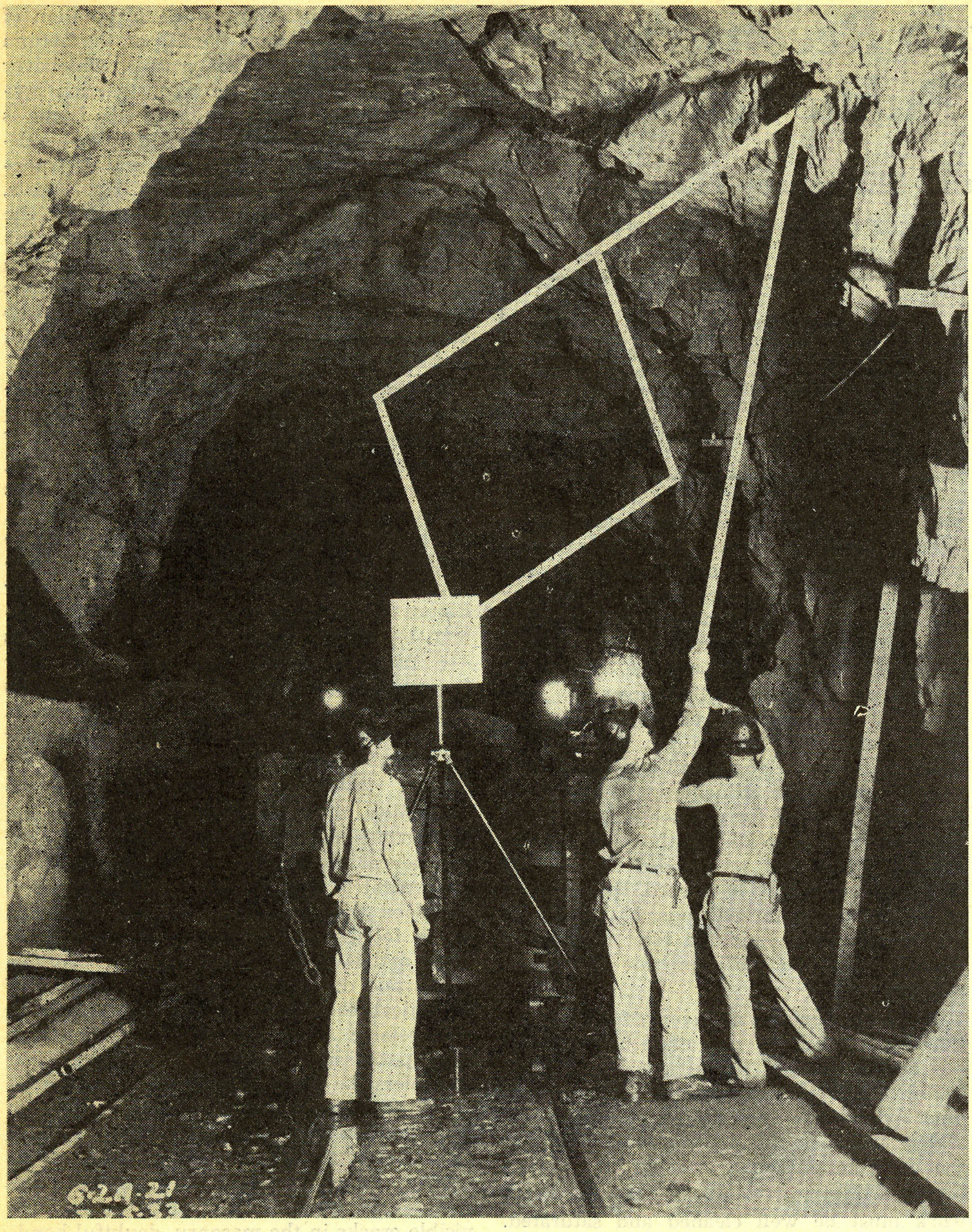


This picture shows giant pantograph being used to make cross section of 16-ft. Colorado River Aqueduct tunnel. An accurate reduction 1 to 24 of contour of tunnel excavation is traced on sheet of paper, 12x12 inches. Cross section shows if tunnel is being driven in conformity to engineer's design.

Pantograph Is Used in Making Cross Sections of Aqueduct Tunnels

To determine as the work progresses whether the bores are being driven true to the engineer's design and the fixed grades is one of the important duties of the engineers in charge of the construction of the 91 miles of tunnels on the Colorado River Aqueduct of the Metropolitan Water District of Southern California. This requires the making of accurate cross sections of the tunnels as the excavation advances, generally at intervals of about five feet.

Because of the size of the tunnels which will have an inside diameter of approximately 16 ft., use of methods formerly in vogue would have been cumbersome and expensive. However, by resort to giant pantographs this work has been greatly simplified and expedited. This instrument was originally designed for the enlargment or reduction of pictures but it has been adapted to practical engineering work with remarkable success and it is possible with the pantograph to make an accurate cross section of rock excavation in the tunnels in about ten minutes, whereas by some old methods it would have required hours of laborious and patient effort.

For the making of tunnel cross sections the District purchased at the outset a number of Proebsteel pantographs capable of making a reduction of about 1 to 24 to accurate scale of the tunnel excavation. The pantograph is mounted on a tripod made of steel pipe. An aluminum plate 16x16 inches with a hub in the center, to which the pantograph is attached, is bolted to an adjustable upright standard clamped in the top of the tripod. This plate is set at the proper level on the main axis of the tunnel and adjusted with a spirit level. A sheet of cross section paper, 12x12 inches, on which the cross section of the excavation is traced, is fastened on the aluminum plate. The frame of the pantograph is made of spruce and has a workable range of from 5 ft. 3-in. to 12 ft. 4-in. A rubber roller is fitted on the end of the extension arm which is drawn over the surface of the rock with a long rod by the man operating the pantograph.

Conformity of the excavation to the engineer's design is determined by placing a template made to exact scale over the cross section tracing made by the pantograph. This template is cut to the line of excavation required. Any projection over this required line is at once indicated and instructions are given for the amount of additional excavation necessary at that point. The amount of overbreak is also indicated by the template. Area of the cross section as recorded by the pantograph is determined by a planomiter.

Plasterer Loses Appeal to Save State License

Nelson S. McCartney, Burlingame plastering contractor, lost his appeal to obtain reinstatement of his state contractor's license, which was revoked several months ago.

Petition for reinstatement was denied by Glen V. Slater, assistant registrar of contractors, in confirming recommendations made by Orman Lutz, referee, as a result of a formal hearing and investigations conducted during the last three months.

Revocation of the original license followed a hearing on charges that McCartney had substituted a cheaper and inferior grade of plaster in construction of a Burlingame home than the quality which his customer had authorized.

In applying for reinstatement, McCartney was given an opportunity to introduce expert testimony to disprove the report of Abbot A. Hanks, Inc., San Francisco, engineers and chemists, which resulted in cancellation of his license.

While several contractors and associates of McCartney testified in his behalf at a recent hearing conducted by Referee Lutz at Burlingame, McCartney failed to produce technical or expert testimony to refute the Hanks report, according to Mr. Slater.

Woman With Clever Talk Imposes on Architects

Recently several architects have received a call from a prospective client—a woman seemingly well educated and refined who talks quite deliberately and at considerable length about a residence she proposed to build upon a large lot, a hillside site, in Hollywood. She is much interested in music for which provisions must be made. She discusses the style of architecture, the cost, her finances, the family relations and draws a floor plan. She then suggests an appointment, will leave her card, but upon opening her hand bag suddenly discovers she has lost or misplaced her purse. Then follows a pantomime of distress, a story of a trip with a friend who has gone on to a beach town and she has no way of reaching her destination. It concludes in the architect advancing just a car fare to reach a suburban town.

Economical and Safe Reconstruction Is Proposed for Public Schools

Engineer Offers Plan to Eliminate Hazards Conclusive Answers to Many Problems Involved

By Martin Pohl

(Copyrighted 1933 by Martin Pohl)

(Continued from last week)

Sixth installment of report entitled "Survey of Earthquake Damage and General Recommendations for Redesign and Reconstruction of Public School Buildings," by Martin Pohl, consulting engineer, architectural department, Los Angeles Board of Education. First installment was published in issue of December 15.

XIII—GENERAL DETAILS OF CONSTRUCTION FOR CLASS ROOM BUILDINGS

Means of Tying Existing Buildings to the New Strengthening Devices. To derive the full benefit from the various reinforcement devices proposed in Section VII, it is of utmost importance that all parts of the building be securely tied and interlocked with the new strengthening device. This purpose is achieved by the various pilasters and bracing details shown in Figs. 37 and 39. These details, however, will require modifications to suit the needs and special conditions of the individual projects.

Pilasters. The installation of pilasters will be necessary at each vertical bent, panel, or other strengthening device.

In Class "A" frame structures pilasters are required at the exterior face of all outside walls and at corridor side of interior longitudinal walls.

In buildings of the brick bearing wall type with steel lintels supported by brick piers, the concrete encasing the steel column must extend in the form of an inside pilaster, as shown by dotted lines in Fig. 37. This measure is necessary in order to prevent lateral displacement of lintels or "pushing through" at their bearings.

An acceptable alternative is the design of a steel bracket extended from the column to lintel, or any other detail worked in conjunction with the strut member along the inside face of the exterior wall. The strut member is a part of the horizontal bracing system indicated in Fig. 37.

If solid reinforced concrete walls are used as a vertical stiffening device, the inside pilasters at exterior walls are to be cast as an integral part with the wall, as shown in Fig. 34 (see issue of Jan. 19). This method of construction will add greatly to the strength of the vertical panel due to the increase of its moment of inertia.

Even if the ends of the exterior brick walls are rigid enough not to require strengthening by a special transverse frame, it is nevertheless imperative to install pilasters at corners, since the corners of the building receive most strain by impact of earthquake action. Also, torsional moments generally reach their maximum in both the horizontal and vertical plane at the corners of a building.

The pilasters at the intermediate reinforcement devices serve merely as an important detail for tying the existing structure to the new strengthening device, but the corner pilasters fulfill a primary structural function, as explained in Section XI, and are to be designed with this particular purpose in view as discussed in the issue of January 19 under the heading "Strengthening by System of Outside Buttresses."

Keys in brick piers are to be provided for anchors, as shown in Fig. 37. The latter must be provided at all floor levels and at window heads to secure direct lateral support of lintels, girders and floor joists; and at intervals between such points as bending and shear resistance of pilaster will necessitate. The reinforcing steel in the pilaster (Figures 37, 39 and 40), is to be determined by assuming beam action over a span equal to the spacing of anchors; additional reinforcing is to be provided to make up a reinforcing detail consistent with sound construction principles. The anchors are to be designed for one-tenth of all loads tributary by the weight of the pier and the pilaster, lintel and joist reactions, etc. If detail shown in Fig. 40 is used, the net area of the anchors at threads must be introduced for tension value, and the total number of threads must be sufficient for shear resistance.

Preparing Surfaces of Contact for Guniting Pilasters. The brick pier must be "toothed" out at the ends of the pilasters, and horizontal reinforcement encased in concrete must extend into such places, so as to completely interlock the present brick pier and the concrete pilaster.

Before guniting the pilaster the old brick work must be well cleaned and saturated. Complete saturation of all surfaces of contact cannot be achieved by merely wetting the brickwork previous to guniting. It must be kept wet for several days before concrete is applied.

XIV—RECONSTRUCTION SCHEMES PROPOSED FOR AUDITORIUMS AND BUILDINGS OF SIMILAR TYPE

This classification includes auditoriums, gymnasiums, large study halls, libraries and shops. Buildings belonging to this class of structure are characterized by their general layout which extends over a large area without intermediate cross walls or other means of stiffening.

Rigidity for this type of construction may be supplied in two ways:

A. By sufficiently rigid exterior walls.

B. By individual piers or bents.

Which of the two methods is to be adopted depends upon the physical character of the building, its general layout, and proportions of wall openings, etc. If solid or practically solid end walls are available, type "A" may be the most economical as well as the most effective solution.

XV—REDESIGN AND STRENGTHENING OF AUDITORIUMS UTILIZING THE RIGIDITY OF PRESENT WALLS

General Method of Design. This method of strengthening requires the installation of a framework in either the top or the bottom chord plane of the roof trusses, whichever is most adaptable to the reconstruction layout. The structural function of this frame is to transfer all lateral forces due to earthquake motion and wind to the end or transverse walls or to some other rigid parts, which in turn must be investigated for the loading imposed by the frame and the increments of lateral force due to their own weight. In Figure 38 is given a diagramatic layout of a structure belonging to the group of buildings under consideration which wlil illustrate all necessary design operations.

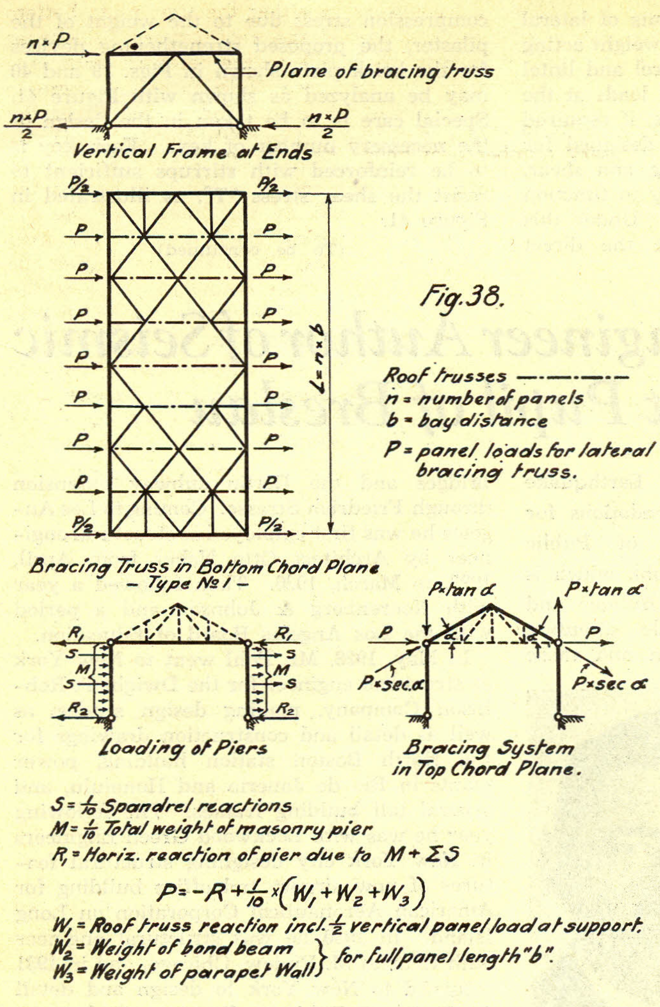
Determination and Application of Seismic Loading—The panel loads "P" represent all increments of lateral earthquake forces. If the bracing system is located in an inclined top chord plane, the horizontal panel loads "P" are to be resolved into their components, as shown. The vertical component is to be resisted by the pier. The panel load "P" due to all loads "W" (see explanatory note under Figure 38), is transmitted to the bracing system by direct application. "R" represents the reaction of the pier when the pier is considered laterally supported at its footing and at height of bracing truss. All seismic forces due to lintel and girder reactions in the wall are transmitted to the bracing system by the agency of the piers.

Stability of Present Masonry Piers and Suggestions for Additional Security. Under the lateral earthquake loading tributary to the weight of the masonry and the spandrel reactions, the pier acts as a beam supported at its footing and at height of the bracing truss and is to be investigated for bending, shear, and direct compression load. Even if a statical calculation shows the maximum compressive stress and shear to be within their permissible limits, it is suggested that the piers be strengthened individually, as shown in Figures 39 and 40. The purpose of this new strengthening device is to prevent single bricks and small units of masonry from sliding out or "pushing through" at lintel seats or at other heavy concentrations. Invisible cracks in the masonry, doubtful bonding, the customary type of workmanship, and the impossibility of determining the quality and strength of the mortar in each and every case throughout the entire job, would necessitate the strengthening device as a precautionary measure, even though not required for purely statical purposes.

Recommendations for Reconstruction. The reconstruction work recommended for this class of buildings is chiefly confined to parapets, bond beams, strengthening of masonry walls and piers, and the installation of lateral bracing in the roof truss system.

A. Remodeling Scheme Pertaining to All Buildings of this Group. As shown in Fig. 39, parapets, bond beams forming the architrave, and strengthening devices of piers are cast as an integral part, and the reinforced concrete pilasters bind all brickwork, lintel seats, and other concentrations in the piers into one homogeneous structural unit. Under the design proposed, parapet walls and cornices could not be shaken down by even a violent quake. By the use of this design and by careful attention to all details of construction, a hazard of serious consequence to human life would be eliminated.

B. Buildings with Steel Trusses. Buildings with steel trusses are in most cases equipped with sway bracing. These bracings, however, are merely a matter of conventional design.



Diagramatic Layout Illustrating General Method of Design.

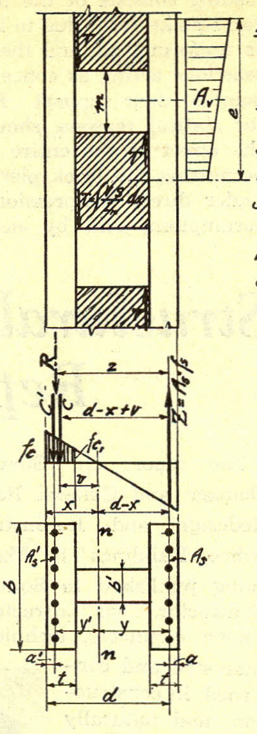


Fig. 41

Notations:

T = Shear stress in keys.
V = Total shear at any section.
I_n = Moment of Inertia of entire section.
z = Leverarm of resisting couple.
S = Statical moment of pilaster with resp. to neutral axis.
S_c = " " " compression flange.
S_s = " " " steel area at tension face.

$$S = S_c = n \cdot S_s = 15\, S_s$$

A_v = Area of shear diagram
C = Total compressive stress in concrete.
C' = " " " " steel.
R = Resultant force of C and C'.

$$C + C' = Z$$

All other symbols are standard in concrete design and evident by figure.

Shear Design

$$T = \int \frac{V \cdot S}{I_n} dx = \int_0^e \frac{V}{z} dx = \frac{A_v}{z} = V \cdot b' \cdot m \qquad m = \frac{A_v}{z \cdot b' \cdot v}$$

Total Stress "T" is to be resisted by reinforcing steel.

Bending Analysis

$$f_{c_1} = f_c \frac{x-t}{x}, \quad f_s' = n \cdot f_c \frac{x-a'}{x}; \quad f_s = n f_c \frac{d-x}{x}$$

$$C' + C = \frac{f_c + f_{c_1}}{2} \times t \times b + f_s' \cdot A_s' = Z = f_s \cdot A_s$$

$$x = \frac{\frac{t^2 \cdot b}{2} + n \cdot (A_s' \cdot a' + A_s \cdot d)}{t \cdot b + n \cdot (A_s' + A_s)}$$

$$I_n = \tfrac{1}{3} x^3 b - \tfrac{1}{3}(x-t)^3 \cdot b + n \cdot (A_s' \cdot y'^2 + A_s \cdot y^2)$$

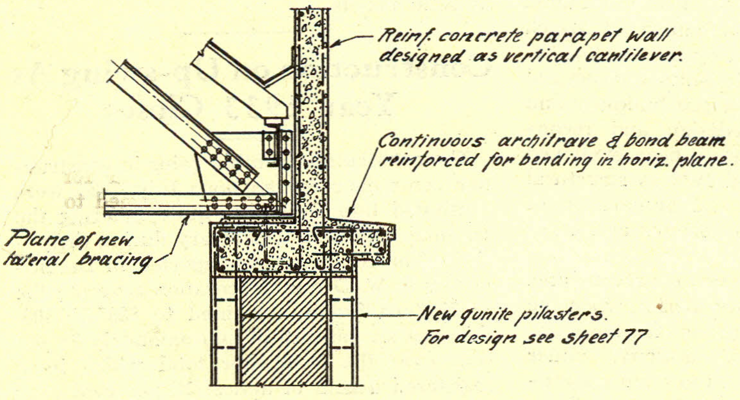
Using structural shapes as shown in Fig. 40, their moment of inertia must not be neglected. In this case "I_n" is to be computed by the following formula:

$$I_n = \tfrac{1}{3} \cdot x^3 b - \tfrac{1}{3}(x-t)^3 b + n \times I_t' + n \cdot A_t' y'^2 + n \cdot I_t + n \cdot A_t \cdot y^2 + n[A_s' \cdot y'^2 + A_s y^2]$$

I_t = I of struct. member in tension face. A_t & A_t' = Area of struct. members
I_t' = I " " " compression face.

$$f_c = \frac{M \times x}{I_n}; \qquad f_s = \frac{n \times M \times y}{I_n}$$

Figure 37 was printed with fifth installment of report in issue of January 19 but is reproduced here because of frequent references to it in the portion of the report published this week.



Cross Section through Wall.

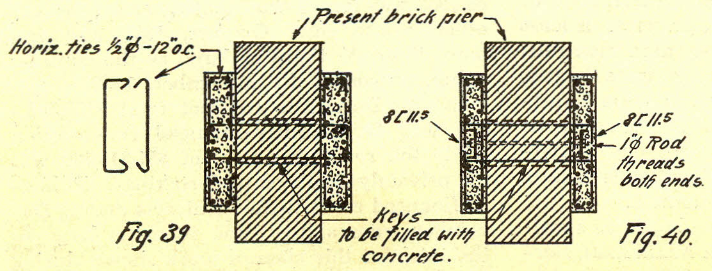


Fig. 39 Fig. 40.

Suggested Methods of Strengthening Walls and Piers of Brick Masonry.

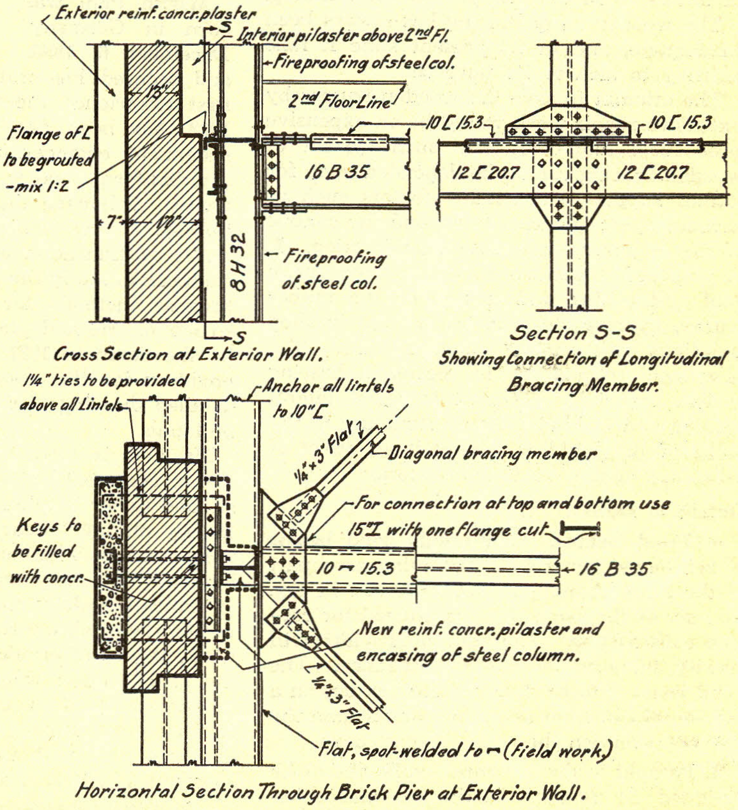


Fig. 37.

General Details of Construction. Portal Frame Connections & Method of Bracing in Horizontal & Vertical Longitudinal Plane.

Although they add to the rigidity in the truss system, they have usually been found to be either incomplete in their layout as a lateral bracing truss, or inadequate in the members and their connections to resist the stresses transmitted to them by seismic loading.

It is suggested that the present systems of bracing be completed by rearranging or adding members and strengthening their connections for their actual design loads. Where this is not feasible, a new bracing truss must be installed. Figure 38 presents a method of bracing which will adapt itself most favorably to similar conditions. Horizontal bracing trusses must be designed for the panel loads "P" as illustrated in the problem shown in Figure 38.

Various other types of bracing trusses will be shown in the next issue when details of design and construction will be given and the adaptability of these types to various reconstruction schemes will be discussed.

C. Buildings with Timber Trusses. During investigations it was often found that large timber trusses over auditoriums, etc., were badly deformed, their tension rods slackened due to shrinkage in timber, and their seats in the masonry attacked by dry rot, to such an extent, in some cases, as to render questionable the value of anchors at support and the shear resistance of the timber. Excessive deformation of the main trusses often caused complete reversal of stresses in the smaller trusses carried by the main trusses, thus putting compression into tension members and causing the latter to buckle, and as a result the weight of the roof was actually brought down to the ceiling joists. These conditions are not the result of earthquake damage, but on the theory that the effect of a quake wave spreads progressively throughout the building until it finds a weakness in the structure, such weak parts are apt to become the cause of failure of some other part of the building.

The wooden sway bracing has always been inadequate, and in its present state it fails entirely to answer its purpose.

The attempt to save the wooden trusses by extensive repair work would be an expensive procedure; moreover, any such system of wooden bracing cannot be depended upon for lateral support, since many factors such as shrinkage, etc., which are beyond the control of the engineer, enter into consideration. It is proposed, therefore, to remove the entire roof and ceiling and to replace the timber trusses by new steel trusses, except in shop buildings and similar small layouts. The new steel trusses are to receive a bracing system especially designed to resist lateral loading and to bind the roof structure to the walls of the building, giving it an effect of unit mass against earthquake motion.

Details of Design

A. Bond Beams. Continuous bond beams of reinforced concrete, functioning as a part of the strengthening device proposed in Fig. 39, may at the same time be utilized for cornice construction. The additional width will greatly increase its lateral resistance. The bond beam is to be designed for bending in a horizontal direction assuming for its span the distance between the center line of piers.

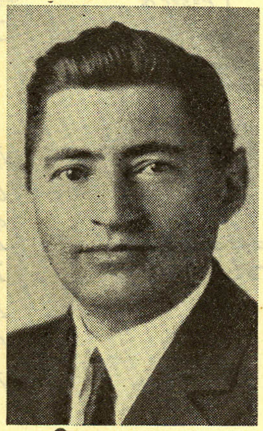
B. Parapet Walls. Parapet walls are to be designed as vertical cantilevers and reinforced for their fixed end-moments at the bond beam.

C. Pilasters. The pier is to be investigated as a simple beam supported at its footing and at height of horizontal bracing truss. The loading consists of the increments of lateral earthquake force due to its one weight acting as uniform load, and the spandrel and lintel reactions acting as concentrated loads at the point of their support. Pilasters, if required for statical reasons, should be designed for the effect of the entire bending and shear, permitting the brick pier merely to function under direct compression load. Under this assumption, and by neglecting the direct compression stress due to the weight of the pilaster, the proposed strengthening devices for the brick piers shown in Figs. 39 and 40 may be analyzed as shown with Figure 41. Special care is to be taken in the design of the necessary number of keys. Each key is to be reinforced with stirrups sufficient to resist the shear stress "T", as illustrated in Figure 41.

(To be continued)

Structural Engineer Author of Seismic Report Pupil of Breslau

The report on "Survey of Earthquake Damage and General Recommendations for Redesign and Reconstruction of Public School Buildings" by Martin Pohl, which is being published in Southwest Builder and Contractor, has attracted wide attention among engineers, technical men and those interested and concerned, in construction, and naturally an interest has been aroused in the author, concerning whose experience and qualifications little has been said, the report being published entirely on its own merits. It is only fair, however, that those interested should know something about him.



Martin Pohl

Martin Pohl was born in Germany, December 6, 1896, and secured his professional schooling and first experience there, but since February, 1923, has been continually employed with prominent engineering firms in New York and California and is a citizen of the United States. Following the earthquake of March 10, 1933, he was employed by the Los Angeles Board of Education as consulting structural engineer to study the damaged buildings and propose methods for redesign and reconstruction. He had previously been employed by the board in 1927 and 1928 as structural engineer for the design and preparation of detailed drawings for auditoriums and other buildings.

Mr. Pohl is a graduate of Staatsbauschule. Zittau Technical University, Berlin, where he studied under Mueller Breslau, foremost authority on the "theory of elasticity," under whose guidance he became thoroughly familiar with the fundamentals in design methods of statically indeterminate structures, such as the theory of elastic energy, principle of least work, theory of elastic curve, area moments, theory of elastic weights, conjugate beams, kinetic theory of structures and slope deflection method. During his entire practice he has kept pace with the present day knowledge and practice in design and construction of the principal types of structures.

His professional experience began as structural engineer with Beton & Monierbau, Berlin, April, 1920, to February, 1923, engaged in design and supervision of construction of reinforced concrete arch, slab and girder bridges and the Berlin subway extension through Friedrich Strasse. Coming to Los Angeles he was first employed as structural engineer by Architect Otto Neher from April, 1923, to March, 1926. Then followed a year with Noerenberg & Johnson and a period with the Los Angeles Board of Education.

In May, 1928, Mr. Pohl went to New York as structural engineer for the Dwight P. Robinson Company, making design studies as well as detail and construction drawings for the North Boston station building, power plants in Rio de Janerio and Honolulu, and several tall building frames. The following year he was with Lockwood Green Engineers in New York city designing structural features of new airport and office building for American Aeronautical Corporation on Long Island. In 1930 he was structural engineer with Charles M. Rudow, Chicago, and in 1931 returned to New York to design and detail for Dwight P. Robinson Company steel structure American Potash & Chemical Corporation at Trona, Calif.

Mr. Pohl states it is his intention to supplement his report on public school buildings later with practical design problems illustrating the practicability of the theories advanced and the benefit resulting therefrom with regard to economy and safety and the method of determining the relative degree of rigidity of various types of framed structures as well as solid panels.

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Construction on Up-swing As Year 1933 Closes

The consecutive monthly gains in construction contracts recorded since July, 1933, were continued into December quite ignoring the seasonal tendencies customary during the period. The contract total reported in December by F. W. Dodge Corporation covering the 37 Eastern States amounted to $207,209,500; this was an increase of approximately 28 per cent over the November total which itself registered a gain of almost 12 per cent over October. In fact the total for the final month of 1933 was larger than that recorded for any other month since October, 1931, and was more than 2½ times as large as the contract volume recorded for December, 1932.

Of the December contract total $155,862,-800 was for publicly-financed construction while the remaining total of $51,346,700 was for privately-financed undertakings. Publicly-financed construction contracts during December were almost nine times as large as the total for this class of work shown during April when such construction contracts were at their lowest point. Privately-financed contracts let during December were higher