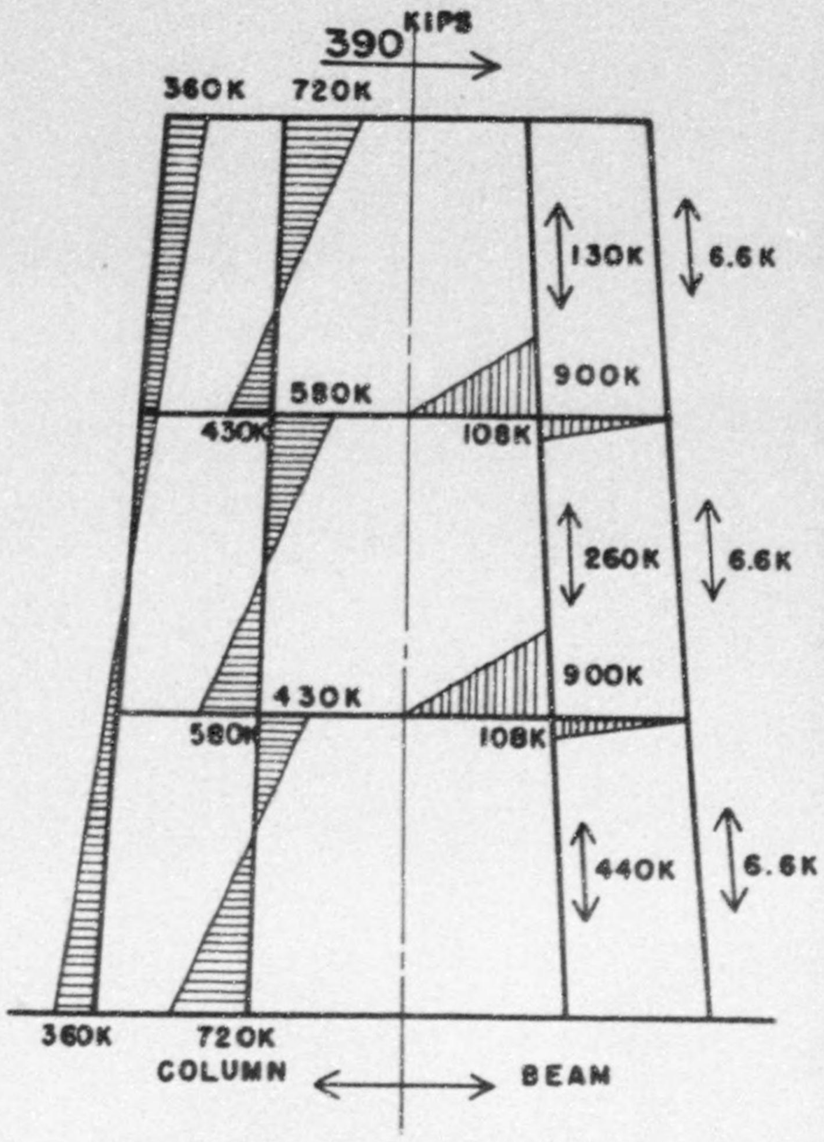

416 KIPS
390 KIPS
A4
B4
C4
D4
A3
B3
C3
D3
A2
B2
C2
D2
A1
B1
C1
D1
390 KIPS
A
B
C
D

Fig. 261



BENDING MOMENT IN FRAME

(WITH DIRECT STRESS IN COLUMN)

MOMENT UNIT FT. KIPS
DIRECT STRESS UNIT KIPS

Fig. 262

Fig. 263

Reinforced Concrete Elevated
Fukui City Railroad Ya

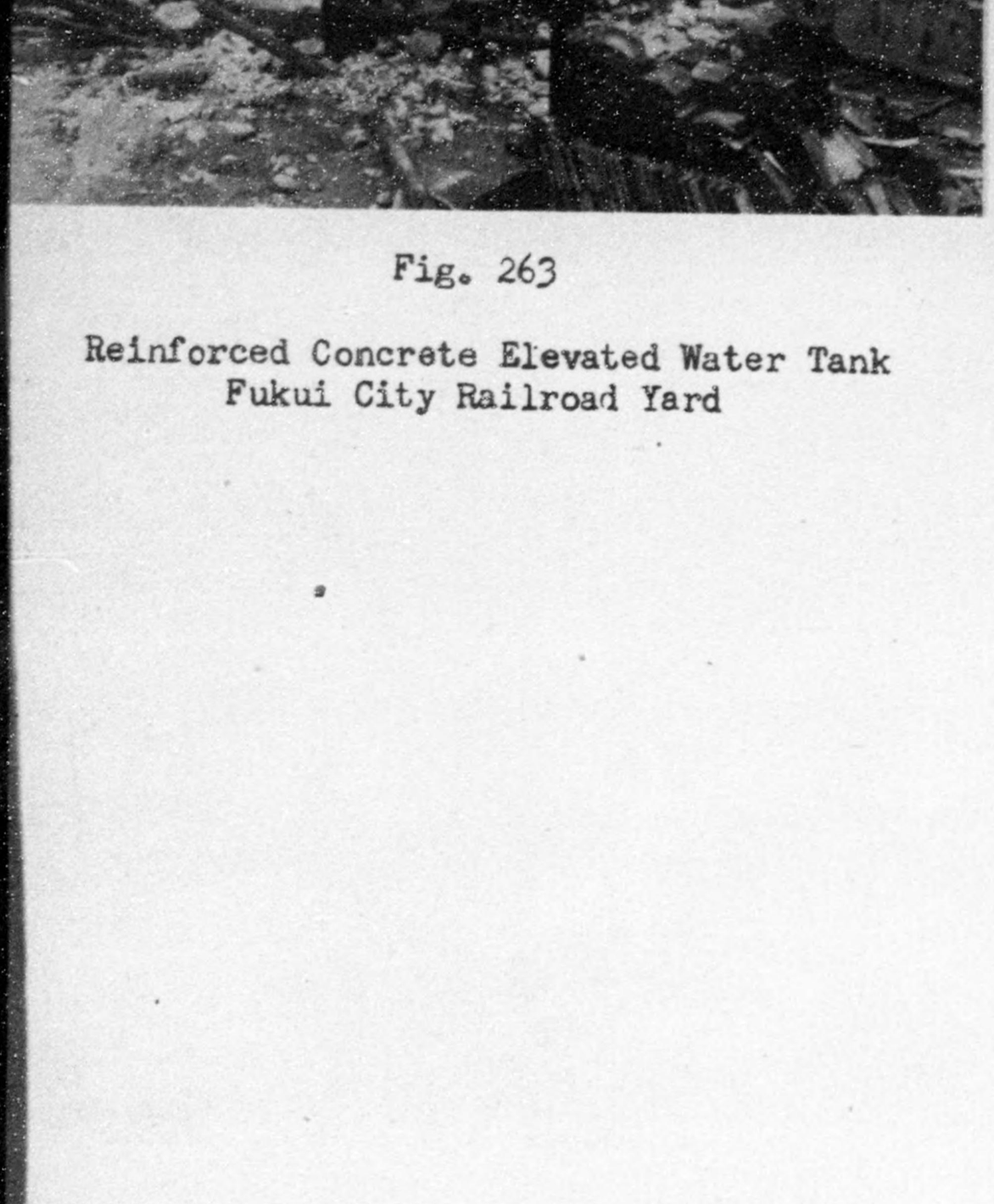


Fig. 263

Reinforced Concrete Elevated Water Tank
Fukui City Railroad Yard

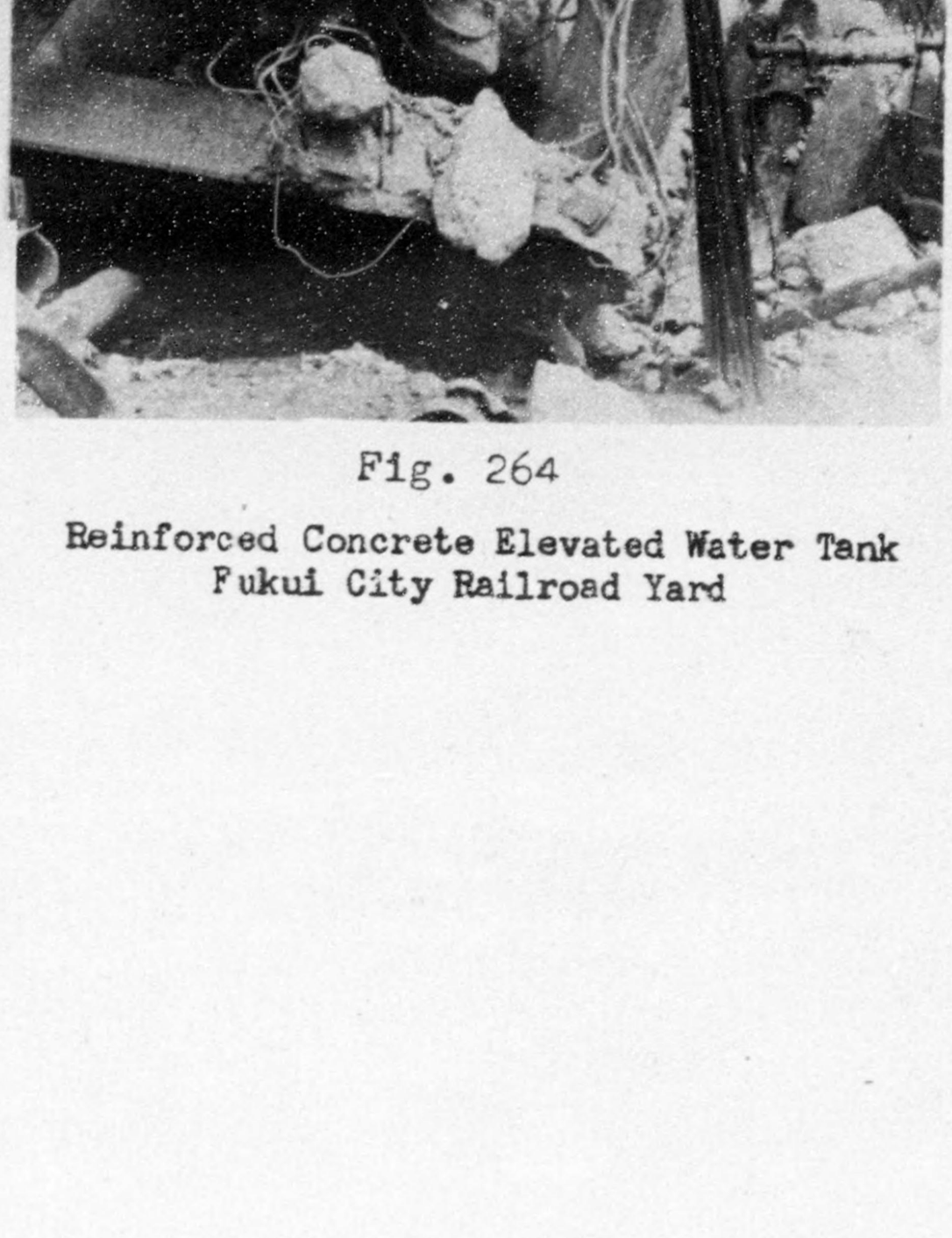


Fig. 264

Reinforced Concrete Elevated Water Tank
Fukui City Railroad Yard

The Heart of Fukui City Was Almost
Entirely Destroyed by Fire

CHAPTER VI
FIRE DAMAGE

Cities in Japan which have experienced violent earthquakes rarely have escaped resultant fires. This area was no exception. Shortly after the shocks subsided, numerous fires broke out in many of the cities and towns. It is fortunate that this earthquake occurred in the summer and before the time when fires would be started in the homes to prepare the evening meals, or the fire damage would certainly have been much heavier. A slight breeze of about 10 ft/sec., blowing from southeast to northwest, aided the fire in Fukui City initially but fortunately abated by seven o'clock that evening.

Fire caused by far the large majority of the casualties in the disaster area. The numbers of houses lost by fire in different areas were as follows: in Fukui City about 2000 houses, in the town of Maruoka about 1400, in the town of Kanazu about 200, and about 100 in each of the towns of Harue and Matsuoka. The burned over areas of most of these towns are shown in Chapter IV, Section A-2. A view of a portion of the downtown area of Fukui City which was ravaged by fire is shown by the chapter frontispiece. Smoke was still (14 hours after the earthquake) issuing from the rayon warehouse to the rear of the Daiwa Department Store.

Japanese investigators have placed, in this case, the primary sources of the conflagration as first, fires used for cooking in b

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The most effective means found to combat the fires that break out, seemingly everywhere, is to extinguish them at their sources. This of course is most obvious and actually was practiced in Fukui City with good effect. A group patrolled an area around the Fukui Technical School immediately after the earthquake and by extinguishing numerous small blazes stopped what could have developed into a conflagration in that area. It is realized that an earthquake leaves the inhabitants of any area with grave handicaps such as blocked streets, broken water mains, other trapped and injured inhabitants, natural fear, and a desire to seek open areas. Any effort, individual or group, toward extinguishing nearby small blazes should, however, be made.

One particular thing obvious to Military Government personnel was the difficulty of rescuing people trapped under fallen structures. Carl Mydans, internationally known reporter and photographer, made similar comments in his reports of the earthquake. Tools for wrecking wood structures, placed so that they are readily accessible would undoubtedly save many lives.

The fire damage suffered by Japanese cities and towns where practically all construction is wooden would be greatly alleviated by the installation of either natural of artificial firebreaks such as parks, wide thoroughfares, or rows of concrete office buildings or residences.

RECONSTRUCTION
OCTOBER 1948

CHAPTER VII
RECONSTRUCTION

A. GENERAL - The disaster area was visited for the second time by an inspection party from the Office of the Engineer, GHQ, FEC on 12 October 1948. The party was composed of Mr. Henry J. Brunnier, internationally known American structural engineer, who, as the officially designated representative of the American Advisory Committee on Engineering Seismology, came to Japan to inspect the aspects of the Hokuriku earthquake damage; Mr. Don W. Butler, Civil Engineer, Engineering and Operations Division; Mr. John J. Collins, Geologist, Intelligence Division; Dr. Tachu Naito, internationally known Japanese structural engineering authority; and Professor J. Kazuo Minami, structural engineering department, Waseda University, Tokyo, Japan.

This party spent several days in the Fukui area visiting and examining the evidences of the earthquake. A prominent feature noticed was the rapid rate of reconstruction of houses, transportation facilities, and public utilities. Conclusions drawn from information gained by this party in addition to that obtained during the first inspection of the area immediately after the earthquake have been incorporated in the main body of the report. Some of the reconstruction efforts observed will be described briefly for the purpose of acquainting the reader with progress made under extreme difficulties.

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B. HOUSING - The panorama shown in Fig. 265 was taken from the roof of the south portion of the Daiwa Department Store. By comparison with the frontispiece, Chapter VI, a photograph of the same area made immediately after the earthquake, it is evident that the claim of almost 90 per cent rehabilitation of houses in Fukui City is probably correct. The large building to the right in Fig. 265 is the City Hall, the one in the center is the former Military Government office, and the one to the left is the Fukui Bank. It was desirable that more general consideration than previously would be given to bracing against lateral forces in reconstruction, but inspection of a few of these new units does not disclose any particular efforts having been made in this direction.

C. BRIDGES

1. KEIFUKU ELECTRIC LINE RAILROAD BRIDGE - This bridge over the Kuzuryu River had been partially repaired and repairs were in progress at the time of the second visit. Fig. 266 shows the reconstructed pier which is the eighth from the southside (fourth from left). This pier failure is shown in Chapter III, Section C, Fig. 71. Fig. 267 shows the effort of reconstruction on the second pier from the southside. This pier failure is shown in Fig. 73. The reconstruction method used was to enclose the pier in its position of failure inside a new shell of reinforced concrete.

Fig. 265

Panorama - Reconstruction of Downtown Area
Fukui City

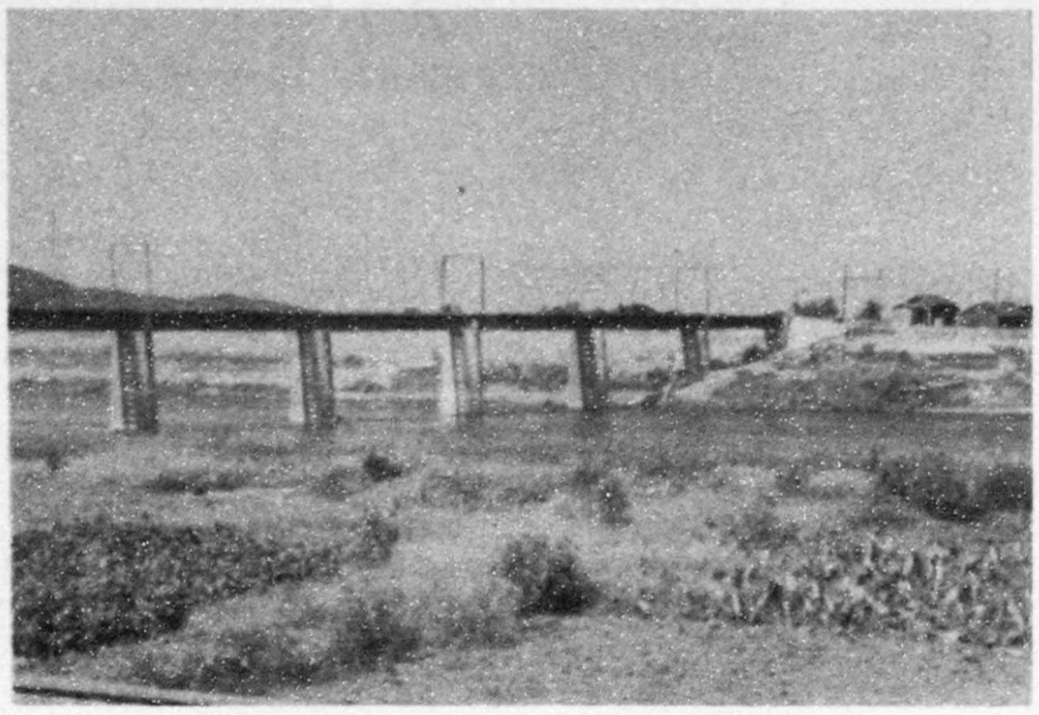

Fig. 266

Reconstructed 8th Pier
Keifuku Electric Line Bridge

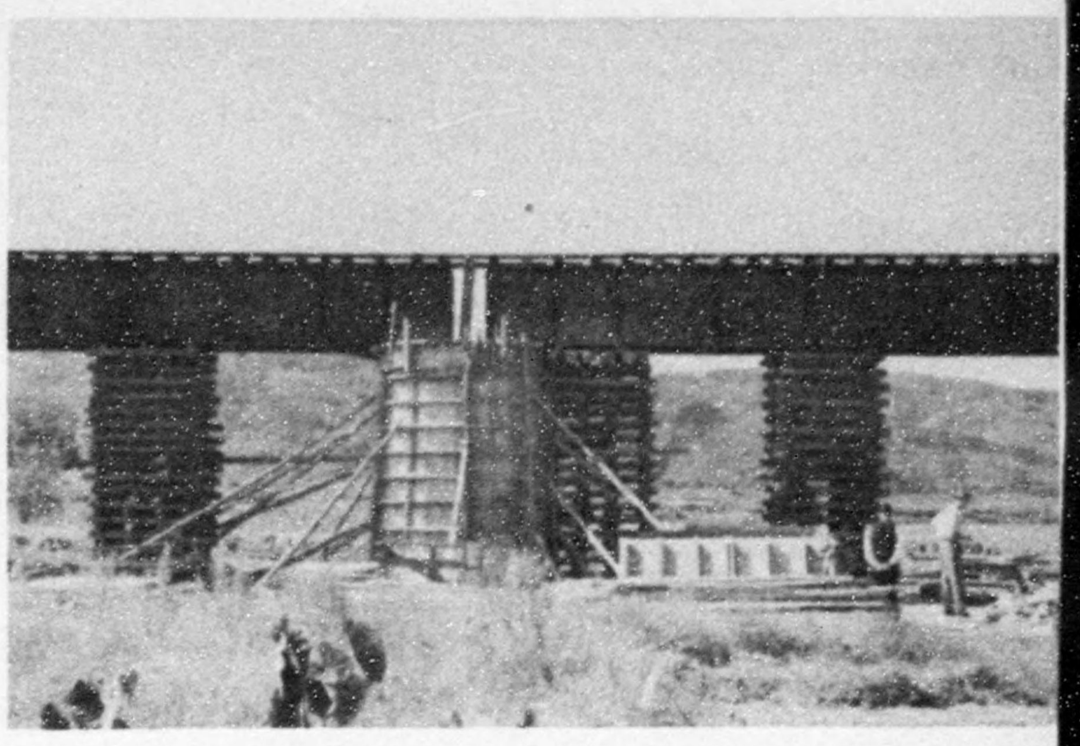

Fig. 267

Reconstruction of 2nd Pier
Keifuku Electric Line Bridge

2. GOVERNMENT RAIIROAD BRIDGE - This bridge over the Kuzuryu River has been undergoing reconstruction. A temporary wood pile bridge has been put up to handle traffic over the river on this main line. The temporary bridge, shown in Fig. 268, has been offset to the west of the former bridge centerline. It is intended to erect combination plate girder end spans and three center trussed spans in the reconstruction of the bridge. Two concrete pier construction.

3. NAKATSUN making use of the inclined piers. reconstruction ef as of 13 October show the sixth pi

Fig. 265

construction of Downtown Area
Fukui City

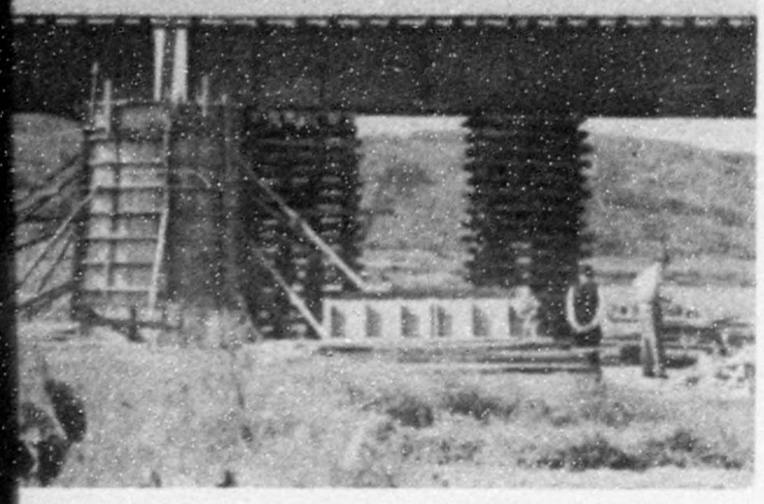

Fig. 267

construction of 2nd Pier
Fuku Electric Line Bridge

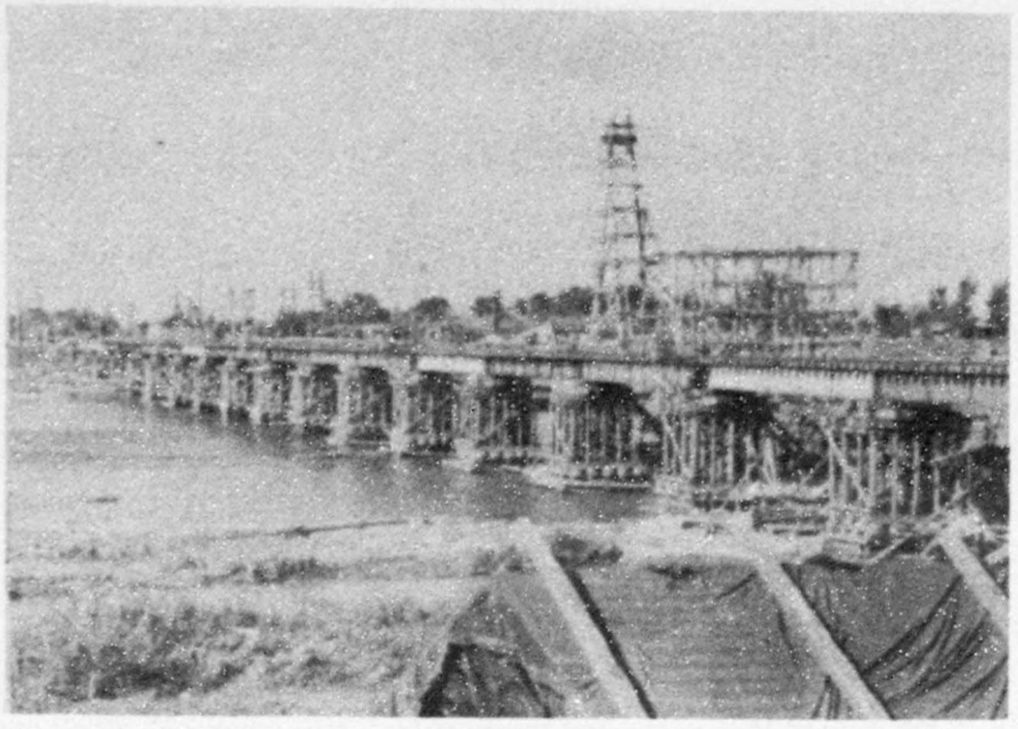

Fig. 268

Temporary Wood - Pile Bridge
Kuzuryu River - Government Railroad

Two concrete pier shells are shown, Fig. 269, in the process of construction.

3. NAKATSUNO HIGHWAY BRIDGE - This bridge is being rebuilt making use of the original plate girder spans and the erect and inclined piers. Fig. 270 and Fig. 271 are views taken before reconstruction efforts commenced and of the stage of progress as of 13 October 1948, respectively. Fig. 272 and Fig. 273 show the sixth pier from the south side in its inclined position

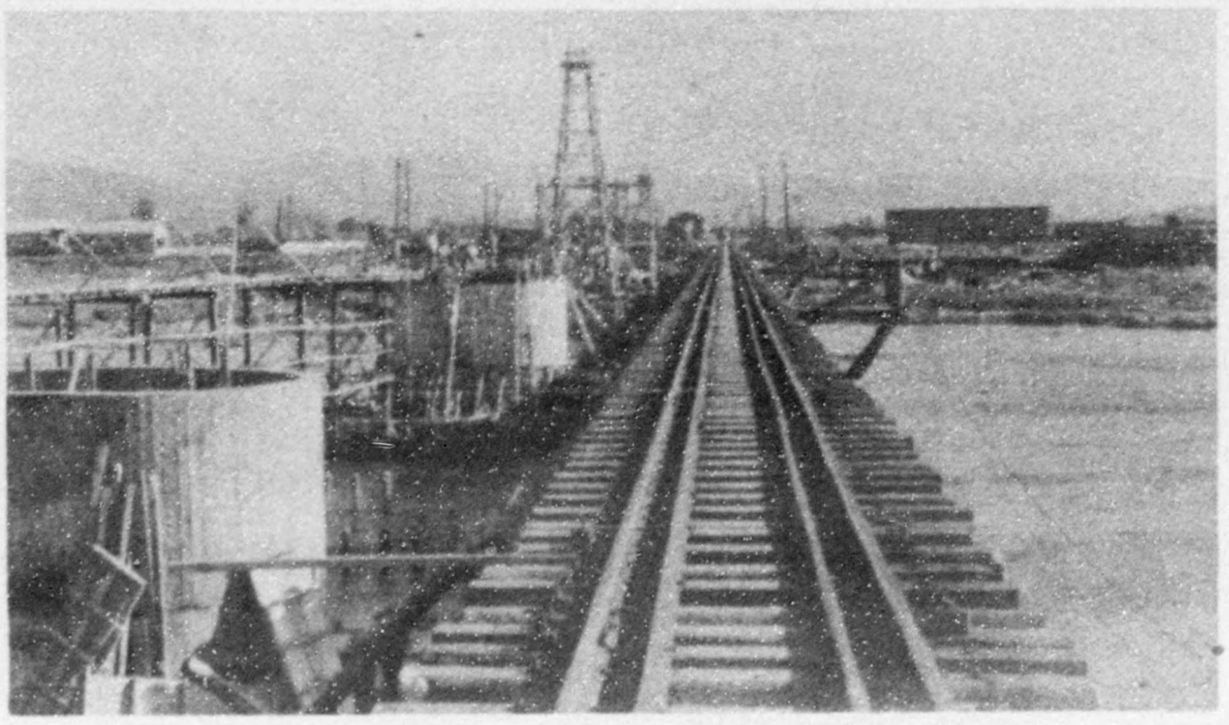

Fig. 269

Concrete Pier Shells - Reconstruction
Kuzuryu River Bridge - Government Railroad

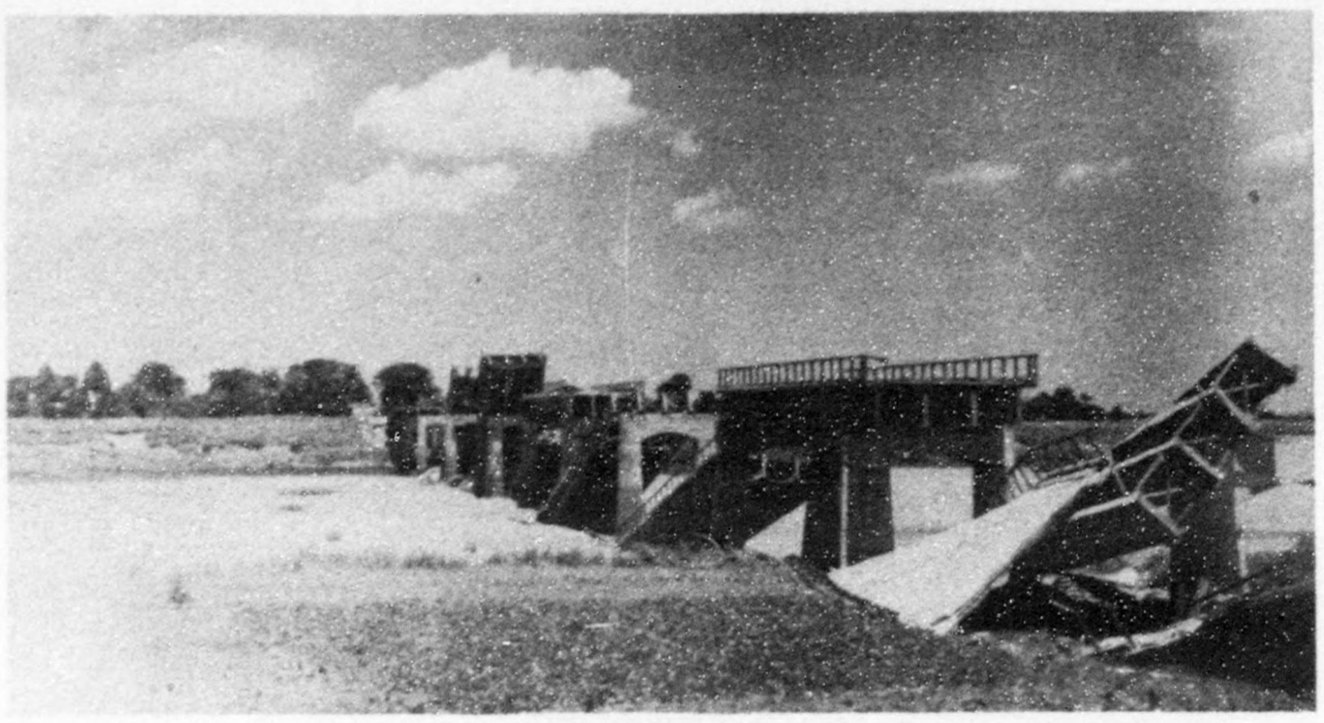

Fig. 270

View of Collapsed Nakatsuno Bridge
Kuzuryu River

towards the south, immediately after the earthquake, and its position in the bridge reconstruction. It will be noted that this pier has been straightened somewhat but is still inclined about 5 degrees off vertical. The pier is roughened and dash coated as the lift progresses. The old pier will thus form an inclined core in a new solid reinforced concrete pier.

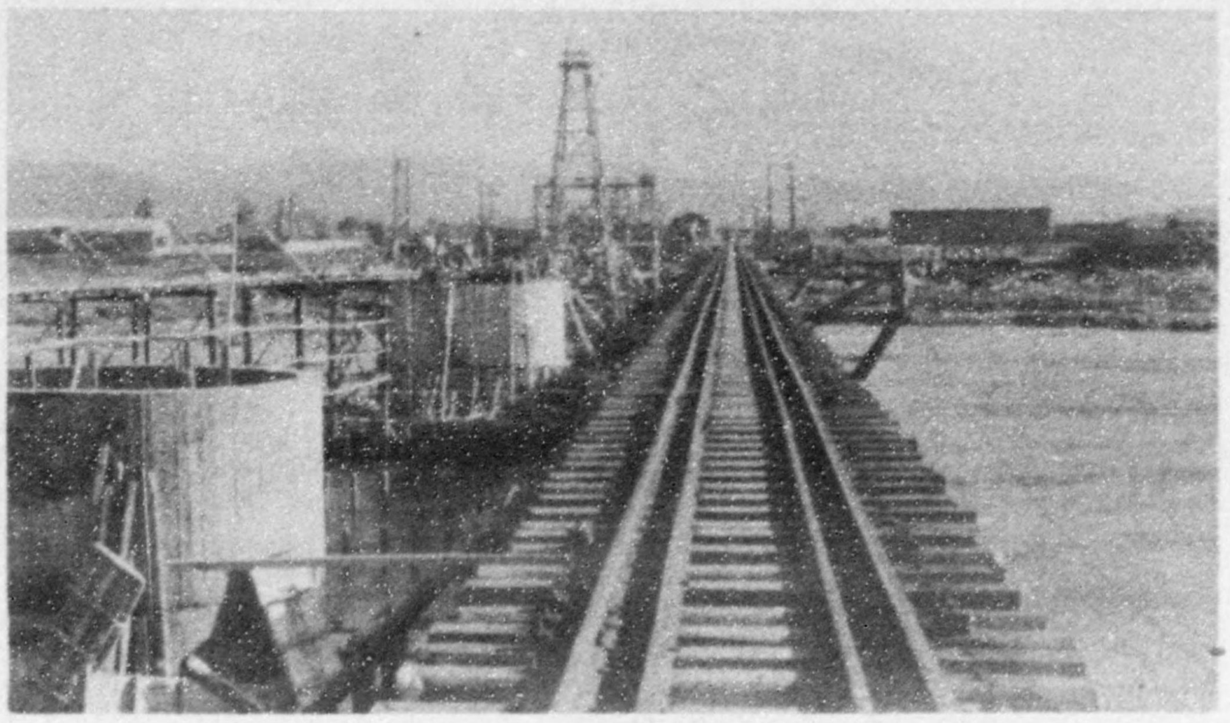

Fig. 269

Concrete Pier Shells - Reconstruction
Kuzuryu River Bridge - Government Railroad

Fig. 270

View of Collapsed Nakatsuno Bridge
Kuzuryu River

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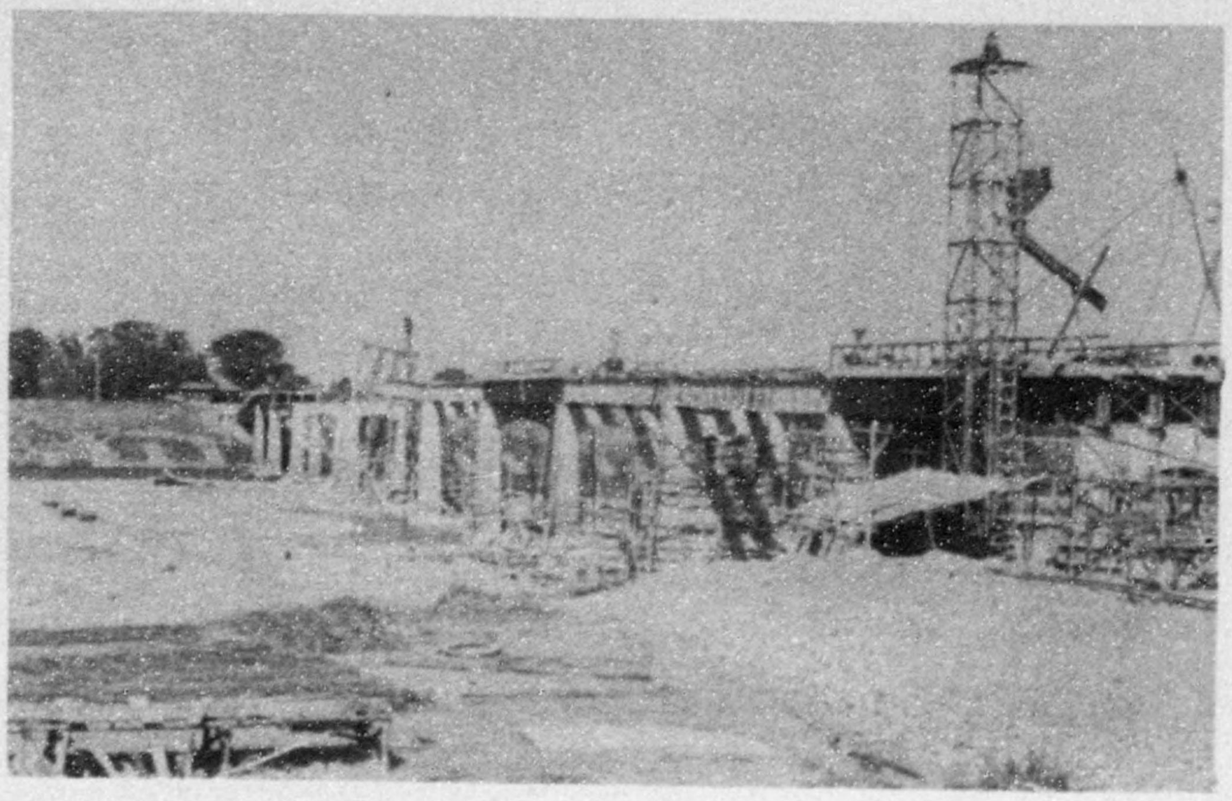

Fig. 271

Reconstruction - Nakatsuno Bridge

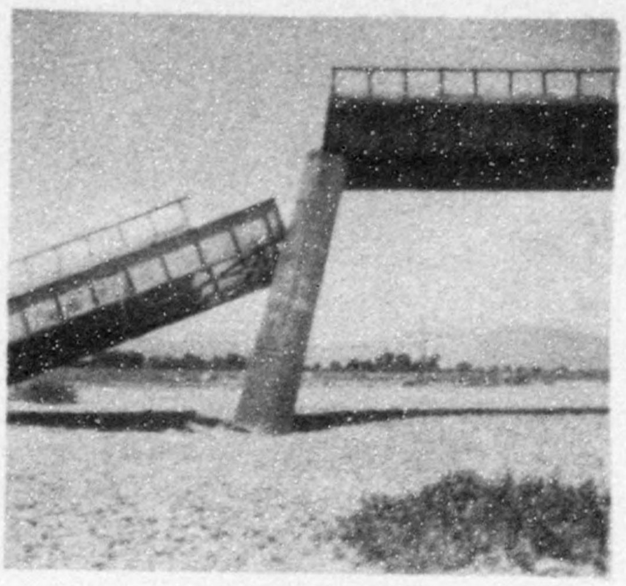

Fig. 272

View of Inclined 6th Pier
Nakatsuno Bridge

Provisions are being made for a pier shoe to be securely anchored to the pier top. This will then provide an anchorage for the plate girder units.

4. ASAHI BRIDGE - This small reinforced concrete road bridge has been described in Chapter III, Section C. The bridge deck structure was cut in two down the longitudinal centerline and each half jacked back into place. Fig. 274 and Fig 275 show the results of the repair efforts.

D. BUILDINGS

1. DAIWA DEPARTMENT STORE - This much discussed building was being demolished at the time of the second visit. The fifth and sixth floors of the sales portion and the front stair-

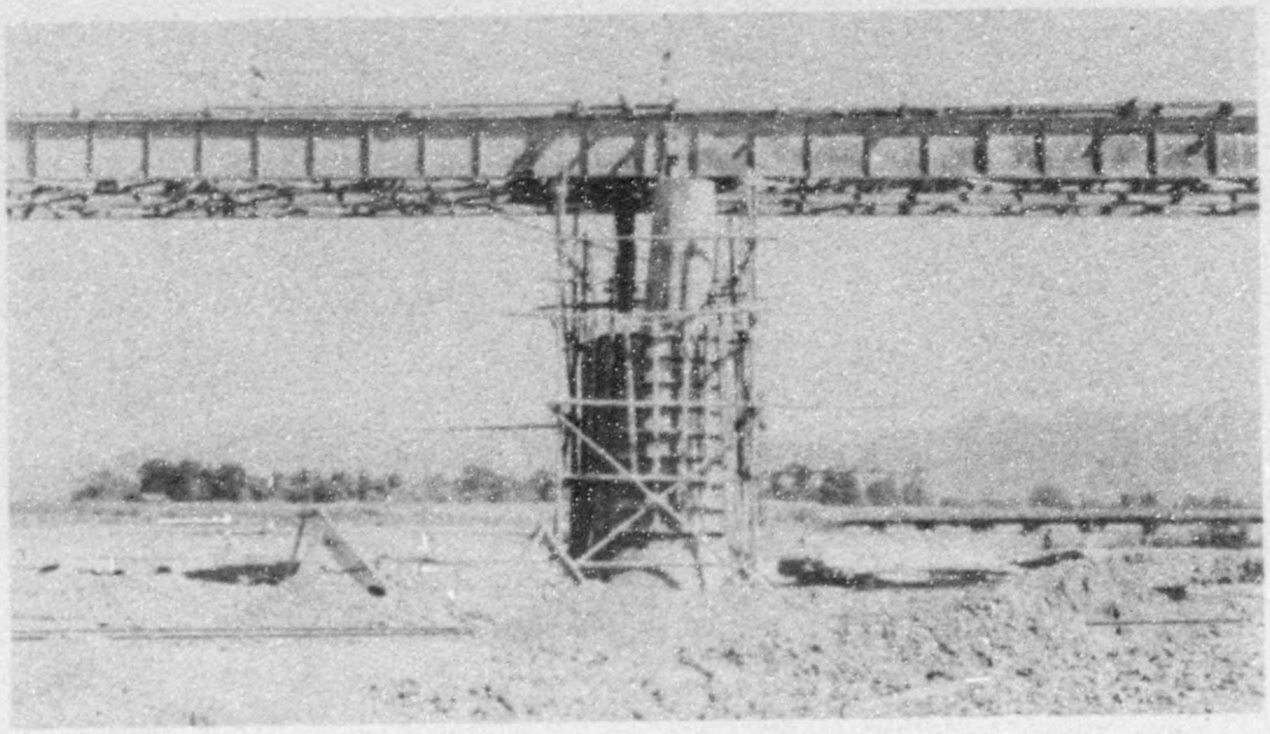

Fig. 273

Reconstruction of 6th Pier
Nakatsuno Bridge

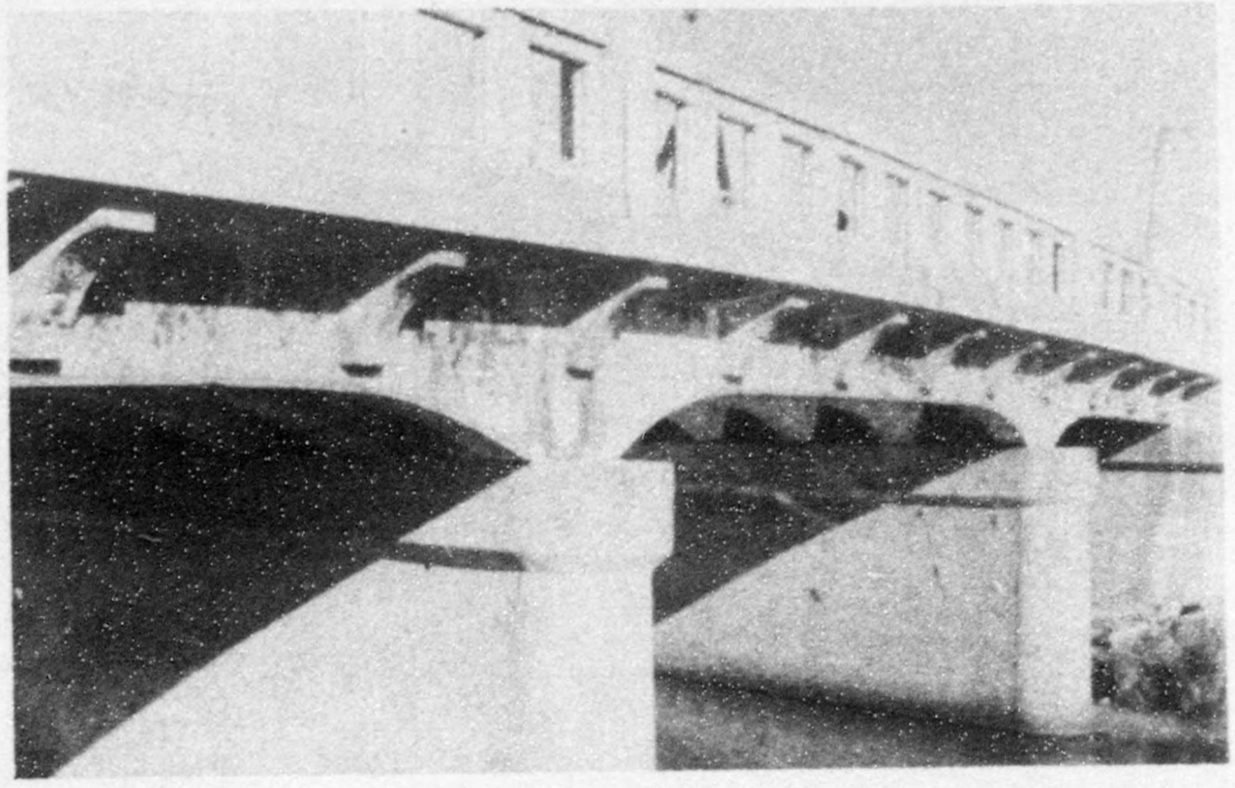

Fig. 274

Asahi Bridge After Repair
Fukui City

well tower of the building had been taken down, see Fig. 276.

The demolition is proceeding by means of sledgehammers and cutting torches. All reinforcing steel is being recovered as scrap. The intention regarding portion III of the building has not been made public by the owners. It was hoped that a

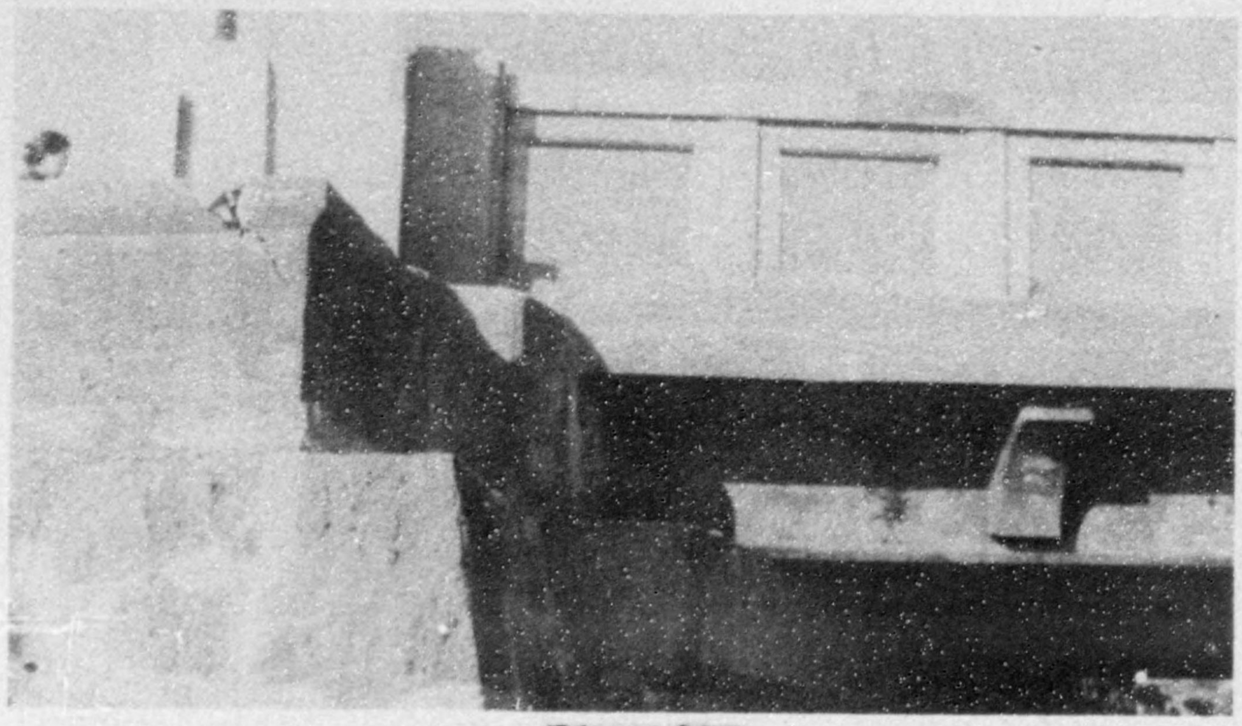

Fig. 275

South View of West Abutment
Repaired Asahi Bridge - Fukui City

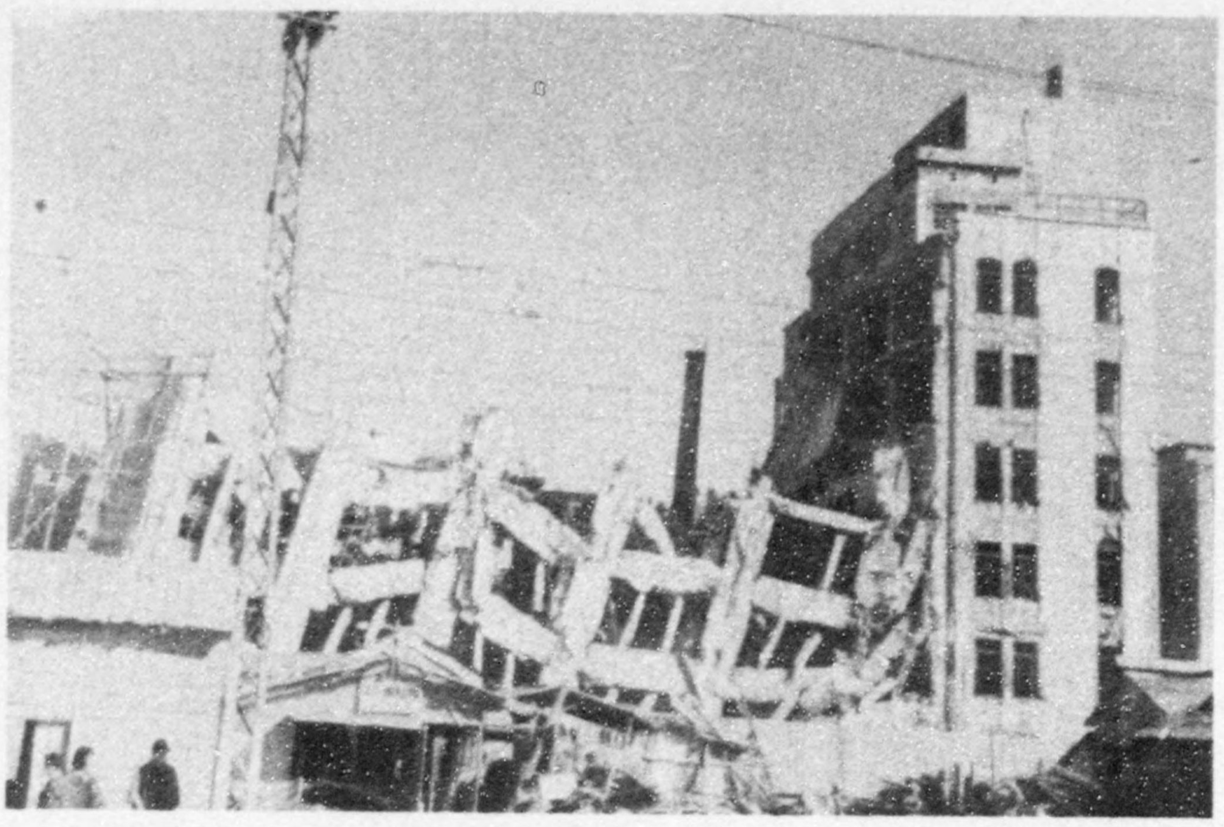

Fig. 276

Daiwa Department Store - Demolition
Fukui City

physical investigation of the foundations could be made prior to publication of this report, but the rate of progress of the demolition does not indicate that such an investigation will be possible.

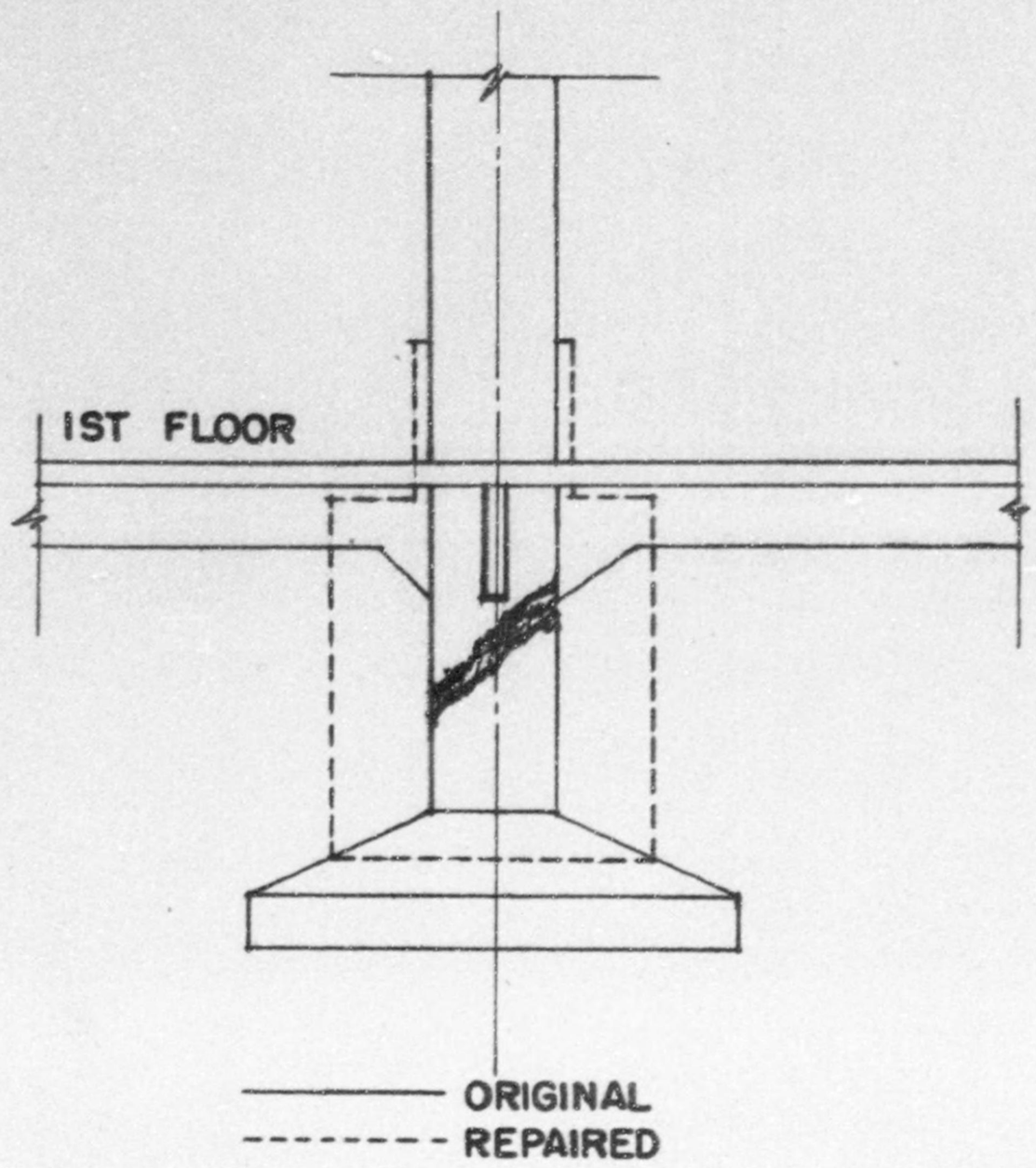

Fig. 277

Sketch of Repair Method
Foundation Columns
Hokuriku Light and Power Co. Building
Fukui City

2. HOKURIKU LIGHT AND POWER BUILDING - The foundation column failure of this building has been repaired by jacking up the sagged sections and enclosing the fractured columns in new columns. The first floor slabs around the original columns were chipped out to allow the new columns to be poured. The sketch, Fig. 277, shows the position of the new column encase-

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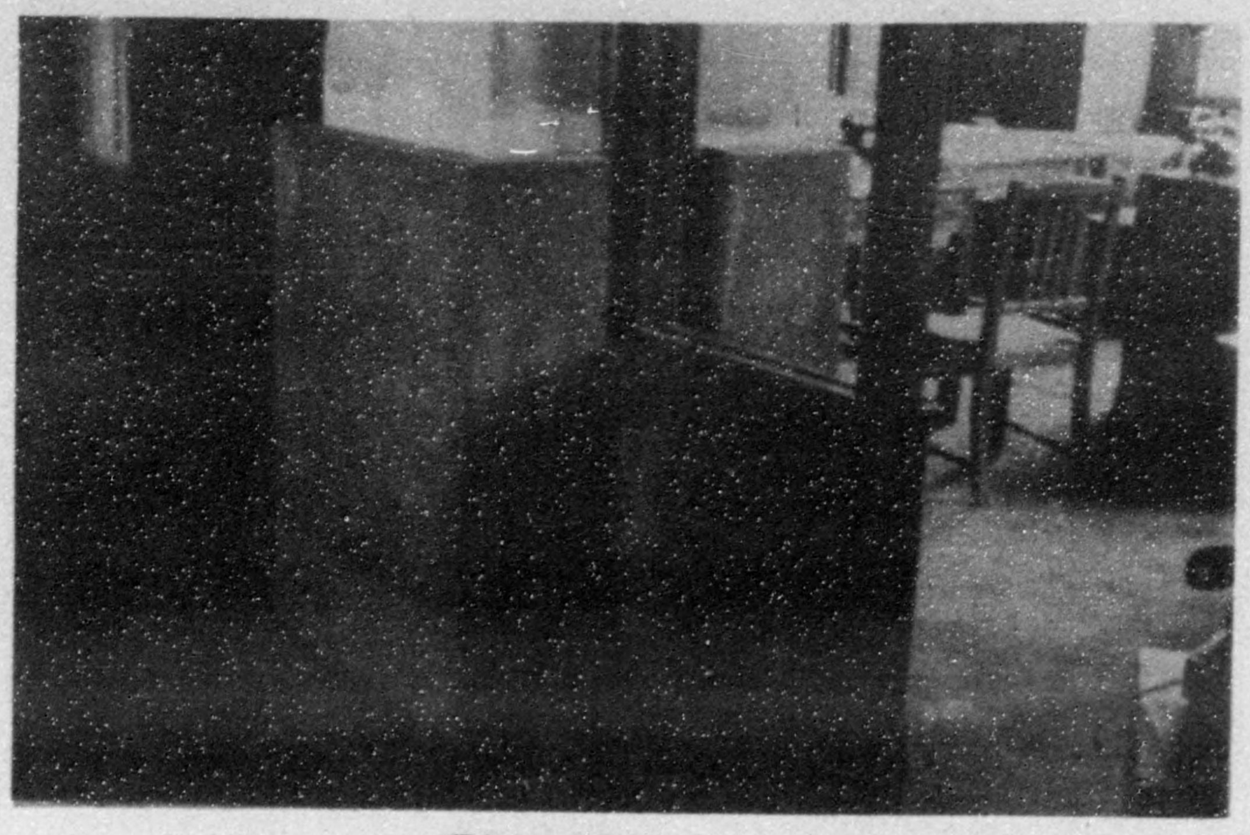

Fig. 278

Repaired Columns - Above First Floor Line
Hokuriku Light and Power Co. Building
Fukui City

ment. The new column was carried 2.5 feet above the first floor line as shown in Fig. 278.

E. COMMENTARY - It should be remembered that reconstruction efforts in the Fukui area are being conducted with the very minimum of materials and equipment. In the majority of cases the difficulty of obtaining necessary materials will cause considerable delay. Although large quantities of building materials were received by the Fukui area through a SCAP authorized release of occupation force supplies and a release by Japanese Government at the expense of the overall Japanese rehabilitation program, much more materials are necessary to permit reconstruction of the devastated area to equal the condition existing before the earthquake.

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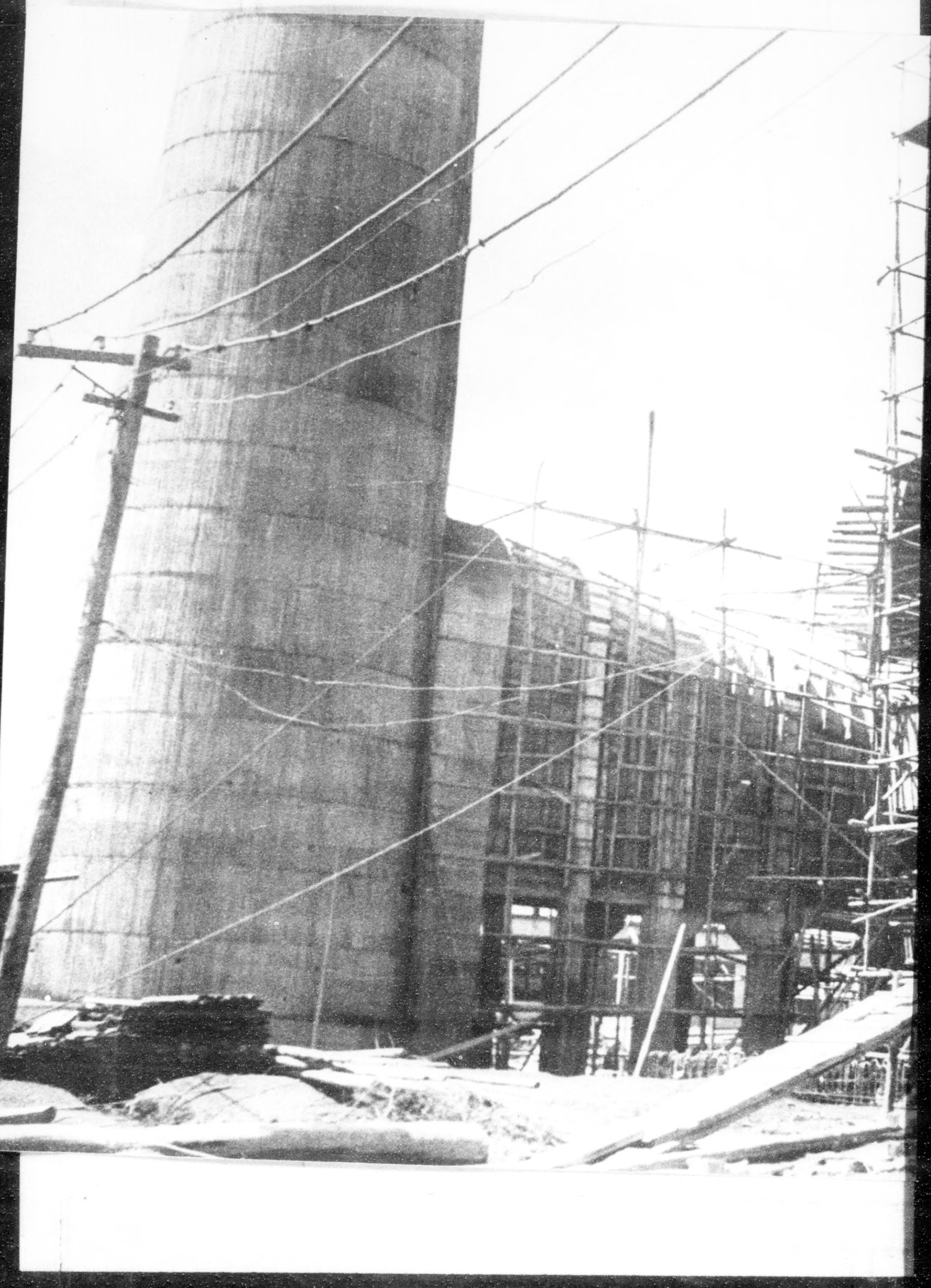

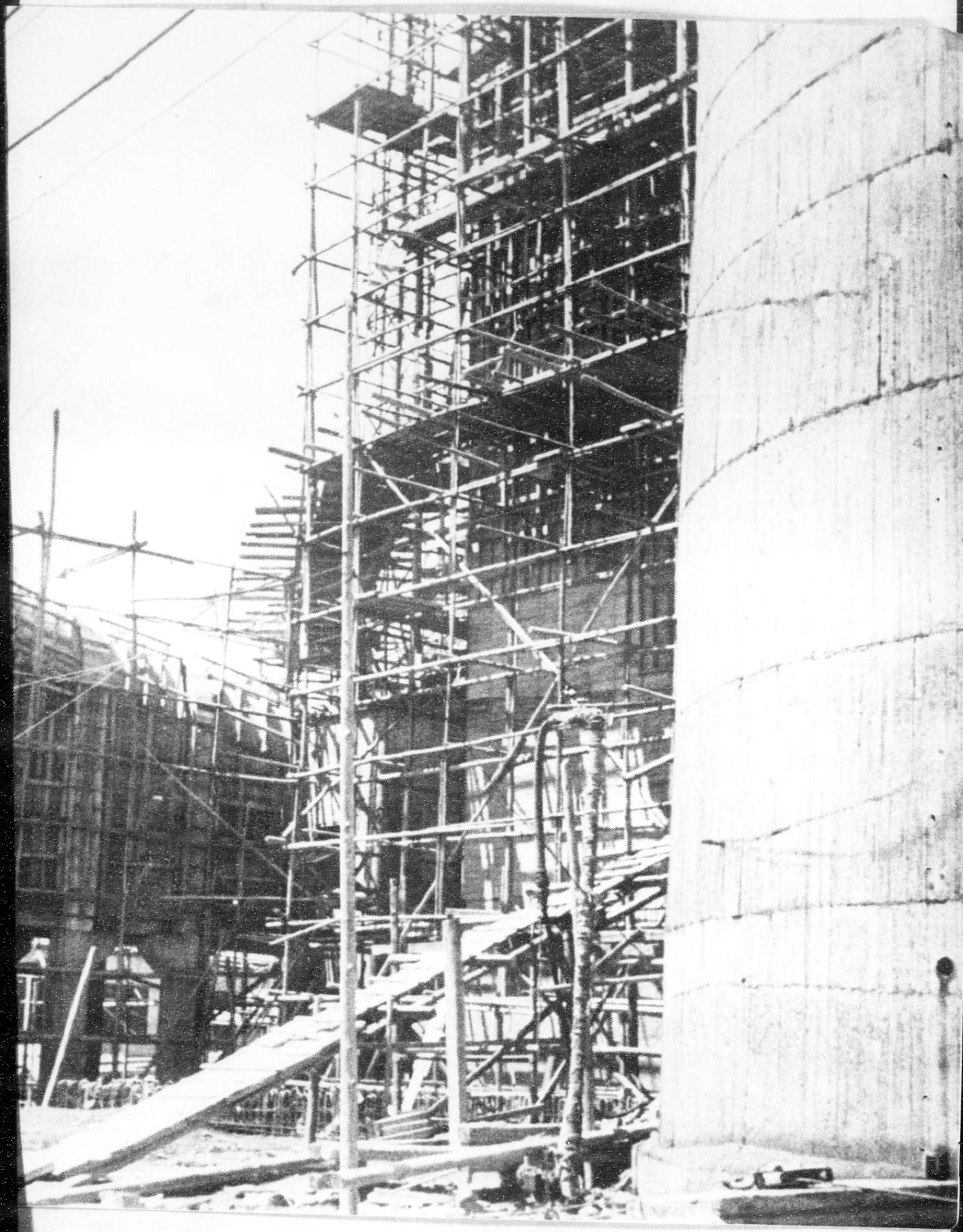

CHAPTER VIII
CONCLUSIONS AND RECOMMENDATIONS

A. GENERAL - Conclusions and recommendations pertinent to each particular chapter have been incorporated in that chapter. A few general conclusions and recommendations will be presented in this section. The methods of design presently known are sufficient, and building materials and construction techniques are generally satisfactory. The main problem in building for earthquake resistance seems to be the dissemination of information already at hand.

1. BUILDINGS

a. Site and Ground - Level, hard ground is the most desirable site for building purposes. Bedrock is the optimum, then compact gravel or sand, hard clay, clay with gravel or sand intermixed, with soft soil or reclaimed land being the least desirable site. Sloping ground and tops of cliffs and ridges are undesirable. The bottom of a slope is also dangerous because of landslides and increased earthquake forces due to reflected and refracted seismic waves. Protection against landslides should be provided using strong retaining walls. Soils of uneven hardness should be avoided. If location on this type of soil is necessary, piles should be used and constant foundation unit loads should be carefully maintained or compensated for by foundation construction. Area history should be investigated and old watercourses and fault zones should be

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b. Foundations - All foundations should be made as rigid and strong as is economically feasible in each case. For house construction continuous outside wall foundations should have steel reinforcement. Wherever possible, buildings should have a basement story with a mat foundation. Individual footings, if used, should be rigidly connected by tie beams; however, preference should be given to continuous footings.

Foundations should be carried down to a hard firm stratum either by excavation or by use of some type of piling. The use of friction type piling requires consideration for reduction of friction coefficients during periods of ground vibrations.

c. Plan and Elevation - The plan and elevation of the building should be as simple as possible. The structural engineer should have the cooperation of the architect in the balancing of the plan for rigidity. Large irregularities in the elevations of the various portions of the building should be avoided. Bracing walls extending from the top to the bottom of the building are recommended. Corners of plans whose shapes are L-U-H should be reinforced.

d. Wooden Construction - Lightness is important especially in the roof and upper portion of the structure. Japanese practice which has morticed and tenoned joints for all trusses and columns has disadvantages in that much cross sectional area is cut away at panel points and column joints and in that

mortiсed joints have little strength in tension. Accordingly it is recommended that structural joints be bolted, wherever possible. Tension members should be bolted as friction joints. In two story framing columns should be full two story height at corners and other critical points. Diagonal bracing must be provided in walls and partitions, and in floors and at plate lines.

e. Reinforced Concrete Construction - Columns should be fixed to foundations and all girder and column connections should be made rigid. In the case of encased steel girders, reinforcing bars should be placed to provide additional reinforcement. Wall girders should be made as deep and strong as economically possible. The maximum depth between windows should be designed to act as a beam. All of the vertical portions of the exterior wall (deducting window openings) should be designed to act as columns.

Bearing walls are capable of resisting large horizontal forces. These walls should be of reinforced concrete and of substantial thickness (6-8-10 inches). The reinforcing should be placed both ways and, in addition, diagonal bars should be provided.

Slabs should be made as rigid as possible. To do so requires the placement of diagonal bars.

Dowels should be provided in concrete wall joints to secure good bond. The area of dowels to be provided usually is

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ould be stiffened by diagonal braces or by filling them with rein-

ns forced concrete walls.

Ornamental stone and tile should be firmly fixed to rein-

- forced concrete walls or the steel frame.

s B. APPENDIX - Work has been started on the compilation of a

ould volume dealing with earthquake resistant construction for build-

of ings. This volume will deal primarily with Japanese methods.

ned Contained therein will be sections devoted to the historical development of earthquake resistant construction; engineering aspects of earthquakes together with theoretical seismology; practical considerations in earthquake resistant design and construction; theory of design; methods of design application using approximate and exact methods; examples of outstanding method applications to actual buildings; design data; comparison of building codes; and briefs of research studies on earthquake resistant construction. Leading Japanese authorities in this field will be contributors to this volume. It is expected that this volume will be published in the late summer of 1949.

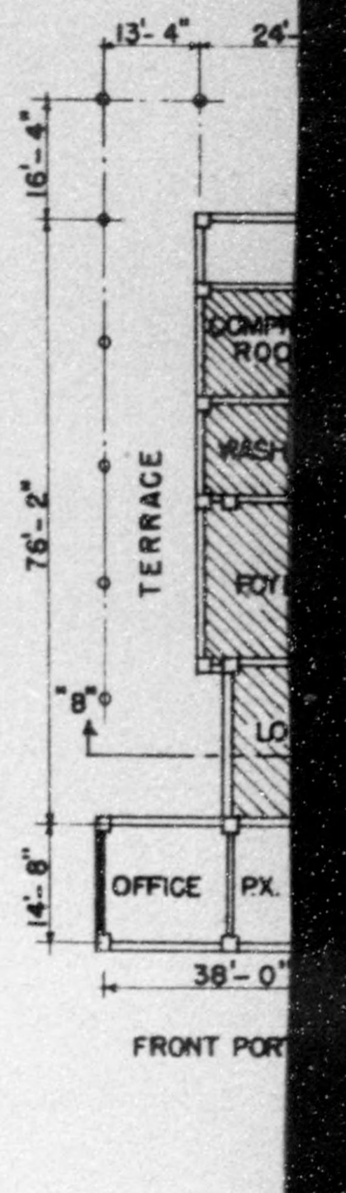
C. STRUCTURAL PLANNING

1. GENERAL - Presented herein is the structural planning for two diverse types of structures done by the Design Branch. Also presented are suggestions for structural planning of the Daiwa Department Store using the original floor plan.

2. 1004 SEAT ARMY THEATER

a. General - The design of an army theater, for construction in active seismic areas, was done early in 1948 utilizing a standard Department of the Army floor plan. The plan of this building is shown by Fig. 279. The main portion of the building consists of a funnel shaped auditorium, having a seating capacity of 1004 persons, and a stage. The front portion is of two story construction; the first floor consists of terrace, toilets, lobby, offices, and equipment rooms while the second floor houses projectors and air conditioning equipment. The rear portion, of single story construction, houses the various dressing rooms.

The reinforced concrete auditorium floor rests on the ground. An underground, reinforced concrete air duct, utilized as a return air duct and pipe trench, runs along the longitudinal axis of the auditorium. The auditorium roof trusses are all-steel construction and support I-beam purlins to which are secured, by a special clip, feather-weight precast concrete channel roof slabs. The auditorium, proscenium, and



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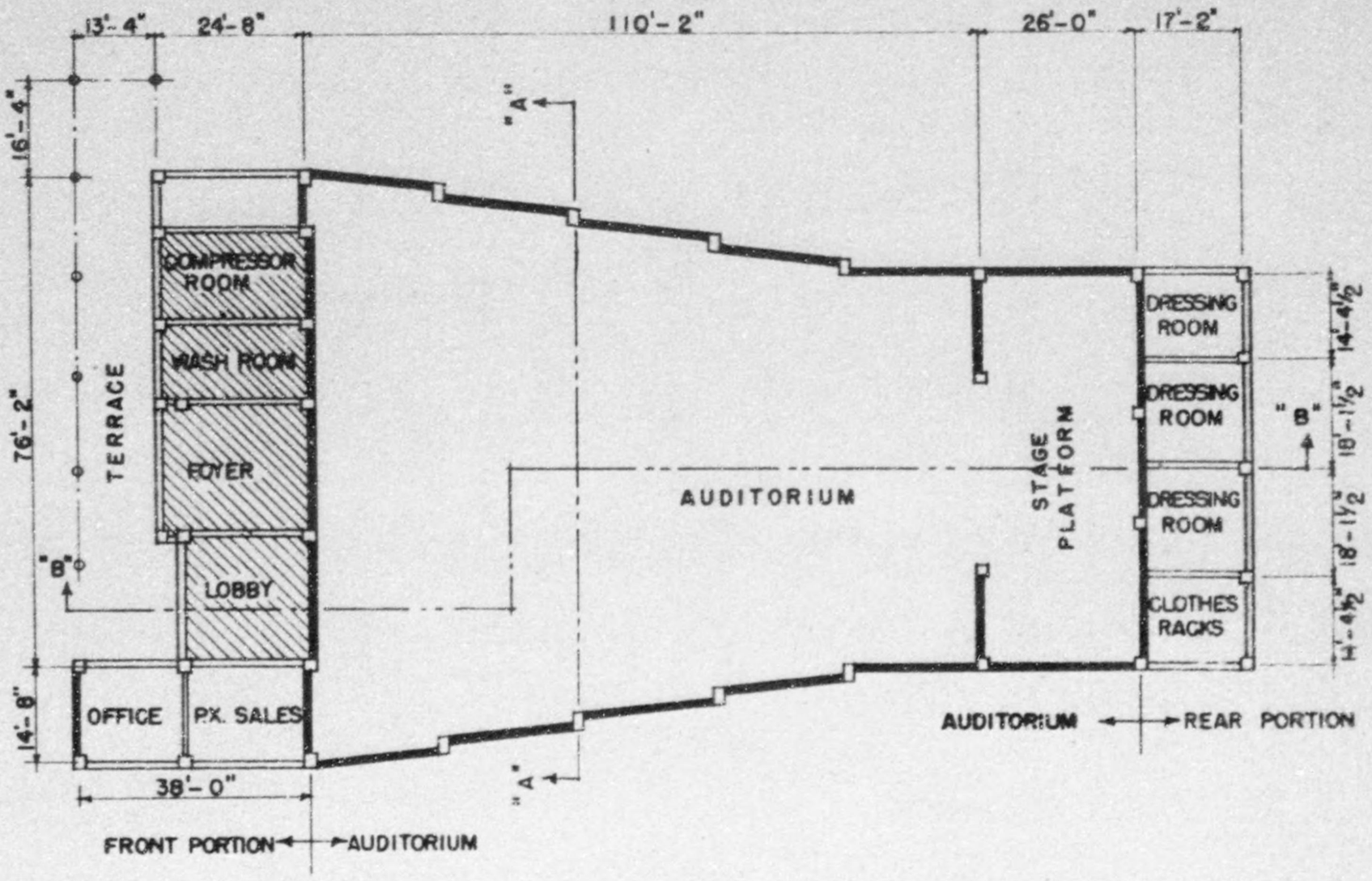
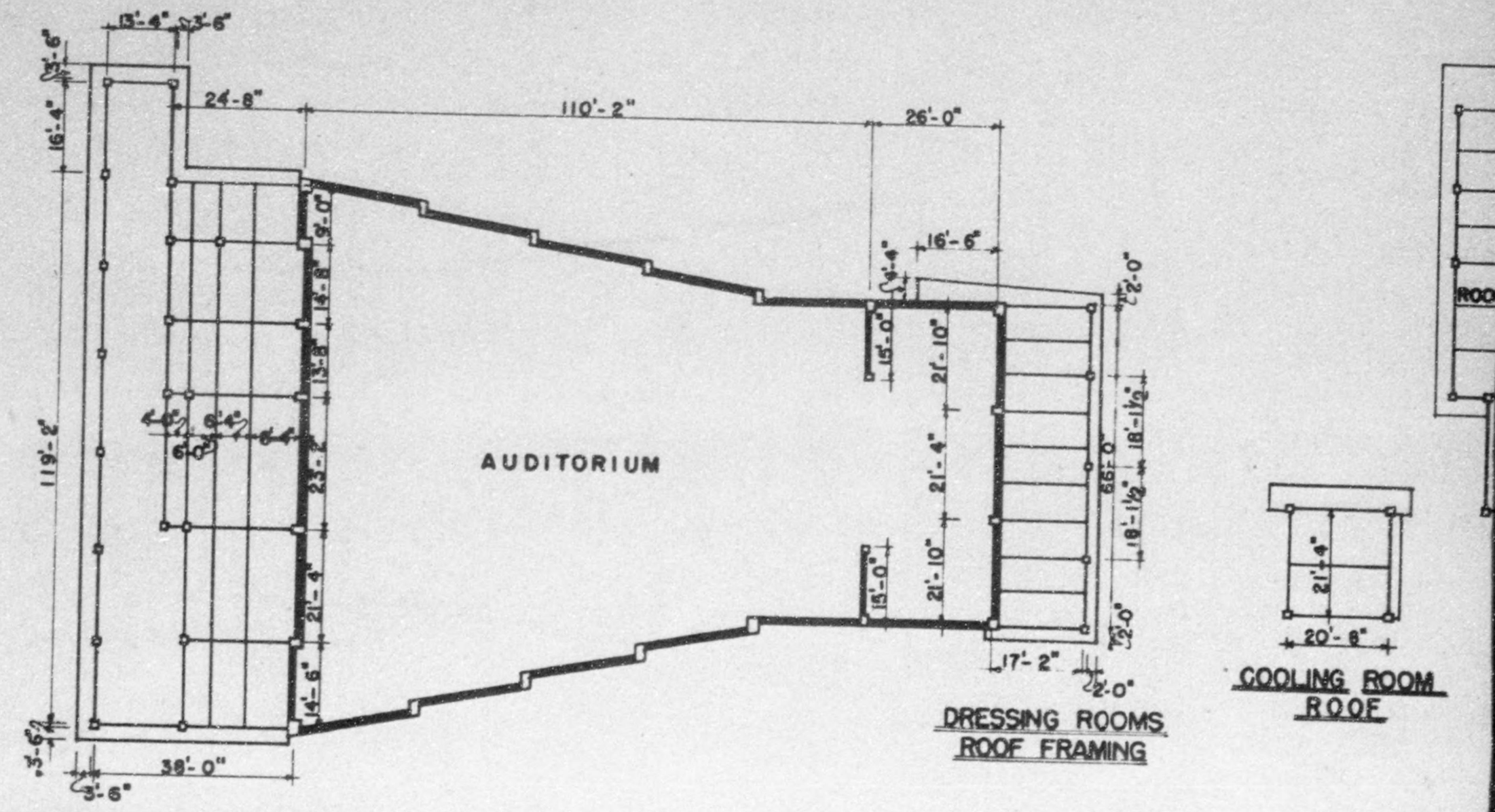


Fig. 279

1st Floor Plan - Army Theater

stage walls are all of reinforced concrete and resist earthquake forces acting in the longitudinal and transverse directions. The auditorium walls and the proscenium wall are 8 inches thick and the stage walls are 6 inches thick. The steel roof trusses are placed parallel to the screen with their end posts being securely anchored into the column tops to insure full transmission of earthquake forces.

The framing of the front portion is of reinforced concrete. The exterior walls are of concrete block and the



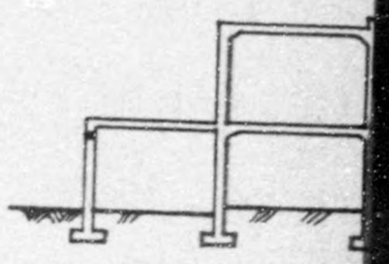
DRESSING ROOMS
ROOF FRAMING

COOLING ROOM
ROOF

FRONT PORTION 2ND FLOOR
AND TERRACE ROOF FRAMING

Fig. 280

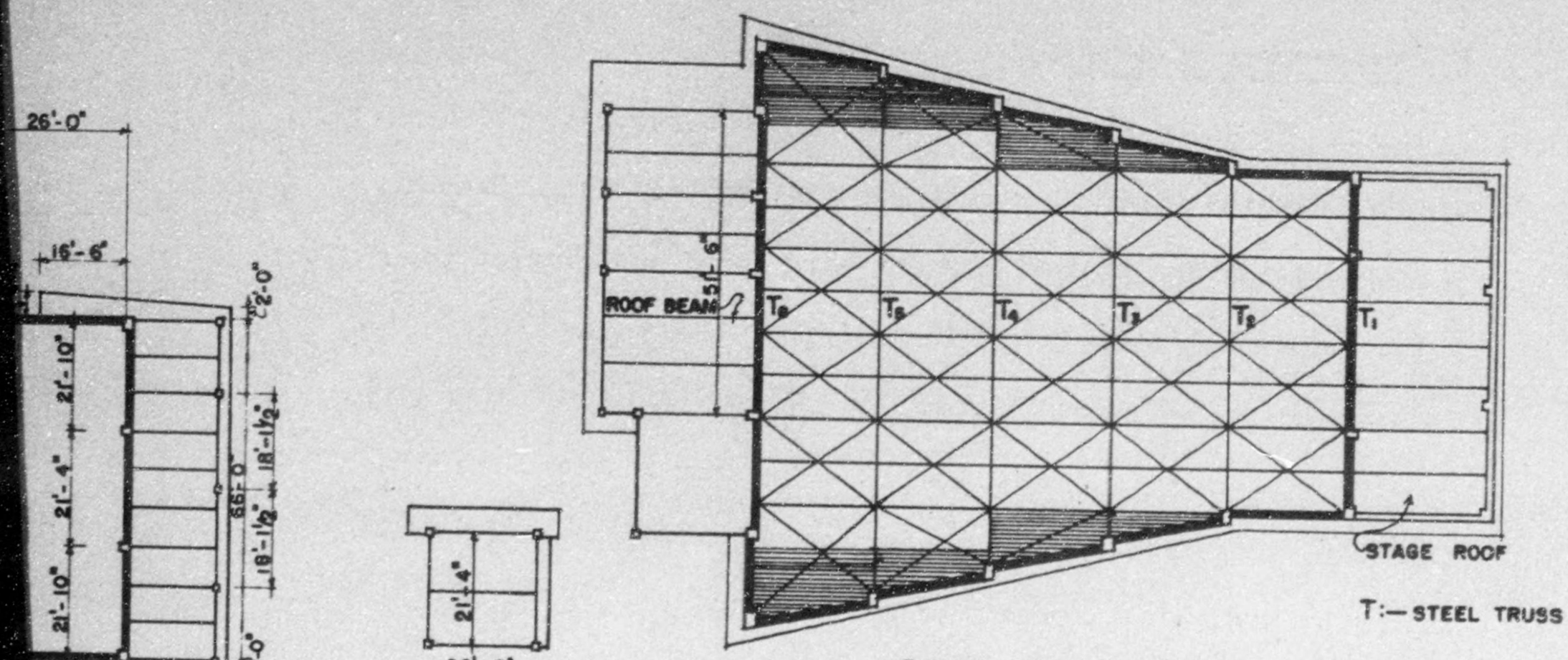
2nd Floor Plan - Army Theater



interior partitions are of hollow clay tile. An 8-inch reinforced concrete wall is provided in front of the office to carry a proper portion of the earthquake force acting in the direction of this wall. The second floor framing, roof framing, and elevation sections are shown by Figs. 280, 281 and 282, respectively.

b. Structural Planning

(1) General - The Joint Committee Recommendations and the Specifications of the American Institute of Steel Construction were used in the structural calculations. A wind



ROOF FRAMING PLAN

Fig. 281

Roof Framing - Army Theater

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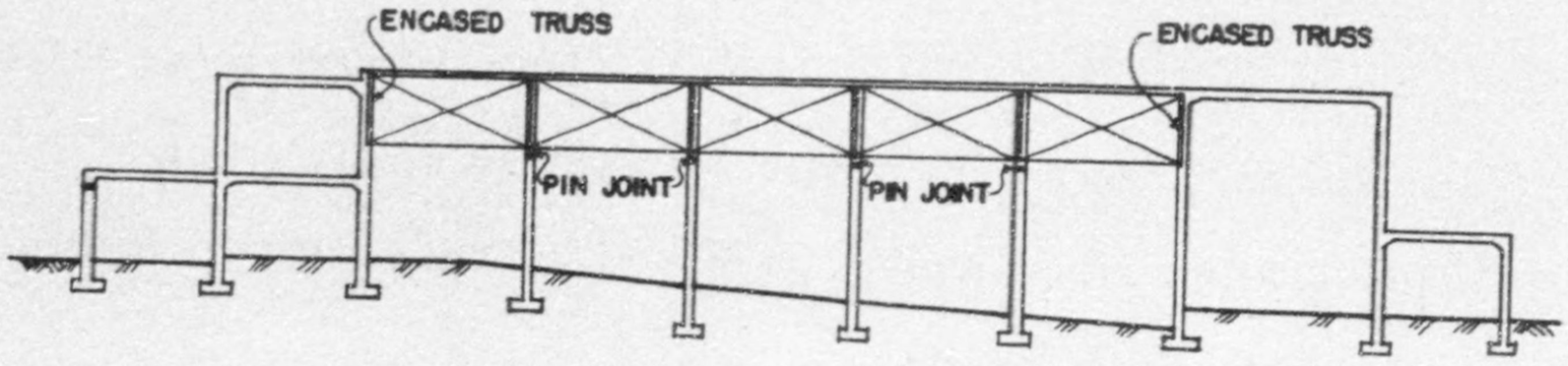

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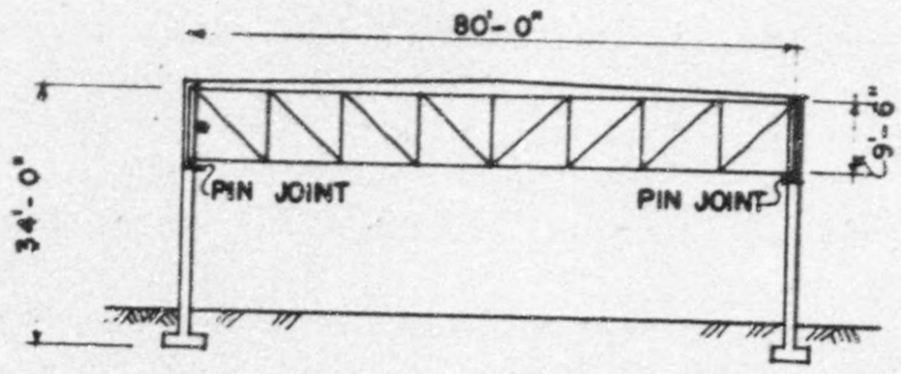

SECTION "A-A"

Fig. 282

Sectional Elevations - Army Theater

loading of 48 pounds per square foot on a projected vertical plane, and a seismic coefficient of k = 0.08g were considered in this design.

The connections of reinforced concrete beams and columns were considered as forming rigid joints. However, the connections of the beams and the tops of the columns of the terrace, and the joints formed by the roof trusses and the supporting columns are assumed to constitute pin joints. The bottoms of columns have been assumed as hinged. The location of the pin has been assumed to be 1 foot 6 inches above the bottom of the footing.

T beams and columns, integral with reinforced concrete walls, have been considered as T shaped sections and the moments of inertia have been computed accordingly. Effective flange width has been taken from Joint Committee Specifications, Section 804.

The direct compression of columns and footings under vertical loads was determined on the basis of supporting area, except when the effect of frame continuity is great in producing unsymmetrical moments at ends of members. The weight of reinforced concrete walls has been considered as transmitted to the columns of the lower story and not directly to columns where the walls are located.

(2) Earthquake Forces and Wind Loads - It was assumed that earthquake forces and wind pressure do not act