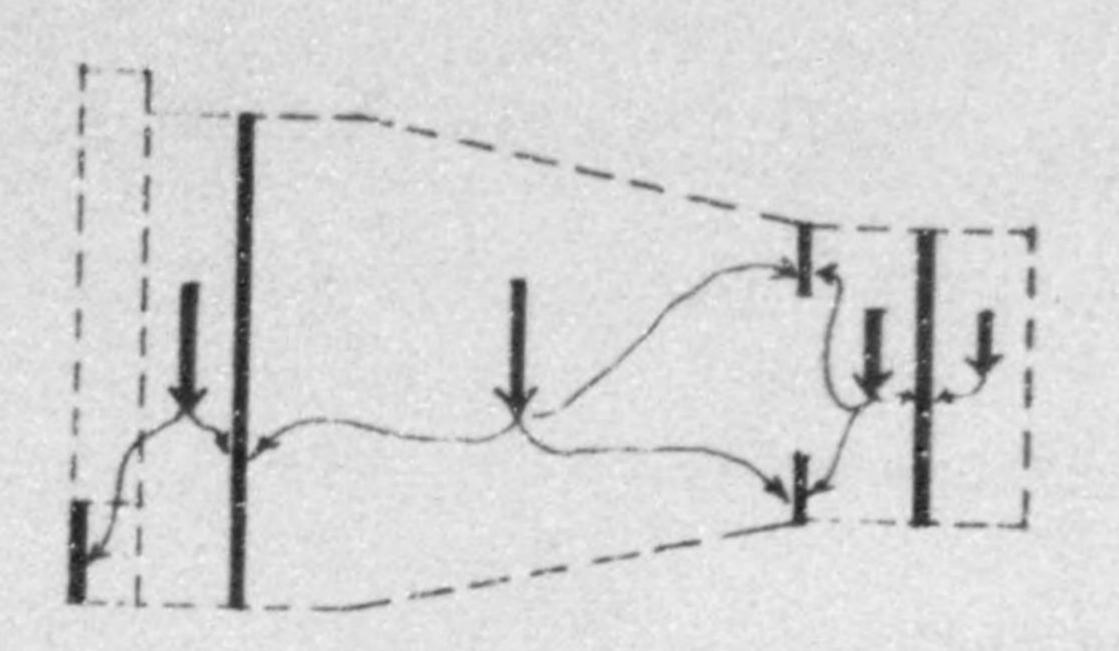
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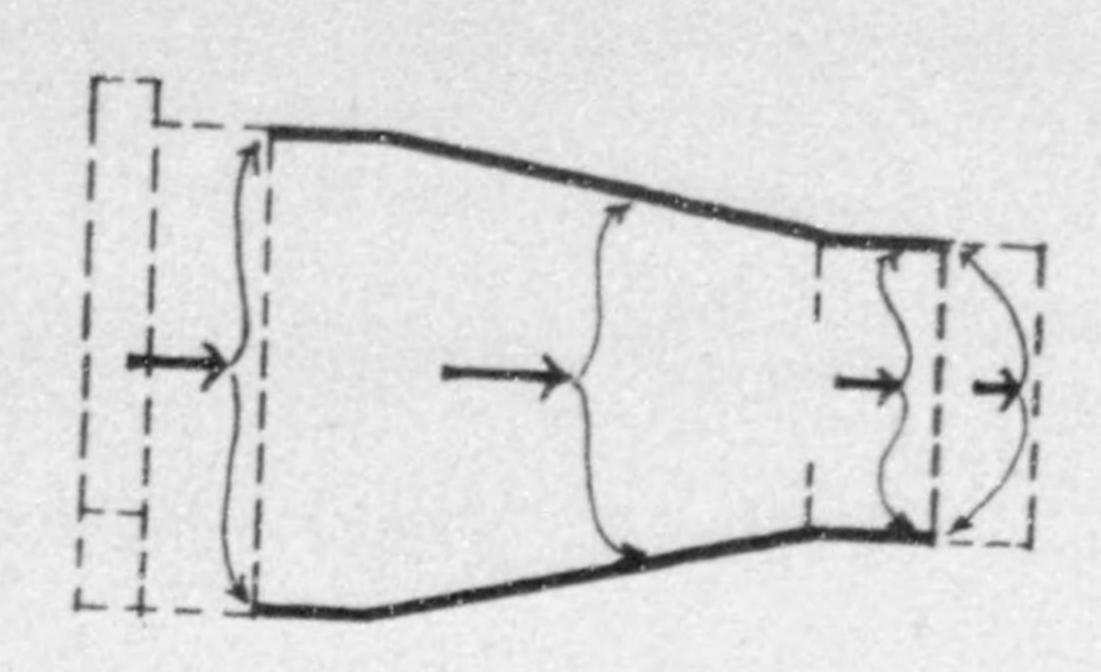
forced concrete
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TRANSVERSE FORCES



LONGITUDINAL FORCES
Fig. 283

Action of Earthquake Resisting Walls Army Theater

simultaneously but separately,
in longitudinal and transverse
directions. The action of horizontal forces in the diagonal
direction was not considered.

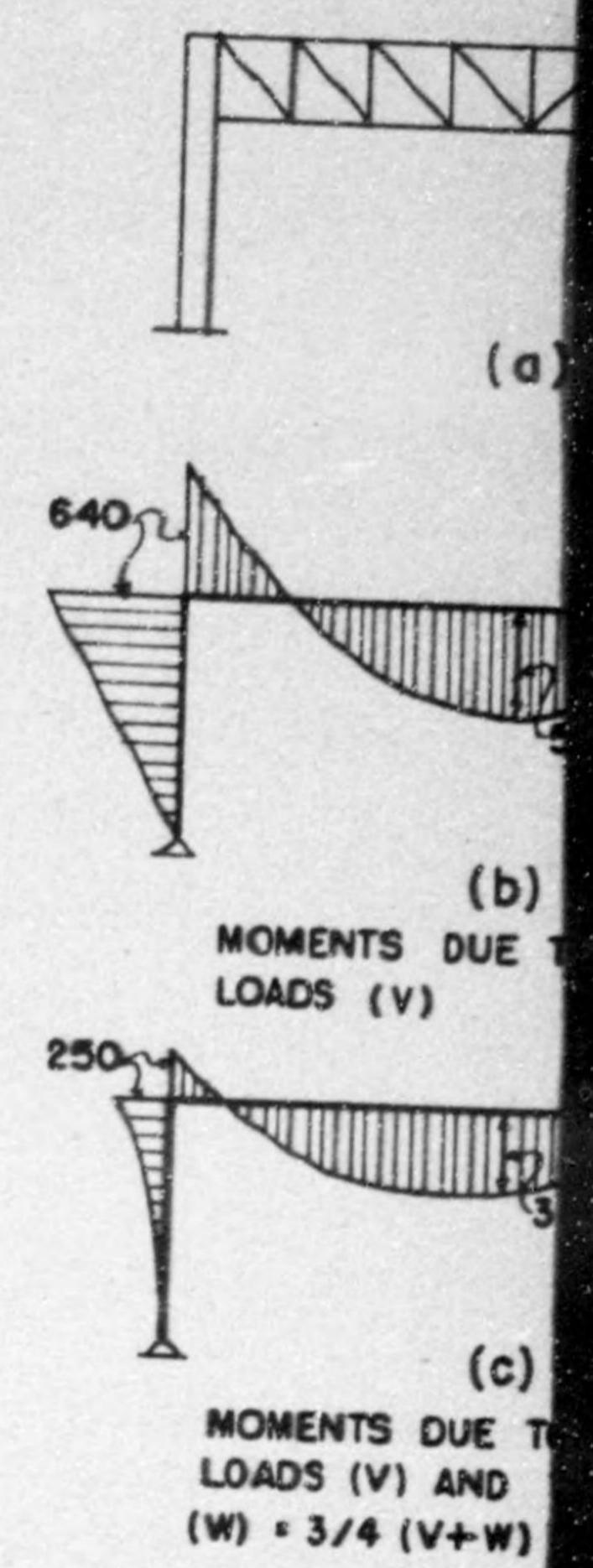
of the roof trusses was obtained by using I section joists.

Vertical bracing was provided at proper intervals to insure the steel roof trusses' acting as a unit. Additional horizontal bracing was provided at

the top chord level so that, together with the purlins, a netlike construction was obtained. This framing not only will
resist the earthquake forces at roof level but will transmit
them to earthquake resisting walls. The horizontal forces of
the front portion are likewise transmitted to earthquake resisting walls. See Fig. 283 for a sketch of the locations of
these earthquake resisting walls and their contemplated action.
The trusses T₁ and T₆ are encased trusses and have been provided
to permit secure and rigid anchorage for the horizontal bracing
members, thus ensuring effective transmission of earthquake
forces to the resisting walls.

- (3) Study of Auditorium Frame Types The most important structural element in the building is the framing of the auditorium. In adopting the final framing, three frame types were studied and computed and are briefly described:
- (a) Continuous Frame The frame shown in Fig. 284 is the usual type of framing used to withstand vertical and horizontal loads. The stresses resulting from the above loadings are as indicated in Fig. 284-b. The effective moments due to vertical loads and wind pressure are on the basis of 3/4 of the sum of moments resulting from vertical and horizontal loadings and are shown in Fig. 284-c. In order to resist these moments, the column section at the level of the roof truss must be approximately 16" x 75" with 2 per cent reinforcement. But a column section of such size would compel radical changes to be made in the architectural design; consequently this type of framing was abandoned.
- (b) Continuous Frame Provided with Horizontal Roof Trusses Providing horizontal roof trusses at the
 level of the top chord of the roof trusses, shown in Fig. 285-a,
 may decisively reduce the moments in the columns due to horizontal forces by transferring these loads to reinforced concrete
 walls on both sides of the frames. Moments, as shown by Fig.
 285-b, due to vertical loads are still large, however, resulting
 in a column section approximately 16" x 65" with 2 per cent rein-

forcement. Suc able changes in was also abando



Study of Continuation Army The producing negligible moments were small becomes approximate A column section

the architectural most satisfactory

3. CENTRAL central steam gen

forcement. Such a section would still have required considerable changes in architectural design, so this type of framing was also abandoned.

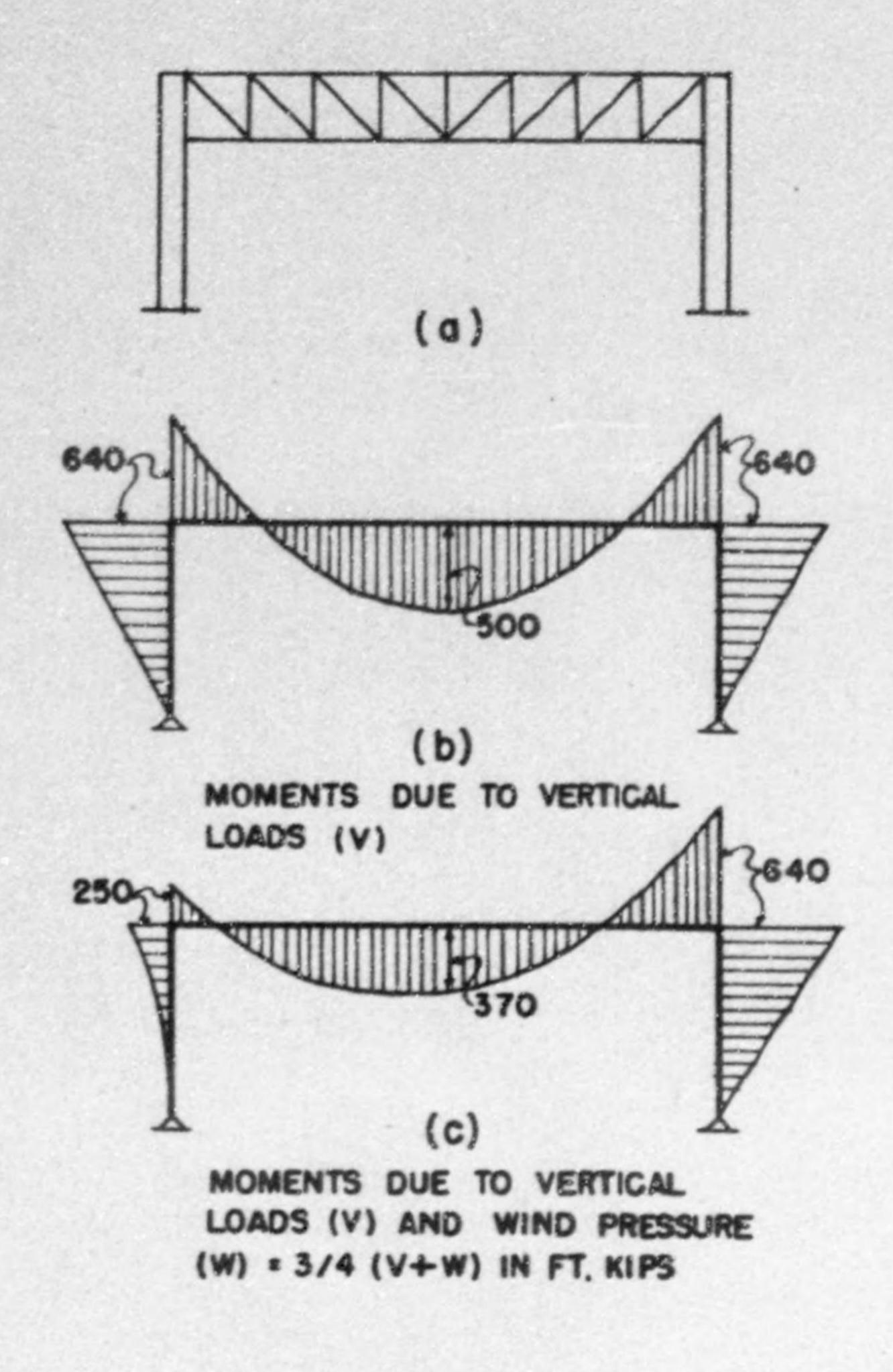


Fig. 284
Study of Continuous Frame
Army Theater

With Horizontal Trusses and
Simply Supported by Columns The framing shown in Fig. 286-a,
wherein the roof truss is simply
supported on columns, disrupts
continuity and eliminates
column moments due to vertical
loads, as shown in Fig. 286-b.
The horizontal forces are transmitted by horizontal trusses on
the top of the roof trusses to
the reinforced concrete walls
on both sides of the frame,

producing negligible moments in the columns. The design column moments were small, as shown in Fig. 286-c, so that the section becomes approximately 16" x 24" with 2 per cent reinforcement.

A column section of this size did not require alterations in the architectural design. This type of framing was adopted as most satisfactory in the final design.

3. CENTRAL STEAM GENERATION STATION - The design of a central steam generation station having a capacity of 270,000

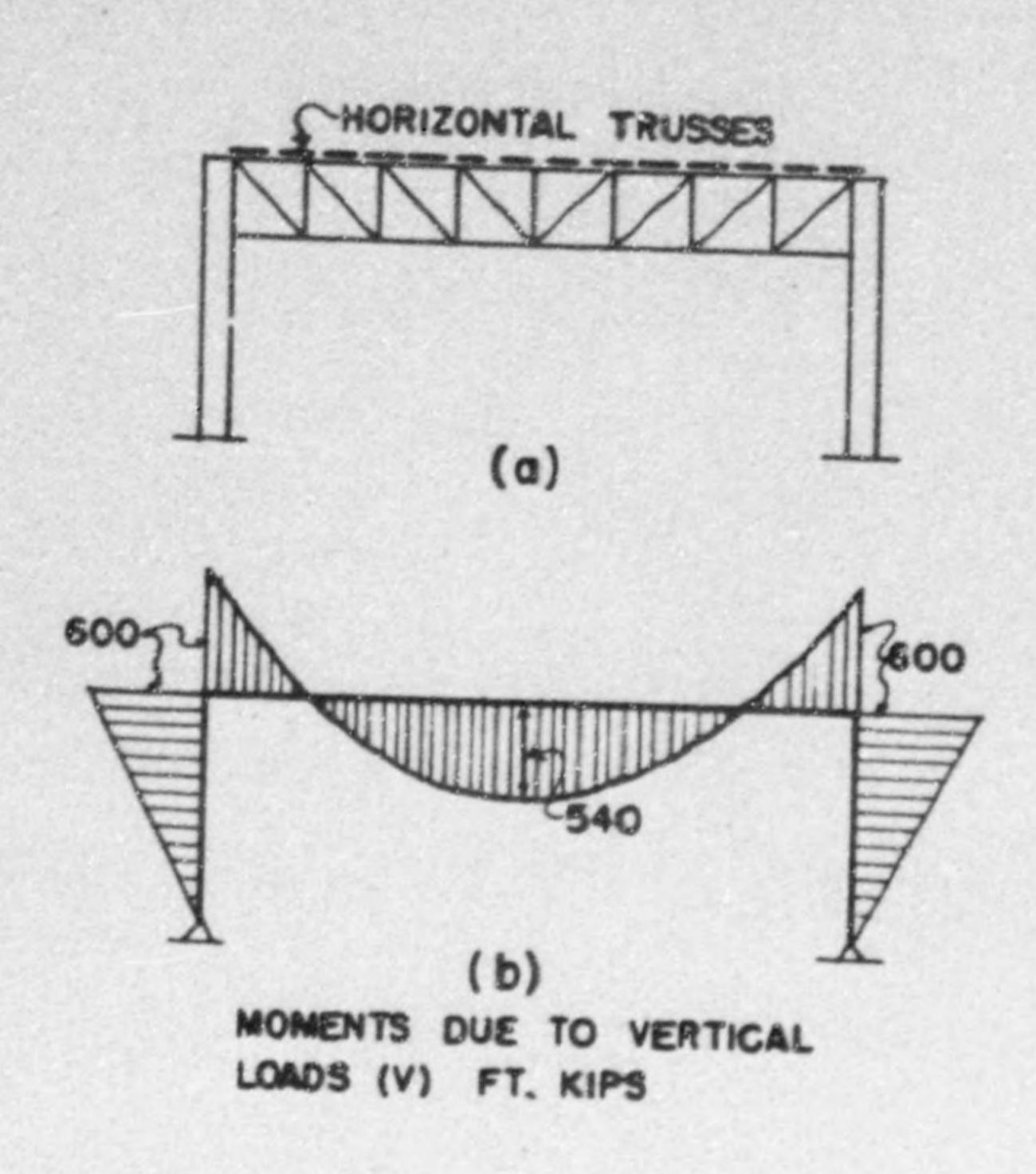


Fig. 285

Study of Continuous Frame With Horizontal Truss - Army Theater

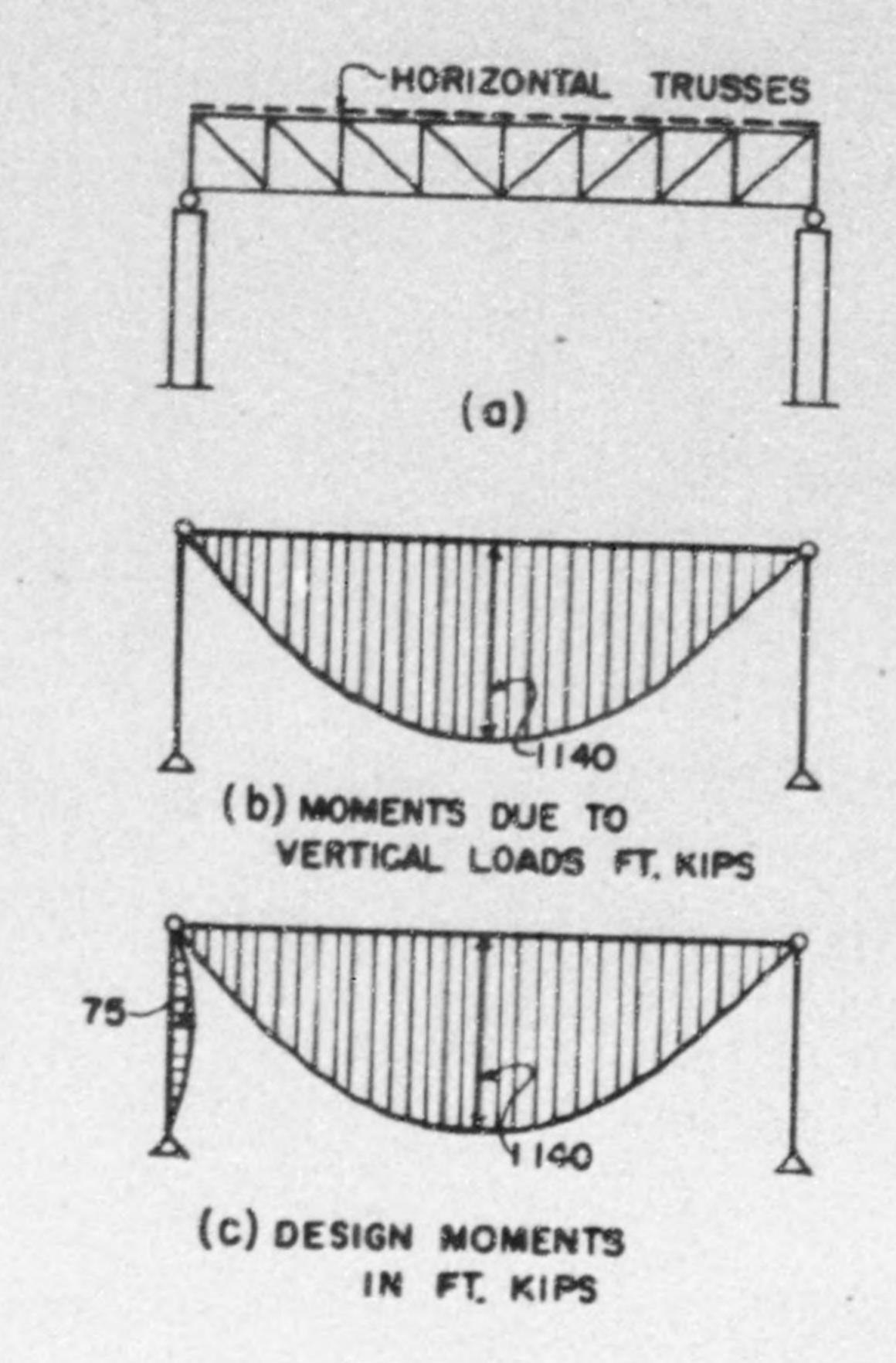


Fig. 286
Study of Continuous Frame With
Horizontal Truss - Simply
Supported Columns

pounds of steam per hour, at normal rating, was done in the summer of 1947. The plant has since been constructed and is now in operation. Fig. 287 shows a longitudinal sectional elevation of the building.

The ground conditions at the site were a major factor in keeping the building above ground. A hard layer of yellow clay was found to be located from minus 2.0 feet to minus 10.0 feet throughout the area. Below this depth the yellow clay was interlensed with soft blue clay. Borings were made on 25 feet grid points and large diameter wells, allowing in-place investigation of the soil, were drilled near the building site.

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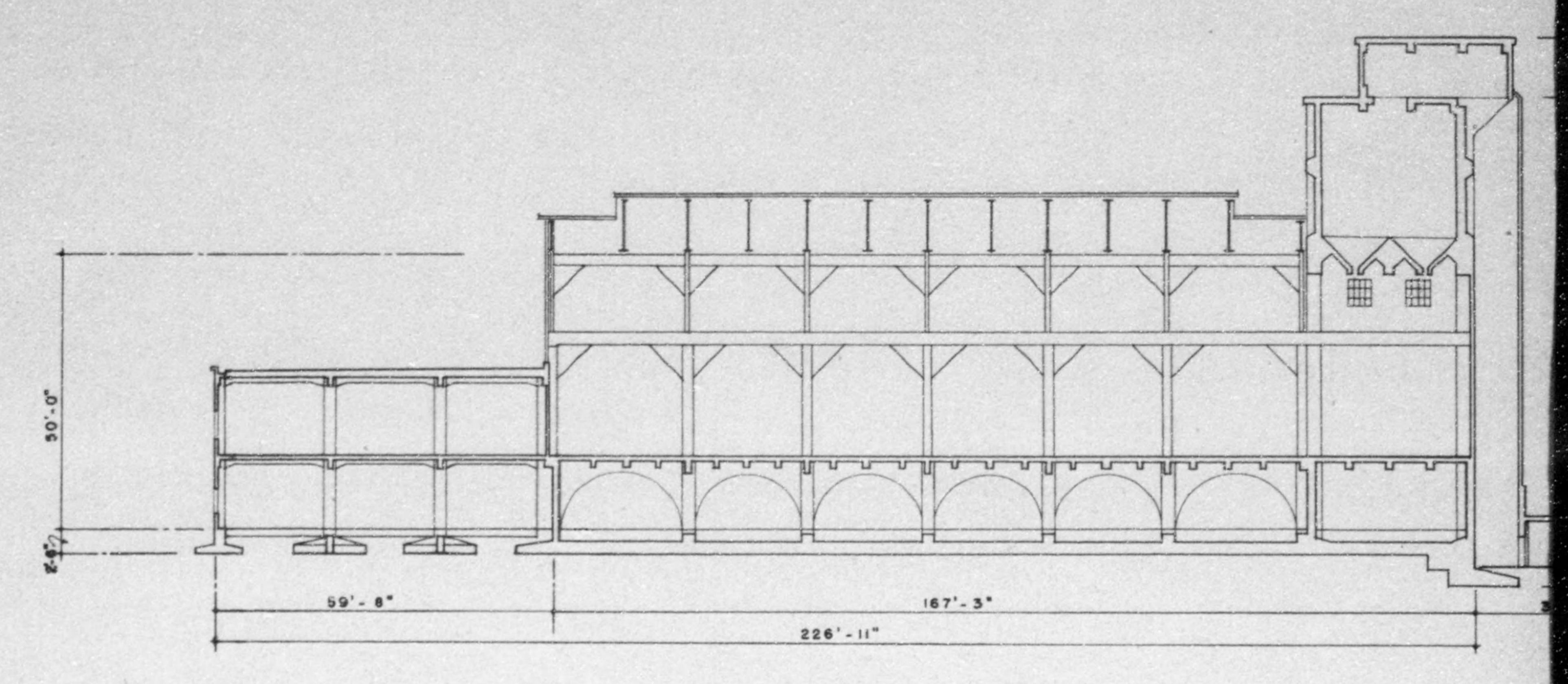
Circular plate bearing and pile loading tests were conducted at different elevations and locations throughout the building area.

The main portion of the building housing the boilers is of reinforced concrete up to the firing floor level. The front portion of the building consists of utility rooms, offices, and toilets and is entirely of reinforced concrete. The elevated coal bunker is entirely of reinforced concrete. Factors used in the design, in addition to the usual assumptions, were a seismic coefficient of k = 0.15g and a wind loading of 25 pounds per square foot on a projected vertical plane.

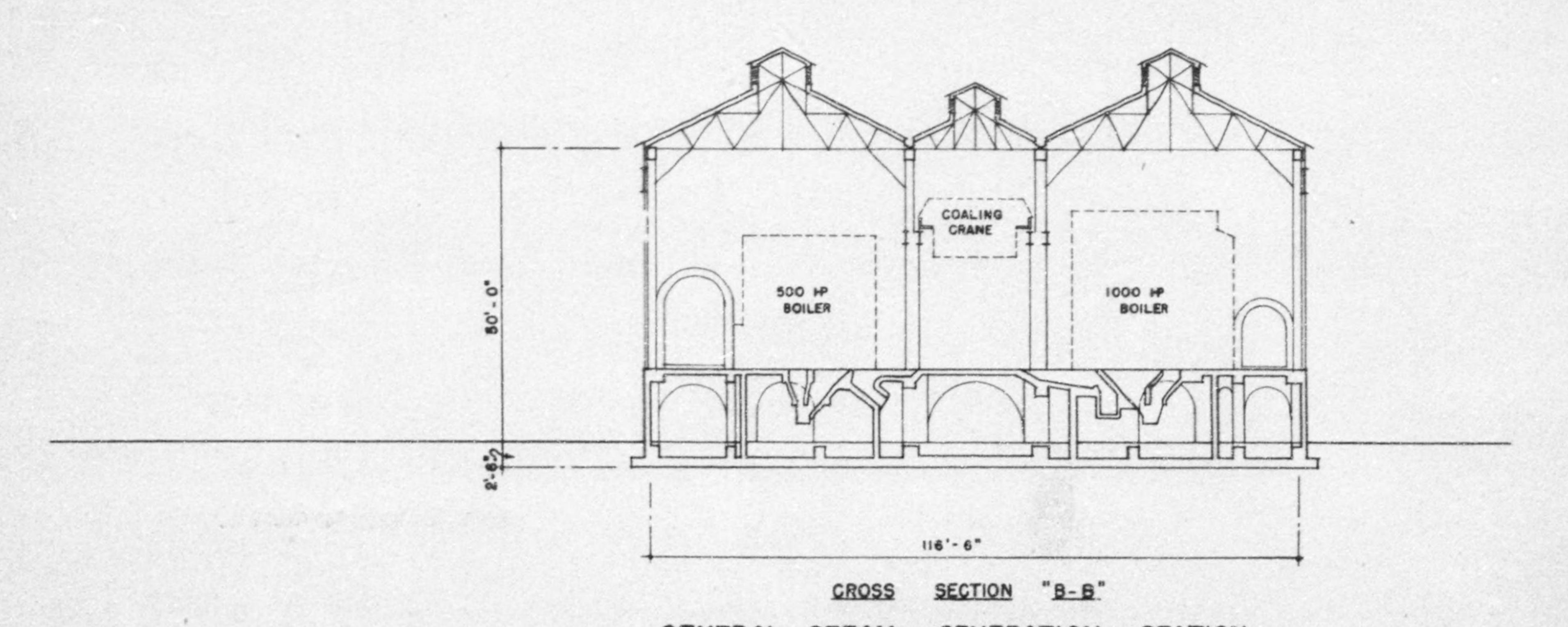
Separate mat foundations under each boiler and under the coal storage bunker were originally considered. Unit foundation loadings and a study of the effect of earthquake action on the building made it necessary, however, to put a mat foundation under the entire main portion and to make it integral with the coal bunker foundation, see Fig. 288. Solid walls, in the main, were utilized as boiler supports and stiffening arches were used longitudinally and transversely. The first floor area houses ash pits, ash sluicing equipment, feedwater pumps, forced draft fans and air ductwork.

The chimney foundation footing bottom elevation is minus

10.0 feet. Soil bearing capacity was considerably lower at
this elevation and unit loads, particularly those that would develop during an earthquake, required that piling be used. The

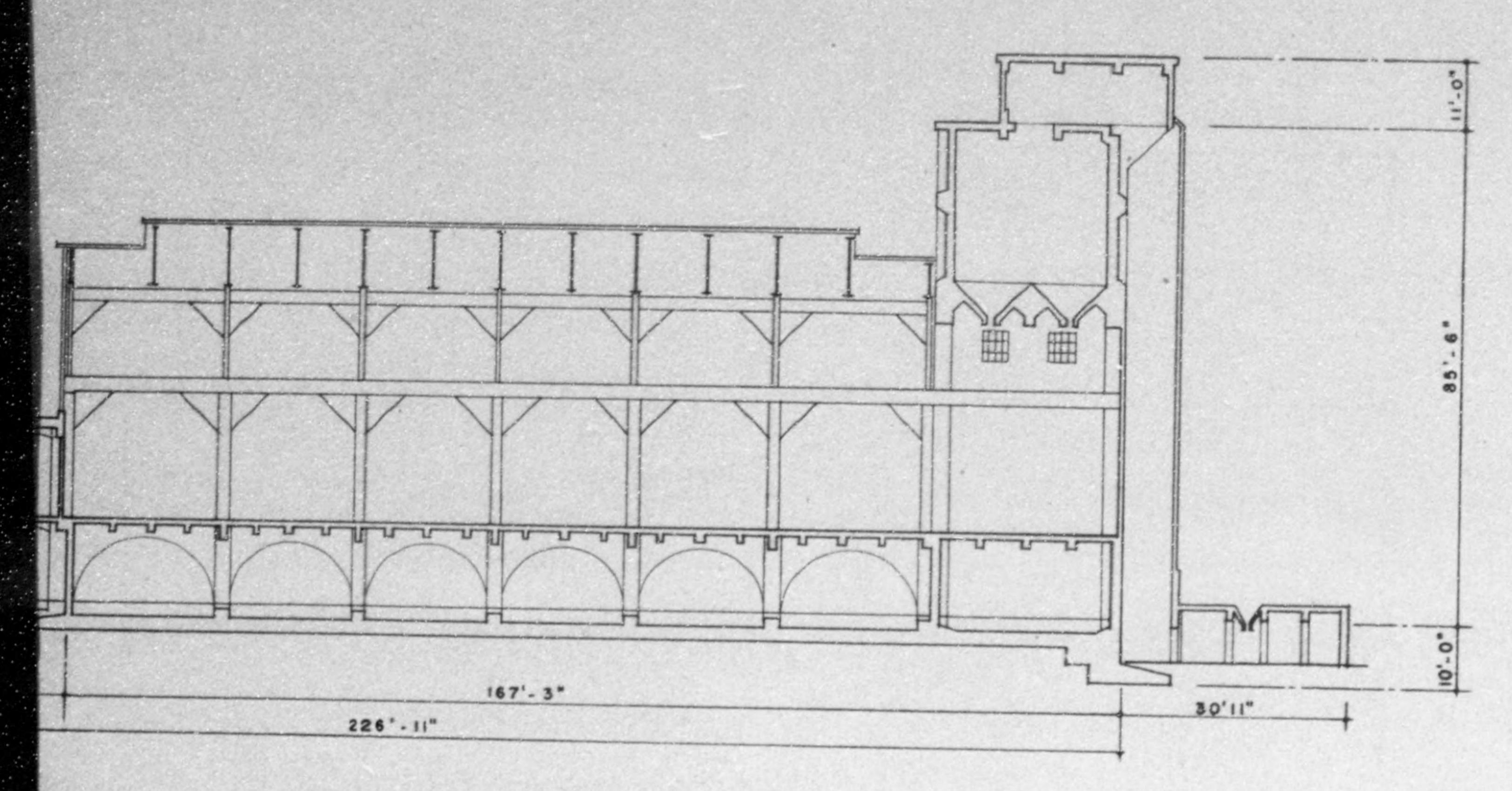


LONGITUDINAL SECTION "A - A"

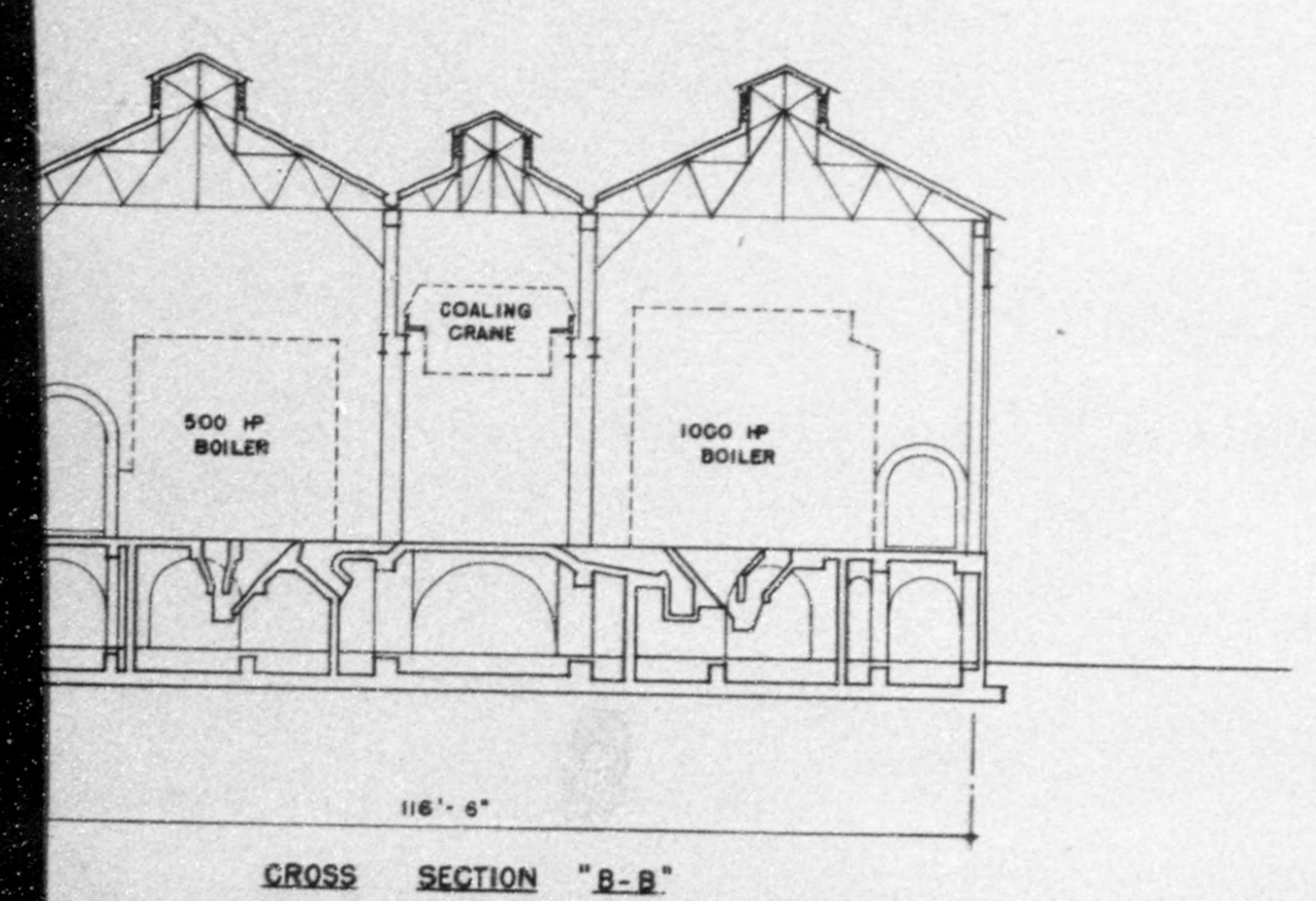


CENTRAL STEAM GENERATION STATION
ELEVATION SECTIONS

Fig. 237

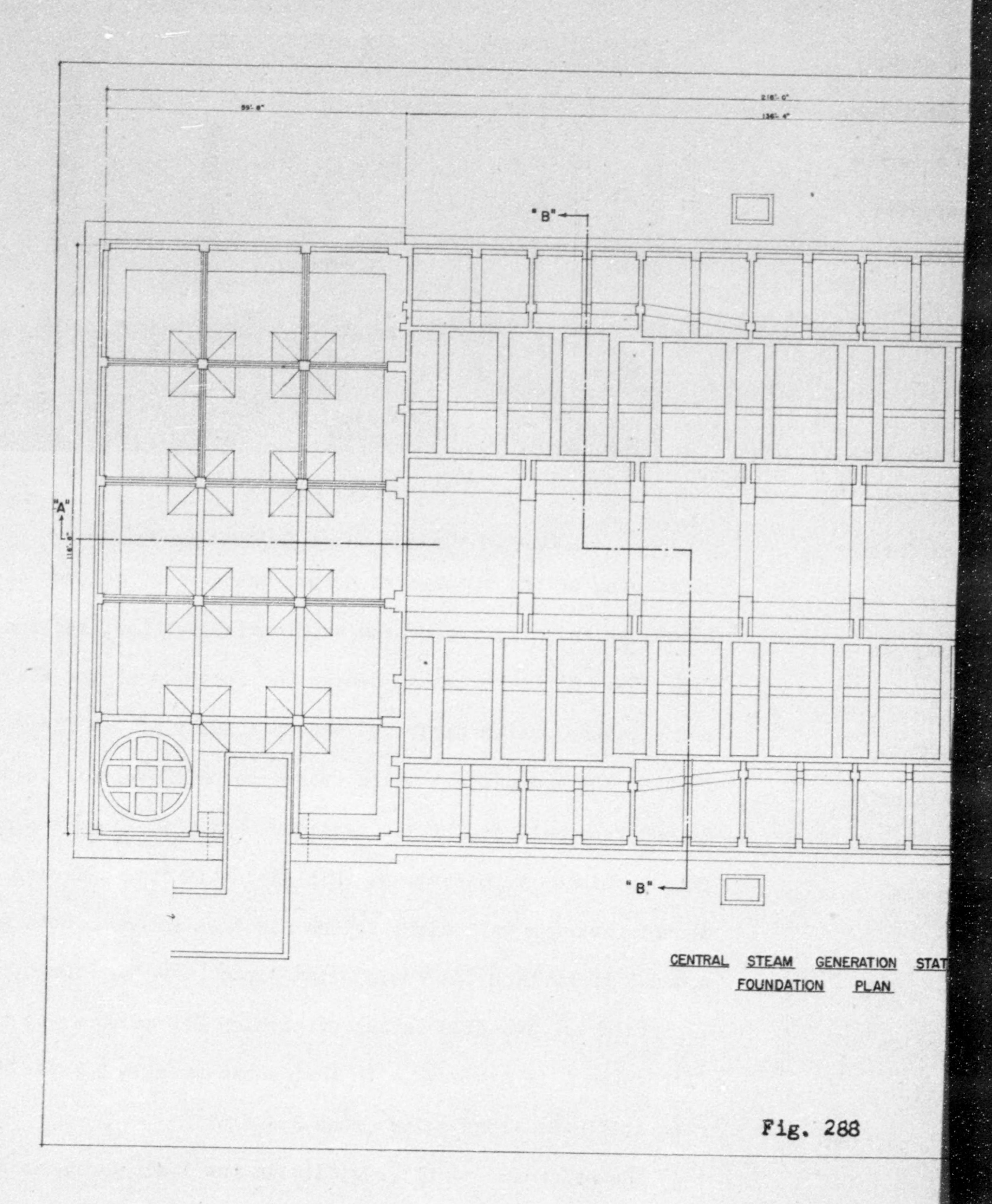


LONGITUDINAL SECTION "A - A"



NTRAL STEAM GENERATION STATION
ELEVATION SECTIONS

Fig. 237



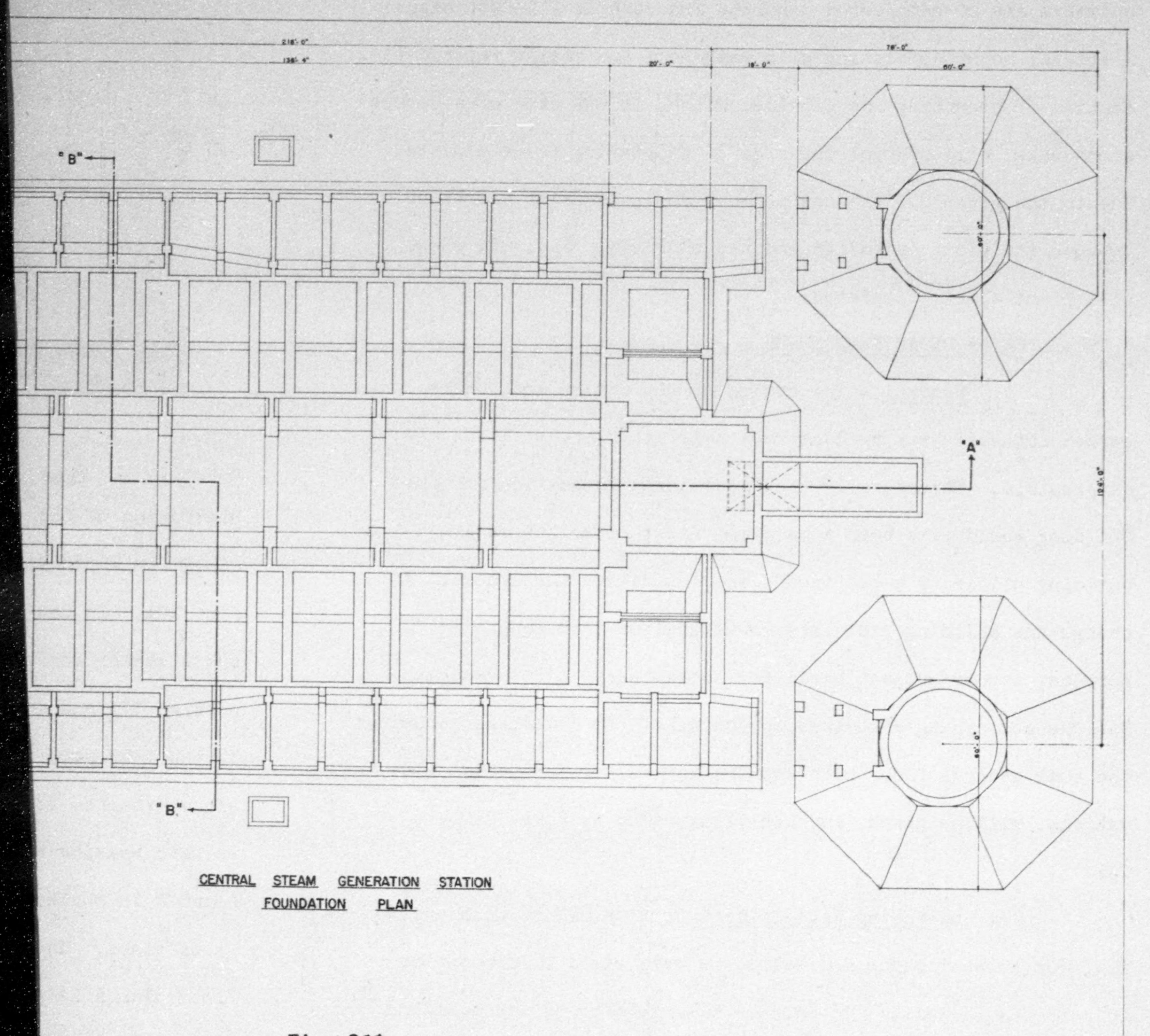


Fig. 288

chimneys are of reinforced concrete and each is 210 feet high.

A special joint consisting of a weakened, and easily replaceable, section of breeching was provided which, in the case of a severe earthquake, will control the area of fracturing to be expected due to the great differences in mass and periods of vibration between the plant buildings and the chimneys. Fig. 289 shows this plant almost completed.

4. DAIWA DEPARTMENT STORE

- a. General The desire of the owners and others concerned naturally was to construct this store at as little cost as possible. Structurally, the most ideal foundation for this building would have been a basement floor under the entire building utilizing a continuous mat foundation throughout. To change the building radically makes it, for this report's purpose, a poor subject basis for comparison. A few recommendations concerning structural planning of the building, assuming the same general floor plan and the same floor foundation elevations, will be given, and are illustrated by Figs. 290 and 291.
- b. Balancing Rigidities Portion III of this building, due to staircases and walls, is very rigid in proportion
 to portions II and I. To balance the rigidity of the building
 in the Y-direction (transverse), the framing of the front portion
 is stiffened by addition of seismic bearing walls and the



stiffening of 4C-4D, as show Fig. 291. In and elevator a portion III as to the rear was ent approaches seismic bearing A and B in portion slabs. floor plan A in more stiffness

The balar complished by

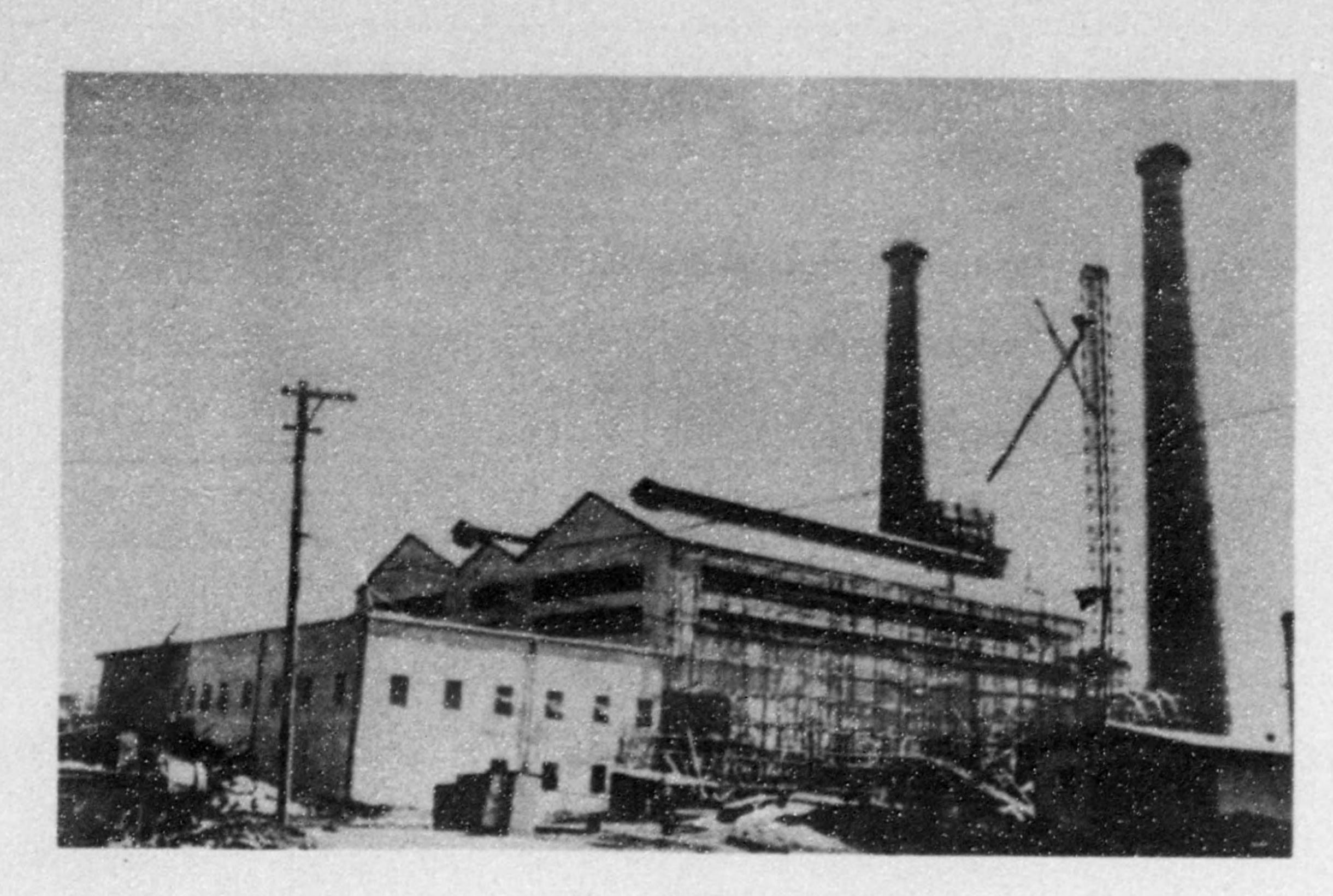


Fig. 289

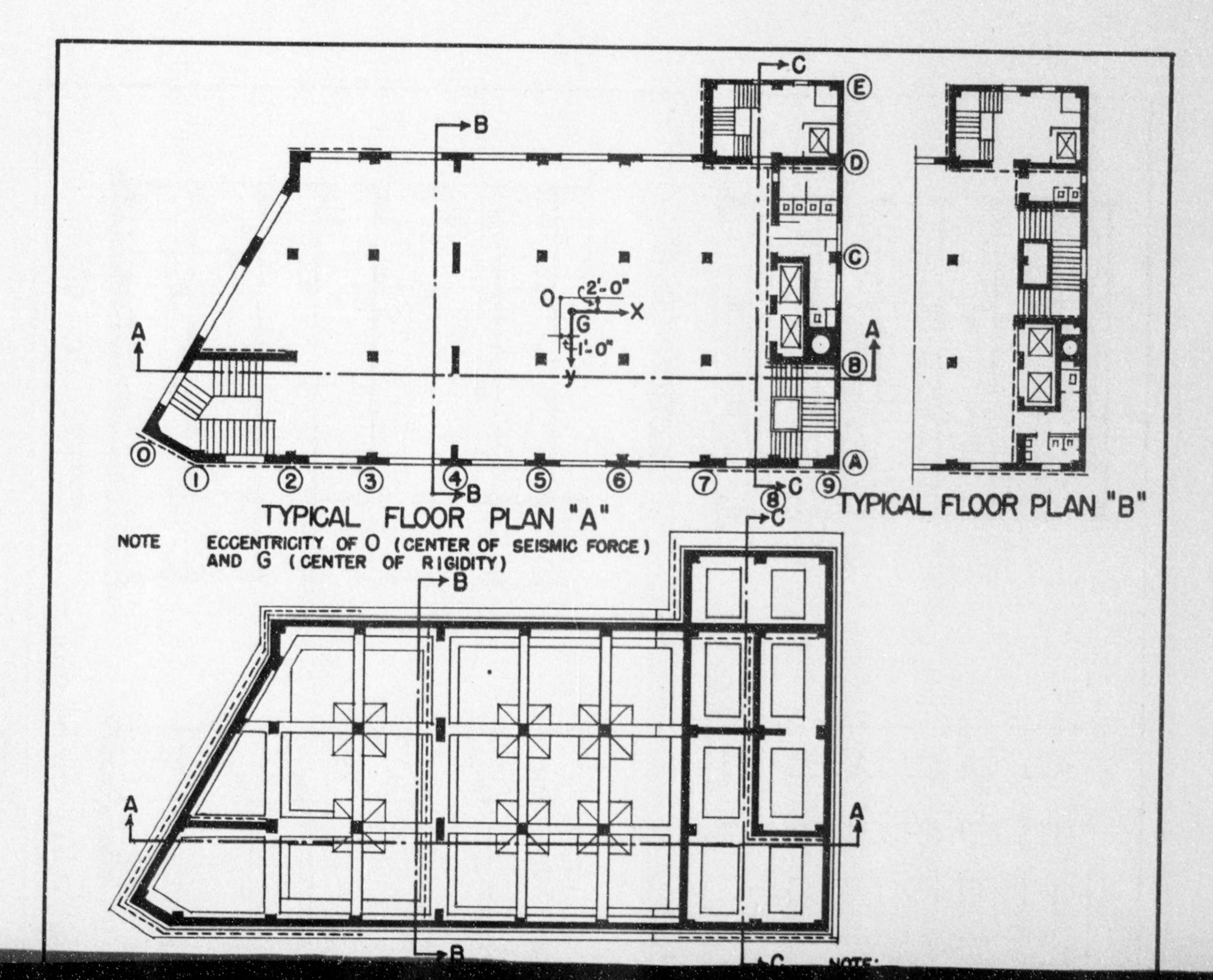
View of Central Steam Generation Plant stiffening of the transverse frame, composed of columns 4A-4B-4C-4D, as shown in transverse sectional elevation, section "B-B", Fig. 291. In the original design the location of the stairwells and elevator shafts destroyed the continuity of the floor in portion III and did not allow transmission of seismic loadings to the rear wall in the Y-direction. Fig. 290 shows two different approaches to circumvent this discontinuity. Placing a seismic bearing wall along column row 8 as shown on both plans A and B in portion III would allow direct transmission by the floor slabs. The plan layout of portion III as shown by typical floor plan A is preferable to that shown by plan B as it has more stiffness along column rows B and C.

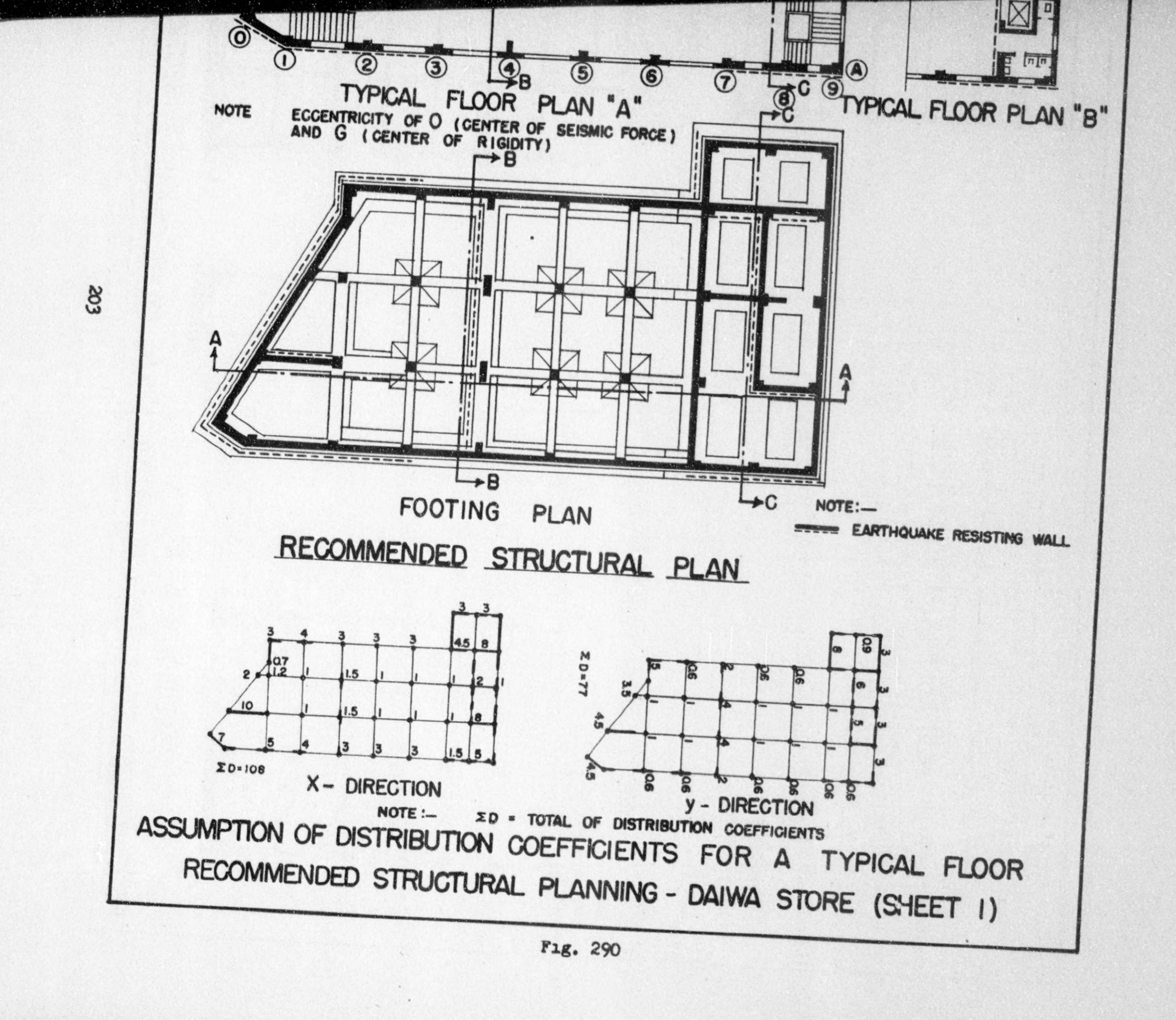
The balancing of the rigidity in the X-direction is ac-

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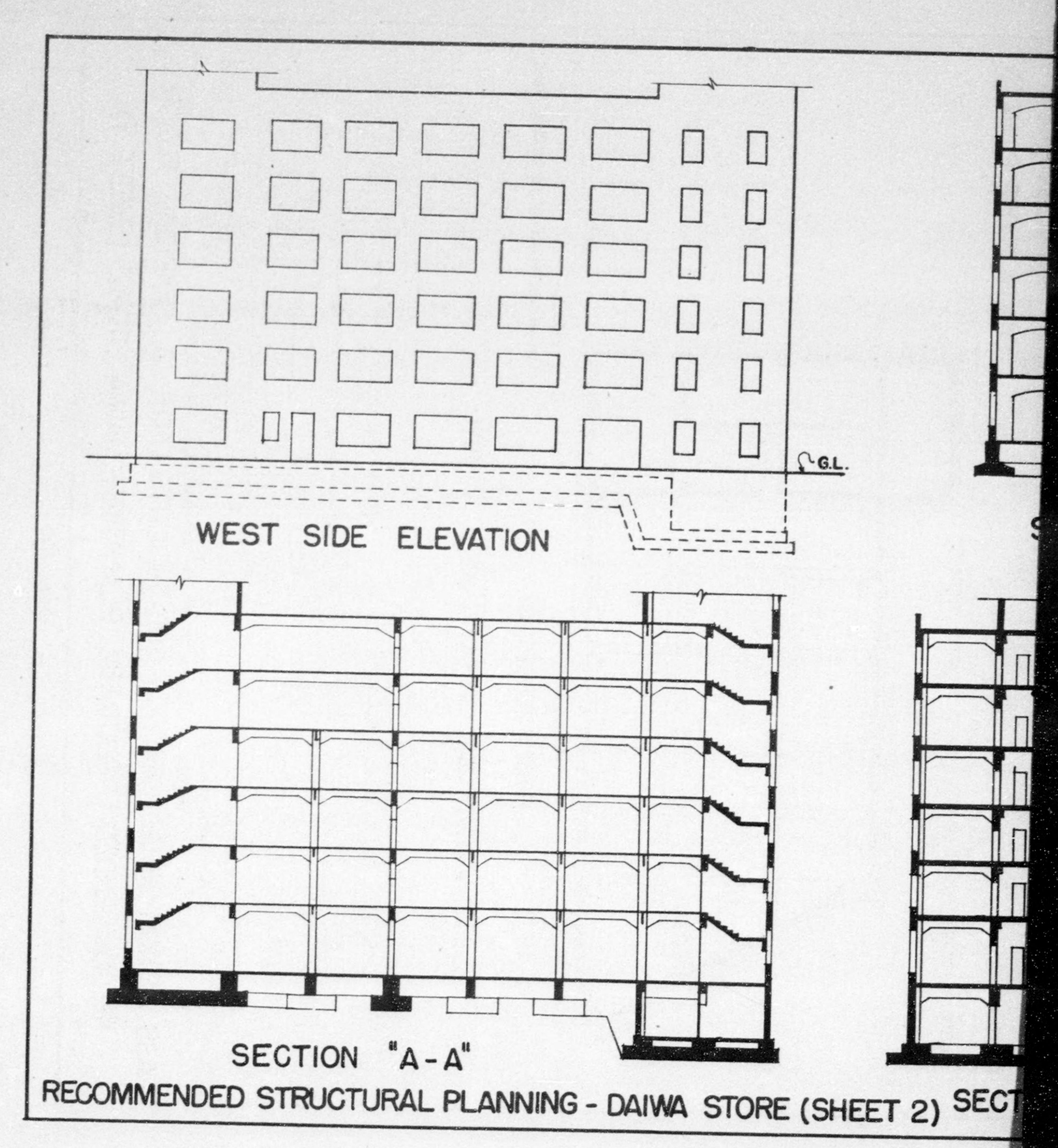
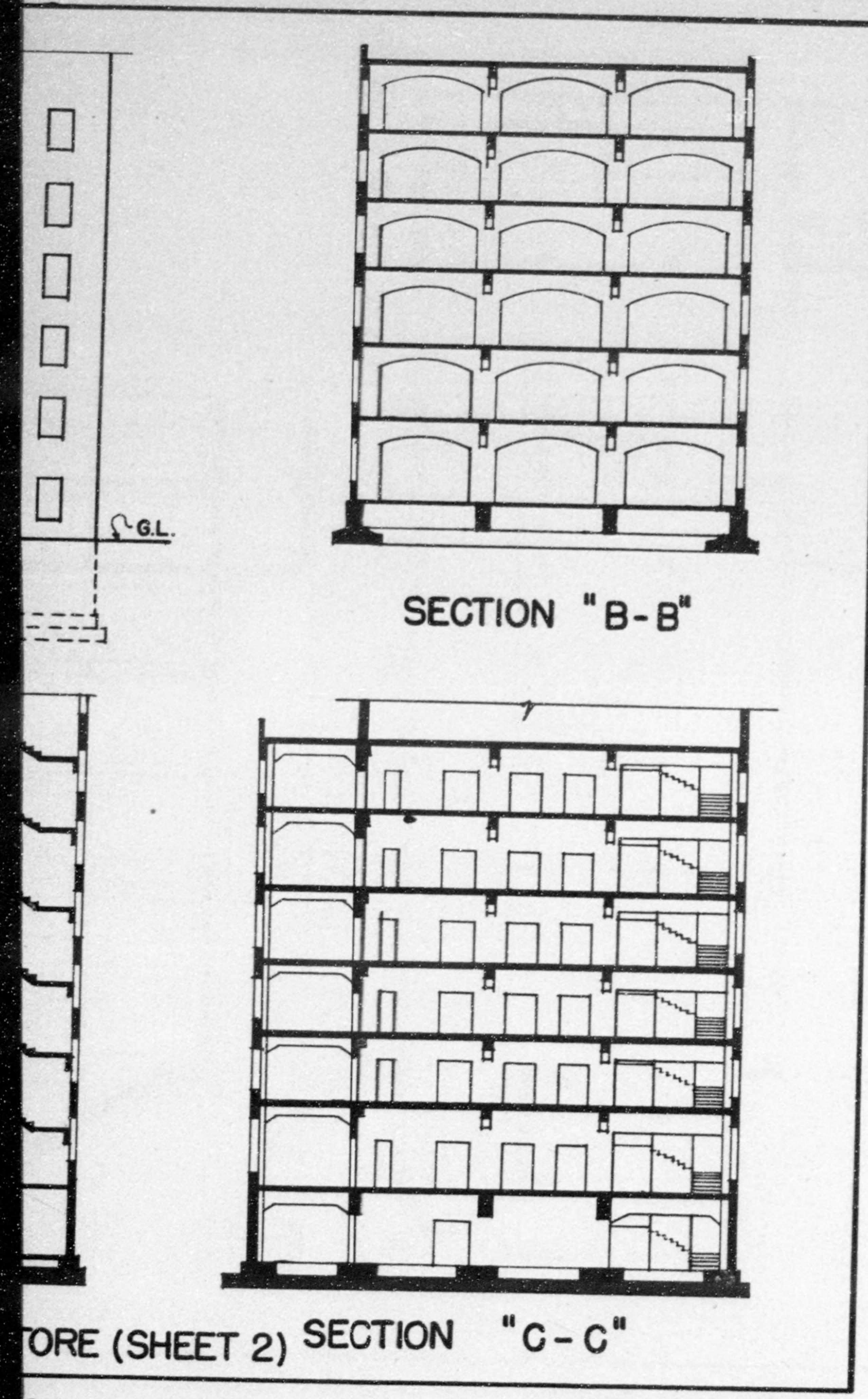


Fig. 291



shown in Fig. 290. The center of gravity and the center of rigidity of the building are shown in Fig. 290. It is seen that these two centers are close enough together so that the torsional effect corrections may be neglected. Corrections for eccentricity when the building is set into the ground are generally made discounting the restraining effect of the foundations.

Seismic force distribution coefficient assumptions for the X and Y directions are also shown in Fig. 290.

The foundation plan, shown by Fig. 290, provides a mat foundation under portions I and III. Continuous outside foot-

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ings are provided in Portion II with a continuous footing under column row 4, and a continuous footing between columns 2B and 3B to resist the loading provided by the bearing wall between columns 1B and 2B in the X-direction.

The west side elevation of the building and the longitudinal sectional elevation, section "A-A", Fig. 291, show the recommended method of foundation transition between portions II and III, along the column rows A-B-C-D. It is advisable to continue the flanges on the outside footing to effect continuity.

Section "C-C", Fig. 291, shows a section through portion 3 and the recommended mat foundation.