BEHAVIOUR OF A STEEL PLANT UNDER MAJOR EARTHQUAKES

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S Y N O P S I S

This paper refers to the effects of the earthquakes of May 21st and 22nd, 1960, on the installations and facilities of an integrated Steel Plant owned and operated by Compañía de Acero del Pacífico, CAP, at Huachipato, San Vicente Bay, 10 miles West of Concepción, Chile.

Design bases used in 1946-1950 are presented, and the structural behaviour of major installations reviewed. Comments are offered on the effects of the earthquakes upon the operations and emergency measures taken to overcome them.

Time has not yet allowed the undertaking of detailed theoretical investigation of observed cases of failure or success, and no attempt is made in this paper to dwell upon such subjects. Nevertheless some preliminary practical recommendations are offered.

2.- S O I L C O N D I T I O N S

The Huachipato Steel Plant is located in the province of Concepción, Chile, a very active seismic zone. See Fig. 1. The city of Concepción, 10 miles away from the Plant, has been destroyed or seriously damaged several times in the last 400 years. The tabulation at the end of this paragraph lists the major earthquakes that have affected the zone from 1570 to date.

The Plant is located on a reasonably compacted sand filling. A typical boring sample is described below:

65'	Disintegrated rock, medium grain-sized, greenish-gray.
55 - 65'	Argillous silt, with shells, greenish-gray
41 - 55'	Dark gray silt.
0 - 41'	Black sand.

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It has been suggested that a slip downwards took place at the end of the quaternary period of at least 120', as indicated by the maximum boring depth at which fundamental rock was found.

The last recorded slippage was produced during the earthquake of January 1939 (no records for 1960 are available). The Concepción-Talcahuano plain descended 24 in. while from 2,4 miles to the East of Concepción ground level raised 38 in., representing a total differential of about 5 ft. No changes relative to the sea level could be recorded due to the ladk of previous data.

RECORDED MAJOR EARTHQUAKES IN THE CONCEPCION AREA

	DATE		EPICENTER	DESCRIPTION
1	Feb.8,1570	9 hrs	Concepción(?)	Earthquake & tidal wave.Very high destruction.
2	Mar.15,1675	19 "	Concepción(?)	No data.
3	170?		No data	No data.
4	Jul. 8,1730	4 "	Santiago (?)	Earthquake & tidal wave. that destroyed 200 buildings at Penco.
5	May 25,1751	1-2"	Concepción(?)	Tidal wave destroyed city of Penco, that was moved to present location of Concepción.
6	Dec.24,1832	18 "	No data	Earthquake, little destruction.
7	Feb.20,1835	11^{30}"	Chillán (?)	Earthquake & tidal wave,called "la ruina", caused total destruction.
8	Aug.13,1868	11^{30}"	No data	Earthquake & tidal wave, little destruction.
9	Jul.22,1898	10 "	Concepción(?)	Earthquake, little destruction.
10	Jun.13,1907	4 "	Concepción(?)	Earthquake, little destruction.
11	Jan.24,1939	23^{30}"	Chillán	Earthquake & tidal wave, great destruction,over 20.000 deaths
12	May 6, 1953	13^{15}"	Angol	Earthquake, little destruction.
13	May 21,1960	6^{05}"	Concepción	Earthquake, great destruction.
14	May 22,1960	15^{15}"	Valdivia , Chiloé	Earthquake & tidal wave, great destruction.

3.- PLANT DESCRIPTION AND EFFECT ON OPERATIONS

The Huachipato Plant of Compañía de Acero del Pacífico is an integrated Steel Plant, built in 1947-1949, located alongside the San Vicente Bay as shown on Fig. 1. It has been continually expanded since its inception and is now producing 450.000 Tons of ingots annually.

The main units are the following:

a). A Pier for ocean going vessels as will be described later on.

b). A system of raw material crushing, screening and storing (ore, limestone, coal, etc.).

c). A Coke Oven battery of 70 ovens to produce the coke requirements of the Blast Furnace.

d). A Blast Furnace of 20' -9" hearth diameter with all auxiliaries like blowers, sintering plant, cast house, etc.

e). A Steel Shop equipped with two Bessemer converter shells, 3 100-Tons Open Hearths and 1 200-Tons Open Hearth.

f). Rolling Mills that include a 32" - 26" Blooming, Billet and Slab Mill line, a 3 Hi-4 Hi reversing Hot Mill Plate and Strip Line, a Merchant Mill for rounds skelp and sections up to 3" high, a 3 Stand Cold Mill for Sheets and Tin Plate with all related auxiliaries like Pickler, Anhealing Furnaces, Temper Mill, Shear Lines, etc., etc.

g). 15.000 M.T. per year Fabricating and Pipe Plants for the manufacture of Steel structures, large diameter pipe and heavy plate work.

h). Service facilities as required by the Steel Plant including Maintenance Shops, Power Station, Laboratories, Water supply system, etc.

The Huachipato Plant did not suffer severe damage to the vital parts of its installations as a result of the major earthquakes of May 21st and 22nd. Nevertheless all operations were suspended immediately upon occurrence of the earthquake of May 21st, and units were placed back in operation as soon as repairs to the damaged auxiliaries permitted.

The iron and steel producing departments and most of the mills were operating normally 5 days after the May 22nd earthquake.

The Huachipato Plant has a well conceived system of services with emergency units such as an elevated storage reservoir for industrial water of 30.000 cu.mt. capacity, a 2.500 KVA steam driven emergency generator, an emergency steam driven Blast Furnace blower and two multifueled Boilers that can burn blast furnace gas, coke oven gas and coke breeze.

The earthquake of May 21st showed that these systems must be improved if absolute safety is desired in case of major earthquakes. The following example illustrates the need for providing one security measure upon another when an absolute guarantee of performance is a must:

As a result of the earthquake, electric power from the Public Power System failed, nevertheless water in the reservoir kept the Blast Furnace cooling system in operation.

However:

A 36" main line broke wide open and drained the reservoir. Within 35 minutes after the shock only a small amount of water was left to feed the furnace. No recirculation, to save water, could be effected at the Blast Furnace since no power was yet available. Power could not be produced in the emergency generator since steam was not available, first due to broken pipes, and subsequently to malfunctioning of the coke breeze aprons. When power finally became available from the Public Power System, one and a half hours after the shock, the electrical lines to the river pump were grounded due to failure of several insulators, and no water could be pumped.

Eventually water was pumped, steam became available for the purging of gas mains, gas was available to keep the coke ovens fired and hot, etc.

However, a delay in the resumption of operations of some of the facilities, by as little as half an hour, would have caused a major disaster.

The above illustrates only one of the many instances where disasters could have occurred. Similar situations were encountered throughout the Plant during the first few hours after the earthquake.

4.- DESIGN STANDARDS

Seismic design standards for the Huachipato Steel Plant were established in 1946 closely following current Chilean trends at the time, predicated on the assumption that danger of resonance existed for periods of around 1.5 sec.

Early during the design, the above criteria were abandoned, and uniform seismic factors of 0.15 g. acting both horizontally and vertically adopted for all structures, excepting highly important units, such as the blast furnace, or special cases of tall vessels or chimneys. The change of standards was brought about by the complexity of the methods as well as by publications from researchers in California and Japan, mainly professors G.W.Housner and Tachu Naito.

Tall vessels and chimneys were designed for seismic factors variable between 0.15 g. and 0.40 g., depending on their natural period. To account for higher modes of vibration, bending moments were increased to a value equal to 1/3 of base moment from mid-height to the top third of the stack, with lineal variation in between.

Unit stresses were adopted following current practices in the United States, namely 20.000 psi for ASTM-A7 structural steel and 950 psi for 3.000 psi concrete. An increase of 33-1/3% was allowed for seismic design. Bearing soil pressure was limited to 4.000 p.s.f. for ordinary and 5.000 p.s.f. for earthquake conditions.

5.- PIER AND UNLOADING FACILITIES

The pier is a structure 890' in length and 70' in width, with a heavy reinforced concrete deck resting on steel H friction piles driven to 60 tons load according to the Engineering News Record formula(Fig.2). Over the pier run a 5,5 M.T. ore unloader and two 66" gage railroad tracks.

Lengthwise the pier was designed as a frame, assuming a 2" horizontal deflection of the deck in relation with the point of fixity of the piles, 52' below as an average. The assumption is equivalent to a seismic factor of 0.008 g. applied on top of this very flexible structure, with a computed period of vibration of 5 sec.

Transversally the structure is rigid, due to the action of two battered piles driven on each row. Seismic design factor is 10% and natural period of vibration 0.17 seconds.

The pier has not suffered in any of the earthquakes of 1953 and 1960, except for the cutting of rails at the approach mole, and damages to some unconsulted piles at the shore end. (Fig. 2).

The earthquake of May 21st, 1960 caused the failure of connections between the main columns and wheel trucks of the ore unloader, as illustrated in Fig. 3 and 4. At the time of the earthquake the tower was not in operation and the bucket had been removed for repairs.

The structure was designed for seismic coefficients of 0,15 g both vertically and horizontally, and 35.000 lbs. concrete counterweights were provided to avoid overturning. Natural period of vibration was computed at 0.33 sec. in a transversal direction.

6.- COKE OVENS

The design of a coke oven battery to withstand earthquake forces was a very challenging engineering problem. The particular nature of the construction of a Coke Ovens battery - a series of parallel slender brick walls forming narrow oven and checker chambers, between pairs of walls - and the expansion requirements characteristic of any furnace, compounded themselves in such a way that it was difficult to apply the known principles of resistance to lateral forces.

Standard design of coke oven batteries considers a larger number of oven chambers (60, 80 and sometimes up to 100) supported at the ends by reinforced concrete pinion walls. Such design obviously could not be used in a plant where seismic forces had to be considered .The substantial cost involved in the installation of a battery of 57 ovens - as the one originally required for the Huachipato Plant - made it mandatory to take every precaution to prevent major damages in case of earthquakes. Fortunately, as standard coke oven design has been the subject of extensive investigation, there is a great deal of information available, particularly as regards adequate mixtures of coal to prevent excessive expansion in the oven chambers which may cause the walls to collapse. Thus, many batteries have been known to collapse due to the expansion of the coal inside the oven chambers. Coking coal experiments include the use of movable wall test ovens where the pressures exerted by the expansion of the coal are measured and recorded at all times during the coking period. Such information as provided by Koppers Co. Inc., Consulting Engineers and Contractors for CAP's coke ovens, was used in the determination of the stability of the oven walls subjected to lateral forces and the conclusion was reached that each wall individually was able to withstand lateral accelerations equal to 0.15 g and even higher.

To assure the lateral stability of the whole system of vertical walls, it was assumed that lateral forces could be carried to the pinion walls by the three pads marked a, b and c (See Fig. 5); however, pad a and b are refractory brick masonry and have 1/2" expansion joints at each oven chamber. This lack of continuity may disappear when the coke battery is hot but there was no certainty that the 1/2" gap would be completely closed down.

There was only one obvious answer to the stability requirements: reduction of the number of ovens between pinion walls. To fix an order of magnitude for this number of ovens, it was necessary to resort to past history on oven failures. A maximum deflection at the top of the brick wall of 5" was reported as having been found in non-damaged walls. Expansion joints of 1/2" could be considered completely open in a cold oven and probably half closed in a new hot oven. A range of 10 to 20 ovens between each pair of pinion walls would therefore appear as a reasonable number that would at least confine the damage to a small number of units. To ensure uniform heating of the batteries and also to facilitate scheduling of the pushing sequence of the ovens, it is usually recommended that the number of ovens be a multiple of 4 plus 1. This reduced the alternates to 13 or 17 ovens per battery. The first, 13 ovens per battery, was finally chosen as the maximum number of ovens that should be included between pinion walls. The required 57 ovens were therefore built as 4 batteries of 13 ovens plus one battery of 5 ovens (See Fig. 6).

Behaviour of the coke oven batteries under the two major earthquakes of May 21st and 22nd was highly satisfactory. Without considering minor damages to steel parts not related with the general structure of the ovens, such as breakage of a steam pipe, displacing of the two gas collecting mains alongside the top of the ovens, etc., the only real damage experienced was the appearance of cracks shown in Fig. 6. These cracks appeared on two pinion walls and are located around the doors. While no exact explanation has been yet developed, there are reasons to believe that they are due to careless detailing of the reinforcing concrete bars around the door openings.

7.- BLAST FURNACE

The Blast Furnace is the heart of a steel plant, It is a very tall, heavy and expensive structure supported by columns (See Fig.7). It is therefore a unit where seismic forces can produce damages of such magnitude that the very life of a steel plant may be impaired.

Here again the standard design of a blast furnace as generally adopted in the U.S.A. and Europe was not able to withstand the earthquake shock. The supporting structure of a blast furnace of standard design consists of a number of wide flange sections arranged radially, connected to a poligonal cast iron bottom ring and a welded steel top ring. Lateral force resistance, therefore, depends only on the rigidity of the connections and is of limited value.

Operating reasons did not permit the design of cross bracing between columns since serious interferences would occur with cooling pipes, cooling plates, etc.

An entirely new structure has to be developed in close cooperation with our Blast Furnace Consulting Engineers, H.A.Brassert and Co. This structure as shown in Fig. 7 is a monolithic welded unit where the bottom ring, columns and top ring are made of box sections with ample flexural and torsional rigidity.

If the energy method developed by Lord Rayleigh is applied to the Blast Furnace structure shown in Fig. 7 a natural period of vibration of 0.6 seconds is obtained. A seismic coefficient of 17 % was applied to the design of the structure and the foundations.

The Blast Furnace of Compañía de Acero del Pacífico, has withstood successfully the three major earthquakes of May 6th, 1953 and May 21st, and 22nd, 1960.

8.- STACKS AND TALL VESSELS

There are 10 tall steel stacks in the Huachipato Plant. Eight of them were built before 1957, applying design standards described in paragraph 3 of this paper. The other two stacks were built after 1957, following design theories developed by professor N.M. Isada on paper presented at the First World Conference on Earthquake Engineering, at San Francisco, California in 1956.

During the May earthquakes the most common failure was buckling of the shell plate, at the second ring over the tapered section, accompanied by vertical cracks in the concrete footings, both on the northern and southern faces (Figs. 9, 10 and 11). The above type of failure was noted on two of the old (pre-1957) and both of the new stacks. Other defects were: failure of anchor bolt connections (in two cases) and opening of the girth weld by tension, opposite a buckle (one case).

It is interesting to note that most failures occurred as a result of the May 21st earthquake. The second shock, on May 22nd, did little additional damage, even though the stacks had their anchor bolts loose, one of them was out of plumb, and buckled shells had not been repaired.

A description of structural properties and damages of some of the stacks is given in Figs. 9 and 10.

Failures of the stacks are attributed to two main couses: inadequate methods of seismic design and unreliability of formulas used to determine buckling strength of the shells.

BLAST FURNACE STOVES. Four hot blast stoves are located south of the blast furnace. The structures are heavy cylindrical steel shells, 94' tall and 21' in diameter, lined and filled with refractory brickwork. The weight of each stove, including brick, is estimated at 2.300.000 lbs. Each one is connected to the concrete foundation by a ring of 48 bolts, 2 inches in diameter (Fig. 12). Three of the stoves are on a common footing, and the other over an independent foundation, but damages were similar on all four. The computed natural period is 0.61 sec. and a seismic coefficient of 0.16 g was used.

The earthquake of May 21st stretched all bolts in a more or less uniform pattern, from 1 3/8" to 2 3/8" and many of them showed signs of lateral contraction (Fig.12). The second earthquake, the following day, worsened the condition and a few bolts were cut.

A rough computation, assuming uniform horizontal acceleration, indicates that static seismic factors of 0.3 g and 0.5 g are necessary to cause yield and rupture of the bolts.

ELEVATED WATER TANKS. Only two rather small elevated water tanks exist in the area. Both are of the same type, with a cylindrical shell supported by a single central pipe column (Fig. 13). One of the tanks is 68' tall, and the other 41', with 4.000 gals capacity. The periods of vibration are 1.9 and 1.2 seconds, and the seismic design coefficient 0.20 g. These flexible structures withstood the seismic shocks well with no damages whatsoever.

9.- B U I L D I N G S

Most of the buildings are steel structures of the classical "Mill type". Spans vary from 40' to 110',80' spans being the most common. Normal bay spacing is 25'. High speed cranes, up to 150 tons are supported on independent columns. Heavy bracing is generally provided at the top and bottom chords and side and end walls. Lateral forces are resisted by the rigid frame formed by the columns and trusses, except in the largest building, the Steel Shop, where lateral forces are resisted entirely by braced towers on both sides (Fig. 14). All columns are supposed to be hinged at the base.

Buildings were designed to resist the following lateral forces: 30 p.s.f. wind, 0.15 g earthquake, and a crane thrust equal to 20% of the load plus the weight of the trolley. Load combinations were: a). Earthquake plus dead load; b). Wind plus dead load; c). Crane thrust plus dead load, and d). 2/3 wind plus crane thrust plus dead load. Not in a single case was the earthquake a governing factor in the design, and its effect amounted, normally, to 55% or 60% of the critical combination.

The behaviour of the buildings was generally satisfactory, none of them collapsed and damage to main members, such as columns, trusses or girders was negligible. The earthquake did cause the failure, however, of a number of secondary members and connections that had been correctly designed according to the assumptions stated above. Main damages observed were:

a). Shearing of rivets on diaphragms between crane girders and main building columns (Fig. 17). This effect was more noticeable in the older buildings and generally happened at places where cranes were stationed at the moment of the shock. Upon examination, the rivets showed indications of fatigue due to operating loads.

b). Buckling of diagonals designed as tension members. (Fig. 15). This type of failure was general, whereas bracing capable of resisting both tension and compression proved undamaged.

c). Failure of bracing at front walls. Bracing was generously provided on front walls, both to furnish lateral supports to columns and to obtain additional strength at low cost. The wall was thus converted into a plane of much highter rigidity than the frames and, because of the effect of roof bracing which acts in a manner similar to a slab, it took earthquake forces for which it had not been designed, Collapse of the diagonals is shown in Fig. 15.

d). Twisting of gusset plates. Gusset plates of the type shown in Fig. 17 were badly bent. During the second earthquake on May 22nd one of the writers had the opportunity of seeing one plate actually bend, due to flexure of both diagonals in a plane perpendicular to their own.

e). Stretching of anchor bolts. Even though columns were supposed to be pin connected at the base, the typical detail shown in Fig.18 has some moment resistance. When this moment was surpassed the bolts yielded and in a few cases were cut.

f). Sliding of column base and bending of anchor bolts. Because of the large size of base plates, they were shipped loose to the field with no connection provided to the column. (Fig. 16). Friction and the bending strength of the bolts were not sufficient to resist horizontal shears, and the columns slid over the base plates as much as 2", deforming the anchor bolts.

g). Failure of brick masonry around steel columns. Brick masonry was tied to steel columns by means of 1/4" steel rods, placed every fifth course, sometimes welded to the steel. This type of joints failed in most cases.

h). Insufficient separation between buildings. In only one instance a steel building knocked against a masonry structure because of insufficient separation. Separation deserves close study during the design, specially between structures of a very different nature.

10.- CONCLUSIONS AND RECOMMENDATIONS

The Steel Plant has a value of US$ 130.000.000 and the earthquakes of May 21st and 22nd, 1960 caused only minor damages estimated at US$ 500.000, less than 1/2 of 1%. Operations were partially started 3 days after the first shock and became almost normal on the sixth day. From these standpoints the seismic resistance of the Huachipato Steel Plant must be considered a success.

An exhaustive study of both operative and structural effects of the earthquake is being made by CAP, with the cooperation of the University of Chile and the Chilean Steel Institute (ICHA), and the results will be published as soon as they are available. It is our earnest hope that others as well as ourselves will benefit from this experience. This report is, therefore, preliminary and the conclusions offered must be considered as such.

A. RECOMMENDATIONS OF AN OPERATING NATURE.-

a). Design of vital service facilities such as water, electrical power, steam, etc. should be the subject of very close scrutiny. Independent emergency or stand-by sources should be provided for each utility with separate distribution systems. Preferably all emergency services should be independent from each other. Thus, water must be made available even when there is no supply of steam and electricity; electrical power is to be furnished even after water and steam facilities break down, etc.

b). The most important of all the services is man power. Properly trained personnel, at all levels, from laborers to managers, must be available at every minute for several hours after the earthquake shock, in order to ensure that proper decisions and actions are taken.

c). A procedure manual with instructions for actions to be taken in various cases of emergencies, seems to be a valuable instrument.

d). Proper maintenance of Industrial Steel Buildings with heavy cranes should include periodical checks of all connections. Buildings damaged during operations are apt to suffer with the earthquake shock. Maintenance should also cover inspection and cleaning of all structural members overhead to avoid the fall of materials, wood planks, etc., left by painters, electricians, etc.

e). Resumption of operation after a major earthquake shock should be allowed only after a check has been made on all structures. Large damages may occur if this recommendation is overlooked.

B. STRUCTURAL RECOMMENDATION.-

a). Current Earthquake Codes cannot be indiscriminately applied to steel structures. The codes have been written for common construction,and are based as much on experience as on theoretical considerations. Industrial steel buildings are clearly not common construction; damping coefficients are low and secondary building elements such as masonry walls and partitions are usually not present. On the other hand, steel has large strength reserves derived from its yielding properties, non-existent in masonry. The writing of a code for steel structures with low damping coefficients and the possibility of yielding is, we believe, a necessity.

b). In earthquake computations, more than in any other, design assumptions should be respected by the engineer and the builder. If a column is designed as hinged, the connection should resemble a hinge as closely as possible; otherwise, it will take moments within the limits of its capacity and might fail before becoming a true articulation. Also, all structural elements incorporated in the construction must have been considered in the design; if not, they might be more harmful than beneficial, as in the cases mentioned of unconsulted piling at the Steel Plant pier or front walls with too many diagonals.

c). The use of tension diagonals should be avoided, unless means are provided to furnish initial stresses.

d). Close attention should be given to details. The possibility of forces acting in any direction must be considered and strength provided accordingly. In Figs. 17 and 18 some recommended details are suggested.

e). Anchor bolts are up to yield, and details should be such that an appreciable length of the bolt is visible, out of the foundation, in order to allow the possibility of tightening and locating failures.

f). Columns should probably be welded or riveted to the base plates, and these anchored to the foundations with ribs or any other suitable means. In any event, base connections must be designed to resist horizontal shears.

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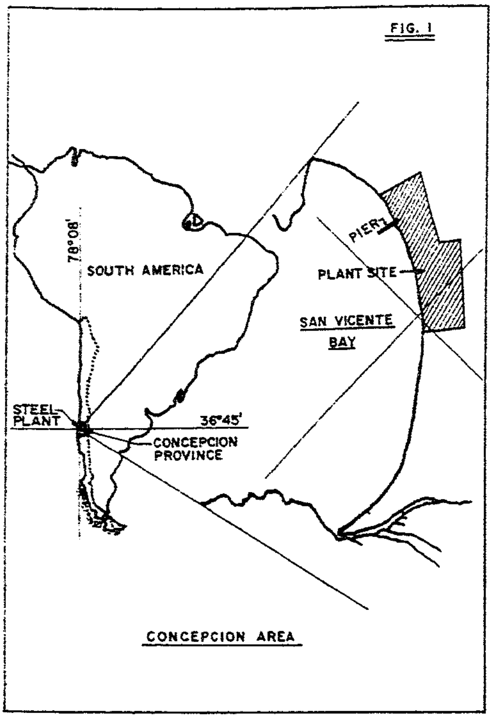

Fig. 1 Concepción Area

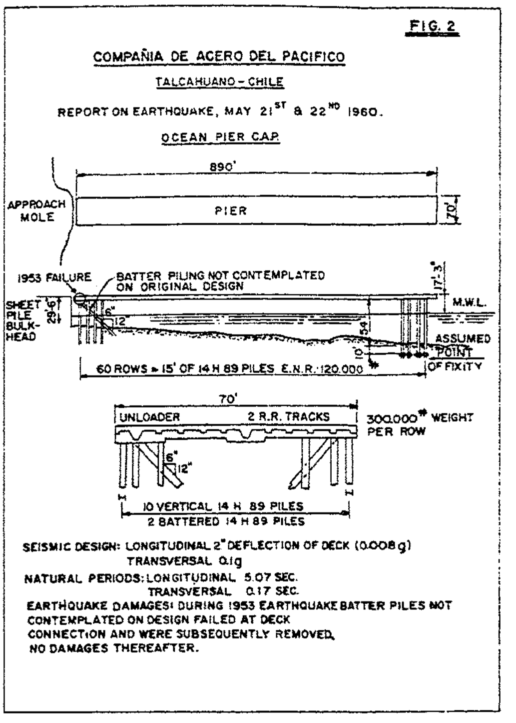

Fig. 2 Pier

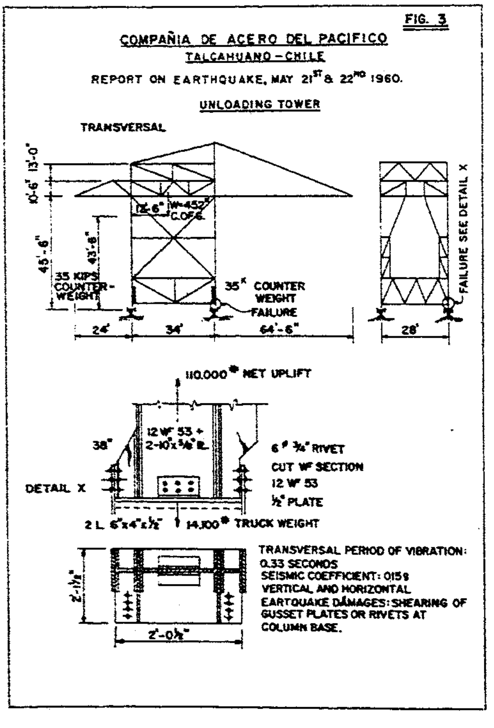

Fig. 3 Unloading Tower

Fig. 4 Unloading Tower - Failure of Column Base

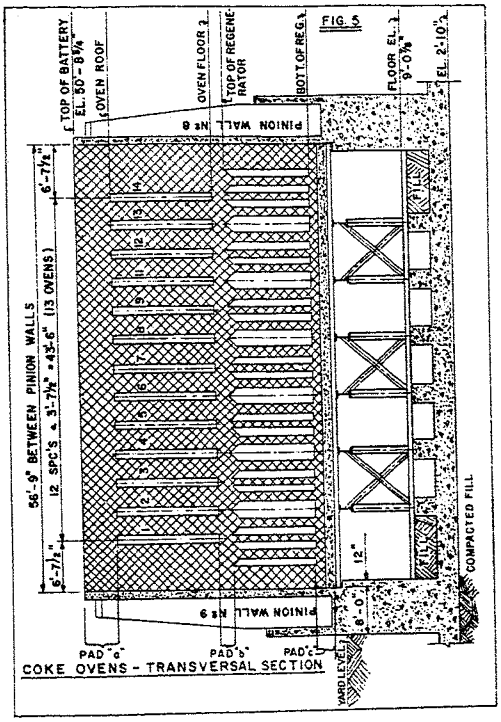

Fig. 5 Coke Ovens - Transversal Section

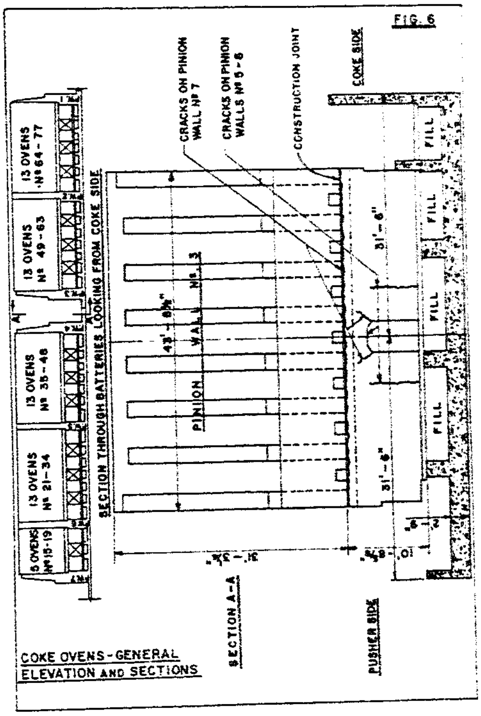

Fig. 6 Coke Ovens - General Elevation and Sections

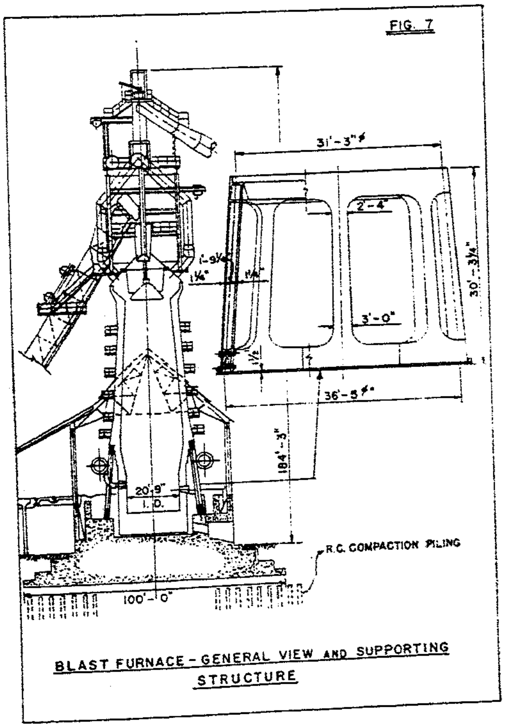

Fig. 7 Blast Furnace - General View and Supporting Structure

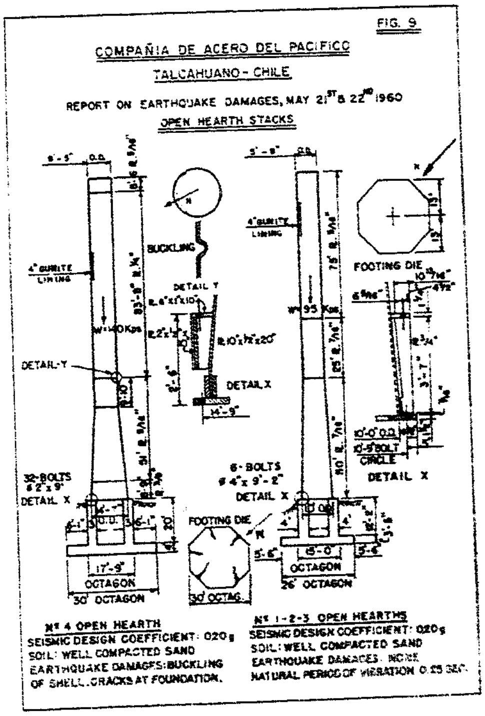

Fig. 9 Open Hearth Stacks

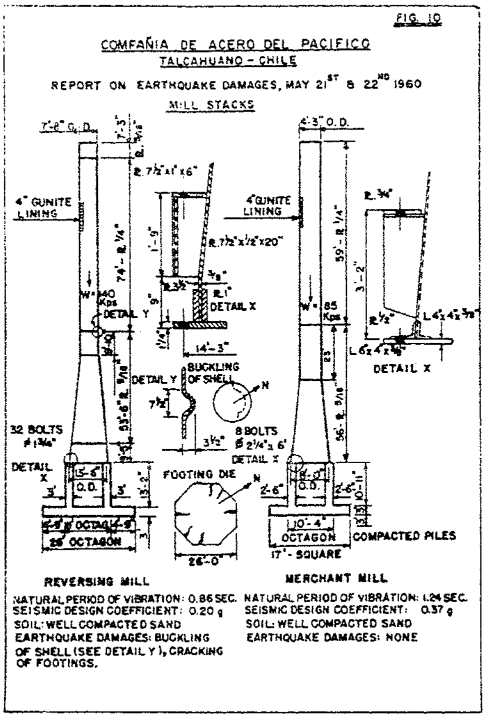

Fig. 10 Mill Stacks

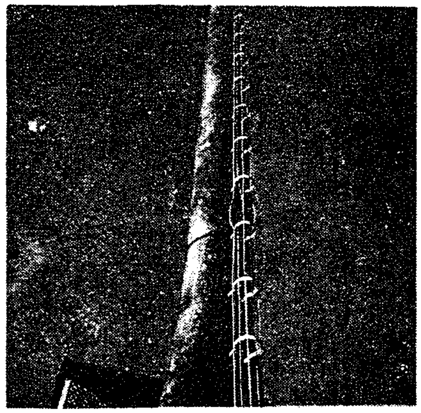

Fig. 11 Buckling of Reversing Mill Stack

Fig. 15 Mill Buildings - Diagonal Bracing

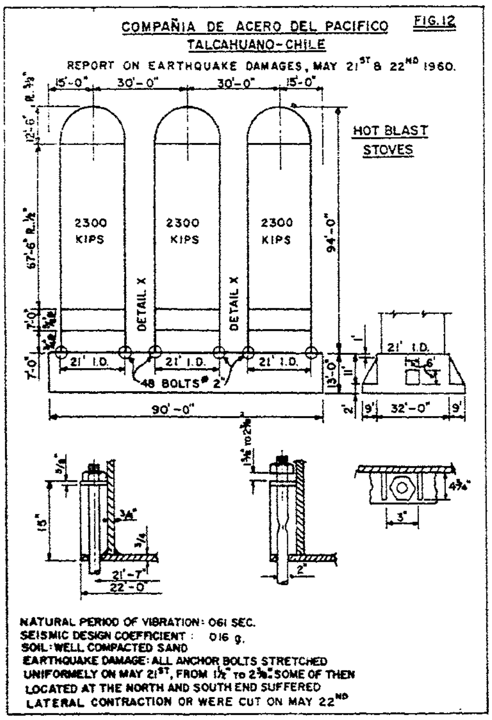

Fig. 12 Hot Blast Stoves

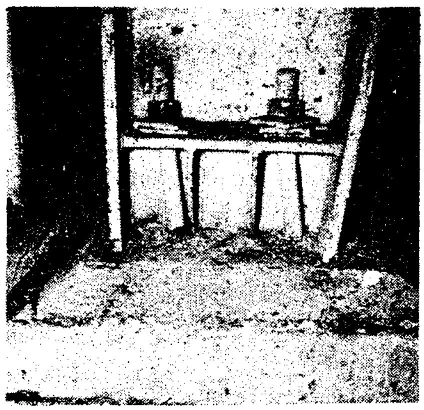

Fig. 16 Mill Buildings - Column Base

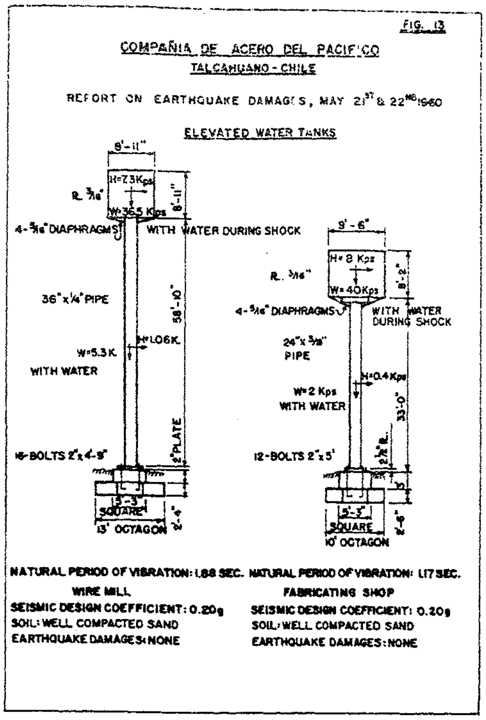

Fig. 13 Water Tanks

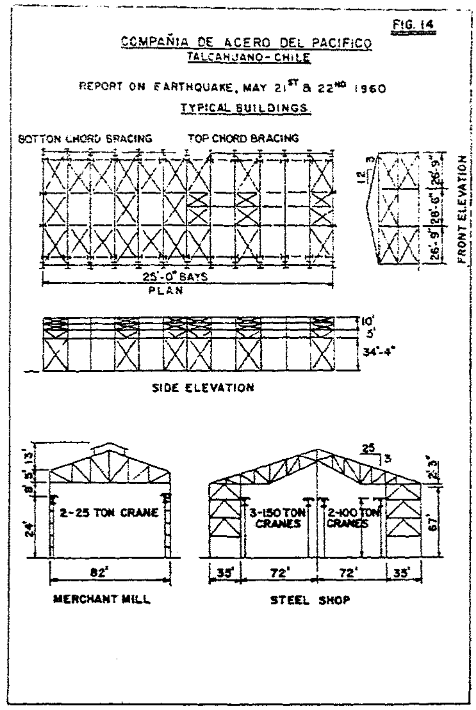

Fig. 14 Typical Buildings

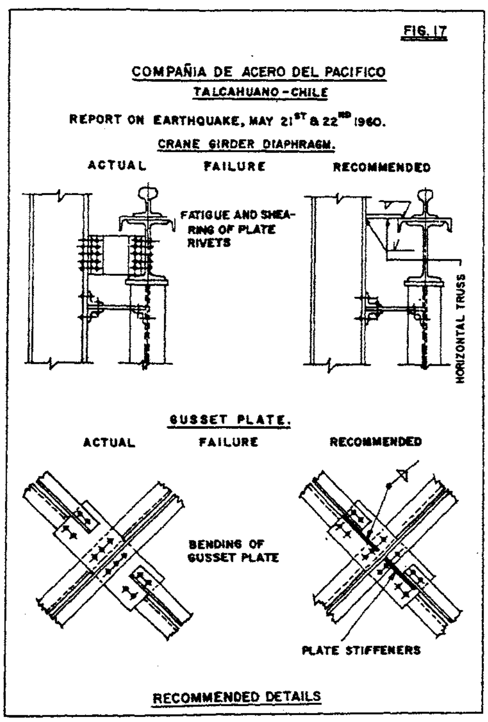

Fig. 17 Recommended Details

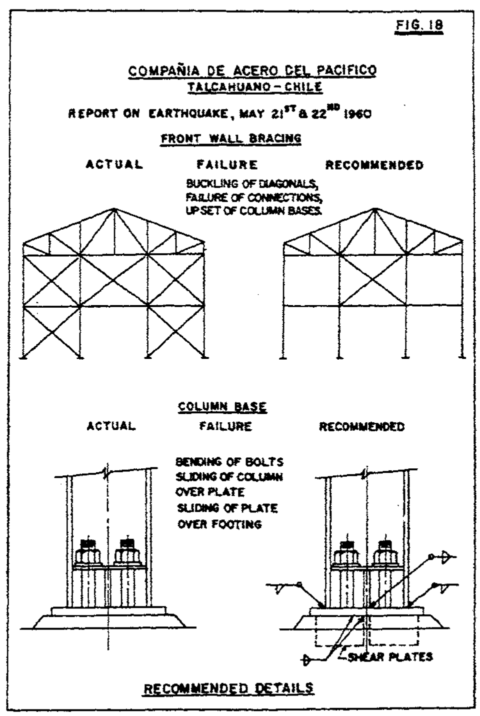

Fig. 18 Recommended Details