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THERMAL ELECTRIC POWER PLANTS OF THE U.S.S.R.

Report No. 92

VOLUME I

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TABLE OF CONTENTS

Volume I

LIST OF PLATES

Declassified in Part - Sanitized Copy Approved for Release 2013/02/25 : CIA-RDP81-01043R001600080008-4

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PREFACE

This report is designed to provide the analyst with a brief but comprehensive account of the layout, structural design, and structural details of steam-powered electric power plants in the Soviet Union. Included are some plants producing both electric power and steam for heating.

The subject is extensive, and the time (3 months, including extensions) for preparing the work is limited. Seventy-five per cent of the electric power produced in the USSR comes from thermal plants. Reports on 45 Soviet hydroelectric installations made up nearly half of a two-year program completed by the Air Information Division in 1955. It was obvious from the outset that a strict limit must be placed on the coverage of the present work. The Structural Engineering Section therefore decided to exclude the following phases of the subject:

Small plants, especially those serving single industrial installations:
Design of the mechanical and electrical equipment of the plants, even of the boilers, turbines, and generators;
Water and fuel supply installations;
Layouts of electric systems;
Transformer and switch yards;
Transmission lines and substations;
Structure of cooling towers (views of several examples of these, however, appear in the plates).

Only large steam turbine power plants are included in the report.

The scanning of possibly useful periodicals, too, had to be cut off where the probable take was too thin to justify the expenditure of man-hours. The coverage exploited was still extensive, as may be seen from the Bibliography. A general list of titles explored (whether fruitfully or not) is on file with the SES copy of the report.

In the introduction, the general course of the Soviet electrification development is briefly outlined.

The first chapter deals with the layouts of different sections of the main power plant building. The four basic types of widely used plant design are described, and examples of some existing power plants typifying these different layouts of sections are shown.

The second chapter analyzes the structural details of the main plant building.

The third chapter deals with separate large Soviet thermal power plants giving for each all available and some estimated data and photographic illustrations.

A word of caution is in order concerning data on plants in those parts of the Soviet Union that were overrun by the Germans in World War II. All existing power stations in those regions were presumably demolished by the invaders. It is understood that most of the early reconstruction work was based on the original designs. This Section, however, has no information as to the total amount of new and updated design work that went into the reconstruction effort. It is therefore anybody's guess whether the data in this report on any plant in regions which were under German occupation represents what is there now, or whether it has been replaced by a more modern installation.

INTRODUCTION

The electrification of Soviet Russia went through several stages in the course of its development. The original electrification plan "Goelro" (State Commission for the Electrification of Russia) was approved in 1921 and the construction of electric power plants received first priority. The guiding principles of the "Goelro" plan were: the restoration and reconstruction of old power plants, (part A of the Plan); the construction of 30 large regional power stations, mostly thermal (GRES), built in places where local low-grade fuel resources were available (peat, brown coal, anthracite culm, etc.); and connected with high-tension lines to form several main regional electric power grids (part B of the Plan).

Part A of "Goelro" plan was fulfilled by 1923, and part B around 1930. In the First Piatiletka (1928 - 1932) the original "Goelro" plan was extended, first by the increase of installed capacities in the already existing plants and secondly, by building 20 more power plants, almost all thermal. (See maps on Plates 1 and 2)

The main building of these power plants built up to the end of the first Piatiletka was usually a poured-in-place reinforced concrete frame structure with masonry curtain walls or brick panel walls and steel or wooden roof trusses. Columns were rigidly connected to poured-in-place pile-supported foundations and were entirely separate from the special pile-supported poured-in-place reinforced concrete foundations for the machinery.

Up to the end of the First Piatiletka very few hydroelectric stations were built. The main reason for this slow-down was that previously made geological surveys proved to be inadequate for the construction of dams and that more accurate surveys and plans were necessary.

Beginning with the Second Piatiletka (1933 - 1937) the construction of hydroelectric stations was started on a much larger scale.

The thermal power capacity was further expanded, mostly by additional installations in existing plants, and by building not only some new large power plants but also smaller local electrical stations which were switched into the steadily expanding regional grids. Larger new power plants were built, mostly in cities and around industrial centers. (See map on Plate 3).

The poured-in-place reinforced concrete type of construction was being gradually replaced by construction with precast reinforced concrete elements, such as columns, beams, and girders; the plant foundations and substructures, however, were still built of monolythic reinforced concrete.

This trend continued during the Third Piatiletka (1938 - 1941). During this time larger power and heat-and-power stations were built in the East, in the Ural Mountains, in Siberia, and in Central Asia; this building program in the East was especially intensified during the War.

During the War, precast reinforced concrete elements were not used in the construction of power stations because the precast fabricating plants were not operating.

After the War, during the Fourth Piatiletka (1946 - 1950) the first task was to rebuild the destroyed power plants. Next, rural electrification was greatly increased and extended whenever possible by the construction of small local hydroelectric plants. During the Fifth (1951 - 1955) and the present Sixth (1956 -) Piatiletkas large hydroelectric projects were carried out. The share of electrical energy produced by hydroelecrric plants rose to 25% of the total.

In the design and construction of new thermal power plants more standardization was attempted.

The power plants are under the Ministry of Electric Power Stations and plans for new stations are designed by the Institut Teploelektro-proyekt. The Institute prepares suggested master schemes for thermal plant construction. Precast reinforced concrete elements were prevailingly used in construction of plants; also rigid welded steel reinforcing skeletons which form the main reinforcement for the concrete frames of the building. Certain elements, usually roof beams, are of prestressed reinforced concrete. In the design of power plant foundations, instead of providing a separate foundation for each piece of auxiliary equipment, the design called for a basement with a heavy reinforced-concrete slab foundation. Only (1) the columns have separate footings, and (2) the boilers and turbogenerators rest on separate monolithic poured-in-place concrete blocks.

With boiler efficiencies increased so that a single boiler is sufficient to serve one turbogenerator, the design of thermal power plants has become simpler. The turbine hall is parallel to the boiler house, the turbogenerators are placed lengthwise to the turbine hall, and since one turbogenerator set longitudinally occupies the same space as its boilers, the condensing equipment being placed underneath in the basement, a block system boiler-turbine can be introduced. Instead of a central fuel preparation system (pulverizing for coal and milling for peat) at present each boiler has its own separate fuel preparation unit. Because boilers and turbogenerator sets are now (1956) larger and heavier, power plant construction has become stronger, foundations heavier, travelling bridge cranes of greater load-lifting capacities (100 - 200 m. tons), and consequently columns and girders stronger. The cross-section widths, however, of the boiler and turbine rooms can remain unchanged. One of the building's end walls is made as a temporary structure to be moved in case the plant is extended for the installation of new boiler-turbine block units. The unit capacities of these new boiler-turbogenerator sets have been increased. The boilers presently (1956) installed, work at higher pressures (up to 300 atm.) and higher temperatures (up to 650°C = 1200°F), and turbogenerators can be installed at higher capacity ratings (up to 150,000 kw.)

MAP OF DISTRICT ELECTRIC POWER PLANTS

AT THE END OF THE PIATYLETKA (OCTOBER 1ST, 1933) INDICATING POWER PLANTS WORKING IN 1930, THOSE BEING ENLARGED AND OTHERS UNDER ERECTION

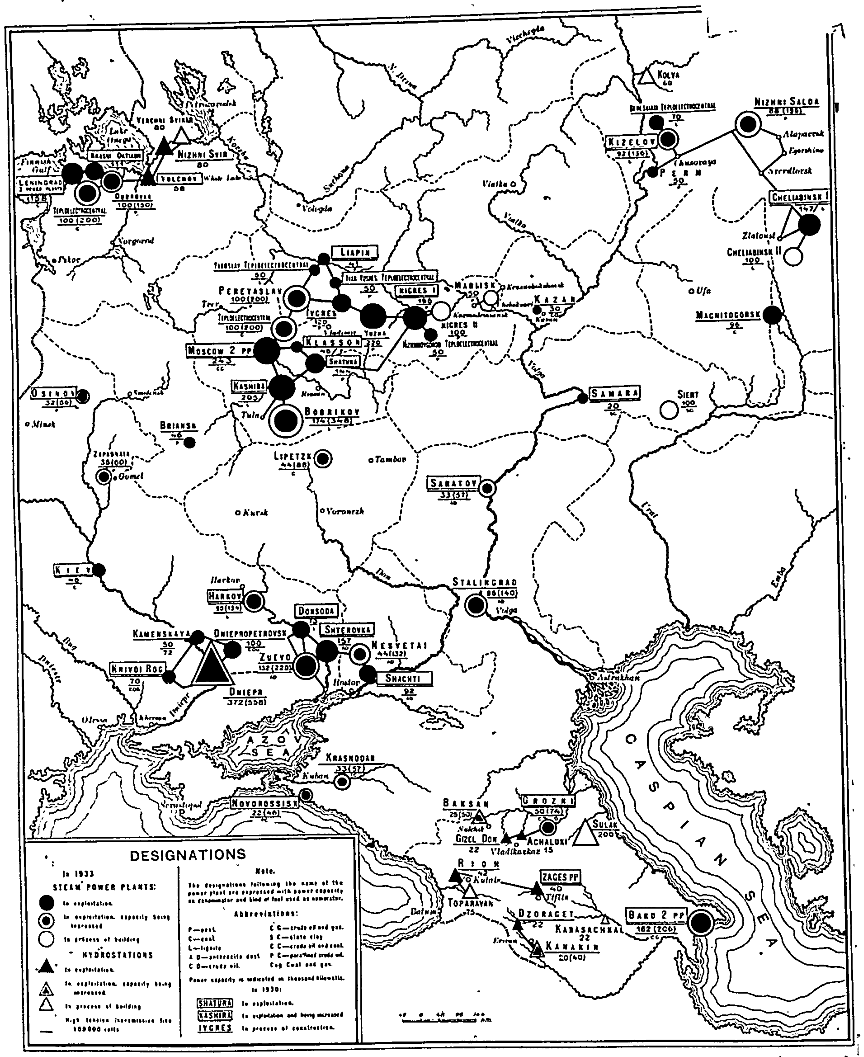

PLATE 1.

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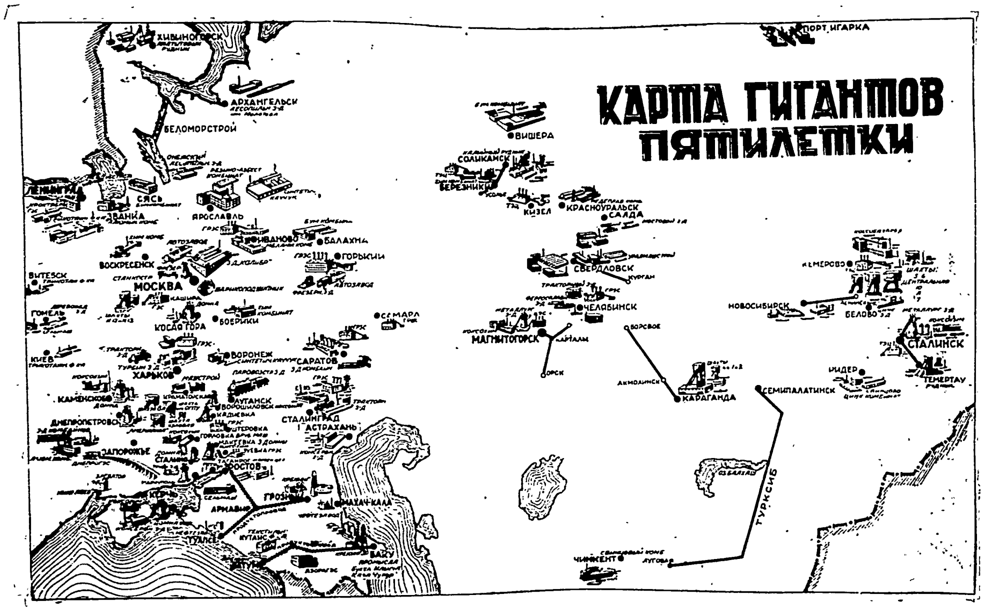


MAP SHOWING THE MAIN SOVIET POWER STATIONS AT THE END OF THE FIRST PIATILETKA IN PRINCIPAL INDUSTRIAL CENTERS.

Source: Prozhektor, 1933, No. 1, AP50.P76

PLATE 2.

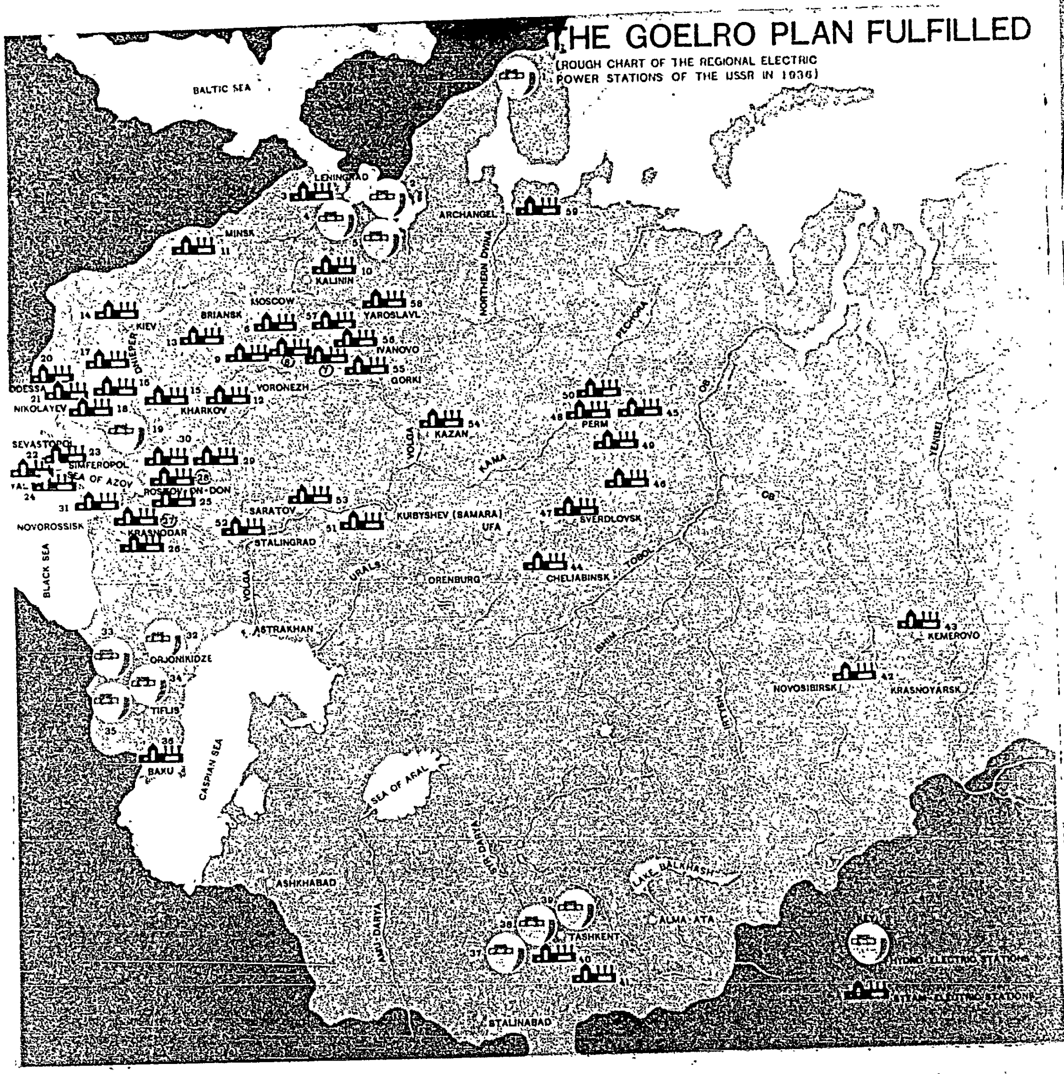
LIST OF REGIONAL ELECTRIC POWER STATIONS IN THE USSR IN 1936

POWER STATION	CAPACITY
1. NIVA HYDRO-ELECTRIC STATION	30,000 KW.
2. KONDOPOGA HYDRO-ELECTRIC STATION	4,500 »
3. LENINGRAD POWER STATIONS	378 000 »
4. VOLKHOV HYDRO-ELECTRIC STATION	66,000 »
5. LOWER SVIR HYDRO-ELECTRIC STATION	100,000 »
6. MOSCOW POWER STATIONS	314,000 »
7. SHATURA REGIONAL POWER STATION	180,000 »
8. KASHIRA REGIONAL POWER STATION	186,000 »
9. STALINOGORSK REGIONAL POWER STATION	100,000 »
10. KALININ POWER STATIONS	19,000 »
11. WHITE RUSSIAN REGIONAL POWER STATION	20,000 »
12. VORONEZH REGIONAL POWER STATION	24,000 »
13. BRIANSK REGIONAL POWER STATION	22,000 »
14. KIEV POWER STATION	64,000 »
15. KHARKOV POWER STATIONS (TOGETHER WITH KRASNOZAVODSK HEAT-AND-POWER STATION)	98,000 »
16. KRIVOI ROG REGIONAL POWER STATION	44,000 »
17. RYKOV POWER STATION	6,000 »
18. KAMENSKOYE POWER STATION	48,000 »
19. DNIEPER HYDRO-ELECTRIC STATION	558,000 »
20. ODESSA POWER STATION	37,000 »
21. NIKOLAYEVSK POWER STATION	13,500 »
22. SEVASTOPOL REGIONAL POWER STATIONS	8,000 »
23. SIMFEROPOL CITY POWER STATION	1,700 »
24. YALTA CITY POWER STATION	1,300 »
25. ROSTOV POWER STATIONS	128,000 »
26. KRASNODAR REGIONAL POWER STATION	10,000 »
27. SHTEROVKA REGIONAL POWER STATION	152,000 »
28. ZUYEVKA REGIONAL POWER STATION	200,000 »
29. NORTHERN DONETS REGIONAL POWER STATION	65,000 »
30. OTHER CENTRAL POWER STATIONS OF THE DONBAS	45,000 KW.
31. NOVOROSSISK REGIONAL POWER STATION	20,000 »
32. GIZELDON HYDRO-ELECTRIC STATION	23,000 »
33. RION HYDRO-ELECTRIC STATION	48,000 »
34. ZEMO-AVCHALI HYDRO-ELECTRIC STATION	25,000 »
35. DZORAGET HYDRO-ELECTRIC STATION	22,500 »
36. BAKU REGIONAL POWER STATION	176,000 »
37. KADYRYA HYDRO-ELECTRIC STATION	13,000 »
38. BURJAR HYDRO-ELECTRIC STATION	3,300 »
39. BOZ-SU HYDRO-ELECTRIC STATION	3,000 »
40. TASHKENT POWER STATION	3,500 »
41. FERGANA REGIONAL POWER STATION	8,200 »
42. NOVOSIBIRSK HEAT-AND-POWER STATION	35,500 »
43. KEMEROVO REGIONAL POWER STATION	48,000 »
44. CHELIABINSK REGIONAL POWER STATION	150,000 »
45. KIZEL REGIONAL POWER STATION	98,000 »
46. YEGORSHINO REGIONAL POWER STATION	24,500 »
47. SVERDLOV REGIONAL POWER STATION	11,000 »
48. PERM REGIONAL POWER STATION	8,000 »
49. MIDDLE URALS REGIONAL POWER STATION	50,000 »
50. BEREZNIKI HEAT-AND-POWER STATION	93,000 »
51. KUIBYSHEV REGIONAL POWER STATION	27,000 »
52. STALINGRAD REGIONAL POWER STATION	75,000 »
53. SARATOV POWER STATIONS	22,000 »
54. KAZAN POWER STATION	24,000 »
55. GORKI REGIONAL POWER STATION	204,000 »
56. IVANOVO POWER STATIONS	113,000 »
57. VLADIMIR HEAT-AND-POWER STATION	3,500 »
58. YAROSLAVL REGIONAL POWER STATION	36,000 »
59. ARCHANGEL REGIONAL POWER STATION	16,000 »

(THE NUMBERS AGAINST THE STATIONS IN THIS LIST CORRESPOND TO THE NUMBERS SHOWN IN THE CHART).

CAPACITIES IN KW OF REGIONAL POWER STATIONS IN THE USSR IN 1936 SHOWN ON MAP, PLATE 2.

<u>PLATE 3A.</u>



MAP OF REGIONAL POWER STATIONS IN THE USSR IN 1936

Source: USSR in construction, 1936, No. 6, DK267.A1U3

PLATE 3.

CHAPTER I

GENERAL LAYOUT OF THE MAIN PLANT BUILDING

The larger thermal electric power plants in the USSR have steam turbines as primery movers. Steam piston engines have been completely discarded, gas turbines are still only in an experimental stage and Diesel motors are used in power plants of smaller capacity serving local needs. The layout of plants has been influenced principally by the kind, efficiency and size of equipment installed, by the kind of fuel fired, and also, to certain extent, by the type of the station, i.e. whether it is a condensing or a heat-and-power producing plant.

The general scheme of a plant layout is mainly centered on the proper placing of the boiler house as the most complex section of the plant and on the grouping of the turbine hall and other sections in relation to it.

When the steam turbine was introduced as the prime mover, the total capacity of several boilers was necessary to produce a sufficient amount of steam for one turbine. In the first peat-firing plants built in the twenties therefore, we find that either: 1/ the boiler rooms are placed at right angles to the turbine hall; or 2/ the turbine hall is placed parallel between two boiler rooms.

Examples of the first type of arrangement are: 1/ the first priority of the 204,000 kw. Gor'kiy GRES (see Plate 4, Fig. 1) which has three double-row boiler rooms, 6 boilers to a turbine; 2/ the first and second priorities of the 180,000 kw. Shatura GRES (see Plate 4. Fig. 2) which has three single-row boiler rooms, 6 boilers to a turbine.

An example of the second type of arrangement is the 100,000 kw. Dubrovka GRES where the turbine hall is placed between two boiler rooms and parallel to them (see Plate 5).

With the rapid advance of boiler engineering, the improved boiler-turbine ratio made it possible to lay out electric power stations with the turbine hall and one boiler house parallel. At first, when 2 boilers were still needed for one turbine, the boilers were arranged either in a double row, as in the 152,000 kw. Shterovka pulverized-coal GRES (Plate 6) or in a single row but with somewhat spread-out turbo-generator sets as in the peat-burning 120,000 kw. Ivanovo GRES (see Plate 7). Further increase in boiler capacities made it possible to design stations with one boiler installed per turbine.

The turbogenerator sets are arranged lengthwise to the axis of the turbine hall in all stations, with the exception of the first priority of the Shatura and the Dubrovka plants, where the turbogenerators are installed crosswise.

The main building of a thermal electric power (or heat-and-power) station is composed of the following sections:

1. The fuel bunker gallery section, in most cases with the peat-milling or coal-pulverizing installation (central or individual for each boiler);
2. The boiler house;
3. The smoke discharge section (smoke exhaust flues, induced-draft fans;)
4. The section of feedwater tanks, feedwater pumps, deaerators etc;
5. The turbogenerator hall and auxiliary equipment.

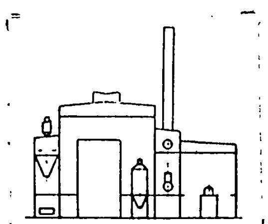
Those sections can be clearly identified on the model of a 5 x 50,000 kw. pulverized-coal station (see Plate 8) or on the model of a 200,000 kw. peat-firing power plant (see Plate 8a).

Different arrangements of these sections in relation to each other characterize the specific types of the main building design of thermal-electric power plants. The central point to be considered in a power plant design is the location of the boiler house as the most complex section of the plant.

The boiler house can have one or two free side walls - that is, walls that do not adjoin the turbine room or its auxiliary section. Outside a free side wall the designer can place the auxiliary sections of the boiler house, namely: 1. the bunker section (bringing and storing of fuel, and its preparation for burning, i.e. the milling or pulverizing equipment) and; 2. the smoke discharge section (smoke exhaust flues, induced-draft fans, smoke stacks).

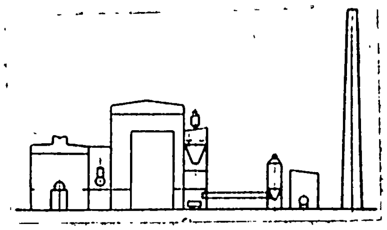
According to the location of the boiler house, the main buildings of thermal-power and heat-and-power plants can be classified in four general groups.

First Group



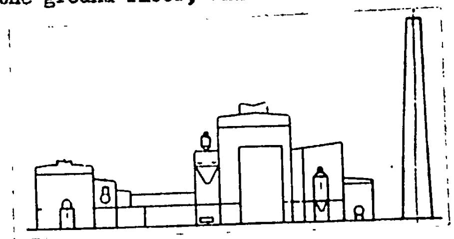
In the first group, the boiler house has one free wall, outside of which is the fuel bunker section. In this group, the stacks are in or next to the boiler house; they and the induced-draft fans are set on a high specially-constructed frame, or on top of the frame that supports the feedwater tank, pump, and deaerator section.

Second Group



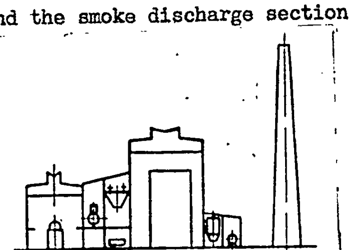
The Second Group includes power plants where the boiler house also has one free side wall, with the fuel bunker section adjoining; however, the smoke discharge installations are separated from the boiler house. Smoke flues are brought underneath the fuel bunker section, the induced-draft fans are placed low, on the ground floor, and the stacks are separated from the main building.

Third Group



In the Third Group, the boiler house has two free side walls, the fuel bunker section on one side and the smoke discharge section on the other.

Fourth Group



The fourth Group is like the Third in principle, but arranged more compactly, with the fuel bunker section set in a frame common to that of the turbine room.

The design of each of the groups is based on the general scheme of arrangement of the sections; however, within the framework of one group, some individual solutions in the design of the plant are feasible. The first group is the most common, especially in power plants built in the twenties and thirties.

Examples of the First Group arrangement are:

a. Zuyevka 96,000 kw. pulverized-coal GRES (see Plates 9 & 9a).

b. Kuznetsk 108,000 kw. pulverized-coal heat-and-power TETs. (see Plate 11)

c. Stalinogorsk pulverized coal GRES (see Plates 12 & 12a)

d. Ivanovo peat-burning GRES (see Plate 7)

e. Stalingrad pulverized-coal GRES (see Plate 10)

The layout of the First Group has the drawback that the smoke eliminating arrangement is located either totally or partially in the boiler house, and the induced-draft fans are placed at a high level, so that their draft capacity is reduced. In the Second Group design, this problem is solved by bringing the smoke exhaust flues to the lower level and placing the induced-draft fans and smoke stacks outside the boiler house. Examples of the Second Group are:

a. Nesvetay GRES (see Plate 13)

b. Stalinsk (Mosenergo #11) TETs in Moscow (see Plate 14)

The First and Second Groups form the "compact" groupings of the station sections. The Third Group is called the "disjointed design", the boiler house and the turbine hall being in different buildings. An example of this arrangement is the Orsk TETs (see Plate 15).

The Fourth Group arrangement is the latest design of the power plant building. Some power plants have already been built according to this arrangement, probably the Cherepet' GRES; also Mironovsk GRES and Slavyansk GRES.

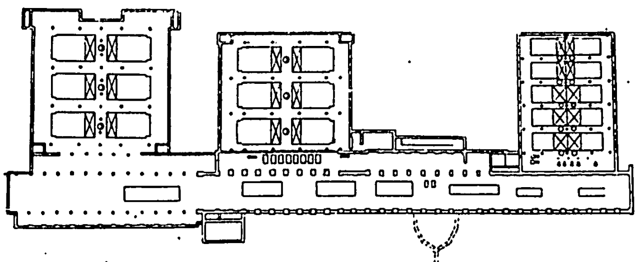


Fig. 37.—Gorki regional station (plan view).

Fig. 1 - Gor'kiy Peat-Firing GRES

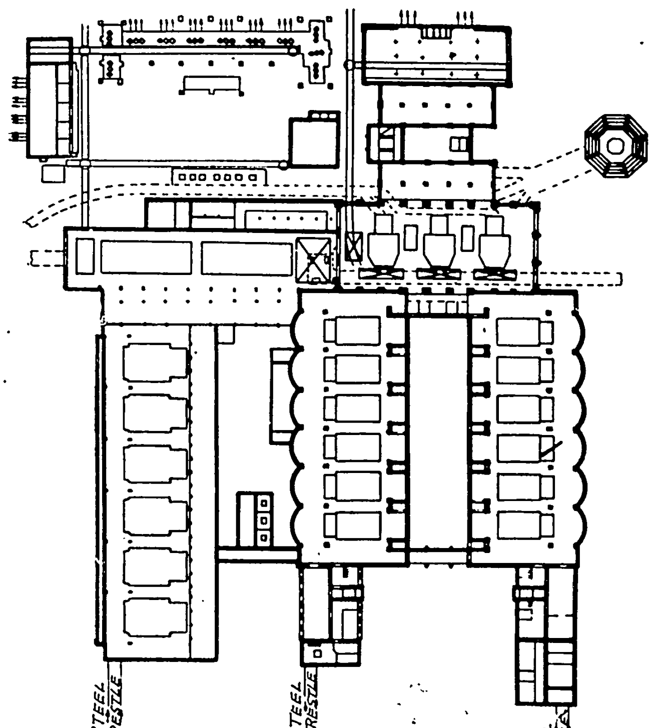


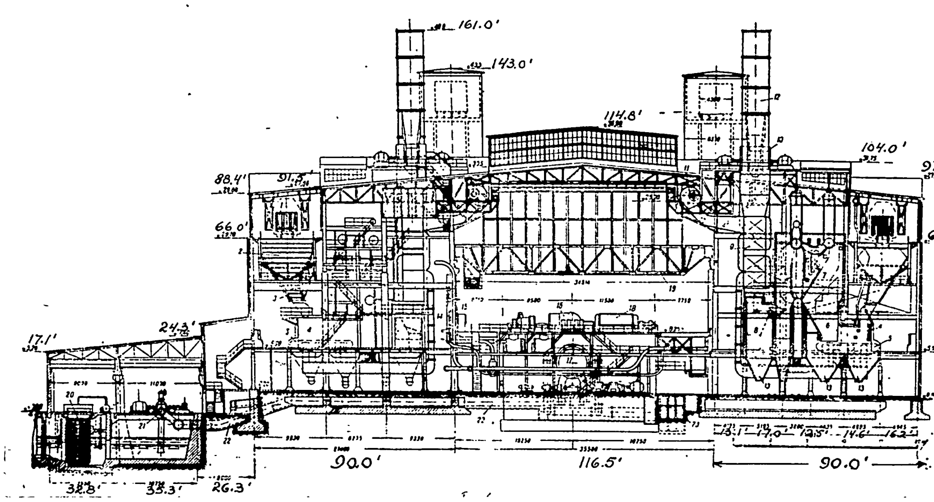
Fig. 38.—Shatura regional station (plan view).

Fig. 2 - Shatura Peat-Firing GRES

THERMAL POWER PLANTS WITH BOILER ROOMS AT RIGHT ANGLE TO TURBINE HALL.

Source: Weitz, 1. ed. Electric Power Development in the USSR, 1936. TK85.E6.

PLATE 4.

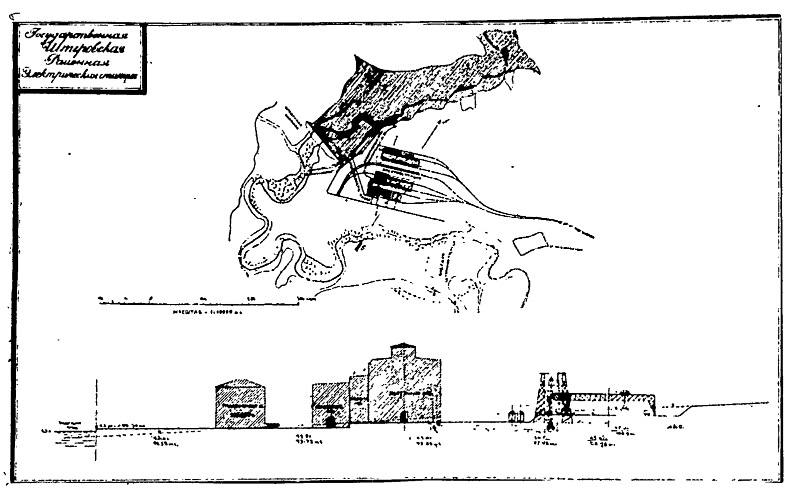


1 - Peat car; 2 - Steel peat bunker; 3 - Peat chute to the furnace well; 4 - Furnace well; 5 - Mechanical chain grate; 6 - Furnace; 7 - 3-drum water-tube boiler, capacity - 135/160 t./h.; 8 - Water economizer; 9 - Air preheater; 10 - Smoke-exhaust fan (induced draft); 11 - Forced draft fan; 12 - Stack; 13 - Ash and cinder bunkers; 14 - Feedwater pipeline; 15 - Steam pipeline; 16 - Steam turbine, capacity 50,000 kw.; 17 - Condenser; 18 - Three-phase generator; 19 - Crane bridge; 20 - Mechanical filter mesh; 21 - Circulation pumps; 22 - Water main; 23 - Drainage channel; 24 - Feedwater tanks.

DUBROVKA PEAT - FIRED GRES.

Source: Weitz, b. Electric Power Development 1936. TK85.E6, 1936.
Antipov, I. P. Arkhitektura elekrostantsiy, 1939. TH4581.AS.

PLATE 5.

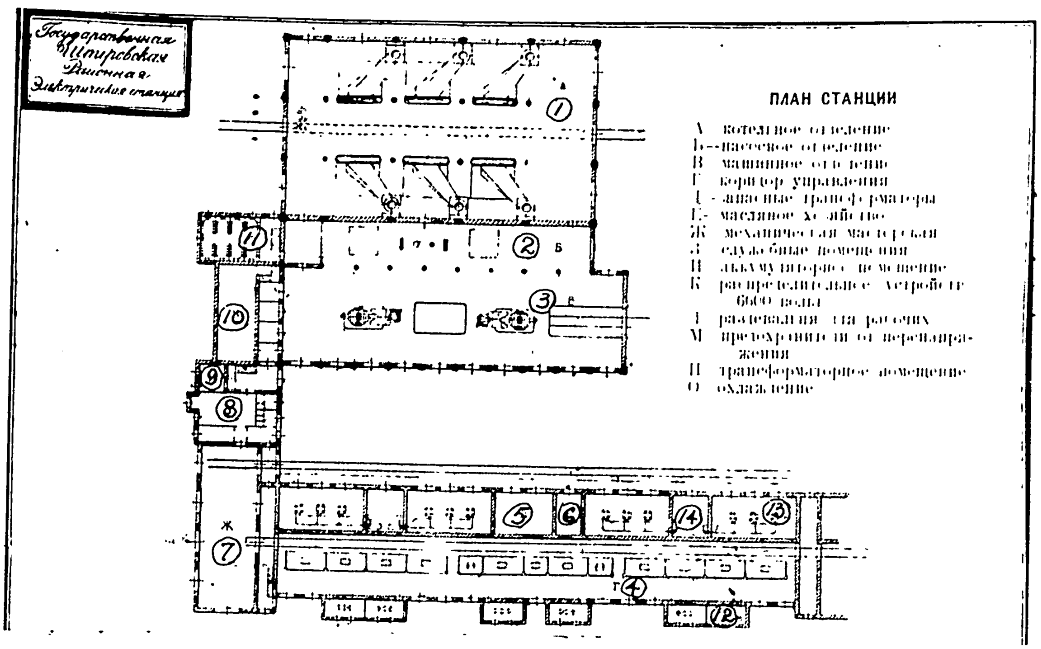


<u>Site plan and sections of main buildings.</u>

SHTEROVKA PULVERIZED COAL FIRING GRES (Capacity: 152,000 kw.)

Source: Stroitel'naya Promyshlennost' 1924, #4, p. 242.

PLATE 6A



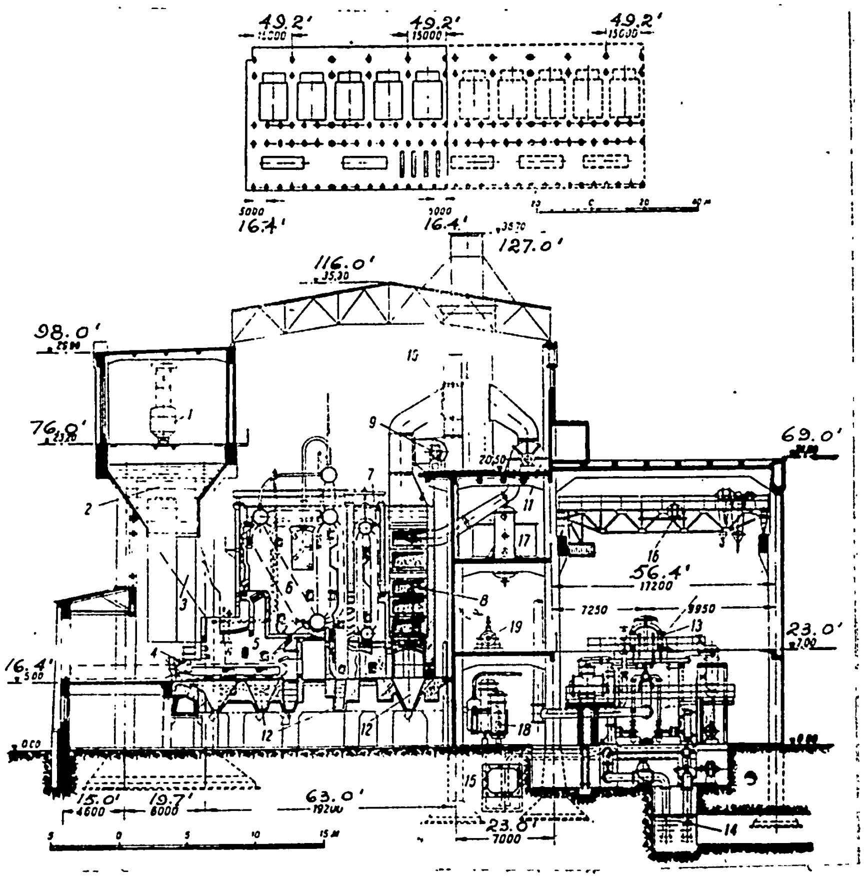
Plan of power plant.

1. Boiler house
2. Pump section
3. Turbine hall
4. Control section
5. Reserve transformers
6. Oil section
7. Work shop
8. Service quarters
9. Accumulator section
10. 6,600 Volt distributing installation.
11. Locker room for workers
12. Circuit breakers
13. Transformer section
14. Cooling section

SHTEROVKA PULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: Stroitel'naya Promyshlennost', 1924, #4, p 243.

PLATE 6)



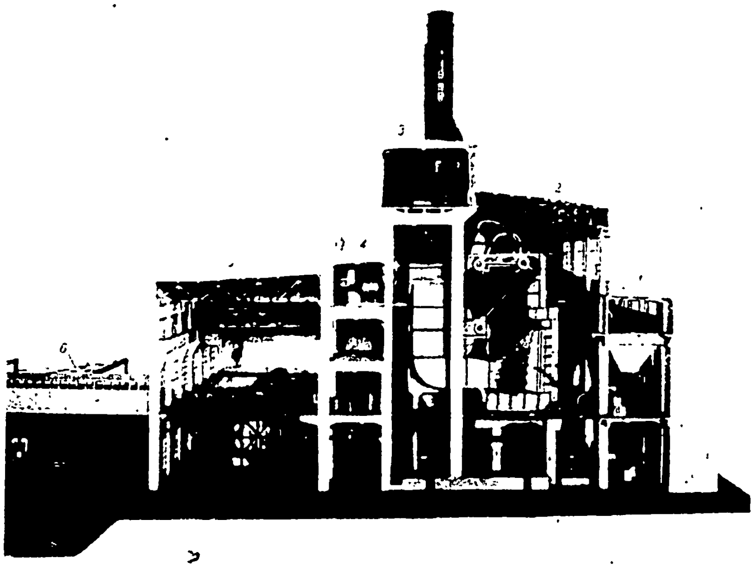
Main building, section and plan:

1 - Electric locomotive in the over-bunker gallery; 2 - Peat bunker; 3 - Peat chute to the furnace; 4 - Mechanical chain grate; 5 - Furnace; 6 - Boiler; 7 - Water economizer; 8 - Air preheater; 9 - Smoke-exhaust fan (induced draft); 10 - Stack; 11 - Forced-draft fan; 12 - Ash and cinder bunker; 13 - Turbogenerator, capacity - 24,000 kw.; 14 - Water channel and sump; circulation pumps; 15 - Drainage channel; 16 - Crane bridge; 17 - Feedwater tank; 18 - Feedwater preheaters; 19 - Boiler feedwater pumps.

IVANOVO PEAT-FIRING GRES

Source: Antipov, I. P. and S. S. Rakita. Arkhitektura elektrostantsiy, 1939, TH4581.A5.
Weitz, B. ed. Electric power development, TK85.E6 1936.

PLATE 7.

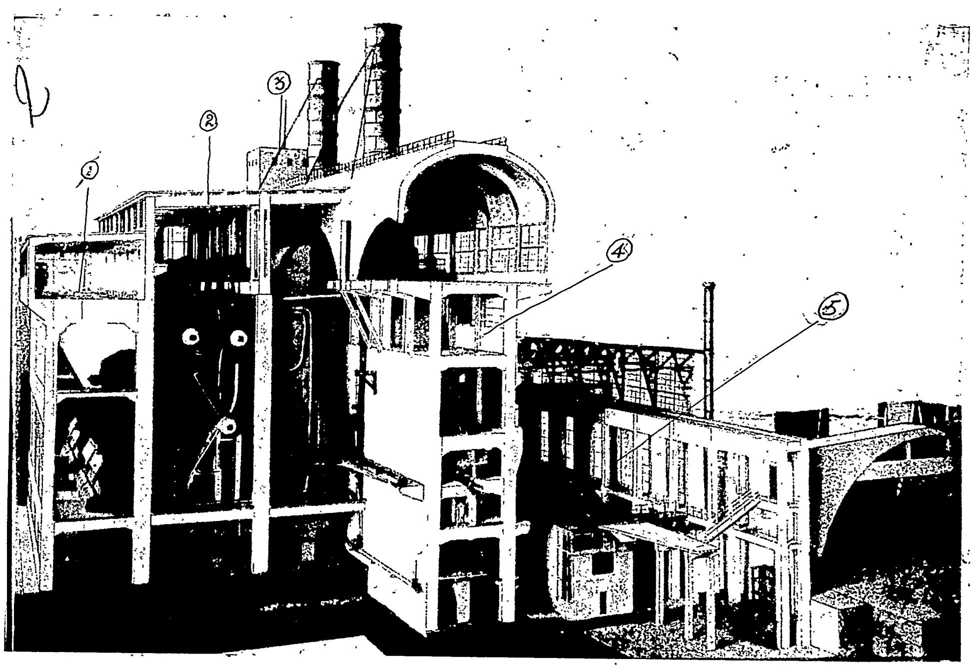


1. Bunker gallery
2. Boiler house
3. Induced draft fan gallery
4. Pump gallery
5. Turbine hall
6. Catwalk

MODEL OF A 5 x 50,000 kw. PULVERIZED-COAL-FIRING GRES.
(Section through the Main Power Plant Building.)

Source: Antipov, I. P. and S. S. Rakita, Arkhitektura Elektrostantsiy, 1939 TH.4581,A5.

PLATE 8.

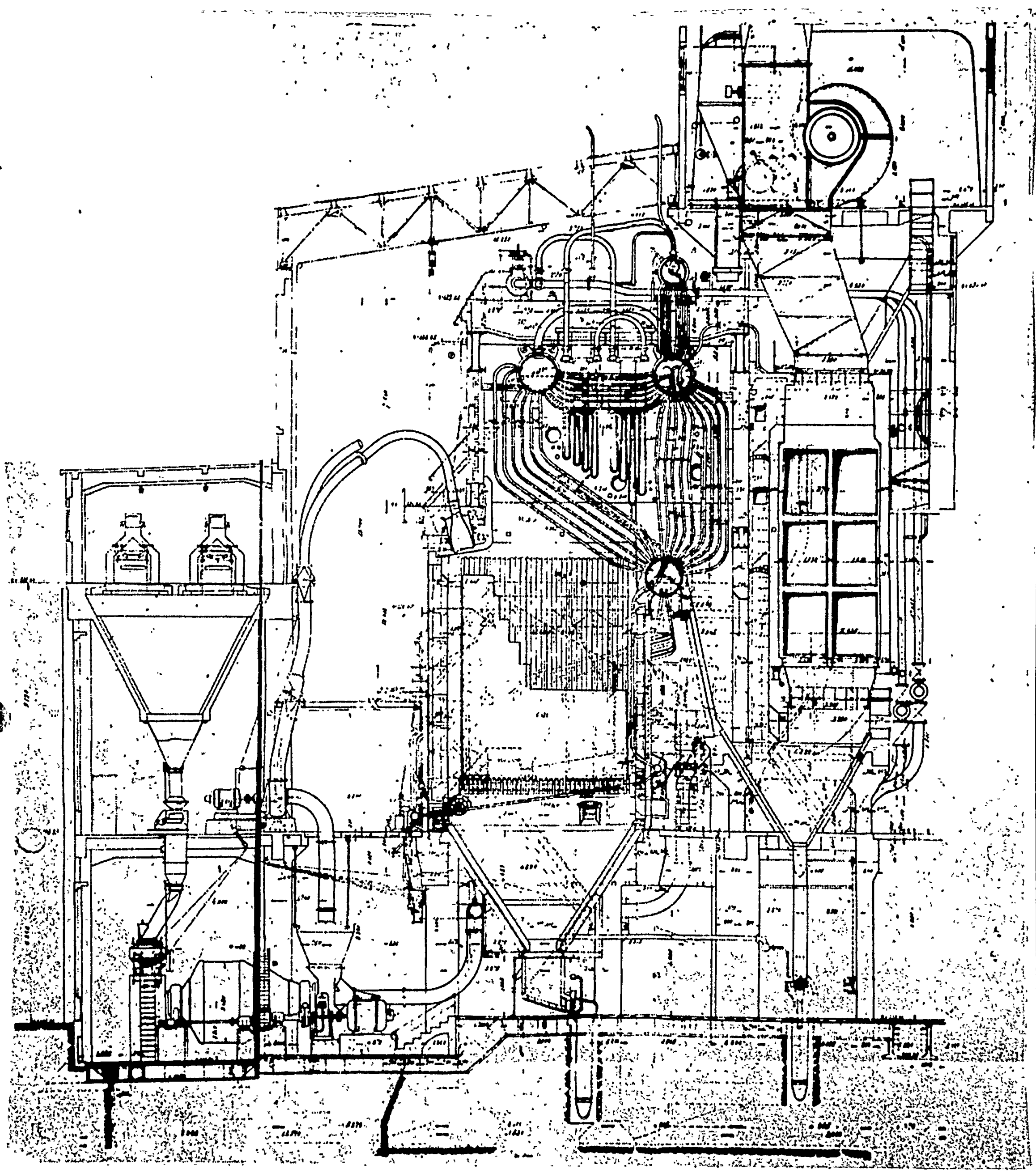


1 - Bunker section with the peat pulverizing installation; 2 - Boiler house; 3 - Forced draft fans and smoke exhaust installation; 4 - Feedwater tanks, feedwater pumps, deaerators etc. bay; 5 - Turbine hall.

MODEL OF A 200,000 kw. PEAT-FIRING ELECTRIC POWER PLANT.

Source: Elektricheshiye Stantsii, 1932, No. 7, front cover TK4.E725.

PLATE 8A.

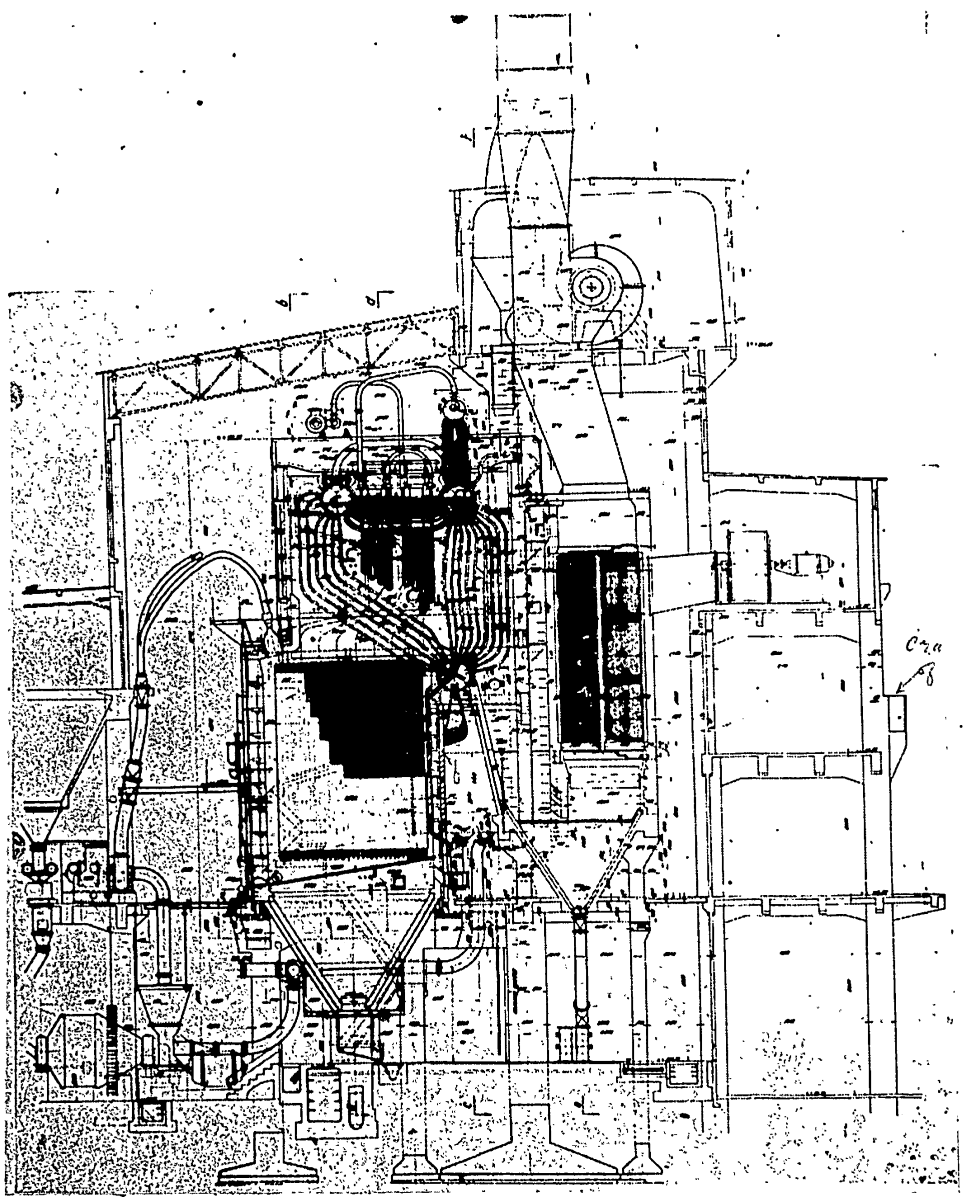


Cross section through the boiler house and the bunker gallery. The feed pump and tank section and the turbine hall are not shown - they adjoin the boiler house to the right.

ZUYEVKA 200,000 kw PULVERIZED-COAL GRES

Source: Elektroenergetika SSSR 1934 p. 68 TK 1193 R9 E4

PLATE 9.



Cross section through the boiler house and the feedwater pump and tank section. The bunker section is to the left of the boiler house and the turbine hall to the right of the feedwater pump and tank section.

ZUYEVKA 200,000 kw PULVERIZED-COAL GRES

Source: Elektroenergetika SSR 1934 p. 68, Tk 1193 R9 E4

PLATE 9A.

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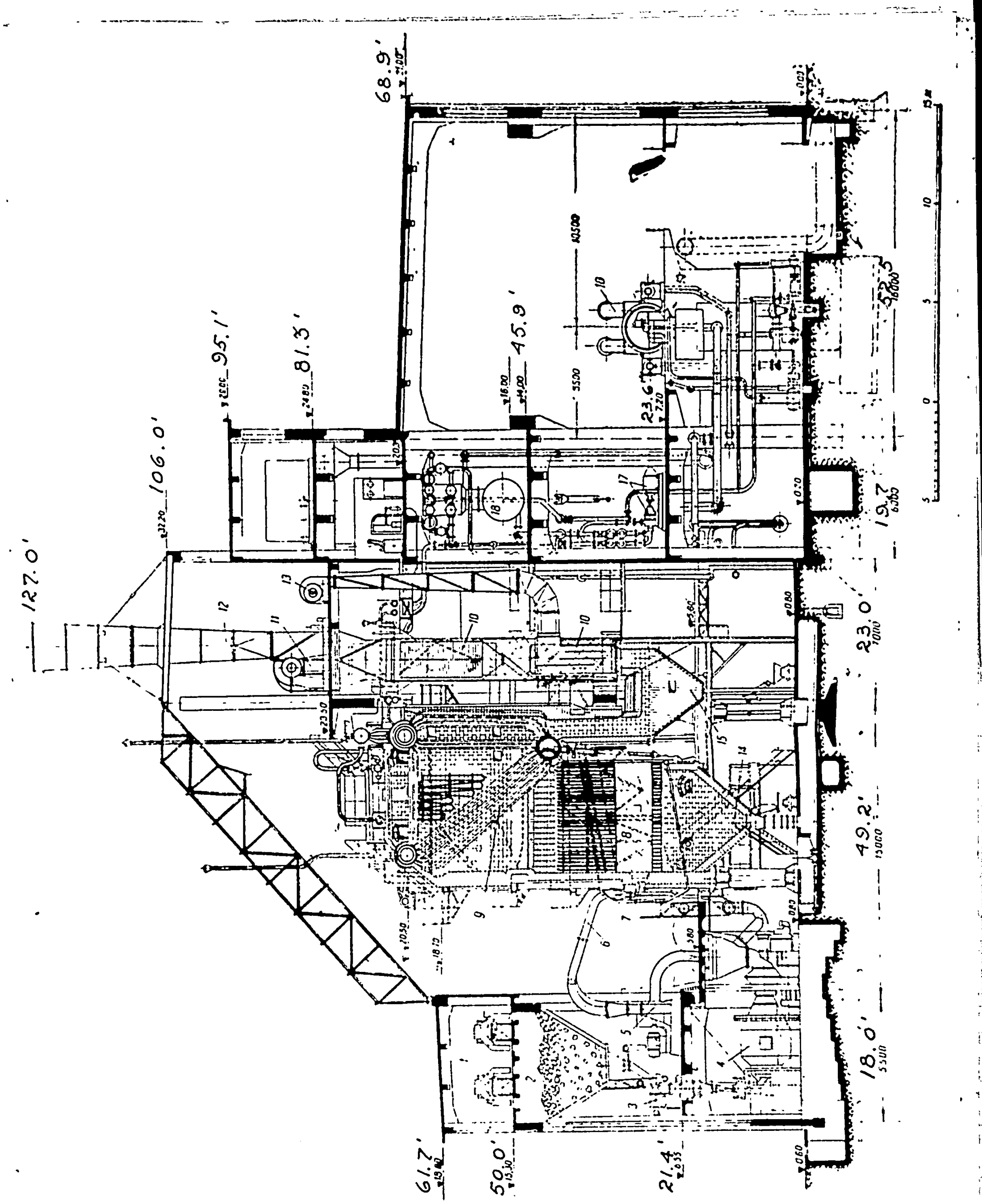
LEGEND: PLATE 10

1. Belt conveyers
2. Raw coal bunker
3. Automatic scales
4. Ball mills
5. Pulverized-coal injector
6. Pulverized-coal ducts
7. Pulverized-coal burners
8. Furnace chamber
9. Boiler
10. Air preheater
11. Smoke exhaust fan (induced draft fan)
12. Stack
13. Forced-draft fan
14. Cinder bunker
15. Ash bunker
16. Steam turbine, capacity 24,000 kw.
17. Boiler-feed pump
18. Feedwater tank

STALINGRAD PULVERIZED-COAL GRES

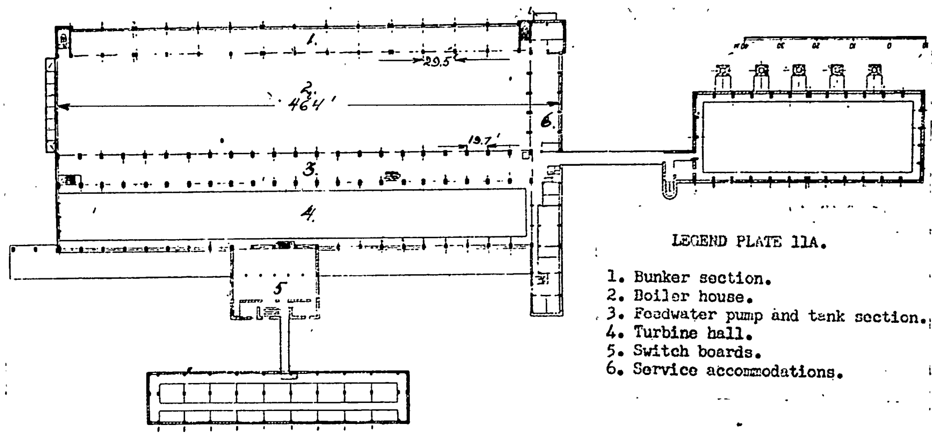
Source: Antipov, I. P. and S. S. Rakita, Arkhitektura Elektrostantsiy, 1939, TH.4581.A5
Weitz, B. Elektroenergetika SSR. V. 1, 1934, TK1193.R9E4.



Section through the main power plant building. Smoke-discharge section is part of boiler house with exhaust arrangement in upper part of structure.

STALINGRAD PULVERIZED-COAL GRES

PLATE 10.



LEGEND PLATE 11A.

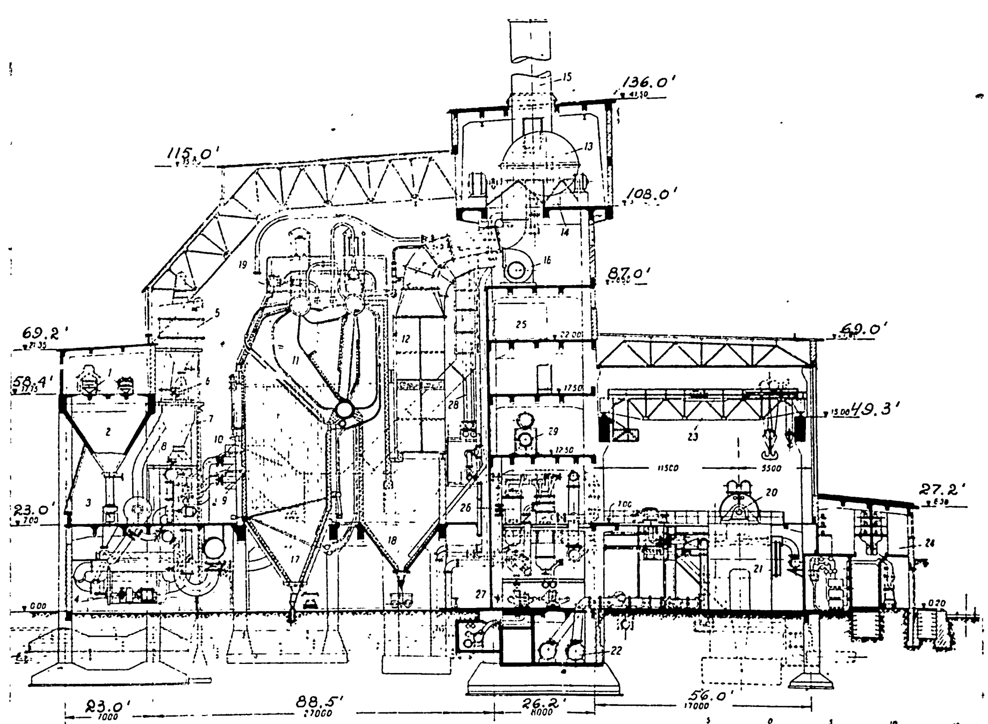
1. Bunker section.
2. Boiler house.
3. Feedwater pump and tank section.
4. Turbine hall.
5. Switch boards.
6. Service accommodations.

LEGEND: PLATE 11

1 - Belt conveyers; 2 - Raw coal bunker; 3 - Automatic scales; 4 - Ball mills; 5 - Dust remover conveyers; 6 - Worm dust conveyers; 7 - Pulverized-coal burners; 8 - Worm conveyer feeders; 9 - Pulverized-coal burners; 10 - Furnace; 11 - Boilers; 12 - Air preheater; 13 - Ash intercepting cyclone; 14 - Induced-draft fan; 15 - Stack; 16 - Forced-draft fan; 17 - Cinder bunker; 18 - Ash bunker; 19 - Steam line to the engine room; 20 - Steam turbine; capacity - 24,000 kw. 21 - Condenser; 22 - Water outlet lines; 23 - Crane bridge; 24 - Station distributor; 25 - Feedwater tanks; 26 - Feedwater preheaters; 27 - Boiler-feedwater pumps; 28 - Feedwater line; 29 - High pressure heaters.

KUZNETSK PULVERIZED-COAL TETs OF THE METALLURGICAL PLANT.
Source: Antipov, I. P. Arkhitekura Elektrostantsiy, 1939. p. 177. TH 4581.A5.

PLATE 11A.



Section through the main plant building.

KUZNETSK PULVERIZED-COAL HEAT-AND-POWER STATION TETs

PLATE 11.

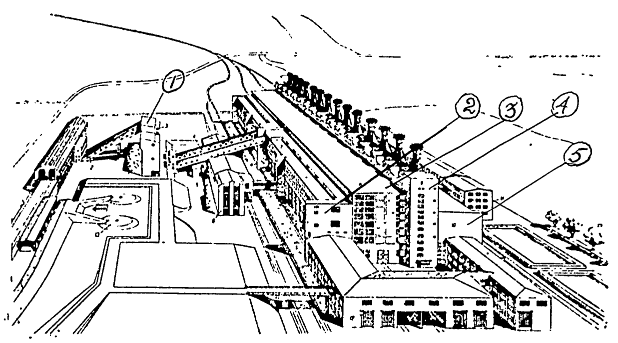


PLATE 12a: STALINOGORSK PULVERIZED-COAL GRES
General layout of the power plant building on the bunker side and coal conveying and crushing installations.

1 - Coal-crushing installation; 2 - Coal-bunker section; 3 - Boiler house; 4 - Water tank and pump section, with smoke exhausts and stacks are installed above; 5 - Turbine hall.

Source: Weitz, B., Electric Power Development in the USSR, 1936 TK85.E6, and Antipov, I. P. Arkhitektura elekrostantsiy, 1939. TH4581.A5

LENGEND: PLATE 12

6 - Ball mills; 7 - Pulverized-coal bunker; 10 - Boiler room; 11 - Boiler-feedwater pump; 12 - Switchboard; 13 - Water tanks; 15 - Induced-draft fan gallery; 18 - Steam turbine; 19 - Circulating pump; 21 - Control room; 22 - Switching and distributing installation.

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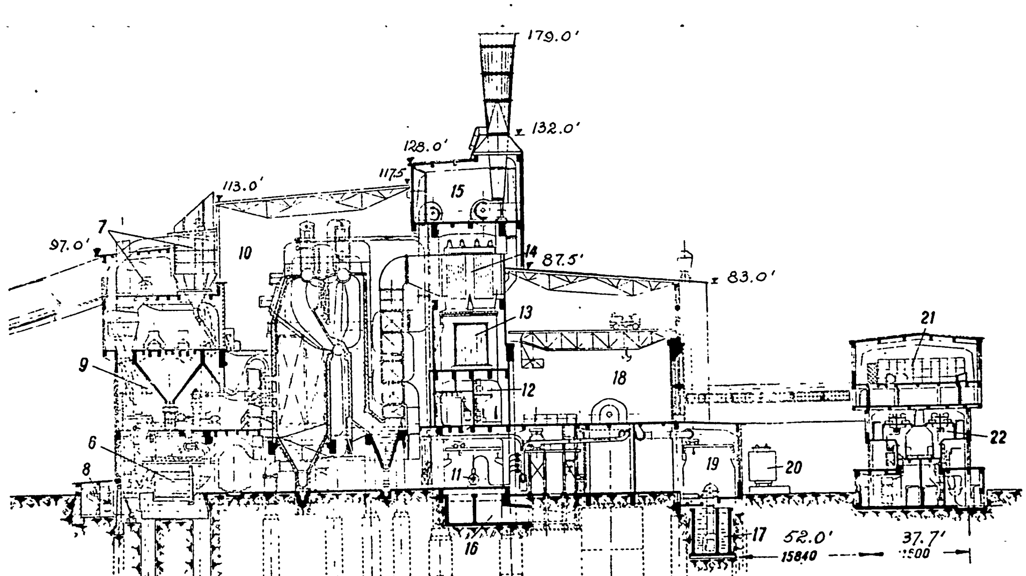


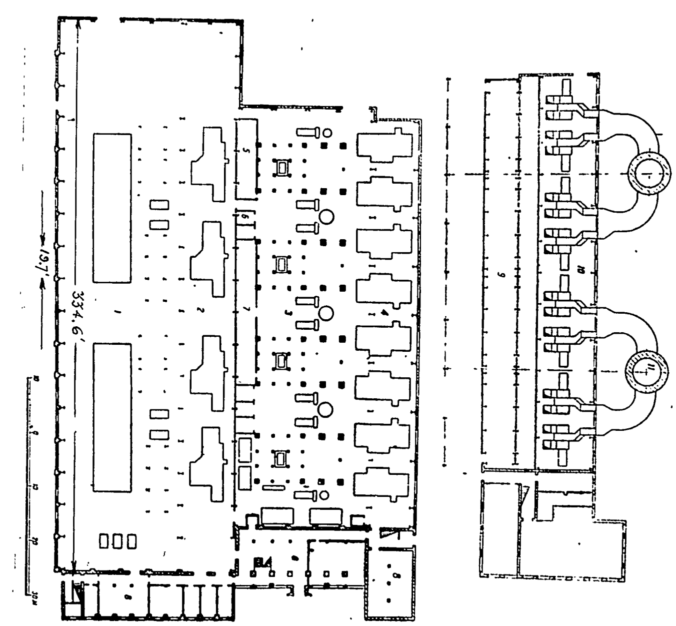
Cross section through the main power plant building.

STALINOGORSK PULVERIZED-COAL GRES

PLATE 12.

Legend To Plate 13a

1. Turbine hall
2. Feedwater-pump and tank section
3. Boiler house
4. Bunker section
5. Switchboard for lights
6. Transformers
7. Switchboard for 500 volts
8. Personnel accommodations
9. Electric filters.
10. Smoke eliminating section
11. Stacks.
12. Chemical water-cleaning installation



Plan of the first floor of the main building.

NESVETAY PULVERIZED COAL GRES.

PLATE 13A.

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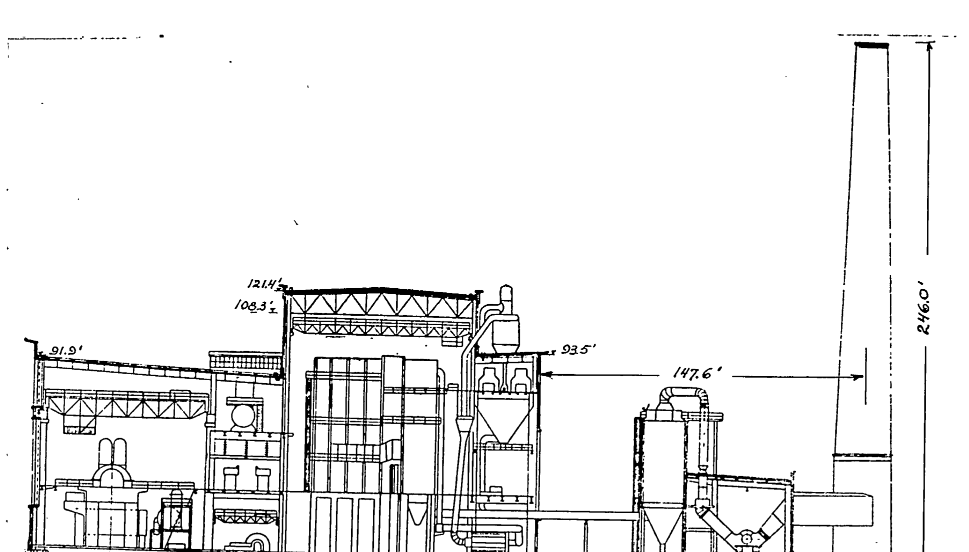


Section through the main building.

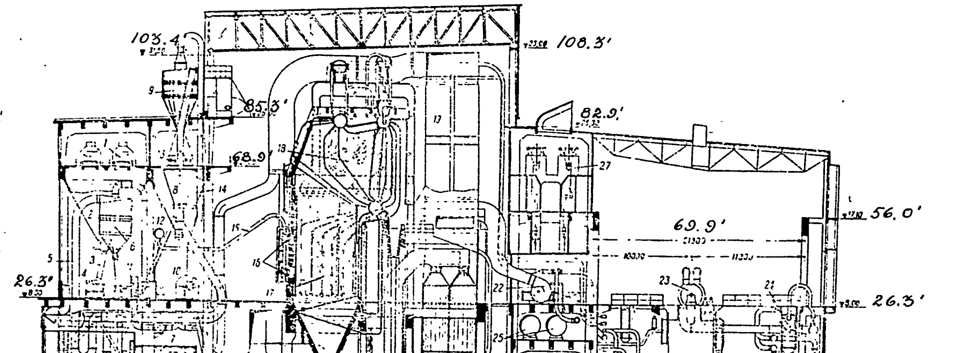
NESVETAY PULVERIZED-COAL GRES

Source: Antipov, I. P. Arkhitektura Elektrostantsiy, 1939, TH.4581.A5, and Elektricheskiye Stantsii, 1947, No. 5, TK4.E725.

PLATE 13.

LEGEND: PLATE 14

1. Belt conveyers
2. Raw coal bunker
3. Duct
4. Automatic scales
5. Drying duct
6. Drying cyclone
7. Mills
8. Pulverized-coal separator
9. Pulverized-coal cyclone
10. Mill fan
11. Primary air fan
12. Primary air collector
13. Pulverized-coal worm conveyers
14. Pulverized-coal bunker
15. Pulverized-coal ducts
16. Pulverized-coal burners
17. Furnace chamber
18. Three-drum boiler; capacity 150-180 t./1.
19. Air heater
20. Electric filters
21. Induction fans
22. Forced-draft fans
23. Steam power and heat turbines; capacity - 25,000 kw.
24. Water heating apparatus for district heating lines
25. Deaerators
26. Feedwater tanks
27. Stations own - need switchboard

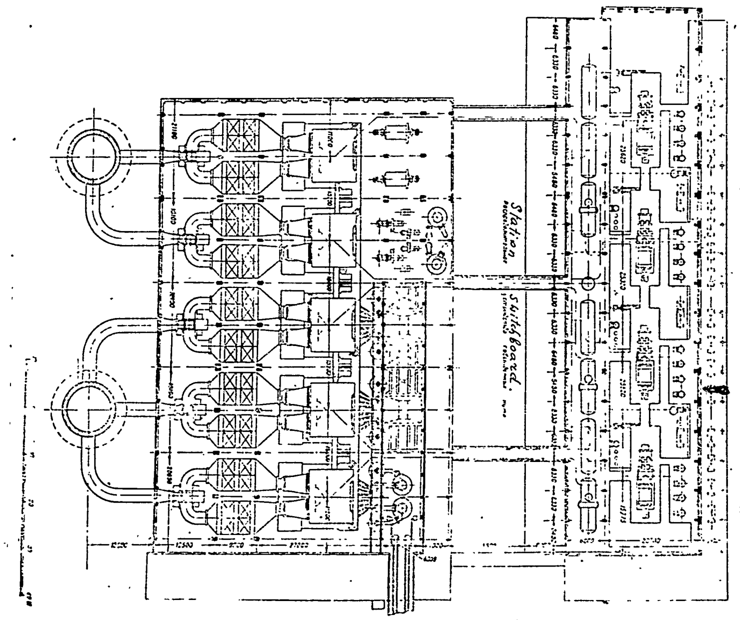


Section through the main building - Stacks located 40 m. apart from the building on side of the bunker section are not shown on the drawing.

STALINSK (MOSENERGO #11) PULVERIZED-COAL TETs IN MOSCOW

Source: Elektricheskiye Stantsii, 1947, #5 and #8, TK4.E725
Antipov, I. P. Arkhitektura Elektrostantsiy, 1939, TH4581.A5.

PLATE 14.



Plan of the main building of the station.

ORSK PULVERIZED-COAL TETs

Source: Weitz, b., Electric Power Development in the USSR, 1936, TK85.E6, Antipov, I. P. Arkhitektura elekrostantsiy, 1939, TH4581.A5.

PLATE 15A.

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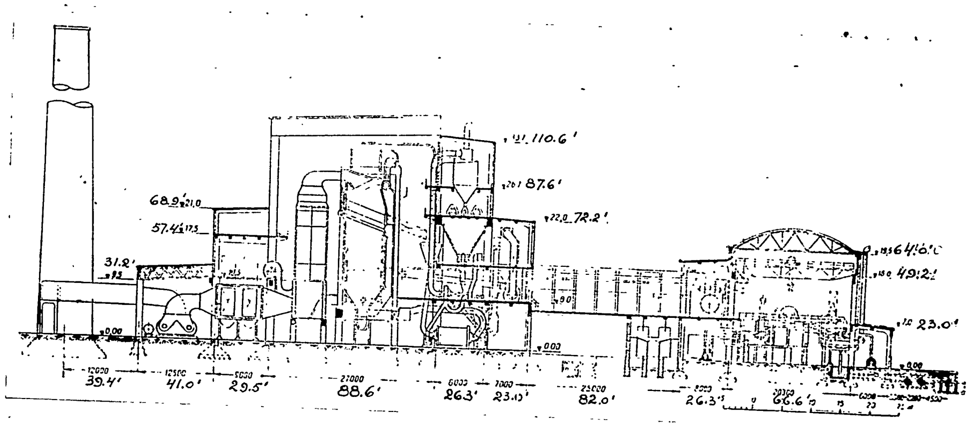
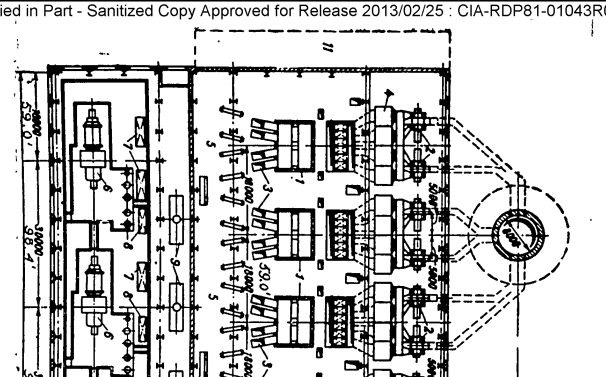


Type of "disjointed" design of station with low-set induced-draft fans and with the front of boilers facing the turbine hall.

ORSK PULVERIZED COAL TETs CROSS SECTION THROUGH THE MAIN PLANT BUILDING

Source: Weitz, B. Electric power Development in the USSR, 1936, TK.85. E6, Antipov, I. P. Arkhitektura elekrostantsiy, 1939, TH4581.A5.

PLATE 15.

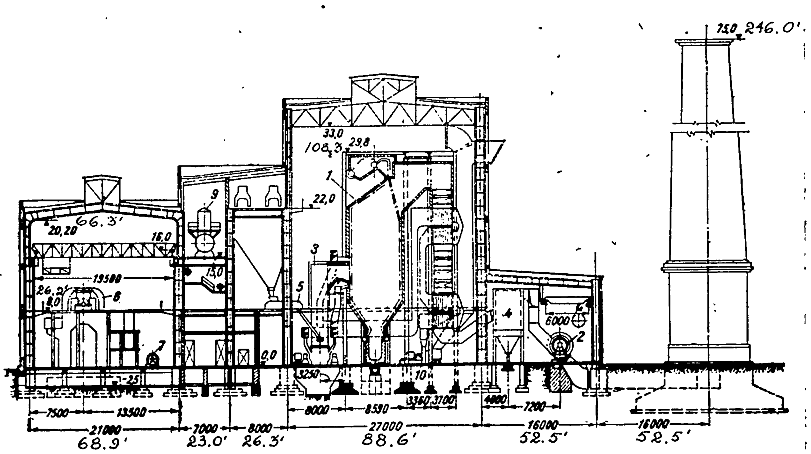


Plan of the main building.

CHEREPETS PULVERIZED-COAL HIGH PRESSURE GRES

Source: Elektrichevsikiye Stantsii, 1947, No. 5, p. 11 TK4.E725.

PLATE 16A.



Cross section of the main building.

1 - Boiler; 2 - Induced-draft fan; 3 - Shaft mill; 4 - Cyclones; 5 - Belt feeder; 6 - Turbogenerator; 7 - Electric feed pump; 8 - Turboine-driven centrifugal feed pump; 9 - Deaerator; 10 - Forced-draft fan; 11 - Staff accommodation.

CHEREPET' PULVERIZED-COAL HIGH-PRESSURE GRES

Source: Elektricheskiye Stantsii, 1947, No. 5, p. 11, TK4.E725.

PLATE 16.

CHAPTER II

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING

1. BUILDING MATERIALS

The principal building materials used in the construction of the main power plant building and the switch house are the following:

a. Reinforced Concrete - monolithic poured-in-place (with either bar ~~and~~ or rigid welded steel skeleton reinforcement) and precast (with bar ~~and~~ or pre-stressed wire reinforcement). Reinforced concrete is used for frames, columns, beams, girders, floors, foundation blocks, slabs and mats, roof supporting beams, wall panels, roof covering slabs, and stacks;

b. Steel - used for frames, columns, beams, girders, roof trusses, open steel grid floorings, window sash, steel plate stacks (when the stacks are built on top of the building);

c. Brick - used for curtain walls braced to structural steel or reinforced concrete frames, also for stacks.*

d. Wood - now used very seldom in any part of the power plant building. In earlier plant construction wood was used for roof trusses (later replaced by steel trusses) and roof sheathing (later replaced by reinforced concrete slabs). At present wood is till used for roof trusses, roof sheathing, window sash, etc. in office and staff accommodation buildings.

2. A. CONSTRUCTION TREATMENTS OF THE MAIN POWER PLANT BUILDING

The construction of the main power plant building has received different kinds of treatment, depending mainly upon the time when the plant was built and the type of the plant layout.

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* In older stations brick was used for wall-bearing construction, but now is very seldom used for the main power plant building.

Structural practice in the building of power plants has varied widely with time mainly in that the prevailing use at the outset of Soviet rule of poured-in-place reinforced concrete with bar reinforcing for the main supporting frames and elements of the building has been gradually shifting to the use of poured-in-place reinforced concrete with rigid welded steel skeleton reinforcement and precast reinforced concrete parts with reinforcement of bars and prestressed wire.

The type of the plant layout influenced its construction mainly because in the main power plant building the two-story wide-span and high-roof transverse frames of the boiler house and the turbine hall can be differently arranged with the multistory short-span frames of the bunker, smoke-eliminating and feed-water pump and tank sections, and with those different arrangements different parts of the building can serve as the principal structural supporting frames.

The first Soviet power plants (Shatura, Dubrovka, first part of the Gor'kiy plant) have special layout designs and are built in a different way from any others. The Shatura peat-firing plant (Plate 4, Fig. 2) had two and later three separate boiler houses with separate bunker sections built transversely to the turbine hall which connects them. Separate steel trestles for peat delivery are built for each boiler house. The main power plant building is therefore not one compact structure, but consists of four separate buildings (3 boiler houses and one connecting turbine hall). The buildings are constructed as separate reinforced concrete frames with brick curtain walls and steel roof trusses. The Dubrovka peat-firing power plant (Plate 5) has the turbine hall located between two boiler houses and its steel roof trusses are placed on brackets supported by the columns of the two adjoining boiler houses frames.

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These early power plant designs did not form any patterns to be followed in later built plants. The subsequent power plant layouts were of the four principal types described in Chapter One. Power plants built in the twenties and early thirties belong mostly to the First Type. In the second part of the thirties the design of the Second Type was introduced and became the prevailing pattern up to and after the war, when Type Four was designed. Type Three design did not become popular and there are few examples of this type of layout.

B. CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FIRST TYPE DESIGN

The main structural features are:

Structural types: poured-in-place reinforced concrete frames or steel frames (in cases when the load-bearing capacity of the soil was low and there was danger of foundation settlement, or when the construction job had to be completed in a short period of time).

Wall coverings: brick curtain walls.

Roof construction: Steel trusses, and reinforced concrete roof beams.

Roof coverings: wood sheathing boards, later changed to reinforced concrete slabs.

Floor construction: reinforced concrete slab, beam and girder construction and steel grids on steel stanchions.

Crane girders in the machine wall: steel, solid web.

Overhead bridge cranes: in the generator room.

Reinforced concrete columns of the outside wall of the generator room are built in four different ways;

1. Column of a rigid bent.
2. Steppod column.
3. Double column.
4. A-type column.

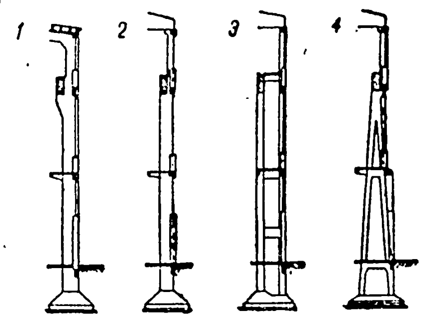

Fig. 5 - Reinforced concrete crane columns for generator room.

depending on the type of roof-supporting structures (steel trusses or reinforced concrete beams) and the load-lifting capacities of the over-head bridge cranes.

Foundations: poured-in-place reinforced concrete slab and mat footings (often on wood or concrete piling in lines or clusters) and separate monolithic reinforced concrete blocks for each boiler and each turbogenerator set. The reinforced concrete blocks for the turbogenerator sets have a special form in order to permit the installation of pipes and cables also the condenser. Typical reinforced concrete foundation block for a turbogenerator set of 50,000 kw. and 1,500 rpm. is shown on Plate 18a, Fig. 1. These foundations are set 4 m. (13 ft.) underground, and as a rule, are placed on clusters of wood or concrete piles.

Stacks: steel plate with an inside protective coating. They are placed on the roof of the main building extending 7-8 m. (23 - 26 ft.) above the roof top, 2.5 - 3.0 (8 -10 ft.) in diameter.

The Bay length: 5-6 m. (16-20 ft.) with the exception of the inner wall of the bunker section and the boiler room where the columns placed between boilers are spaced 15-18 m. (49-59 ft.)

Frames supporting the smoke-eliminating installation

In some buildings of the first Group design the smoke-exhaust flues, induced-draft fans and stacks are placed on a special reinforced concrete frame built in the boiler house. This special frame is supported on one side by the columns of the feed pump and tank section frame and on the other side by reinforced concrete columns erected between boilers.

In other stations of this first Group the smoke discharge installations are placed directly on top of the upper floor multi-story frame of the feed pump and tank section.

Construction of some main power plant buildings of the First Group.

The Ivanovo 100,000 kw. peat-firing GRES is shown in Plate 7. The trusses of the boiler house rest on the reinforced concrete frames of the bunker section on one side and of the turbine hall on the other side. Inside the boiler house space the 3-story poured-in-place reinforced concrete frame of the feed pump and tank section is built, and on the top floor of this frame are placed the induced-draft fans and the steel stacks. In the frame for the turbine hall and feed pump and tank sections the columns are on 16.4 ft. centers; in the bunker section frame on 49.2 ft. centers. The roof of the boiler house is supported by Warren-type cambered steel trusses of 86 ft. span. In the bunker section and the turbine hall the roof is on reinforced concrete beams. The roof covering, originally of wood sheathing, was later replaced by reinforced concrete slabs covered with ruberoid. The outside walls are brick, of the curtain type.

In the Stalingrad pulverized-coal GRES (Plate 10) the smoke-eliminating installations are placed on a special monolithic reinforced concrete frame in the boiler house space; it rests on reinforced concrete columns placed between the boilers at one end and on reinforced concrete columns of the feed pump and tank section frame at the other. On top of this special frame short columns are placed, which support one end of the Warren-type parallel-chord steel roof trusses. The other ends of these rest on the reinforced concrete three-story bunker frame. The special frame for the smoke-eliminating section adjoins a 5-story reinforced concrete frame of the feed pump and tank section, which forms a part of the reinforced concrete frame of the turbine hall. The roofs of the bunker, feed pump and tank sections and the turbine hall are supported by reinforced concrete beams. The roofing is of ruberoid laid on reinforced concrete slabs.

The Zuyevka 200,000 kw. pulverized-coal GRES (Plate 9 and 9a) has a construction very similar to the Stalingrad GRES.

In the Kuznetsk pulverized-coal TETs (Plate 11 and 11a) and in the Stalinogorsk pulverized-coal GRES (Plate 12 and 12a) the smoke-eliminating section is placed directly on the monolithic reinforced multi-story frame of the feed water pump and tank section. In each plant the steel roof trusses of the boiler house are supported by the reinforced concrete frame of the bunker section at one end and by that of the feed water pump and tank section at the other. In both stations the frames of the building are of poured-in-place reinforced concrete with reinforced concrete roof beams, except in the boiler house and in the turbine hall, where the roof is supported by steel trusses. The roofing is of ruberoid laid on reinforced concrete slabs.

The construction of other power plants belonging to the First Type built in the twenties and thirties is similar to the one described above, but with some minor differences.

A postwar design of the First Type, also has main supporting frames of poured-in-place reinforced concrete, not with ordinary bar reinforcing but with welded rigid steel skeletons (see Plates 17a). This method has the advantage of eliminating the necessity of scaffoldings for the entire building to support the forms for the poured-in-place concrete; used instead are small portable forms suspended from the rigid welded steel skeleton. This rigid steel skeleton is strong enough to carry its own weight and also the weight of the suspended forms filled with fresh wet concrete. When the concrete hardens in one section, the portable forms are transferred to the next. The rigid welded steel skeleton, embedded in the poured concrete, acts as its reinforcing. On Plate 17a the two frames of the bunker and the feed-water pump and tank sections are shown. The roof trusses of the boiler house are supported by those two frames. The turbine hall, located to the left of the feed-water pump and tank section, is not shown on this diagram. Induced-draft smoke fans are placed directly on top of the feed-water pump and tank reinforced concrete frame which also supports the steel plate stack 14 m. (46 ft.) high, 4 m. (13 ft.) in diameter and weighing 26 m. tons.

The details of the load-bearing skeleton reinforcement are shown on Plate 17b. The welded steel skeleton of a column is composed of four laced angles. Supplementary reinforcing bars are placed at the outer edges of the section. Special supporting angles are provided for the assembly of rigid frames which consist of girders, beams and braces. The connection of column sections is made with the principal four angles and with the supplementary bar reinforcements.

C. STACKS

Main power plant buildings of the First Type design can easily be recognized as they are the only ones ~~are~~ having stacks placed on the roof. The other three types have freestanding stacks outside the main building. Those stacks are of brick or reinforced concrete construction (Plate 19). They are located approximately 50 m. (164 ft.) from the main power house building, and are connected with it by 2-3 underground smoke flues. At the base of the stack the flues are brought to the surface and are connected with the stack shaft. The connection of the flue to the stack is reinforced with a cast iron ring 5 cm. (2 in.) thick.

At the bottom beside the flue connection two soot doors are provided. The stacks stand on reinforced concrete foundation mats supported by reinforced concrete piles. The height of stacks is determined by the fuel combustion draft requirements and also by the amount of sulphur in the smoke discharge. Some Soviet coals have a high sulphur content, and special smoke cleaning installations have been devised to eliminate injurious sulphuric gases from the smoke. Previously, without these cleaning installations, much higher stacks were needed than those built under current practice.

The brick masonry stack shown on Plate 19 Fig. 1 is 120 m. (394 ft.) high . Its foundation is a reinforced concrete mat supported by 252 reinforced concrete piles. Up to the 20 m. (66 ft.) level the stack is octagonal with an inside diameter of 9.95 m. (32 ft.) and an outside diameter of 13.20 m. (43 ft.). The shaft is straight vertical up to the 66 ft. level. Above 66 ft. the stack is round and conical with an outside diameter of 12 m. (39 ft.) at the bottom and 5.76 m. (18 ft.) at the top. The brick walls are 103 cm. (40 in.) thick at the bottom and gradually are reduced to a thickness of 0.38 m. (15 in.) at the top. Up to the 20 m. (66 ft.) level the shaft is lined with a fire brick lining 25 cm. (9.8 in.) thick, which is insulated from the structural brick masonry by a 10 cm. (4 in.) layer of tripoli. In the upper conical part of the shaft the fire brick lining is 12 cm. (48 in.) thick and the insulation tripoli layer is 5 cm. (2 in.) thick.

The monolithic reinforced concrete stack shown on Plate 19 Fig. 2 is 115 m. (377 ft.) high. It is divided into 7 sections, each section being of constant wall thickness decreasing from 1.05 m. (41.3 in.) in the bottom section to 0.8 m. (31.5 in.) in the top section. In order to protect the reinforced concrete from the corrosive effect of the sulphuric gases the stack has a fire brick lining 25 cm. (9.8 in.) thick in the first three lower sections and 12.5 cm. (4.9 in.) thick in the four upper sections. The outside diameter of the stack is 10 m. (32.8 ft.) at the base and 5 m. (16 ft.) at the top.

D. CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE SECOND TYPE DESIGN

Structurally the most important difference between the First and the Second Type design is the removal from the boiler house of the smoke-eliminating installations. The boiler house is located between the bunker section on one side and the feedwater pump and tank section on the other. These two multistory short-span frames and the outside wall of the turbine hall constitute the main load-bearing elements of the building. The columns of the bunker and of the feedwater pump sections adjacent to the boiler house have stepped extensions which support the roof trusses of the boiler house. The smoke flues at the back of boilers are laid on the ground floor of the bunker section, where the smoke-cleaning cyclones and induced-draft fans are also installed; sometimes they are placed outside the main building in a special small annex building which stands between the main building and the freestanding stacks.

Main structural features of the main building of the Second Type design are similar to those of the First Type design with some changes:

Structural type: reinforced concrete frames are built mostly of precast structural members or complete structural parts of the frame. After the War, in 1946, a new method for reinforced concrete frames was designed by the Teploelektroproyekt, namely monolithic reinforced concrete frames with rigid load-bearing welded steel skeletons as reinforcement. This method was also applied to some power plant buildings of the First Type design built after the War.

Plates 20A - 20E show the details of this kind of construction. The steel welded skeletons are prefabricated in sections of one or a group of structural members, forming a part of the frame. Those prefabricated parts weighing 3 to 5 m. tons (depending on the lifting capacity of the assembly crane), with wood or steel forms attached to them are then lifted and welded to each other in the process of building erection, and the concrete is poured into these forms. Sometimes the monolithic parts are combined with some precast structural members as shown on Plate 20E fig. 8. Besides reinforced concrete frames steel frames are also used. (See Plate 21a)

(See Plate 21). The main steel frames are shown in Fig. 1 with the section of principal structural members. Figs. 2 and 3 give two different treatments of principal frame connections, the rigid-frame type (Fig. 2) and the later type with hinged frame connections (Fig. 3).

At present (1957) the construction design of steel frames is based on the following principles:

1. The transverse combined frame is composed of main supporting rigid frames and adjoining hinged structural members.

2. The number of rigid joints is kept at a minimum and they are executed in the form of connections between columns and trussed cross girders.

3. The transfer of transverse stresses in joint connections is accomplished through supporting plates and the connection between continuous girders and columns is made by flange joints.

4. The solid web column and continuous girder sections are made in I-form composed of three built-up plates.

Wall coverings: brick curtain walls for the turbine hall outside wall are 51 cm. (20 in.) thick; for the bunker section outside wall, 38 cm. (15 in.) thick.

Roof construction: steel trusses and reinforced concrete beams.

Roof covering: reinforced concrete slabs.

Roofing: ruberoid or corrugated steel sheets. (When roof supporting construction is of steel trusses with steel purlins.).

Floor construction: reinforced concrete slab, beam and girder construction and steel grids on steel stanchions.

Overhead bridge cranes: in the turbine hall and often in the boiler house.

Crane girders: steel.

Reinforced concrete columns: for the outside wall of the turbine hall - as in First Type design (see p. 39.).

Foundations: the separate poured-in-place reinforced concrete foundation footings for each piece of auxiliary machinery has been replaced by a monolithic reinforced concrete basement floor mat on which all auxiliary machinery equipment and cables are laid. (See Plates 18B and 18C.) Separate poured-in-place reinforced concrete footings were still used for each column and for each boiler, and special reinforced concrete blocks, placed 4 m. (13 ft.) underground for each turbogenerator unit (see Plate 18A). In monolithic construction with rigid steel welded skeleton reinforcement the embedment of the column in its foundation footings is shown on Plate 20D, Fig. 5.

Stacks: are built outside the building as described above under C.

Examples of the Second Type design construction are: the power plants in Nesvetay (Plates 13 and 13A) with separate building for the smoke-eliminating section; and in Moscow, the Stalinsk TETs (Plate 14), where the smoke-eliminating installations (electric cyclone filters and induced-draft fans) are placed on the ground floor of the boiler house. The Nesvetay main power plant building, built in 1936-38, has steel frames, one for the bunker section, the other a combined transverse two-aisle frame for the feed-water pump section and the turbine hall with continoous two-span transverse girders. The steel roof trusses of the boiler house are supported by the stepped columns of the bunker section and of the feed-water pump section. The transverse frames of the machine hall and feed-water pump section are on 6 m. (20 ft.) centers and of the bunker section

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on 9 m. (30 ft.) centers. The smoke-eliminating section annex has transverse steel bents. The boiler house and the machine hall are equipped with overhead bridge cranes. The Kurakhovka GRES (47° 59' N, 37° 19' E) is similar to the power plant in Nesvetay.

The Stalinsk TETs in Moscow has reinforced concrete frames partly of precast structural members. The roof trusses in the turbine hall and the boiler house are of steel; the bunker and the feed-water pump sections have reinforced concrete roof beams. The boiler house has no overhead bridge crane. Of interest are the foundations of the boiler house, built as one monolithic reinforced concrete mat.

E. CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE THIRD TYPE DESIGN

Structurally the main difference between the Second and the Third Type design is the "disjointing" of the main power plant building into two separate buildings. One contains the boiler house, placed between the bunker section on one side and the smoke-eliminating section on the other. The other building contains the feedwater pump and tank section and the turbine hall; it is connected with the first building by an 80-100 ft. long gallery housing the switching installations. The main structural features do not differ from those described for the Second Type design. This type of construction has not been widely used, as it proved more expensive and required a wider area than other types. An example of this type if the Orsk TETs (plates 15 and 15A), built as reinforced concrete frames with reinforced concrete roof beams.

F. CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH TYPE DESIGN

This type is the latest in Soviet power plant construction. It was designed by the Institut Teploelektroproyekt in 1947. At that time also new regulations were issued under the title "Principle Regulations Governing the Design of Electric Power Plants, Substations, and Heat and Power Distributing Systems." Main articles of these regulations relating specifically to the structural part of the main power plant building construction are annexed at the end of this chapter.

The Fourth Type design has been prevalent since 1950, in the Fifth and Sixth Piatiletkas; some power plants of the Second Type design, however, are still being built. The layout of the Fourth Type design is shown on p. 12. The main structural features are similar to those of the Second Type design (see p. 45).

The Fourth Type layout has been designed in steel, in monolithic reinforced concrete with rigid welded steel skeleton reinforcement and in precast reinforced concrete with some prestressed-wire-reinforced structural members.

The steel frame construction is shown on Plates 22A and 22B. The transverse frames of the building are on Fig. 1 with the main structural members given. (Allowable steel stresses 1400 kg/cm^2 - 19,900 lb/in^2). Figure 2 shows the boiler house and the turbine hall as rigid steel frames. The steel columns between the bunker and the feed water pump section are connected with those two frames by hinged structural members. All steel columns are rigidly connected with their foundation footings. The weight of steel required by this design is given in the table on Plate 22B. Plate 22C shows the details of the principal joints.

Another design of the Fourth Type in steel is shown on Plates 23A and 23B. Here two alternatives are considered. In the first scheme the combined frame of the bunker and feedwater pump sections is made the principal rigid supporting element of the whole building, with the two outside walls of the turbine hall and the boiler room attached to this central frame by hinged structural members.

In the second scheme the two other frames of the building, namely of the boiler house and of the turbine hall are made rigid; they are connected by hinged structural members to form the middle frame of the bunker and the feed-water pump section. The second scheme is considered more advantageous and easier in assembly, having a smaller number of rigid joints.

The construction of the Fourth Type in monolithic reinforced concrete with rigid welded steel skeleton reinforcement is shown on Plates 24A and 24B. The transverse frames of the building are shown on Fig. 1.

For this medium-size power plant the weight of equipment on one square meter of floor space is calculated to be 1 m. ton (20.48 lb./ft^2). The floor heights are 12-18 m. (39-59 ft.). The load on a column in the lower floor sections is calculated to be 400-600 m. tons.

The combined two-aisle multistory frame of the bunker and feed-water pump sections is built as a monolithic reinforced concrete frame with rigid welded steel skeleton reinforcement. It consists of a lattice of steel angles and round bars welded together. The frame of the outside wall of the turbine hall is of reinforced concrete. The frame of the boiler house (except the inside wall up to the 25 m. (82 ft.) level) with the upper floor of the bunker section, and the structural members supporting the roofs of the turbine hall and boiler house are of steel.

The inter-story floors in the bunker and the feed-water pump sections are of beam-and-slab monolithic reinforced concrete construction.

Details of the steel skeleton reinforcement of the monolithic reinforced concrete frame of the bunker and the feed-water pump sections are shown on Plates 24A and 24B. In the process of construction the rigid welded steel skeleton supports only its own weight and the weight of forms which are attached to it and filled with poured-in-place wet concrete for the height of one story.

When the building is finished the frame members must carry the complete dead load of the building and also of the equipment. This combined load is much greater than that during construction: for girders and beams 2-3 times greater, for columns in lower story sections 8-10 times greater. Thus the original rigid welded steel skeleton is not sufficient, as the final reinforcement and additional reinforcing steel round bars must be added. The steel skeleton is mounted in prefabricated sections of steel members for one floor together with attached wood or steel forms as shown on Fig. 7.

All columns are rigidly embedded in their reinforced concrete stepped footings. The erection of the rigid steel skeleton for the bunker section is shown on Plate 24B, Fig. 8.

In 1956 the Institut Teploelektroproyekt designed a uniform construction scheme of the main power plant building of the Fourth Type layout in precast reinforced concrete. (Plates 25A - 25D)

On Plate 25A, Fig. 1 the cross-section through the building is shown with the transverse precast reinforced concrete frames, and sections of their principal structural members. The suggested dimensions of the plant make it possible to install turbogenerators of various capacities (25,000, 50,000 and 100,000 kw.) and boilers of 160 to 220 t./hr. steam producing capacities without changing the aisle span or the bay length of the structural frames. In case some additional units should be installed the building need only be extended lengthwise.

According to this design a TETs of 150,000 kw. capacity will have 26 bays each 6 m. (19.7 ft.) long making the total length of the building 156 m. (512 ft.). The two-aisle multistory rigid frame - Plate 25a, fig. 1 (rows B, C, D) of the bunker and feed water pump sections ensure the transverse stability of the building. The outside columns of the turbine hall (row A) and of the boiler house (row E) are connected to the central rigid frame by hinged roof beams. The longitudinal stability of the building is ensured by longitudinal girders connecting the transverse frames and columns in the outside rows. All the elements of the building superstructure (columns, girders, beams, floor and roof panels) are of precast reinforced concrete. Wall coverings are of reinforced foam concrete panels.

For columns, concrete "300" is used (with a compressive strength of 300 kg./cm^2 = 426 lb./in^2); for all other members concrete "200" is used (with a compressive strength of 200 kg./cm^2 = 284 lb./in^2.)

The cross sections of the columns have a uniform width of 0.6 m. (24 in.) and depths of 2, 1, 0.8 and 0.6 m. (78, 39, 31 and 24 in.).

For the inter-story floor covering a uniform type of panel is selected (Fig. 2). These panels have two longitudinal and five transverse ribs. The panels are 5.35, 5.55, 5.65, 5.97 m. (17.6, 18.2, 18.6, 19.6 ft.) long and 1.49 m. (4.9 ft.) wide. Rebates 60 x 70 mm. (2.36 x 2.76 in.) are provided along the longitudinal ribs of the panels for setting flat cover plates for floor openings. The openings between panels are obtained by moving apart the panels for the necessary distance.

The bunkers are designed in the form of two longitudinal partitions made up of precast reinforced concrete ribbed panels which are supported by beams resting on the cross girders of the frame. The transverse partition walls of the bunker also serve as cross-girders of the bunker section frame; on them rest the floor panels at floor level 23.00 m. (76 ft.). The bunker feed hoppers are of steel.

The roof beams of the turbine hall (span 27 m. = 88.6 ft.) of the boiler house (span 24 m. = 78.7 ft.) are of precast reinforced concrete with pre-stressed wire reinforcement. They support large reinforced concrete roof panels.

The precast structural elements are fabricated in sections weighing not more than 15 m. tons. At the building site the prefabricated sections of the columns are joined together to form sections weighing up to 40 m. tons, if they are to be assembled in places where the crane beam can lift such a load. Thus most columns are erected completely assembled, with the exception of the outside wall of the boiler house, where the assembly jib crane beam is most extended and can lift only shorter sections, which must be connected in the process of erection.

The column sections are connected on the ground by welding the protruding bars of their reinforcement and then pouring concrete over the joint to make it monolithic (Fig. 3). The connection of column sections in the process of erection (field joints) along row D is accomplished by welding the reinforcing bars of these sections, but without pouring concrete over the joint, because of the considerable height at which the work must be done (Fig. 4). These joints are provided with central steel plates for better transfer of vertical forces, and with steel side plates welded to steel hoops, to fasten the two connecting column elements so as to withstand any bending moment which might occur in the column joint (Fig. 4). The ends of

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column elements to be connected are reinforced with steel mesh which increases the strength of the concrete in the area of local compression. To align the two connecting column elements assembly bolts are provided. After adjustment, the side plates are welded and the assembly bolts and supporting angles are cut off. Then the joint is cemented. Such a column joint can withstand a vertical force of 250 m. tons and a moment of 200 ton-meters (1400 ft. kips). When the building has a basement and a floor above it at gound level 0.0 the joint of the column with the foundation is formed by embedding the column in a recess in the foundation footing (Fig. 5); the floor cover then rests on the top edge of the foundation footing.

To ensure the transfer of vertical loads to the foundation footing, the end of the column is provided with a pin, thus providing for filling the space under the base of the column with concrete.

The column is adjusted and fixed by means of wedges. The foundation recess is provided with an opening for cleaning and flushing. The vertical force transferred by such a column joint can reach 600 m. tons and the bending moment 50 m. ton meters (360 ft. kips).

When the building has no basement and no floor at gound level 0.0, the joint of the column with the foundation is formed by welding the projecting reinforcing bars of the foundation and the column (Fig. 6). While the column is being set up the vertical load is transferred through a steel pipe. The bending moments can be transferred by the reinforcing bars only after welding. The welding of the four corner reinforcing bars ensures the necessary stability to the column in the process of erection. After all reinforcing bars are welded and the joint poured with concrete, all the live load is transferred from the column to the foundation.

The joint of a cross girder with a column shown on fig. 7 is formed by welding the projecting upper and lower reinforcing bars of the two members with the aid of steel insertion rods. Welding is done by the submerged-arc method. Concrete is then poured into the joint to make it monolithic, in order to provide compressive strength. Wall coverings are of reinforced foam concrete panels (see fig. 8). On the outside the panels are covered with a 35 mm. (1.4 in.) rough-finished layer of heavy concrete "200". The foam concrete is of the mark 75 (with a compressive strength of 75 kg./cm^2 = 1066 lb./in^2.) and a volume weight of 900 kg./m^3 = 55 lb./ft^3.

The fastening of panels to the columns is flexible, with bolts, so that the panel load is not transmitted to the columns. The substructure of the main power plant building is usually built in the form of a great number of separate foundation slabs for various units of equipment, tunnels, channels, etc. placed at different levels. In this project, one basement floor is designed, under the turbine hall, bunker, and feed water pump sections (see plates 18B & 18C). Foundations for auxiliary equipment are placed in the basement at level 0.0. The basement floor is of flat precast reinforced concrete slabs with prestressed wire reinforcement 3 m. x 3 m. (10 ft. x 10 ft.) and 0.25 m. (10 in.) thick. The slabs rest at their corners on precast reinforced concrete columns with caps. This construction has the necessary height to accomodate all the auxiliary equipment.

Column foundation footings, supporting walls, and other structural elements in the substructure are of prestressed reinforced concrete. The foundation slab under the building, and the foundations blocks under the turbogenerator and boiler are of monolithic reinforced concrete. In special cases when ground water is high, the basêment floor is placed on a continuous monolithic ribbed reinforced concrete mat. The amount of reinforced concrete necessary for the new-design of basement floor is 1.2 m^3/m^2 (0.146 yd^3/ft^2) of

TABLE I

LIST OF PRECAST REINFORCED CONCRETE STRUCTURAL MEMBERS OF THE MAIN POWER PLANT BUILDING UNIFIED FOURTH TYPE LAYOUT DESIGN OF A TETS 150,000 KW CAPACITY.

Name of the structural member	Number of sizes of each type	Weight of each structural member m. tons	Number of units	Volume of reinforced concrete m^3	Volume of reinforced concrete yd^3
A. Superstructure	~~24~~	~~3.5 – 15.6~~	~~549~~	~~2,400~~	~~3,110~~
Columns	24	3.5 – 15.6	549	2,400	3,110
Cross girders	13	6.5 – 10.3	228	778	1,010
Roof supporting beams	2	17.5 – 21.0	60	462	600
Floor beams	6	0.5 – 3.4	721	540	700
Floor and roof panels	2	114 – 2.6	1943	1,579	2,020
Bunker members	5	1.1 – 6.2	360	566	740
Various members (flat panels monitors etc.)	–	———	—	375	490
Total				6,700	8,700
B. Substructure					
Column foundation footings and shoes	9	2.0 – 14.5	270	1,174	1,520
Columns	3	1.0 – 3.0	480	428	560
Concrete blocks	4	0.4 – 1.2	922	434	565
Supporting walls	1	1.7	86	146	190
Flat slabs and panels for floors and ceilings	6	0.07 – 5.6	1,046	1,138	1,480
Beams, ducts, passage tunnels cable support blocks	8	0.07 – 1.8	1,414	213	277
Various elements	–	–	–	87	117
Total				3,620	4,720
C. Reinforced foam concrete wall panels	21	0.31 – 2.3	1,930	2,480	3,230

Source: Stroitel'naya Promyshlennost', 1956, No. 6, page 24, 57 floor space as compared to the required volume of 1.35 m^3/m^2 (0.164 yd^3/ft^2) of reinforced concrete for the previous method of separate foundations for auxiliary equipment.

The list of precast reinforced concrete structural members for the construction of the main power plant building is given on table 1.

The total volume of reinforced concrete for the superstructure of the main building amounts to 7100 m^3 (250,000 ft^3) of which 95% or 6700 m^3 (236,000 ft^3) consists of precast members.

In the substructure (together with the foundations for the turbo-generators) the total volume of reinforced concrete amounts to 15,020 m^3 (530,000 ft^3), of which 24% or 3620 m^3 (128,000 ft^3) consists of precast members. The monolithic reinforced concrete in the substructure is used for the continuous foundation mat and for foundation blocks for the turbo-generators and boilers.

A photo showing a main power plant building in erection according to the above design is shown on Plate 25, fig. 9.

The latest regulations, issued in 1947, concerning the design of thermal power plants recommend instead of a building frame entirely in reinforced concrete (Plate 25e, fig. 2) a mixed type construction for the main building, (Plate 25e, fig. 1). The recommended scheme consists of precast reinforced concrete for the main structural frame and for its heavily loaded members and steel for lightly loaded members; for members placed at high levels, such as the upper part of the turbine hall wall above the crane girder, the upper floor frame of the bunker and feed water pump sections, the upper parts of the boiler house inside wall above the roof of the bunker section; and for crane girders.

TABLE 2

1947 STANDARD MIXED-TYPE CONSTRUCTION

COMPARISON OF QUANTITIES OF STEEL AND CONCRETE FOR 18 m. (59 ft.) OF BUILDING

Upper structural members of the building frame built either in steel or in reinforced concrete.	Mixed-type construction design (Table 25e, fig. 1). Weight of required steel.	All-reinforced-concrete construction design (Table 25e, fig. 2). Volume of required reinforced concrete.	
	m. tons	m^3	yd^3
Members of the upper part of the turbine hall outside wall (above crane girders).	5.94	6.3	8.2
Members of the upper floor frame of the bunker and feed-water pump section.	17.2	57.0	74.2
Members of the upper part of the boiler house inside wall (Above the roof of the bunker section).	8.7	8.4	11.0

Source: Elektricheskiye Stantsii, 1947, No. 7, page 10,

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The amount of material (steel or reinforced concrete) required for the construction of the upper structural members for 18 m. (59 ft.) length of building (width of one boiler unit) - see Plate 18a - according to the two alternative treatments (Plate 25e, fig. 1 and 2) is shown on table 2.

The total amount of material (steel and reinforced concrete) required for the construction of the entire frame for 18 m. (59 ft.) according to those two alternatives is:

	Alternative 1 (fig. 1)	Alternative 2 (fig. 2)
Reinforced concrete	451 m^3 (590 yd^3)	553 m^3 (720 yd^3)
Steel construction	107 m.tons	67 m.tons

Examples of the Fourth Type design

The 3 large newly built thermal power plants, the Mironovskaya 400,000 kw. coal-fired GRES near Artemovsk in Stalinskaya oblast', the Slavyanskaya GRES, and the Cherepet' GRES (projected capacity 600,000 kw.) are probably built according to the Fourth-Type layout design (see also plate 16 and 16a). There may be only small differences in their transverse dimensions; the length of the buildings will change with the number of installed boiler and turbo-generator units.

APPENDIX

"PRINCIPAL REGULATIONS GOVERNING THE DESIGN OF ELECTRIC POWER PLANTS, SUBSTATIONS, HEAT AND POWER DISTRIBUTING SYSTEMS" ISSUED BY THE MINISTRY OF ELECTRIC POWER STATIONS USSR IN 1947.

PARAGRAPHS RELATING TO THE CONSTRUCTION OF THE MAIN POWER PLANT BUILDING AND THE DISTRIBUTING INSTALLATIONS.

Construction of the Main Power Plant Building

317. The bay length in the main buildings should be uniform and a multiple of the width of the boiler unit.

318. Deformation joints are provided in the main building at the end of a boiler unit.

Settlement joints are provided in places where any structure adjoins the main building.

Deformation joints in the principal frame of the main building are built in the form of double columns.

319a. Stiffness and stability of the building are achieved by means of rigid frames, both transverse and longitudinal.

321. In the case where the skeleton of the main building is of steel, the construction is as follows:

rigid bents with columns rigidly connected with the foundations;

hinged members between the bents;

continuous welded members;

field joints on erection bolts.

322. In cases where the skeleton of the main building is of reinforced concrete, the structural members carrying heavy loads are of reinforced concrete. Steel is used for the lightly loaded elements, which for the most part carry their own load and that of the facing; steel is also used for long-span members subject to bending.

323. The crane girders in the turbine hall are of steel as a rule.

324. Bunkers, as a rule, are wholly of steel.

325. The boiler and engine room end walls (permanent and temporary) should be built on a steel frame.

326. A temporary end wall must be built so that the work in the part of the building which is to be expanded may proceed without dismantling of the wall; construction of an end wall should also be such that the wall can be moved to form a new end wall.

327. Concrete "140" (2,000 lb./in^2) should be used in the main reinforced concrete construction of the main building.

328. Steel "3" (23,000 lb./in^2) is to be used for the steel construction of the main building skeleton.

329. The following construction is adopted for the load-bearing roof members of the engine and boiler room in both reinforced concrete and steel structures with 6-7 m. (19.7 - 23.0 ft.) beys:

a) with span up to 24 m. (78.7 ft.): steel frames with solid web beams;

b) with span over 24 m (78.7 ft.): frames with latticed beams or roof trusses.

330. The inside walls separating the boiler room from other compartments (engine room, service quarters, bunker gallery) are made of fireproof material; the thickness of the walls is:

a) not less than 25 cm. (9.8 in.) for brick walls;

b) not less than 20 cm. (7.9 in.) for walls of block material (ceramic blocks, slag-concrete blocks, etc.).

331. The wall covering for the main building frame is to be of fireproof and frost- and humidity-resisting construction.

332. The outside walls (brick construction) of the boiler room above the boiler servicing level are to be half a brick (one brick, American notation) in thickness for any climatic conditions.

333. Walls thicker than one and a half brick (three bricks American) (in steel frame constructions) should be self-supporting.

334. The use of reinforced concrete walls and partitions is not allowed as a rule.

335. Compartment partitions of plant-service distributing installations with small-size oil or air circuit breakers consist of steel skeleton and gypsum or asbestos-cement panels.

336. Roof covering load-bearing members are of fire-resistant or semi-fire resistant materials.

337. The roofing of the main building consists of three layers: a layer of ruberoid over two layers of artificial parchment paper with mastic adhesive.

338. A special basement is provided under the engine room and the deaerator compartment to accommodate the underground communications.

Auxiliary equipment and platform supports are installed directly on the reinforced concrete floor above the basement.

339. The underground communication lines of the engine room are laid in ducts and tunnels in cases where the ground water level is high.

340. The boiler room underground communication lines are laid as a rule in ducts and channels.

341. The foundations of the turbogenerator units are of reinforced concrete (for any unit capacity), or steel (for unit capacity up to 25,000 kw.).

342. The boiler foundations are of reinforced concrete in the substructure and of steel in the superstructure; this is achieved by extending the steel skeleton to the ash compartment floor or by building individual steel foundations.

343. Ball mill foundations are built, as a rule, as separate reinforced concrete bases - (under the bearings, the electric motor and the reduction gear) which are located on the solid foundation slab of the bunker section.

344. Induced-draft fan foundations and the foundations for similar large size auxiliary equipment are built of lightly reinforced concrete on undisturbed ground.

345. Floors of service quarters and ceilings are of reinforced concrete or wood; floors and ceilings in toilets, showers and washrooms, laboratories, workshops, and archives are made of reinforced concrete exclusively. The partitions in the above-mentioned rooms are of brick, half a brick (one brick American notation) in thickness.

Main Distributing Installations and Switch Panel

346. Whenever the control panel and main distribution installation are located in a structure outside the main building, a heated passage should be provided to connect these two buildings.

347. The building housing the main distribution installation with small size or non-oil circuit breakers is designed to have load-bearing walls, and floors and ceilings; the partitions are not included in the number of load-bearing structural elements of the building. Partitions of compartments are assembled from precast gypsum or similar panels, which are set in steel frames. Light steel frames joined to the main skeleton of the building are provided for the panel erection.

348. Low walls and racks in bus-bar construction are also made of gypsum or similar panels which are fastened to a frame.

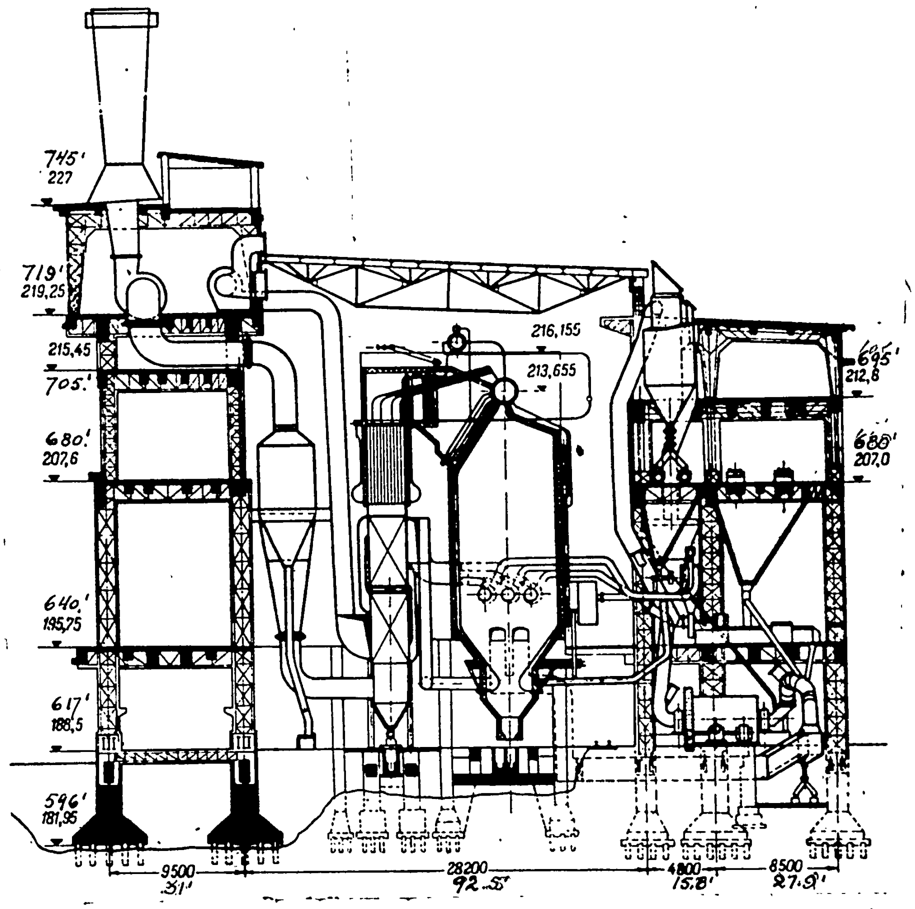
349. The thickness of outside brick walls of the main distribution installation are not to exceed 38 cm. (15 in.) irrespective of the climatic conditions.

350. Floors, ceilings and roofs are to be of reinforced concrete laid on steel beams.

351. The roofing of the main distribution installation and that of the control panel should consist of three layers: a layer of ruberoid over two layers of artificial parchment paper with mastic adhesive.

352. No windows are provided in the building of the main distribution installation. Natural lighting is to be provided in the control panel building. Window casings and sash are to be of wood.

(Source: Elektricheskiye Stantsii, 1947).



Monolithic reinforced concrete frame construction with rigid welded steel skeleton reinforcement. The turbine hall is to the right of the feed-water pump and tank section (not shown on this diagram)

SECTION THROUGH THE MAIN POWER PLANT BUILDING OF THE FIRST-TYPE DESIGN

Source: Elektricheskiye Stantsii, 1949, #1, p. 28, TK4.E725

PLATE 17A.

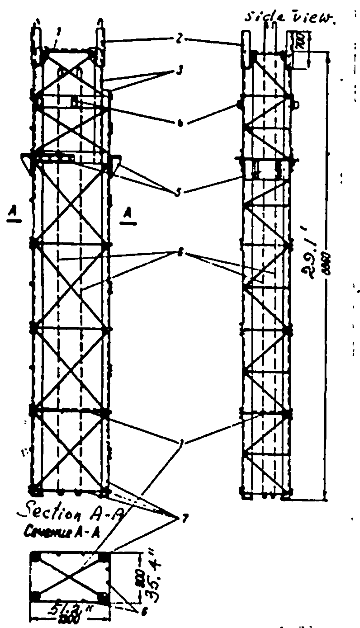


Fig. 1 - Legend

1. Stiffening Cross bars.
2. Joint connecting angles of the load-bearing reinforcement.
3. Reduction of the column to a smaller cross section.
4. Plates for adjusting connecting lattices.
5. Erection supports.
6. Bar reinforcement.
7. Rigid welded steel skeleton reinforcement.

Fig. 1 - Rigid Welded Steel Skeleton Reinforcement of a Column.

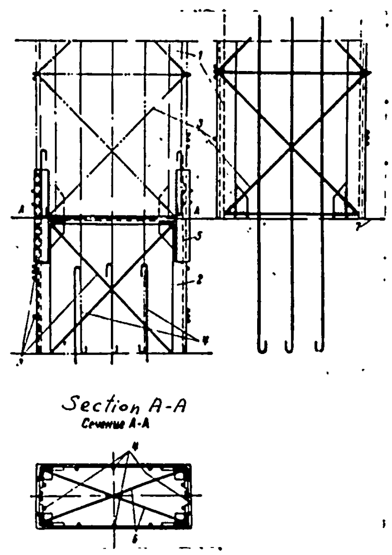


Fig. 2 - Details of a Joint of the Column Sections

1. Upper column section.
2. Lower column section.
3. Reinforcement of the skeleton lattice of the column.
4. Bar reinforcement of the column.
5. Joint angle.
6. Stiffening cross bars of the column.

Fig. 3 - Legend

1. Bar reinforcement of the beam.
2. Bar reinforcement of the beam-column joint.
3. Rigid welded steel skeleton reinforcement of the beam.
4. Plates for adjusting connecting lattices.
5. Erection suppo rts.
6. Erection angles.

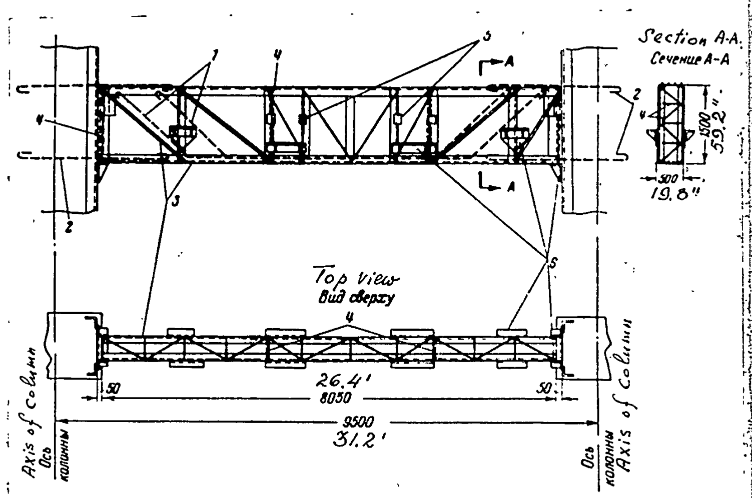


Fig. 3 - Rigid Welded Steel Reinforcement of a Cross Girder.

DETAILS OF THE RIGID WELDED STEEL SKELETON REINFORCEMENT FOR REINFORCED CONCRETE FRAMES SHOWN ON PLATE 17A.

Source: Elektricheskiye Stantsii, 1948, #6, p. 14, TK4.E725

PLATE 17B.

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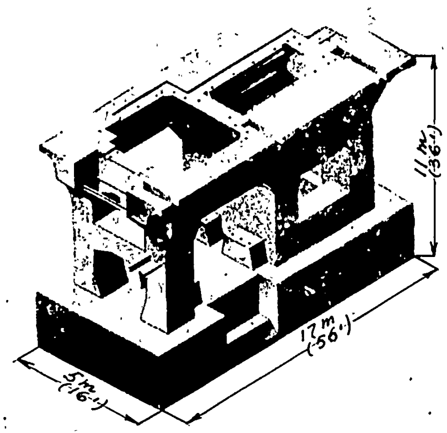


Fig. 1

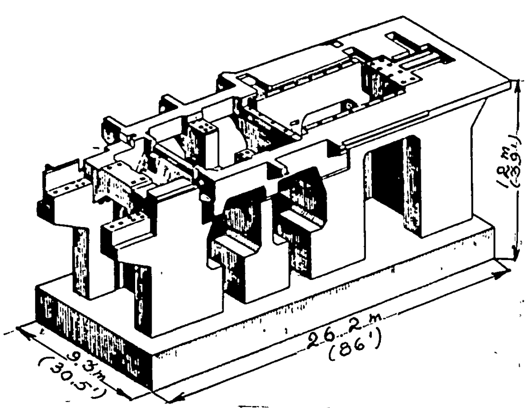
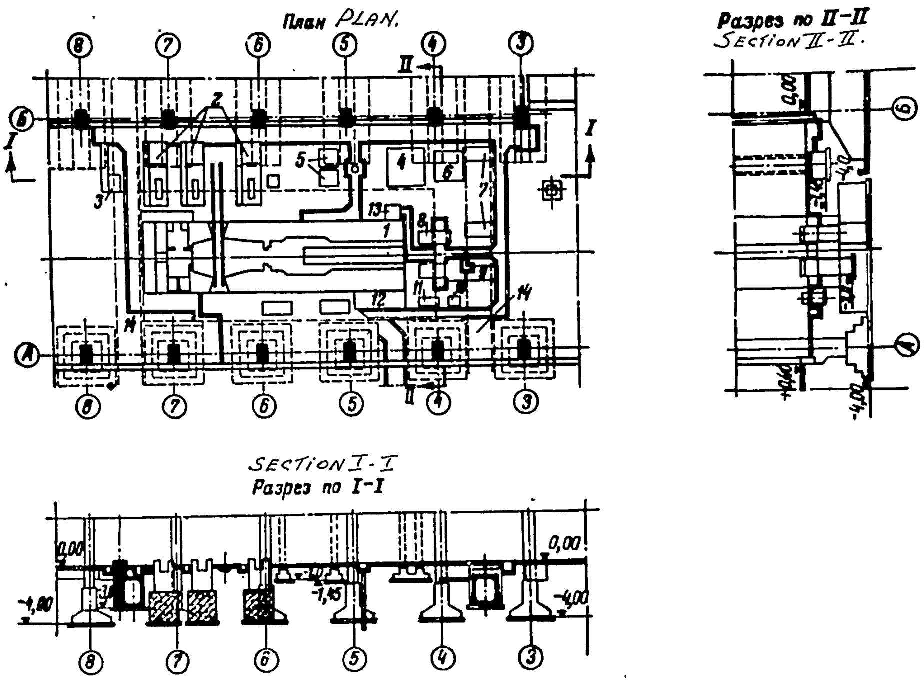


Fig. 2

Monolithic reinforced concrete foundations of turbogenerator units.

Source: Antipov, I. P. Arkhitektura elektrostantsiy, 1939. (TH 4581.A5)
Elektricheskiye stantsii, 1954, No. 9, p. 22 (TK4.E725).

PLATE 18A.



Legend

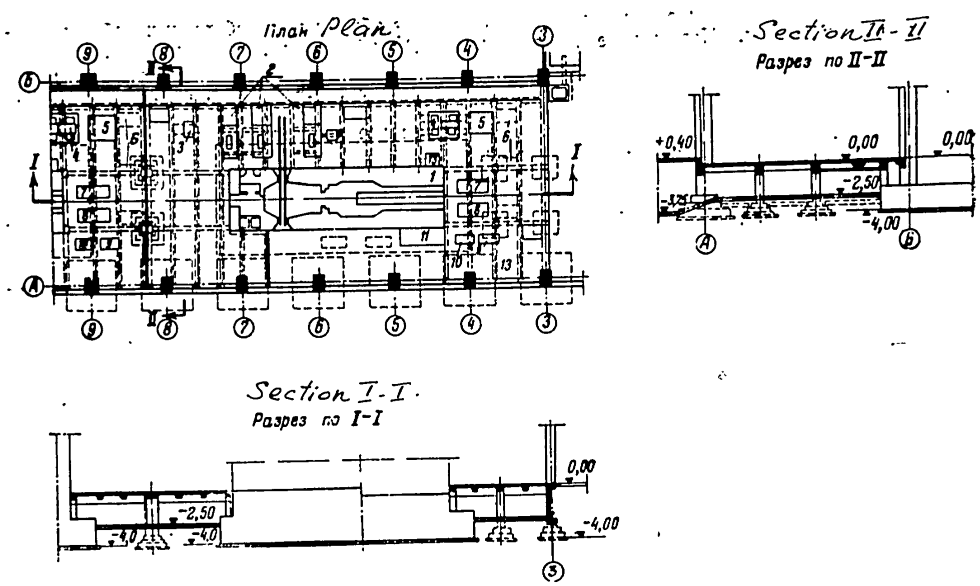
1. Foundation under the turbogenerator;
2. Foundations under the electric feed pumps;
3. Foundation under the turbine feed pump;
4. Foundation under the overflow pump and plantforms;
5. Foundation under the platforms;
6. Foundation under the separator;
7. Foundations under the platforms;
8. Foundation under the electric condenser pump;
9. Foundation under the condenser turbo-pump;
10. Foundation under the lubricating turbo-pump;
11. Foundation under the lubricating turbo-pump;
12. Foundation under the lubrication-oil cooler;
13. Foundation under the overflow tank;
14. Cable tunnel.

Foundation of a turbogenerator unit and of its auxiliary equipment underneath the machine hall.

TYPE OF CONSTRUCTION WITH INDIVIDUAL FOUNDATIONS.

Source: Elektricheskiye stantsii, 1947, #7, p. 11

PLATE 18B.



<u>Legend</u>

1. Foundation under the turbogenerator;
2. Electric feed pumps;
3. Turbine feed pump;
4. Overflow pump and platforms;
5. Separator;
6. Posts under platforms;
7. Electric condensing pump;
8. Condensing turbo-pump;
9. Lubricating oil turbo-pump;
10. Electric lubricating oil pump;
11. Lubricating oil cooler;
12. Overflow tank.

Foundations of a turbogenerator unit and of its auxiliary equipment underneath the machine hall.

TYPE OF CONSTRUCTION WITH A BASEMENT FLOOR PLACED ON A MONOLITHIC REINFORCED CONCRETE MAT.

Source: Elektricheskiye stantsii, 1947, No. 7, p. 12.

<u>PLATE 18C.</u>

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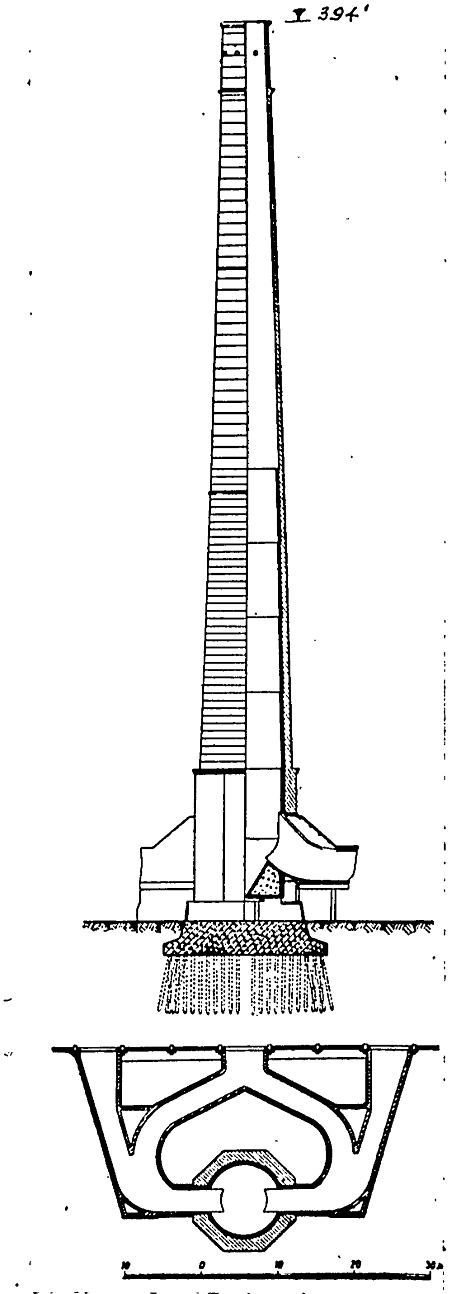


Fig. 1 - Brick Stack.

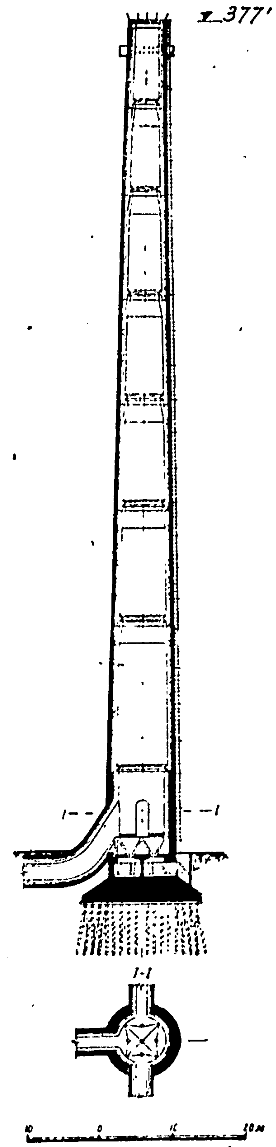
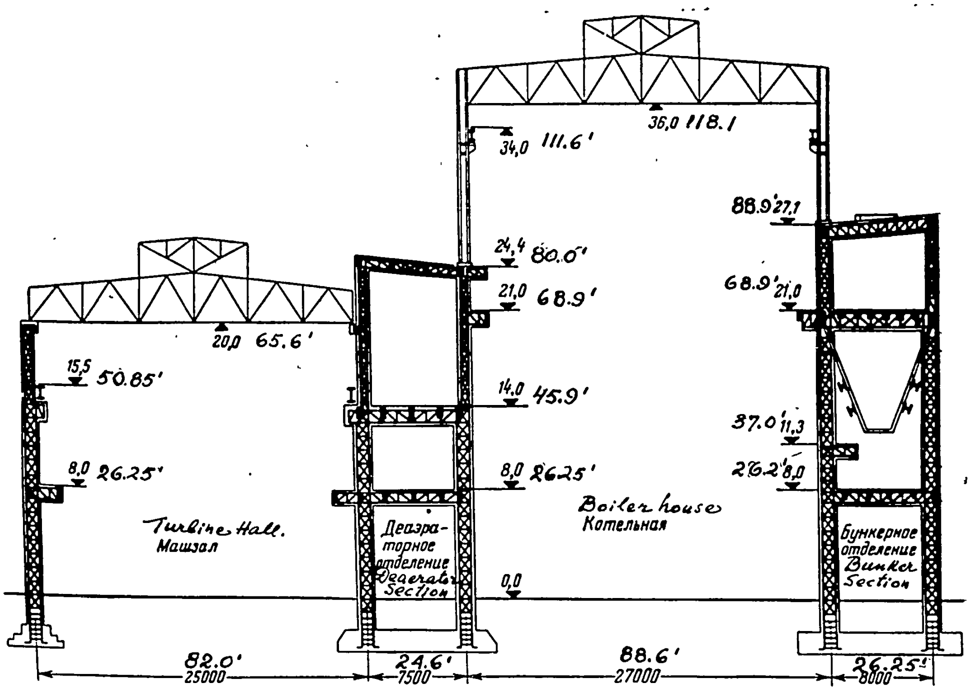


Fig. 2. 1. Reinforced Concrete Stack.

STACKS

Source: Antipov, I. P. Akhitektura elektrostantsii, 1939 (TH.4581.A5)



The separate frames of the bunker, the deaerator sections and the outside wall of the turbine hall have rigid steel welded skeleton reinforcement.

THE LOAD BEARING MONOLITHIC REINFORCED CONCRETE TRANSVERSE FRAMES OF THE MAIN POWER PLANT BUILDING OF THE SECOND-TYPE DESIGN

Source: **Elektricheskiye Stantsii, 1952, #2, p. 25.** (TK4.E725)

PLATE 20A.

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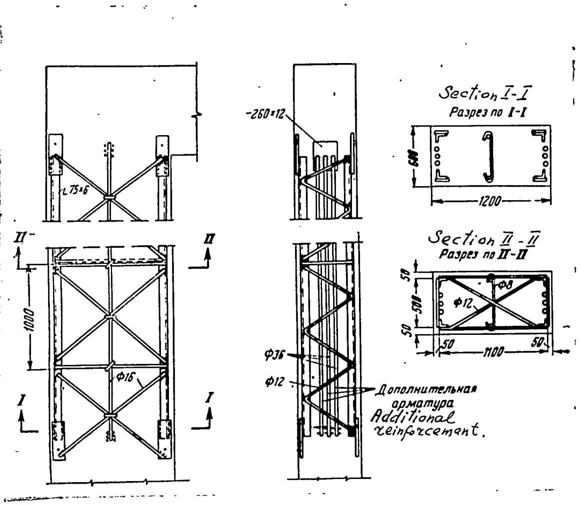


Fig. 1 - Reinforcing skeleton of the column.

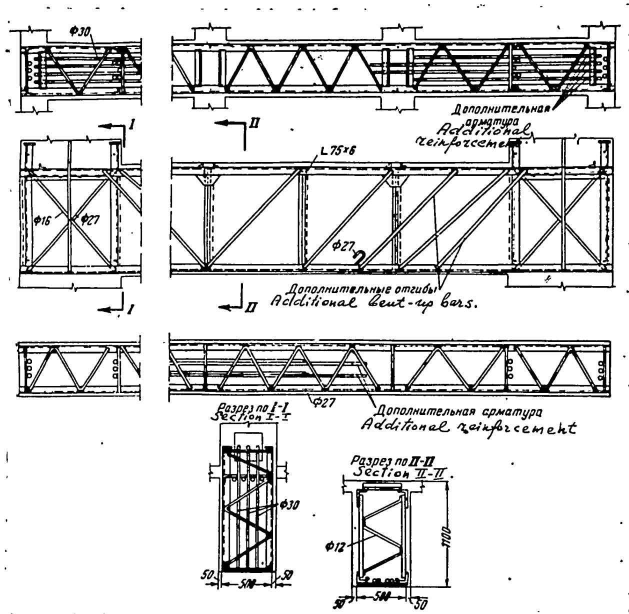


Fig. 2 - Reinforcing skeleton of the cross girder

DETAILS OF THE RIGID STEEL WELDED SKELETON REINFORCEMENT
OF THE MONOLITHIC REINFORCED CONCRETE FRAMES SHOWN ON PLATE 20A.

Source: Elektricheskiye Stantsii, 1952, #2 p. 23-26.

PLATE 20B.

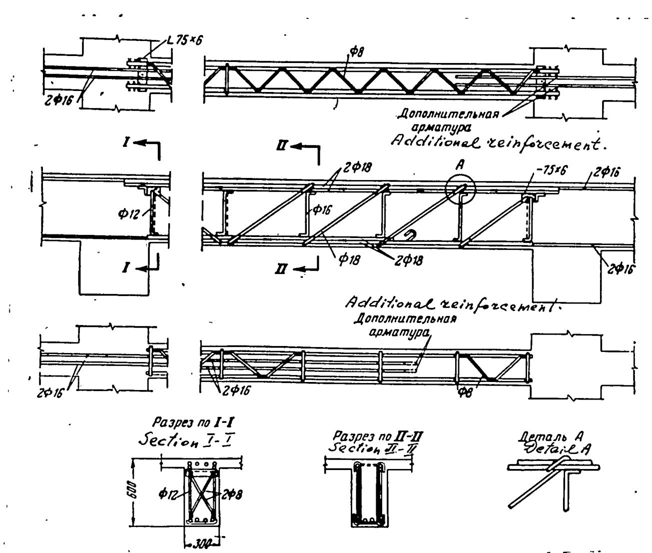


Fig. 3 - Reinforcing skeleton of the beam

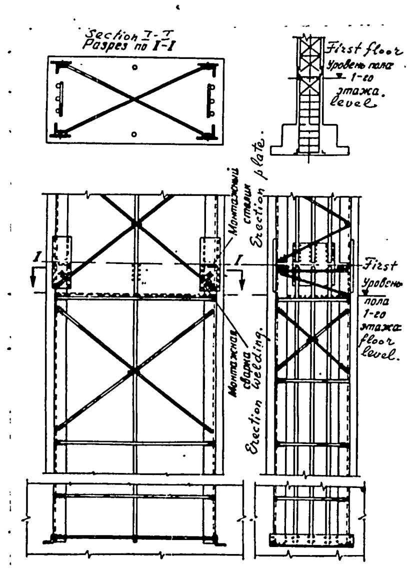


Fig. 4 - Connection of the skeletons of the column and the foundation footing at the level of the floor of the first story.

DETAILS OF THE RIGID STEEL WELDED REINFORCEMENT
OF THE MONOLITHIC REINFORCED CONCRETE FRAMES SHOWN ON PLATE 20A.

Source: Elektricheskiye Stantsii, 1952 No. 2, p. 24

PLATE 20C.

Declassified in Part - Sanitized Copy Approved for Release 2013/02/25 : CIA-RDP81-01043R001600080008-4

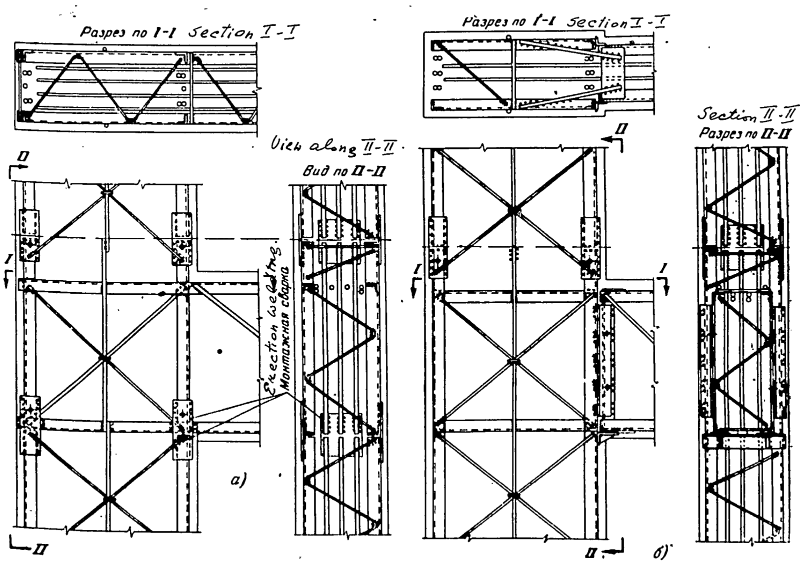


Fig. 6 — Connection of the skeletons of the column and the cross girder.

a. When the girder crosses the column
b. When the girder ends at the column.

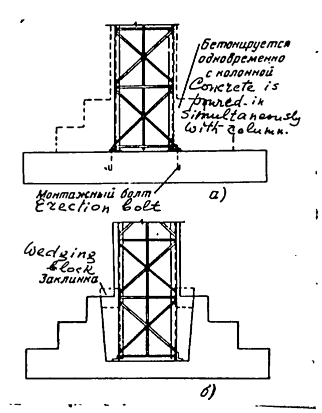


Fig. 5 — Embedment of the column-skeleton in the foundation footing.

a. Attachment of the skeleton to the foundation plate
b. Embedment of the skeleton in the foundation well.

DETAILS OF THE RIGID STEEL WELDED REINFORCEMENT OF THE MONOLITHICS REINFORCED CONCRETE FRAMES SHOWN ON PLATE 20A.

Source: **Elektricheskiye Stantsii, 1952, #2, p. 25**

PLATE 20D.

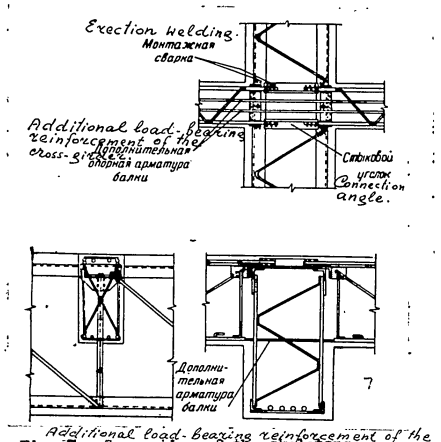


Fig. 7 – Connection of the skeletons of the girder to the beam.

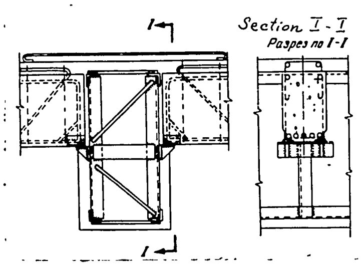


Fig. 8 — Connection of the precast reinforced concrete beam to the skeleton of the transverse girder.

Fig. 9 - Legend

Fastenings of the form to the skeleton

a. Construction of the fastening of plank forms for the beams.
b. Construction of the fastening of plank forms for the girders.
c. Construction of the fastening of plank forms for the column.

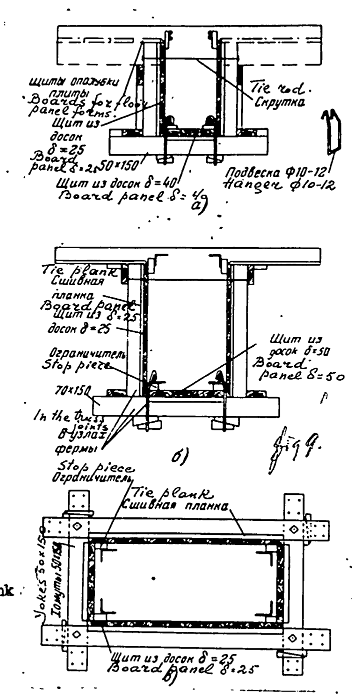


Fig. 9

-75-

DETAILS OF THE RIGID STEEL WELDED REINFORCEMENT OF THE MONOLITH REINFORCED CONCRETE FRAMES SHOWN ON PLATE 20A
Source: Elektricheskiye Stantsii, 1952, #2, p. 26

PLATE 20E.

Weight of Steel: for 18 m. (59 ft.) of building length (boiler unit).
(Allowable Steel stresses 1400 kg/cm^2 (19,900 lbs/in^2)

	Scheme No. 1			Scheme No. 2		
	Total Weight m tons	Weight of steel kg/m^3	Weight of steel lb/ft^3	Total Weight m tons	Weight of steel kg/m^3	Weight of steel lb/ft^3
Roof-supporting members	60.5	1.81	0.112	87.1	3.87	0.240
Columns	172.2	5.20	0.323	137.6	6.04	0.374
Crane girders	16.8	0.4	0.0249	9.8	0.43	0.0266
Inter-story floor-supporting members of the feed water pump section	40.5	1.20	0.0745	33.7	1.70	0.1055
Inter-story floor supporting members of the bunker section	58.5	1.76	0.109	38.16	1.69	0.1049
Bunkers	41.3	1.24	0.077	29.3	1.30	0.0805
Longitudinal frame members	40.0	1.20	0.0745	28.44	1.27	0.0788
Total	429.3	12.9	0.800	369.0	16.3	1.01

mm.	in.
100/12	= 3.94/0.472
120/16	= 4.72/0.630
120/18	= 4.72/0.709
150/10	= 5.91/0.394
300/14	= 11.8/0.551
400/16	= 15.8/0.630
480/14	= 18.9/0.551
500/12	= 19.7/0.472
500/16	= 19.7/0.630
500/18	= 19.7/0.709
900/12	= 35.4/0.472
1100/10	= 43.3/0.394
1100/12	= 43.3/0.472

Conversion figures for steel sections on Plate 21A, Fig. 1

PLATE 21B.

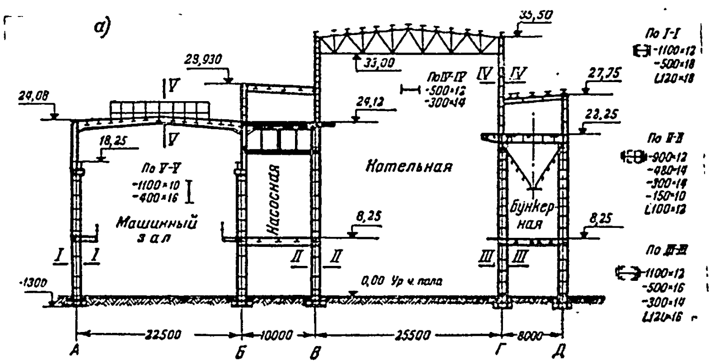


Fig. 1. Transverse steel frames of the main power plant building with steel sections for a rigid-frame connection design (as per scheme 1 - fig. 2).

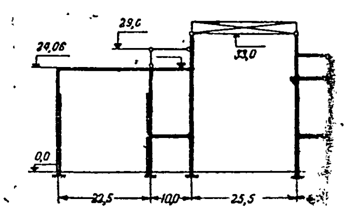


Fig. 2. Scheme 1 - The multi-story four-aisle combined transverse steel frame is rigidly connected.

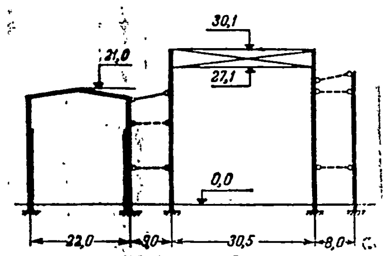


Fig. 3. Scheme 2. The steel rigid frames of the boiler house, of the turbine hall and the outside wall of the bunker section are interconnected by hinged steel members.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE

SECOND-TYPE DESIGN IN STEEL.

Source: Elekricheskiye Stantsii, 1948, #9.

PLATE 21A

Weight of Steel: for 18 m(59') bldg. length(boiler unit)
(Allowable steel stresses 1400 kg/cm^2 - 19,900 lb/in^2 -)

	Total Weight m tons	Weight of steel	
		kg/m^3	lb/ft^3
Roof-supporting members	81.8	3.10	0.1920
Columns	121.6	4.61	0.2860
Crane Girders	8.5	0.32	0.01984
Inter-story floor-supporting members of the feed water pump section	17.4	0.66	0.0410
Inter-story floor supporting members of the bunker section	65.6	2.49	0.1545
Bunkers	35.0	1.32	0.0820
Longitudinal frame members	29.5	1.10	0.0682
Total	359.4	13.6	0.8450

⅃L 75/8 = 2.95/0.315
⅃L 150/12 = 5.91/0.472
⅃L 150/16 = 5.91/0.630

100/10 = 3.94/0.394
300/20 = 11.8/0.787
350/12 = 13.8/0.472
350/16 = 13.8/0.630
350/30 = 13.8/1.18
400/20 = 15.8/0.787
500/10 = 19.7/0.394
500/18 = 19.7/0.709

500/20 = 19.7/0.787
500/30 = 19.7/1.18
750/8 = 29.5/0.315
750/10 = 29.5/0.394
750/12 = 29.5/0.472
750/14 = 29.5/0.551
1000/10 = 39.4/0.394
1000/14 = 39.4/0.551
1000/20 = 39.4/0.787
1200/12 = 47.2/0.472
1800/14 = 70.9/0.551

Conversion figures on steel sections on Plate 22A, Fig. 1

PLATE 22B.

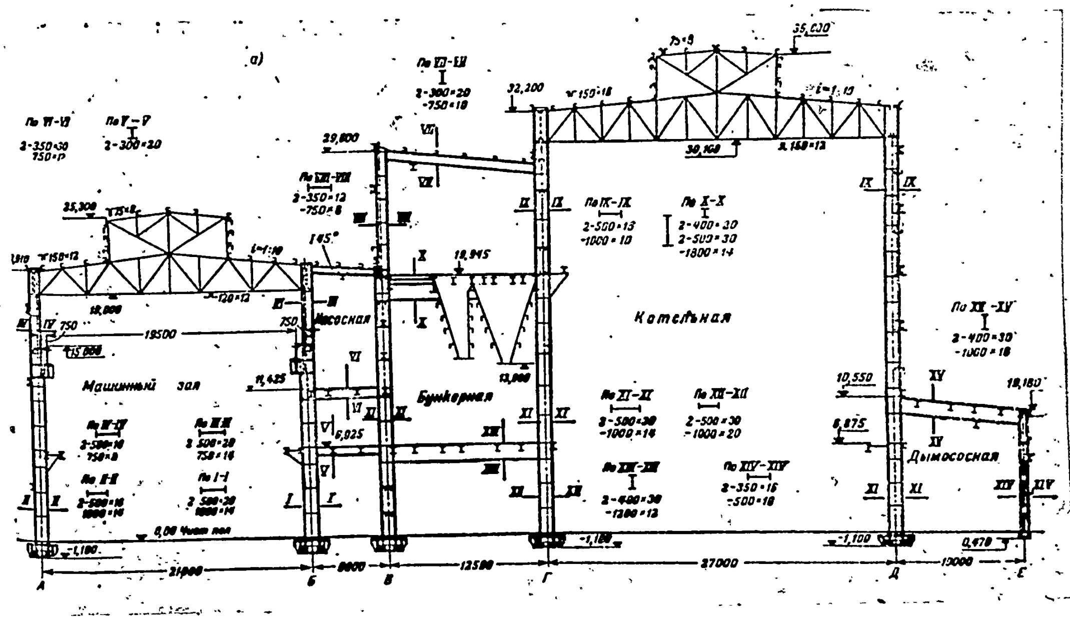


Fig. 1 – Transverse steel framesof the main power plant building.

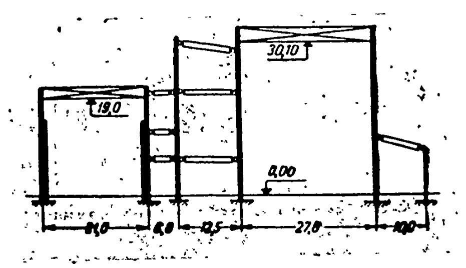


Fig. 2 – Construction scheme of the main building steel frame connections.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH-TYPE DESIGN IN STEEL

Source: Elektricheskiye stantsii, 1948, No. 9 (TK4.E725).

PLATE 22A.

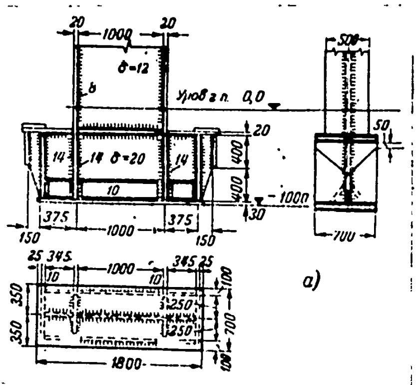


Fig. 1 – Base of a column

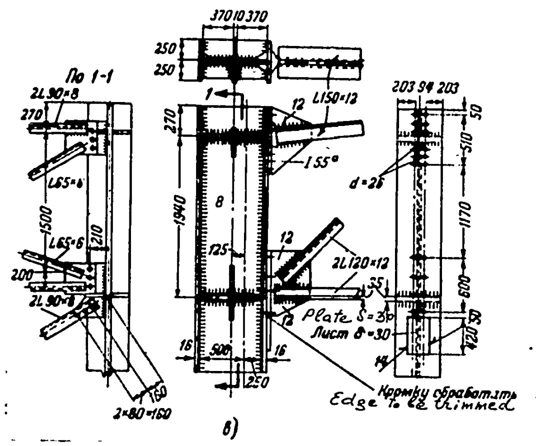


Fig. 3 – Connection between truss members and a column.

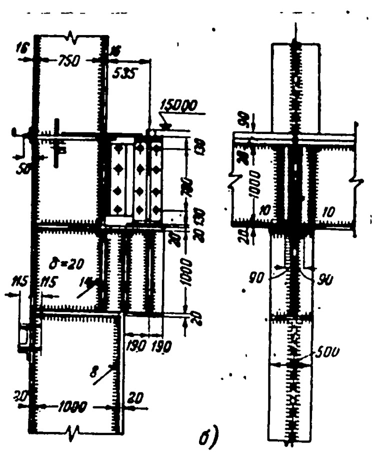


Fig. 2 – Connection of a crane girder and a frame girder to a column.

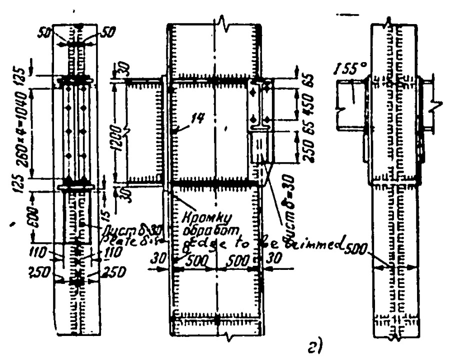


Fig. 4 Connection between a column, a girder and a beam

DETAILS OF THE PRINCIPAL JOINTS OF THE STEEL FRAME IN THE MAIN POWER PLANT BUILDING

Source: Elektricheskiye Stantsii, 1948, #9 (TK4.E725)

PLATE 22C.

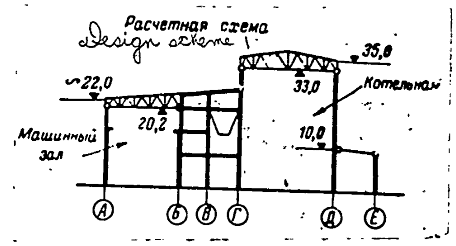


Fig. 1 - Design scheme 1

Scheme I

The combined rigid steel frame of the bunker and feed-water pump section is connected with the outside walls of the turbine hall and the boiler house by hinged structural members

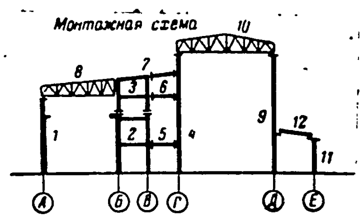


Fig. 2 - Erection scheme 1

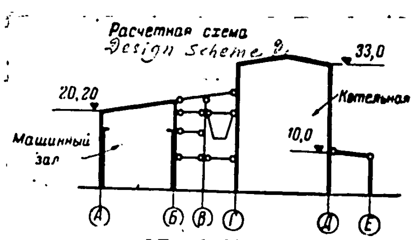


Fig. 3 - Design scheme 2

The rigid frames of the boiler house and the turbine hall are connected by hinged structural members formin the middle hinged combined frame of the bunker and feed-water pump stations.

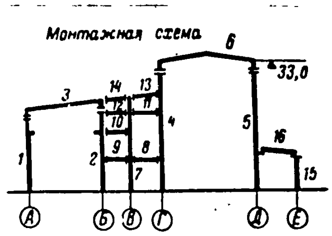


Fig. 4 - Erection scheme 2

PLATE 23B.

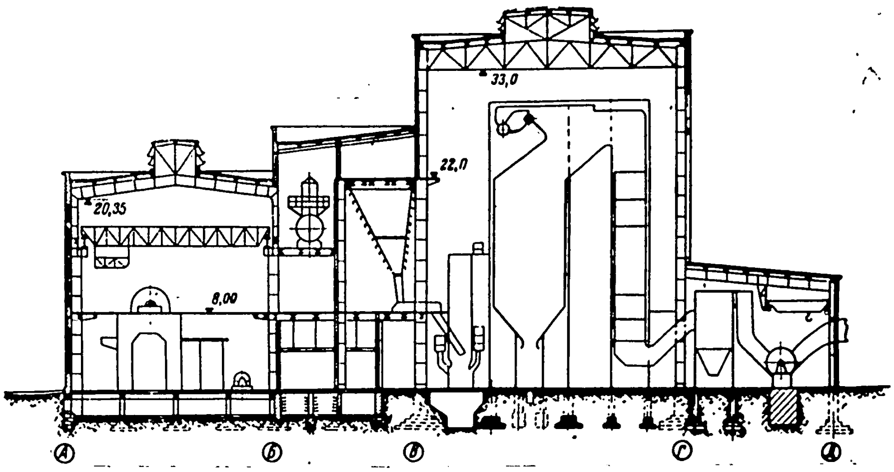


Fig. 1 - Main transverse steel frames of the building.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH-TYPE DESIGN IN STEEL

Source: Elektricheskiye Stantsii, 1947, #7.

PLATE 23A.

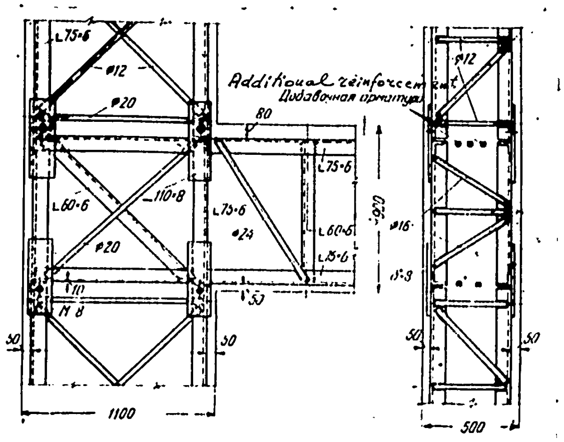


Fig. 5 - Connection between the steel welded skeletons of a cross girder and a column.

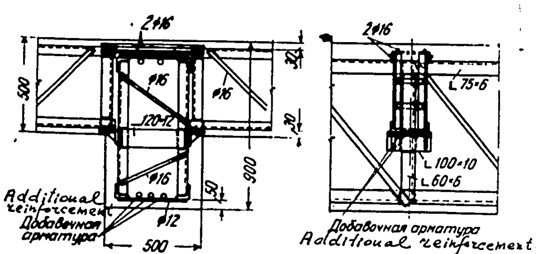


Fig. 6 - Connection between the steel welded skeleton of a girder and a beam.

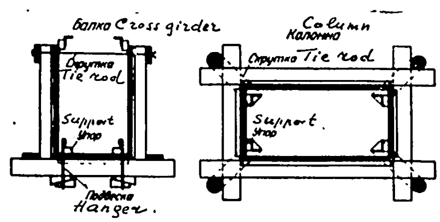


Fig. 7 - Fastening of the concrete forms to the welded steel skeleton of a girder and of a column.

Fig. 8 - Assembly of welded steel reinforcing skeleton parts for the bunker section frame by means of a swing jib crane.

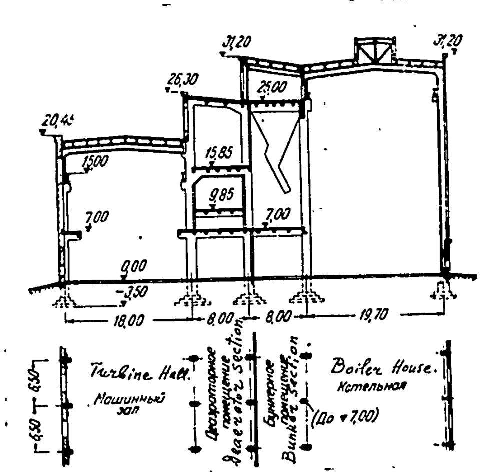


Fig. 1 - Principal transverse frames and partial plant showing the bay lengths of the main power plant building.

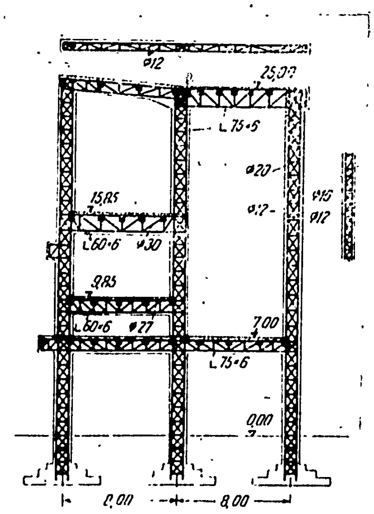


Fig. 2 - Welded rigid steel skeleton of the monolithic frame of the bunker and feed-water tank sections.

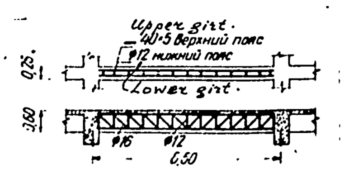


Fig. 3 - Rigid steel welded skeleton of the floor supporting beam.

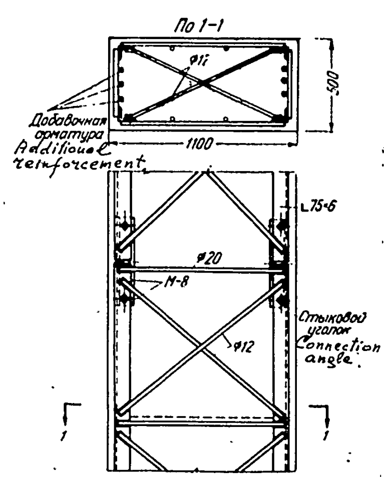


Fig. 4 - Joint of the welded steel skeleton of a column.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING IN MONOLITHIC REINFORCED CONCRETE WITH WELDED RIGID STEEL SKELETON REINFORCEMENT

Source: Stroitel'naya promyshlennost', 1950, No. 4.

PLATE 24A

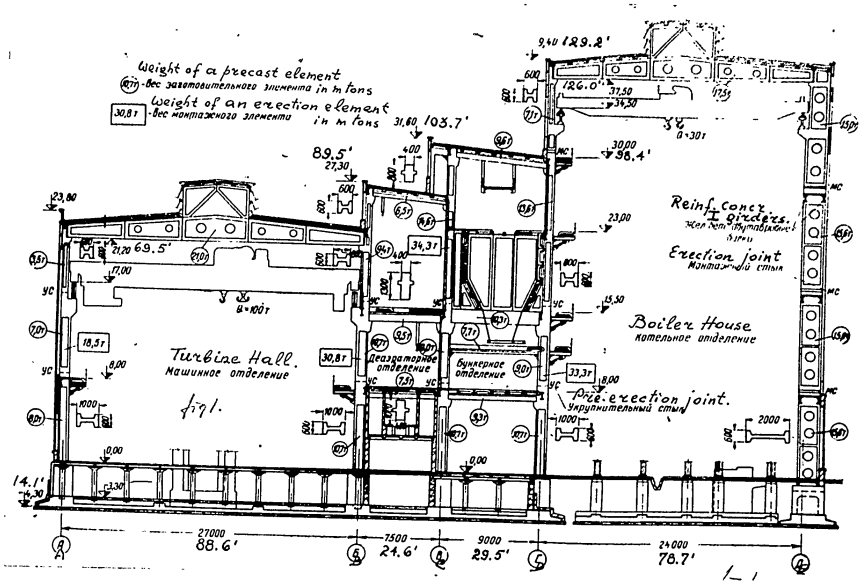
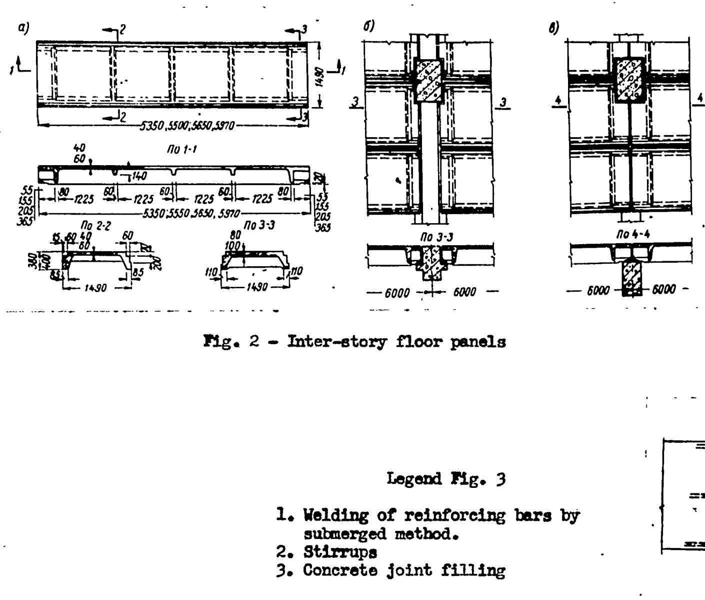


Fig. 1 - Transverse precast reinforced concrete frames of the main power plant building. The sedtions of the principal structural members are shown. In small circles are given the weights of the prefabricate members, and in small squares are the weights of members pre-assembled at the building site.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH TYPE LAYOUT DESIGNED IN PRECAST REINFORCED CONCRETE

Source: Stroitel'naya Promyshlennost' 1956, #6.

PLATE 25A



Legend Fig. 2

a. Geometric dimensions
b. Panels resting on brackets of the cross girder.
c. Panels resting on top of the cross girders.

Fig. 2 - Inter-story floor panels

Legend Fig. 3

1. Welding of reinforcing bars by submerged method.
2. Stirrups
3. Concrete joint filling

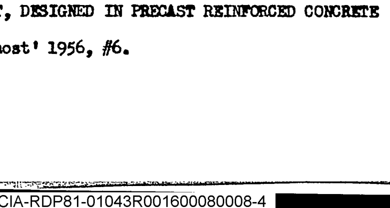
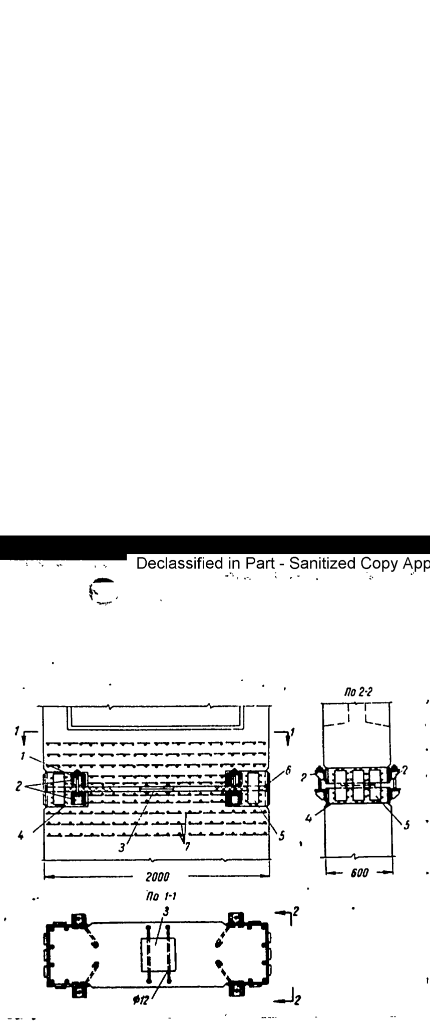


Fig. 3 - Preassembly joint of a column

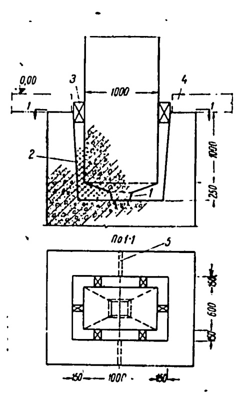
CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH-TYPE LAYOUT, DESIGNED IN PRECAST REINFORCED CONCRETE

Source: Stroitel'naya Promyshlennost' 1956, #6.

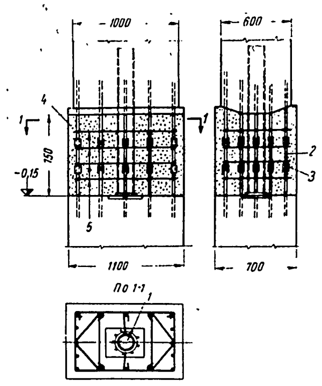
PLATE 25B



Legend Fig. 4.
Erection joint of a column
1. Erection adjustment belt.
2. Erection angles ∠ 120x10 mm.
3. Steel plates 300 x 200 x 16 mm.
4. Steel hoops placed around the column joint, 140 mm. x 10 mm.
5. Side steel plates 100 x 280 x 16 mm.
6. Cement plaster applied around the column joint.
7. Horizontal steel mesh set in the column (indirect reinforcing)



Legend Fig. 5.
Joint of a column with its foundation footing in case a basement is built.
1. Pin in the column of the butt.
2. Concrete filling.
3. Wedges.
4. Floor covering.
5. Scupper 50 mm (2 in.)



Legend Fig. 6.
Joint of a column with its foundation
1. Pipe 160/10 mm (10.2/0.4 in.) filled with concrete for adjustment of the column.
2. Insertion of reinforcing bar pieces.
3. Welding of ends of the reinforcing bars by the submerged arc method.
4. Filling with concrete.
5. Column stirrups.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH TYPE LAYOUT DESIGNED IN PRECAST REINFORCED CONCRETE

Source: Stroitel'naya Promyshlennost' 1956, #6.

PLATE 25C

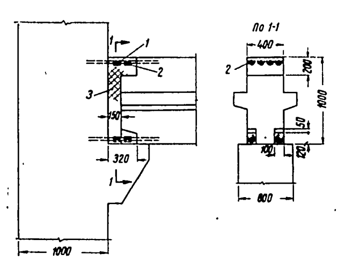


Fig. 7 - Legend
Joint of a transverse girder with a column
1. Insertion of pieces of reinforcing bars
2. Welding of ends of the reinforcing bars by the submerged arc method.
3. Concrete filling.

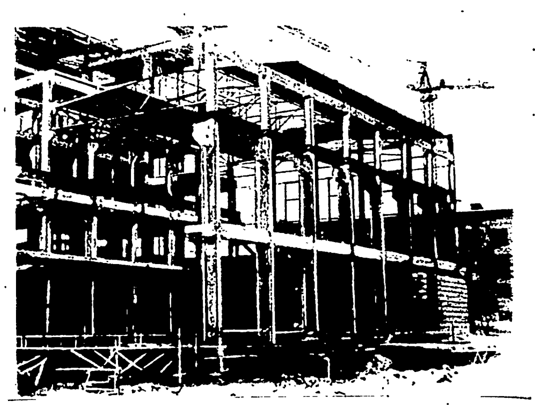


Fig. 9 - Legend
Construction of the frames of a TETs in precast concrete

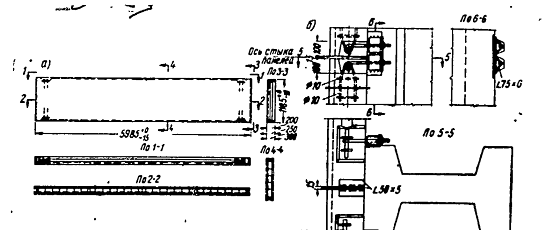


Fig. 8

Fig. 8 - Legend
a. Geometrical dimensions.
b. Details of the attaching of panels to a column.

CONSTRUCTION OF THE MAIN POWER PLANT BUILDING OF THE FOURTH TYPE LAYOUT, DESIGNED IN PRECAST REINFORCED CONCRETE

Source: Stroitel'naya Promyshlennost' 1956, #6

PLATE 25D.

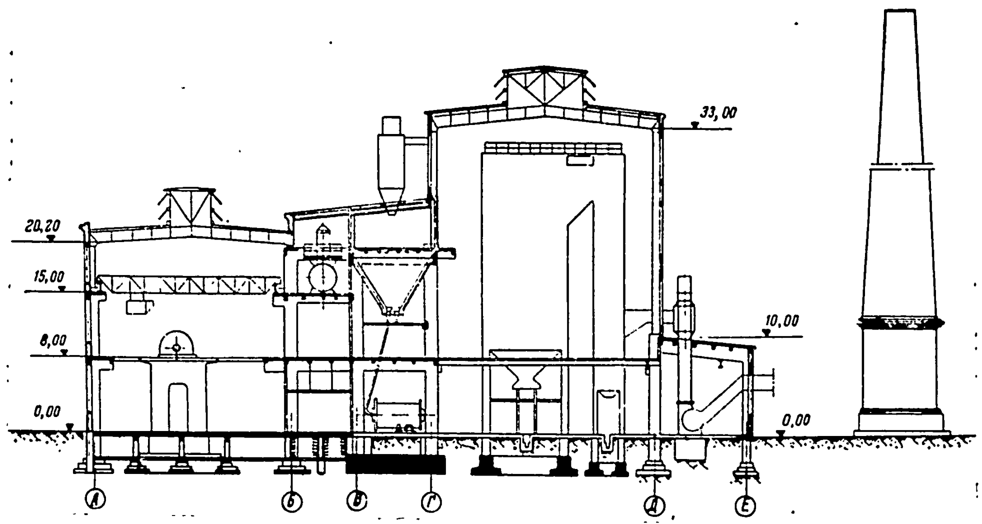


Fig. 1 - Frame construction in precast reinforced concrete and steel.

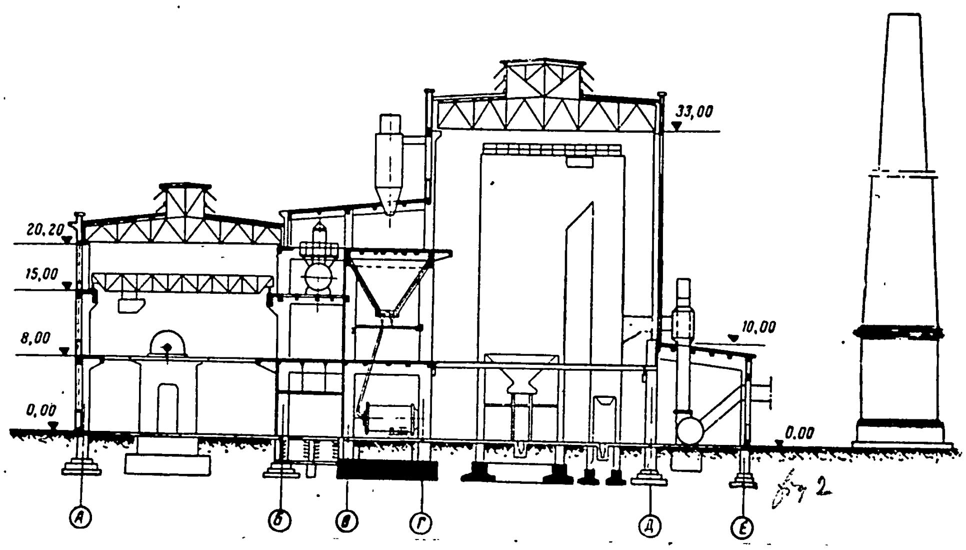


Fig. 2 - Frame construction entirely in precast reinforced concrete.

Two Alternatives in Construction.

MAIN POWER PLANT BUILDING

Source: Elektricheskiye Stantsii 1947, #7.

PLATE 25E

1.

THERMAL ELECTRIC POWER PLANTS

IN THE U.S.S.R.

Report No. 92

9014466

VOLUME II

COPY 3

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THERMAL ELECTRIC POWER PLANTS OF THE U.S.S.R.

Report No. 92

VOLUME II

Information Prepared
by

Air Information Division, Structural Engineering Section
Library of Congress

For
United States Air Force
September 1957

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CHAPTER III

Some of the Principal Soviet Thermal Power Plants.

Data (available and estimated) and photographs.

Abbreviations used:

- GRES – State Regional Electric Power Plant.
- GES – State Electrical Power Plant.
- TETs – Heat-and-Power Electric Plant.

List of Plants

List of Plants (Cont'd)

SHATURA PEAT-FIRING GRES

(Plates: 4, fig. 2; 26, 26A, 26B, 26C, 26D, 26E, 26F)

Location: Shatura, near Moscow.

Coordinates: 55° 35' N, 39° 36' E.

Date of construction: 1925, enlarged in 1933.

Layout type: Of special design, three separate boiler house buildings placed crosswise to the turbine hall (see Plate 4, fig. 2, also description on page)

Installed capacity: 180,000 kw. (1933)

Dimensions: Overall: L. - 180 ft.; W. - 82 ft.; H. - 44 ft.

Structural type: Poured-in-place reinforced concrete frame.

Wall covering: Brick curtain or panel walls.

Roof construction: Steel roof trusses.

Roofing: Ruberoid on wooden later changed to reinforced concrete sheathing.

Window sash: Steel.

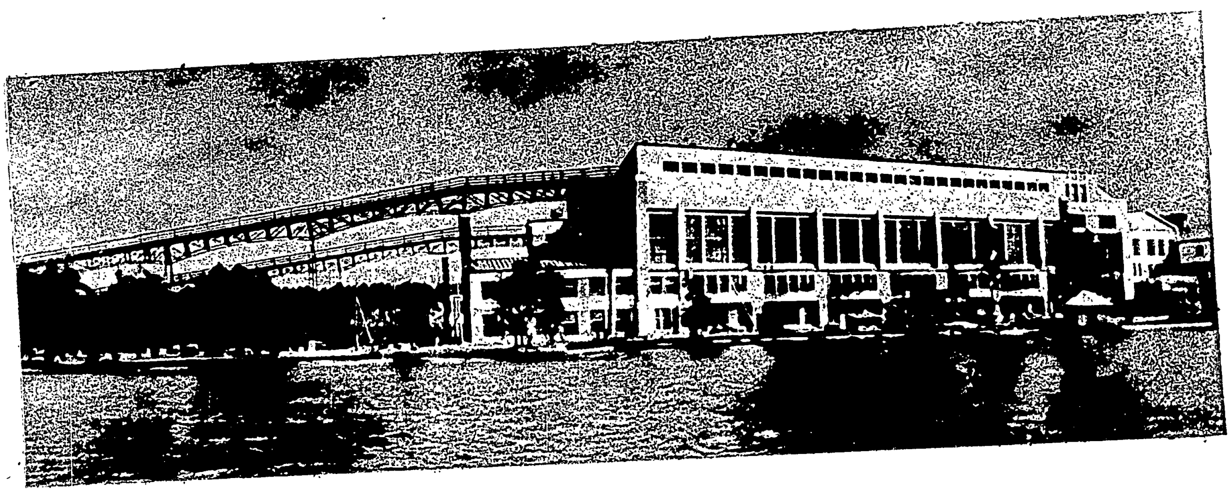
Cranes: In the turbine hall, cap. 75/15 mm tons (estimate).

Crane girders: Steel.

Stacks: Steel on the roof of the boiler house.

Trestles for peat delivery: Steel.

Substation: Open-air, adjoining the power plant building.



Side view of the GRES and peat delivery trestles.

SHATURA PEAT-FIRING GRES
(Capacity: 180,000 kw.)

Source: USSR in Construction 1930, #3, p. 8 top

PLATE 26

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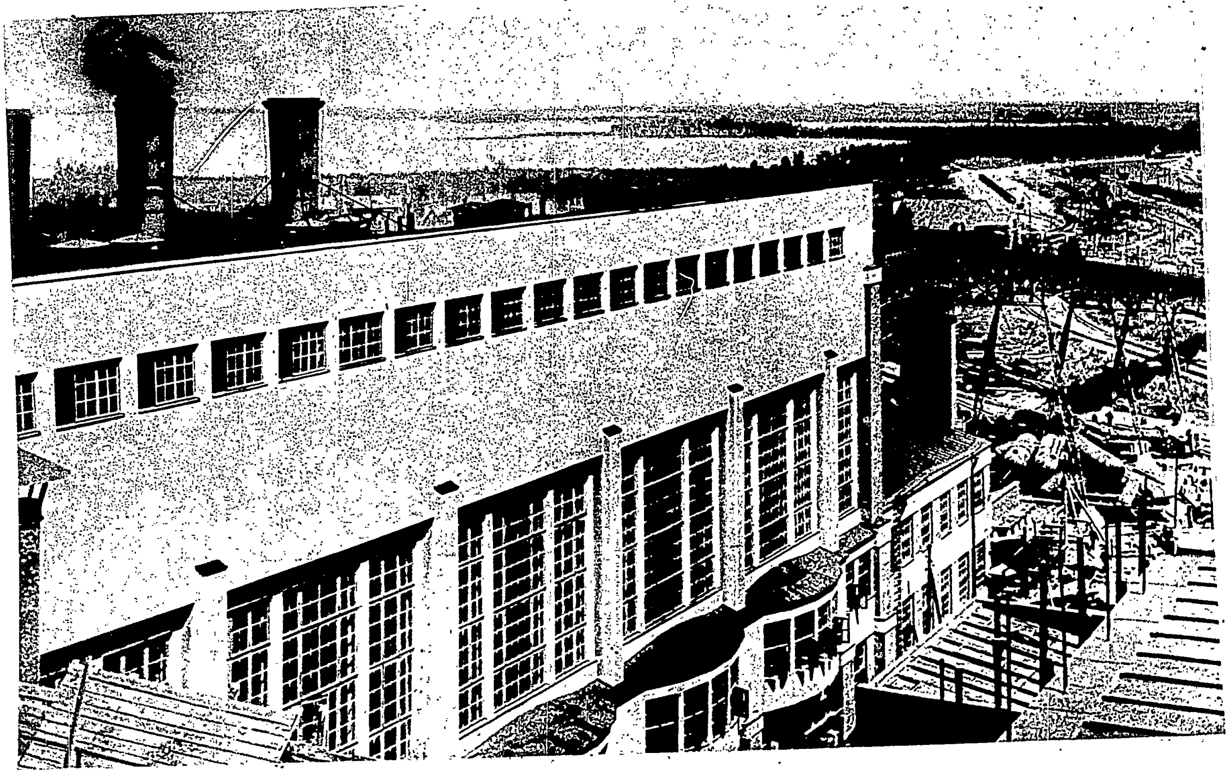
View from the turbine hall side.

SHATURA PEAT-FIRING GRES
(Capacity: 180,000 kw.)

Source: Elektrotechnische Zeitschrift 1930, p. 387 bottom (TK3.E8)

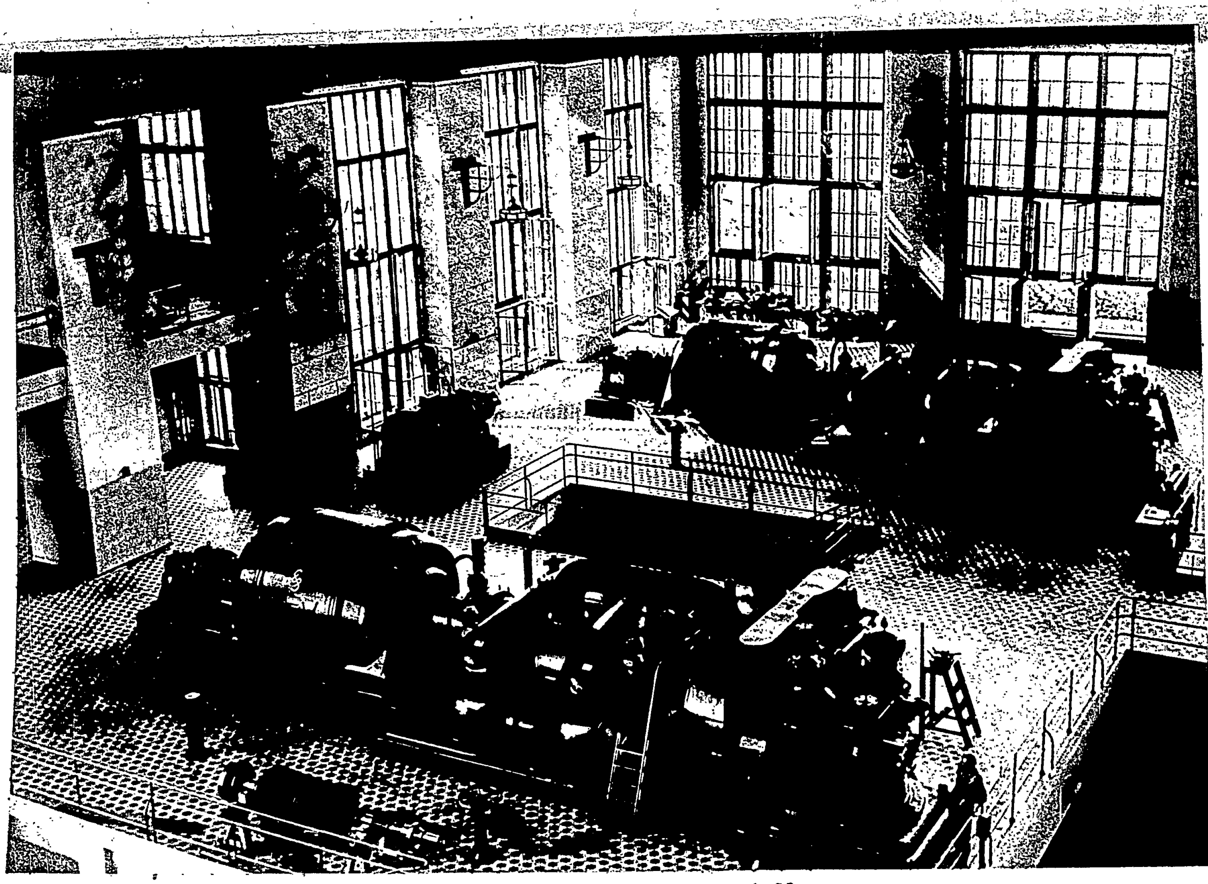
PLATE 26A



Partial view of the boiler house #2.

SHATURA PEAT-FIRING GRES (Capacity: 180,000 kw.)
Source: USSR in Construction 1930, #3, p. 9 top

PLATE 26B

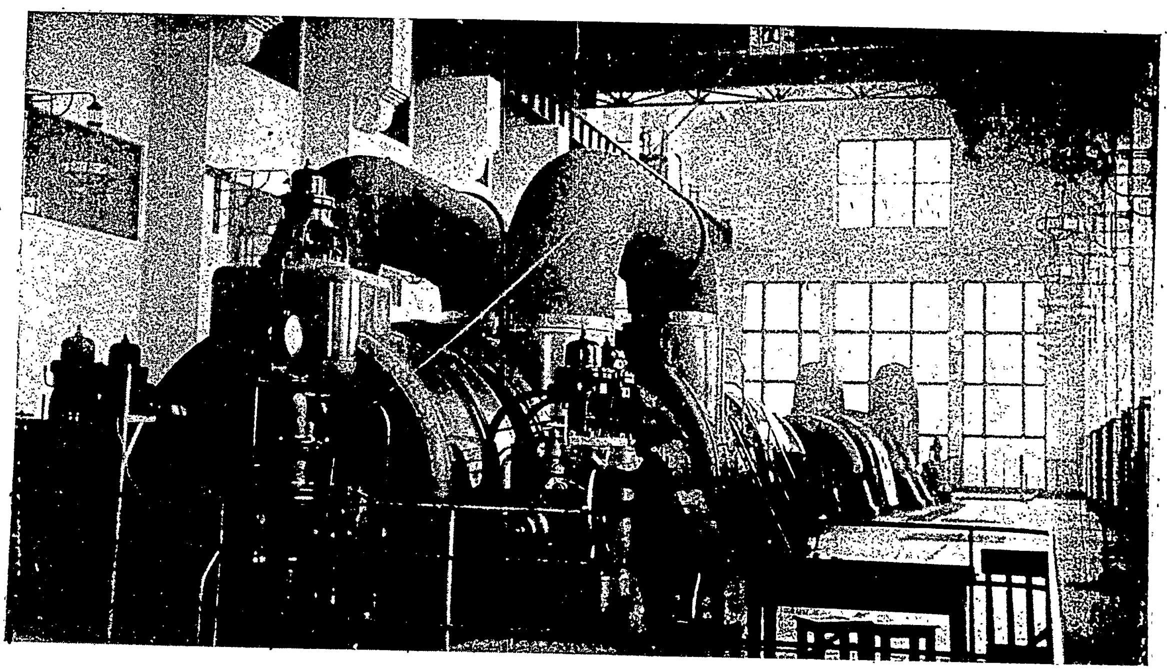


Partial view of a turbogenerator hall.

SHATURA PEAT-FIRING GRES (Capacity: 180,000 kw.)

Source: USSR in Construction 1930, #3, p. 9 bottom

PLATE 26C



Partial view of second section of the turbogenerator hall.

SHATURA PEAT-FIRING GRES
(Capacity: 180,000 kw.)

Source: Economic Review of the Soviet Union 1931, #8, p. 178.

PLATE 26D

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Partial view of the transformer bank.

SHATURA PEAT-FIRING GRES
(Capacity: 180,000 kw.)

Source: Economic Review of the Soviet Union 1929, #11, front cover

PLATE 26E

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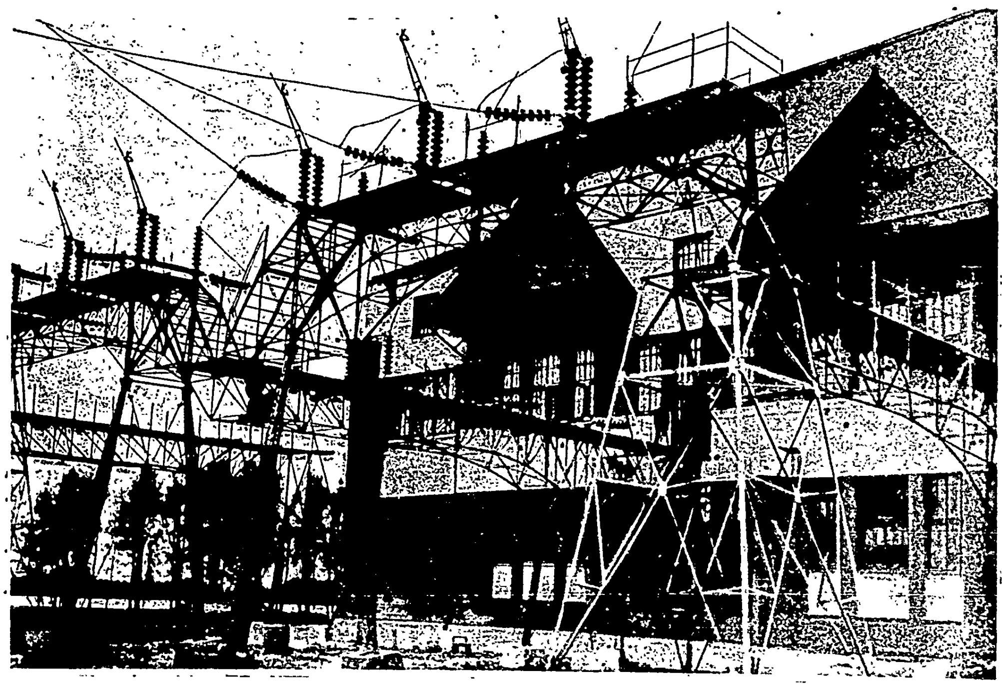

Partial exterior view of a sub-station and steel structures carrying high tension wires.

SHATURA PEAT-FIRING GRES
(Capacity: 180,000 kw.)

Source: Prozhektor 1925, #20, p. 8 bottom

PLATE 26F

DUBROVKA PEAT-FIRING GRES

(Plates: 5; 27, 27A, 27B, 27C)

Location: Village on the Neva River near Leningrad

Coordinates: 59° 50' N, 31° 00' E.

Layout type: Of special design, two boiler houses, between them the turbine hall (see Plate 5, also description p.)

Date of Construction: Built in 1933, reconstructed in 1946.

Installed capacity: 200,000 kw (1946)

Structural type: Steel frame (probably)

Outside dimensions: L. ; W. 297 ft.

Wall covering: Originally brick curtain walls, reconstructed - reinforced concrete panels.

Roof construction: Steel trusses.

Roofing: Ruberoid on reinforced concrete sheathing (prob.).

Boiler house: W. - 59 ft; Height to roof truss: 69 ft. (est).

Bunker sect.: W. - 31 ft.

Turbine Hall: W. - 116 ft; Height to roof truss: 79 ft (est).

Cranes: In the turbine hall, cap. 75/15 m tons (est).

Crane girders: Steel

Stacks: Steel 2 rows of 4, on roof of turbine houses.
Height above roof 18 ft., ∅ 12 ft.

Trestles for peat delivery: Steel.

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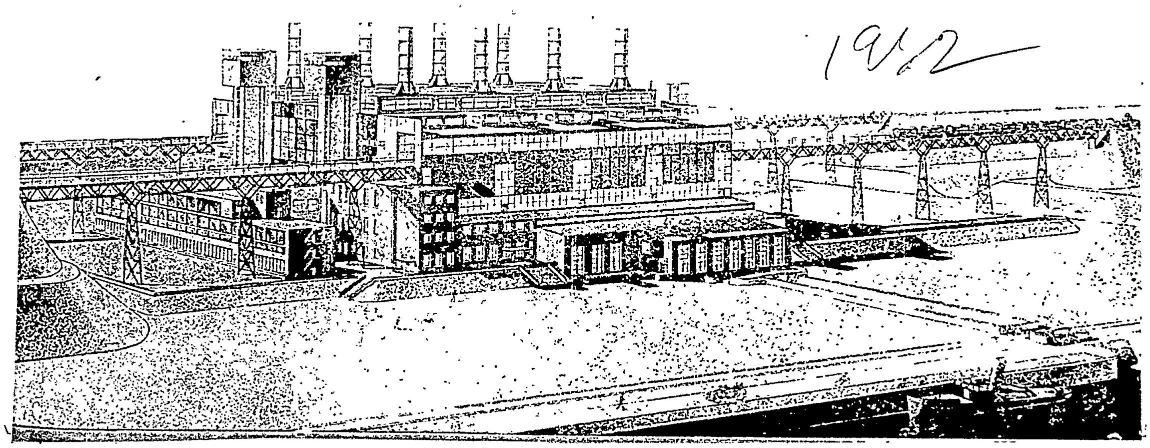
2

1932

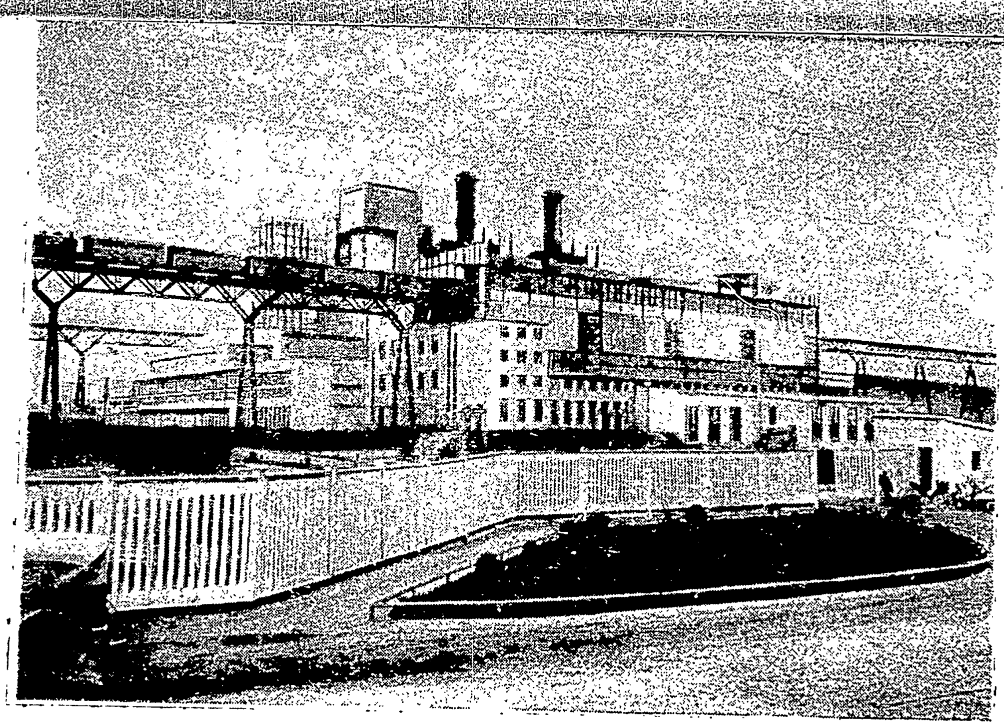


General view

DUBROVKAA PEAT-FIRING GRES
(Capacity: 200,000 kw.)

Source: Elektricheskiye Stantsii 1932, #3, front cover, (TK4.E725).

PLATE 27



Side view of the plant showing steel trestles for peat delivery and a boiler house.

DUBROVKA PEAT-FIRING GRES
(Capacity: 200,000 kw.)

Source: Pyatnadsat' let leninskogo plana elektrifikatsii 1936, p. 69 (TK85.D6)

PLATE 27A

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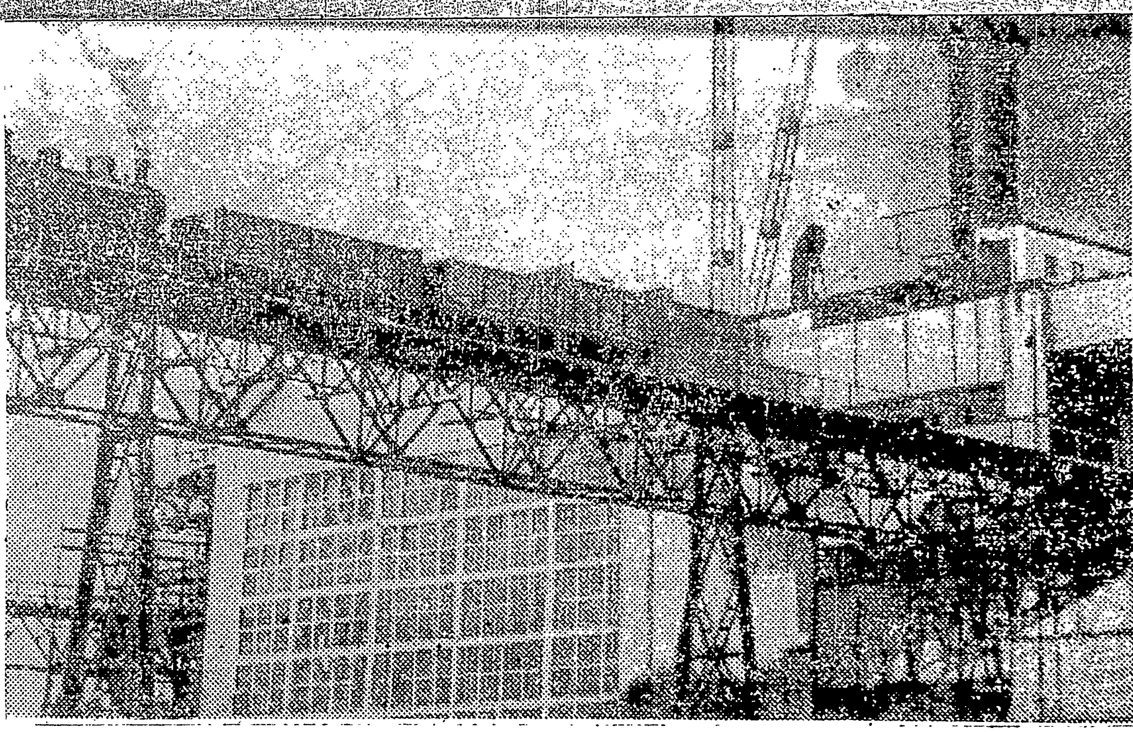
-103-

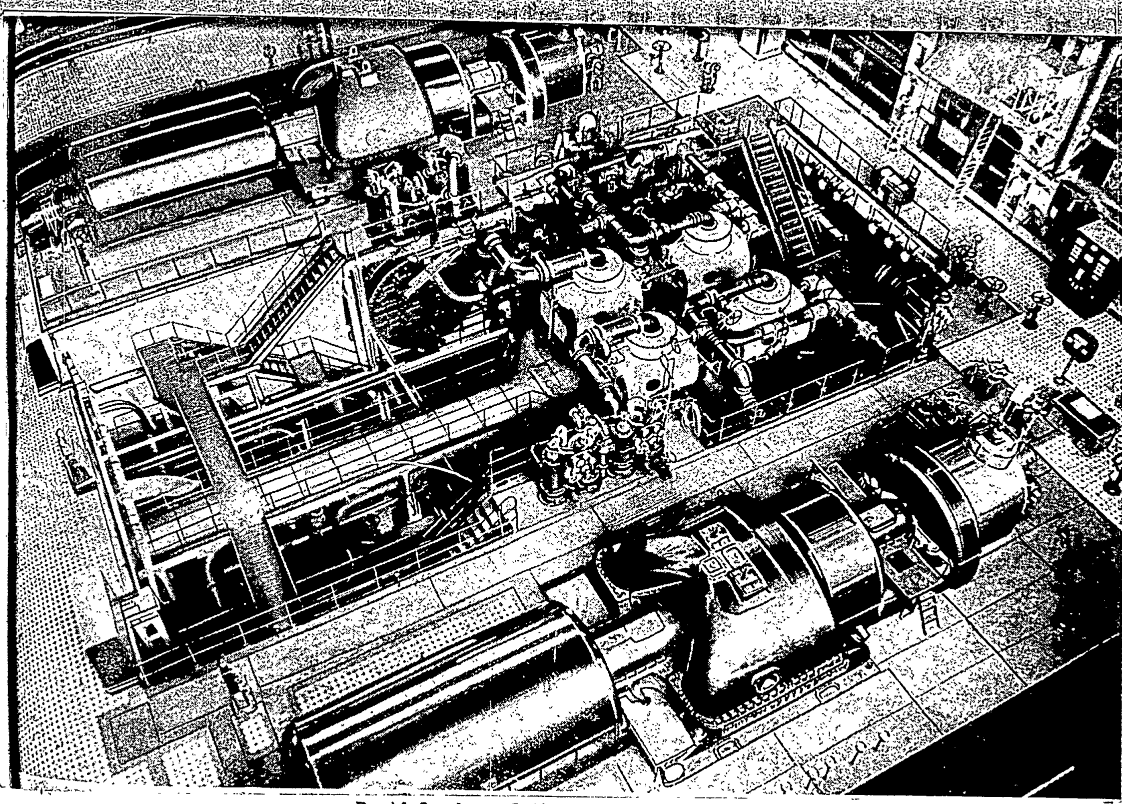


Partial view of the GRES showing steel trestles for peat delivery.
(during the reconstruction in 1946)

DUBROVKA PEAT-FIRING GRES
(Capacity: 200,000 kw.)

Source: Leningradskaya Pravda 1947, #14, p. 4.

PLATE 27B



Partial view of the turbine hall.

DUBROVKA PEAT-FIRING GRES (Capacity: 200,000 kw.)

Source: Elektricheskiye Stantsii 1937, #10, front cover, (TK4.E725).

PLATE 27C

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GOR'KIY GRES (Mixed Fuel: Peat and Coal Firing)

(Plates: 4, fig. 1; 28, 28A, 28B, 28C)

Location: Town of Balakhna on the banks of theVolga River, 33 klm from the city of Gor'kiy.

Coordinates: 56° 20' N, 44° 00' E.

Date of Construction: First part - 1930.
Second part - 1933.

Layout type: First part - First type design.
Second part - Second type design (prob).

Installed capacity: 204,000 kw (1933).

Structural type: Poured-in-place reinforced concrete frame

Wall covering: Brick curtain walls.

Roof construction: Monolithic reinforced concrete beams.

Roofing: Ruberoid.

Cranes: In the turbine hall.

Crane girders: Reinforced concrete.

Floor construction in the turbine hall: Steel grid on steel stanchions.

Stacks: Steel on roofs of boiler houses.

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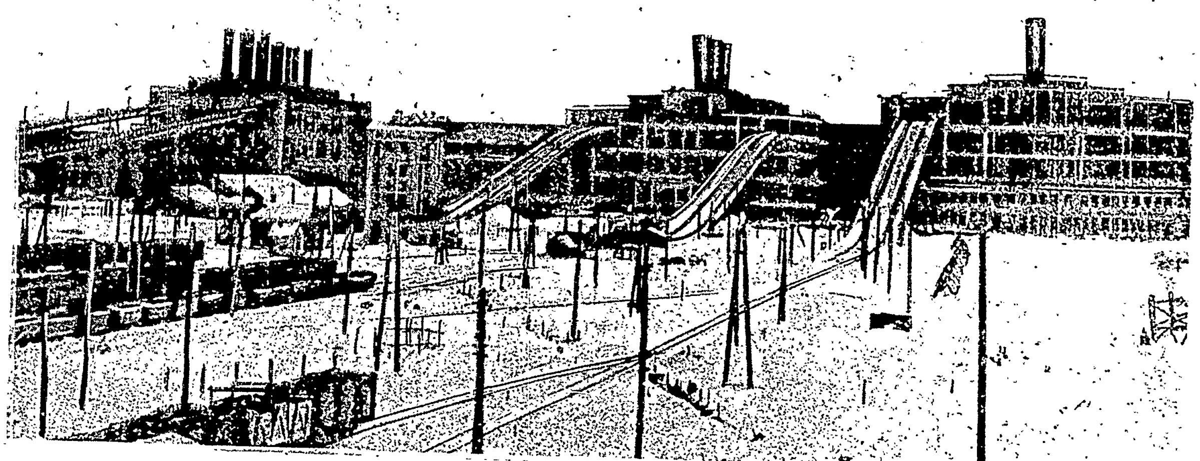

General view of the boiler houses - first part of construction.

GOR'KIY GRES (mixed peat and coal firing)
(Capacity: 204,000 kw.)

Source: Elektricheskiye Stantsii 1934, #4, front cover (TK4.E725)

PLATE 28

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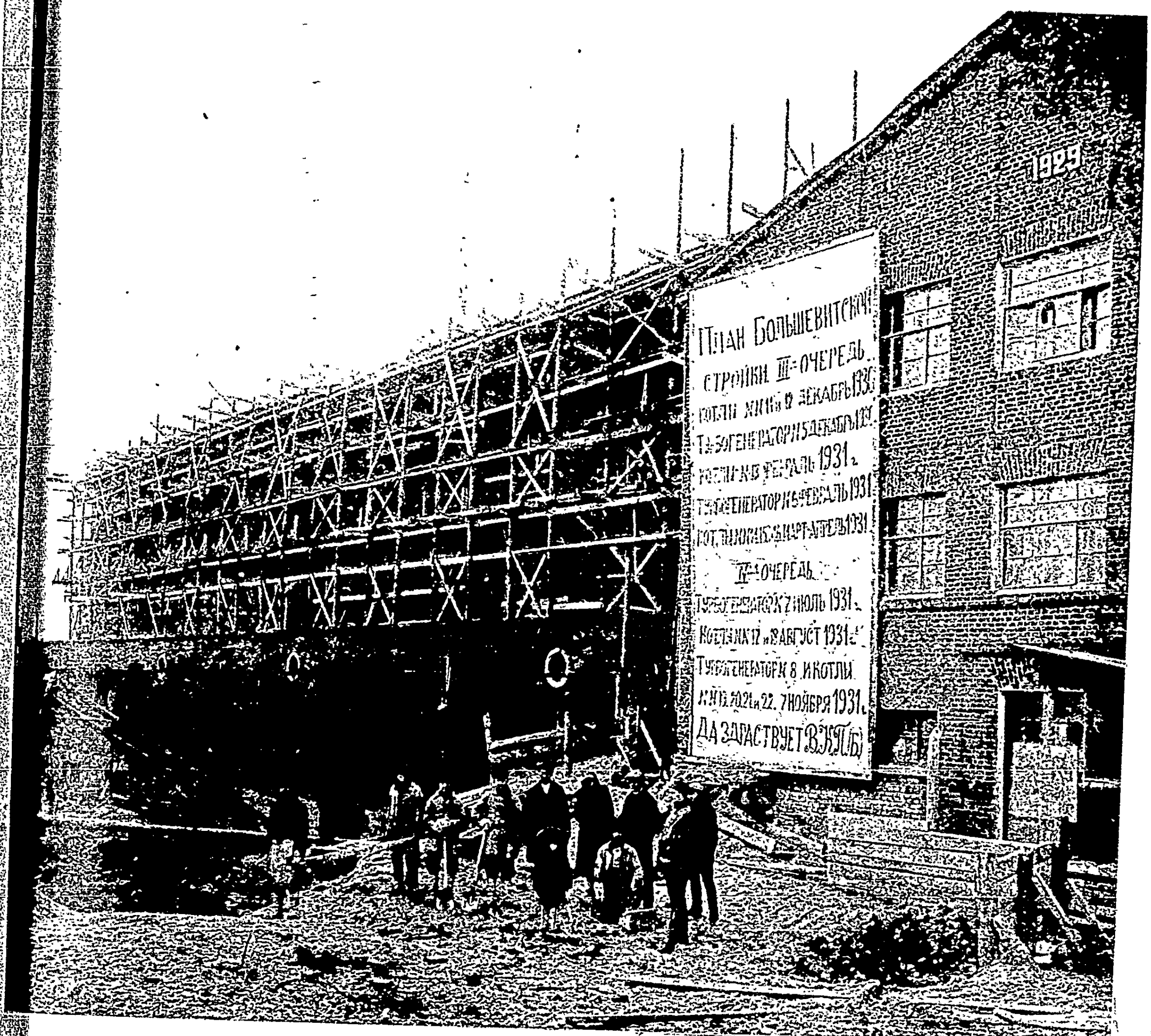

One of the GRES buildings under construction.

GOR'KIY PEAT AND COAL-FIRING GRES
(Capacity: 204,000 kw.)

Source: Prozhektor, 1930, #34, p. 9.

PLATE 28A

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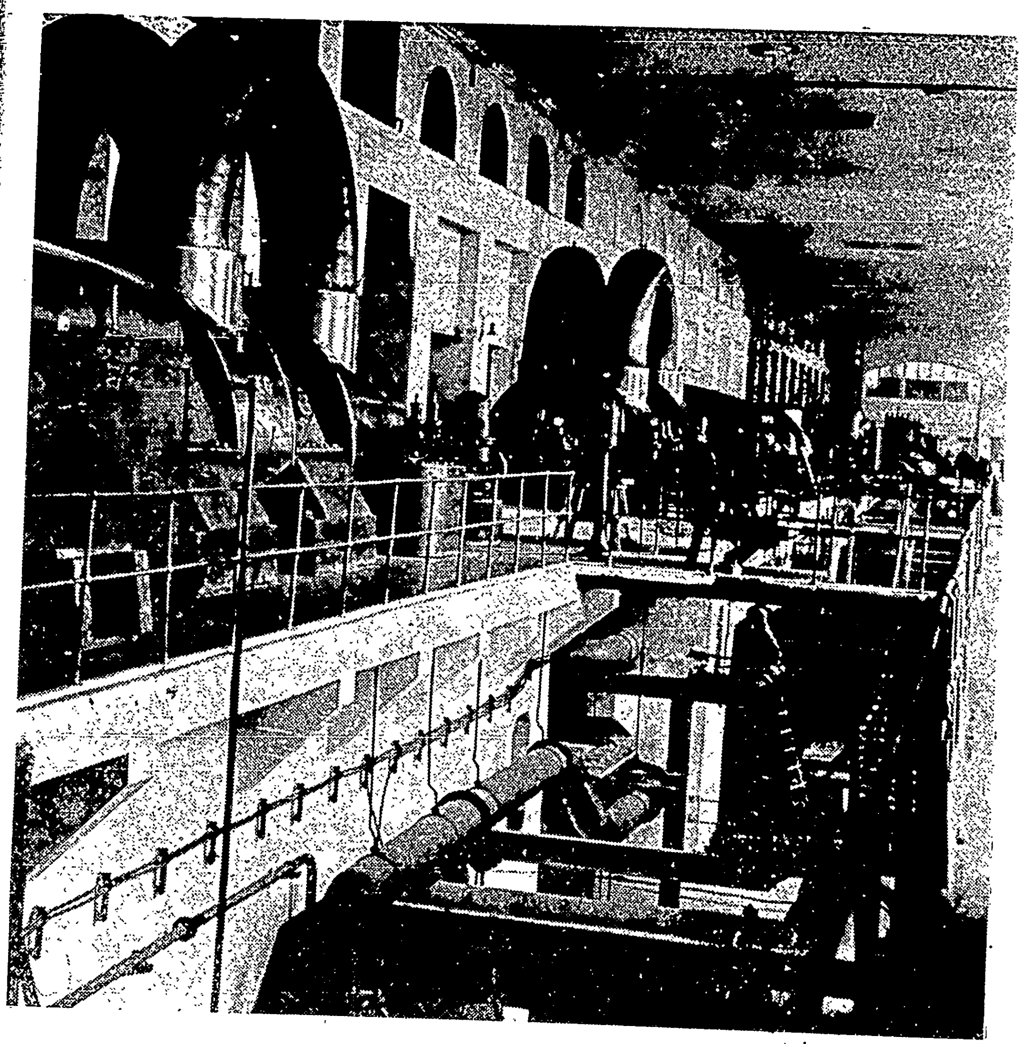
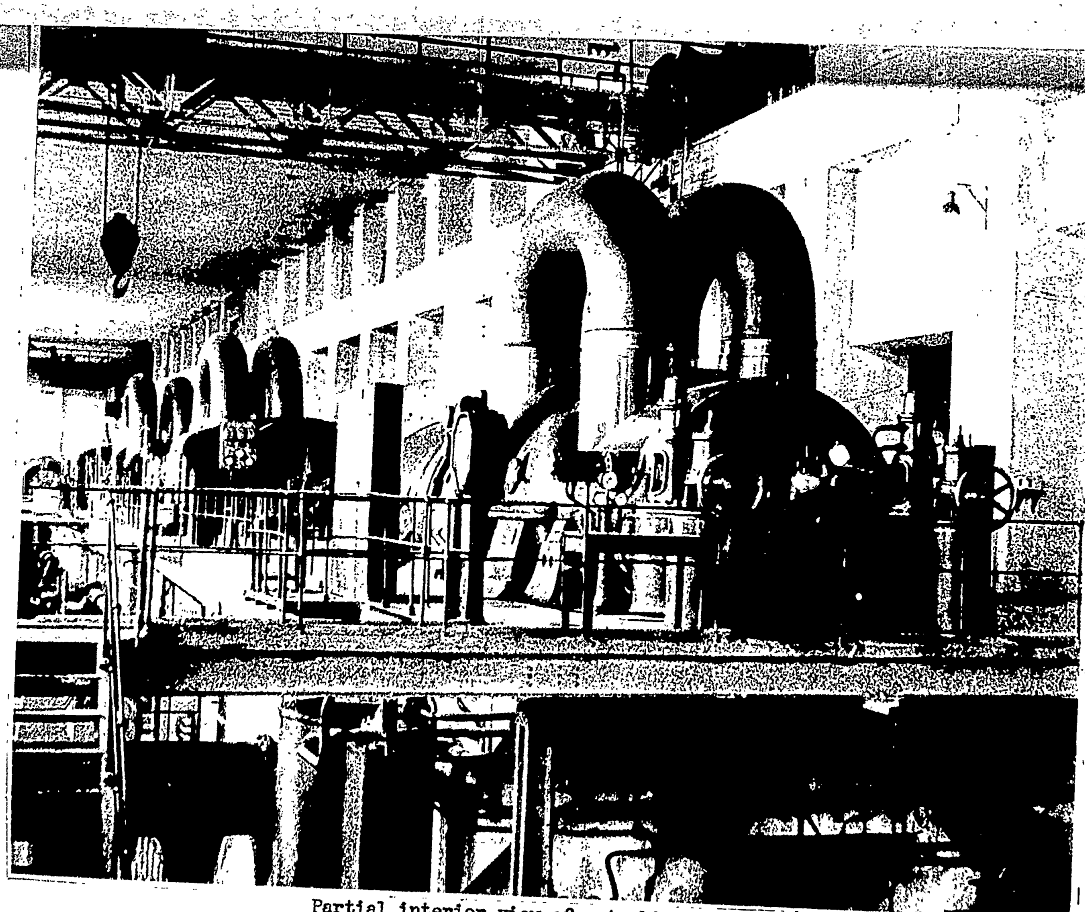


Partial view of the turbine hall.

GOR'KIY GRES (mixed peat and coal firing)
(Capacity: 204,000 kw.)

Source: Economic Review of the Soviet Union 1932, #7, p. 164.

PLATE 28B



Partial interior view of a turbine hall.

GOR'KIY PEAT AND COAL-FIRING GRES
(Capacity: 204,000 kw.)

Source: USSR in Construction 1933, #3, p. 12 middle
PLATE 28C

KASHIRA PULVERIZED COAL GRES

(Plates: 29, 29A, 29B, 29C, 29D, 29E, 29F, 29G, 29H)

Location: Kashira, town in Moscow Region.

Coordinates: 54° 50' N, 38° 12' E.

Date of construction: First built in 1923, extended in 1931 and 1932.

Layout type: First type design.

Installed capacity: 186,000 kw (1932).

Structural type: Poured-in-place reinforced concrete frame.

all covering: Brick curtain walls ; wall section around the coal bunkers finished with hollow concrete blocks.

oof construction: Steel trusses and reinforced concrete beams.

oofing: Metal sheets and ruberoid.

ranes: In the turbine hall, originally 35 m. tons capacity later propably increased to 75/15 m tons.

ane girders: Steel.

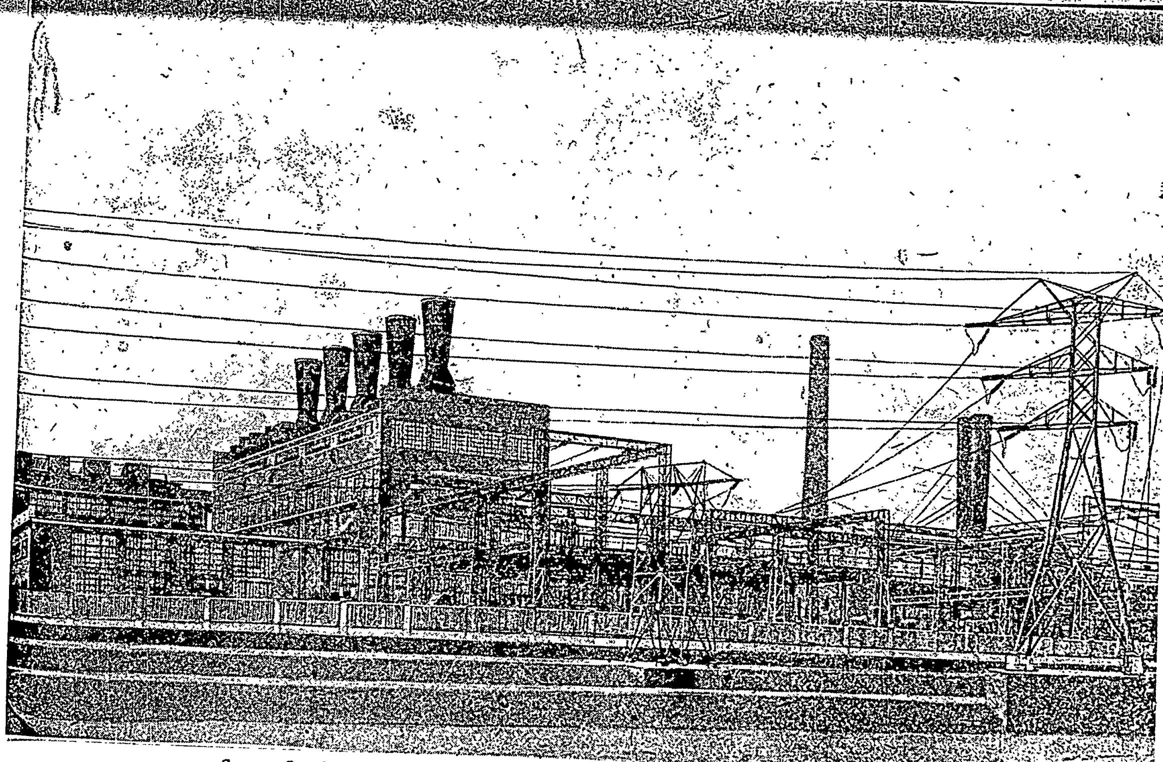
rbine hall floor: Steel frame on steel stanchions, covered with tile.

al bunkers: Reinforced concrete.

e general power distribution control panel is located in the continuation of e turbine hall section.

acks: Steel on roof of building.

bstations: Steel frames - outdoor.



General view of the boiler house and the open-air sub-station.

KASHIRA PULVERIZED COAL-FIRING GRES. (Capacity: 186,000 kw.)

Source: Elektricheskiye Stantsii 1933, #5, front cover (TK4.E725)

PLATE 29

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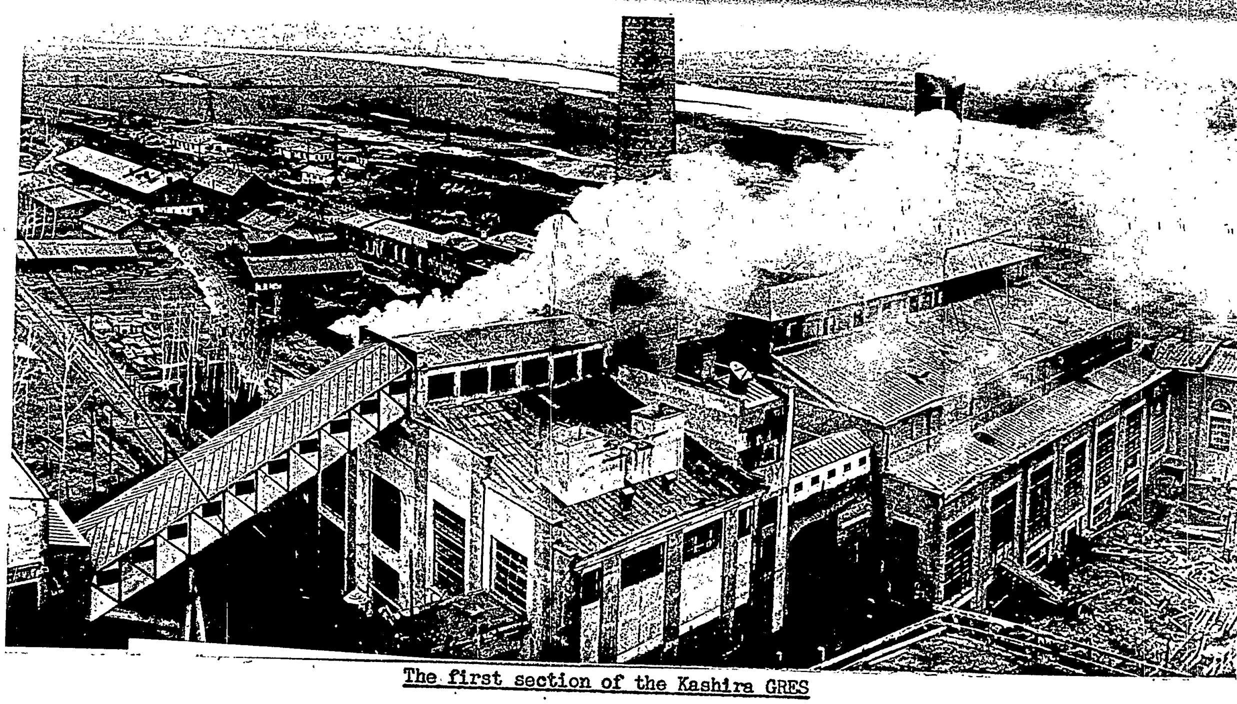

The first section of the Kashira GRES

KASHIRA PULVERIZED COAL-FIRING GRES
(Capacity: 186,000 kw.)

Source: USSR in Construction 1930, #3, top p. 6

PLATE 29A

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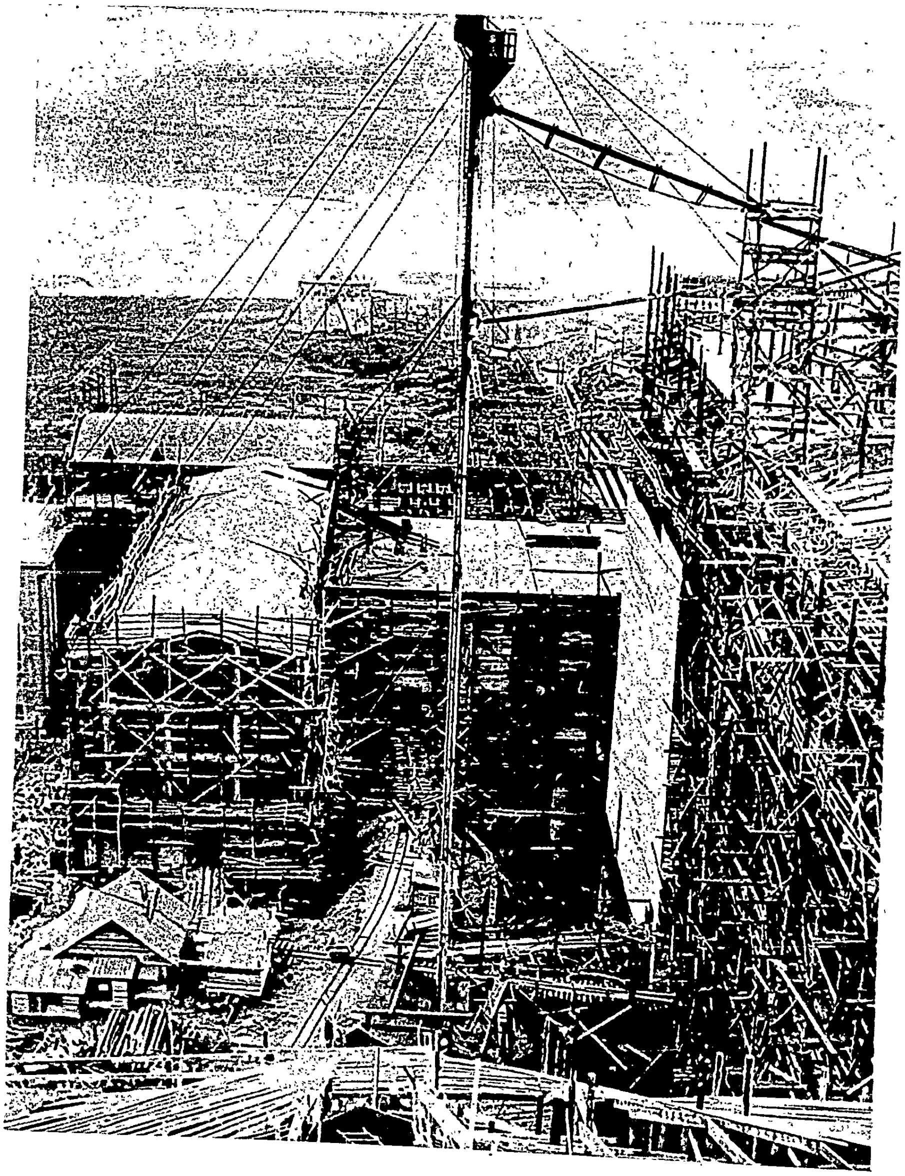

The second section of the Kashira GRES under construction

KASHIRA PULVERIZED COAL-FIRING GRES (Capacity: 186,000 kw.)

Source: USSR in Construction 1930, #3, p. 7, bottom.

PLATE 29B

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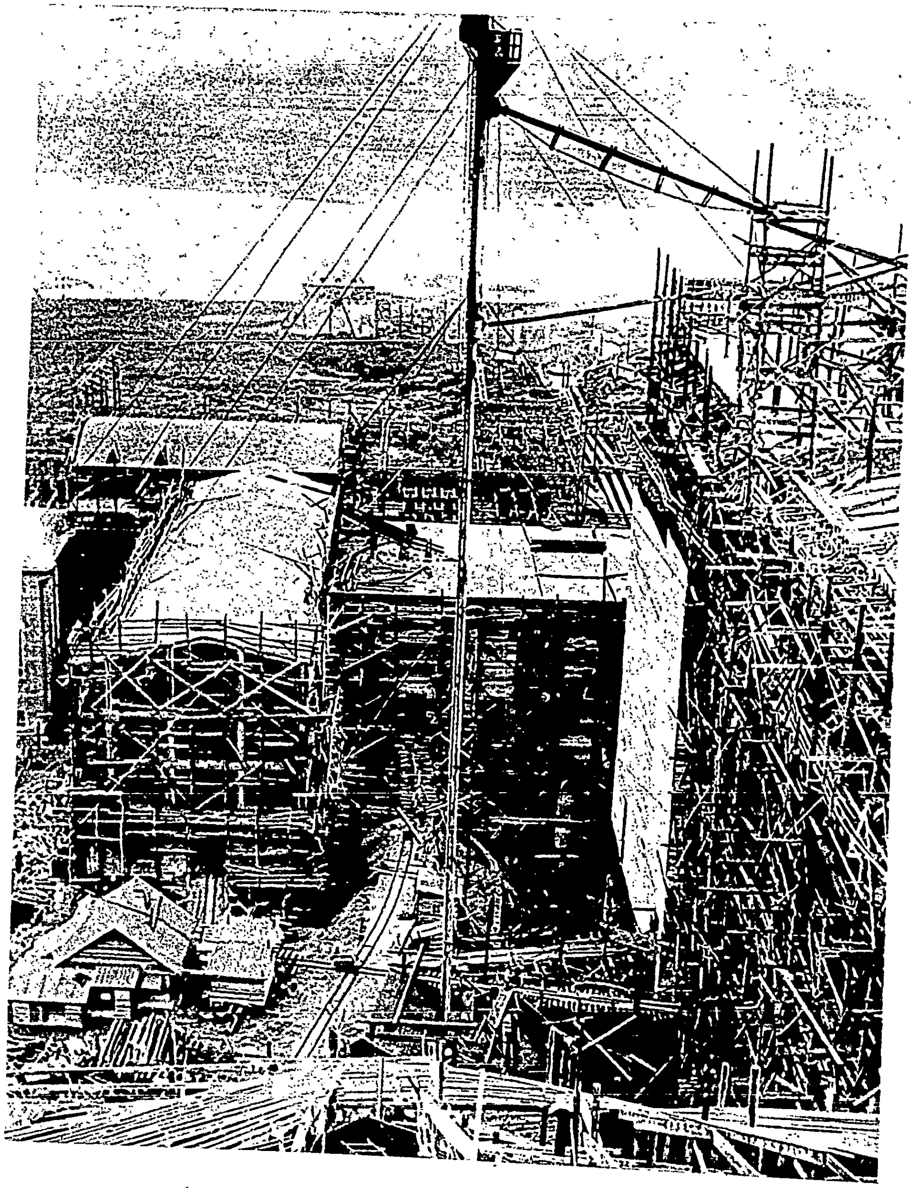

The second section of the Kashira GRES under construction

KASHIRA PULVERIZED COAL-FIRING GRES (Capacity: 186,000 kw.)

Source: USSR in Construction 1930, #3, p. 7, bottom.

PLATE 29B

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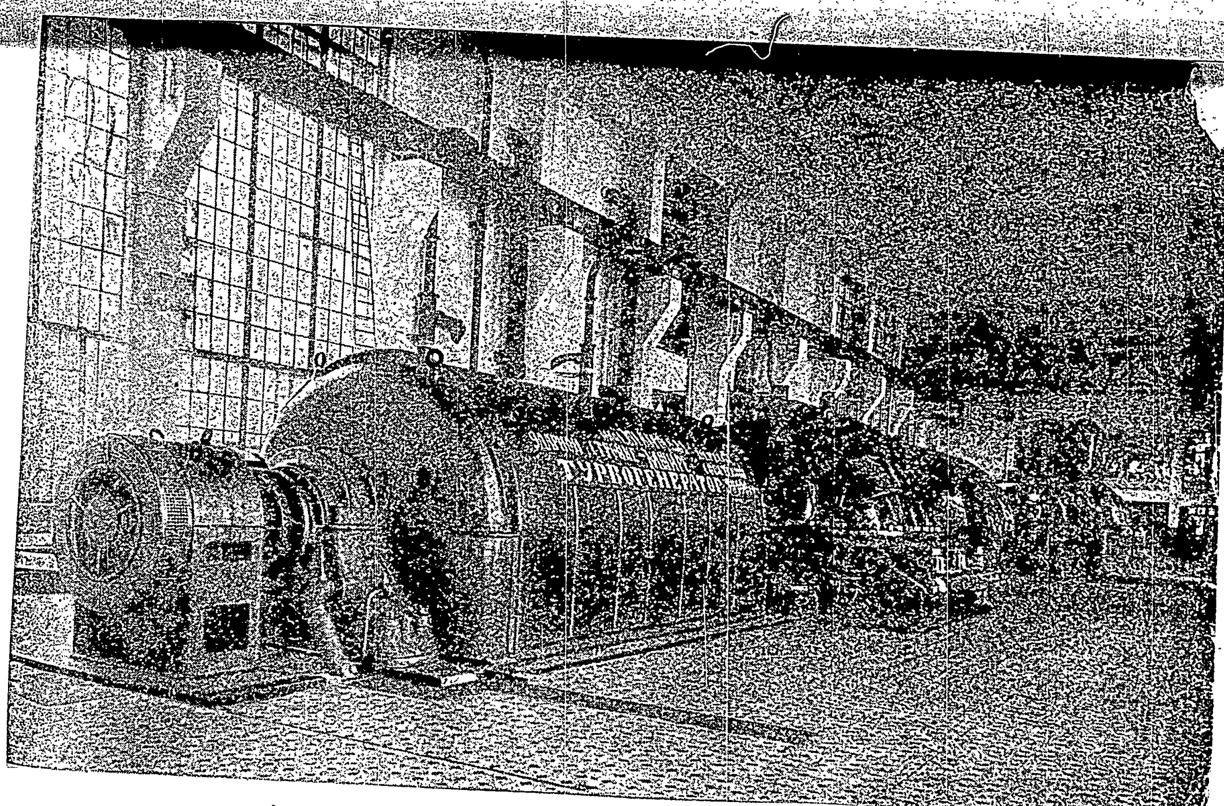

A 50,000 kw. turbogenerator built by the "Elektrosila" Plant.

KASHIRA PULVERIZED COAL-FIRING GRES (Capacity: 186,000 kw.)

Source: Elektricheskiye Stantsii, 1933, #6, front cover (TK4.#725)

PLATE 29C

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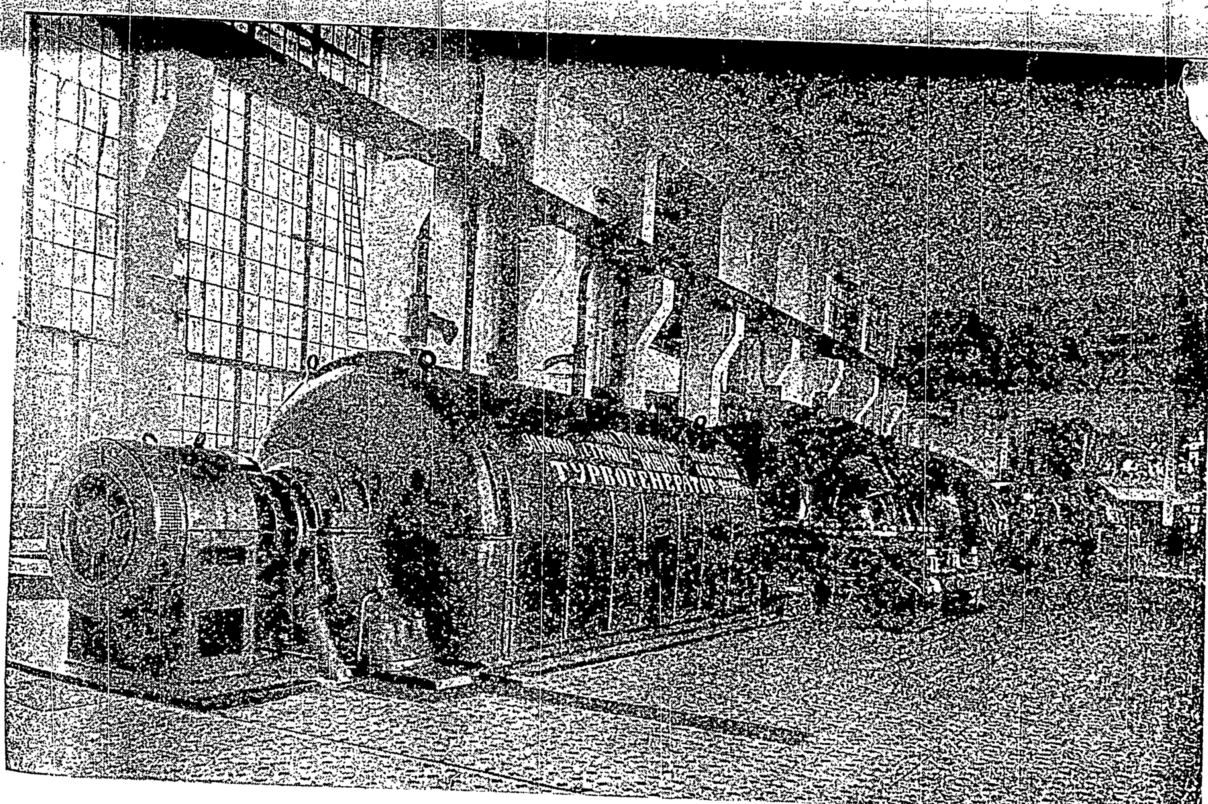

Partial view of the turbine hall.

KASHIRA PULVERIZED COAL-FIRING GRES (Capacity: 186,000 kw.)

Source: USSR in Construction 1930, #3, bottom p. 6.

PLATE 29D

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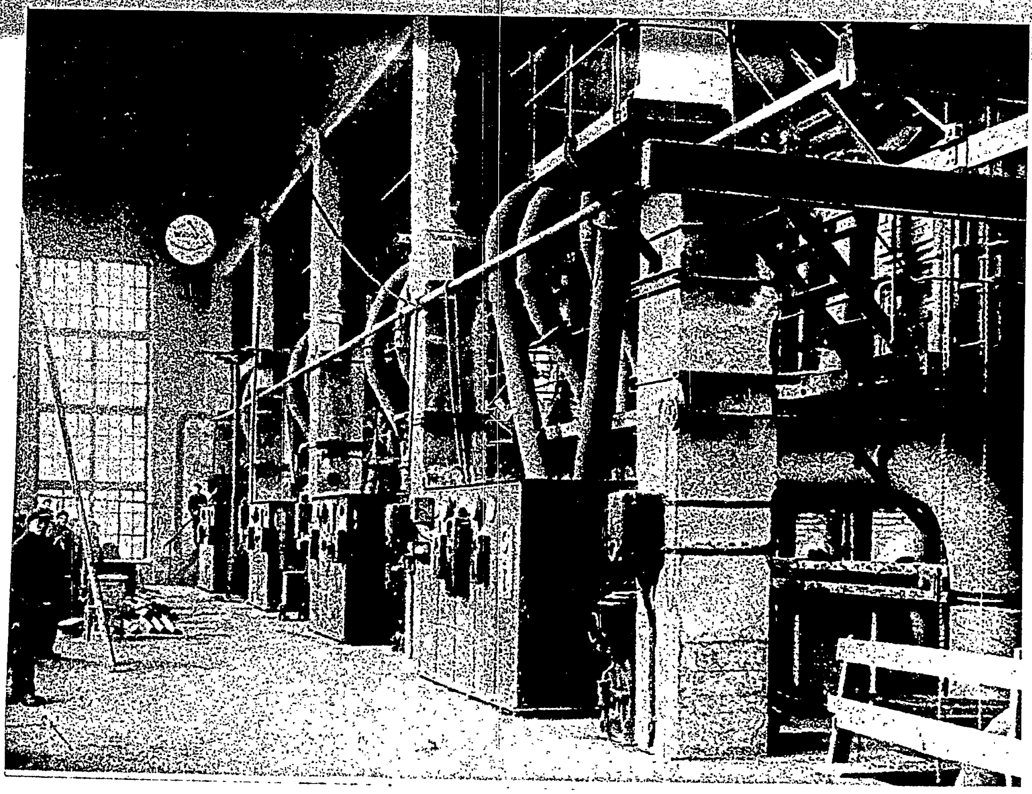
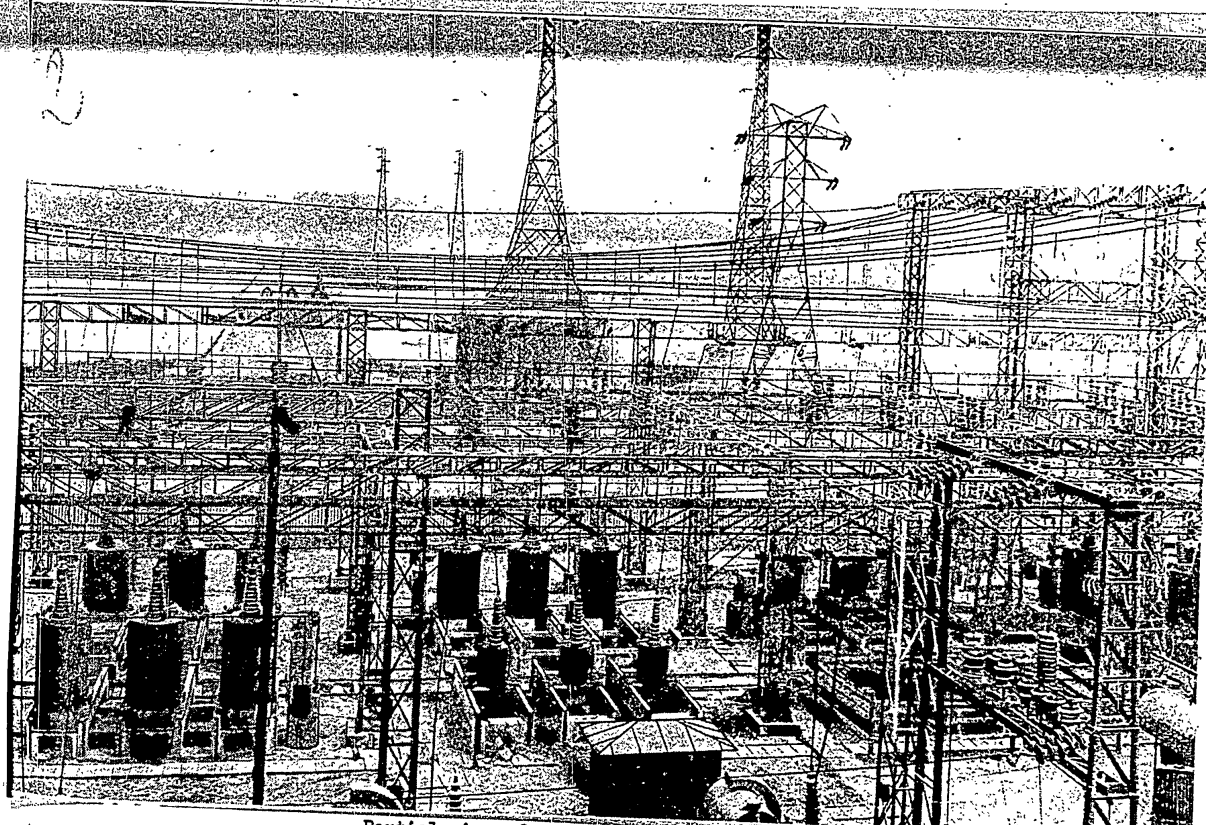


Coal pulverizing equipment.

KASHIRA PULVERIZED COAL-FIRING GRES (Capacity 186,000 kw.)

Source: USSR in Construction 1930, #3, p. 7, top right.

PLATE 29E



Partial view of an outdoor sub-station

KASHIRA PULVERIZED-COAL FIRING GRES (Capacity: 186,000 kw.)

Source: Elektricheskiye Stantsii 1933, #7, front cover, (TK4.E725)

PLATE 29F

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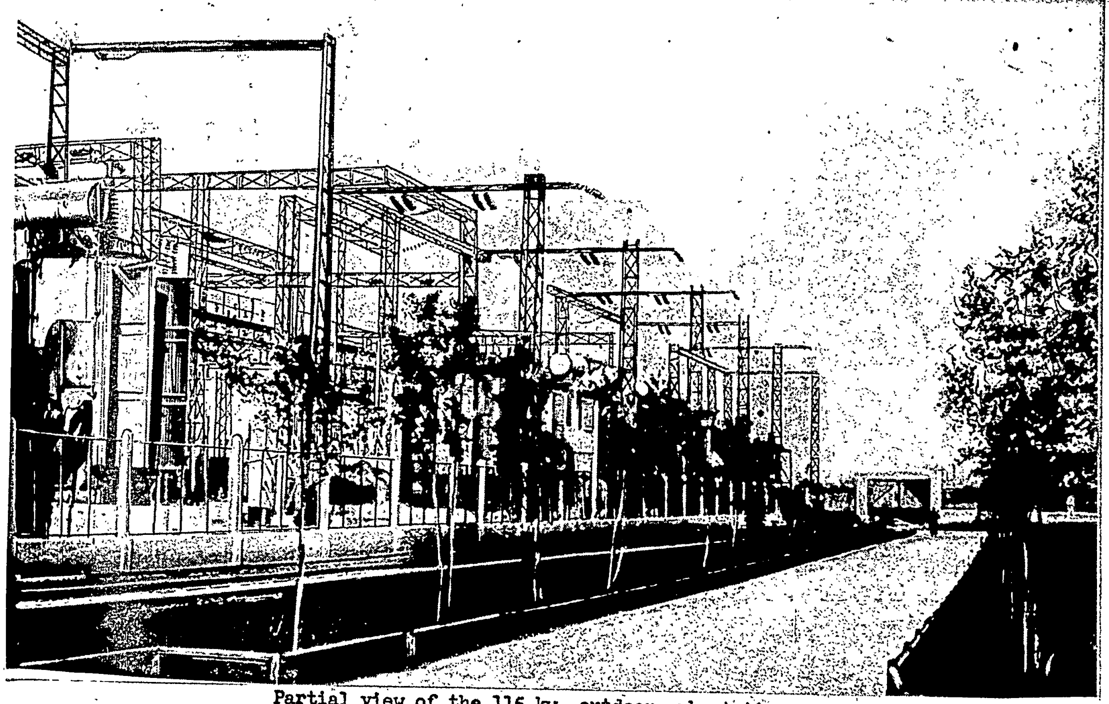


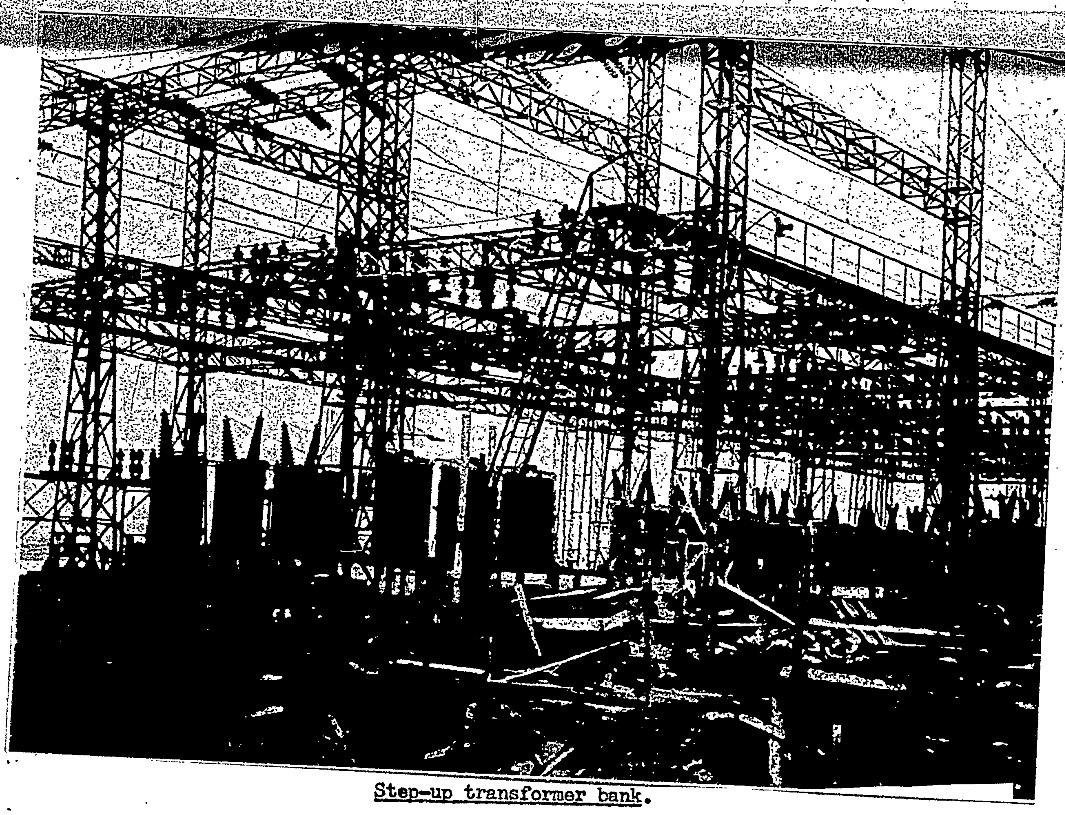
Partial view of the 115 kw. outdoor sub-station.

KASHIRA PULVERIZED COAL-FIRING GRES (Capacity: 186,000 kw.)

Source: Elektricheskiye Stantsii, 1937, #6, front cover (TK4.E725)

PLATE 29G

-119-



Step-up transformer bank.

KASHIRA PULVERIZED COAL-FIRING GRES
(Capacity: 186,000 kw.)

Source: USSR in Construction 1930, #3, p. 7, top left.

PLATE 29H

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SHTEROVKA PULVERIZED COAL GRES

(Plates: 30, 30A, 30B, 30C, 30D, 30E, 30F, 30G)

Location: Shterovka in the Ukranian Donbass.

Coordinates: 48° 05' N, 38° 55' E.

Date of construction: Built in 1926, extended in 1930 and 1931. Reconstructed after the war.

Layout type: First type design

Installed capacity: 152,000 kw. (1931)

Dimensions: Turbine hall: Length - 115 ft.; Width 50 ft.; Height - 65 ft.
Feedwater pump section: Length - 98 ft.; Width 23 ft.; Height 67ft.
Boiler house: Length - 164 ft.; Width 98 ft.; Height - 93 ft.

Structural type: Poured-in-place reinforced concrete frame.

Wall covering: First construction - brick curtain wall, later extensions, reinforced concrete panels.

Roof construction: Steel trusses (prob).

Roofing: Ruberoid (prob).

Cranes: In the turbine hall - cap. 75/15 (est).

Crane girders: Reinforced concrete.

Turbine hall floor: Steel frame on steel stanchions tile floor cover.

Stacks: Steel on roof of building.

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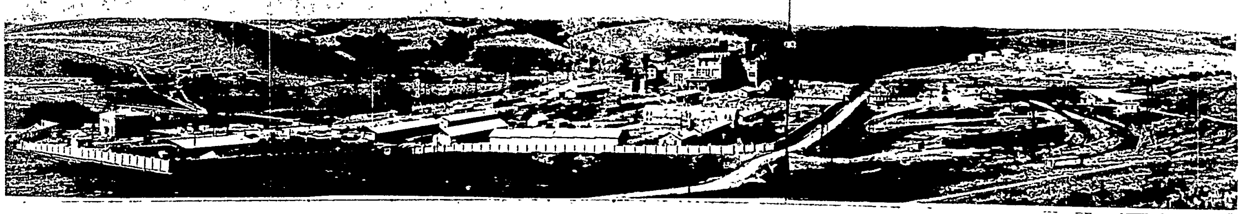
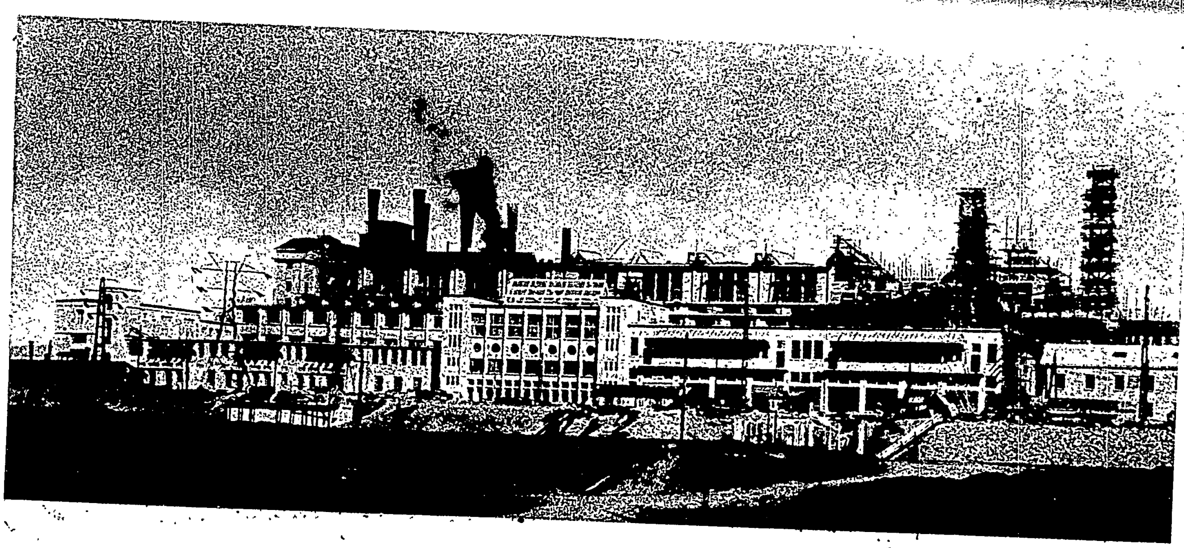


General view of the plant.

SHTEROVKA PULVERIZED COAL FIRING GRES (Capacity: 152,000 kw.)

Source: USSR in Construction 1930, #3 top. pp. 10-11.

PLATE 30

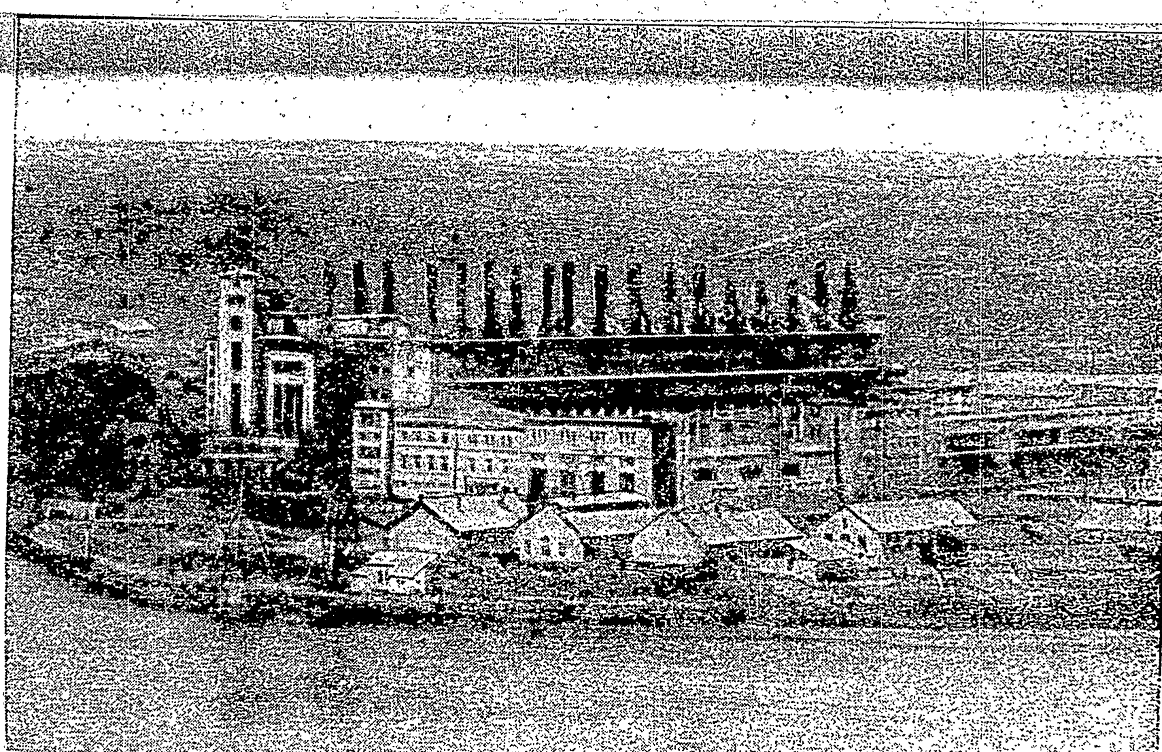


View of the main structure from the south.

SHTEROVKA PULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: USSR in Construction 1930, #3, p. 10 middle.

PLATE 30A



General view of the GRES.

SHTEROVKA PULVERIZED COAL-FIRING GRES (Capacity: 152,000 kw.)

Source: Elektricheskiye Stantsii, 1936, #1, p. 9 (TK4.E725)

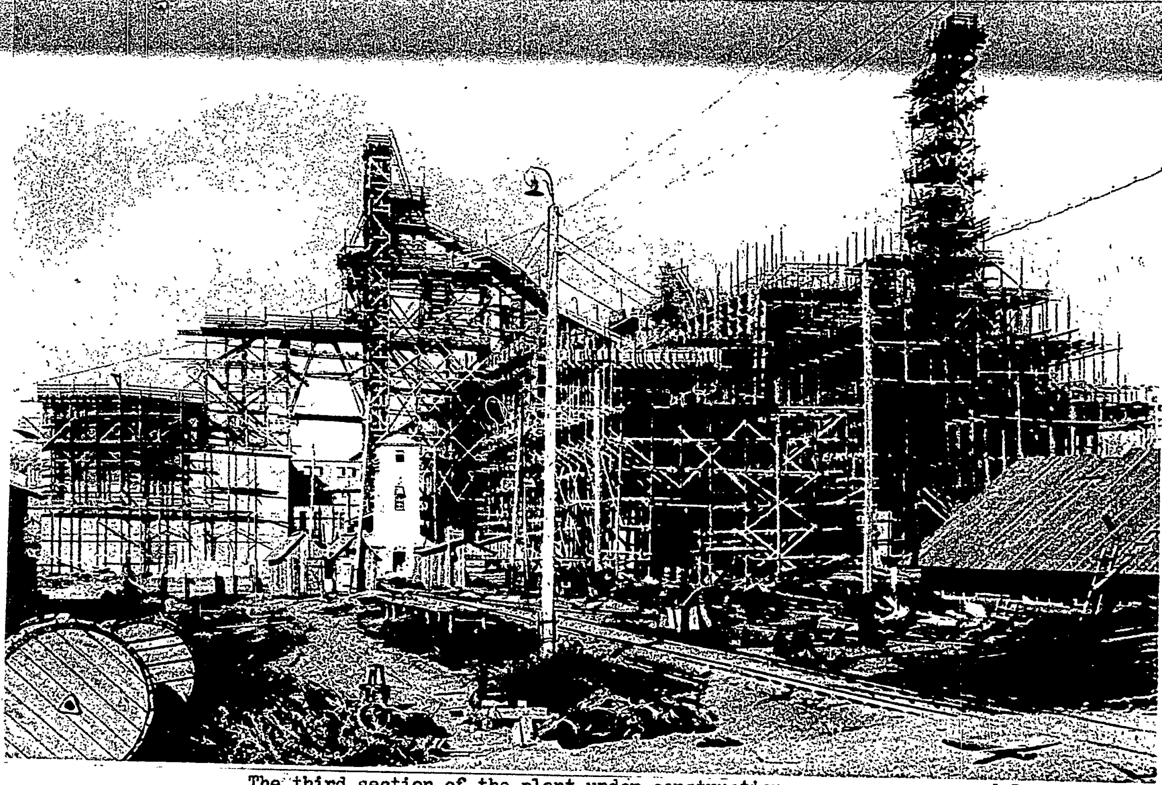
PLATE 30B

Boiler room under construction.

SHTEROVKA PULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: Prozhektor, 1930, #16, p. 29.

PLATE 30C

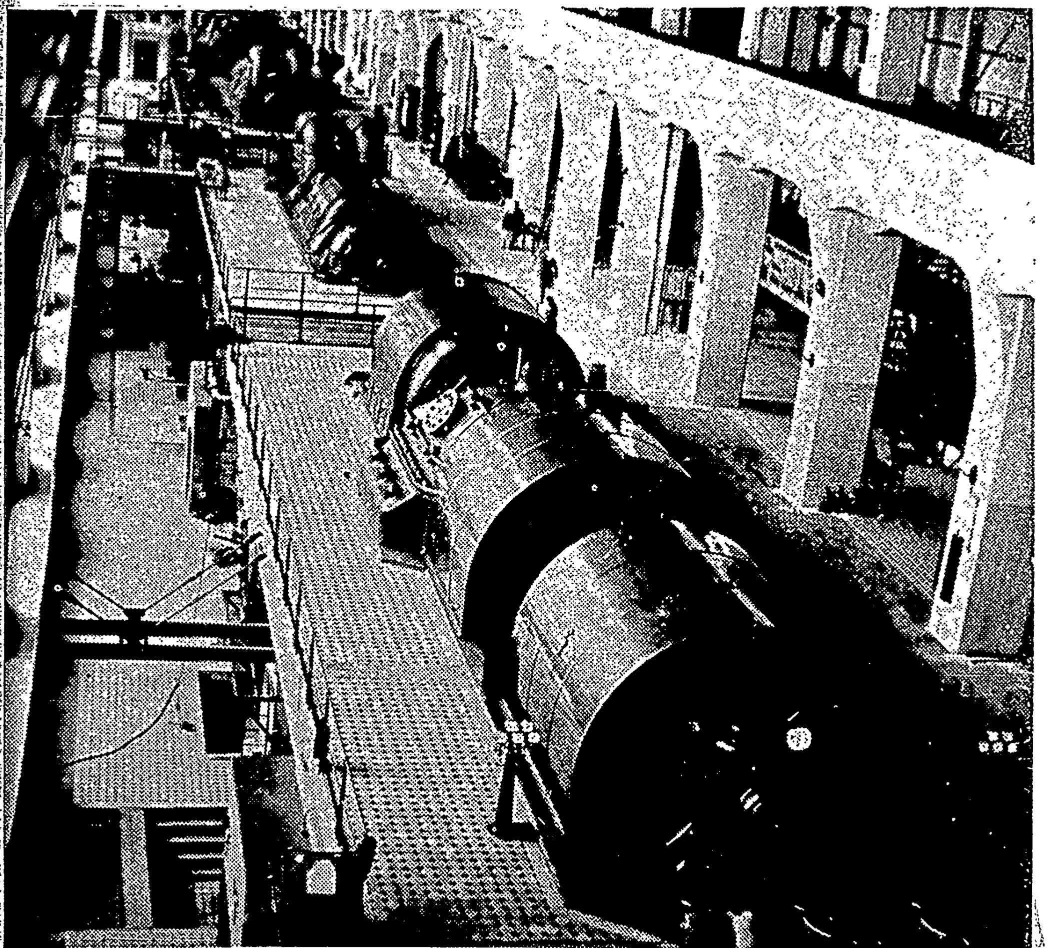


The third section of the plant under construction.

SHTEROVAKAPULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: USSR in Construction 1930, #3, p. 10 bottom

PLATE 30D



Partial view of the turbo-generator hall.

SHTEROVKA PULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: Economic Review of the Soviet Union 1932, #13-14, p. 294.

PLATE 30E

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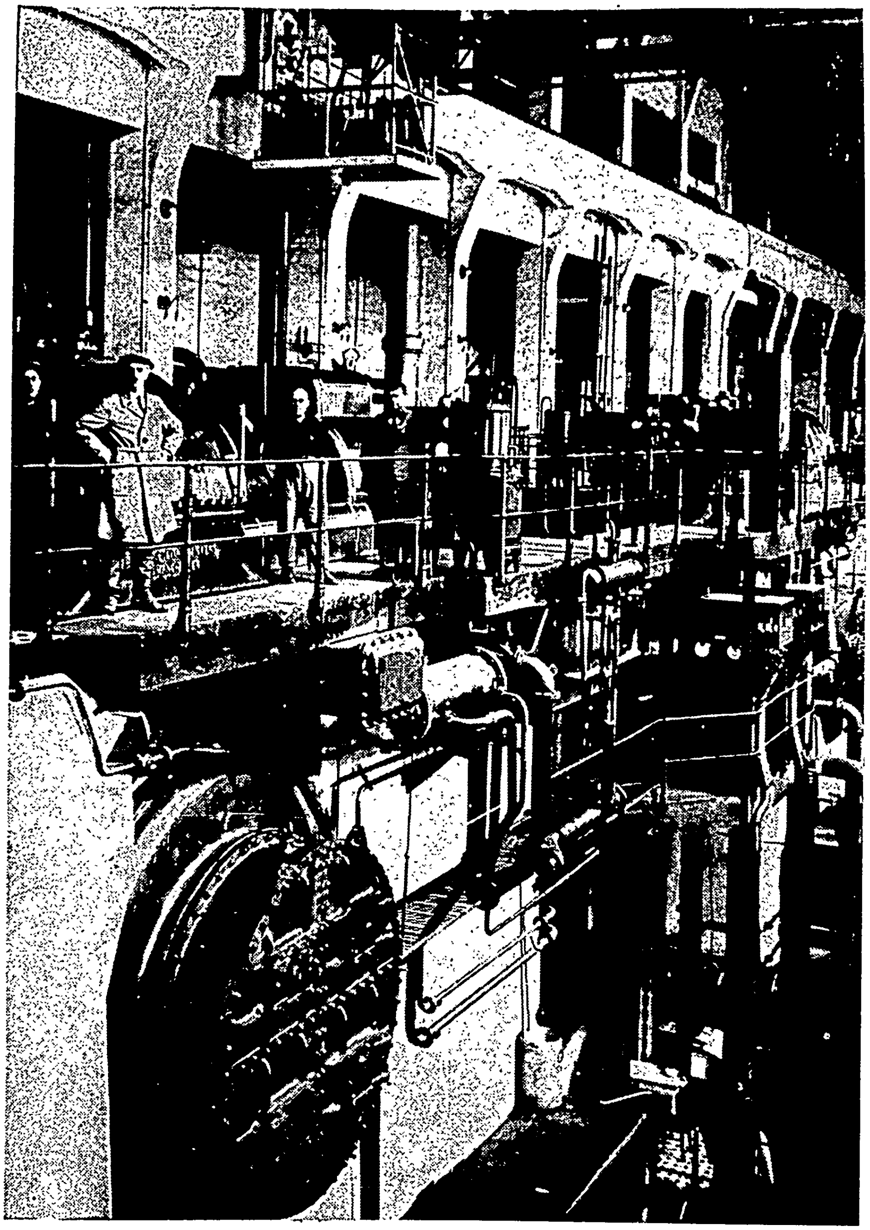
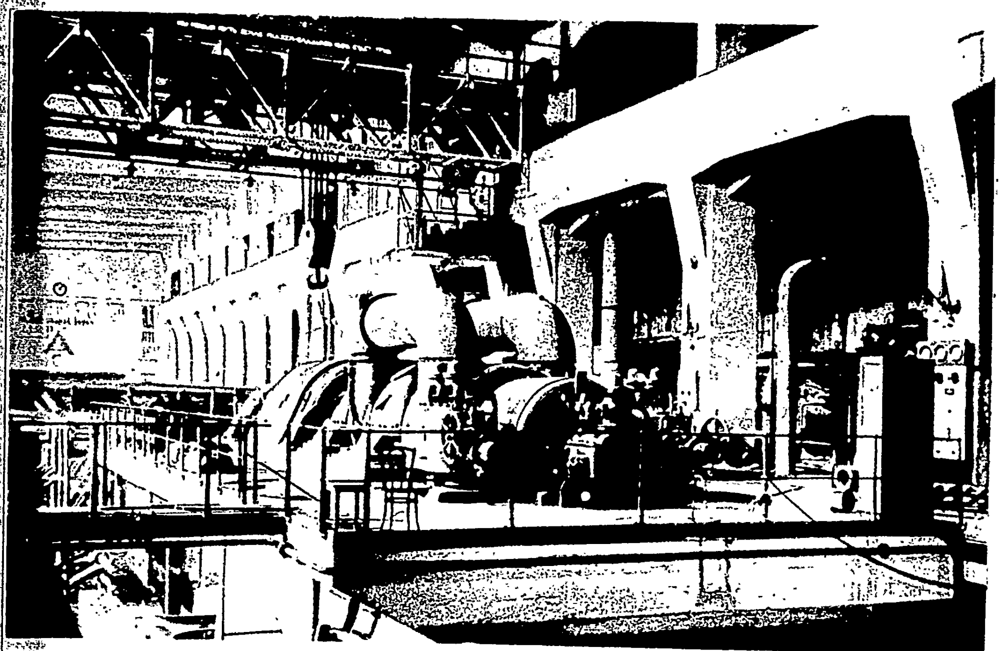


Side view of the generator hall.

SHTEROVKA PULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: Economic Review of the Soviet Union 1931, #3, p. 57.

PLATE 30F



Partial interior view of the turbine hall, second section.

SHTEROVKA PULVERIZED COAL-FIRING GRES
(Capacity: 152,000 kw.)

Source: USSR in Construction 1930, #3, p. 11 middle.

PLATE 30G

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ZUYEVKA PULVERIZED-COAL GRES

(Plates: 9, 9A; 31, 31A, 31B)

Location: In the Ukranian Donbass.

Coordinates: 48° 04' N, 38° 15' E.

Date of construction: Built in 1931, extended in 1935, and 1939, reconstructed after the war in 1949.

Layout type: First type design.

Installed capacity: 200,000 kw (1935).

Dimensions: Boiler house: W - 86 ft; H. to bot. of truss - 98 ft.
Feedwater pump section: W. - 24.6 ft; Total Height - 90 ft.
Bunker section: W - 29 ft. (est); Total Height - 94 ft. (est).

Structural type: Poured-in-place reinforced concrete frame.

Wall covering: Reinforced concrete panels (prob).

Window sash: Steel.

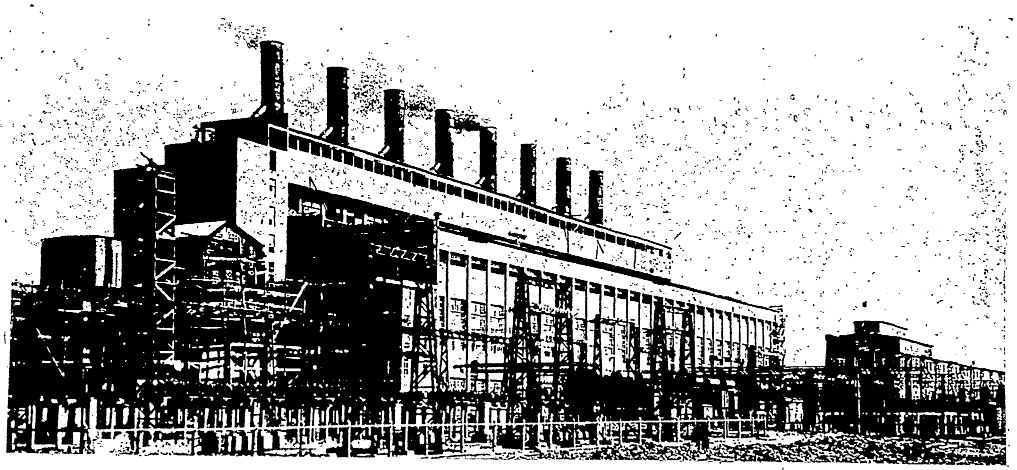
Roof construction: Steel trusses.

Roofing: Ruberoid (prob).

Cranes: In the turbine hall cap. 75/15 (prob).

Crane girders: Steel

Stacks: Eight steel on the roof of the building.



View of the GRES and open air sub-station.

-130-

ZUYEVKA PULVERIZED COAL-FIRING GRES
(Capacity: 200,000 kw.)

Source: Arkhitektura SSSR 1933, #3-4, p. 30 top, (NA6.A74)

PLATE 31

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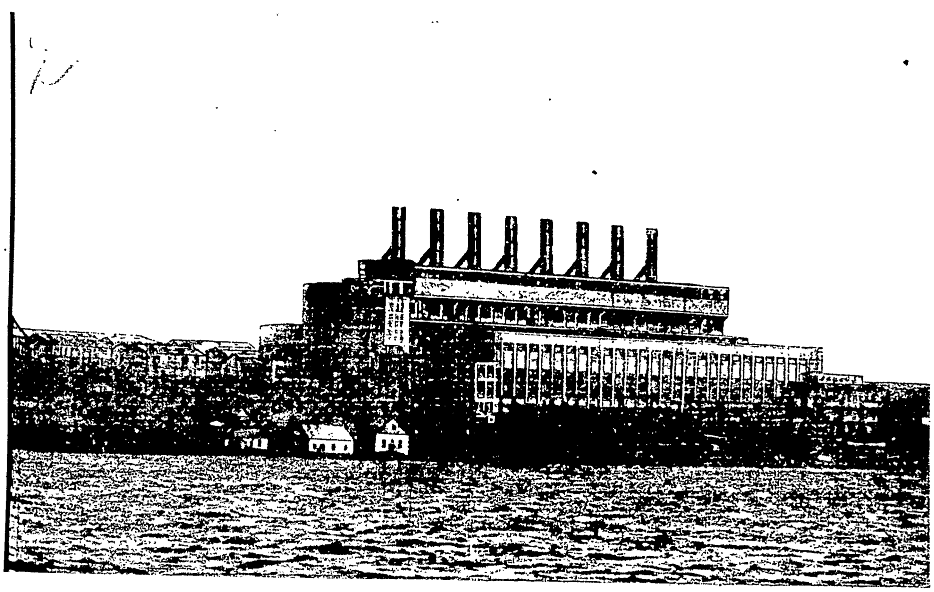

General view.

ZUYEVKA PULVERIZED COAL-FIRING GRES
(Capacity: 200,000 kw.)

Source: Elektricheskiye Stantsii, 1933, #3, front cover, (TK4.E725)

PLATE 31A

100,000 kw. Turbo-generator at the partially reconstructed Zuyevka GRES.

ZUYEVKA PULVERIZED COAL-FIRING GRES
(Capacity: 200,000 kw.)

Source: Trybuna Ludu 1950, #52, p. 6 center.

PLATE 31B

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IVANOVO (formerly IVANOVO-VOZNESEUSK) PEAT-FIRING GRES

Plates: (7, 32, 32A, 32B, 32C, 32D, 32E).

Location: City of Ivanovo (previously Ivanovo-Voznesensk) industrial center

Coordinates: 57° 00' N, 40° 59' E.

Date of construction: 1931.

Installed capacity: 113,000 kw.

Layout type: First type

Dimensions: Overall L - 490 ft, W - 177 ft., H - 116 ft.

Structural type: Reinforced concrete.

Wall covering: Brick curtain or panel wall.

Roof construction: Boiler room - steel trusses; bunker, feed-water pump and turbine sections - reinforced concrete beams and panels.

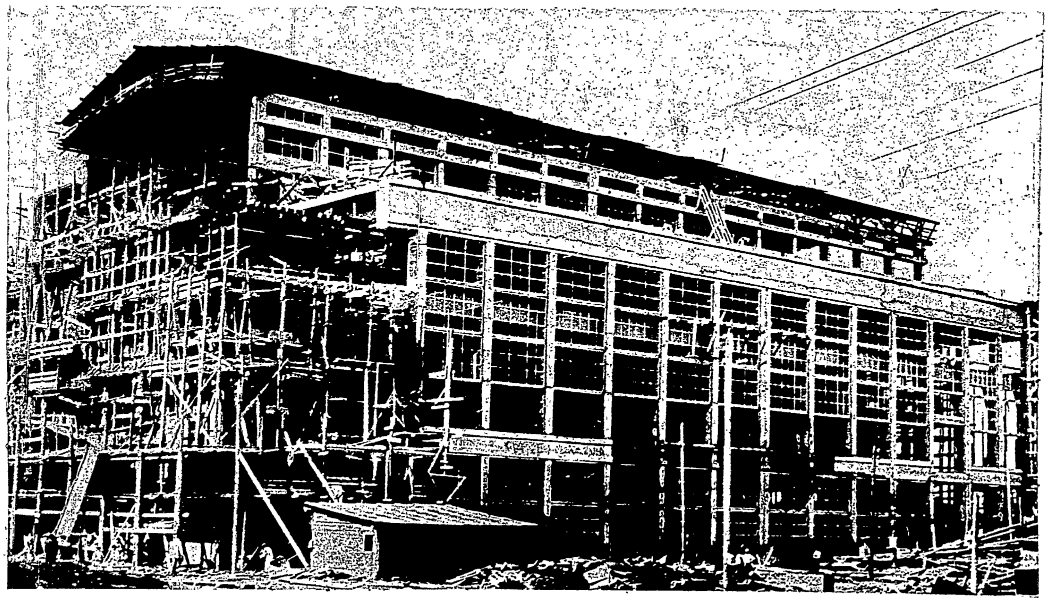
Roofing:

Cranes: In the turbine hall.

Crane girders: Reinforced concrete.

Stacks: Steel, located on the roof.

Fuel delivery: Over steel trestle

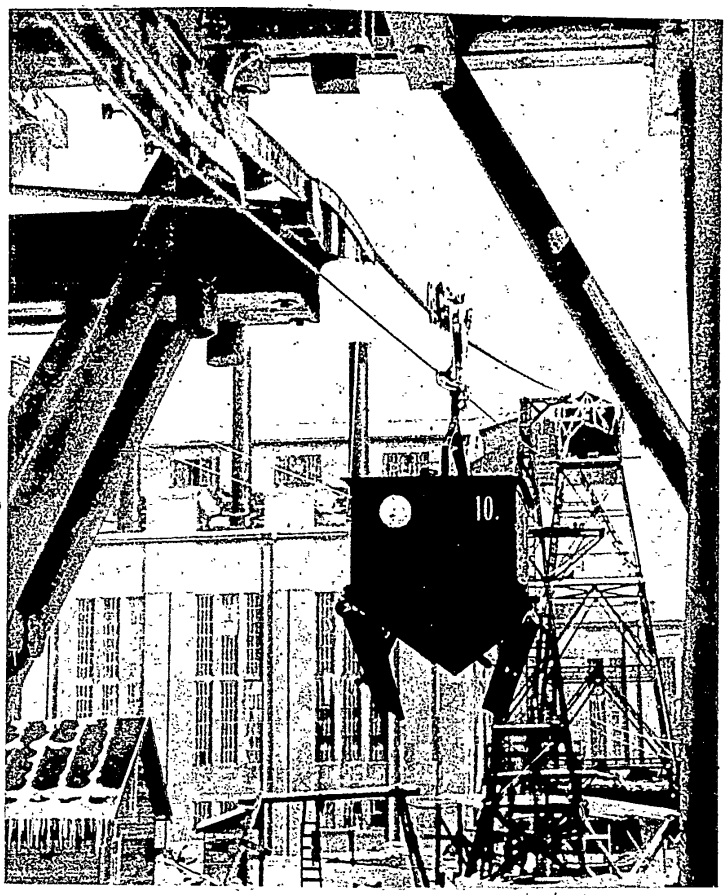


Side view of the main building under construction.

IVANOVO (formerly Ivanovo-Vooznesensk) PEAT-FIRING GRES (Capacity: 113,000 kw.)

Source: USSR in Construction 1930, #2, p. 13 middle.

PLATE 32

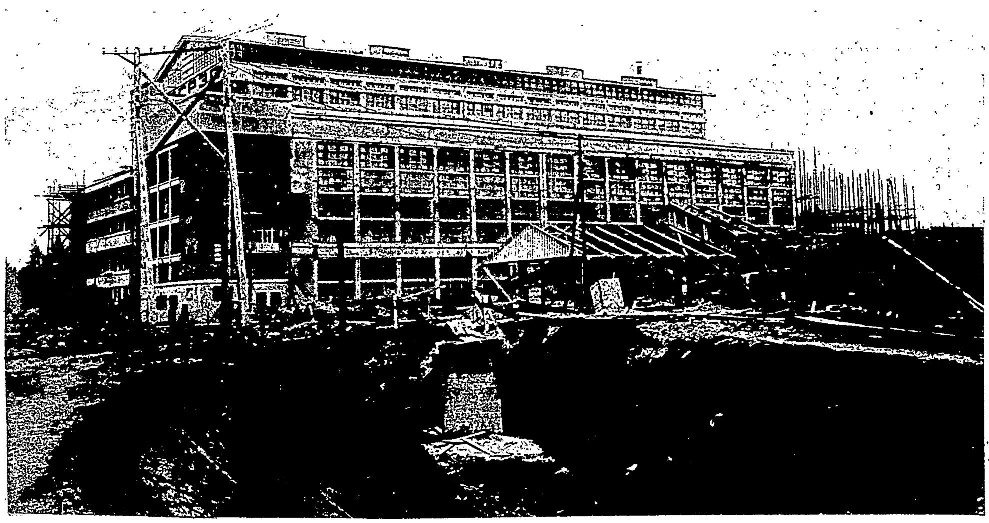


Construction work at the GRES.

IVANOVO (formerly Ivanovo-Voznesensk) PEAT-FIRING GRES
(Capacity: 113,000 kw.)

Source: Fünfzehn Eiserne Schritte, Berlin 1932, p. 61.

PLATE 32A



Main structure in the advanced stage of construction.

IVANOVO (formerly Inavovo-Voznesensk) PEAT-FIRING GRES
(Capacity: 113,000 kw.)

Source: USSR in Construction 1930, #3, p. 20 center.

PLATE 32B

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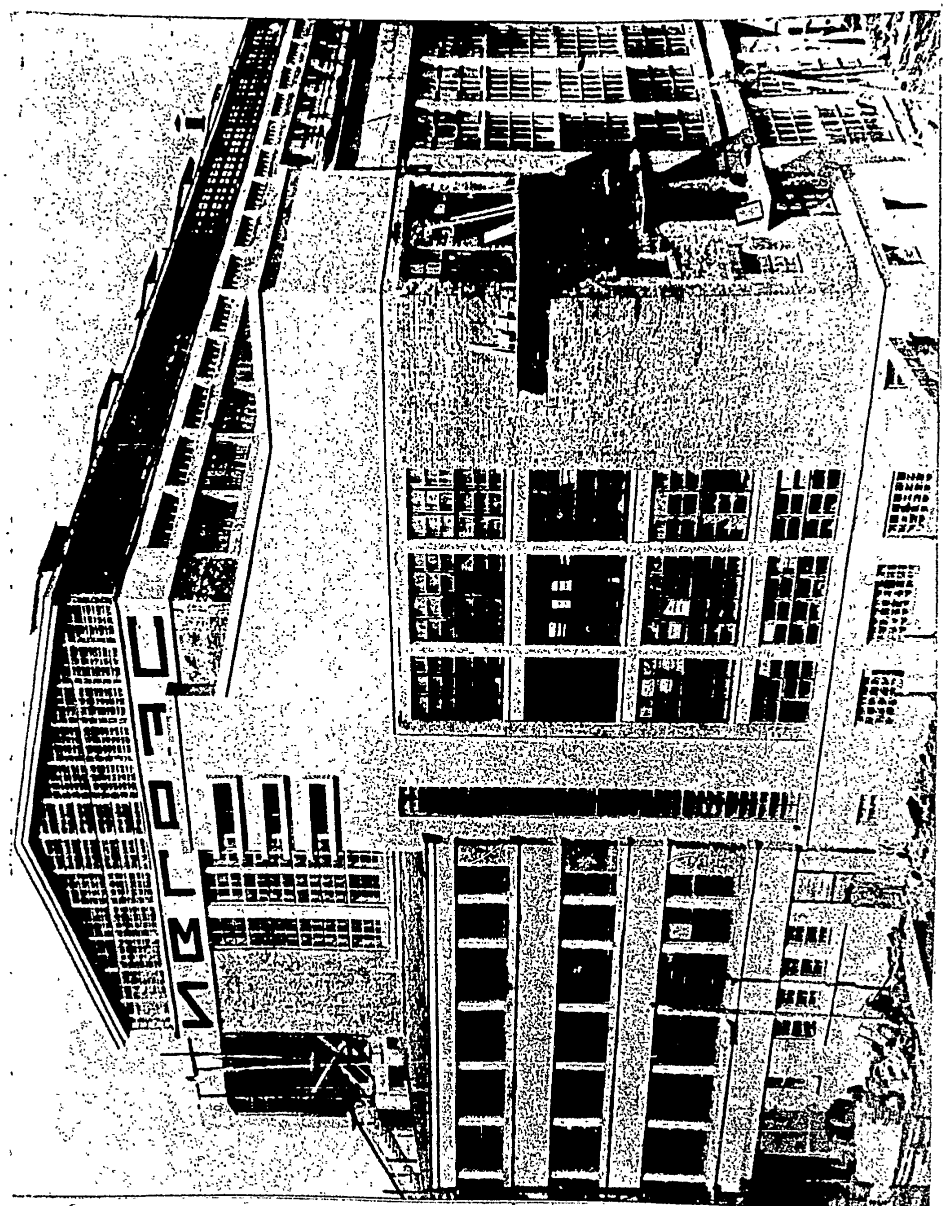
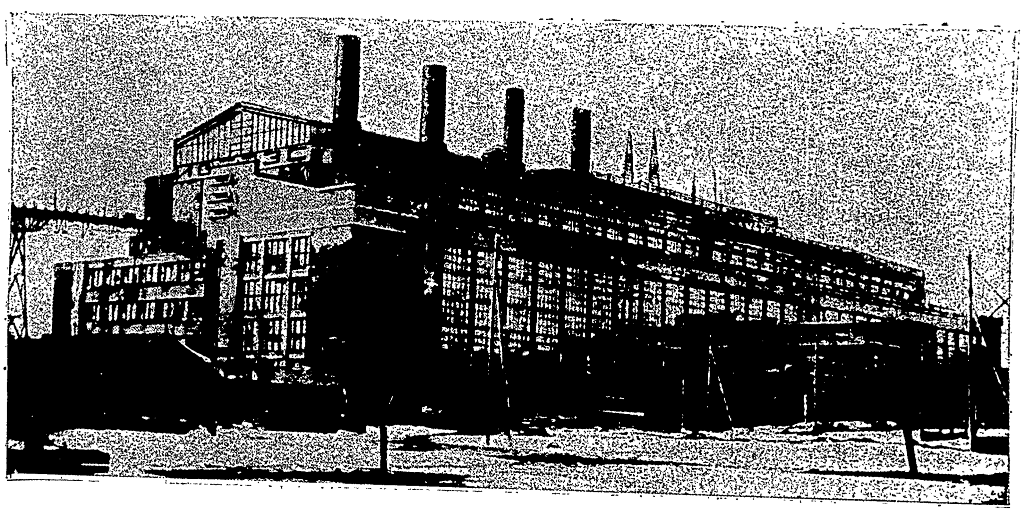


GRES under construction.

IVANOVO (formerly Ivanovo-Voznesensk) PEAT-FIRING GRES (Capacity: 113,000 kw.)

Source: Prozhektor 1930, #17, p. 26.

PLATE 32C



Side View.

IVANOVO (formerly Ivanovo-Voznesensk) PEAT-FIRING GRES
(Capacity: 113,000 kw.)

Source: Arkhitektura elektrostantsiy 1939, p. 168 bottom, (TH4581.A5)

PLATE 32D

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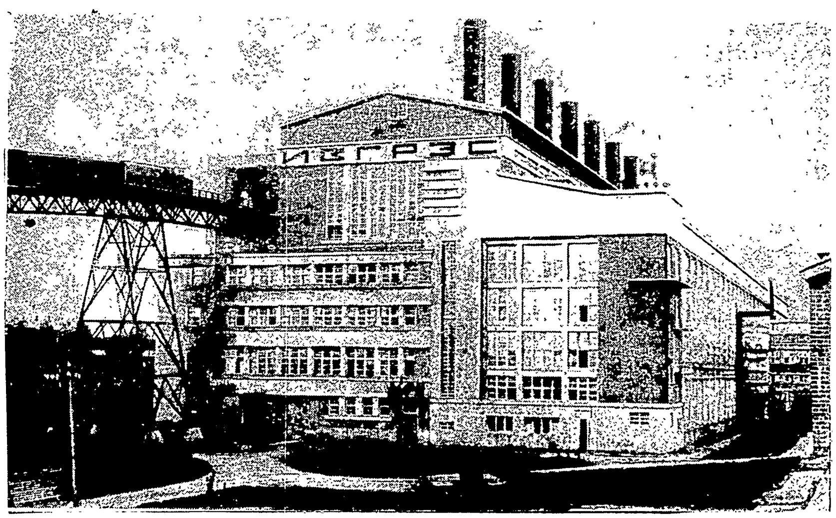

IVANOVO (formerly Ivanovo-Voznesensk) PEAT-FIRING GRES
(Capacity: 113,000 kw.)

Source: Pyatnadsat' let leninskogo plana elektrifikatsii 1936, p. 57
(TK85.D6)

PLATE 32E

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STALINOGORSK COAL-FIRING GRES

(Plates: 12, 12A, 33, 33A, 33B, 33C, 33D, 33E)

Location: Stalinogorsk in Moscow Region.

Coordinates: 54° 05' N, 38° 14' E.

Date of construction: 1934 - expanded in 1936 (auxiliary building).

Installed capacity: 150,000 kw.

Dimensions: Overall: W. 220 ft.
Main building: Bunker section - W. 36 ft.
Boiler house - W. 90 ft.
Feedwater tank section - W. 27.8 ft.
Turbine hall - W. 66.4 ft. H. to truss - 39 ft. (est).

Layout type: First type design.

Structural type: Poured-in-place reinforced concrete frame.

Wall covering: Brick curtain walls and reinforced concrete panels (est).

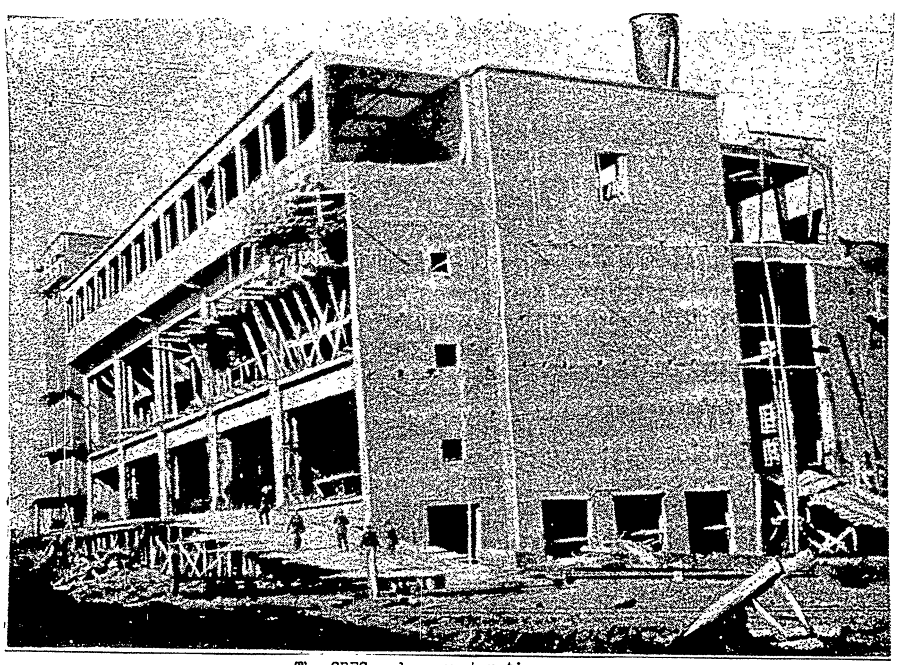
Roof construction: Steel trusses (boiler house and turbine hall); reinforced concrete beams (bunker section, feedwater tank section, switch house)

Roofing: Ruberoid (prob).

Cranes: In the turbine hall, cap. 75/15 (est).

Crane girders: Steel (est).

Stacks: Steel on the roof of the building.



The GRES under construction.

STALINOGORSK COAL-FIRING GRES
(Capacity: 100,000 kw.)

Source: Aviatsiya i Khimiya 1931, #10-11, p. 33, (TL 504.Z3)

PLATE 33

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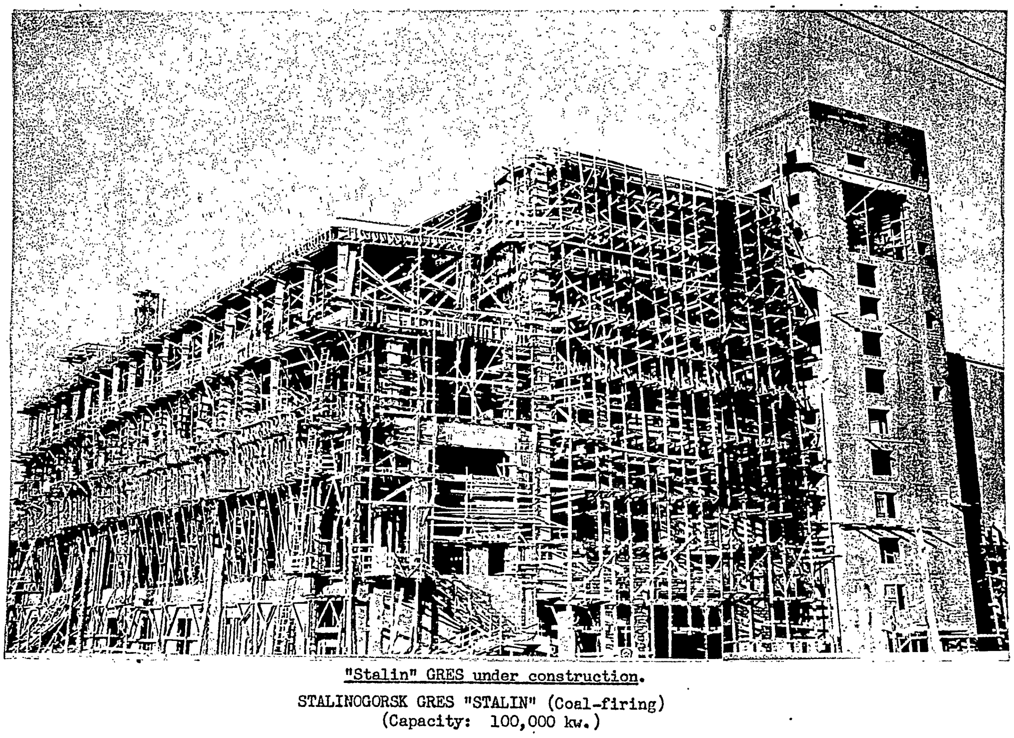


"Stalin" GRES under construction.

STALINOGORSK GRES "STALIN" (Coal-firing)
(Capacity: 100,000 kw.)

Source: USSR in Construction 1934, #1, p. 24 top.

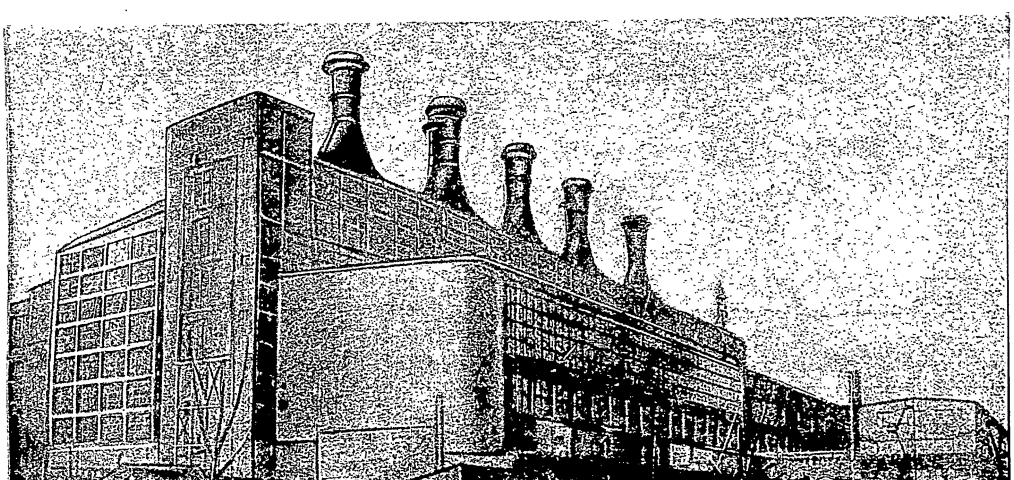
PLATE 33A

Exterior view.

STALINOGORSK COAL-FIRING GRES (Capacity: 100,000 kw.)

Source: Prozhektor 1932, #9-10, p. 33, left third from top.

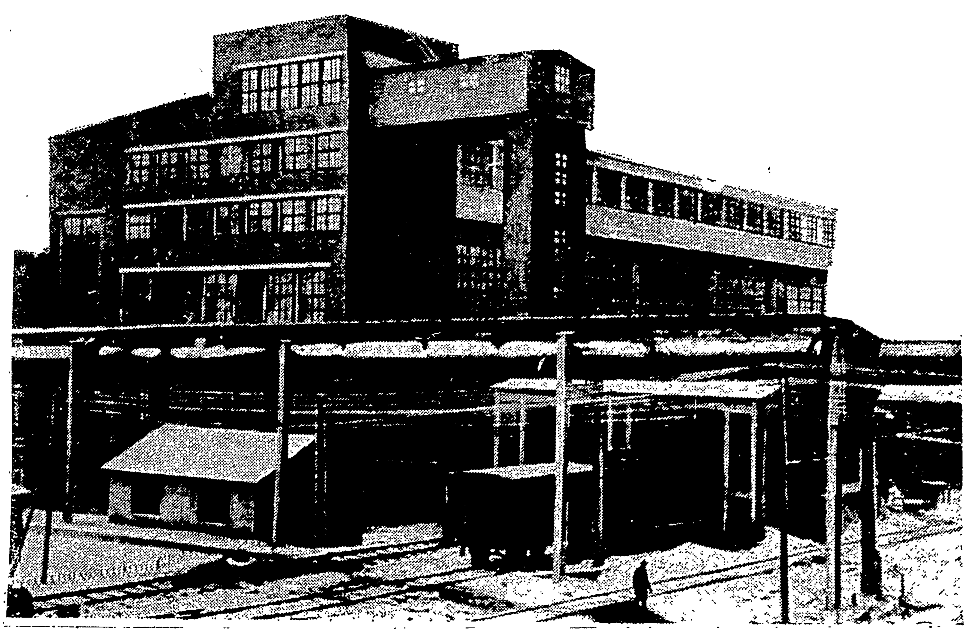
PLATE 33B



Exterior view.

STALINOGORSK COAL-FIRING GRES
(Capacity: 100,000 kw.)
Source: Elektricheskiye Stantsii 1936, #2, front cover, (TK4.E725)

PLATE 33C

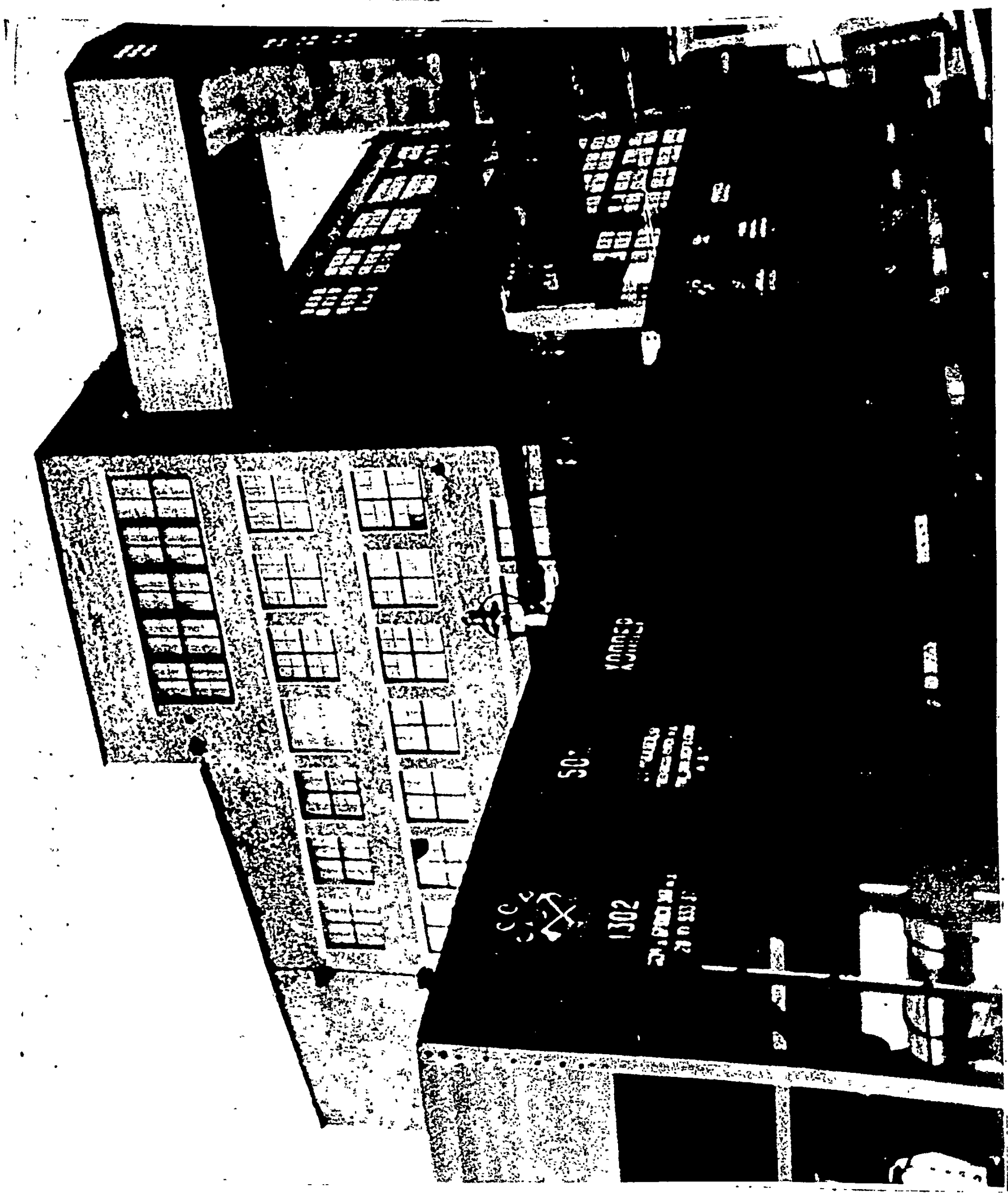


General view.

STALINOGORSK AUXILIARY POWER PLANT

Source: Pafos Osvoyenia, p. 83, (T26.R9M6)

PLATE 33D



Partial view of auxiliary power plant.

STALINOGORSK AUXILIARY POWER PLANT

Source: USSR in Construction 1934, No. 1, back cover.

PLATE 33E

STALINGRAD PULVERIZED COAL-FIRING GRES

(Plates: 10, 34, 34A, 34B)

Location: City of Stalingrad on the bank of Volga River

Coordinates: 48° 42' N, 44° 30' E.

Date of construction: 1930

Installed capacity: 75,000 kw. (1934)

Layout type: First type.

Outside dimensions (available): W - 162.5 ft.; H - 106.0 ft.

Structural type: Reinforced concrete.

Wall covering:

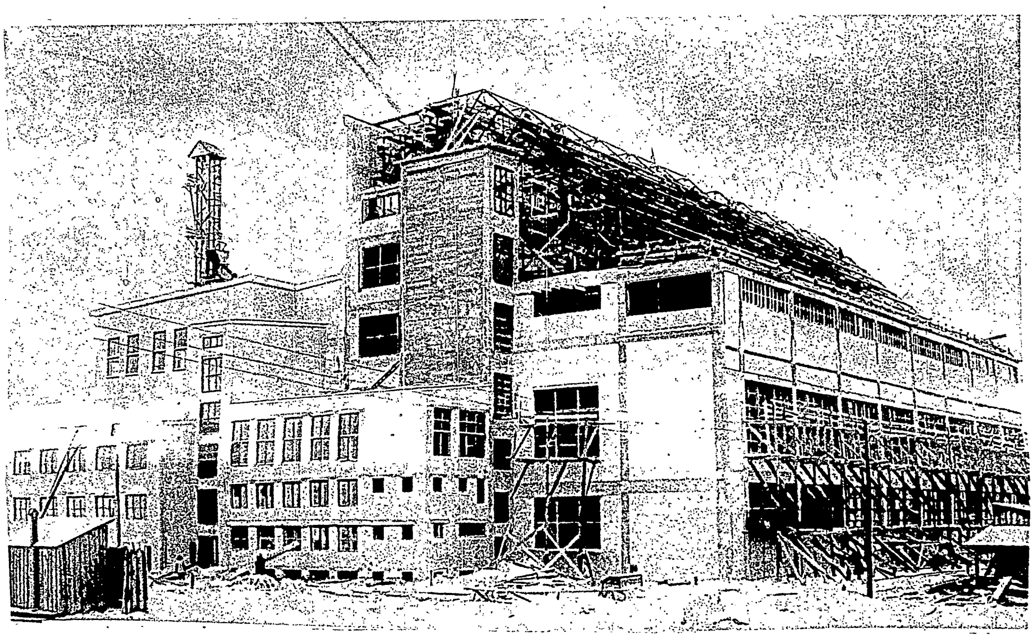
Roof construction: 1) Boiler room - Warren parallel chord steel trusses.
2) Bunker section, feed-water pump section and turbine hall - reinforced concrete beams and slabs.

Roofing: Presumably ruberoid.

Crane: In the turbine hall.

Crane girders: In the turbine hall - reinforced concrete

Stacks: Steel, located on the roof.

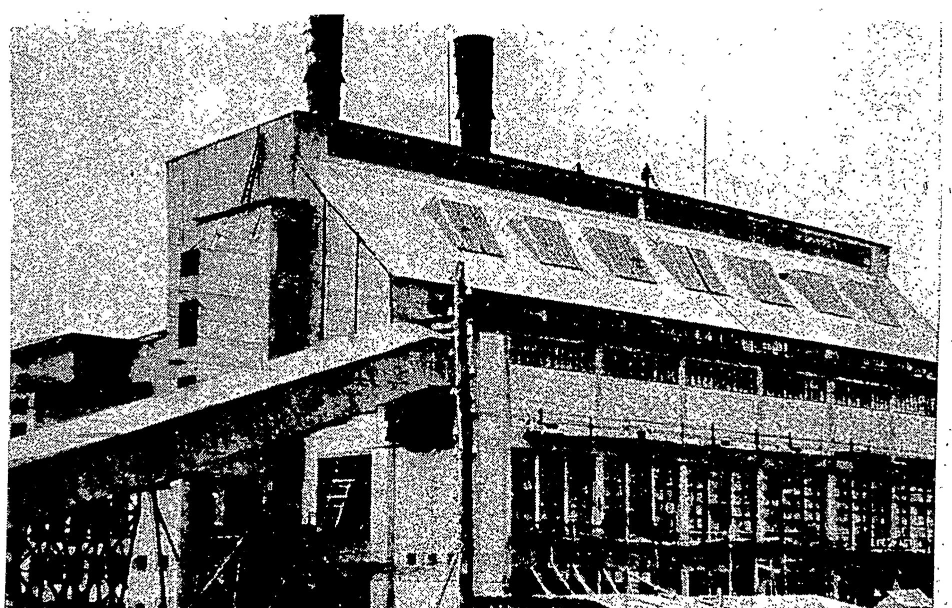


Main building under construction.

STALINGRAD PULVERIZED COAL-FIRING GRES
(Capacity: 75,000 kw.)

Source: USSR in Construction 1930, #3, p. 21 middle.

PLATE 34

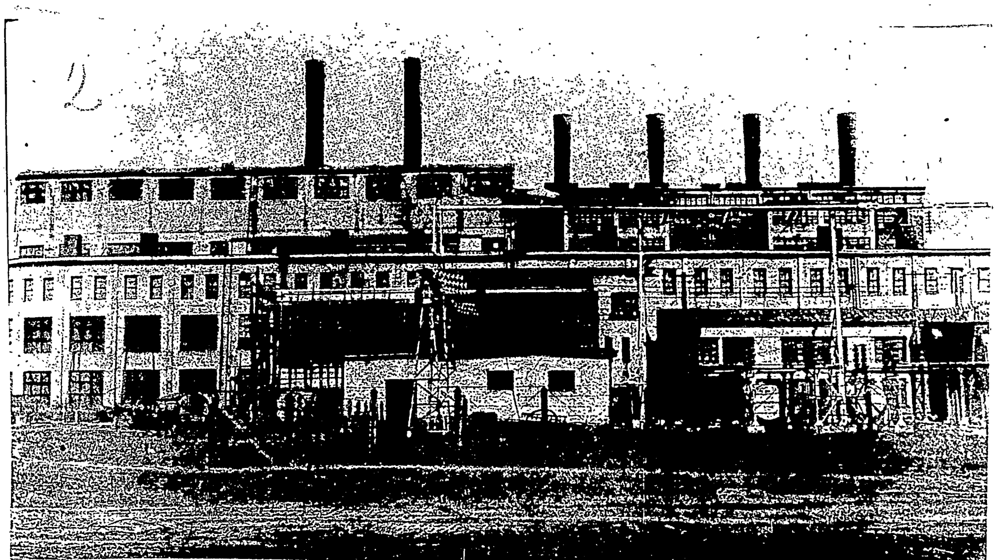


Partial exterior view.

STALINGRAD PULVERIZED COAL-FIRING GRES
(Capacity: 75,000 kw.)

Source: Prozhektor 1930, #35, p. 6 bottom left.

PLATE 34A



Exterior view.

STALINGRAD PULVERIZED COAL-FIRING GRES
(Capacity: 75,000 kw.)

Source: Elektricheskiye Stantsii 1936, #8, front cover, (TK4.E725)

PLATE 34B

STALINGRAD TRACTOR PLANT TETS

(Plates: 34C)

Location: In the northern part of the city of Stalingrad, on the bank of the Volga River

Coordinates: 48° 42' N, 44° 30' E.

Date of construction: Destroyed during the war, reconstructed during the Fourth Piatiletka (1946-1950)

Layout type: First type.

Installed capacity: 19,000 kw. (1936).

Dimensions:

Structural type: Appears to be a combination of steel and brick. Steel frame with continuous steel sash windows - for side walls. Brick wallbearing construction for front walls.

Wall covering:

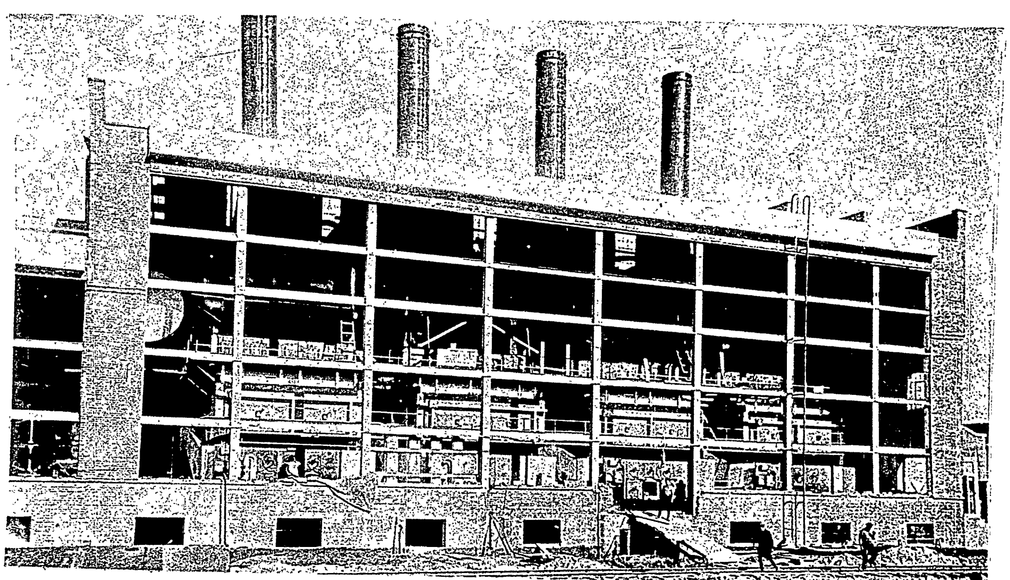
Roof construction:

Roofing:

Cranes: In turbine hall

Crane girders: Steel

Stacks: Steel, located on the roof.



View of the boiler room.

STALINGRAD TRACTOR PLANT POWER STATION (pulverized coal-firing)
(Capacity: 19,000 kw.)

Source: Economic Review of the Soviet Union 1931, #17, p. 395.

PLATE 34C

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WHITE RUSSIAN PEAT-FIRING GRES.

(Plates: 35, 35A, 35B, 35C, 35D)

Location: Near or in Orekhovsk, White Russia.

Coordinates: 54° 42' N, 30° 30' E.

Date of construction: 1930

Layout type: First type.

Installed capacity: 20,000 kw.

Dimensions:

Structural type: Reinforced concrete frame. Brick panel walls.

Wall covering:

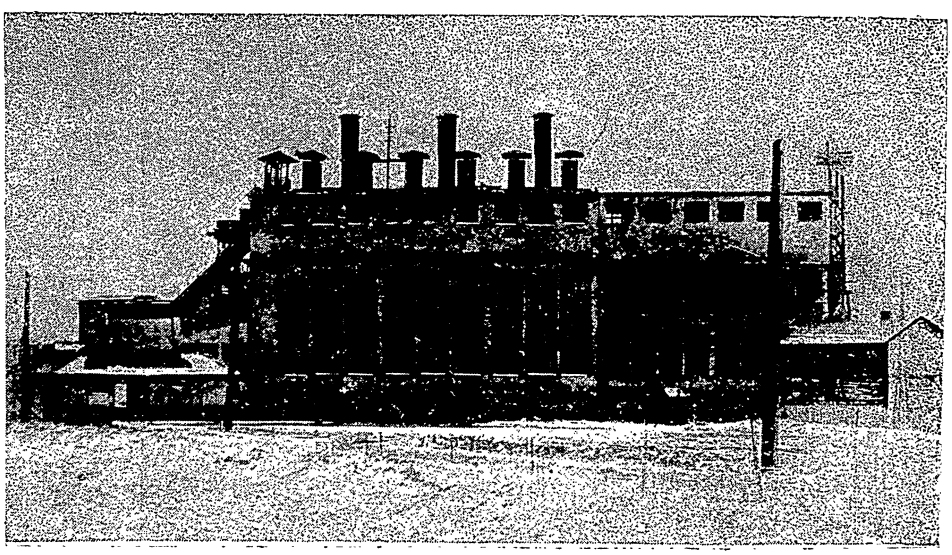
Roof construction:

Roofing:

Cranes:

Crane girders:

Stacks: Steel, located on the roof.

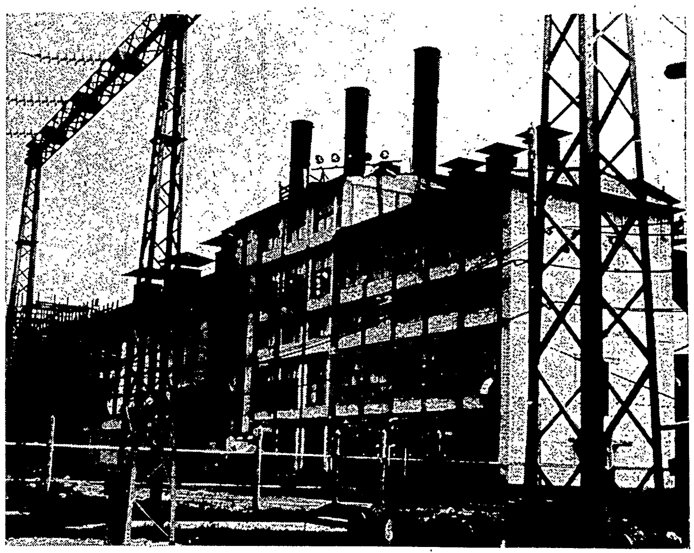


General view.

PEAT-FIRING GRES IN OR NEAR OREKHOVSK, WHITE RUSSIA
(Capacity: 20,000 kw. - 1931)

Source: Elektricheskiye Stantsii 1936, #1, p. 19, (TK4.E725)

PLATE 35



The first section of the plant.

PEAT-FIRING GRES IN OR NEAR OREKHOVSK, WHITE RUSSIA.
(Capacity: 20,000 kw. - 1931)

Source: Economic Review of the Soviet Union 1932, #7, p. 151

PLATE 35A

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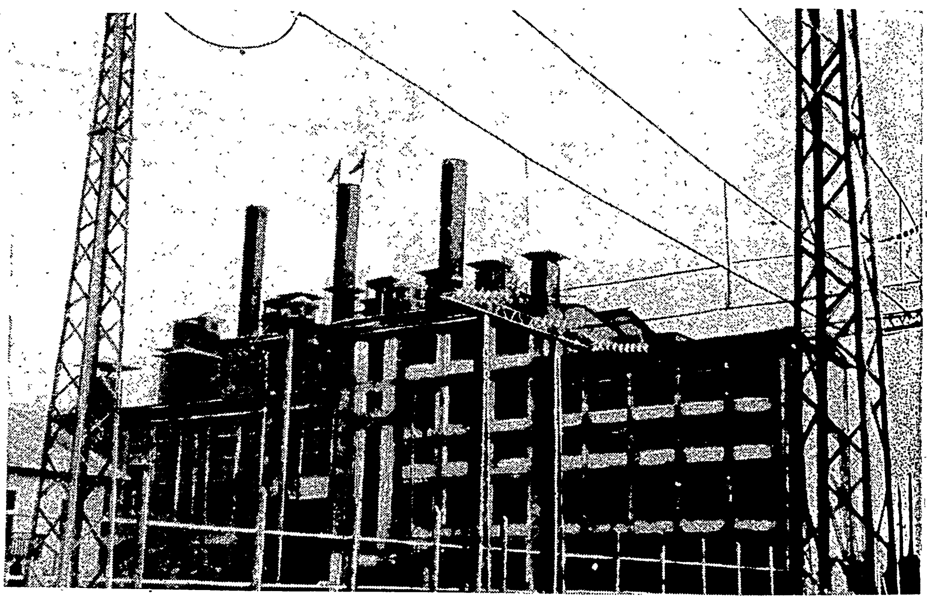
-156-

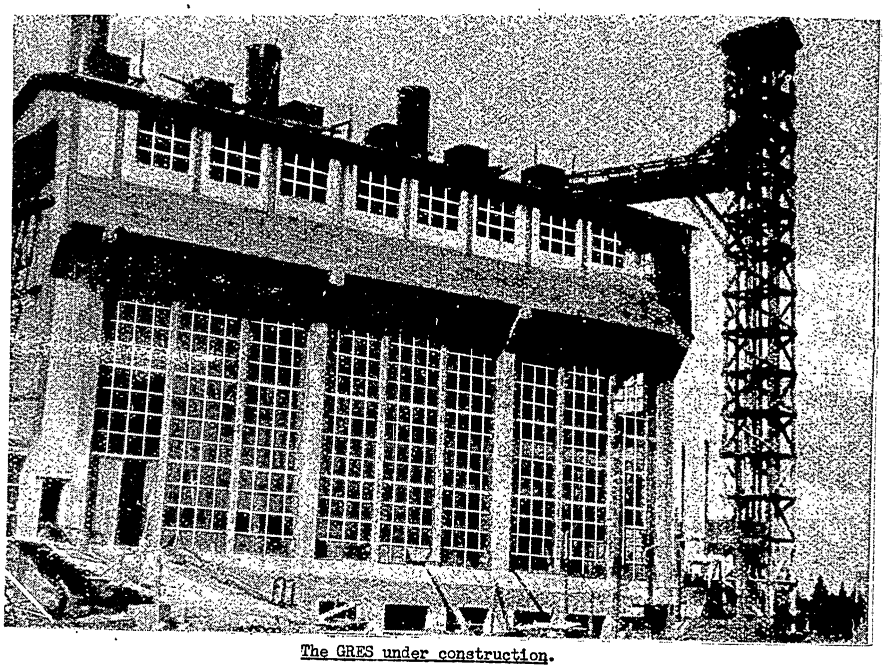


Exterior view.

PEAT-FIRING GRES IN OR NEAR OREKHOVSK, WHITE RUSSIA
(Capacity: 20,000 kw.)

Source: Prozhektor 1930, #35, p. 6 bottom right.

PLATE 35B

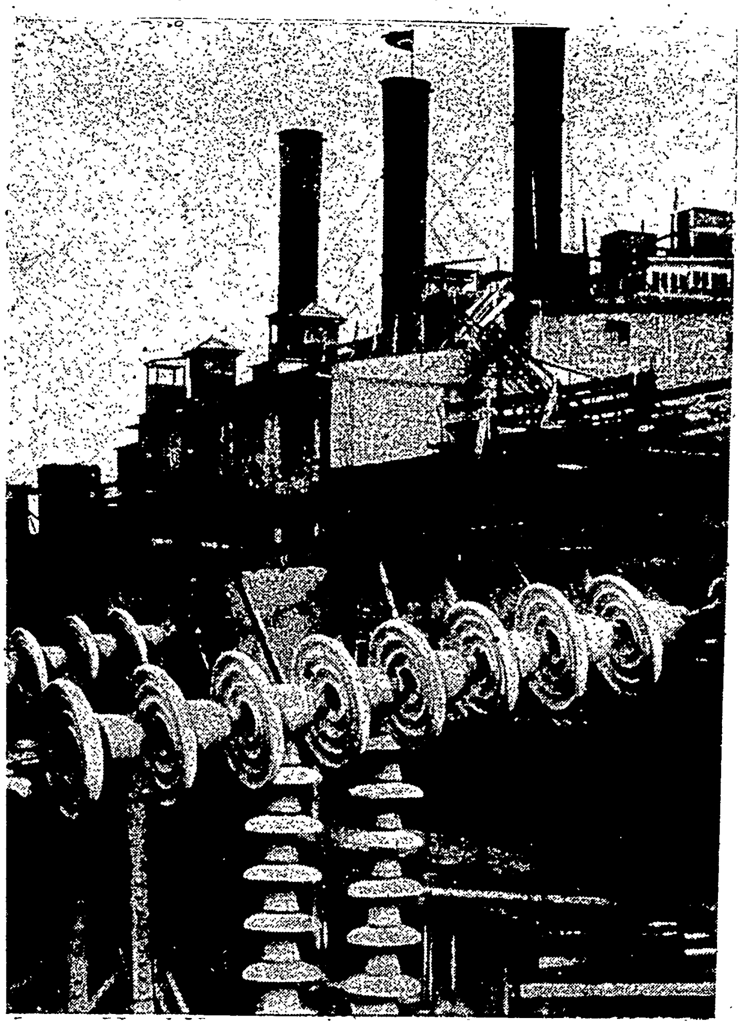


The GRES under construction.

PEAT-FIRING GRES IN OR NEAR OREKHOVSK, WHITE RUSSIA
(Capacity: 20,000 kw. - 1931)

Source: Prozhektor: 1930, #26-27, p. 22 center left.

PLATE 35C



Partial view of the roof.

PEAT-FIRING GRES IN OR NEAR OREKHOVSK, WHITE RUSSIA
(Capacity: 20,000 kw.)

Source: Prozhektor, #29, p. 30 top center.

PLATE 35D

BEREZNIKI PULVERIZED COAL-FIRING TETS

(Plates: 36, 36A, 36B, 36C)

Location: On the site of the Berezniki Chemical Works.

Coordinates: 59° 21' N, 56° 40' E.

Date of construction: 1931.

Layout type: First type.

Installed capacity: 93,000 kw.

Dimensions:

Structural type: Reinforced concrete and steel.

Wall covering: On the side wall - continuous sash windows and reinforced concrete panels

Roof construction: In turbine hall - steel trusses

Roofing:

Cranes: In the turbine hall

Crane girders: Reinforced concrete.

Stacks: Steel, located on the roof.

Trestles for fuel delivery: Steel

Remarks: Two 10,000 kva. step-up transformers are installed in the open air sub-station; the Berezniki TETS is thus connected with the Kizelovsk and Solikamsk power stations.

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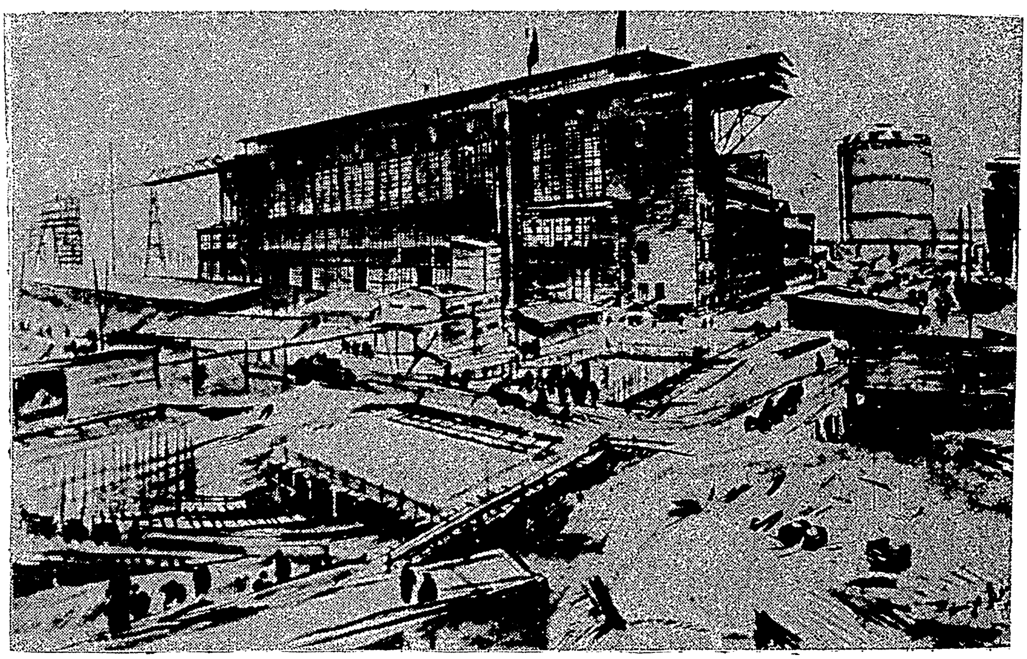
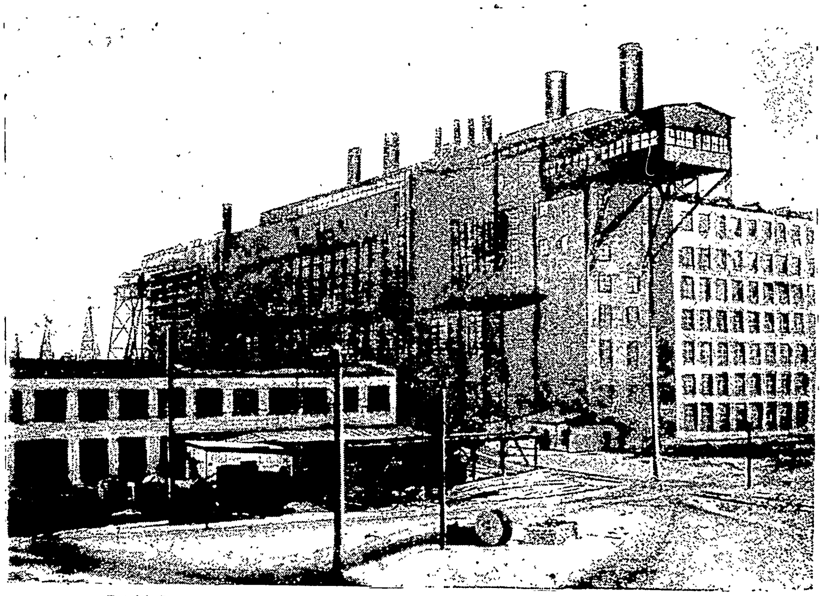


Bereznikі TETs under construction.

BEREZNIKI PULVERIZED-COAL FIRING TETs
(Capacity: 93,000 kw.)

Source: Khimstroy 1932, #1, p. 1379, (TPl.K5)

PLATE 36



Partial exterior view of the TETs serving the Berezniki Chemical works.

BEREZNIKI TETs (pulverized coal-firing)
(Capacity: 93,000 kw.)

Source: Pyatnadsat' let leninskogo plana elektrifikatsii, 1936, p. 51, (TK85.D6)

PLATE 36A

Exterior view of the TETs serving the Berezniki Chemical Works.

BEREZNIKI PULVERIZED COAL-FIRING TETs
(Capacity: 93,000 kw.)

Source: USSR in Construction, 1932, #5, p. 26.

PLATE 36B

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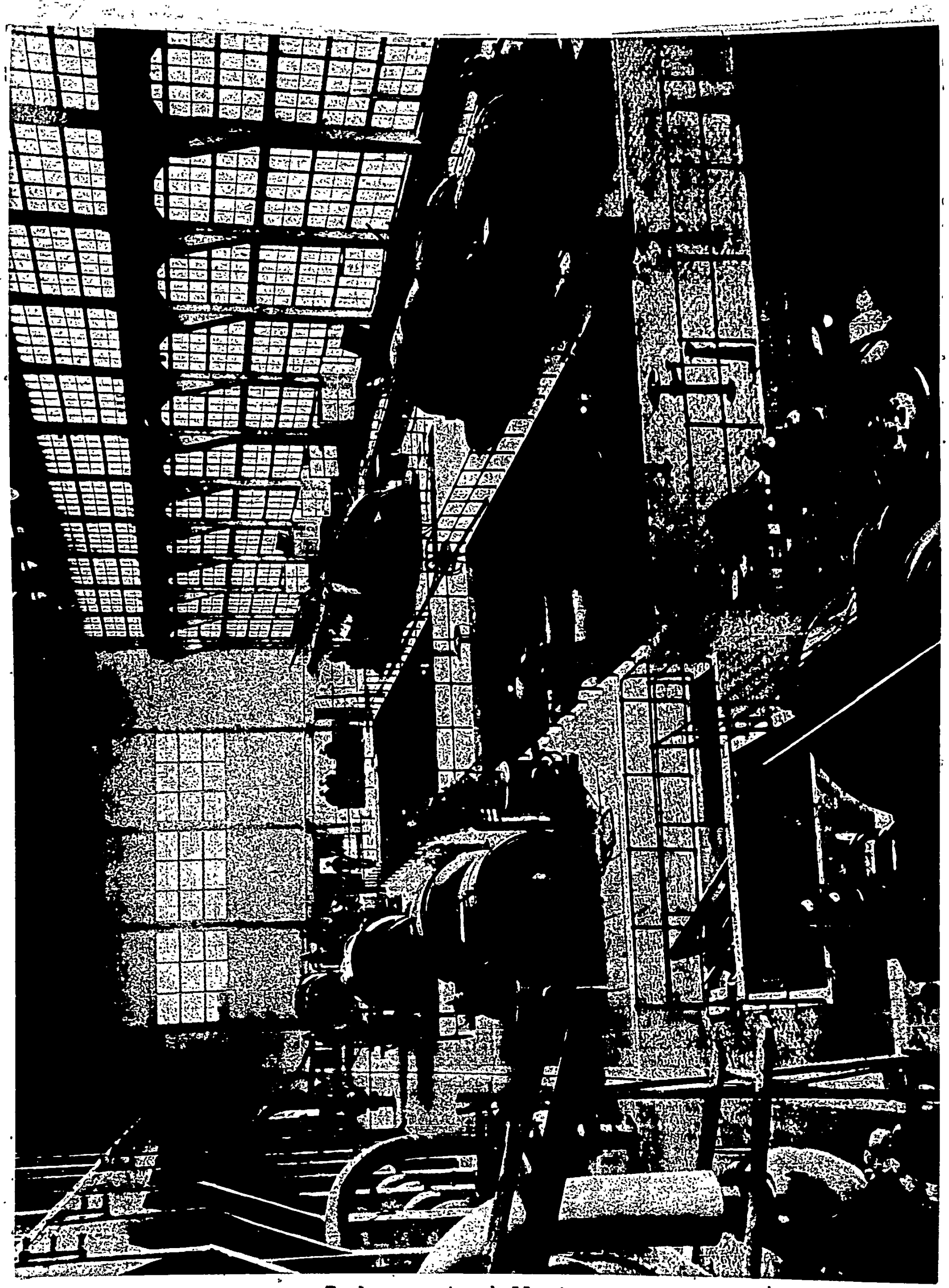

Turbogenerator hall.

BEREZNIKI TETs (pulverized coal-firing)
(Capacity: 93,000 kw.)

Source: DK267.A1U1, 1932, #5, p. 26.

PLATE 36C

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SOLIKAMSK COAL-FIRING GES

(Plates: 37)

Location: Apparently on the site of the Solikamsk Chemical Plant.

Coordinates: 59° 38' N, 56° 47' E.

Date of construction: Prior to 1932.

Layout type: First type.

Installed capacity:

Dimensions:

Structural type: Reinforced concrete, poured-in-place.

Wall covering: Probably brick panel walls.

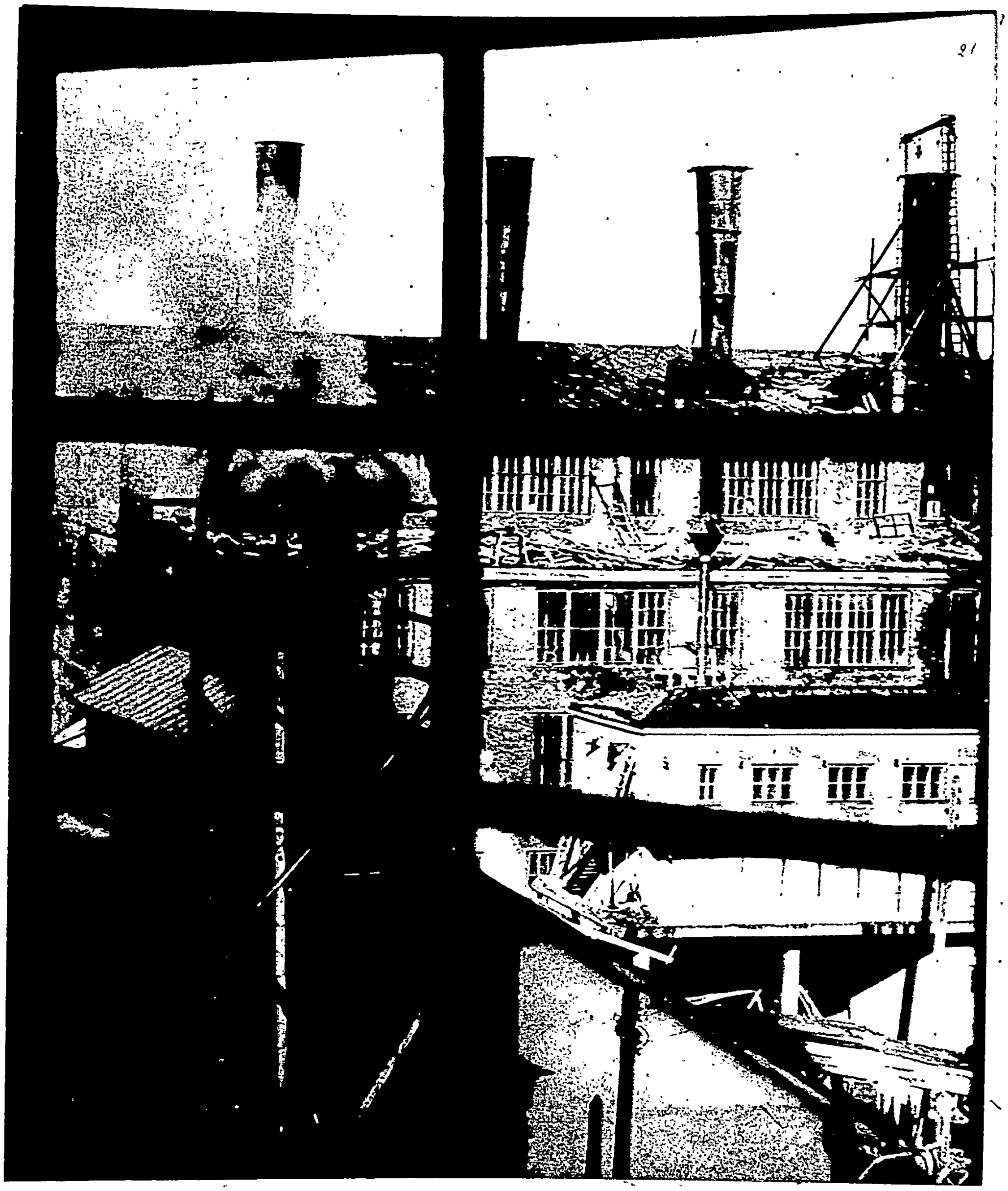
Roof construction:

Roofing: Sheet iron.

Cranes:

Crane girders:

Stacks: Steel, located on the roof.



Partial view from the window of another building.

SOLIKAMSK COAL-FIRING GES

Source: USSR in Construction 1932, #5, p. 21.

PLATE 37

KUZNETSK COAL AND BLAST FURNACE GAS-FIRING GES

(Plates: 11, 11a, 38, 38A, 38B, 38C, 38D)

Location:

Coordinates: 55° 00' N, 85° 00' E. —— (approximate)

Date of construction: 1931.

Installed capacity: 109,000 kw.

Layout type: First type.

Dimensions: Overall - L. - 462.5 ft.; W. - 193.7 ft.; H. - 136,0 ft.

Structural type: Poured-in-place reinforced concrete,

Wall covering:

Roof construction: Steel trusses in the boiler house and turbine hall; reinforced conrete beams in bunker and feed water pump section.

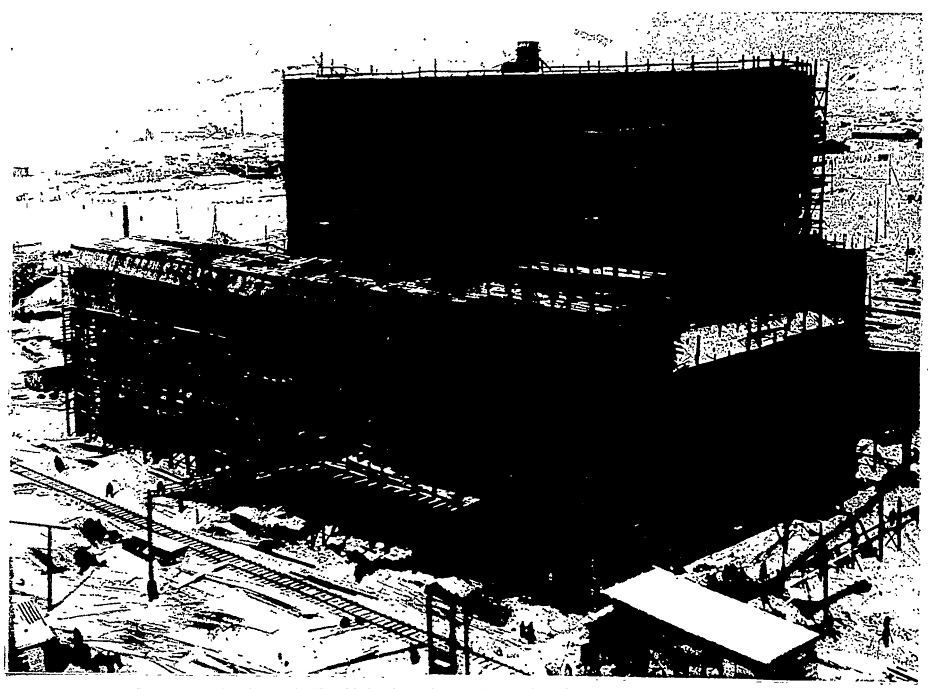
Roofing: Presumably ruberoid.

Cranes: In the turbine hall; probable capacity - 75/15 tons.

Crane girders: Reinforced concrete.

Stacks: Steel, located on the roof.

Switch house: Reinforced concrete.



Central power station at the Metallurgical Plant in the process of construction.

KUZNETSK COAL AND BLAST FURNACE GAS FIRING TETs
(Capacity: 60,000 kw. - Projected 109,000 kw.)

Source: USSR in Construction 1932, #4, p. 19 bottom, (DK267.A1U3)

PLATE 38

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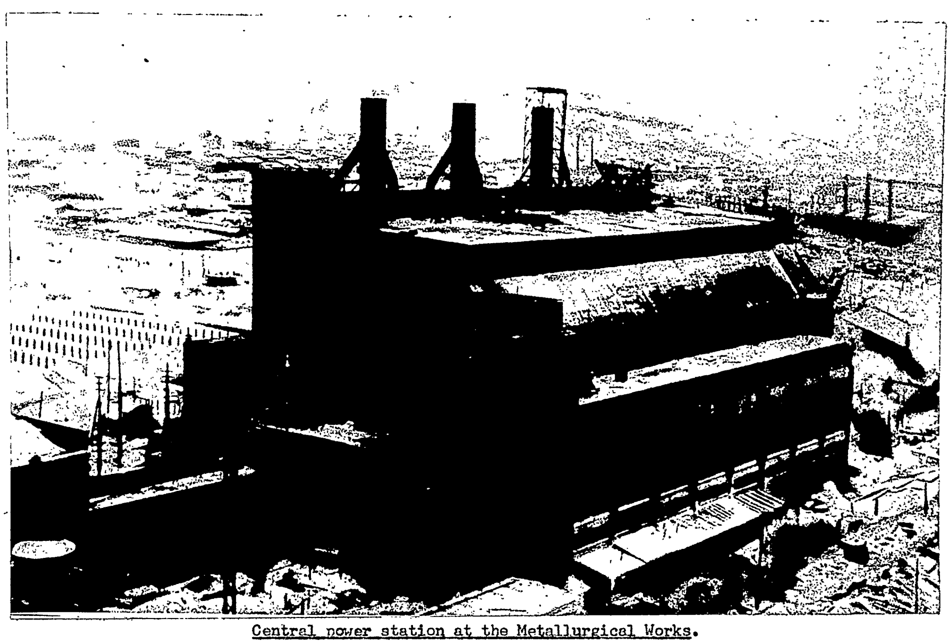

Central power station at the Metallurgical Works.

KUZNETSK COAL AND BLAST FURNACE GAS FIRING TETs
(Capacity: 60,000 kw. - Projected 109,000 kw.).

Source: USSR in Construction 1932, #4, p. 20 top, (DK267.A1U3)

PLATE 38A

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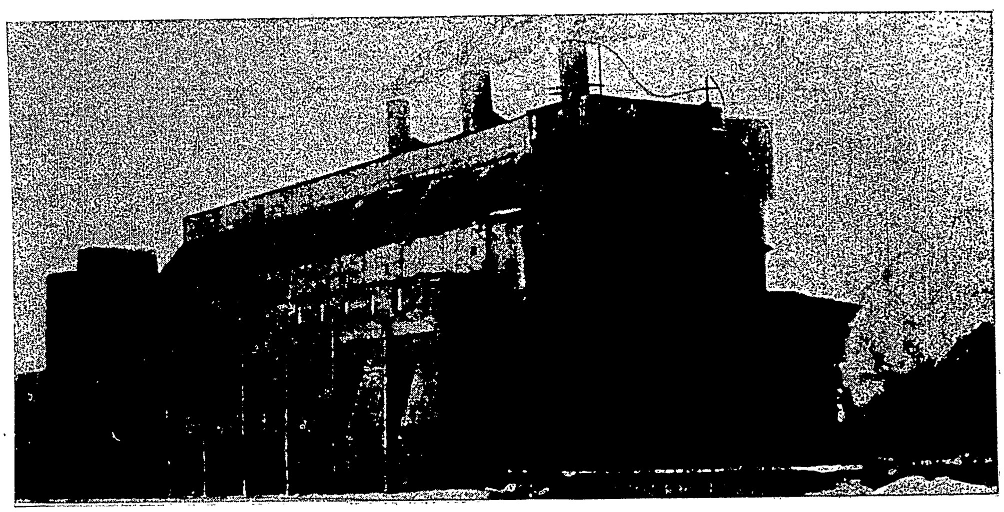
Central power station at the Metallurgical Plant.

KUZNETSK COAL AND BLAST FURNACE GAS FIRING TETs
(Capacity: 60,000 kw. - Projected 109,000 kw.)

Source: USSR in Construction 1932, #4, p. 10 bottom, (DK267.A1U3)

PLATE 38B

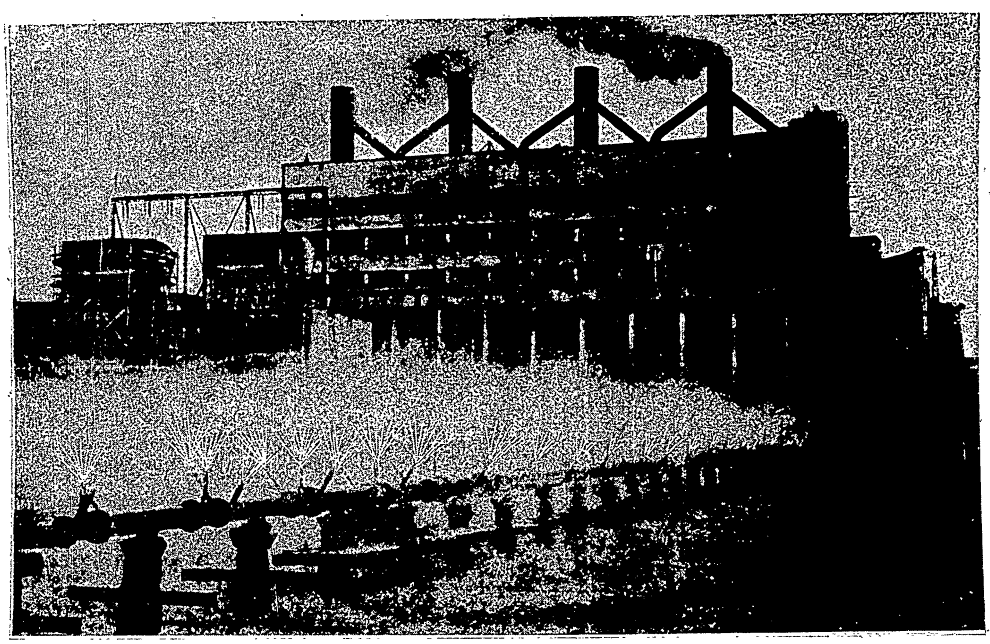


Partial view of the TETs

KUZNETSK COAL AND BLAST FURNACE GAS-FIRING TETs
(Capacity: 60,000 kw. - Projected 109,000 kw.)

Source: Arkhitektura elektrostantsiy 1939, p. 179, (TH4581.A5)

PLATE 38C



Partial view of the TETs showing the spray pond.

KUZNETSK COAL AND BLAST FURNACE GAS-FIRING TETs
(Capacity: 60,000 kw. - Projected 109,000 kw.)

Source: Arkhitektura elektrostantsiy 1939, p. 178, (TH4581.A5)

PLATE 38D

CHELYABINSK COAL-FIRING GRES

(Plates: 39, 39A)

Location: Chelyabinsk, industrial city in the Ural' Mountains

Coordinates: 55° 10' N, 61° 24' E.

Date of construction: 1930.

Layout type: First type.

Installed capacity: 150,000 kw.

Dimensions:

Structural type: Reinforced concrete frame

Wall covering: Reinforced concrete panels (prob.)

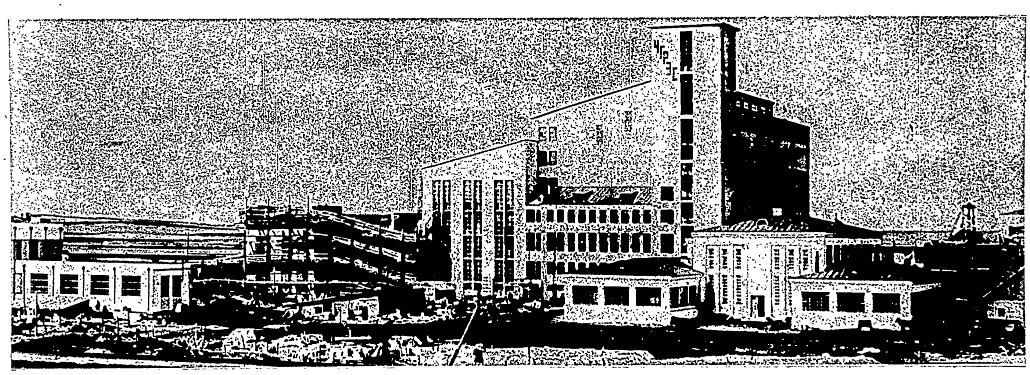
Roof construction:

Roofing: Ruberoid (prob.)

Cranes:

Crane girders:

Stacks: Steel, located on the roof.



GRES under construction.

CHELYABINSK COAL-FIRING GRES
(Capacity: 150,000 kw.)

Source: USSR in Construction 1930, #3, p. 20 bottom.

PLATE 39

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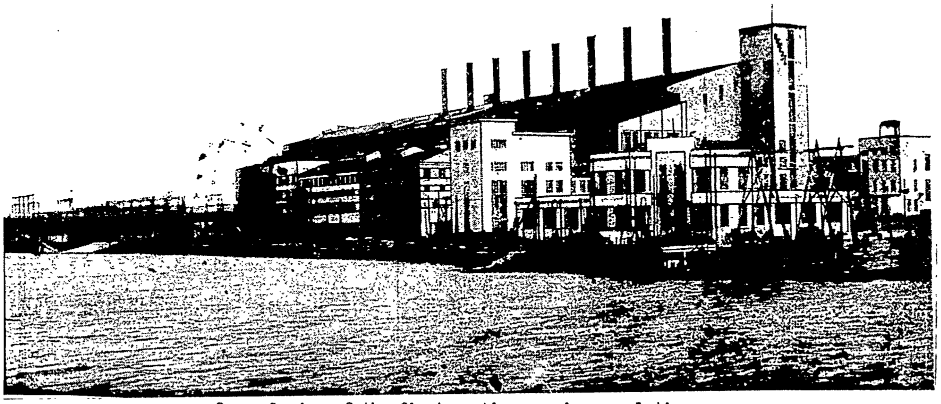

General view of the first section nearing completion.

CHELYABINSK COAL-FIRING GRES
(Capacity: 150,000 kw.)

Source: Elektricheskiye Stantsii 1932, #9, front cover, (TK4.E725)

PLATE 39A

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MAGNITOGORSK COAL AND BLAST FURNACE GAS-FIRING GES

(Plates: 40, 40A, 40B)

Location: Magnitogorsk Industrial city in Ural Mountains

Coordinates: 53° 27' N, 59° 04 E.

Date of construction: Before 1933.

Layout type: First type.

Installed capacity: 98,000 kw.

Dimensions:

Structural type: Cost-in-place reinforced concrete frame

Wall covering: Brick panel walls

Roof construction: Steel trusses, probably wood sheatting

Roofing: Probably tar and gravel

Cranes:

Crane girders:

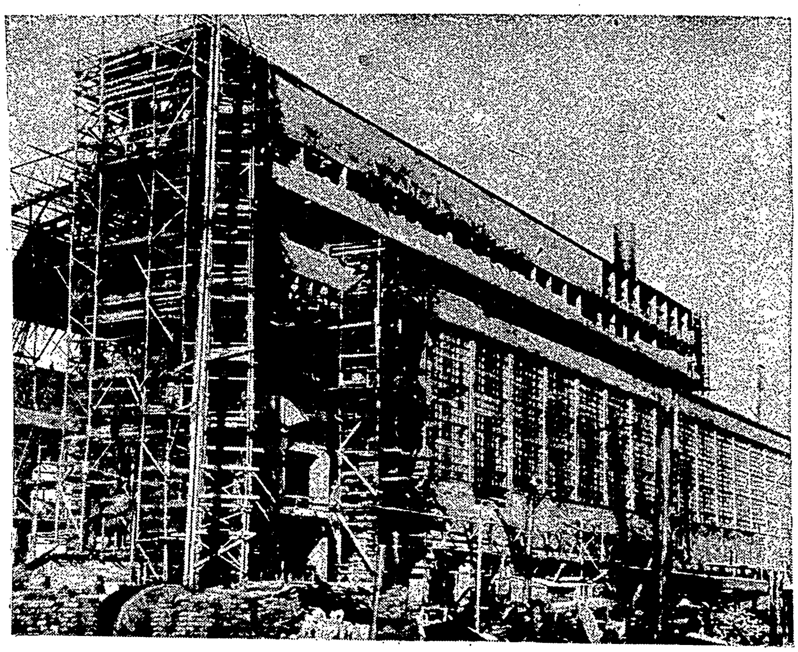
Stacks: Steel, located on the roof.

Aerial view. GES under construction.

MAGNITOGORSK COAL AND BLAST FURNACE GAS FIRING GES SERVING THE METALLURGICAL PLANT
(Capacity: 98,000 kw. - Projected 350,000 kw.)

Source: USSR in Construction 1932, #1, p. 32 bottom.

PLATE 40

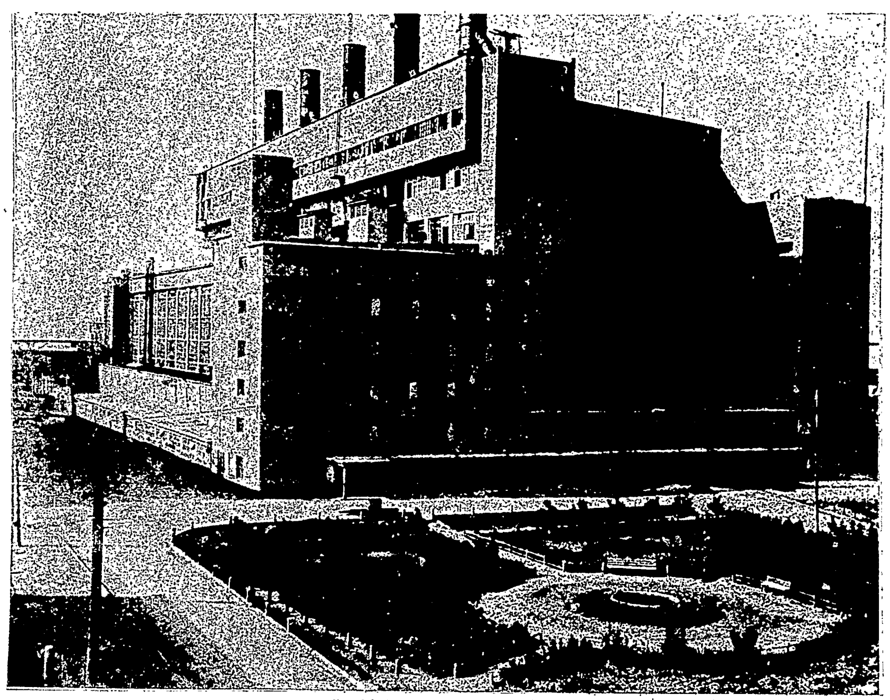


GES under construction.

MAGNITEGORSK COAL AND BLAST FURNACE GAS FIRING GES
(Capacity: 98,000 kw. - Projected 350,000 kw.)

Source: Ogonek 1932, #18, p. 13 bottom right, (AP50.042)

PLATE 40A



Partial exterior view.

MAGNITOGORSK COAL AND BLAST FURNACE GAS FIRING GES SERVING THE METALLURGICAL PLANT
(Capacity: 95,000 kw. - Projected 350,000 kw.)

Source: Arkhitektura elektrostantsiy 1939, p. 170, (TH4581.A5)

PLATE 40B

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KRAMATORSK COAL AND BLAST FURNACE GAS FIRING POWER STATION

(Plates: 41, 41A)

Location: Don Basin (The power station serves the metallurgical plant)

Coordinates: 48° 43' N, 37° 33' E.

Date of construction: Before 1932.

Layout type: First type.

Installed capacity: 25,000 kw.

Dimensions:

Structural type: Appears to be reinforced concrete.

Wall covering:

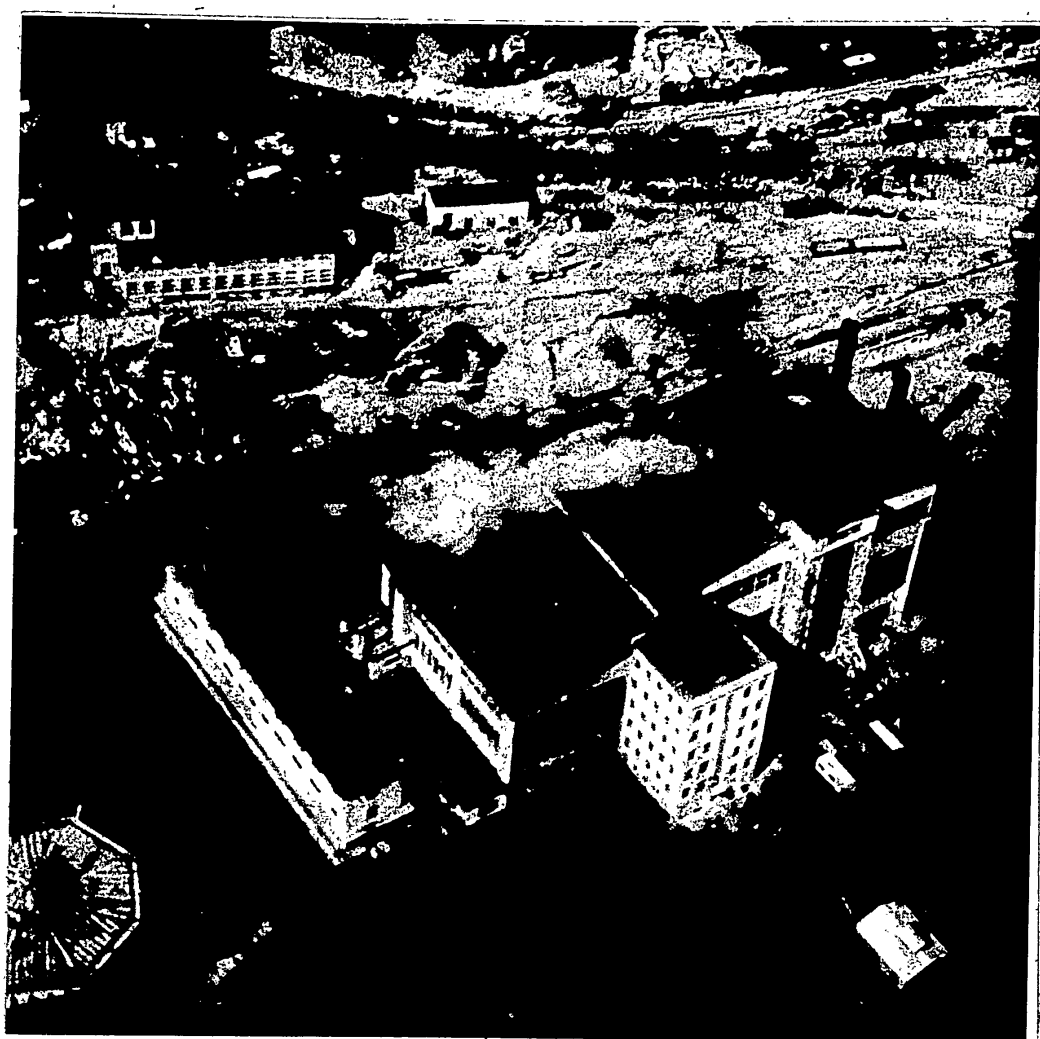
Roof construction:

Roofing: Sheet-iron.

Cranes:

Crane girders:

Stacks: Steel, located on the roof.

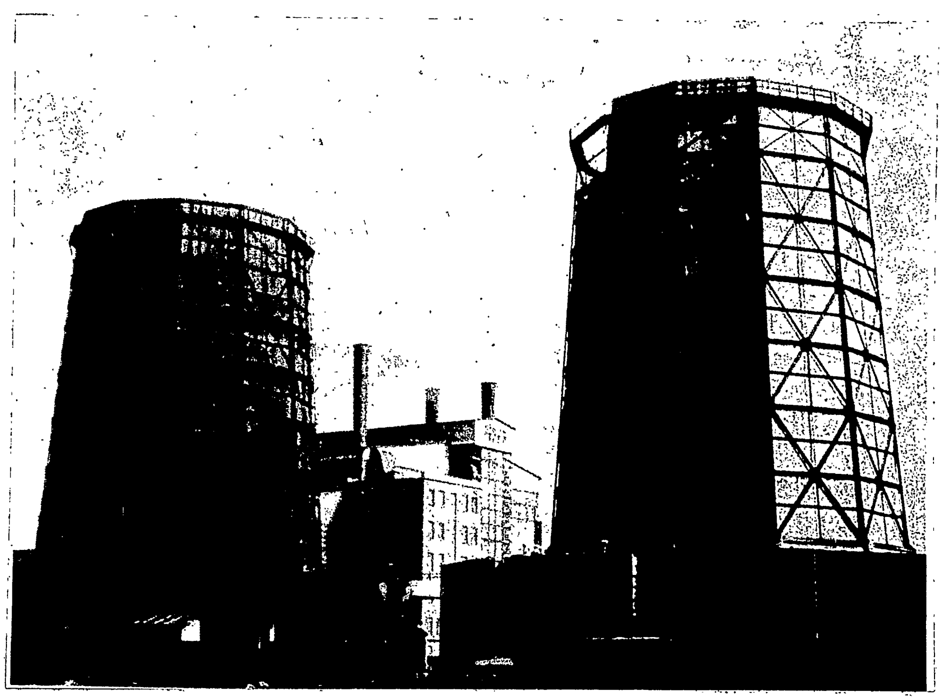


Aerial view.

KRAMATORSK COAL AND BLAST FURNACE GAS FIRING GES
(Capacity: 25,000 kw. - Projected 57,000 kw.)

Source: USSR in Construction 1932, #7, p. 8.

PLATE 41



Water cooling towers.

KRAMATORSK COAL AND BLAST FURNACE GAS FIRING GES
(Capacity: 25,000 kw. - Projected 57,000 kw.)

Source: USSR in Construction 1932, #7, p. 8 bottom

PLATE 41A

KEMEROVO COAL-FIRING TETS

(Plates: 42)

Location: City in Kuznetsk Basin, central Siberia

Coordinates: 55° 20' N, 86° 05' E.

Date of construction: 1934.

Layout type: First type.

Installed capacity: 48,000 kw.

Dimensions:

Structural type: Reinforced concrete.

Wall covering:

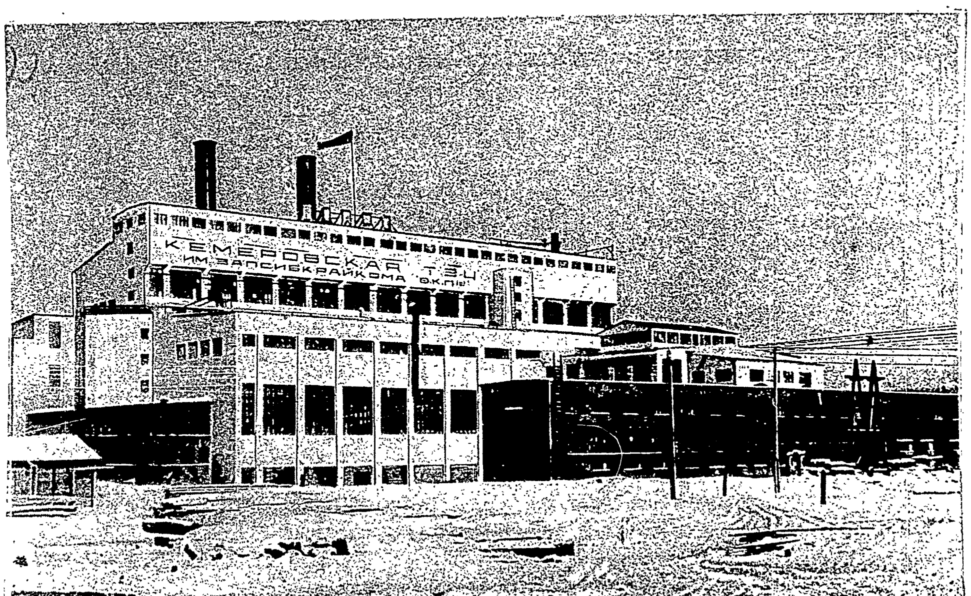
Roof construction:

Roofing: Probably sheet iron

Cranes:

Crane girders:

Stacks: Steel, located on the roof.



Partial exterior view.

KEMEROVO COAL-FIRING TETs
(Capacity: 48,000 kw.)
Source: Elektricheskiye Stantsii 1934, #3, front cover, (TK4.E725)

PLATE 42

Declassified in Part - Sanitized Copy Approved for Release 2013/02/25 : CIA-RDP81-01043R001600080008-4

SVERDLOVSK PEAT-FIRING TETS

(Plates: 43, 43A)

Location: City east of Urals, between Nizhniy Tagil and Chelyabinsk

Coordinates: 56° 50' N, 60° 38' E.

Date of construction:

Layout type: First type.

Installed capacity: 10,000 kw.

Dimensions:

Structural type: Reinforced concrete frame.

Wall covering: Brick panel walls.

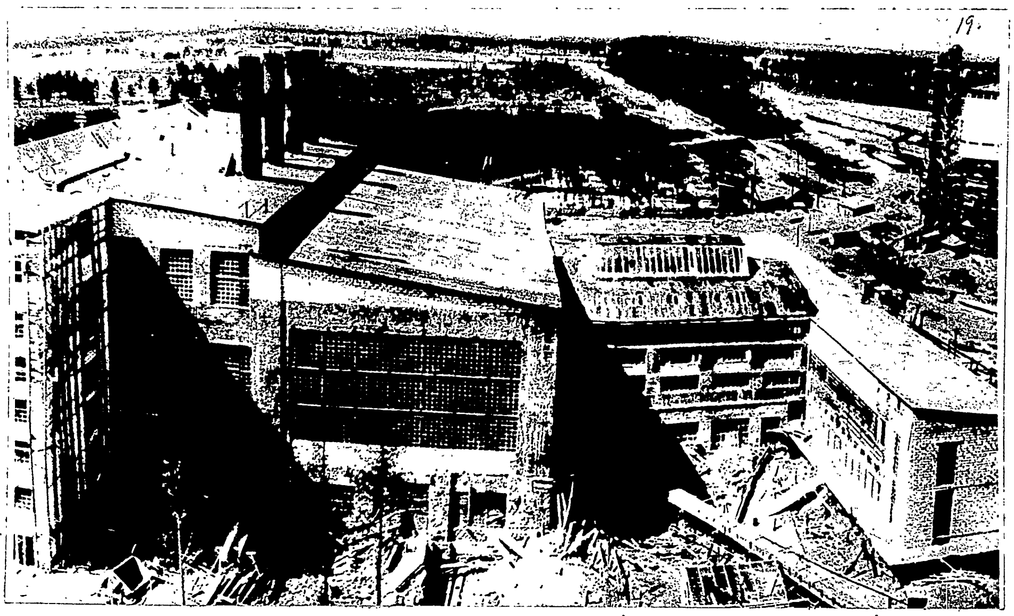
Roof construction:

Roofing: Sheet-iron on sloping part; tar & gravel on flat part.

Cranes:

Crane girders:

Stacks: Steel, located on the roof.

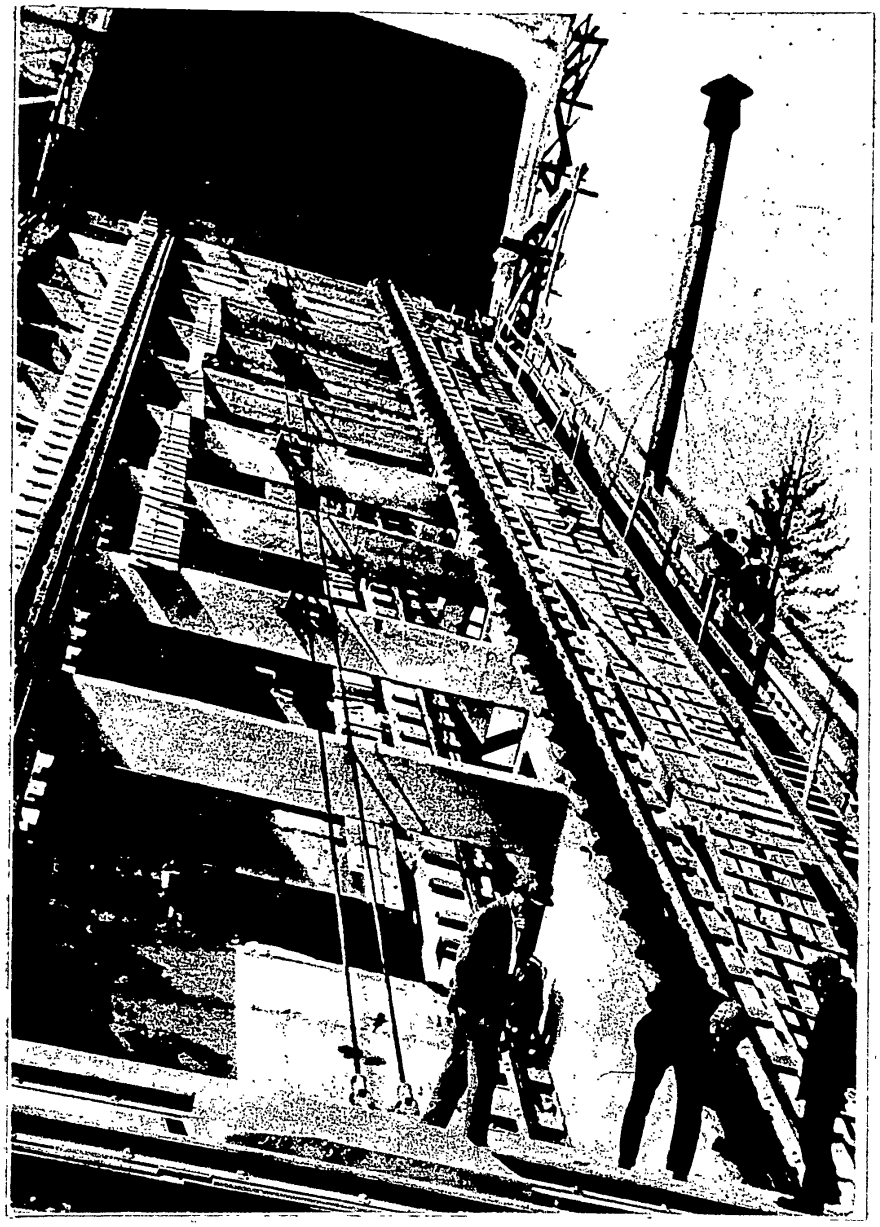


Partial exterior view of the TETs serving "Uralmashstroi" (Ural Machine Building Works)

SVERDLOVSK PEAT-FIRING TETs
(Capacity: 10,000 kw. - Projected 18,000 kw.)

Source: USSR in Construction 1932, #7, p. 19 top.

PLATE 43



Fuel supply trestle leading to the bunker gallery of the TETs.
(Serving "Uralmashstroi" - Ural Machine Building Works)

SVERDLOVSK PEAT-FIRING TETs
(Capacity: 10,000 kw. - Projected 18,000 kw.)

Source: USSR in Construction 1932, #7, p. 19 bottom.

PLATE 43A

VOROSHILOVSK COAL AND BLAST FURNACE GAS FIRING POWER PLANT

(Plates: 44).

Location: City in Southern Donbass, Ukraine

Coordinates: 48° 30' N, 38° 47' E.

Date of construction: After 1936.

Layout type: First type.

Installed capacity: 24,000 kw.

Dimensions:

Structural type:

Wall covering:

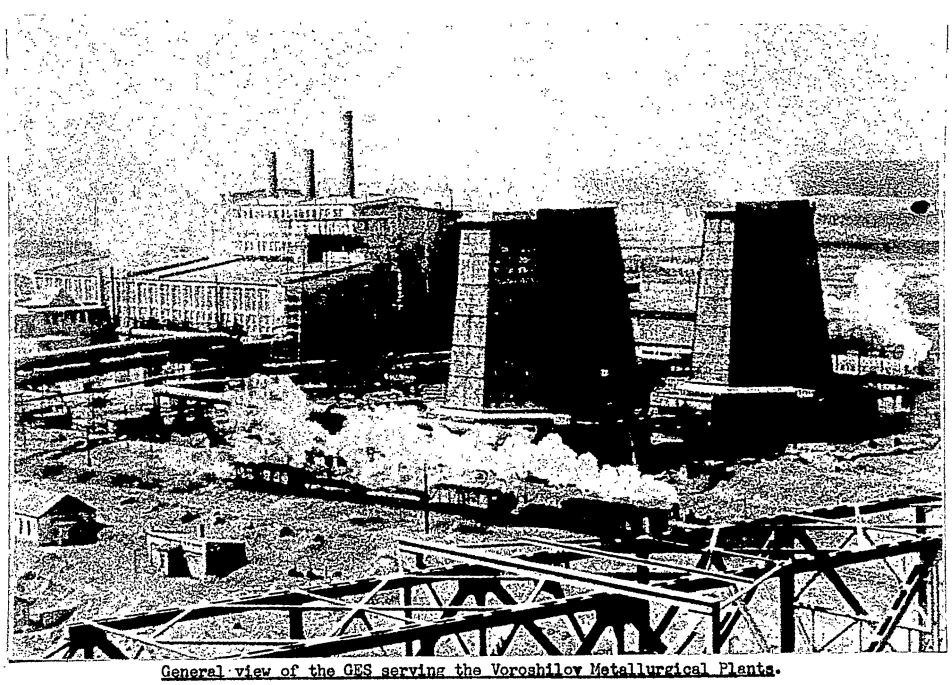
Roof construction:

Roofing: Probably sheet iron

Cranes:

Crane girders:

Stacks: Appears to be steel, located on the roof.



General view of the GES serving the Voroshilov Metallurgical Plants.

VOROSHILOVSK COAL AND BLAST FURNACE GAS-FIRING GES
(Capacity: 24,000 kw.)

Source: Ogonek, 1947, December, p. 6 top right.

PLATE 44

SARATOV COAL-FIRING GRES

9 (Plates: 45, 45A, 45B, 45C, 45 D, 45E.)

Location: On the shore of Volga in the city of Saratov

Coordinates: 51° 34' N; 46° 02' E

Date of construction: 1930

Installed capacity: 22,500 kw

Layout type: First type

Dimensions:

Structural type: Reinforced concrete frame

Wall covering:

Roof construction:

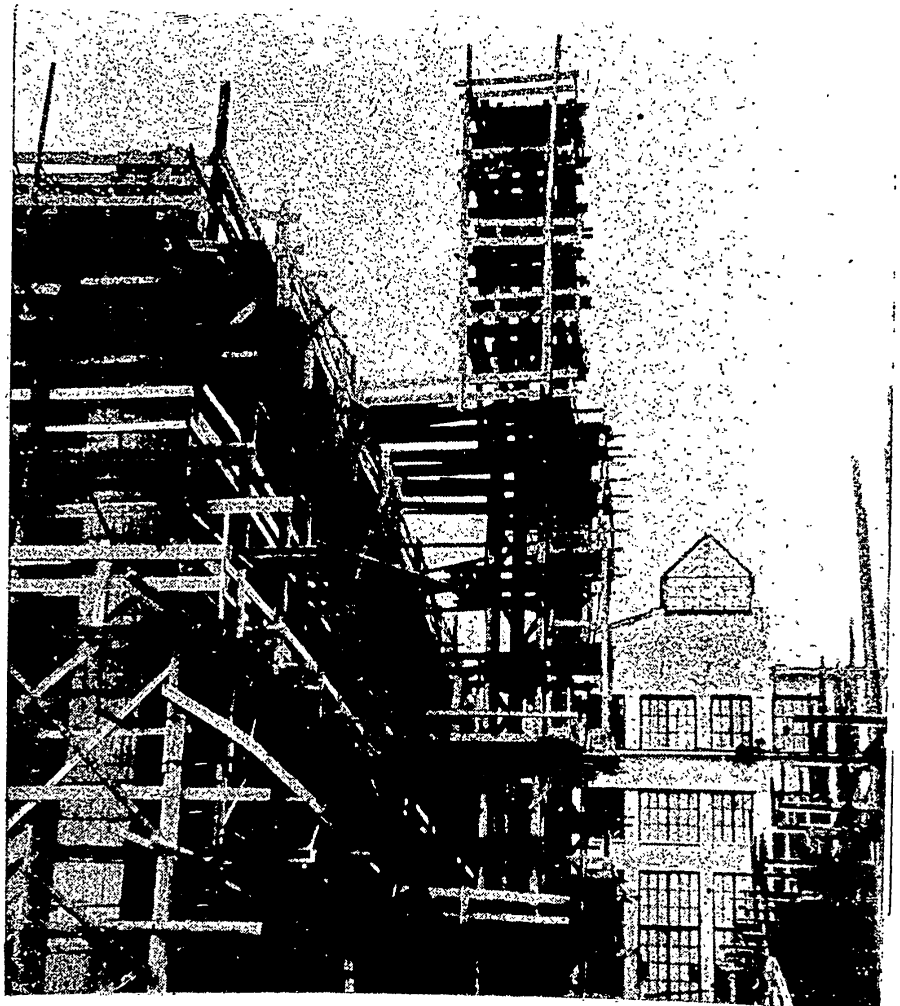
Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof

Remarks: There is also a 12,000 kw TETs at Saratov.

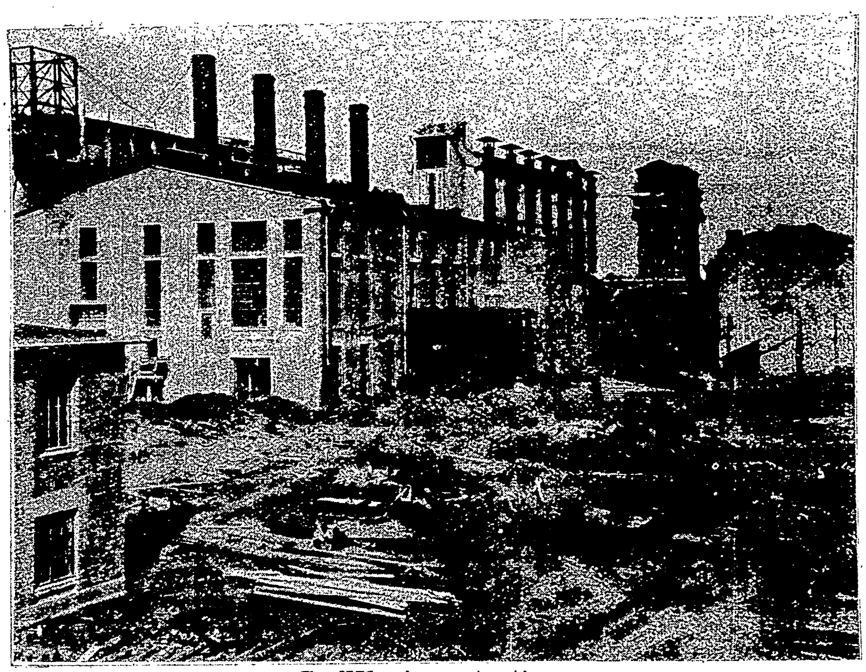


The GRES under construction.

SARATOV COAL-FIRING GRES
(Capacity: 22,500 kw.)

Source: Prozhektor 1930, #9, p. 14 bottom left.

PLATE 45



<u>The GRES under construction</u>.

SARATOV COAL-FIRING GRES
(Capacity: 22,500 kw.)

Source: Prozhektor 1930, #26-27, p. 24 bottom left.

PLATE 45A

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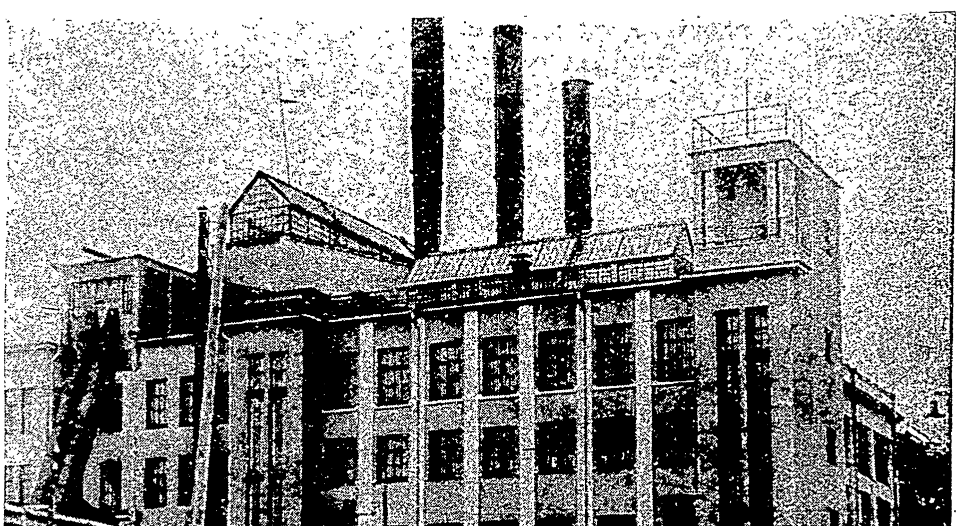

Partial view of GRES under construction.

SARATOV COAL-FIRING GRES
(Capacity: 22,500 kw.)

Source: Prozhektor 1930, #35, p. 9, bottom right.

PLATE 45B

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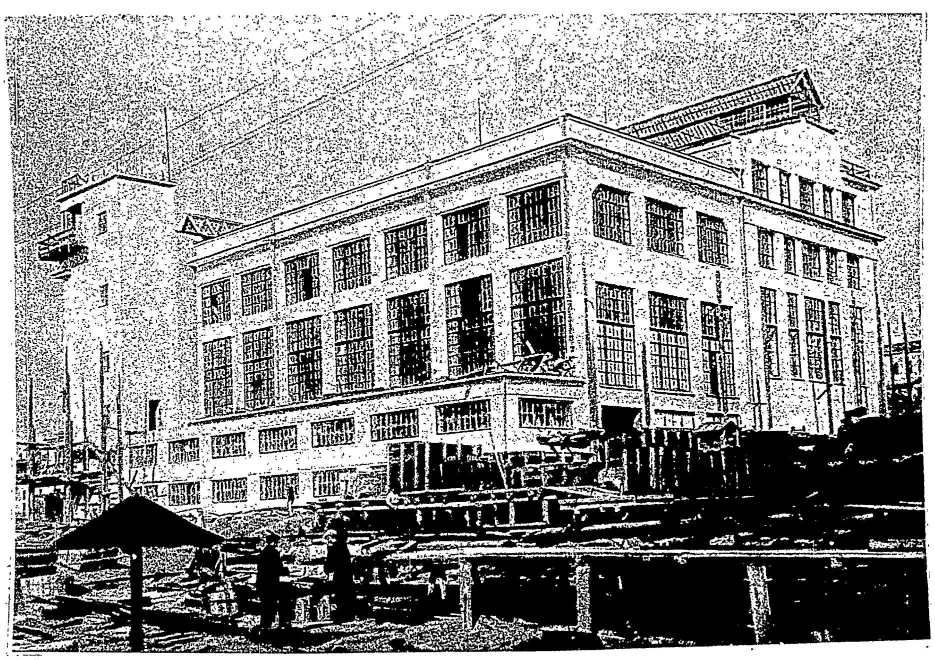

GRES under construction.

SARATOV COAL-FIRING GRES
(Capacity: 22,500 kw.)

Source: USSR in Construction 1930, #3, p. 21 top.
PLATE 45C

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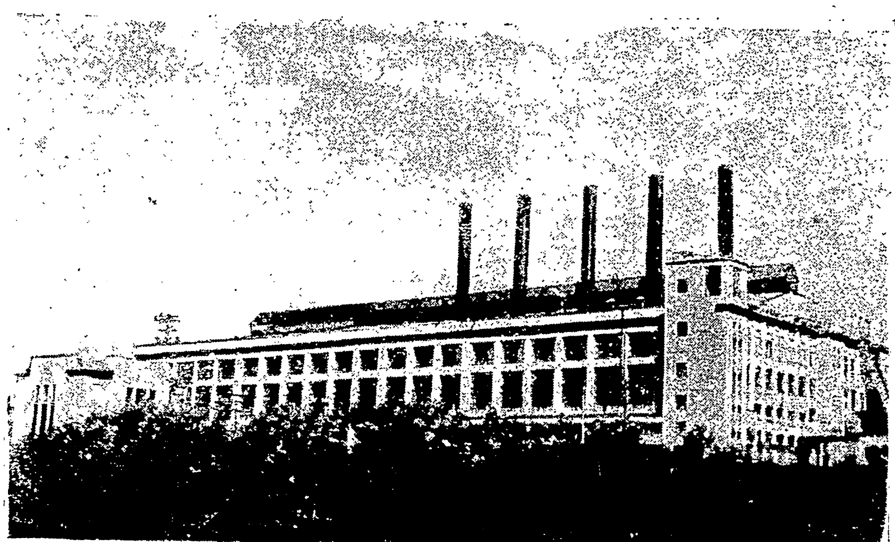
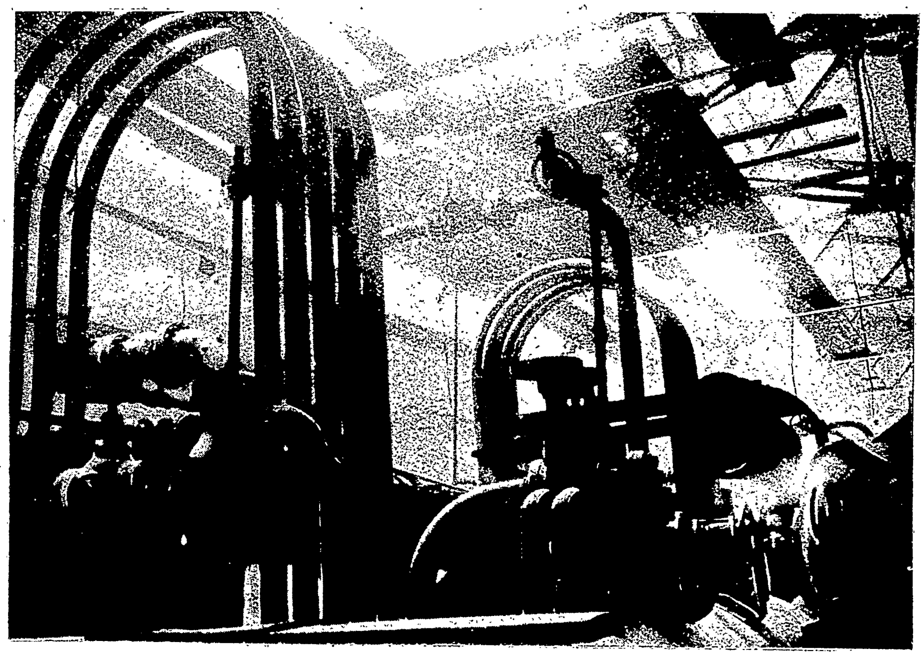


General view.

SARATOV COAL-FIRING GRES
(Capacity: 22,500 kw.)

Source: Pyatnadtsat' let leninskogo plana elektrifikatsii 1936, p. 43, (TK85.D6)

PLATE 45D



Partial view of the steam piping system.

SARATOV COAL-FIRING GRES
(Capacity: 22,500 kw.)

Source: Prozhektor 1930, #9, p. 14 bottom right.

PLATE 45E

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YAROSLAVL' PEAT-FIRING GRES

(Plates: 46)

Location: Left bank of the Volga River, near the city of Yaroslavl'

Coordinates: 57° 35' N; 39° 50' E.

Date of construction: 1931

Installed capacity: 41,000 kw

Layout type: First type

Dimensions:

Structural type: Reinforced concrete

Wall Covering: Brick, and block panel walls and curtain walls

Roof construction:

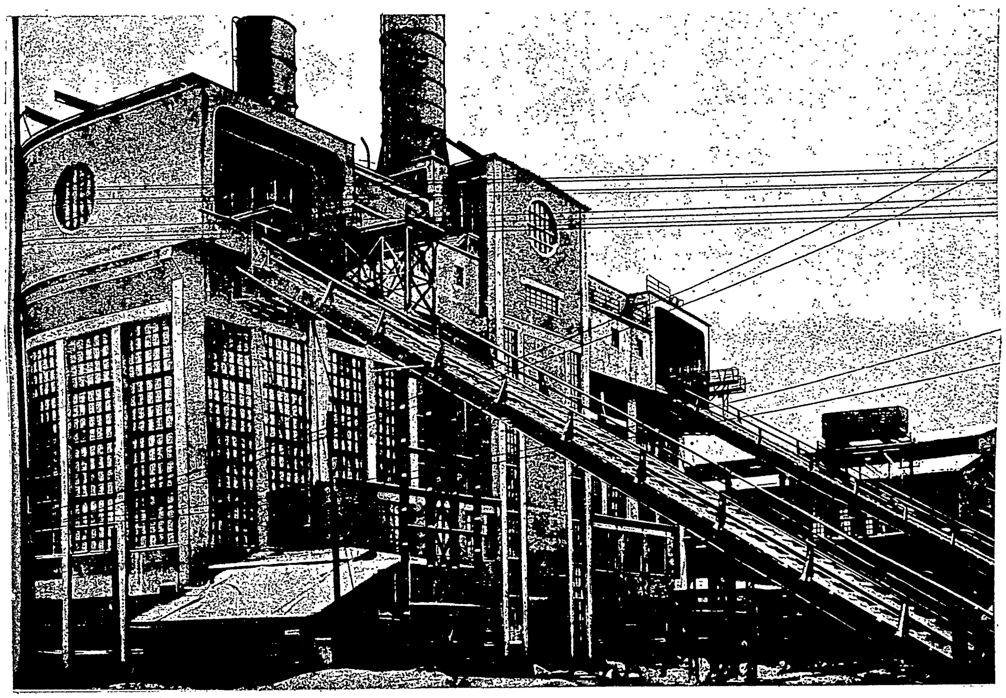
Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof

Fuel delivery: Over steel trestles



Side view of the boiler house.

YAROSLAVL' PEAT-FIRING GRES
(Capacity: 41,000 kw.)

Source: Elektricheskiye Stantsii 1936, #6, front cover, (TK4.E725)
PLATE 46

Declassified in Part - Sanitized Copy Approved for Release 2013/02/25 : CIA-RDP81-01043R001600080008-4

YAROSLAVL' PEAT FIRING TETS

(Plates: 46A)

Location: Site of the Yaroslavl' Rubber and Asbestos Plant

Coordinates: 57° 35' N; 39° 50' E (approximately)

Date of construction: 1933

Installed capacity: 130,000 kw

Layout type: First type

Dimensions:

Structural type:

Wall covering: Brick

Roof construction:

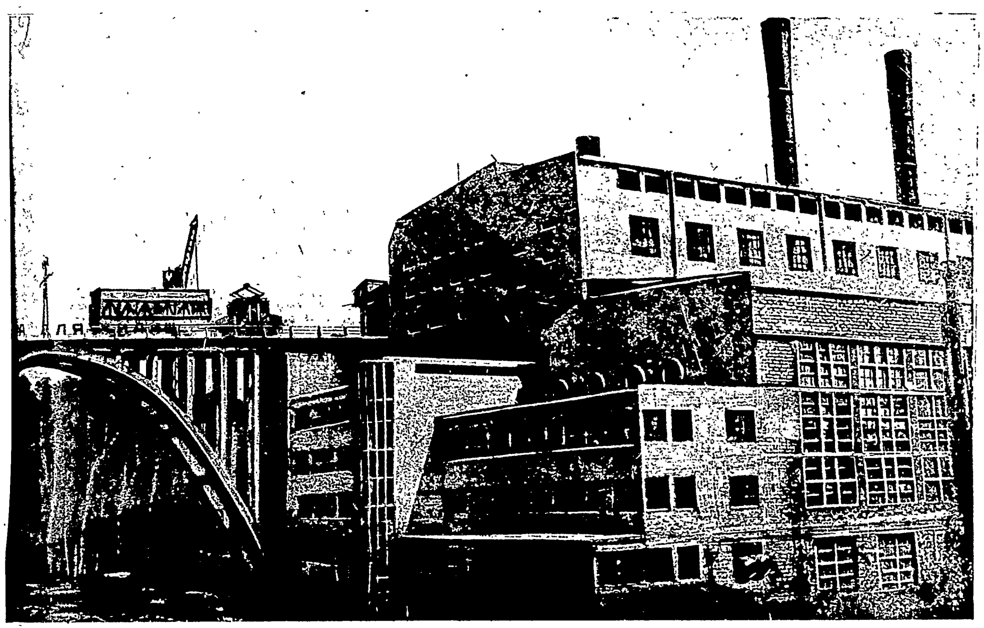
Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof

Fuel delivery: over a reinforced concrete trestle



Partial exterior view showing peat delivery reinforced concrete trestle.

YAROSLAVL' PEAT-FIRING TETs AT THE YAROSLAVL' RUBBER AND ASBESTOS PLANT
(Capacity: 130,000 kw.)

Source: Elektricheskiye Stantsii 1934, #7, front cover (TK.E725)

PLATE 46A

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BAKU CRUDE OIL, GAS AND MAZUT (FUEL OIL) FIRING GRES "KRASNAYA ZVEZDA"

(Plates: 47, 47A)

Location: Baku, Azerbaydzhanskaya SSR, on east Caspian Sea

Coordinates: 40° 23' N; 49° 55' E.

Date of construction: Prior to the 1921 "GOELRO" plan; expanded since then

Installed capacity: 109,000 kw

Layout type: First type

Dimensions:

Structural type:

Wall covering:

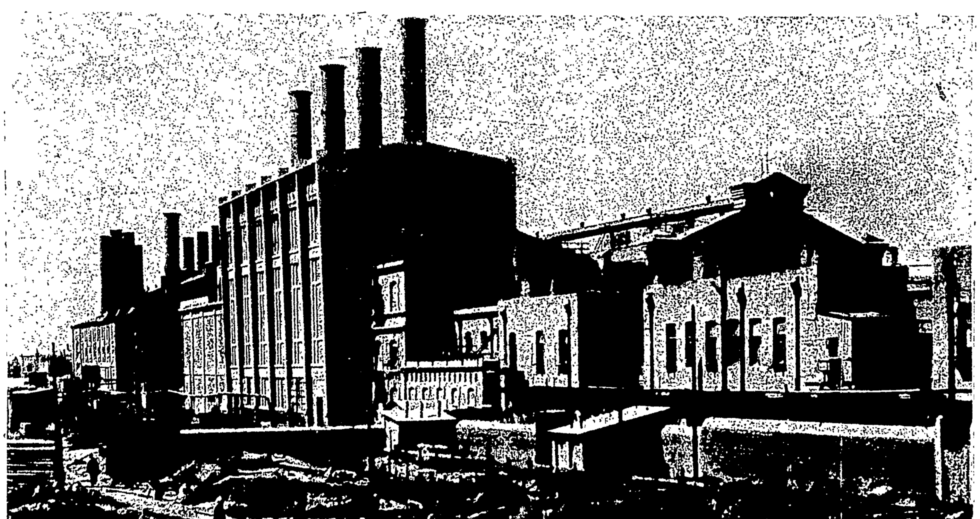
Roof construction:

Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof



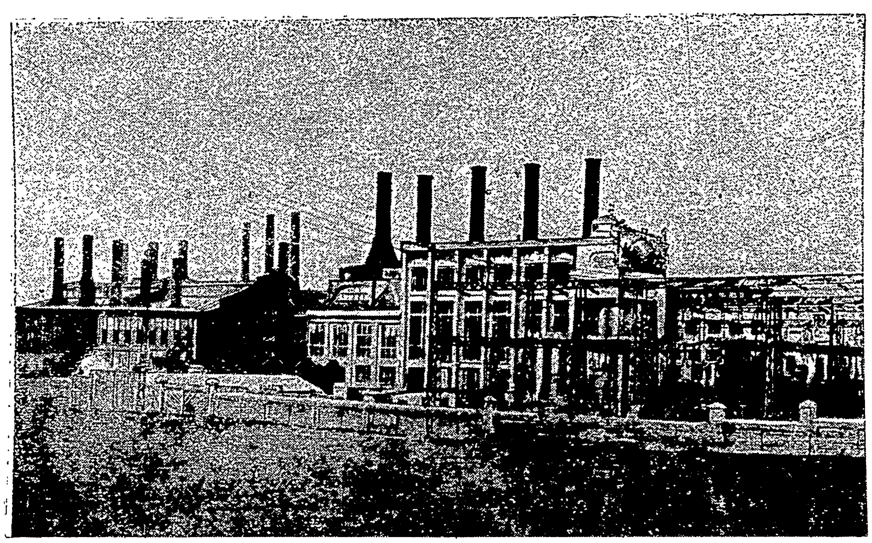
General view.

BAKU "KRASNAYA ZVEZDA" CRUDE OIL, GAS AND MAZUT (fuel oil) FIRING GRES
(Capacity: 109,000 kw.)

Source: USSR in Construction 1932, #3, p. 25, center left.

PLATE 47

Declassified in Part - Sanitized Copy Approved for Release 2013/02/25 : CIA-RDP81-01043R001600080008-4



Side view.

BAKU CRUDE OIL, GAS AND MAZUT (fuel oil) FIRING GRES "KRASNAYA ZVEZDA".
(Capacity: 109,000 kw.)

Source: Elektricheskiye Stantsii 1936, p. 37, (TK4.E725)

PLATE 47A

BAKU OIL, GAS AND MAZUT (FUEL-OIL) FIRING GRES "KRASIN"

(Plates: 47B)

Location: Baku, Azerbaydzhanskaya SSR, on east Caspian Sea

Coordinates: 40° 23' N; 49° 55' E.

Date of construction: Prior to the 1921 "GOELRO" plan; expanded since then

Installed capacity: 67,000 kw

Layout type: First type

Dimensions:

Structural type:

Wall covering:

Roof construction:

Roof covering:

Cranes:

Crane girders:

Stacks: Steel, located on the roof

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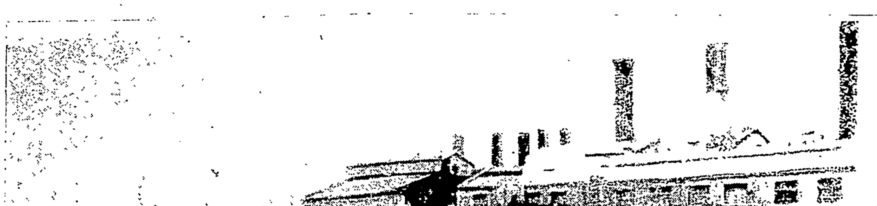


Partial view of the plant and 100 kw. open sub-station.

BAKU "KRASIN" OIL-GAS AND MAZUT (fuel oil) FIRING GES
(Capacity: 67,000 kw.)

Source: Pyatnadsat' let leninskogo plana elektrifikatsii 1936, p. 37, (TK85.D6)

PLATE 47B

NOVOROSSIYSK MAZUT (FUEL*OIL) FIRING GRES

(Plates: 48)

Location: Port on Black Sea, east of Sea of Azov

Coordinates: 44° 43' N; 37° 47' E

Date of construction: 1930

Installed capacity: 20,000 kw

Layout type: First type

Dimensions:

Structural type: Reinforced concrete

Wall covering:

Roof construction:

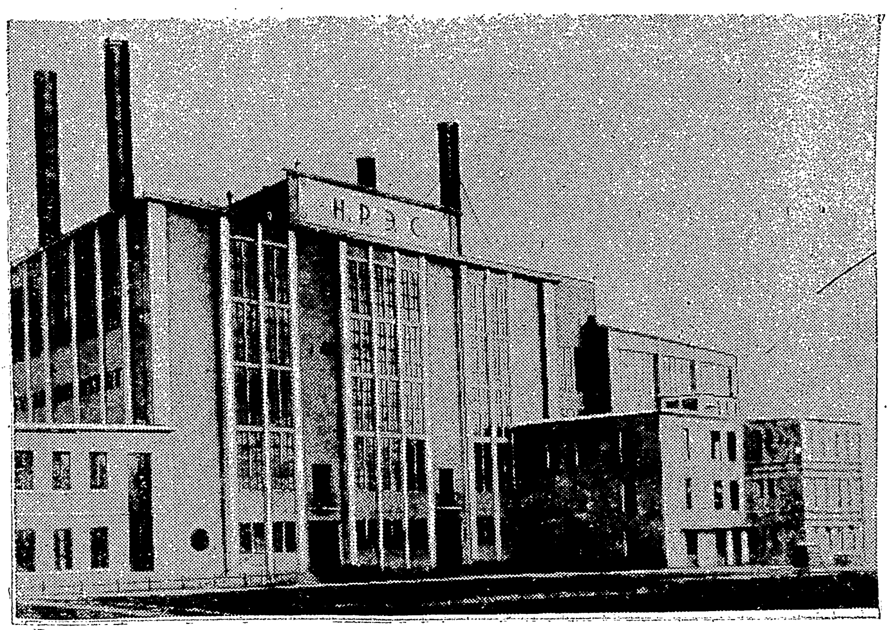
Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof

-206-



Front view.

NOVOROSSIIYSK MAZUT (fuel-oil) FIRING GRES
(Capacity: 20,000 kw.)

Source: Elektricheskiye Stantsii 1936, #1, p. 41, (TK4.E725)

PLATE 48

VORONEZH PULVERIZED COAL-FISHING GRES

(Plates: 49)

Location: City on Don River, 300 miles south of Moskva

Coordinates: 51° 38' N; 39° 12' E

Date of construction: 1934

Installed capacity: 24,000 kw (1934)

Layout type: First type

Dimensions:

Structural type: Reinforced concrete

Wall covering:

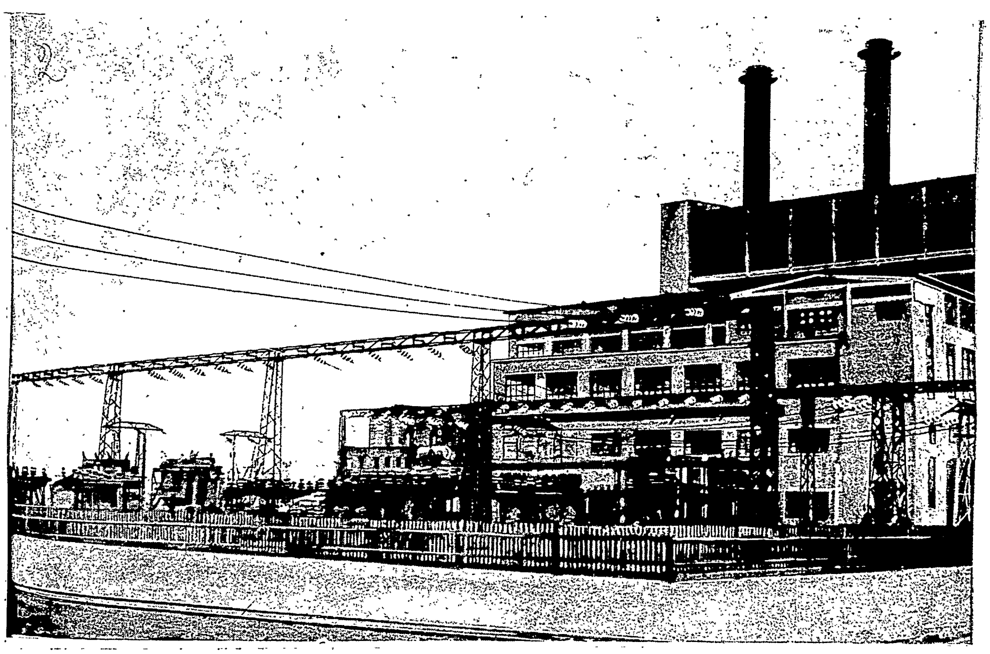
Roof construction:

Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof



Partial view of the GRES and the open air sub-station.

VORONEZH PULVERIZED COAL-FIRING GRES
(Capacity: 24,000 kw.)

Source: Elektricheskiye Stantsii 1936, #11, front cover, (TK4.E725)

PLATE 49

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KAZAN' COAL-FIRING TETS

(Plates: 50, 50A)

Location: City on Volga River, 440 miles east of Moskva

Coordinates: 55° 45' N; 49° 08' E

Date of construction: 1932

Installed capacity: 20,000 kw

Layout type: First type

Dimensions:

Structural type: Reinforced concrete

Wall covering:

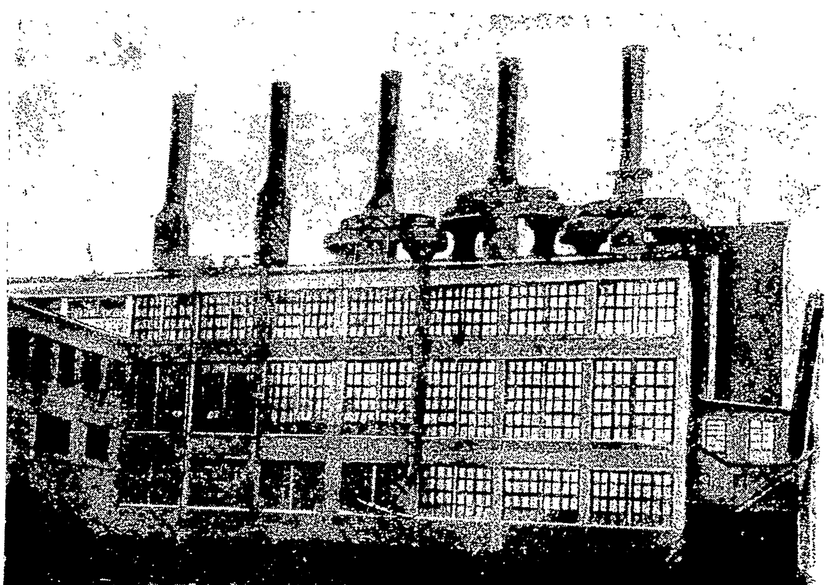
Roof construction:

Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof



Partial exterior view.

KAZAN' COAL-FIRING TETs
(Capacity: 20,000 kw.)

Source: Piatnadsat'let leninskogo plana elektrifikatsii, 1936, p. 63, (TK85.D6)

PLATE 50

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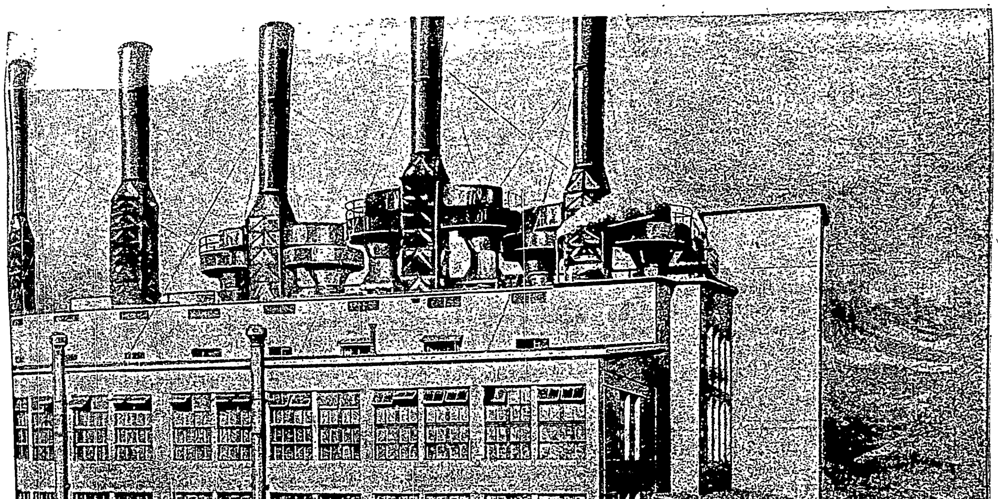

Partial exterior view.
KAZAN' COAL-FIRING TETs
(Capacity: 20,000 kw.)
Source: Elektricheskiye Stantsii 1936, #7, front cover (TK4.E275)

PLATE 50A

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MOSKVA COAL-FIRING TETS "STALIN"

(Plates: 51) (51a)

Location: Moskva

Coordinates: 55° 45'; 37° 35' E

Date of construction: Between 1931 and 1935

Installed capacity: 25,000 kw (1936). Projected capacity: 100,000 kw

Layout type: Second type

Dimensions:

Structural type:

Wall covering: Probably brick panel walls

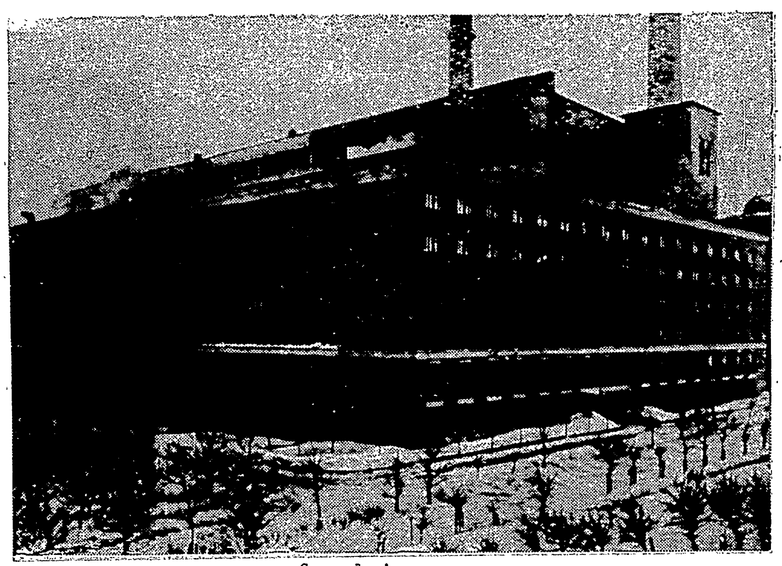
Roof construction:

Roof covering:

Cranes:

Cranesgirders:

Stacks: Appear to be separate from the building

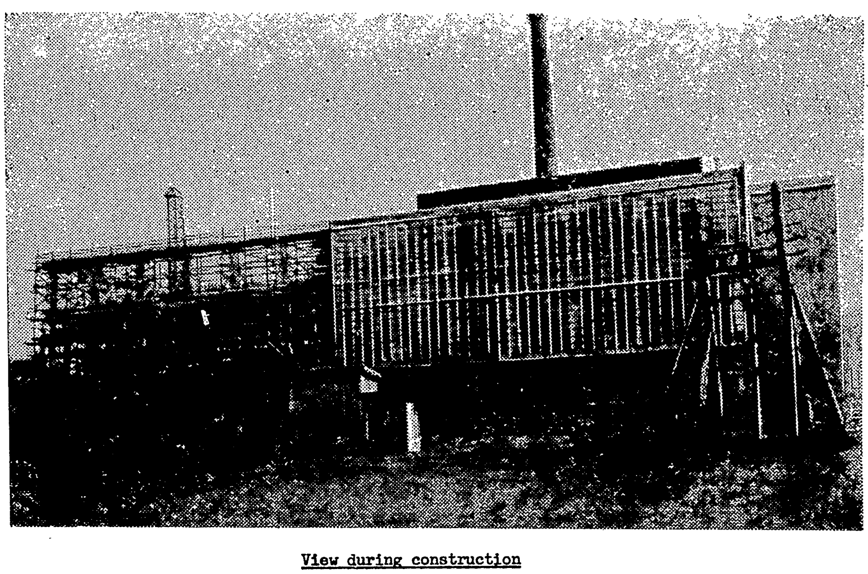


General view.

MOSKVA COAL-FIRING TETs "STALIN"
(Capacity: 25,000 kw.)

Source: Markin, A.B., Budushchyeye Elektrifikatsii SSSR, 1956, p. 22,
(TK85.M3)

PLATE 51



View during construction

MOSKVA COAL-FIRING TETs "STALIN" (Capacity: 25,000 kw.)

Source: Elektricheskiye Stantsii 1936, No. 1, top p. 12.

PLATE 51a

MOSKVA MAZUT (FUEL-OIL)-FIRING POWER STATION "SMIDOVICH"

(Plates: 51A, 51B, 51C, 51D)

Location: Moskva

Coordinates: 55° 45'; 37° 35' E

Date of construction: 1886; reconstructed several times; a 12,000 kw turbine added in 1933 for district heating

Installed capacity: 120,000 kw

Layout type: First type

Dimensions:

Structural type: Probably partly brick wall, partly reinforced concrete frame

Wall covering: Various; probably partly concrete, partly brick curtain walls

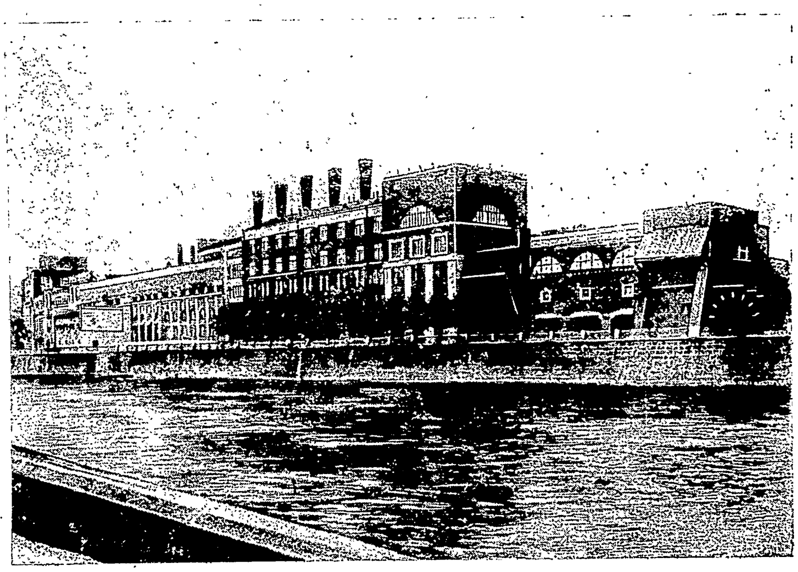
Roof construction:

Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof

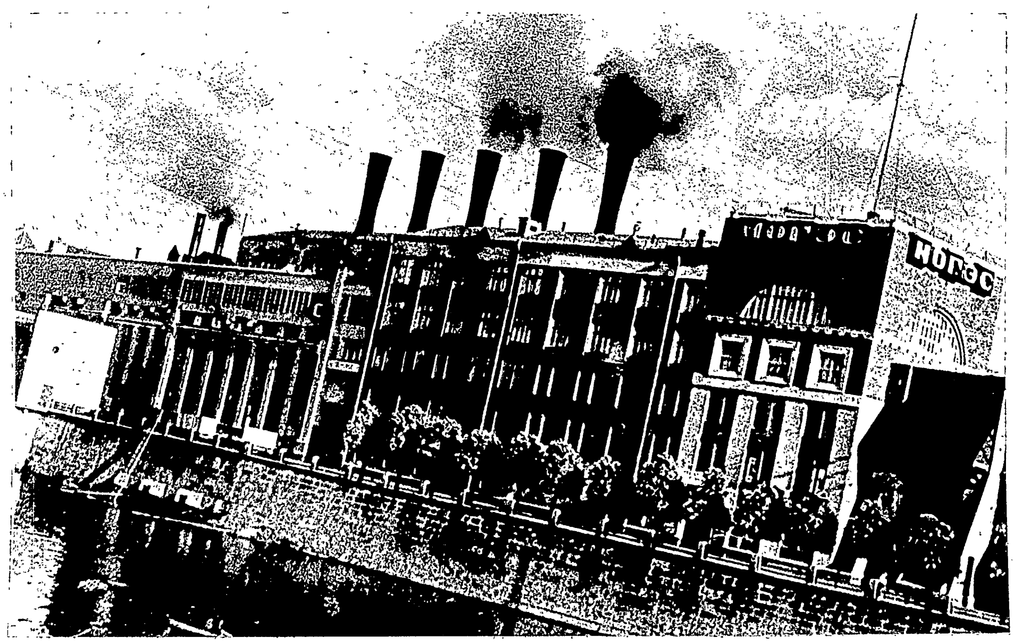


General view.

MOSKVA MAZUT (fuel-oil) FIRING GES "SMIDOVICH"
(Capacity: 120,000 kw.)

Source: Pyatnadsat' let leninskogo plana elektrifikatsii 1936, p. 31, (TK85.D6)

PLATE 51A



Side view.

MOSKVA MAZUT (fuel-oil) FIRING GES "SMIDOVICH"
(Capacity: 120,000 kw.)
Source: USSR in Construction 1931, #9, p. 27 middle.

PLATE 51B

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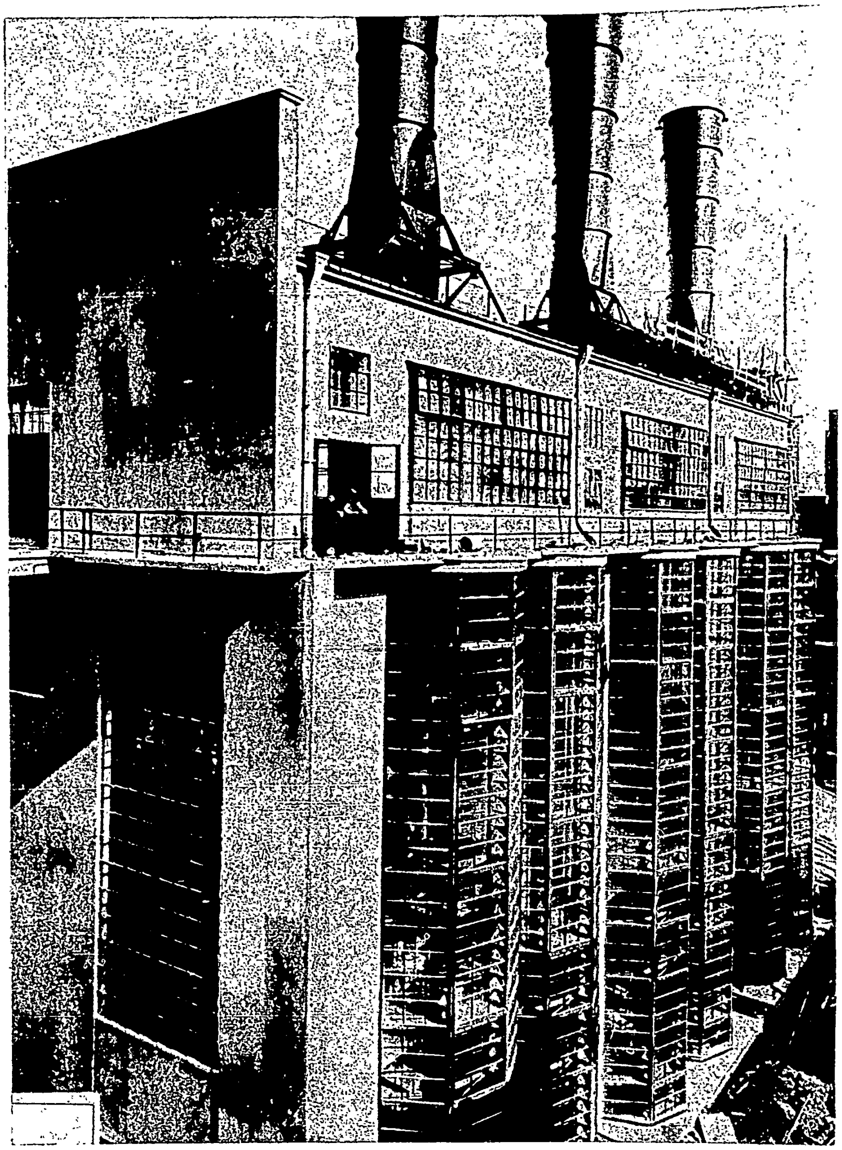

Exterior view of the boiler house.

MOSKVA MAZUT (fuel-oil) FIRING GES "SMIDOVICH"
(Capacity: 120,000 kw.)

Source: USSR in Construction 1930, #3, p. 19 top

PLATE 51C

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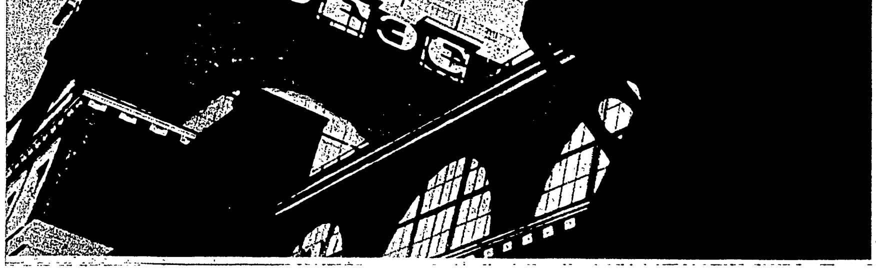

Construction detail.

MOSKVA MAZUT (fuel-oil) FIRING GES "SMIDOVICH"
(Capacity: 120,000 kw.)

Source: USSR in Construction 1931, #9, p. 27 top

PLATE 51D

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MOSCOW HIGH PRESSURE COAL-FIRING TETS (ENERGO)

(Plates: 51E)

Location: Site of the All-Union Heat Technical Institute in the suburbs of Moscow

Coordinates: 55° 45'; 37° 35' E

Date of construction: 1933

Installed capacity: 60,000 kw

Layout type: First type

Dimensions:

Structural type: Probably poured-in-place reinforced concrete frame

Wall covering: Probably brick panel walls

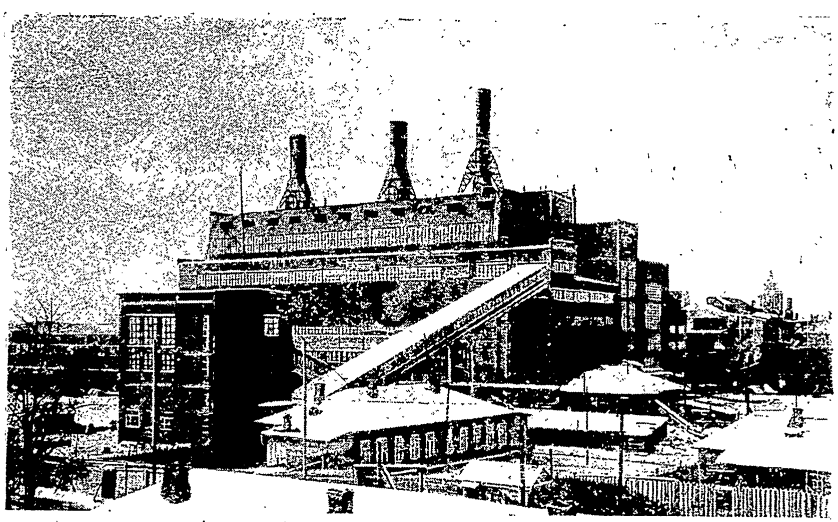
Roof construction:

Roof covering:

Cranes:

Crane girders:

Stacks: Steel; located on the roof



General view

MOSCOW HIGH PRESSURE COAL-FIRING TETs (ENERGO)
(Capacity: 60,000 kw.)

Source: Pyatnadsat' let leninskogo plana elektrifikatsii 1936, p. 67, (TK85.D6)

PLATE 51E

ELEKTROGORSK (FORMERLY ELEKTROPEZEDACHA) PEAT-FIRING PLANT "KLASSON", MOSENERGO GRES

(Plates: 51F)

Location: 12.5 miles from Noginsk, Moscow oblast'

Coordinates: 55° 53' N; 38° 47' E

Date of construction: Before 1930

Installed capacity: 46,000

Layout type: First type

Dimensions:

Structural type: Reinforced concrete frame

Wall covering: Presumably brick

Roof construction:

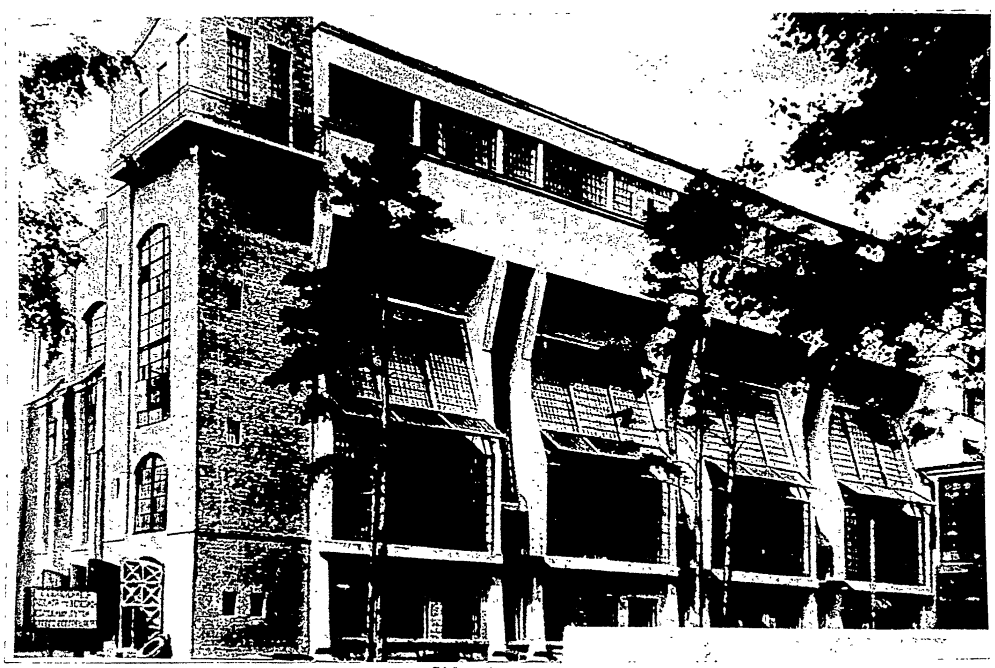
Roof covering:

Cranes:

Crane girders: Steel (prob.)

Stacks: Steel; located on the roof

-223-



Side view.

ELEKTROGORSK (formerly Elektroperedacha) "KLASSON", PEAT FIRING PLANT #3, MOSENERGO GES
(Capacity: 46,000 kw.)

Source: USSR in Construction 1930, #3, p. 19 middle.

PLATE 51F

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COAL-FIRING ARTEM GRES "KIROV" IN PRIMORSKIY KRAY

(Plates: 52, 52A)

Location: Approximately 25 miles from Vladivostok on Suchanskaya Railroad.

Coordinates: 43° 22' N, 132° 19' E.

Date of construction: Apparently in 1936.

Installed capacity: 50,000 kw.

Layout type: First type.

Dimensions:

Structural type: Reinforced concrete frame.

Wall covering:

Roof construction:

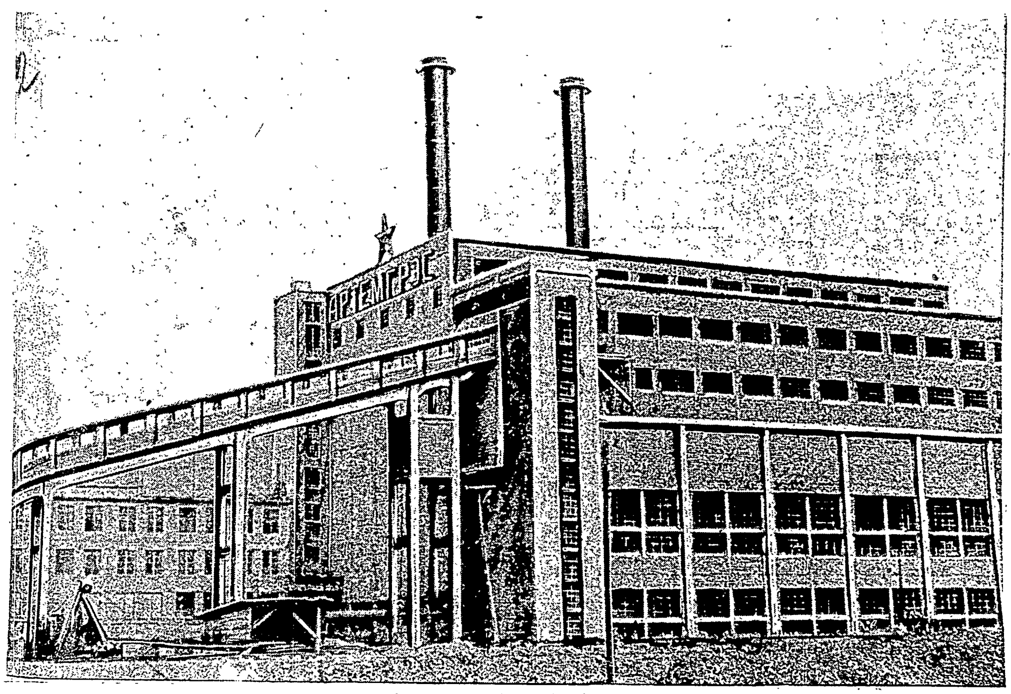
Roofing:

Cranes:

Crane girders:

Stacks: Steel, located on the roof.

Fuel delivery: Over a reinforced concrete trestle.



Side view of the GRES.

ARTEM GRES "KIROV" IN PRIMORSKIY KRAY (coal firing)
(Capacity: 50,000 kw.)

Source: Elektricheskiye Stantsii 1937, #5, front cover (TK4.E725)

PLATE 52

General view.

ARTEM GRES "KIROV" (Primorskiy Kray) (coal firing)
(Capacity: 50,000 kw.)

Source: Krasnoye Znamya 1947, #10, p. 4

PLATE 52A

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MIRONOVSKAYA COAL-FIRING GRES NEAR ARTEMOVSK (STALINSKAYA O.)

(Plates: 53)

Location: In Donbass, southern Ukraine

Coordinates: 48° 36' N, 38° 01' E.

Date of construction:

Installed capacity: 400,000 kw.

Layout type:

Dimensions:

Structural type:

Wall covering:

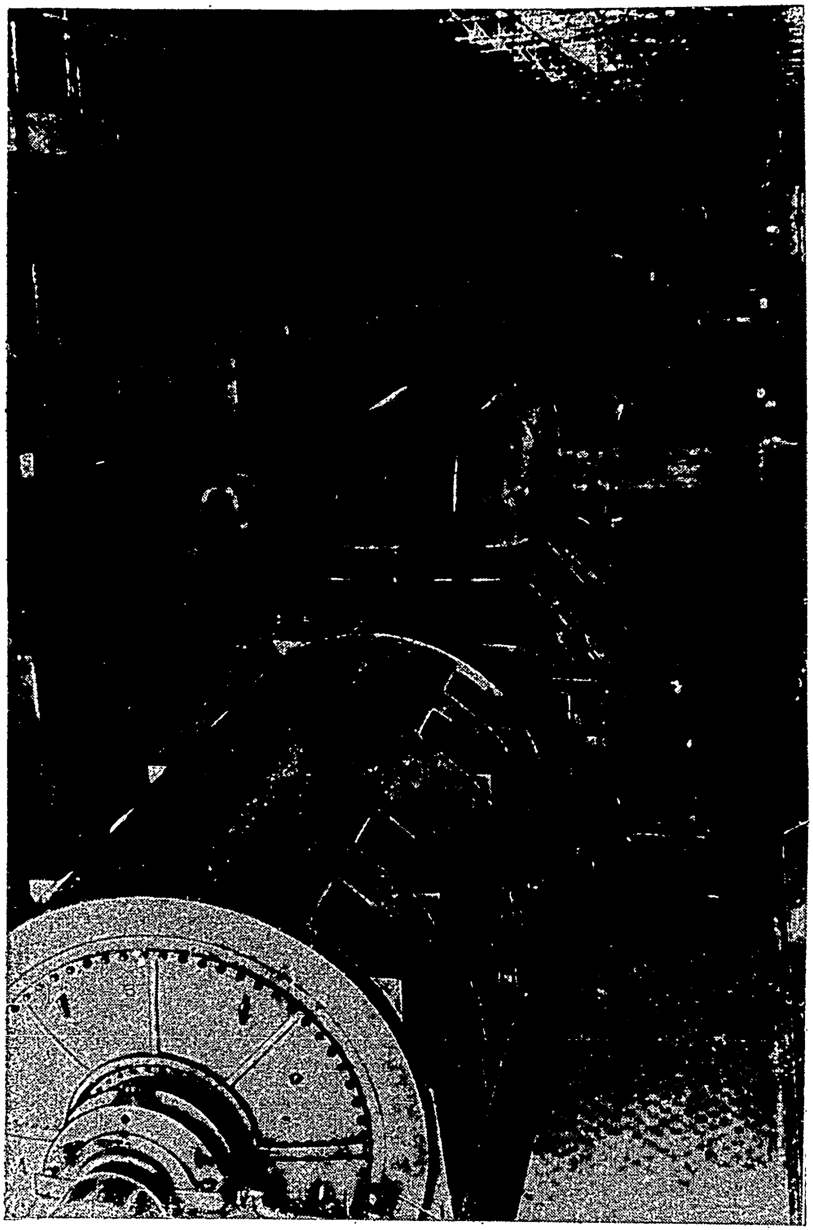
Roof construction: Turbine hall - longitudinal monitor, braced light steel trusses.

Roofing:

Cranes: In the turbine hall.

Crane girders:

Stacks:



Interior view of the turbine room.

MIRONOVSKAYA COAL-FIRING GRES NEAR ARTEMOVSK (Stalinskaya o.)
(Capacity: 400,000 kw.)

Source: Markin, A.B., Budushchyeye Elektrifikatsii SSR 1946, p. 21 (TK85.M3)

PLATE 53

SLAVYANSK GRES IN STALINSKAYA OBLAST'

(Plates: 54, 54A)

Location: Slavyansk, Stalinskaya Oblast', Ukrainskaya SSR.

Coordinates: 48° 52' N; 37° 36' E

Date of construction: 1951-1955.

Installed capacity: 200,000 kw (1955)

Layout type: Fourth type.

Dimensions: (est)

Structural type: (est) Lower part of building-reinforced concrete frame; upper part of building: steel members

Wall covering:

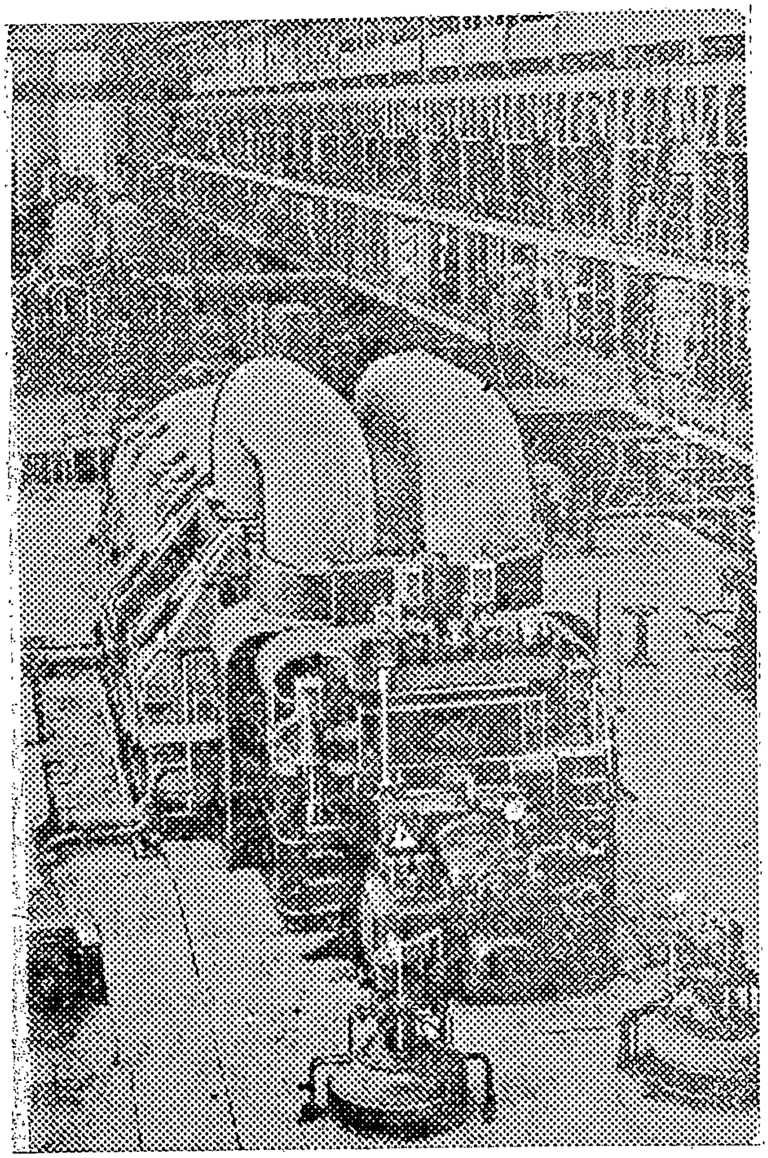
Roof construction:

Roof covering: Ruberoid over reinforced concrete slabs (est.)

Cranes: In turbine hall-est. cap. 150/30 m ton in boiler house

Crane girders: Steel solid web (est).

Stacks: Reinforced concrete (prob.)



Partial exterior view of the turbine room.

SLAVYANSK GRES

Source: Zarya Vostoka 1955, 5 Oct., #236, p. 2, (AP50.23)

PLATE 54

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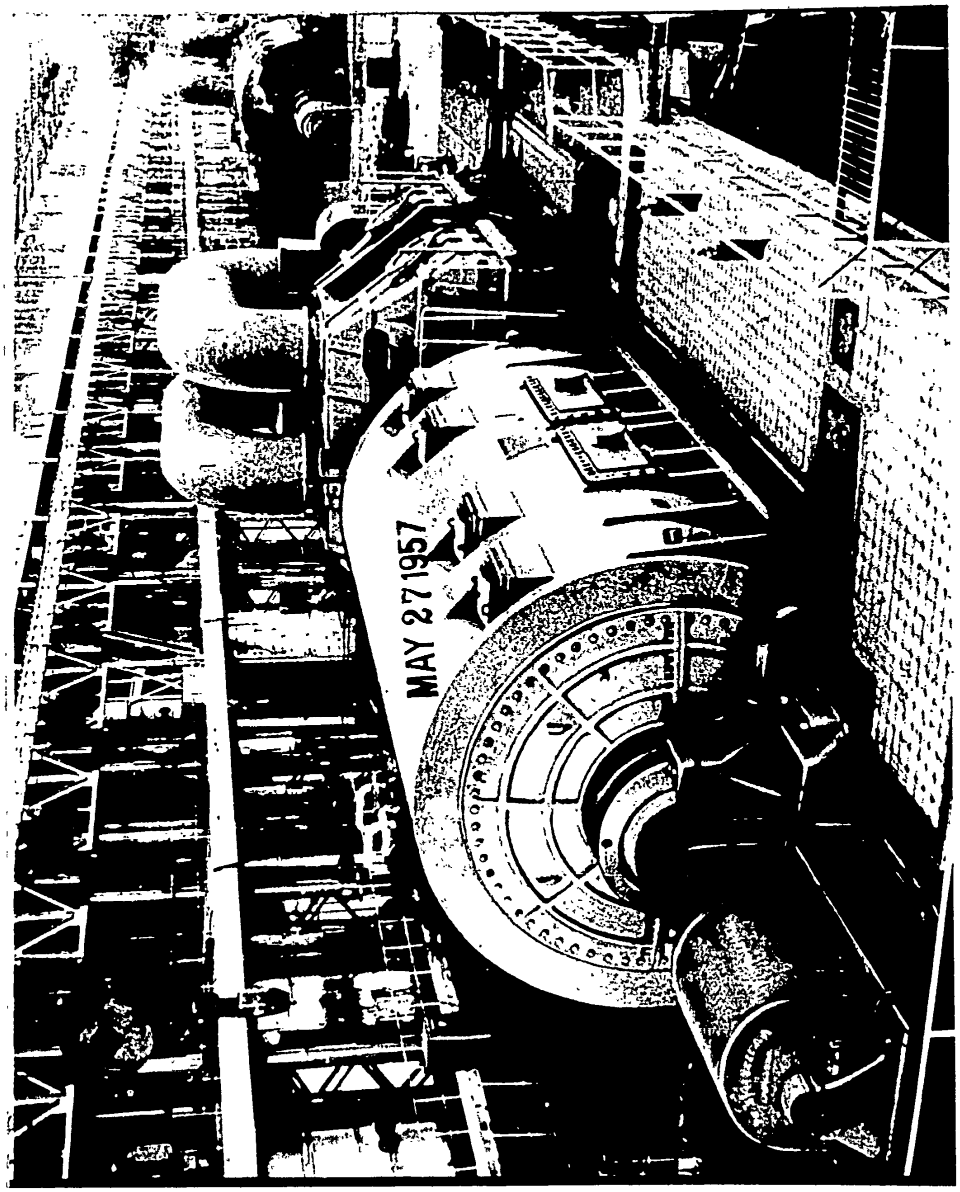

Turbine Hall

SLAVYANSK GRES

Source: Teploenergetika, 1957, #4, front cover

PLATE 54A

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CHEREPET' COAL-FIRING GRES IN TUL'SKAYA OBLAST'

(Plates: 16, 16A, 55, 55A, 55B)

Location: Town 160 miles south of Moskva

Coordinates: 54° 07' N, 36° 24' E.

Date of construction: 2nd section was under construction in 1956.

Installed capacity: 300,000 kw. in 1956; projected capacity - 600,000 kw. before 1960.

Layout type: Fourth type.

Dimensions: Overall - W. - 259.3 ft.; H. - 131.0 ft.; L - 433.0 ft.

Structural type: Steel frame.

Wall covering. Probably brick curtain walls.

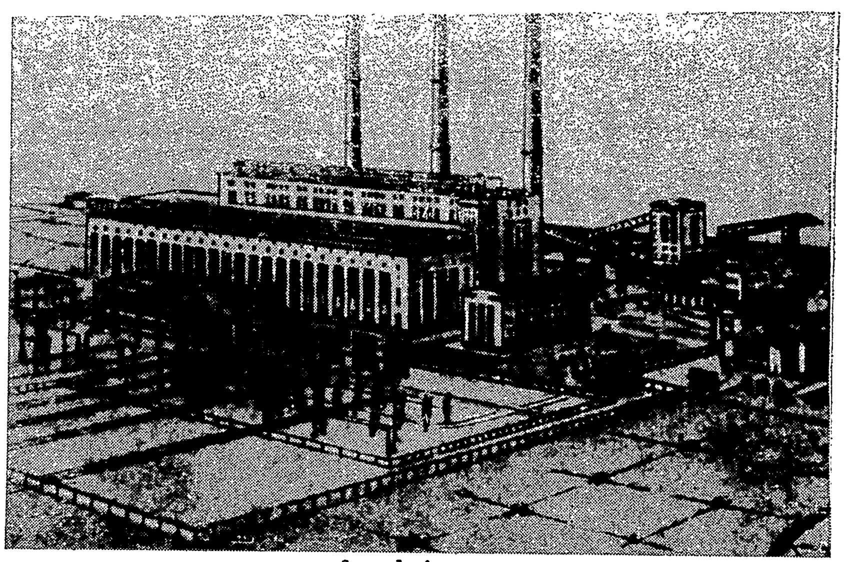
Roof construction: Boiler room - longitudinal monitor; braced light steel trusses.

Roofing:

Cranes:

Crane girders: Steel.

Stacks: 2 stacks presumably of reinforced concrete; located 52.5 ft. from the smoke exhaust installation.

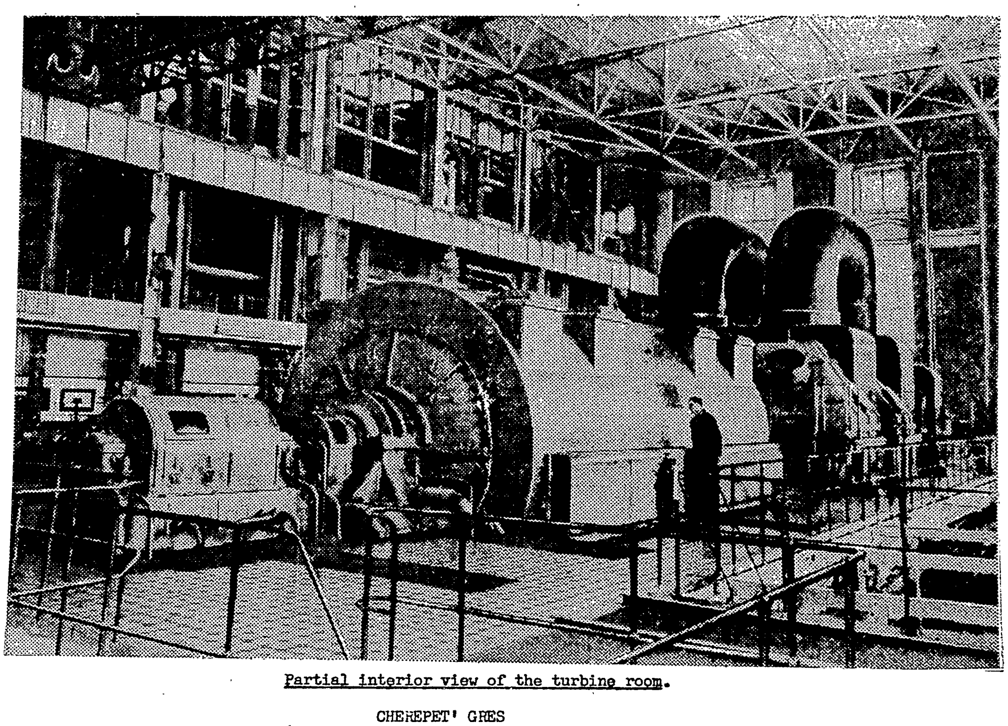


General view.

CHEREPET' GRES
(Projected final capacity: 600,000 kw.)

Source: Markin, A.B., Budushchyeye Elektrifikatsii SSSR, 1956, p. 51, (TK85.M3)

PLATE 55



Partial interior view of the turbine room.

CHEREPET' GRES
(Projected final capacity: 600,000 kw.)

Source: Izvestiya 1956, #72, p. 1

PLATE 55A

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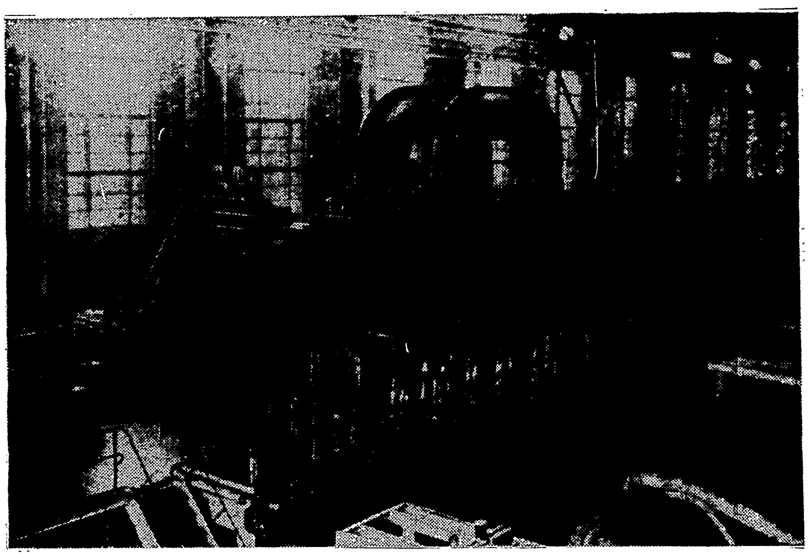

Turbine hall housing a 150,000 kw. turbine.

CHEREPET' GRES
(Projected final capacity: 600,000 kw.)

Source: Markin, A.B., Budushchyeye Elektrifikatsii SSR 1956, p. 53, (TK85.M3)

PLATE 55B

9014466

INVESTIGATIONS ON REINFORCED
CONCRETE PANELS AND DEEP BEAMS

Report No. 93

Copy No. 3

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INVESTIGATIONS ON REINFORCED CONCRETE PANELS AND DEEP BEAMS

Report No. 93

Translation Prepared
by
Air Information Division, Engineering Section
Library of Congress
for
United States Air Force
November 1957

Declassified in Part - Sanitized Copy Approved for Release 2013/02/25 : CIA-RDP81-01043R001600080008-4

INVESTIGATIONS ON REINFORCED CONCRETE PANELS AND DEEP BEAMS

Forwarded herewith is a Summary and translation of the article headed "Investigations on Reinforced Concrete Panels and Deep Beams", by Henrik Nylander. The article appeared in P: "Kugl. Tekniska Högskolans Handlingar" (Translations of the Royal Technical University), 1946, No. 2, pp. 7-53, DLC: TA445.N9.

This article gives the results of a series of photoelastic tests and static load tests to failure of deep, narrow, short-span concrete beams and of wide, thin concrete compression panels with special reference to shear and lateral tension failure. On the basis of these tests and of earlier tests and analyses cited, rules are given for placing and proportioning reinforcement in members of the shapes and functions described.

LIST OF INCLOSURES

LIST OF ILLUSTRATIONS

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Excerpt from Author's Foreword*

Reinforced concrete compression panels ["slabs"] and deep beams are used as load-bearing structural members. Numerous instances could be cited: deep grade beams in housing construction; exterior and interior bearing walls in concrete structures; walls in silos of various kinds, etc. On account of the scarcity of knowledge of the static behavior and load-carrying capacity in general, these structural members as currently used are overreinforced. In the present investigation, some of the most important problems of design have been treated. The study of phenomena (which have been as far as possible clarified) was aimed at presenting results which could be applied to actual structural designs. To enlarge our knowledge of structural problems, further investigations are needed. This is particularly true in relation to cases where the main part of the load is applied to the lower edge of the compression panel ["slab"] or beam, as for instance in certain types of silo.

The investigations in question were primarily carried out by means of tests partly on concrete slabs and beams and partly on model specimens in which the states of stress were studied by the photoelastic process.

*p. 5, Text

SUMMARY*

Comparatively few laboratory tests have hitherto been made on deep concrete beams. The tests, which have been made show however that the formation of cracks in concrete prior to failure gives rise to essential deviations from that stress distribution which is obtained from calculations based on the assumptions of the classic theory of elasticity.

The difference in static function between a deep beam and a beam of ordinary depth lies primarily in the fact that the additional stresses set up by concentrated loads in deep beams, in contradistinction to those in beams of ordinary depth, produce an effect of paramount importance on the state of stress throughout the whole beam. Furthermore, the local stresses at supports and below concentrated loads in a deep beam are larger than in a beam of ordinary depth. Special attention has therefore been given in the present investigation to the study of stress distribution and liability to failure in the vicinity of concentrated loads.

After the first cracks have formed, the stress distribution determining the design is dependent on the method of application of load. Thus, when the load is applied to the top surface of the beam, the stress distribution will be essentially different from that due to a load applied to the bottom surface.

The results obtained from the present investigation are represented in the form of design rules for beams which are subjected to loads applied to the top surface of the beam. However, the special rules set up for determining the permissible compressive stresses at supports and below concentrated loads, and the general investigation of the stress distribution in the neighborhood of concentrated loads, hold good and are of interest also in those cases where the loads are not applied to the top surface of the beam.

In designing deep reinforced concrete beams or slabs, provision must be made against the following principal types of failures:

1. Failure of concrete at a support or below a concentrated load.

2. Splitting failure or failure under tension in shearing.

3. Failure of reinforcement under tension in bending.

4. Failure of concrete under compression in bending.

The first two types of failures may be taken to be most conspicuous. Primary failure of concrete under compression in bending occurs in deep beams even more seldom than in beams of ordinary depth, and this eventuality may generally be left out of account.

*pp. 50-53, Text.

Beam design must provide, not only against failure, but also against formation of detrimental cracks. This is achieved by providing those portions of the beam which are in tension with special reinforcement for the purpose of crack distribution.

1. In studying the danger of concrete failure at supports or under concentrated loads, recourse must be had to strength tests.

The following permissible stresses can be suggested on the basis of the strength tests described in this paper.

a) When the load is applied at a corner:

   the same stress as for a column loaded in the direction of its axis, assuming that there is no danger of failure by buckling.

b) When the load is applied at a distance 3 c (c = width of support) from a free edge:

   twice the stress specified under a.

In those cases where the reactions at the supports exceed the permissible values, the strength of the beam can be increased by means of local reinforcement of the type shown in Figs. 5 and 9.

2. The liability to failure under tension of a slab (a splitting failure) subjected to concentrated loads can be estimated with the aid of Table 6 which gives the maximum horizontal tensile stress. (For notations, see Figs. 17 and 18.) This table contains a summary of the results obtained from a series of photoelastic tests made by Mr. Lars Forsblad and Bo Kõhlmark under the direction of the author, and a theoretical investigation by Goodier (see footnote on page 25) dealing with states of stress due to concentrated loads. If the tensile stress exceeds the permissible value in shear, reinforcement should be provided. The design of this reinforcement should be based on the stress distribution prevailing after crack formation. The reinforcement used for prevention of splitting is proportioned by the force computed from equation (α), page 35, or from Table 5, on the simplest possible design assumption, viz. that the line of compressive stress falls within the core area in each portion of the slab formed by cracking.

When using reinforcement against failure under tension in shear, distinction must be made between continuous beams and freely supported beams. The photoelastic tests (see Figs. 24 and 25 illustrating the lines of principal stresses and the distribution of principal stresses in cracked and non-cracked members) and the direct tests on concrete beams described in this paper have shown that no special reinforcement is required at the supports on condition that the member is provided with crack distribution reinforcement in accordance with the recommendations given in Fig. 36.

The design of the reinforcement for members resting freely on the supports should not be based on the safety against failure - after the formation of tensile cracks the member acts as a straining trestle-work provided with tension rods, and the load-bearing capacity is therefore dependent on the anchorage of the bending tensile reinforcement. For this reason, the shear reinforcement should be proportioned so as to provide safety against formation of detrimental cracks.

The intensity of the principal tensile stresses in members subjected to concentrated loads applied at 1/3 points of span was determined by means of photoelastic tests made on models of reinforced and plain beams. This investigation is described in Part II by my collaborator, Mr. Hans Holst, C. E. An example of the distribution of these stresses is shown in Fig. 20 ($\underline{1/h}$ = 413) [sic probably $\underline{1/h}$ = 4/3], and the values of principal tensile stresses are given for various span to depth ratios $\underline{1/h}$ in Table 7. As may be seen from Fig. 20, recourse must be had to a subjective estimate in determining the range where the dangerous tensile stresses in shear are not influenced by the bending cracks, and Table 7 gives therefore the approximate limit values. In those cases where the principal tensile stress given in Table 7 exceeds 1.5 times the permissible tensile stress in shear, the members should be provided with reinforcement.

The design of the reinforcement is based on the requirement that this reinforcement should be proportioned by the force needed in order to keep in equilibruim that portion of the beam which is separated by a probable tensile crack.

The procedure used in designing "shear" reinforcement is illustrated in Fig. 21. The tensile force acting on the shear reinforcement S_j and the requisite area of shear reinforcement computed on the basis of the suggested permissible value of steel stress are given in Table 8. The results thus obtained can also be applied to the conditions of loading shown in Fig. 22 where, for large values of the ratio $\underline{b/a}$, the state of stress in the neighborhood of the right-hand support is essentially the same as under the conditions of loading dealt with in this investigation.

3. The design of bending tensile reinforcement has hitherto generally been based on the stress distribution in a homogeneous beam. The reason is obviously to be found in the imperfect knowledge of the effects of cracking. With a view to elucidating the function of members which are loaded so heavily that crack formation exerts an influence on the stress distribution, the author has made a series of tests. In order to throw light upon the interaction of the moments at the supports and between the supports, the tests were made on continuous beams resting upon three supports. The shape of the test beams and their reinforcement are shown in Fig. 26-a and 26-b. The reinforcement of the beam shown in Fig.26-a (beam No. 1) was designed on the basis of the probable stress distribution in the cracked beam,

obtained from the photoelastic tests, that is to say, the reinforcement at the supports was placed for the most part near the upper surface of the beam. In the beam shown in Fig. 26-b (beam No. 2), on the other hand, the reinforcement was designed on the basis of the state of stress prevailing in a homogeneous non-cracked beam. The steel stresses were measured by means of tensometers which were located as shown in Fig. 26. Photographs illustrating crack formation are reproduced in Fig. 31. The main results of the test can be summarized as follows.

After the beginning of crack formation, the reinforcement at the supports and between the supports was on the whole subjected to equal stresses, irrespective of whether the reinforcement was designed on the basis of the stress distribution in a homogeneous non-cracked beam, as in beam No. 2, or on the basis of the probable stress distribution in a cracked beam, as in beam No. 1. Since the inner lever arm of the tensile force acting on the steel in beam No. 1 was about twice that in beam No. 2, it is evident that the reinforcement used in beam No. 1 is preferred from the point of view of load-bearing capacity. As regards crack formation, the two alternatives referred to in the above may be regarded as equivalent. Throughout the loading range up to 60 percent of the ultimate strength, the width of cracks in beam No. 1 did not exceed 0.1 mm at any point.

The bending reinforcement required under several typical conditions of loading is shown in Fig. 36. The present investigation has shown that no additional shear reinforcement except the bent-up bars is needed at the central supports. At the end supports, on the other hand, shear reinforcement should be provided in accordance with the recommendations given under 2.

PREFACE

This is a full translation of the assigned text, as close to literal as is consistent with the tranlator's objective of making the author's exact meaning clear to the technically trained English-speaking reader. In particular, ambiguities are avoided. Under this program, a word-for-word dictionary translation was out of the question, particularly as some important words are not given in any dictionary available at the Library of Congress, and in other cases no translation given by the dictionaries is appropriate to the sense of the text.

Attention is called to the procedure followed in translating the Swedish term skiva (plural: skivor). This word is widely used to mean slab; and the test specimens are indeed of slabby shapes. But to the American engineer the term slab in the present connection inevitably recalls the familiar problem of analyzing floor and roof elements - flat members loaded and supported at right angles to their broad face. To avoid confusion, therefore, the tested members here have been designated by their load-bearing functions as deep narrow beams and wide thin struts, or compression panels.

In the translation, the term stress has been used to mean internal forces, and strain for deformation in the material of the stressed element.

Inclosure #1 to AFCIN-1A1 IR - 1804 - 57 31 October 1957

INVESTIGATIONS

ON REINFORCED CONCRETE PANELS AND DEEP BEAMS

by Henrik Nylander

(Stockholm, 1946).

Inclosure #2 to AFCIN-1A1 IR - 1804 - 57 31 October 1957

1. INTRODUCTION

For the last twenty years design rules for reinforced concrete deep beams have been established by different researchers in the field of static theory (1-15*).

The general procedure has been to determine the design for a stress distribution based on the assumptions provided by the classic theory of elasticity. In only two investigations initiated in recent years have the effect of certain specific properties of reinforced concrete been tested (13-15*). In these tests simply supported beams were under discussion. In an article, Mr. Bay (13*) analyzed the extent to which the formation of cracks preceding the yield point in the reinforcement affects the internal lever arm, i.e. the distance between the compression and tension centers in a beam with the depth greater than the span.

Mr. Bay's analysis shows: that crack formation is accompanied by an increase of the internal lever arm over the value in stage I; that the proportioning of reinforcement based on the initial state prior to cracking implies an unjustified increase in the safety factor; it thus can be assumed that tension cracks appear in concrete before the yield point in steel bars is reached.

Mr. Bay, together with O. Graf and E. Brenner, carried out a laboratory test on a simply supported deep beam with loads applied to the bottom surface of the beam (14*). The resulting report carries an announcement regarding forthcoming additional investigations which due to circumstances have probably been suspended. The results thus obtained from the investigation show that the stress distribution depends to a great extent on the depth of embedment of the suspension steel transmitting the exterior loads to the beam. In the final analysis it was pointed out that when designing a very deep beam (in the test specimen the ratio of the

*[See Bibliography]

depth to the theoretical span was equal to 2.2), more attention should be paid to principal oblique tensile stresses than to principal tensile stresses due to bending.

Finally, Horst Klingroth has carried out large scale tests on simply supported beams with the depth equal to the theoretical span loaded at the upper surface (15). On the basis of Klingroth's tests, it appears that in beams with ordinary depth the tensile stresses in bending are of less importance than the oblique principal tensile stresses, and especially than the local compressive stresses at supports.*

When scrutinizing the above-mentioned test, which was carried out on simply supported deep beams, it appears that stress distribution gives rise in certain respects to essential deviations from that stress distribution which is obtained from calculations based on the classical theory of elasticity. This test indicates that the progressive crack formation produces an effect of great importance on the state of stresses throughout the reinforced concrete beam. The formation of cracks varies in character, depending on the method of load application: when the load is applied to the top surface of the beam, the stress distribution will be essentially different from that due to a load applied to the bottom surface. Therefore, when determining the design rules for beams, it is essential to distinguish these two principal cases. The investigation that follows treats the case of beams subjected to loads on the top surface.

In designing deep reinforced concrete beams on the basis of test-proved factors, provisions must be made against the following principal types of liability to failure:

1. Failure of concrete at a support or below a concentrated load;
2. Shear cracking or failure under tension in shear [diagonal shear];
3. Failure of reinforcement under tension in bending;

*This test will be discussed in Section 2.

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4. Failure of concrete under compression in bending.

The first two types of failure may be taken as the most conspicuous. Primary failure of concrete under compression in bending occurs in deep beams even more rarely than in beams of ordinary depth, and may consequently be disregarded.

Beam design must provide safeguards not only against failure but also against formation of detrimental cracks. This is achieved by providing those portions of the beam which are in tension with special reinforcement designed for crack control.

The foregoing classification serves as a basis for further discussion.

2. LOCAL FAILURE UNDER COMPRESSION AT SUPPORT OR UNDER CONCENTRATED LOAD

Fig. 1 shows beam failure resulting from tests carried out by Mr. Klingroth. The arrangement of supports is seen in Fig. 2, and views of tested beams together with various conditions of loading are given in Fig. 3.

The U-shaped steel saddles clamping the beams were primarily designed to effect the centering of the load in plan. In addition, they prevented the transverse expansion of the concrete and thereby possibly raised the value of compression at supports.

Data on values at failure for various beams are tabulated in Table 1, under the assumption that the load pressure and compression at supports are equally distributed. Location of failure is indicated by compression figures not enclosed in parentheses.

The failure at support was probably facilitated by the manner in which the reinforcement was placed: as seen in Fig. 1, the spalling in the concrete which had occurred in the corner at the supports is located outside the reinforcement. On the other hand, as mentioned previously, the U-saddles shown in Fig. 2 had possibly increased the ultimate load.*

The author had carried out loading tests on three concrete panels (as shown in Fig. 4) in order to gain information on the effect upon the ultimate load when local load is applied near the edge of the panel, on the other hand, to determine what increase in strength can be achieved by introducing special reinforcement designed to limit the transverse expansion of concrete at a right angle to the plane of the panel in the vicinity of the local load. In these loading tests the panels were provided with excessively heavy transverse reinforcing bars (5 pieces 20 mm. = 0.8 in. in diameter, Ser. 52) designed to isolate the local failure in

*In beam 1-a this increase could possibly be relatively slight because the flanges of the U-saddles showed outward bending before failure, due to lateral compression.

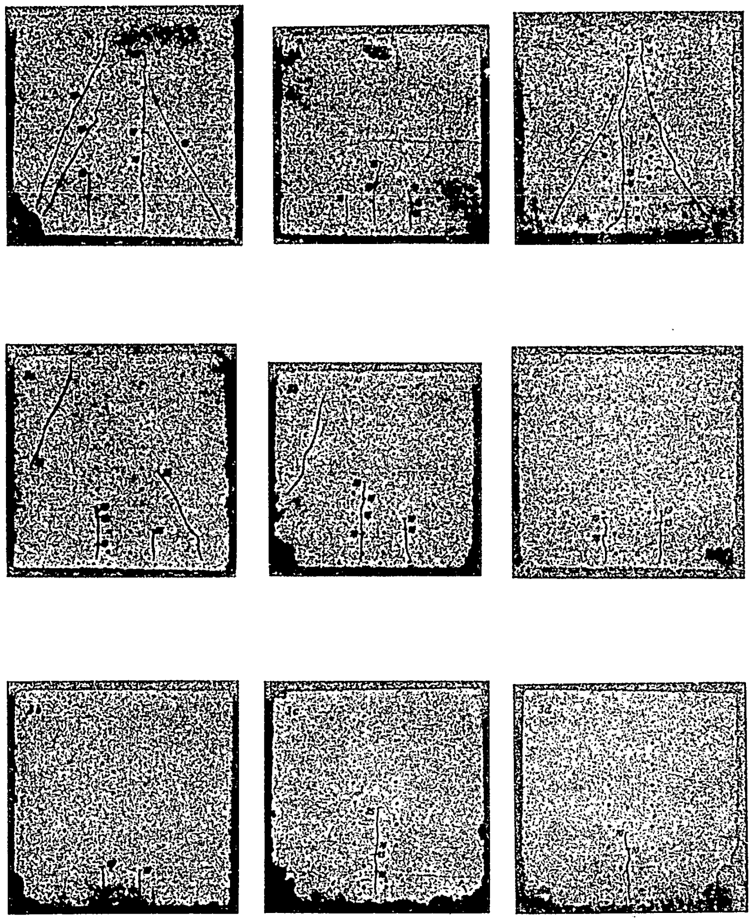


Fig. 1 - Post-Failure View of Beams Tested by Klingroth

Loading methods and notation are shown in Fig. 2 and 3.
Figures are reproduced from Klingroth's account (15*).

*[See Bibliography]

Beam No.	1-a	1-b	1-c	2-a	2-b	2-c	3-a	3-b	3-c
Ultimate Load in m. t.	18	21	20	27	26	25	26	27	26
Compression under Load kg/cm^2	(260)	300	285	(195)	(185)	(180)	(185)	(195)	(185)
		{1.95}	{1.85}						
lb/in^2	(3700)	4250	4050	(2780)	(2640)	(2560)	(2640)	(2780)	(2640)
Compression at Support kg/cm^2	130	(150)	(143)	195	185	180	185	195	185
	{0.85}			{1.23}	{1.20}	{1.17}	{1.20}	{1.23}	{1.20}
lb/in^2	1850	(2140)	(2040)	2780	2640	2560	2640	2780	2640

Table 1. Data on Compression Under Load and at Supports, According to Klingroth's Test.

The strength of the concrete was established in control tests: by means of cubes 20 cm. (7.9 in.) on each side and with prisms 20 x 20 x 80 cm^3 (7.9 x 7.9 x 31.6 in.). As the number of control tests that had taken place is not indicated in the report on the test results, the strength values:

$$\sigma_{cube} = 223 \text{ kg/cm}^2 \text{ (3180 lb/in}^2\text{) and}$$

$$\sigma_{prism} = 154 \text{ kg/cm}^2 \text{ (2200 lb/in}^2\text{).}$$

should be used cautiously. The values shown in braces [{}] signify the relation between compression at failure and the strength of prisms.*

*The strength of prisms was selected for purposes of comparison, while the load-distributing pad was designed to bring about a favorable condition of bearing between loading surface and the concrete.

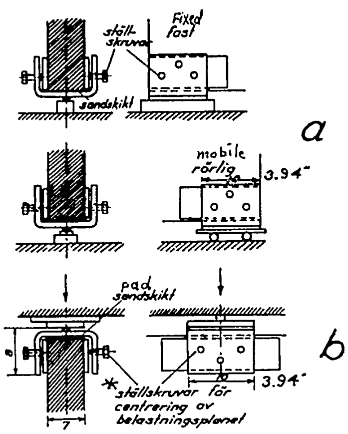


Fig. 2 - Set-up in Klingroth's Test

a. At the Supports
b. Below Loading Force

*Screws for Centering of Loading.

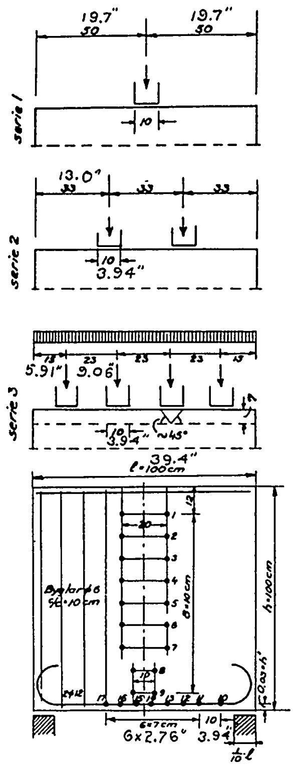


Fig. 3 - Loading Condition and Beam Dimensions in Klingroth's Test

1. Eccentric Loading in Load Test B in Scheme III Without Local Reinforcement.

2. Center Loading in Scheme I and II with Local Reinforcement and Load Test A in Scheme III Without Local Reinforcement.

3. Eccentric Loading in Load Test C in Scheme III Without Local Reinforcement.

4. Local Reinforcement in Scheme I and II

5. Local Reinforcement in Scheme I and II

All Dimensions in mm.

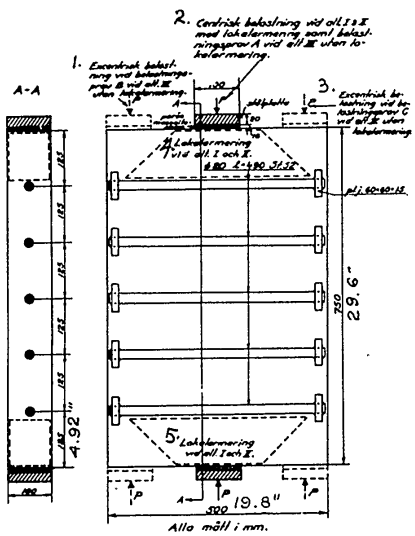


Fig. 4 - Reinforcement and Dimensions in the Test of Partial Compression

Inclosure #10 to AFCIN-1A1 IR - 1804 - 57 31 October 1957

concrete under compression at the points where load is applied. Table 2 summarizes the most significant results of these loading tests.

Moreover, the panels in Schemes (alternative) I and II were provided with local reinforcement, as shown in Fig. 5, designed to limit transverse expansion of concrete under load, and thus to increase local compression. Two alternative forms were obtained as a result of this local reinforcement: Schemes 1 and 2. Scheme 2 differs from Scheme 1 in that the spacing between the transversely held round bars was half of the spacing in the respective bars in Scheme 1. Local reinforcement was omitted in Scheme III.

In Schemes I and II the concrete began to show flaking at a load of 40 to 50 m.t., that is, at approximately the same load at which failure occurred in the unreinforced panel.

Types of failure are shown in Fig. 6; a view of the local reinforcement after failure is given in Fig. 7.

The test results may be summarized as follows:

1. In partial loading of the panel, assuming that failure by splitting is prevented, the value of the ultimate load depends on whether the load is applied at the corner or at the center portion of the panel.
2. In applying the load at the corner, the failure formation and the failure value depend to a great extent on the manner in which the reinforcement is provided at the corner. In Klingroth's test it was shown that in the case where the supporting U-saddles were ineffective, the ratio between the ultimate compression at the support and the resistance in the prism was $\frac{\sigma_k}{\sigma_p} = 0.85$. The

ratio between compression at supports and cube resistance $\left(\frac{\sigma_k}{\sigma_{cube}}\right)$ in this experiment was equal to 0.78 which corresponds to:

$$\frac{\sigma_k}{\sigma_p} = \frac{0.78}{0.80} = 0.97$$

3. In Klingroth's test with load applied at the center portion, the result was $\frac{\sigma_k}{\sigma_p} = 1.90$, and in this test: $\frac{\sigma_k}{\sigma_{cube}} = 1.60$, or $\frac{\sigma_k}{\sigma_p} = \frac{1.60}{0.80} = 2.0$.

4. When the panels were provided with local reinforcement, as shown in Fig. 5, the result obtained was $\frac{\sigma_k}{\sigma_{cube}} = 2.43$, or $\frac{\sigma_k}{\sigma_p} = 3.0$. Failure occurred outside the local reinforcement; consequently the panel was overreinforced. Even under the same load as that which caused failure without local reinforcement, scaling occurred in the concrete outside the local reinforcement similar to what had been observed in testing concrete columns with spiral reinforcement.

Type of Load	Ultimate Load in m.t.	Unit Compression Under Load, σ_k		Character of Failure	Resistance of cube, σ_{cube}		$\frac{\sigma_k}{\sigma_{cube}}$
		kg/cm²	lb/in²		kg/cm²	lb/in²	
Scheme I, Fig. 4, Local Reinf. as in Scheme 1, Fig. 5	68.0	680	9650	Scaling Outside Local Reinf. Crushing Under Local Reinf.	281 (275, 285, 283)	4000 (3900, 4050, 4025)	2.42
Scheme II, Fig. 4, Local Reinf. as in Scheme 2, Fig. 5	69.0	690	9800	Same as Above			2.45
Scheme III, as in Fig. 4, No Local Reinf. Test Load A (Central Load)	40.0	400	5700	Crushing Under Load			1.46
	49.8	498	7100	Same as Above			1.82
Scheme III, as in Fig. 4, No Local Reinf. Test Load B (Eccentric Load)	24.0	240	3400	Crushing Under Load and Spalling of the Corner	273 (274, 268, 277)	3890 (3900, 3800, 3960)	0.79*
Scheme III (Fig. 4), No Local Reinf. Test Load C (Eccentric Load)	23.5	235	3340	Same as Above			0.77*

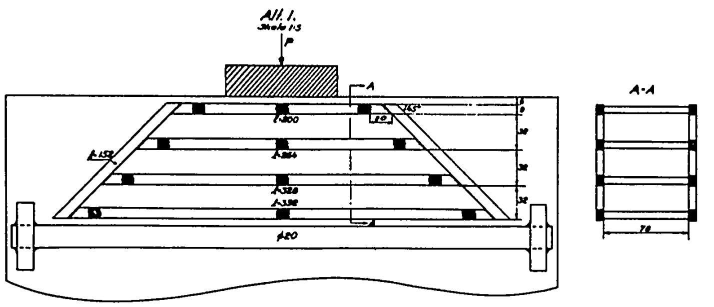
Table 2. Results Obtained from Compression Test of Concrete Panels Under Concentrated Loads (as in Fig. 4).

The proportioning of concrete consisted of the following aggregates: A-cement: sand: coarse gravel (1:3.7:4).

Water-cement ratio 0.70 (7.9 gal/sack). Slumps varied between 3.5 and 4.5 cm. (1.4 and 1.8 in.); maximum size: 28 mm. (1.7 in.).

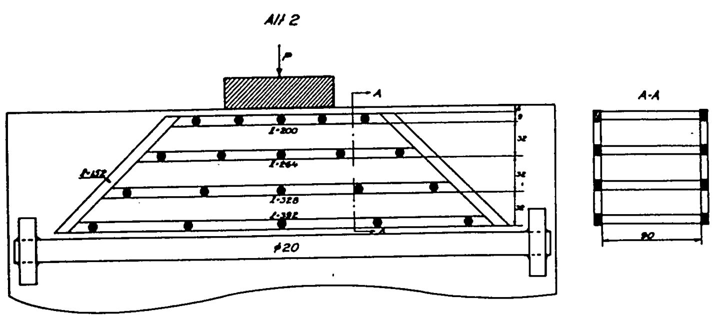
In test load III, Scheme A, renewed loads up to failure could be applied, since crushing failure occurred at one loaded area. In another test, the panel was subjected to load until failure on one side occurred (because the other side was so damaged that the panel could not be subjected to a renewed load). In all other tests, except in Scheme IIIA, the ultimate load indicated the lowest value of the two tests.

*Age of concrete at test: for IIIB and IIIC - 44 days; the rest - 28 days. The concrete cubes were tested after 28 days, and in Schemes IIIB and C, 44 days; the ratio σ_k/σ_{cube} observed for these Schemes was multiplied by 0.90.



All Dim. in mm.

All ▭ Bar Section: 1/4" x 3/8"
Total Length 2 x 3.95 = 8.0 m.
(26.3 ft.)



All Dim. in mm.

All ▭ Bar Section: 1/4" x 3/8"
Total Length 2 x 2.98 = 6.0 m. (19.7 ft.)
Diam. of ○ Bars 8 mm. (0.3")
Total Length 2 x 1.8 = 3.6 mm. (0.1")

Fig. 5 - Shapes of the Local Reinforcement Used in the Test

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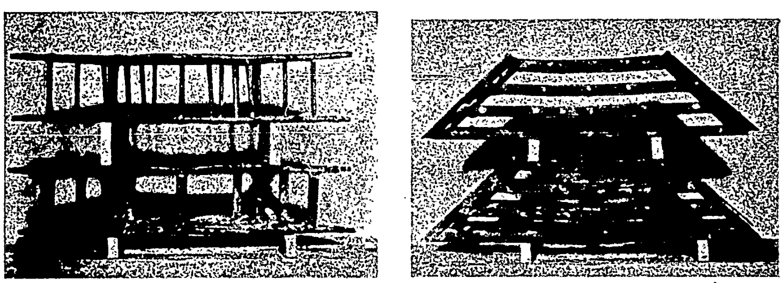
Fig. 6 - Views of Post-Failure Panels in Schemes I and II.

Attention is drawn to the extensive spalling of concrete outside the plane of reinforcement and to the bursting failure in panel I over its entire width.



a b

Fig. 7 - View of the Local Reinforcement after Failure

a. Seen from below
b. Side view

Photos show the steel cold-bent sidewise and vertically.

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Considerations of Design

Fig. 8 shows two methods of anchoring reinforcing bars which obviously affect differently the value of critical stresses at supports. The form at the left support prevents damage of the corner, while the hook at the right support facilitates it. The alternative at left was not tested, but judging from all factors involved, the method of anchorage would presumably afford better results than Schemes IIIB and IIIC, which were tested. The anchorage method shown at right should be avoided because the ultimate load for undertermined reasons can drop, owing to the cracking effect of the hook anchorage with a bent-up end. In the arrangement at left (or similar cases) the permissible reaction at the support and the permissible stresses might be taken as equal in accordance with concrete specifications for columns loaded in the direction of their axes, assuming that there is no danger of failure by buckling.

If the width of supports c is limited to a certain maximum value, (c = width of support) and the reaction at supports exceeds the permissible values, the next best solution is to resort to an interconnection of column and beam such as that shown in Fig. 8-b, i.e. by extending the column part above the support level. In this way the force at supports is partly transmitted to the column from the beam by vertical shearing stresses.

Should the extended column seem cumbersome, recourse could be had to local reinforcement of the same type as that used earlier in the test. On account of the short distance between the support and the corner, a modification in the local reinforcement is required, consisting of vertical bars with straight ends (see Fig. 27-a). It is estimated that with very strong local reinforcement of the type shown in Fig. 27-a, the permissible reaction at supports could be raised

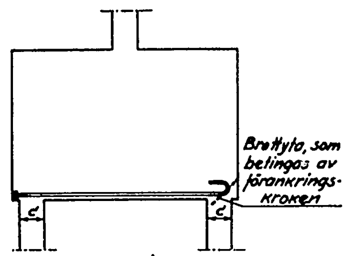
50%* (above that without local reinforcement).

When concentrated load is applied at an adequate distance from the wall corner ($\gtrsim 3c$, where c = column width at the loaded point), the permissible stresses at the loaded area could be estimated as twice the permissible stresses established by concrete specifications for centrally loaded columns, assuming there is no danger of failure by buckling.

As in the case of loads applied at the corner, two alternatives could be presented if the width of supports cannot be adequately increased: to bring the column section above the support level, or to provide local reinforcement below the loaded area. If overreinforced, an increase of permissible compressive stresses up to approximately 50% will result.

An alternative form of reinforcement as a safeguard against crushing is shown in Fig. 9, consisting of exterior steel plates rigidly secured by bolts passing through the concrete.

*No special test was made on the effect of local reinforcement at the corner. However, besides elimination of corner spalling, the local reinforcement (Fig. 27-a) brings about a spread of stresses at supports of the same type as at loads applied in the center of the panel, where the increase of local compression by means of local reinforcement was 50%.



Failure Area depending on the Anchorage

Fig. 8-a - Two Methods of Anchorage Variously Affecting the Values of Critical Stresses at Supports

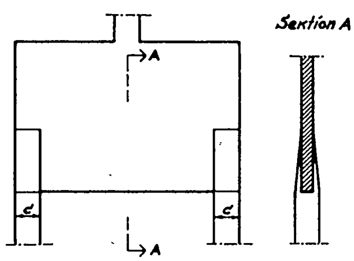


Fig. 8-b - At High Stresses at Supports Exceeding Permissible Values, the Column can be Extended Partway up the Wall.

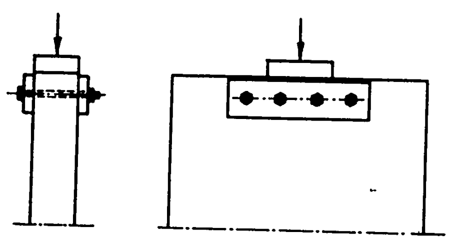


Fig. 9 - Local Reinforcement Consisting of Steel Plates Secured with Bolts to Both Sides of the Concrete Panel

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3. REINFORCEMENT AGAINST SPLITTING

A. Compression Panel on a Fixed Support

Fig. 10 shows a typical case of loading where tensile stresses resulting from concentrated load influence the design.

The horizontal stresses in the vertical section of symmetry of the panel vary from compression to tension immediately under the load at a short distance e from support, as shown in the diagram. If the concrete panel has no reinforcement, these tensile stresses cause failure at a value of P sufficient to bring about the splitting of the panel.

The situation is similar if the panel is embedded in its support at its horizontal section of symmetry (as shown in Fig. 10-b). As the transverse deformation of the panel is thus prevented by the support, the horizontal tensile stresses are weaker than in the loading case shown in Fig. 10-a.

To find the magnitude of horizontal tensile stresses and danger of failure in concrete, photoelastic and theoretical investigations have been carried out, dealing with stress distribution due to concentrated loads.

1. Photoelastic Tests

Four specimens of isolon were tested in the photoelastic investigations carried out in 1941 by Lars Forsblad, C. E., and Bo Köhlmark, C. E., under the direction of the author (as an examination paper on structural mechanics). Dimensions of the test specimens are presented in Table 3 and the notation in Fig. 11*.

*For details on properties of the specimen - an unpigmented resin belonging to the phenol-formaldehyde group - and methods of photoelastic investigations see H. Nylander, "Några spänningsoptiska undersökningar." Tekniska skrifter, No. 101. Stockholm och Norrköping, 1944.

Test No.	$\frac{h}{a}$	a	
		mm.	in.
1	4	80	3.1
2	2	70	2.8
3	1	60	2.4
4	1/2	70	2.8

Table 3. Dimensions of Specimens Used in the Photoelastic Tests.

In other tests the thickness of panel was 10 mm. (0.4 in.).

Fig. 12 gives the lines of principal strain for various cases, and Fig. 13 the distribution of horizontal tensile stresses along the vertical section of symmetry. Tensile stresses are expressed as percentages of σ_k, which is $\frac{P}{\frac{1}{10} a}$. The stresses are calculated from lines of principal elastic strain, isoclinic and isochromatic lines by graphical integration. The best obtainable precision was 5-10% for maximum values, and even poorer for the smaller values. The curves for various lateral ratios take into account the analogies between the four setups [by plotting h as unity for all curves]. The maximum [tensile] stress for the lateral ratio, $h/a = 1/2$ for instance, is 80% greater than that for the specimens of greater relative length. This result is to be expected, in view of the fact that the load concentration on the lower surface makes a greater contribution to the maximum values for tensile stresses at the upper surface than it does in the case of other lateral ratios. On the other hand, the curve for lateral ratio $h/a = 1$ should lie somewhat above that for $h/a = 2$ and 4.

Strikingly enough, the horizontal stresses very near the loaded area are compressive. Immediately under the load an almost hydrostatic stress condition prevails, with a compressive stress value in all directions equal to $\frac{P}{\frac{1}{10} a}$ and changing in all lateral ratios to tension at a distance approximately $0.7\,c$ from the loaded edge - a situation to be taken into consideration when providing reinforcement against splitting.

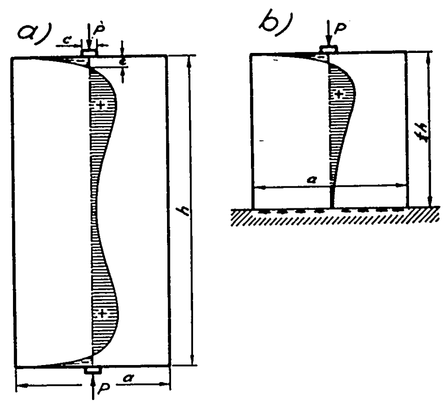

Fig. 10 - Diagram (Typical view) of Distribution of Horizontal Strain Along the Vertical Line of Symmetry in Two Fundamental Loading Cases

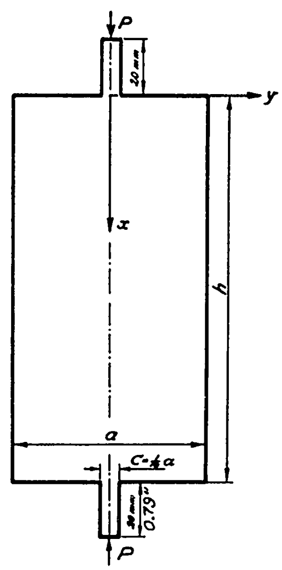

Fig. 11 - Meaning of Symbols and System of Coordinates

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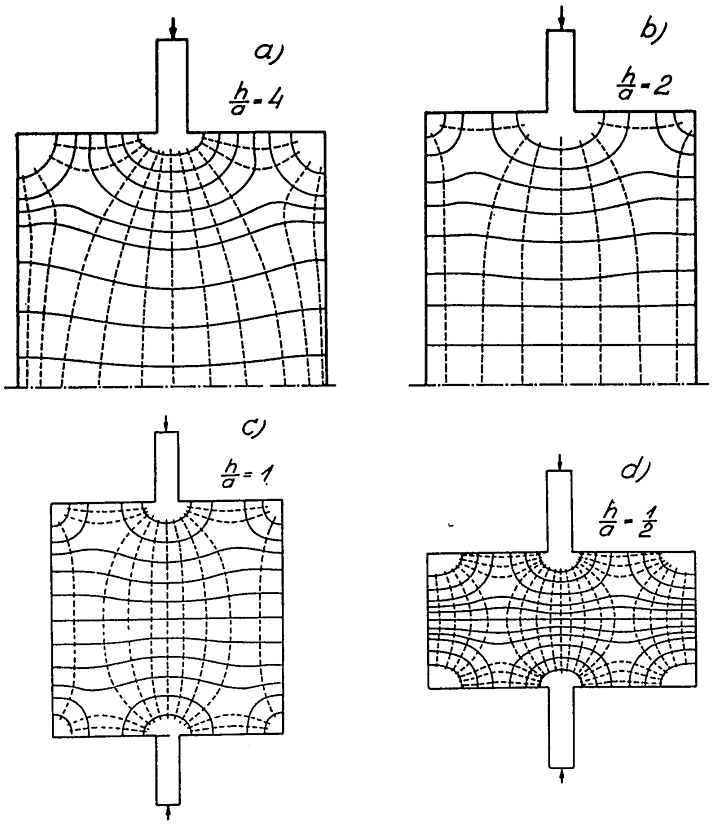

Fig. 12 - Principal Lines of Strain Drawn from the Photoelastic Test

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The broken-line curves in Fig. 13 represent the results obtained from calculations made by Mr. J. N. Goodier and based on the mathematical theory of elasticity.* In view of the fact that only a single report on results of calculations was available, it is difficult to express an opinion on its value; nevertheless, consideration should be given, whether they are made by Goodier or obtained from photoelastic tests. Any deviation would have no important bearing.

In this photoelastic investigation only one c/a ratio (= 1/10) was studied. However, Goodier's calculations, in accordance with St-Venant's principle,** have shown that the spread of the load over the bearing area strongly affects the stress distribution in only one section at the distance c from the loaded area. Therefore, the obtained result can be applied even to other c/a ratios.*** In view of the fact that it represents the total of the concentrated load which affects the state of stresses at distances $> c$ from the loaded area, the value δ shown in Fig. 13 for other c/a ratios should be multiplied by $c/a \cdot 10$.

The position of zero stress depends, however, on the c/a ratio. For $c/a = 1/10$ the zero lies at a distance of $0.7\,c$ from the loaded area, as shown elsewhere. For $c/a = 1/5$ and $h/a = 2$ Goodier's curves give a corresponding distance of $0.55\,c$ and for $c/a = 20$, and $h/a = 1/2$ the distance is $0.6\,c$. It could, therefore, be deduced that for various anticipated cases the zero lies at a distance of 0.6 to $0.7\,c$ from the loaded area.

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* See: J. N. Goodier: "Compression of Rectangular Blocks, and the Bending of Beams by Non-linear Distribution of Bending Forces." Transactions of the American Society of Mechanical Engineers.

** ["All distribution of forces on the ends of a beam which are statically equivalent will produce the same distribution of stress at some distance from the end". (Translator's note)].

***This is valid for $c/a < 1/5$.

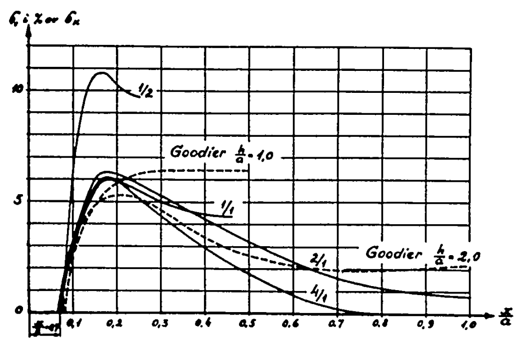


Fig. 13 - Diagram of Horizontal Stress Distribution Along the Vertical Line of Symmetry Obtained from the Photoelastic Test on Thin Compression Panels, According to table 3, Fig. 11, and from Goodier's Calculations

According to results obtained from the photoelastic studies, the compressive stresses in a vertical plane near the load (up to a distance equal to the spread of the load) were mainly calculated under the assumption that compression radiating from the load area in a 45° direction is uniformly distributed.

2. Test on Reinforced Concrete Compression Panels

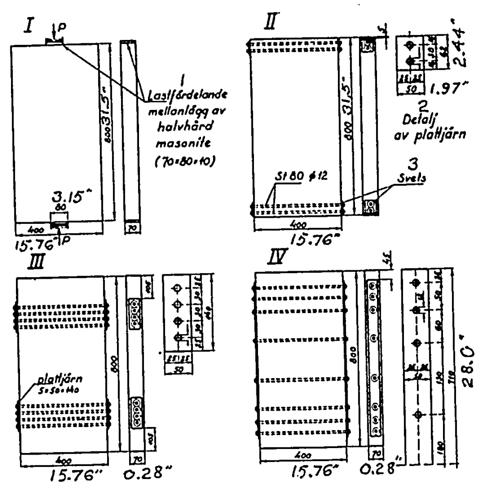
To secure data on the special phenomena occurring in concrete with load pattern discussed earlier, a series of tests on panels were performed, as shown in Fig. 14*. One test (Alt. I) was made with plain panels, the other three with transverse reinforcement variously distributed along the height of the panels. The reinforcing steel bars were of grade St. 80 with tensile yield point of 4000 kg/cm^2 (57000 lb/in^2).

To ensure safe anchorage, the ends of the bars (as shown in Fig. 14) were welded to plates projecting beyond the vertical edge.

A load of three tons was built up in one half minute and held continuously for five minutes. Fig. 15 shows a panel preassembled for testing. The plates at supports, of dimensions 200 x 50 x 10 mm^3 (7.9 x 2.0 x 0.4 in^3), serving to prevent local crushing, were pressed against either side of the panel by clamps clearly visible on the photograph. They proved effective, for wherever a crushing failure occurred, it always originated below the plate. Strain was measured and the cracks traced. A beam was tested in each alternative. The principal results of the test are tabulated in Table 4.

In Alt. I failure occurred immediately after the first crack had appeared. Simultaneously with the formation of a vertical center crack, horizontal cracks began to appear at the outer edge of the panel. The post-failure view is shown

*The test was part of the examination paper on structural mechanics prepared in 1942-43 by Baltzar Klingberg, C. E., and Åke Sandberg, C. E., under the direction of the author.



1. Load distributing pad of semi-hard Masonite (70 x 80 x 10)
2. Details of Steel Plate
3. Weld

Fig. 14 - Dimensions and Reinforcement in the Test on Spalling in Concrete Panels

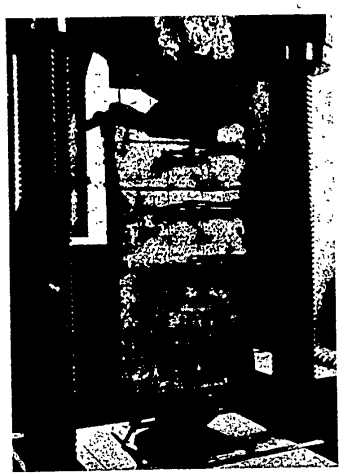


Fig. 15 - Compression Panel as Set-up for Test

Panel No.	First Observed Spalling					Failure				Cube side 15 cm (6 in) Cube Strength	
	Load m.t.	Compression Under Load		Maximum Tensile Stress		Load m.t.	Compression Under Load		Type of Failure		
		kg/cm^2	lb/in^2	kg/cm^2	lb/in^2		kg/cm^2	lb/in^2		kg/cm^2	lb/in^2
I	24.7	440	6250	25.0	356	24.7	440	6250	Spalling and secondary horizontal tension crack at outer edge		
II	27.2	485	6900	27.8	396	28.1	500	7100	Same as I	410 423 404 422 445 Average Value: 420	5840 6000 5750 6010 6340 5980
III	33.0	590	8400	33.5	476	44.7	800	11400	Crushing under load		
IV	23.5	420	5980	24.0	341	41.5	740	10500	Same as III		

Table 4. Results of Tests on Transverse Reinforcement in Concrete Compression Panels Loaded with Two Opposed Concentrations, as Shown in Fig. 14.

Fig. 16-a - Compression Panel
After Failure.
Scheme I

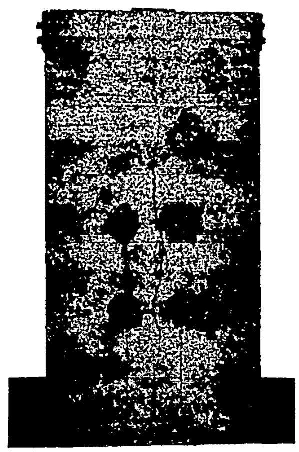


Fig. 16-b - Compression Panel
After Failure.
Scheme II

in Fig. 16-a. After the vertical center crack appeared, the panel obviously acted as two eccentrically loaded columns. Under the assumption that the stress distribution is rectilinear and that half of the compressive force is transmitted to each half of the panel, the vertical tensile stress in the outer edge at failure is calculated at approximately 120 kg/cm^2 (1720 lb/in^2); consequently appreciably higher than the probable failure value under eccentric loading. This tends to show that it is also possible that the ultimate failure marked by a horizontal tension crack in bending will occur immediately after the formation of vertical cleavage. Hence in non-reinforced panels the formation of vertical cleavage carries a greater liability to failure.

In Alt. II, where transverse reinforcement was placed close to the loaded edges in the compression zone of the panel, the failure phenomenon was similar in character to the Alt. I. Post-failure view is seen in Fig. 16-b. The reinforcement had no considerable effect. Nevertheless, it held the panel together so that the load could be slightly increased after the vertical center crack appeared. Ultimate failure was caused by the development of compression failure in the portion between the vertical crack and the horizontal crack, which was longer.

In Alt. III and IV failure consisted of local crushing immediately outside the plate reinforcement, at the loaded areas. The appearance of vertical center cracks denoted a transfer of state of stresses in the panel. In Alt. III the reinforcement was concentrated in the area of greatest tension in the uncracked panel; in Alt. IV the reinforcement was distributed in proportion to the intensity of tensile stresses. No essential difference could be detected in the two alternatives: in fact failure consisted, as indicated above, of local crushing at support, with the load at the first crack somewhat greater in Alt. III.

The measured elongation confirms to a great extent the results drawn from the photoelastic test and Goodier's calculations.

The following conclusions can be drawn from the test:

1. Vertical cracks in unreinforced panels under concentrated load occur when the maximum calculated horizontal tensile stresses become greater than the tensile strength of concrete. During the investigation, control specimens were tested for compressive strength only. The compressive strength obtained for a cube, 420 kg/cm^2 (5980 lb/in^2) corresponds to a tensile strength of 20-25 kg/cm^2 (284 to 356 lb/in^2), according to results drawn from other investigations. This value is close enough to the calculated values at the first crack in Alt. I and II - 25.0 and 27.8 kg/cm^2 (356 and 396 lb/in^2).*
2. The reinforcement close to the loaded edge has no effect on the carrying capacity of the panel. The customary use of bars close to the edge has actually little effect on the carrying capacity under concentrated load.
3. The distribution of reinforcement along the vertical section of symmetry, according to insufficient experiments, has little effect on the amount of the load at the formation of the first crack. The proper place for reinforcement seems to be the area where the tensile stresses are highest. If the reinforcement is placed in a layer, it should be at a distance of $3C$ from the loaded edge with a regular spread of load (C = width of the loaded area).

*It was to be expected that the estimated tensile stresses in panels would lie higher, for the horizontal tensile stresses have an unequal distribution along the vertical section of symmetry. This carries possibilities of stress equilibrium through plastic deformation.

3. Determination of Required Area of Transverse Reinforcement

The reinforcement should be designed with due consideration of the fact that the center crack seen in all the above-described tests had actually occurred. Various possibilities are available for placing reinforcement whether it is concentrated in the area of greatest tensile stresses or distributed vertically in proportion to the intensity of tensile stresses. Here are the rules drawn for the case when reinforcement is concentrated in the area of greatest tensile stresses: The distance from the loaded edge to the center of the reinforcement is designated as b (see Fig. 17). This distance depends on the ratio between the width of the local loaded area c and the width of the panel a, as well as the ratio between the height of the panel h and its width a. With respect to the distribution of horizontal tensile stresses close to the loaded area, the panel shown [diagrammatically] in Fig. 17 is considered to be equivalent to a panel, loaded at two opposite points, of a height $2h$ and width a.*

Values for b in various ratios c/a and h/a obtained as a result of photoelastic tests described above and Goodier's computations are tabulated in Table 5.

*With some slight error, this ought to be comparable to $h/a \geq 1$

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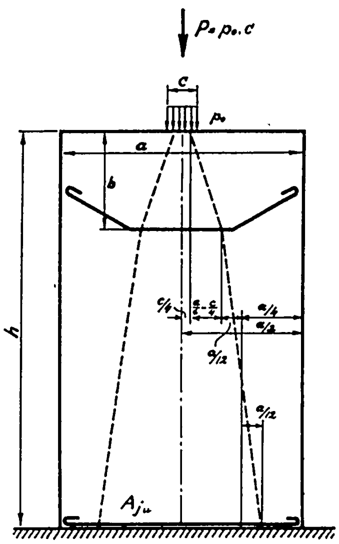

Fig. 17 - Meaning of Symbols Used in Referring to Required Transverse Reinforcement Under Single Concentrated Load.

h/a \ c/a	0		1/10		1/5	
	b	S_j	b	S_j	b	S_j
∞	0.3 a	0.28 P	0.35 a	0.20 P	0.40 a	0.15 P
2	0.3	0.23	0.35	0.15	0.40	0.10
1.5	0.3	0.21	0.35	0.13	0.40	0.05
1	0.3	0.16	0.35	0.07	0.40	0.05
<1	0.3h	0.16	0.35h	0.07	0.40h	0.05

Table 5. The Distance of the Transverse Reinforcement From the Loaded Area b and the Intensity of Tensile Stresses in Transverse Reinforcement S_j at Various Ratios c/a and h/a.

The calculation is limited to the $h/a > 1$ ratios, as the reinforcement in the lower edge to resist bending has considerable effect on the design of transverse reinforcement. The design of this reinforcement is based on the assumption that the line of compressive stresses falls within the core area in each portion of the panel formed by center cracking (see Fig. 17). With the aid of symbols in Fig. 17, the reinforcement (S_j) used to prevent cracks is proportioned by the force computed from equation:

$$S_j = \frac{\frac{a}{6} - \frac{c}{4}}{b} \cdot \frac{P}{2} - \frac{\frac{a}{6}}{h-b} \cdot \frac{P}{2} = \frac{1}{4} P \left[\frac{1}{3} \frac{a}{b} \frac{h-2b}{h-b} - \frac{c}{2b} \right] \cdots (\alpha)$$

By inserting the value for b (in accordance with Table 5) at varying ratios h/a and c/a we obtain the value for S_j drawn from Equation α and computed in the same table.

If the panel is subjected to several concentrated loads, as shown in Fig. 18, the reinforcement is determined in portions outside the outermost loads by the same conditions which prevail outside the lines of compression as for center load. Hence the reinforcement for the outermost portions is determined in Table 5 with the a equal to twice the distance from the outermost load to the free edge.

If the forces under load P are of equal intensity, then the extent of the line of compressive stresses in the center part between the vertical cracks in the extended lines of P-forces depends on the difference between the strength which has been imparted to the outermost portion through transverse reinforcement and the force which can be transmitted to the adjacent middle portion by means of transverse reinforcement (in Fig. 18 A_{j2}). Adequate safeguards against secondary transverse cracks liable to occur because the line of compressive stresses falls outside the core area in the middle portion may be achieved if this reinforcement A_{j2} (in Fig. 18) is determined by the values S_j at $h/a < 1$ presented in Table 5. In many cases it is useless to reduce the transverse reinforcement in the center portions, for the reduction of the reinforcement area may be offset by an increase of weight due to splices.

The rules for transverse reinforcement given above were based on the assumption that cracks in the concrete occurred under the concentrated loads. If the horizontal tensile stresses are not high enough to cause vertical cracks to appear, then obviously transverse reinforcement is unnecessary. It is self-evident that, as in the procedure used for shallow beams subject to shear, reinforcement should be provided only if the tensile stress exceeds the permissible value.

Greatest tensile stresses for various h/a and c/a ratios, tabulated in Table 6, are based on the results of the photoelastic tests discussed above and on Goodier's findings.

When a panel is subjected to several loads, the greatest tensile stresses under the outermost load can be taken as equal to the greatest tensile stresses in a panel under a central load, having the same height and with width equal to twice

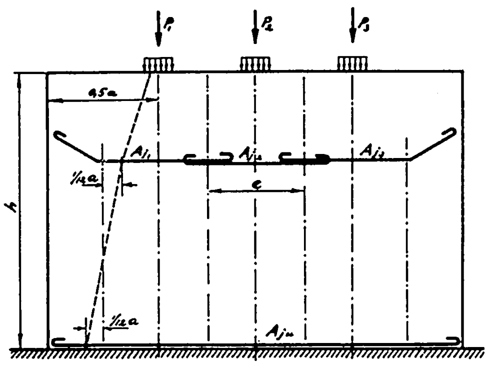

Fig. 18 - Meaning of Symbols Used in Referring to the Required Transverse Reinforcement Under Several Concentrated Loads.

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the distance between the outermost load and the vertical edge (cf. Fig. 18). The ratio between the distance to the free edge and the height of the panel has a considerable effect on the state of stresses below the outermost force.

The state of stresses below the forces nearer midspan is affected, apart from the distance between forces, also by the height of the panel; if the distance is not excessive, the tensile stresses are reduced through the effect of the nearest force.

To establish rules with due consideration to the effect of the various factors involved covering practically all conceivable cases would so complicate the problem that it would have no practical meaning.

On the other hand, should the greatest tensile stresses be instead equal to the calculated greatest tensile stresses for a beam of the same depth as the panel under discussion, and with span equal to the distance between the inside forces (e in Fig. 18), we would be on the safe side in the majority of cases. This is evident, among other things, from the development of the principal lines of stresses in the panel subjected to four opposed forces, according to Fig. 19.* Since compression can be distributed only over an area which is limited by a vertical line dividing the distance between forces in two equal parts, the possibilities for horizontal tensile stresses under load - those tensile stresses which depend on the spread of pressure under concentrated loads - can be reduced.

*The diagram is taken from an examination paper by Lennart Larsson, C. E., written under the author's direction.

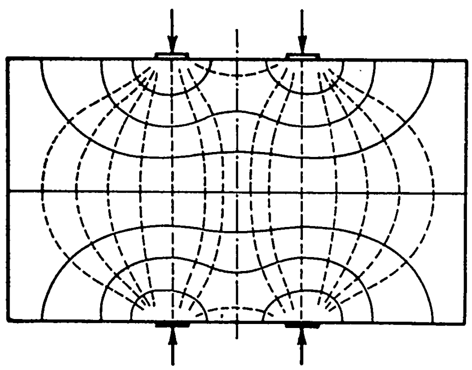

Fig. 19 - Principal Lines of Strain Under Four Opposed Concentrated Loads.

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h/a \ c/a	0	1/10	1/5
∞	$0.60 \frac{P}{da}$	$0.50 \frac{P}{da}$	$0.40 \frac{P}{da}$
2	0.65	0.55	0.40
1.5	0.65	0.60	0.45
1	0.65	0.60	0.45
0.5	0.90	0.65	0.65

Table 6. Dimension for Greatest Horizontal Tensile Stresses Under Concentrated Load. d= Thickness of the panel. For other notations see Figs. 17 and 18.*

4. Proportioning of Steel in the Lower Edge of the Panel

The design rules for transverse reinforcing shapes under concentrated loads, discussed in the preceding chapter, were determined under the assumption that the panel rests on a fixed support. From the standpoint of structural technique, this should correspond to a beam either resting directly on a rock or firm foundation, or supported by piles so closely spaced as to form a practically continuous support. In general practice, a couple of relatively heavy steel bars are placed in the lower edge of the beam. Laid out directly on a firm foundation, they are primarily designed to resist the tensile force produced by shrinkage and temperature variations. If the panel is supported on piles, these tensile forces are considerably weaker, particularly if the piles are long and slender and are driven in soft ground. If, furthermore, the panel or wall has little longitudinal extent, then the tensile stresses set up as a result of compressive deformation in the panel sections formed under load by vertical cracks become important in

*If the tensile stresses given in Table 6 exceed the permissible value τ in shear, transverse reinforcement should be provided as tabulated in Table 5.

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proportioning of reinforcement in the bottom edge. From Figs. 17 and 18, the tensile stresses in the reinforcement S_{ju} are computed as:

$$S_{ju} = \frac{1/6 \cdot a}{h - b} \cdot \frac{P}{2} \quad \text{-----------------} \quad (B)$$

where the meaning of notations a and b is given in Figs. 17 and 18 and the dimension b is tabulated in Table 5. When several concentrated loads are given, P indicates the outermost load.

B. Deep Beams

1. Introduction

While in beams of ordinary depth the design against failure in bending and the design against failure in shear can be fundamentally differentiated - in deep beams, on the other hand, no such difference exists, as for instance, between the principal tensile stresses prevailing in connection with the so called "pure" state of shearing stresses and the principal tensile stresses in bending, or the principal tensile stresses due to concentrated load. Despite the fact that the limits are diffuse, there are, however, sections where a certain character of prevailing stresses can be defined respectively as in beams of ordinary depth.

The essential difference between deep and ordinary beams lies in this: in beams of a depth equal to the span the additional stresses due to concentrated loads - as against the case in ordinary beams - over the entire beam are of importance to the design.

As mentioned in section 1, the tests hitherto conducted on deep beams of reinforced concrete indicate that the progressive crack formation is of great importance in stress distribution.

This point was taken into account in the analysis of photoelastic tests presented in Part II. The analysis made on the basis of my program by my assistant, Hans Holst, C. E., was aimed at studying the effect of a variation in the ratio $\frac{\text{Depth}}{\text{Span}} = h/l$ under a specified load, and in a large measure the influence of extreme crack formation on the development of strain.

Tests were made on two different series: on simply supported beams (Series I) with h/l = 1.0, 3/4, 1/2, 1/4, marked I:1 - I:4; and on continuous beams resting on three supports (Series II) with h/l = 1.0 and 3/4 marked II:1 - II:2 (see Fig. 1 in Part II). With a view to elucidating the effect of the above-mentioned crack formations which might probably take place (under ordinary or slightly higher loading), both series were investigated after the tests on homogeneous beams, and the zone where the greatest tensile stresses in bending were observed in the homogeneous beams, was cut out and replaced by bending reinforcement. The member thus obtained can be characterized as an arch or a curved beam with a tie rod. The effect is real if it is assummed that the tensile force in the reinforcing bars is transmitted entirely to the concrete by means of end anchorage. While the anchoring of tensile reinforcement is not secured in this way but rather by friction and bond, it is still true that at least in the later stage [of a test] the actual condition may lie between those tested in the two limiting cases: the homogeneous panel and the panel with cut-out tension zone.

The load in both Series I and Series II consisted of two equal loads applied at the third points of span. This loading method was selected because it gives conditions intermediate between the two extreme cases of loading: uniformly distributed and midsp a load.

The summary of photoelastic tests will be presented in the form of lines of principal stress, contours of principal tensile and compressive stresses, as well as the horizontal stresses in a certain characteristic section* of selected beams.

2. Simply Supported Beams

The previously discussed test made by Klingroth indicates that the load-bearing capacity was not dependent on the fact that the principal tensile stresses computed on the basis of the elastic ratio exceeded the tensile strength of concrete (15**). Due to the formation of tensile cracks in concrete, a shift of the distribution of internal stresses has taken place, and therefore the member now acts as a deforming trestlework provided with tension rods. Safeguards should be provided against detrimental crack formation by means of reinforcement in the regions of highest tensile stresses. Disregarding those portions where the tensile stresses have the character of bending tension, Table 7 gives the greatest principal tensile stresses for various span ratios, based on the results obtained from the photoelastic tests. Because it is difficult to indicate the exact limit between bending-tensile stresses and other principal tensile stresses, the table gives the approximate limit values.

As may be seen from Fig. 20, where the contour lines of principal tensile stresses are reproduced, the crosshatched portions represent the critical range where the dangerous tensile stresses are not influenced by the bending reinforcement.

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* Concerning methods used in the photoelastic tests see footnote on the article mentioned in Chapter 3, Section 1.

**[See Bibliography]

l/h	Homogeneous Unreif. Beam Stage I	Stage II Reinforced	σ_1 dim
1	$0.8 - 1.1 \frac{P}{dh}$	$0.8 - 1.1 \frac{P}{dh}$	$0.9 \frac{P}{dh}$
4/3	0.85 - 1.15	0.9 - 1.2	1.0
2	0.9 - 1.2	1.2 - 1.6	1.2
4	1.0 - 1.5	1.5 - 2.0	1.5

Table 7. The Values of the Maximum Principal Tensile Stresses Computed on the Basis of the Photo-elastic Test in Series I, Having no Character of Bending Stresses.

d = thickness of the beam;
h = total depth;
l = span.

Two concentrations of value P applied to the upper edge at the third points of the span made up the load.

In the last column of Table 7 the determining values for the principal tensile stresses are suggested. When these values exceed the permissible stresses, the members should be provided with shear reinforcement. Since the load at the first crack lies considerably below the ultimate load, the permissible tensile stresses are relatively high. If the safety factor is placed, for example, 50% lower than the safety factor against failure in shear in beams with high span-depth ratio (> 6), the values in the last column in Table 7 could be compared with 1.5 of the allowable shearing stresses in accordance with the specification for the concrete.

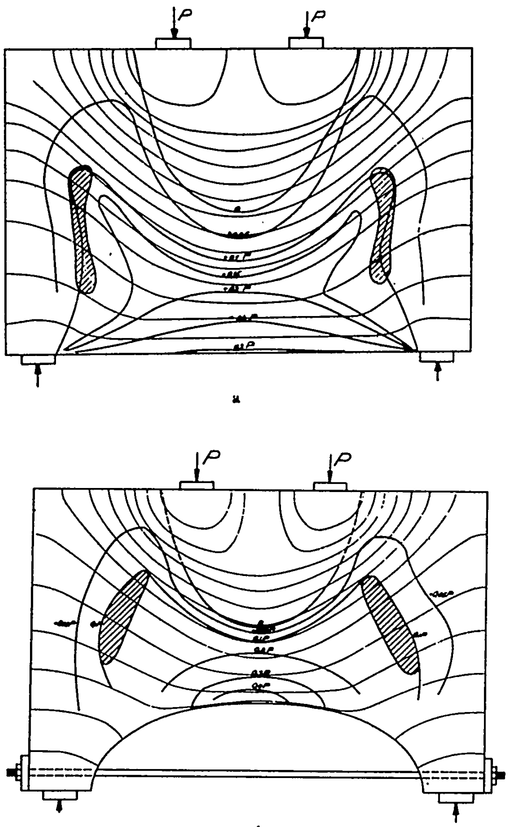

Fig. 20 - Distribution of the Principal Tensile Stresses Drawn from the Photoelastic Test. Numbers show this distribution at $l/h = 4/3$

Fig. 20-a shows "Stage I" and Fig. 20-b "Stage II". For other lateral ratios the distribution is the same. The results are omitted for lack of space. The intensity of principal tensile stresses is expressed in terms of P. The thickness of specimen is 1 cm. (0.39 in.). The more gently curving lines show the principal tensile-strain trajectories.

Shear Reinforcement

The author who treated the present problem up to now has suggested as a rule, that shear reinforcement should be proportioned in line with the stress distribution in a homogeneous uncracked beam. It is obvious that such a procedure has been based on the assumption that the stress distribution was relatively easy to calculate and that reinforcement designed for the uncracked stage would best serve to prevent detrimental cracks. The latter assumption, however, is questionable. A definite answer could be achieved only through comprehensive investigations.

Following are the proportioning rules obtained for shear reinforcement, based on the internal stress distribution after bending and shear cracks have occurred.

The probable location of shear cracks can be determined from contours of principal tensile stresses and from lines of principal strain. The design of the reinforcement is based on the requirements that this reinforcement should be proportioned by the force necessary to keep in equilibrium that portion of the beam which is separated by a probable shearing crack. The procedure used in designing shear reinforcement is illustrated in Fig. 21*. Assuming that the chief purpose of reinforcement is to prevent the formation of harmful cracks (while noting that the proportioning suggested here is based on the probability that the reinforcement is sufficient if it achieves an equilibrium of forces acting on the separated portions like that which would exist if no shear crack had occurred), then a high steel stress can be permitted.

As the shift of the internal stress distribution resulting from a shear crack is least in beams with a relatively low span ratio l/h (judging from the course of the principal lines of stress), the size of cracks must consequently be smaller

*The outside forces acting on the separated portions were derived from the results obtained from the photoelastic test.

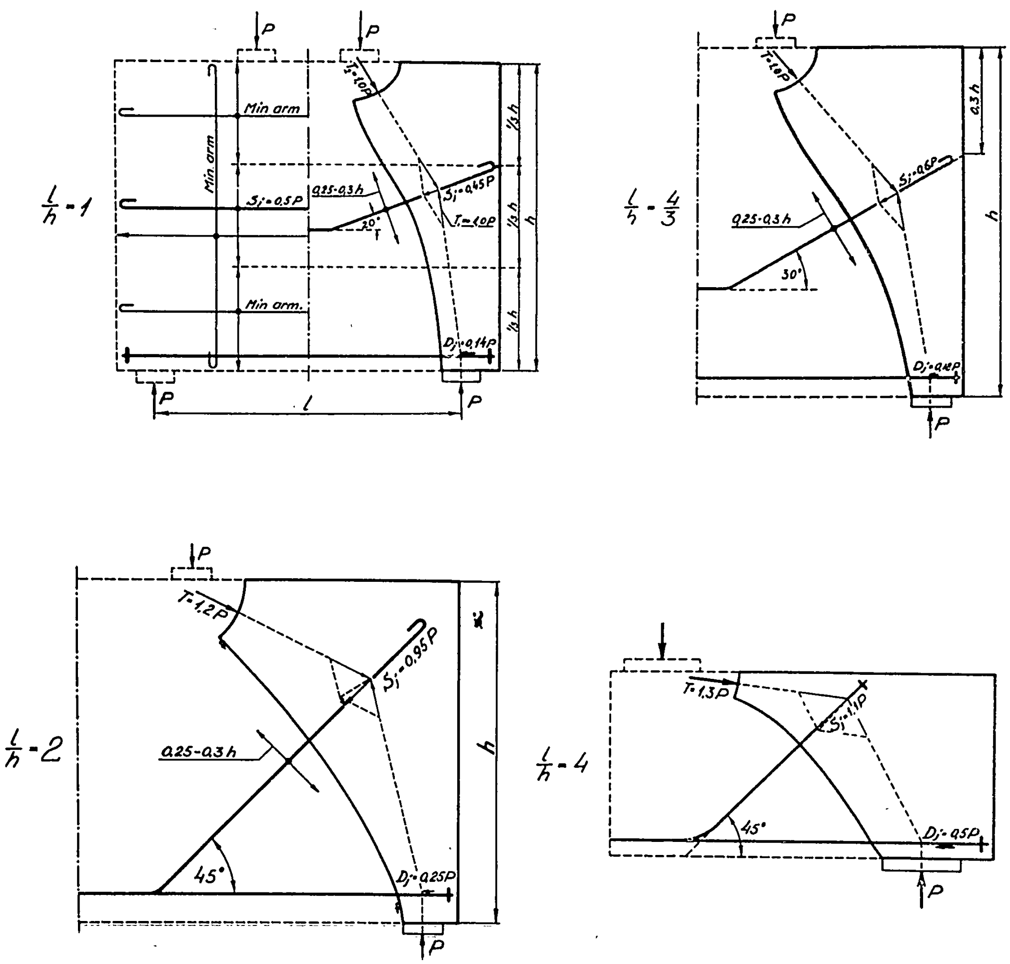

Fig. 21 - Computed Shear Reinforcement and its Distribution

Load: Applied at the third points of span. The location of the shear crack was determined on the basis of the photoelastic test. The tensile force acting on the shear reinforcement was obtained under conditions of equilibrium in that portion of the beam which is separated by a crack in the concrete. The light [broken] lines depend on the uncertainty of the value of tensile forces acting on the bending reinforcement determined on the basis of the photoelastic test. However, a certain adjustment is possible. It is assumed that for example $l/h = 1\ D_j < 0.14\ P$ For loads exceeding the value that produces a yield point stress in shear reinforcement, then $S_j = 0$ and $D_j > 0.14\ P$

Failure occurs when bending reinforcement is stressed to failure.

than in a higher span ratio, and thus a higher permissible steel stress may be assumed for the small span ratio.

The suggested permissible values for steel stresses at various span ratios are given in Table 8. In view of the fact that no direct concrete test has been made, the values indicated in the table should be used cautiously.

l/h	$\sigma_{j\,hll}$	S_j	A_j (Required Area)
1	1.0 σ_{js}	0.45 P	0.45 $\frac{P}{\sigma_{js}}$
1.5	0.9	0.70	0.80
2	0.8	0.95	1.20
3	0.7	1.05	1.50
4	0.55	1.1	2.00

Table 8. Permissible Values for Steel Stresses, Design of Forces Acting in the Shear Reinforcement (Crack Reinforcement), and the Requisite Area of Reinforcement for Load Conditions Indicated in Fig. 21.

σ_{js} = stresses at the yield point of shear reinforcement

[$\sigma_{j\,hll}$ = allowable unit stress in steel]

Modifications for Loading Conditions Other Than Those Already Treated

Results obtained from the loading conditions already discussed can be applied even in beams with greater span-depth ratios if the load is applied close to the supports (see Fig. 22).

The state of stresses in an area between the load forces and the support A depends on:

1. the moment;
2. the shearing force;
3. the local reaction at support;
4. the local forces resisting the load.

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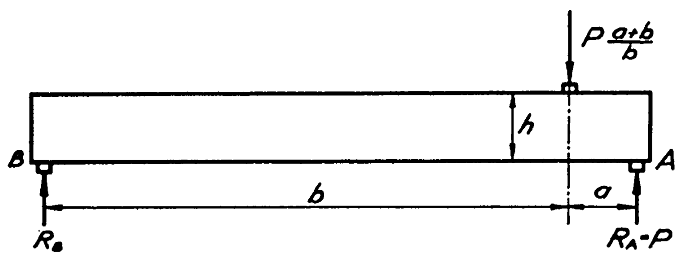

Fig. 22 - Meaning of Symbols Referred to in the Design Rules for a Beam Subjected to Load Close to Support.

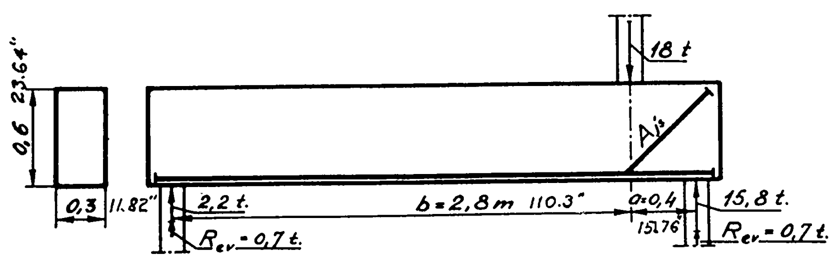

Fig. 23 - Example

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If the allowable principal tensile stresses in concrete and the required area of reinforcement are expressed in terms of reaction R_A which is designated by P, it follows that the loading case shown in Fig. 22 deviates from the loading case in Fig. 21 only in regard to local forces resisting the load.

As is evident from the photoelastic investigations, this deviation, however, is relatively insignificant for a greater b/a ratio.

In the loading case shown in Fig. 21 there is also a certain reaction of the farthest loading force that can be neglected, since the local reaction falls off rapidly with increasing distance from the active forces.*

Consequently, at a sufficiently high b/a ratio the formulas and rules given above for the intensity of principal tensile stresses and for shear reinforcement can be applied even in a case when the load is applied at a single point. The lower limit value can be set at $b/a \gtrsim 5$. The tables for two points of load application are valid if P in Fig. 22 denotes reaction at support A and h the total beam depth. The value l/h with the loading at the third point corresponds here to the value $3a/h$.

<u>Example</u>. Determine the required shear reinforcement for the beam given in Fig. 23.

τ_{till} shear permissible according to concrete specifications = 6 kg/cm^2 (85.3 lb/in^2)

σ_1 weight of specimen $= 1.15 \frac{7.00}{30 \cdot 57} = 0.5$ kg/cm^2 (7.1 lb/in^2)

$$\frac{3a}{h} = \frac{3 \cdot 0.4}{0.6} = 2$$

*See The influence lines of local reaction indicated by F. Seewald: "Abhandlungen Aerodyn. Instit. Technishce Hochschule," Aachen, Vol. 7, p. 11, 1927.

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From Table 7 is obtained $l/h = 2$

$$\sigma_{(dim} = 1.2\,\frac{15800}{30 \cdot 60} = 10.5 \text{ kg/cm}^2 \text{ (149.1 lb/in}^2\text{)}$$

$\Sigma \sigma_1 = 10.5 + 0.5 > 1.56$ for which shear reinforcement is to be provided.

From Table 8, established area of shear reinforcement for $l/h = 2$

$$A_{js} = 1.2\,\frac{15800}{2200} = 8.6 \text{ cm}^2 \text{ (1.4 in}^2\text{)}.$$

3. Continuous Beams

As the situation remains largely the same in continuous beams as in simply supported ones, the rules drawn from the (preceding) Section 2 regarding permissible reactions at supports and the shear reinforcement ought to be applicable here.

The situation between supports differs considerably from that in the simply supported beam. Besides the fact that the oblique principal tensile stresses change imperceptibly to horizontal bending-tensile stress above the supports, there are portions of the beam separated by cracks (which could be termed extended bending cracks due to shearing cracks) in a state of equilibrium, with the special reinforcement not necessarily producing an effect. This is under the assumption that the span and the loadings are equal in different areas. When failure occurs, the system can be characterized as a continuous arch where the condition of equilibrium of all the portions separated by the cracks is fulfilled without the introduction of an additional force. This point is amplified by the results presented in Figs. 24 and 25, which had been obtained from the photoelastic investigations of a continuous beam resting on three supports. Furthermore, Fig. 25 clarifies the point that the crack first to occur brings about a relief of the area close to the support, with the result that the center of tension in bending above the supports is transferred upward. The shear reinforcement at the support ought

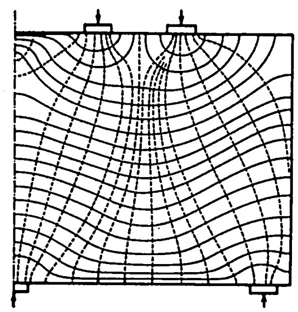

Fig. 24-a - Trajectories of Principal Stress in Stage I of Photoelastic Test. Beam resting on three supports. $l/h = 1$.

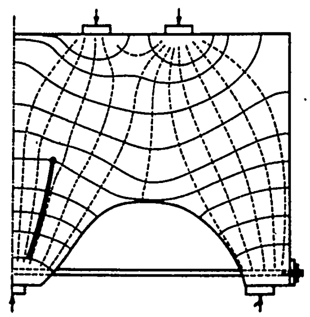

Fig. 24-b - Trajectories of Principal Stress in Stage II of Photoelastic Test. Beam resting on three supports. $l/h = 1$.

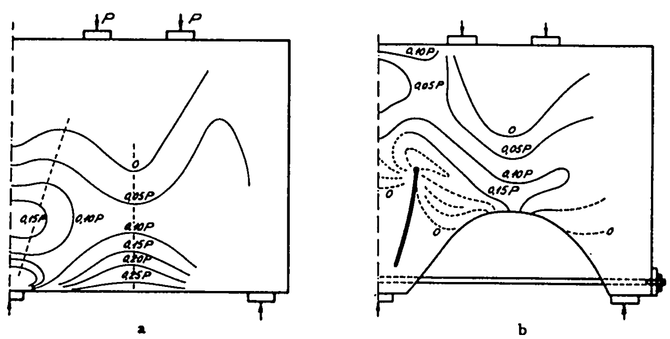

Fig. 25 - Contours of Principal Tensile Stress Distribution in a Beam on Three Supports, Obtained From the Photoelastic Test; $l/h = 1$.

Fig. 24-a shows Stage I; Fig. 24-b - Stage II. Intensity of principal tensile stresses is expressed in terms of P. Thickness of specimen - 1 cm. (0.39 in.)

to be used simultaneously as bending reinforcement, as shown in the photoelastic reproduction. As the bending reinforcement, according to the existing calculation methods, is indispensable, it follows that a certain shear reinforcement also will always be necessary.

To establish the requisite amount of shear reinforcement it is first necessary to elucidate its function. Thus it can be deduced on the basis of the foregoing statement that it [the reinforcement] serves to distribute cracks. As the first cracks bring about a radical shift in stress distribution, the solution of the problem can be achieved by studying direct tests on concrete. The test described in Chapter 4 presents the needed data. On the basis of this test, no extra shear reinforcement is required above the supports, if the entire reinforcement of the beam is designed in accordance with the recommendations given in Fig. 36. This assumption holds, however, with the prerequisite mentioned earlier that the different span areas and loadings are equal.

4. BENDING REINFORCEMENT

A. Introduction

The design of bending reinforcement has hitherto generally been based on stress distribution in a homogeneous beam and on a tensile force in the reinforcement equal to the total tensile force in the homogeneous beam.*

This method of approach obviously deviates from the method used in proportioning shallow beams, where the tensile force in the reinforcement is determined by the stress distribution prevailing at the cracking of concrete.

*With the exception of the test shown in item 13 [of the Bibliography], where the analysis of the effect of crack formation refers to simply supported beams ar for the test in item 15, where a direct attempt was made to establish the tensile forces in steel in certain loading cases of simply supported beams.

The reason for not taking into consideration the cracking of concrete in deep beams is to be found in the imperfect knowledge of the effect of cracking. As far as is known to the author, no direct tests reaching the yield point have ever been made.

With a view to elucidating the behavior of members which are loaded so heavily that crack formation exerts an influence on the stress distribution, the author has made a series of tests. In order to throw light on the interaction of the moments at the supports and between supports, so important to the problem of proportioning, the tests were made on continuous beams resting on three supports.

B. The Test

The shape and reinforcement of the test beams are shown in Fig. 26-a and 26-b. The reinforcement area in beam #2 was designed on the basis of stress distribution prevailing in the homogeneous uncracked beam in the photoelastic test (beam II:1-a); for instance, the reinforcement above the support was concentrated in the area where tension prevailed. The reinforcement in beam #1 was designed on the basis of a probable stress distribution after the cracks have occurred. It was assumed that the reinforcement at the center support placed at the upper edge of the beam, on account of the progressive crack formation, should be effective against the moment at the supports. To eliminate possibilities of local compression failure, local reinforcement was placed in the areas subjected to load and at the supports, as shown in Fig. 27. Then shear reinforcement was introduced consisting of 4 ∅ 8 mm. (0.32 in.). The form of reinforcement mounted for casting is shown in Fig. 28. The proportioning of concrete aggreagates in dry weight was as follows: standard cement: sand: gravel (1 : 3.7 : 4.7). Water-cement ratio 0.75 (8 1/2 gal./sack).

Six compression cubes (each side 15 cm = 6 in) were made for each beam. Also for each series of two beams a control specimen was prepared consisting of:

6 compression cubes,
4 tension prisms (with a cross-section 15 x 15 cm^2 = 6 x 6 in; free tensile length 64 cm (25 in),
3 beams subject to bending (with cross-section 15 x 15 cm^2 = 6 x 6 in)

The summary of results obtained from the control specimens is tabulated in Tables 9 and 10.

Tested Specimen for	Strength of Cube		Tested Specimen for	Strength of Cube	
	kg/cm^2	lb/in^2		kg/cm^2	lb/in^2
Beam 1 a	295 289 278 275 267 266 Av. 278	4200 4100 3960 3920 3800 3780 3960	Beam 2 a	305 300 294 254 276 252 Av. 280	4350 4260 4190 3620 3930 3580 3980
Beam 1 b	280 260 282 282 270 276 Av. 275	3980 3700 4000 4000 3840 3930 3920	Beam 2 b	273 271 262 262 248 267 Av. 264	3880 3860 3730 3730 3530 3800 3750

Table 9. Strength of Cube of Concrete Used in Test Beams

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Control Specimen for	Strength of Cube		Tensile Strength		Tensile Strength (in Bending)	
	kg/cm^2	lb/in^2	kg/cm^2	lb/in^2	kg/cm^2	lb/in^2
Beam # 1	291	4150	18.4	260	26.0	370
	292	4160	17.2	244	26.5	378
	304	4340	16.3	230	31.0	442
	270	3840	17.8	248	Av. 27.8	396
	276	3930	Av. 17.2	244		
	278	3960				
	Av. 285	4050				
Beam # 2	257	3660	18.0	257	26.8	382
	265	3780	16.6	236	31.3	445
	276	3930	16.4	231	28.2	402
	281	4000	Av. 17.0	242	Av. 28.8	410
	267	3800				
	276	3930				
	Av. 270	3850				

Table 10. Strength of Cube, Tensile Strength, and Tensile Strength in Bending. Bending tensile strength is calculated on the basis of rectilinear stress distribution.

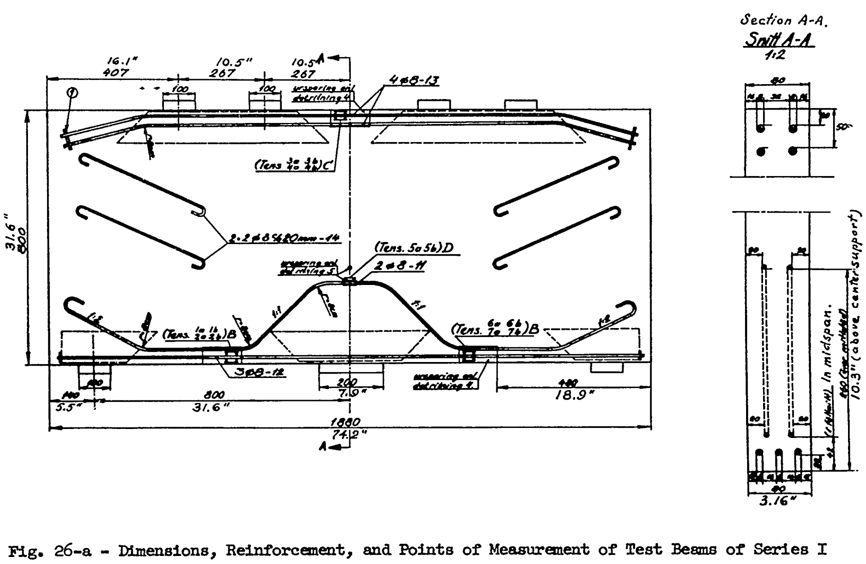

Fig. 26-a - Dimensions, Reinforcement, and Points of Measurement of Test Beams of Series I

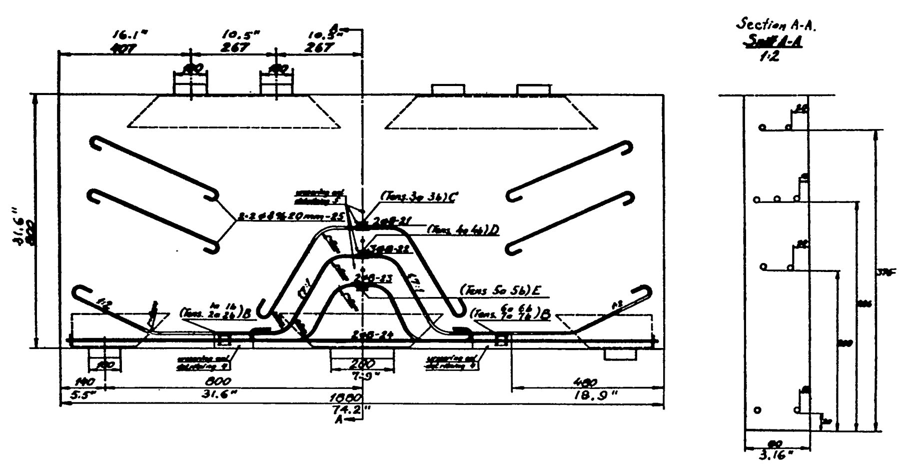

Fig. 26-b - Dimensions, Reinforcement, and Points of Measurement of Test Beams of Series II.

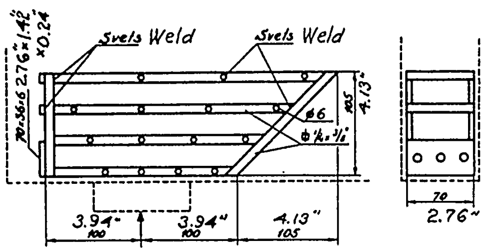

Fig. 27-a - Local Reinforcement at End Support

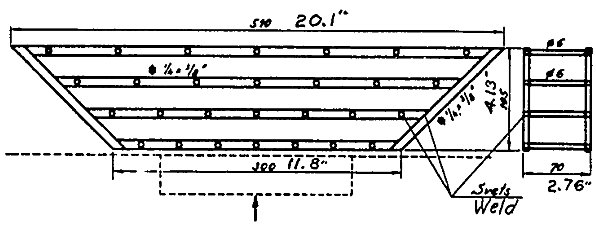

Fig. 27-b - Local Reinforcement at Center Support

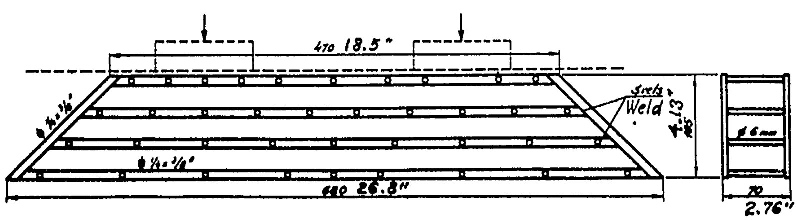

Fig. 27-c - Local Reinforcement Below Loading Forces

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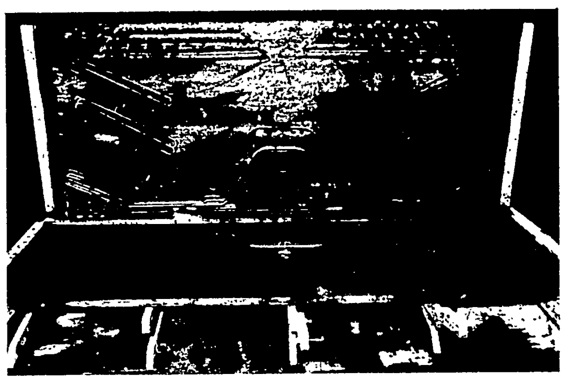

Fig. 28 - Form with Mounted Reinforcement Before Pouring

A control specimen was also taken from each reinforcing bar. Data on the strength of the reinforcement used are given in Table 11.

Beam	Ref. to Fig. 26	Steel Diam.		Yield Point (Lower)		Ultimate Strength		Ult. Elong.	E	
		mm	in	kg/cm^2	lb/in^2	kg/cm^2	lb/in^2		kg/cm^2 10^6 x	lb/in^2 10^6 x
1-a	11	7.9	0.31	2740	39000	3530	50300	Average 28 percent	2.15	30.6
"	12	7.9	0.31	2940	41900	3950	56200		2.21	31.5
"	13	7.9	0.31	2910	41500	4010	58400		2.19	31.2
"	14	7.9	0.31	3080	43900	3800	54100		2.24	31.9
1-b	11	7.8	0.30	2970	42200	4060	58000		2.15	30.6
"	12	7.8	0.30	2920	41500	3990	56600		2.17	30.9
"	13	8.0	0.31	3020	43000	3780	53700		2.14	30.4
"	14	7.9	0.31	3080	43900	3800	54100		2.24	31.9
2-a	21	7.8	0.30	3040	43200	4040	57400		2.25	32.0
"	22	7.9	0.31	3140	44600	3920	55700		2.28	32.4
"	23	8.0	0.31	2700	38400	3740	53200		2.11	30.0
"	24	7.9	0.31	2750	39200	3860	55000		2.19	31.1
"	25	7.9	0.31	3080	43900	3800	54100		2.24	31.9
2-b	21	7.8	0.30	3040	43200	4040	57400		2.25	32.0
"	22	7.9	0.31	3020	43000	4020	57200		2.25	32.0
"	23	8.0	0.31	2700	38400	3740	53200		2.11	30.0
"	24	7.9	0.31	2750	39200	3860	55000		2.19	31.1
"	25	7.9	0.31	3080	43900	3800	54100		2.24	31.9

Table 11. Data on the Strength of Reinforcement.

The steel strain in each test beam was measured by means of tensometers located as shown in Fig. 26. The center deflection in relation to the end supports was measured by means of a "Zeiss Clock" [dial gage]. Since the jack was located below the center support, it was possible to calibrate the height of the support and to determine the center reaction. The photo in Fig. 29 gives a view of the test set-up. The jacks were calibrated for the same loading method as in the experiment. In calibrating, internal friction was taken into consideration when evaluating the results of the experiment.

Data on the load at the first crack to appear, the ultimate load, and failure are tabulated in Table 12.

Beam No.	Load at the First Visible Crack m.t.	Load at Failure m.t.	Failure Phenomena
1-a	75	135.0	Scaling at the center support. Widening cracks
1-b	75	123.5	Ditto
2-a	75	121.5	Widening Cracks
2-b	50	106.5	Scaling at the center support. Widening cracks

Table 12. The Load at the First Visible Crack and at Failure.

The results obtained from deformation measurements are shown in Fig. 30. The relationship between the total load and the center reactions, and between the total load and the deflection at center support relative to the end supports are given in Fig. 30-a; the measurements of steel stress for different beams in Fig. 30-b (points of measurement as in Fig. 26). Photos illustrating the crack formation in test beams are to be found in Fig. 31.

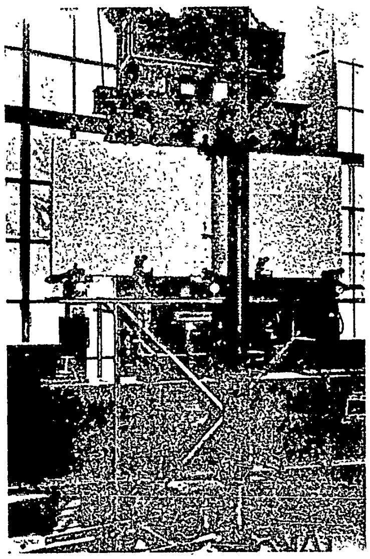

Fig. 29 - Testing and Measuring Setup

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From the results presented in Figs. 30 and 31 the following conclusions can be drawn regarding the behavior of the beams tested.

1. The center reaction in beams #1-a and #1-b due to the load at the first crack formation amounted to between 54 and 62 percent of the total load when the center support was not shifted vertically in relation to the end supports (the load of 25 m.t. was disregarded because the reading of the load value was uncertain). No appreciable change in the relationship between the center reaction and the total load was noticed when the load was increased.

In beams #2-a and #2-b, the reading was uncertain at small load values. At loads exceeding 75 m.t. the center reaction constituted 50 to 52 percent of the total load when the upward shifting of the center support was zero.

From the comparison of beams #1 and #2 it is evident that beam #1 has a greater rigidity in bending in its center portion. This probably could be explained by the fact that beam #1 is provided with reinforcement in its upper edge.

2. The steel stress diagram shows that, after the cracks began to appear in the concrete beam, the reinforcement at the supports and between supports was on the whole subjected to equal stresses, irrespective of whether the reinforcement was designed on the basis of the stress distribution prevailing in a homogeneous and fully elastic beam, as in beams #2, or on the basis of the probable stress distribution in a cracked beam, as in beams #1.

Deviations observed in both types of beams could be explained by the different reactions in the center portion which, by causing cracks to appear, resulted also in a different stress distribution. This, for instance, could explain the high reinforcement stresses observed in beam #2-a even at a load of 75 m.t., when the center reactions had first increased, then considerably decreased. As a result wide cracks appeared above the center support, causing a lowered rigidity above the

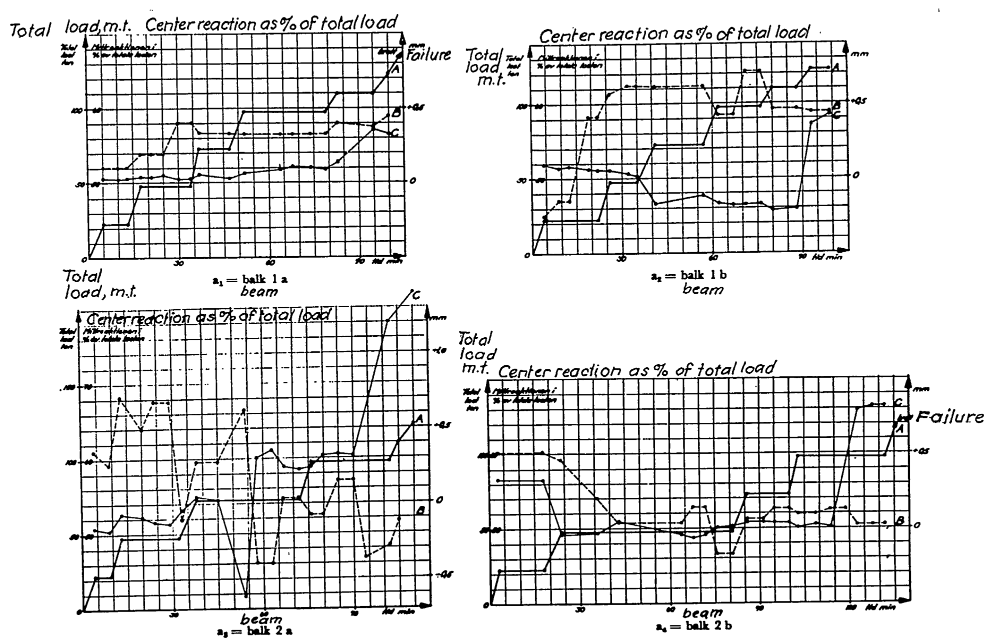

Fig. 30-a - Total Load, Center Reaction as Percentage of the Total Load, and the Vertical Displacement at the Center Support Observed During the Test. The abscissa indicates time elapsed from the start of the test.

Notation: A = Total Load; B = Center Reaction as % of Total Load;
C = Vertical Displacement of the Center Support (+ when the displacement is downward).

The connection between the reaction at the center support or the vertical displacement of that support and the total load is to be found from the diagram by drawing a vertical line from the load curve A to [intersect] the reaction and displacement curves B and C.

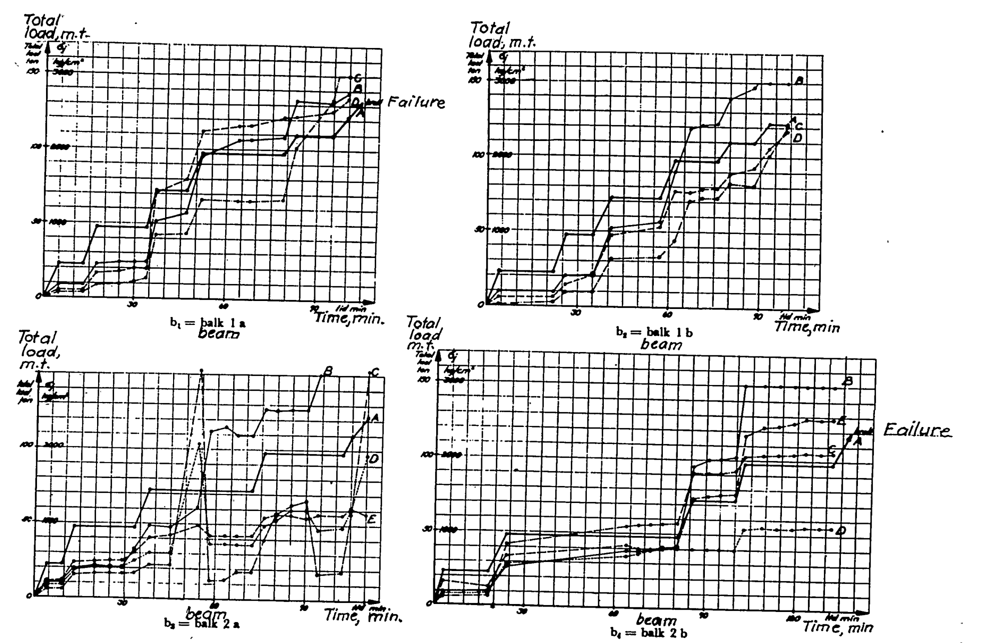

Fig. 30-b - Total Load and Steel Stresses Observed During the Test. Abscissa indicates time elapsed from start of the Test.

Notation in Fig. b_1 and b_2:

A = Total Load; B = Steel Stresses in Midspan (Tensometers 1, 2, 6, & 7); C = Steel Stresses in the Upper-edge Bar Above Support (Tens. 3 & 4); D = Steel Stresses in Bent-up Bars Above Support (Tens. 5) {see Fig. 26-a}

Notation in Fig. b_3 and b_4:

A = Total Load; B = Steel Stresses in Midspan (Tens. 1, 2, 6, & 7); C = Steel Stresses in Upper Bars Above Support (Tens. 3); D = Steel Stresses in Center Bars Above Support (Tens. 4); E = Steel Stresses in the Bottom Bar Above Support (Tens. 5). {see Fig. 26-b}

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support, which in turn, when subjected again to an increased load, caused the span to absorb the greater part of the bending moment.

As the lever arm of the tensile force acting on the steel (above the support) in beam #1 was more than twice that in beam #2, it is evident that the reinforcement used in the beam #1 is preferable from the point of view of load-bearing capacity.

3. In beams #1, where the reinforcement was provided for observation of the moment probably present at failure, the widths of cracks were measured throughout the test. When subjected to load equal to 60% of the ultimate load, the widths of cracks, as found in Fig. 31-a, did not exceed 0.1 mm. (0.004 in) at any point. In beams #2 no such measurements were taken. The cracks were visible at the same or at slightly lower loads than in beams #1. It can thus be concluded that from point of view of crack formation it is permissible to provide the beam with reinforcement mainly at the upper edge if, as in the case of the test, the crack-distributing reinforcement is provided in the portions of the beam where the greatest tensile stresses are present in the uncracked homogeneous beam. The reinforcement used during the test consisted of half of the reinforcement in the upper edge of the beam above the support.

4. In connection with the matter discussed in the preceding paragraph (3) concerning crack formations, the question remains open as to the design importance of the diagonal tensile stresses above the center support. The photographs in Fig. 31 show that the cracks formed as a result of these principal tensile stresses run from center support toward the nearest acting force. It is obvious that such cracks have no effect on the bearing capacity of the beam. As the cracks had no alarming width, as already mentioned earlier (in paragraph 3), there is evidently no need, in a beam ratio such as this $\frac{\text{span}}{\text{depth}} = 1$, to consider the diagonal tensile

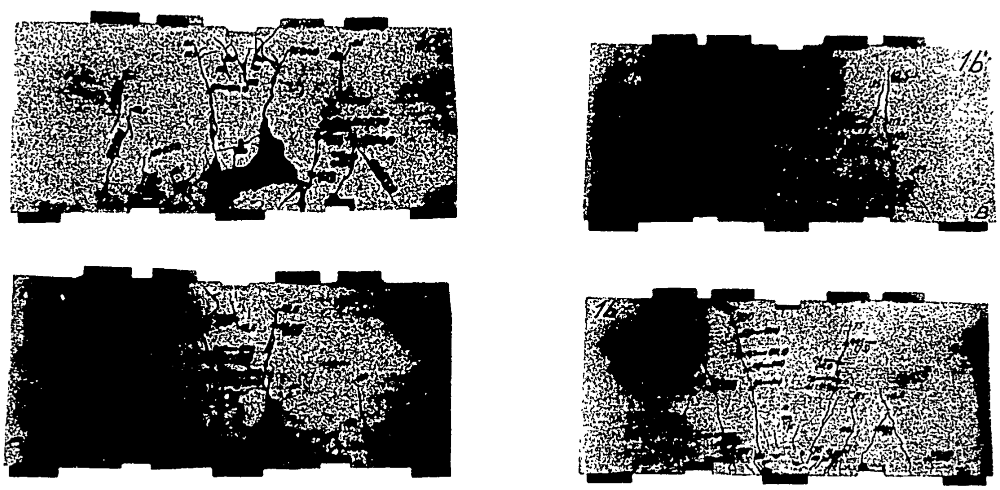

Fig. 31-a - Crack Formation in Beams #1

Load values indicated for transverse lines represent the extent of cracks at loads as in the reading, while the actual load was equal to the reading load reduced by 3.05 m.t., which is the weight of the loading device. In places where the widths of cracks are given, the values for loads are given in parentheses.

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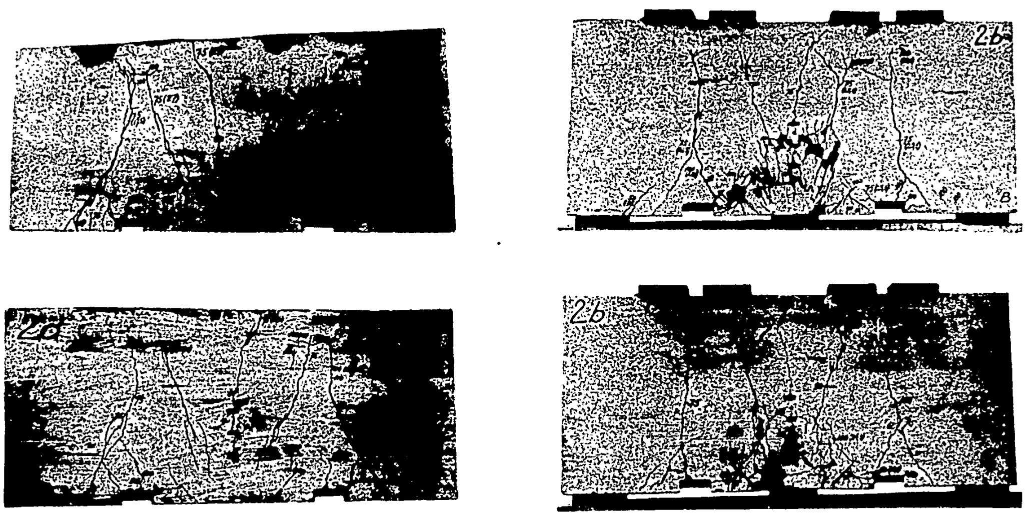

Fig. 31-b - Crack Formation in Beams #2

Load values indicated for transverse lines represent the extent of cracks at loads as in the reading, while the actual load was equal to the reading load reduced by 3.05 m.t., which is the weight of the loading device. The values in parentheses represent the center reaction.

stresses above the center support, if some crack distributing reinforcement is provided.

C. Lever Arm of Internal Moments Under Various Support and Loading Conditions

As no systematic test has been made on beams for various methods of support, loading condition, and span ratio, one had to fall back on the rather rough evaluation based on the available results drawn from tests. Klingroth's test (15*) gives data on the lever arm [of internal moments] in a simply supported beam with a depth = span and loading conditions shown in Fig. 3, namely: 1) load at center, 2) load at third points, and 3) uniformly distributed load. The lever arms for these cases as obtained by Klingroth are:

1) $h/1.04$ 2) $h/1.37$ 3) $h/1.61$.

These values, however, presuppose that the bending reinforcement is distributed over an area equal to 1/4 of the beam depth, which seems to be unnecessary. If it is assumed instead that the center of tension is located at $0.05h$ from the lower edge, then the lever arm values, according to Klingroth, are as follows:

1) $h/0.97$ 2) $h/1.25$ 3) $h/1.45$.

Thus it could be deduced on the basis of the high values at the load points that the concrete absorbs tension. As this capacity depends on the concrete mix, it seems a warranted precaution to establish the lever arm at $h/1.05$.

These values for the lever arm e [of internal moments] deduced from the photoelastic test for homogeneous unreinforced beams are tabulated in Table 13.

*[See Bibliography]

l/h	ϱ
4	$h/1.50$
2	$h/1.54$
4/3	$h/1.53$
1	$h/1.51$

Table 13. Values for Lever Arm of Internal Moments ϱ Obtained from the Photoelastic Investigations. Simply supported homogeneous beam; loads applied at third points.

It is clear from the table that the lever arm in a homogeneous beam varies slightly with variation of the span-to-depth ratio. For loading cases other than the one examined the ratio ought to be similar. Therefore it can be concluded that the variation in stress distribution in varying span ratio should have no important bearing on the ϱ value. On the other hand, the cracking of the concrete in the tension zone has a different effect at various span ratios. In regard to the fact that the lever arm increases with a progressive crack formation, there cannot be a serious error in assuming the lever arm at the span ratio 1:3 equal to the value indicated for an ordinary beam, and for ratios between 1:1 and 1:3, that the value is to be found by straight-line interpolation. The values thus obtained are to be found in Fig. 32.

The above-described test on concrete beams resting on three supports shows that the internal lever arm is largely dependent on crack formation. In all the beams cracks appeared at midspan higher than in the case of Klingroth's test on simply supported beams subjected to loads at the third points. Consequently, in continuous beams the lever arm in the span can be assumed equal to that in a simply supported beam and use made of the values shown in Fig. 32.

1. Uniformly Distributed Load

2. Load Applied at Third Points

3. Load Applied in the Center

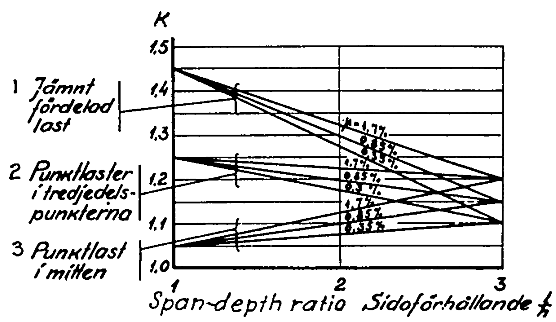

Fig. 32. Lever Arm ϱ of Internal Moments for Various Conditions of Loading, Span-Depth Ratios l/h [and Percentage of Reinforcement μ].

$l/h \; \varrho = h/K$

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The proportioning of the reinforcement above the support (as in the foregoing test) with $l/h = 1$ was designed under the assumption that the center of compression is located at approximately $h/20$ to $h/25$ from the lower edge of the beam and that the reinforcement at supports consists of two parts, $2/3\,A_j$ and $1/3\,A_j$, with $2/3\,A_j$ located at the upper edge of the beam and $1/3\,A_j$ at a distance of $2.5c$ from the lower edge. The value of the lever arm thus obtained corresponds to Klingroth's value for a simply supported beam with load applied at midspan. As to the other value of l/h, the value e found in Fig. 32 can be used in accordance with rules given in Fig. 36.

D. Moment Values at Various Support and Loading Conditions

The above-described test on reinforced concrete beams supported on three points shows that the relation between the moments at supports and at midspan is largely dependent on the manner in which the reinforcement is placed. If the reinforcement at the supports is mainly near the upper edge (as was recommended in the preceding section), then the moment at supports in a beam supported at three points is considerably greater than if the reinforcement were provided on the basis of strain distribution in a homogeneous uncracked beam. Consequently all too rigid formulas for moments are out of place, especially in regard to the design of deep beams. As the moment is highly susceptible to any displacement of points of supports, this behavior should be reflected in the design.

On account of the extraordinary difficulties involved in calculating the moments acting in continuous beams with unequal spans - even in beams of ordinary depth - the following moment values are presented for general and frequently encountered cases fundamental in the analysis of support conditions.

a. Continuous Beams on Multiple Supports with Equal Repeated Spans

In cases where the loading points are applied at midspan, the moments at supports and at midspan are equal to $Pl/8$, as clearly seen from Fig. 33. These moments are independent of the span-depth ratio l/h.

In cases of uniformly distributed loads, shown in Fig. 34, Dischinger's calculations of the moment above the supports and at midspan (9*) were based on the assumption derived from the theory of elasticity. The summary of results drawn from Dischinger's calculations is presented in Table 14.

h/l	Moment at Support	Moment at Midspan
1	$0.071\,ql^2$	$0.041\,ql^2$
3/4	0.072	0.041
1/2	0.073	0.041

Table 14. Moment at Support and Midspan Moment in Continuous Beams on Multiple Supports and Subjected to Uniformly Distributed Load [q kg/m or lb/ft] ($c/l = 1/10$), According to Dischinger.
The deviation of the sum of moment values at supports and in midspan from $ql^2/8$ depends on the spread of the support.

Consequently, even in this loading case the moment is practically independent of the ratio h/l. Therefore there is reason to assume that the distribution of the moment under other loading conditions for continuous beams resting on multiple supports with a uniform load and with equal repeated spans will also be independent of the ratio h/l.

b. Beam Resting on Three Supports with Spans of Equal Value

The loading case examined in the test is shown in Fig. 35: the load was applied at the third points; the moment at support is approximately 80 to 90 percent

*[See Bibliography]

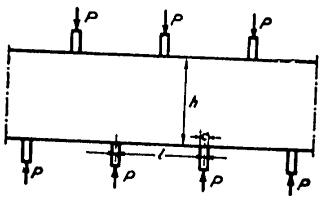

Fig. 33 - Continuous Beam Resting on Multiple Supports. Point of Load in Midspan

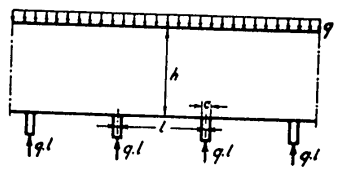

Fig. 34 - Continuous Beam Resting on Multiple Supports. Uniformly Distributed Load

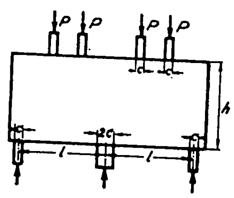

Fig. 35 - Continuous Beam on Three Supports as in the Concrete Test

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of the center moment in the span. (The moment at support = $-0.20Pl$; the midspan moment = $+0.23Pl$). In beams of ordinary depth, the moment at supports = $-0.33Pl$ and the midspan moment = $+0.17Pl$. In the photoelastic test, the value for the moment at supports was very low for the homogeneous uncracked beams, while the cracked and reinforced beams gave almost the same distribution of moments as in the concrete test. The spread of the support is an important factor in local compression above the support, and consequently in the moment distribution. This is evident from the fact that when the ratio $\frac{\text{spread of support}}{\text{span}} = 1/10$ and $h/l = 1$, the compressive stress above the center support is of the same magnitude as the midspan deflection in a simply supported beam.

It is also obvious that the bending resistance of a supporting column is an important factor in distributing the moment. There is no justification for analyzing the effect of the spread of supports and the elastic deflection for every case that comes up in practice.* Instead, the design should be made on the basis of safe values for span and support moments and on estimates of the effects of support displacements which can be both of an elastic and of a constant nature.

The midspan moment in a beam on three supports with a ratio $h/l = 1$ is taken as equal to the midspan moment in a simply supported beam, and the moment at supports equal to the support moment in a fixed beam of ordinary depth. The midspan moment varies in a straight line from a value at $h/l = 1$ to that at $h/l = 1/3$, which is valid for a beam of ordinary depth.

*One should, however, take into account the effect of the spread of the support in levelling off the peak support moments.

The recommendations previously referred to for the design and area of reinforcement required under typical conditions of loading are to be found in Fig. 36. It gives the reinforcement in a continuous beam (Fig. 36-a and b), including shear reinforcement. At the supports where the beam is simply supported (for end supports see Fig. 36-c and d) the shear reinforcement is not shown. In these portions the reinforcement should be provided in accordance with recommendations given in Section 2B. The formulas have a validity for the ordinary values of the ratio c/l, where $c/l \gtrsim 0.2$.

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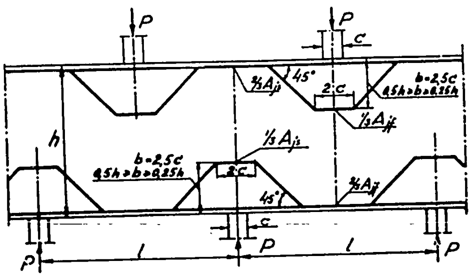


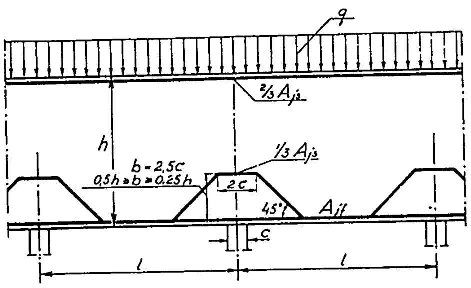
$$A_{js} = A_{jf} = \frac{P\left(\frac{l}{3} - \frac{c}{8}\right)}{\sigma_{j\,till}\left(\frac{7}{8} \cdot \frac{h}{K} + 0{,}2\,b\right)}$$

K - enl. fig. 32 för punktlast i mitten

$\sigma_{j\,till}$ = Allowed unit stress in steel

K - as in Fig. 32 for center load

Fig. 36-a - Point of Load Applied in Midspan of a Continuous Beam on Multiple Supports



$$A_{jf} = \frac{K\,q\,\frac{l^2}{24}}{\sigma_{j\,till}\,h}$$

K - enl fig 32 för jämnt fördelad last

$$A_{js} = \frac{ql\left(\frac{l}{12} \cdot \frac{c}{8}\right)}{\sigma_{j\,till}\left(\frac{7}{8} \cdot \frac{h}{K} + 0{,}2\,b\right)}$$

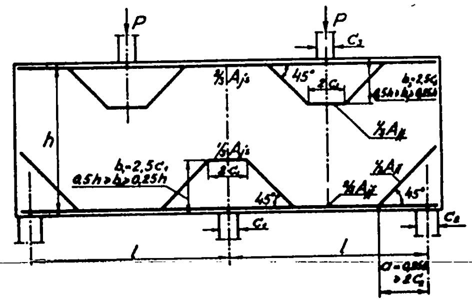
K - enl. fig. 32 för punktlast i mitten

$\sigma_{j\,till}$ = Allowed unit stress in steel

K - as in Fig. 32 for uniformly distributed load

K - as in Fig. 32 for center load

Fig. 36-b - Uniformly Distributed Load in a Continuous Beam on Multiple Supports



$\sigma_{j\,till}$ = Allowed unit stress in steel

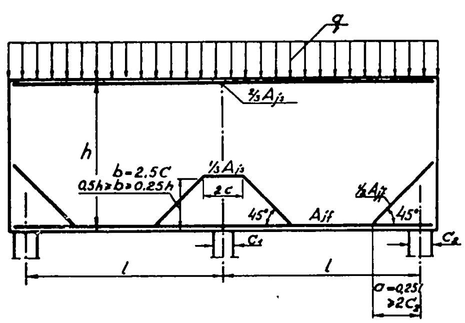
$$A_{js} = \frac{P\left(\frac{3l}{16} - 1{,}4\frac{c_1}{8}\right)}{\sigma_{j\,till}\left(\frac{2}{3}\cdot\frac{h}{K} + 0{,}2\,b_1\right)}$$

$$A_{jt} = \frac{Pl\left(\frac{13}{64} - \frac{3}{64}\cdot\frac{l}{h}\right) - P\cdot\frac{c_2}{8}}{\sigma_{j\,till}\left(\frac{2}{3}\cdot\frac{h}{K} + 0{,}2\,b_3\right)}$$

K - enl. fig 32 för punktlast i mitten

K - as in Fig. 32 for center load

Fig. 36-c - Point of Load Applied in Midspan of a Beam on Three Supports



$\sigma_{j\,till}$ = Allowed unit strees in steel

$$A_{js} = \frac{ql\left(\frac{l}{8} - 1{,}25\frac{c_1}{8}\right)}{\sigma_{j\,till}\left(\frac{2}{3}\cdot\frac{h}{K} + 0{,}2b\right)}$$

K - enligt fig 32 för punktlast i mitten

K - as in Fig. 32 for center load

$$A_{jf} = \frac{Kql^2\left(0{,}1525 - 0{,}0275\frac{l}{h}\right)}{\sigma_{j\,till}\cdot h}$$

K - enligt fig 32 för jämnt fördelad last

K - as in Fig. 32 for uniformly distributed load.

Fig. 36-d - Uniformly Distributed Load in a Beam on Three Supports

FIG. 36 - REQUIRED REINFORCEMENT AND AREA UNDER SEVERAL TYPICAL CONDITIONS OF LOADING. RANGE OF VALIDITY: $3 > l/h > 1$

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