ENGINEERING ASPECTS OF THE EARTHQUAKES IN THE MAIPO VALLEY, CHILE, IN 1958

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1. FOREWORD

This paper deals with the effects of the earthquake that took place on September 4th., 1958, and of its foreshocks and aftershocks, on engineering structures and housing in the epicentral zone located in the Maipo Valley, Central Chile.

The above mentioned earthquakes provided an excellent opportunity for examining the damages that severe seismic movements can produce in manmade structures. From the standpoint of damages the strongest shocks occurred on August 23rd., and September 4th.

Special care was taken to obtain information directly from the field and from qualified sources closely related to the design of the structures. It is not our purpose to cover all phases or to give full data, but merely to indicate the most important effects.

Little is known about the seismic history of the district. According to data supplied by the Institute of Geophysics and Seismology of the University of Chile, seismic activity in the Maipo Valley has been recorded in the years 1850, 1870 to 1880, 1883, 1905 and 1947.

On August 23rd., 1958 a series of earthquakes started in the upper Maipo Valley. These shocks reached a crisis on September 4th., 1958 with three earthquakes. They were followed by several months of aftershock activity.

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Two foreshocks were particularly severe, one on August 24th., at 0 hours 24' local time (5 magnitude), and another on August 28th, at 5.30 hours local time (5.5 magnitude). These foreshocks caused some damage in the close vicinity of the epicenter, particularly in the locality of Las Melosas, which had to be vacated of its population. The time of the three main shocks of September 4th. was as follow: 17 hrs. 50', 17 hrs. 52', and 17 hrs. 55' local time with magnitudes 6.9-6.7 and 6.8, respectively.

Instrumental determination of the epicenter (Lomnitz 1960) gave the following data: Lat. 33°50' S.long. 70 10' W. with 5 Km. probable error. The initiation of the first shock was established at 21 hrs. 51 min. 08 sec., universal time. By a similar procedure the fault strike was established as N.10 E. These data generally agree with the distribution of damage in the zone.

2. DESCRIPTION OF THE ZONE

The epicentral zone appears in Fig. 1. The area encircled in Fig. 2 limits the zone where the most severe damage in structures was observed. Near the center of the zone lies the township of El Volcan where maximum intensity was estimated as 9.5 of the modified Mercalli scale.

As shown in the figures the epicentral zone is a mountainous region about 80 Km. distant from Santiago. The rivers Maipo, Colorado, Volcan and Yeso flow in narrow, deep valleys. The economic activities of this district can be subdivided into three points: a) Public utility services, such as the Maitenes, Volcan and Queltehues hydroelectric plants, and the aqueducts which convey water from Laguna Negra to Santiago; b) Mining activities chiefly related to copper and gypsum (Mercedes, Merceditas, Yeseras and other mines; c) Industrial plants such as the El Volcan and Romeral gypsum plants; d) Tourism, as represented by the Las Melosas summer colonies, and the ski resorts of Lo Valdes and Lagunillas.

3. DESCRIPTION OF DAMAGES

For the purposes of the present paper, damage has been classified as follows:

3.1. Damage by landslides.
3.2 Damage by avalanches.
3.3 Damage by earthquake waves.

Apart from the description of earthquake effects due to the three causes listed above, we shall attempt to give a brief account of the repair work, particularly in the Queltehues hydroelectric plant where service was reassumed after an interruption of only 180 days.

3.1. Damage from slides.

Under this heading we include damages caused by 3.11) large landslides; 3.12) rock sliding, particularly in the "Porphyritic Formation"; 3.13) differential settling of unconsolidated mountain slopes.

3.11. Landslides.

Several spectacular landslides occurred as a consequence of the earthquakes. In the Yeso Valley, about 10 Km. upstream from the Maipo River confluence, a slide of the order of 15 to 20 million cubic meters blocked the river course and formed a small lake which after a few days began to spill over the slide material. No major damage was produced save the interruption of a dirt road along the left bank of the river.

In the Maipo Valley, about 2 Km. downstream from the intake which supplies water to the Queltehues Hydroelectric plant, another large slide having a volume of over 4.000.000 cubic meters took place. The slide dimensions were approximately 500 x 400 meters and its maximum vertical displacement was about 80 meters. (Fig. 3). The slide occurred in débris material grading into alluvial terraces at the place called El Manzanito. Displacement at the surface of rupture reached about 100 m. A telephone line located lower down suffered an approximate displacement of 40 m., and the canal which services the Queltehues Hydroelectric plant and crossed the toe of the slide was displaced about 3 m. to the West and raised 4 m. above its original elevation. Both the channel and the road to the canal intake were destroyed for a length of over 500 m. A small steel bridge spanning the canal was similarly ruined and displaced 3 m. up and 3 m. horizontally from its original position.

Engineering studies for the repair of this section consulted various possibilities. The final solution consisted in a welded steel syphon 3.25 m. ø x 10 mm. thick x 430 m. long and weighing about 400 tons. (Fig.4).

The tube has a protective coating consisting of succesive layers of tar, glass wool and talcum, and it rests on a soil-cement bed reaching to its mid section. The whole structure is covered by a protective fill. This coating will prevent damage in the tubing caused by rock slides and, in addition, it will reduce the effects of temperature variations when the syphon is empty. The work was completed with reinforced concrete inlet and outlet structures. Rain and leakage waters are drained through a valve placed in the bottom of the syphon and an exterior drain system.

Two other large landslides occurred in the Maipo River canal, opposite the locality of Las Melosas. One of these slides with a volume exceeding a million cubic meters completely erased the canal. The reconstruction of an adequate bed in the ruined section required moving about 200.000 cubic meters of débris and laying about 100 m. of welded steel piping 290 m. ø 10 mm. thick. This new tubing was connected to the old canal by means of adequate reinforced concrete structures and sealed joints.

This work was completed with a series of later additions such as drains, filling of faults and fissures and repair of slopes to increase slide resistance and prevent big slides during thaws.

Opposite the Sonámbula creek a considerable flow of mud and stones completely covered a 100 m. section of the canal.

3.12. Rock Slides in the Porphyritic Formation.

The damage suffered by the river intake, the settling basins and Tunnel N° 1 of the Maipo Canal can be ascribed to this cause.

Maipo Canal Intake. (Fig. 5, 6). The intake comprises essentially a dam of gravity type which conveys water from the Maipo river to the canal that services the Queltehues plant, and several bays provided with gates for the control of the flow. Lateral compression by the earthquakes reduced in about 50 cm. the width of whole structure, causing numerous cracks at an angle of 45° in the masonry elements, and horizontal cracks in the piers which support the gates. These horizontal cracks usually coincide with construction joints. The steel beams and girders of the superstructure failed in buckling. Most of this damage appears to be caused by differential motion between the dam founded on sedimentary soil and the piers founded on rock and solidarizing, therefore with the rocky eastern slope. While appearances might indicate an eastward slide of the sedimentary layers on which the dam is founded, geological evidence shows that the eastern rocky bank of the river broke away from the mountain and advanced towards the river bed compressing the intake in such a way that its primitive width was reduced by approximately 50 cm. This was proved by huge crevices appearing some scores of meters above the works level. The western bank of the river did not experience any damage.

The repair of the structure consisted mainly in filling cracks, eliminating leaking joints, and grouting. A reinforced concrete gate pier (90 cu. mts.) was rebuilt. Fig. 7. The lower portion of these piers which had experienced abrasion by stones, gravel and sand was lined with 12 mm. thick steel plates and the spillway apron was lined with a layer of graded rock.

Settling Basins. Two elongated stone masonry settling basins in series, with their longest axis pointing towards the channel were damaged by rock slides. The inlet canal is close to the Valley and the outlet canal is adjacent to the mountain. Spanning this canal there is a series of steel bridges for access of the valves.

The eastern wall of the outlet section was pushed westwards and some steel passageways giving access to the gates failed in buckling. Greater damage was to be found in the downstream direction of the Settling Basins, apparently owing to the fact that the upstream section is founded upon alluvial ground which through damping effect, absorbed most of the compressive forces in this area.

Typical tension cracks were found in the stone masonry, and the concrete bottom of the second Settling Basin was broken by foundation pressure in an upward direction.

N° 1 Tunnel. This 300 mts. long tunnel traverses the Porphyritic formation through what appears to be a fault zone. The poor quality of the ground dictated the use of a lining in the form of an arched concrete floor, with vertical walls of concrete blocks supporting a roof structure formed by reinforced concrete T beams. Most of the damage consisted of failure of the concrete blocks under squeezing and consequent flexural cracks in the walls. (Fig. 8). The roof structure resisted well the vertical stresses which caused the T beams to develop their ultimate strength, with flowing in the reinforcing bars in many places. The work of repair was complicated by the fact that the change in sections downstream required lowering the tunnel to a depth that in some parts amounted to 1,4 m.

The repair program was particularly difficult and had to face the following problems:

a) Relatively unknown acting forces; b) lower flexural resistance of concrete-block walls, and c) necessity of performing the work in the shortest possible time.

After weighing diverse solutions among which we can cite: reconstruction of both walls in reinforced concrete using a double system of stiffening steel frames; transformation of the original rectangular section into a curved shape inscribed in the distorted section, etc., it was decided to reinforce the complete tunnel by means of a rigid steel frame formed by 30 Km. of Decauville rails spaced about 60 cm. apart and filled with concrete in the gaps between these steel frames and the original tunnel section (Fig. 9). By this means a strong rigid frame was obtained capable of resisting both the horizontal pressure of the cracked hillside and the vertical forces acting on the present system of beams. The reactions of the steel frame are transferred in the lower end through its anchor to the new tunnel floor and in the upper end to the riverside wall which suffered less damage.

Operations were planned according the following stages: a) Total lagging of the tunnel for safety purposes, with frames spaced every 0,6 m., 1,2 m. to 2,4 m., depending upon conditions in the tunnel; b) Alternate excavation of sections between frames to new floor elevation; c) Placement of steel frames in excavated sections; d) Concrete pouring in floors and walls to the elevation of the original floor; e) Withdrawal of timbering and placing of underpinning supported on frames already placed; f) Excavation of second portions; g) Placement of remaining steel frames; h) Concrete pouring between original hillside wall, beams and steel frames.

This last stage completes the strengthening of the tunnel. The concrete was pumped in place at a rate of 15 cu. mts. per hour to a mean distance of 250 mts.

3.13. Differential Settling of Slope Material.

Changes in elevation in the canal floors. This effect which at first glance had not been apparent, was one of the most costly and lengthy to repair. Levelling of the canal floor after the earthquake showed substantial deviations from the original elevations. The upheavals and subsidences together with the changes in the hydraulic axis wherever the canal was replaced by steel pipe lines, resulted in a total lowering amounting to up to 2 mts. in the elevation of the computed canal floor compared to the existing floor.

Forebay Basin of the Volcan Plant. This stone masonry forebay has a rated capacity of 1.700 cu. mt. and a total length of 40 m. The masonry was appreciably damaged both outside and inside where it lines the rocky slope. In addition, the entire forebay settled in a downslope direction causing the stone wall to fail by flexure through an action similar to that of a cantilever beam. (Fig. 10).

Penstock of the Volcan Plant. The penstock with a total hydraulic head of 180 mts. is 365 mts. long. It lies on concrete piers spaced 6 m. apart, and at each of the 5 points of slope changes, cyclopean concrete anchors are provided. The sliding of the mountain slopes resulted in a total reduction of 65 cm. in pipe line length. The relative displacements were easily checked by measuring the unpainted length of tubing adjacent to the piers, and it was found that these displacements decreased from the forebay towards the power house. However, the penstock itself was not damaged because the actual reduction was absorbed by the four expansion joints. The total change in elevation between the forebay and the power house is estimated as 31 cm. Both this damage and that mentioned in connection with the Forebay Basin were caused by the same sliding.

3.2. Fallen Rocks and Avalanches.

This type of damage was very extensive and was caused by rocks of all sizes up to several tons.

Volcan River Hydroelectric Canal. This canal flows through a covered concrete section for about one half of its actual length, i.e., 6 Km. The typical damage caused by rocks consisted of shattering and perforation of the upper reinforced concrete slab as well as flexural cracks in the masonry walls. Near the forebay, a rock weighing about 2.000 tons fell upon the canal cover.

Maipo Hydroelectric Canal. Opposite the Melosas locality a section of some 400 m. of the canal was destroyed by a major avalanche. The stripping of rocks and débris, and blasting of the ruined sections started immediately. This section was completely rebuilt.

In other sections, particularly near the intake structure and at this structure itself the open canal was clogged by fallen rocks. Part of the gate structure at the intake was similarly damaged.

Maitenes Hydroelectric Canal. The only damage observed in this area was caused by a large boulder that fell on the canal. The escaping waters eroded the canal through 16 mts. of its length before it could be controlled.

Santiago Aqueduct. This aqueduct supplies about 35% of Santiago water requirements (2,5 cu. mts. per second). It is approximately 80 Km. long between the intake at Laguna Lo Encañado (near Laguna Negra) and Santiago. The main structure is a completely covered conduit of non reinforced concrete built in 1914. The concrete conduit is buried to about 60 cm. depth excepting a section where it is a tunnel. The main damage in an extension of about 20 mts. was caused by a huge rock that shattered the aqueduct even though in this section there existed an overburden of about 3 mts. thickness.

Damage to Roads. In each of the valleys of the Maipo, Volcan and Colorado rivers a road runs alongside the river bed. While the Yeso Valley is flanked by a road on each side. Owing to their location at the bottom of deep valleys, all these roads were damaged in many places by fallen rocks and slides.

Bridges. The bridge spanning the upper Maipo river at Los Ratones was completely destroyed by a large rock fall. It was a steel bridge 15 m. long, located at a narrow gorge some 4 Km. south of Melosas. A new bridge was built on this site and no other damage to bridges were noted in the area.

Railroad Tracks. The narrow gauge track between Puente Alto and Volcan was cut in several spots. This railroad normally services diverse points along the valley hauling both passengers and freight. The most serious damage was registered near Romeral due to a big avalanche, where even the rails were sheared off. (Fig. 11). Damage along the line was promptly repaired.

Aerial Tramline. Big rocks destroyed two towers of the tramline used for the transport of mineral from the gypsum deposit to the Volcan Co. plant in the town of the same name.

3.3. Damage Caused by Earthquake Waves.

After the earthquake the effects of large accelerations both horizontal and vertical, were immediately apparent to observers. Apart from the destruction of houses these effects were evidenced by several railway wagons at the Volcan Railway Station that jumped out of the tracks to a distance of up to 30 cm. from their original position. The direction in which the wagons jumped has been studied by seismologists as an auxiliary in a more precise determination of the epicenter and the fault direction.

Another indication was the case of a 5 HP electric motor that sat loosely on a concrete base directly upon the floor and was overturned. Since the center of gravity in this motor is low, this example reveals the existence of strong accelerations.

Seismological observations recorded at Santiago lead to a similar deduction. There is an empirical formula which relates the intensity I in the Mercalli scale to maximum acceleration in c.g.s. units:

$$\log a = \frac{I}{3} - \frac{1}{2}$$

In Santiago, the maximum acceleration observed was a = 0,05 g = 50 c.g.s., with an intensity I estimated as 6,5 which is precisely what the formula gives. Assuming that in Volcan the intensity was I = 9,5

$$\log a = 3{,}2 - 0{,}5 = 2{,}7$$
$$a = 500 \text{ c.g.s.} = 0{,}5 \text{ g}$$

Accelerations of the order of 0,5 g acting simultaneously in a vertical and an horizontal direction could explain the overturning of the electric motor.

Of the various damage that can be typically ascribed to the earthquake vibratory action, only the most important are considered in this paper, such as:

3.31.- Damage to Silos.
3.32.- Damage to steel structures.
3.33.- Damage to bridge abutments and embankments.
3.34.- Damage to the aqueduct.
3.35.- Damage to canals.
3.36.- Damage to buildings.

3.31.- Damage to Silos.

Gypsum storage silos belonging to Compañía Industrial El Volcan.

These comprise two 15,50 m. high x 4,50 m. inner diameter cylindric hoppers united by a common generatrix, each provided with conical discharge funnels and a cylindric supporting mantle with rectangular openings which give access to the funnels. This mantle is founded to a depth of about 5 mts. on a reinforced concrete annular footing 80 cm. width. The earthquake of September 4th. considerably damaged these silos at + 5 mts. elevation (ground level = 0), and at + 0,70 mts. elevation. In addition to these main cracks there is a number of smaller ones throughout the silos height.

The damage that occurred at + 5 mts. elevation consists of a horizontal fissure that cuts through both silos. In the silo built near the mountain (South side) the upper portion was displaced about 1,5 cm. in relation to the lower portion. The fissure which presents a surprisingly smooth fracture surface corresponds, undoubtedly, to a concrete joint during construction. It should be noted that the vertical reinforcement is only 0,2% and that in the fractured section all the reinforcements that serve to hang the funnels are interrupted. Assuming an acceleration of 0,1 g. the static calculation gives a rather low shear stress. The + 0,70 mts. fissure in the cylindric supporting mantle does not exhibit a horizontal fracture surface. The crack is possibly due to weakening of the base section by the openings and to the absence of special reinforcement in these points. The crevice, seen from the inside, presents a series of converging branches. This suggests a shear crack.

Belonging also to the Compañía Industrial El Volcan there is a smaller silo about 8 m. high used in the industrial process. This silo has diverse cracks, mainly one located at 0,40 mts. elevation and another at about 1,50 mts. elevation. It is interesting to note in the latter longitudinal steel rods that have undergone a considerable plastic elongation and suggest an important flexural work in the fracture zone. Undoubtedly the crack corresponds to a construction joint. (Fig. 12).

Silos for Copper Ores. In the Merceditas copper mine concentration plant the silo for the storage of ores was spectacularly damaged. The structure of about 15 m. height and 6 m. diameter experienced a complete shear failure at about 4 m. above ground level. As in the case of Compañía Industrial El Volcan smaller silo the crack corresponds to a construction joint. Several reinforcing bars ruptured by yielding are plainly visible, thus proving the considerable tensile stresses undergone through the oscillations induced by the earthquakes. The upper portion of the silo was displaced and its wall overlapped the lower portion deviating from the vertical. Along the fracture edges the concrete is ground.

3.32.- Damage in Steel Structures.

Steel structure belonging to Compañía Industrial El Volcan (Fig. 13).

This structure comprises 17 steel trusses with 18.20 m. span, 9,90 m. height and 3,60 m. longitudinal spacing between trusses. Each truss was hinged at the base joints and the top joint. The basal hinges rest upon corresponding masonry walls with abutments in the plane of each truss. The roof is of corrugated galvanized iron sheet on timber purlins. The Company used this structure for the storage of gypsum ores.

At first sight the failure of this kind of structure in an earthquake is surprising, considering that wind loads are many times higher than the design load of 10% g required by codes. It must be noted, however, that due to the orientation of the structure relatively to the valley (longitudinal axis of structure parallel to the valley), the entrapped wind could act only in a longitudinal direction. The structure failed through rupture of the rigid joint defined by the vertical and the slope. (Fig.14). The collapse was determined by the faulty design of the joint which did not contemplate continuity in the bars of the bottom chord in the change of direction, Moreover, the absence of bracing in the bottom chord left the structure with a very low safety factor even for design loads. In addition, welding was of poor quality, showing insufficient penetration, porosity and discontinuities. It should be noted that this structure was not designed by a structural engineer.

3.33 Damage in bridge abutments and embankments.

Damage of this kind was found in the following bridges: railway bridge over the Yeso river, bridge over the Volcan river at Volcan, road bridge over the Yeso river, and bridge over the penstocks at Queltehues. This damage was relatively unimportant. The damage observed in the last three bridges mentioned show the appreciable increase in earth pressure exerted on abutments by seismic action in a confined soil.

3.34 Damage in the Aqueduct.

We must include in this classification the numerous longitudinal, transverse and sloping cracks produced both in the aqueduct and the manholes that represent a menace to the stability of this structure in the event of a new seismic movement. The cracked aqueduct zone covers a length of 8 Km. with its approximate center at the mean point between San Gabriel and Laguna Negra or almost opposite to the Yeso river slide.

The above mentioned cracks are complete fissures that generally cut through the concrete wall. It is interesting to note that in some sections the rich mortar coating maintains the continuity of the aqueduct, while the concrete wall is completely destroyed. It must be observed that the aqueduct is not reinforced and its resistance to the forces set into play by the seismic movement was poor.

The aqueduct was temporarily repaired to leave it water tight and capable of supplying water to Santiago. The repair was superficial and included chipping the concrete along each fissure for a few centimeters and grouting the fissures with a layer of rich mortar.

3.35 Damage to canal.

Some damage as shown in the Fig. 15 and 16 may be considered typical of the vibrational effect of the ground. The severe effects that the seismic wave can produce when it reaches a vertical or a sloping surface are well known. When the vibration action reaches a sloping face it has been compared to the movement of the last billiard ball in a row when the ball at the opposite end receives a sudden impact. Although this comparison is wanting in precision, it helps us to understand the great landslides and rockfalls that frequently occur on such surfaces by the passage of a seismic wave.

In damages like the present the poor quality of wall linings is felt. In other cases like in open canals, the absence of jointing between the bottom and the walls is also apparent.

3.36 Damage to houses.

Most of the buildings in the epicentral area were one story buildings with highly heterogeneous materials and constructive systems.

The observed structural damage in buildings can be classified as follows:

a) Damage due to instability of structures when, even though the materials are of normal quality, the structure collapses or is destroyed through incapacity to resist seismic forces.

b) Damage due to failure in joints.

c) Damage due to poor quality in the materials,

Regardless of this classification and to provide a simple report of facts, the following list is established in increasing order of seismic resistance, taking only into account the behaviour of the walls. A brief account of the damage in walls is given.

3.361.- Stone masonry houses, mud mortar. Total collapse.

3.362.- Houses with gypsum block walls. Destruction of walls and collapse in most cases.

3.363.- Houses with walls of timber frame construction, filled with gypsum mortar (Railroad station at El Volcan, Fig. 17). The gypsum filling failed in most cases.

3.364.- Houses with walls of timber frame construction filled with mud bricks. The filling was destroyed in most cases.

.365.- Houses with mud brick walls. Many walls collapsed and the remainder were shattered and out of plumb.

3.366.- Houses with rammed earth walls. Failures in corners and walls out of plumb,

3.367.- Houses with walls of timber frame construction filled with concrete. Various damage ranging from cracks in plaster to collapse of fill.

3.368.- Cement mortar stone masonry without horizontal reinforced concrete ties. Various damage ranging from collapse to general cracking of masonry in stone joints.

3.369.- Cement mortar stone masonry with horizontal reinforced concrete ties.

A) Without columns.

Aa.- Houses with walls not restrained transversally at the top. All construction included in this group is one story high. Damage in corners and cracks in walls.

Ab.- Houses with walls restrained transversally at the top. All these constructions are two storeys high and have masonry walls. The floors, partitions and in general the complete second floor structures had sufficient rigidity to prevent deformation by flexure at the top of the first floor walls. Small cracks in corners. Few cracks in the wall panels.

Ac.- Constructions with walls restrained transversally at the top by means of a slab. Longitudinal walls work in shear. The only examples of this type are the staircases of the second floor in the Melosas Summer School Camp buildings (Fig. 18), with walls of 25 cm. thickness. These staircases rested in the first floor upon the walls of an old one store construction. Total collapse of walls (Fig. 19) at the second floor level.

B) With columns.

Ba.- Buildings with walls not restrained transversally at the top. The Queltehues Church presents an example of this type (Fig. 20). In all observed cases the walls had collapsed.

Bb.- Buildings with walls restrained transversally at the top. In every case there were structural elements preventing the transverse displacement of walls. Their behaviour was decidedly better than that of construction included under Ab.

Bc.- Construction with walls restrained transversally at the top by means of a slab. Longitudinal walls work in shear. There exist only two examples of this system of construction but unfortunately neither of them corresponds to normal conditions. One of these examples is found in the central two storey body of the Melosas Summer School Camp building. Like the upper portion of the staircases, the second floor of this building is partly constructed upon a one storey structure. As a result of the staircases collapse at the second floor level these walls had to absorb all the horizontal forces transmitted by the roof slab, resulting in shear failure (Fig. 21).

3.3610. Structures of hollow timber walls sheathed with light material. These constructions are formed by posts, lintels and diagonal members of timber sheathed with overlapping or dovetailed boards, asbestos cement boards, cardboard and gypsum boards, or plastered metal lattice, leaving an empty space between both faces. These constructions did not experience damage excepting a few cracks in the sheathing.

Notice should also be taken of specific damage observed in concrete reinforcements like those due to insufficient overlap of bars in joints of reinforced concrete tie beams which were severed by tensile stresses, and to the frequent failures in column to tie beam connections from which a part of the concrete was expelled by the reiterated motion of steel hooks.

We must also include, although abstaining from discussion thereof, other typical damages observed:

a) General foundation damage.
b) Fall of loose structure due to absence of linking to walls.
c) Fall of gypsum board ceilings.
d) Collapse of tile roofs.
e) Fall of chimney stacks.
f) Loosening of ceramic bathroom fixtures.

4. CONCLUSIONS

In the course of this exposition we have indicated the decisive cause or factor of the damages observed.

Damage by landslide.- Much of the damage was unavoidable. Even with the help of geological science it was not easy to anticipate the movement of huge blocks in the porphyritic formation which squeezed the valley and shattered the intake and the settling basins.

In connection with landslides, it must be borne in mind that the cone of débris at El Manzanito and the relatively unconsolidated alluvial soil, opposite San Simón, left no alternative for the passage of the canal. While landslides are frequent phenomena in the history of engineering and measures for their prevention can and must be taken when their avoidance means a saving of human lives, we believe that in cases like the present instance it is more economical in the long range to repair the damaged canal or road than to prevent absolutely all possibility of damage by this cause.

Damage by avalanche.- Can be prevented or at least reduced to a great extent and in this sense the most severe damage caused by this earthquake will be a lesson to the future. For example, in a country of narrow and deep valleys like our Andean valleys (unavoidable location for many engineering works) rock slides, even without earthquakes, are most frequent.

The present earthquake left us some lessons on what should be done in these cases. The canal which supplies the Volcan hydroelectric plant cuts the mountain slope along 6 Km. of its course. Actually it was designed against rock slides and it was covered throughout by a reinforced concrete slab. The idea was, naturally, to avoid obstructions in the canal by eventual rock falls. The present earthquake showed that this protection was insufficient because the stones fell from such heights that they perforated the slab, and some were so heavy that the canal masonry walls failed under flexure. In the future it will be necessary to give attention to slopes above canals and aqueducts (even when they are sunk several meters underground as in the case of the Laguna Negra aqueduct) so that rocks may not fall on the work but would follow their course downhill.

Damage by earthquake wave.- The damage that we have classed as due to the vibratory action of the earthquake is nearly all related to an inefficient construction practice. We shall abstain from enlarging on this point since in the description of damage we have indicated the determining failure. However, we desire to emphasize a reiterated failure in civil structures; this is a weak construction joint through the omission of adequate measures to bind old and new concrete. This bad construction practice determines weak failure planes whose results have been shown. (Case of the silos).

In the case of buildings, the better behaviour of houses in which the transverse displacement of walls at the top was restrained, has been clearly observed. Likewise, the efficiency of the column tie beam system in masonry walls has been proved, and this casts some doubts upon the practice of suppressing the reinforced concrete columns in low cost constructions.

Some instances of poor behaviour in the column tie system originated in the absence of proper overlap in the reinforcing bars of the horizontal tie beams and, in a large measure, to deficiencies in the column tie beam connection. This suggest an increase of concrete section at joints by haunching the reinforced concrete ties which is constructively easy and economical in comparison with the greater earthquake safety obtained by this system.

Finally we must emphasize the excellent behaviour of houses with hollow timber walls.

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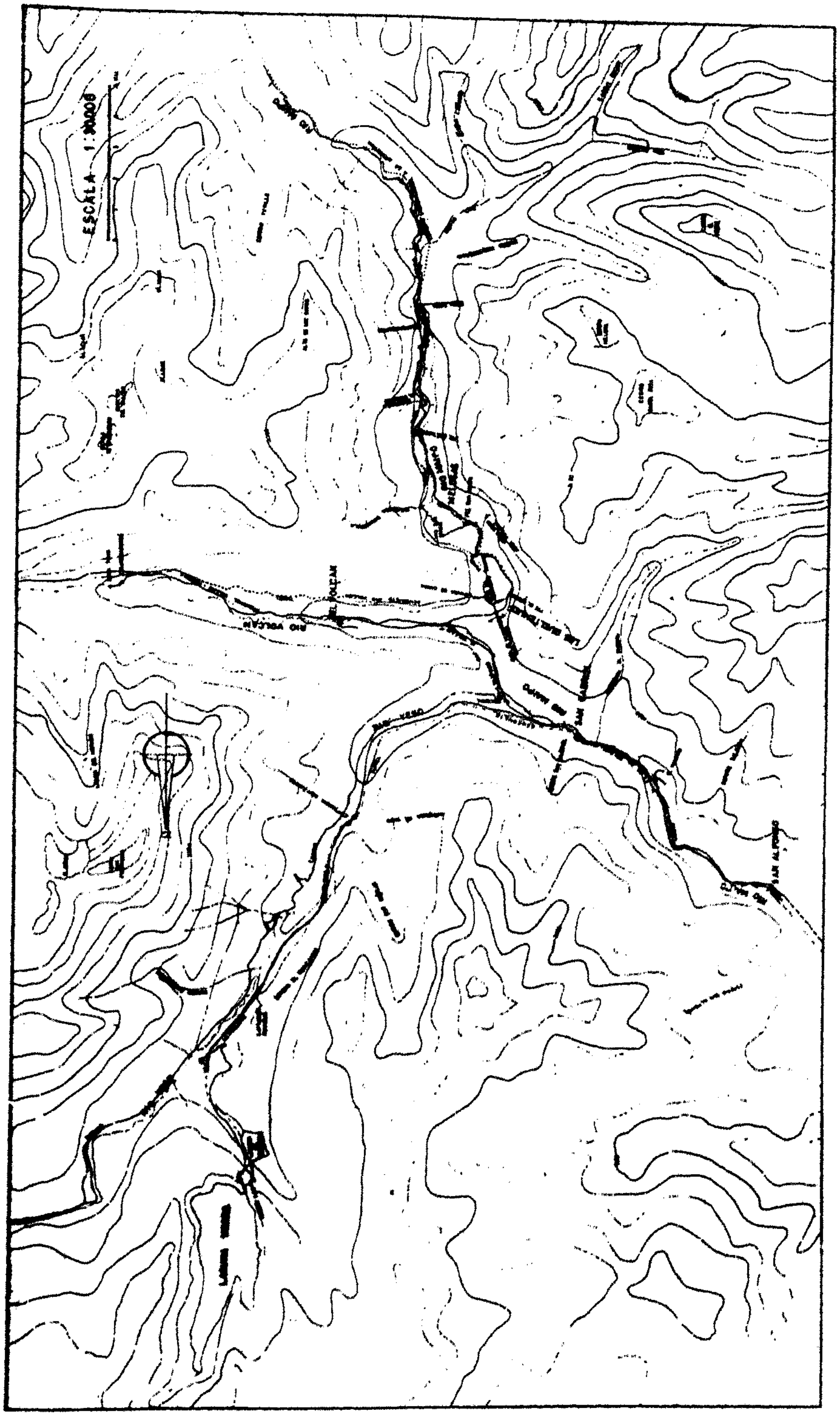


Fig. 1.- Portion of the Maipo Valley most severely affected by the 1958 earthquakes.

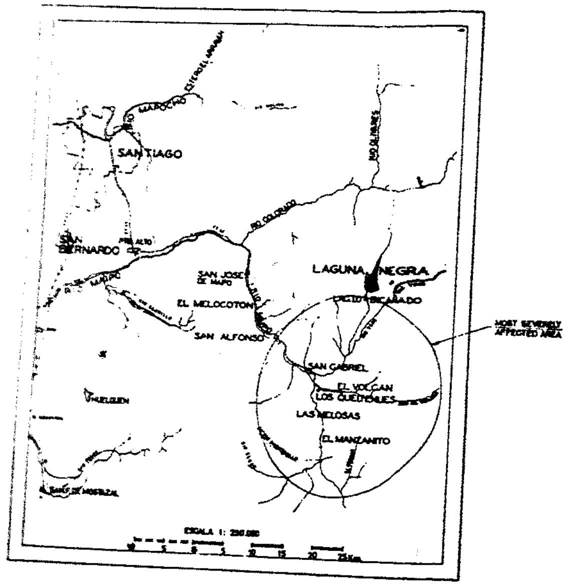


Fig. 2.- Earthquake of 1958. Location map of the most severely affected area.

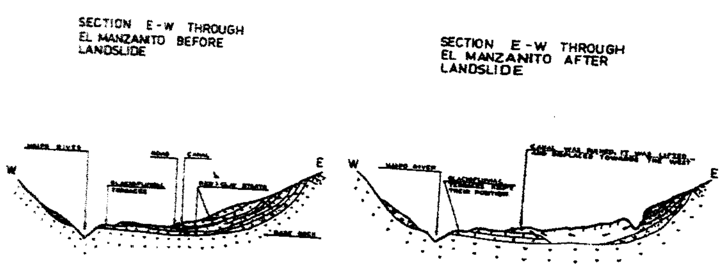


Fig. 3.- Section E-W through El Manzanito before and after landslide.

Fig. 4,- Heavy landslide at El Manzanito. A large part of the fracture surface can be observed. Lower down appears the steel siphon that was built to replace the canal.

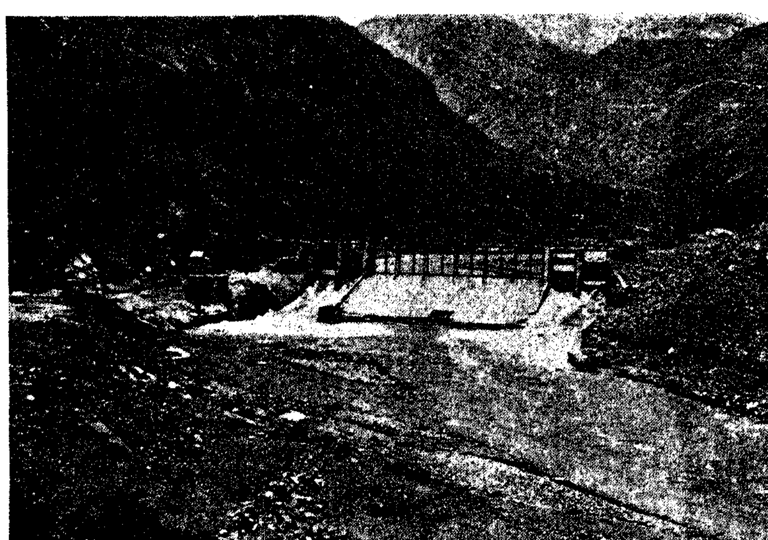


Fig. 5.- Maipo canal intake. Panoramic view. The fracture in the East central pier is seen.

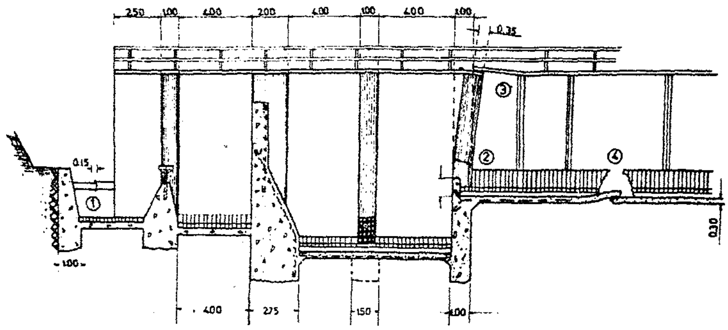


Fig. 6.- Cross section at the intake works on Maipo river.

1.- Fracture and overlapping of small reinforced concrete slab above the canal.
2.- Fracture of reinforced concrete pier.
3.- Fracture and overlapping of the upper R.C. slab.
4.- Fracture and overlapping of the R.C. spillway apron.

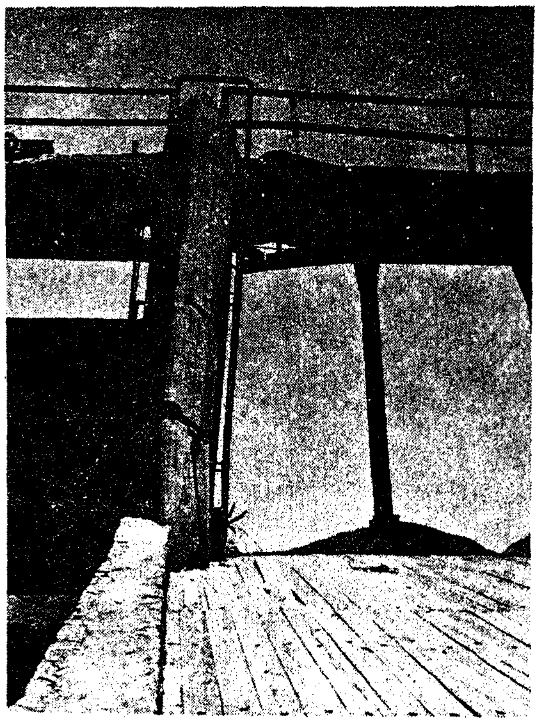


Fig. 7.- Fracture of the pier and buckling of the bridge steel girders.

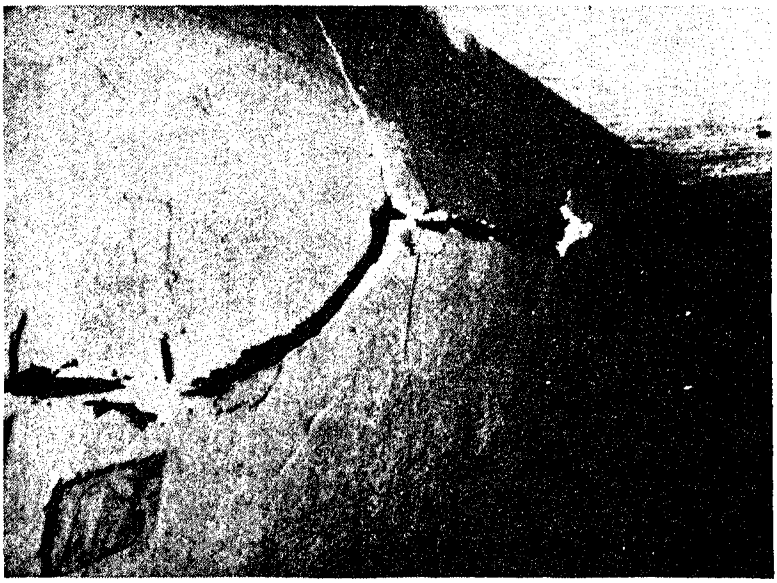


Fig. 8.- N° 1 Tunnel. The concrete blocks of the tunnel eastern wall have failed under rock pressure.

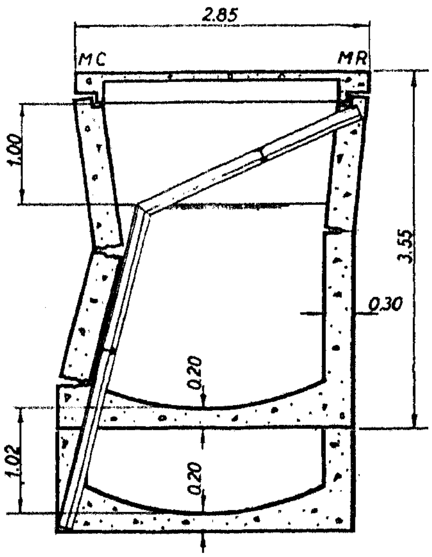
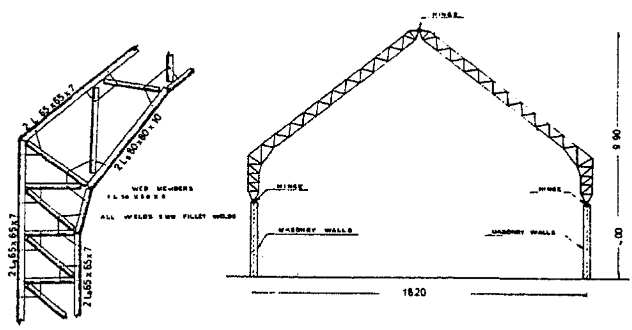


Fig. 9.- Tunnel 1. Maipo Canal - Typical section.

Fig.10.- Forebay basin of the Volcan Plant. The stone wall has failed due to settling of the hill.

Fig.11.- Railroad track destroyed by falling rocks.

Fig. 12.- Volcan Silo. Detail of horizontal crack.



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WELDED STEEL STRUCTURE. CROSS SECTION.

Fig.13.- Welded steel structure. Cross section and details.

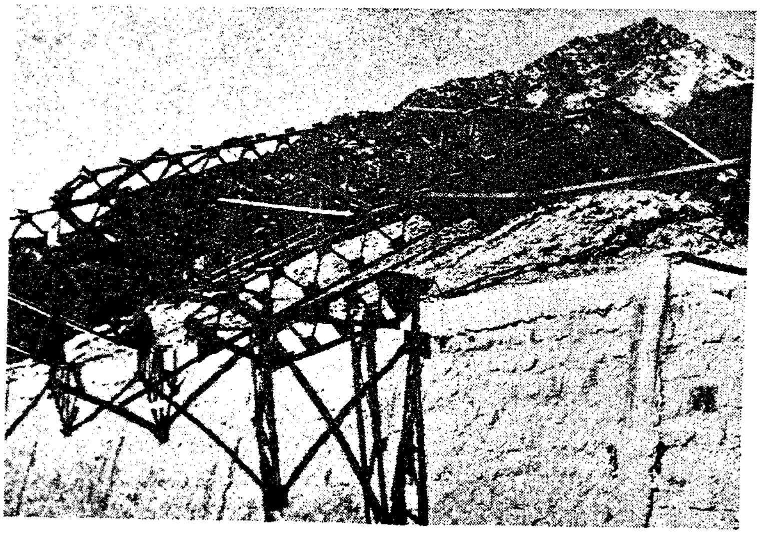

Fig.14.- Steel structure at El Volcan. The galvanized sheet cover has been withdrawn.

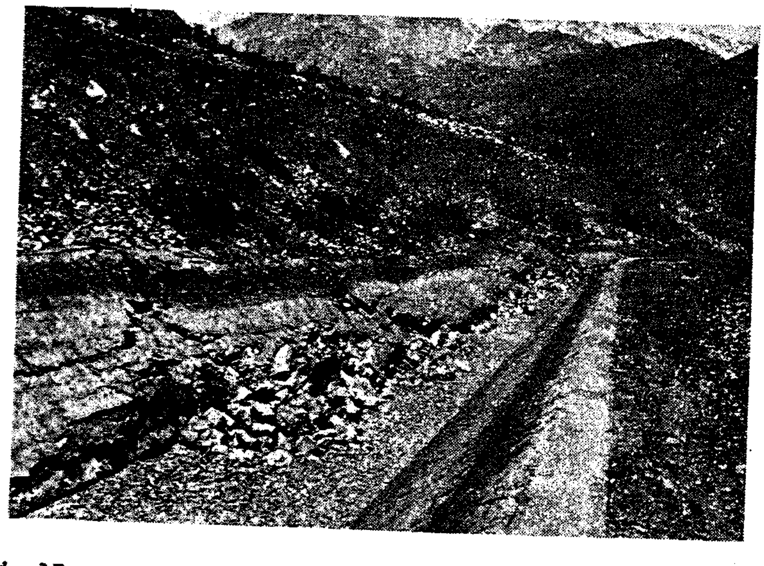

Fig.15.- Maipo canal. Open canal section. Walls lined with stone masonry.

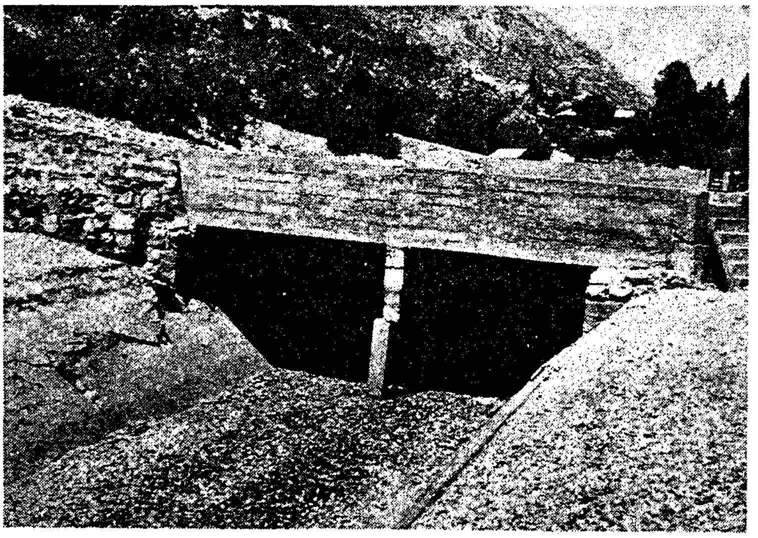

Fig.16.- Maipo canal. Passage over the canal. Shear of the column due to the sudden horizontal movement is seen.

Fig.17.- Railroad station at El Volcan.

Fig.18.- Melosas Summer School Camp. Collapsed staircases.

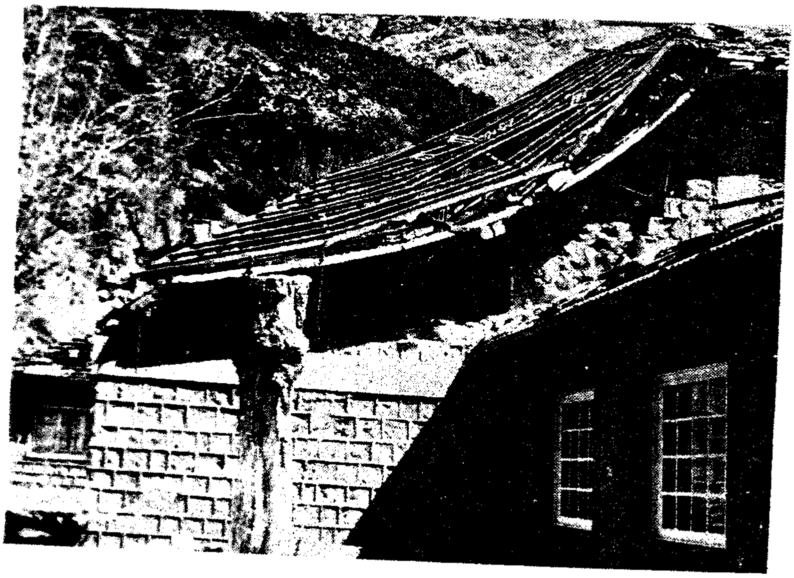

Fig.19.- Melosas Summer School Camp. Collapsed staircases. It shows the concrete slab complete in spite of its large deformation.

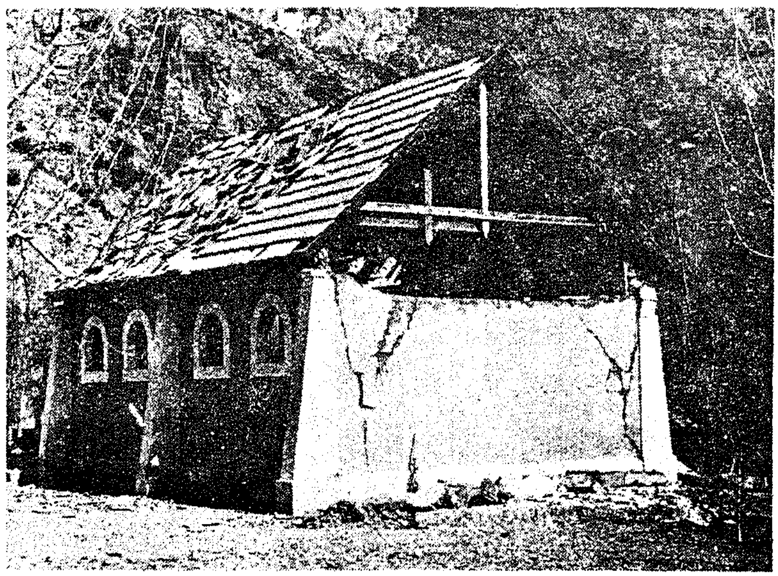

Fig.20.- Queltehues Church. Shows collapsed end wall and cracks caused by movement normal to the wall.

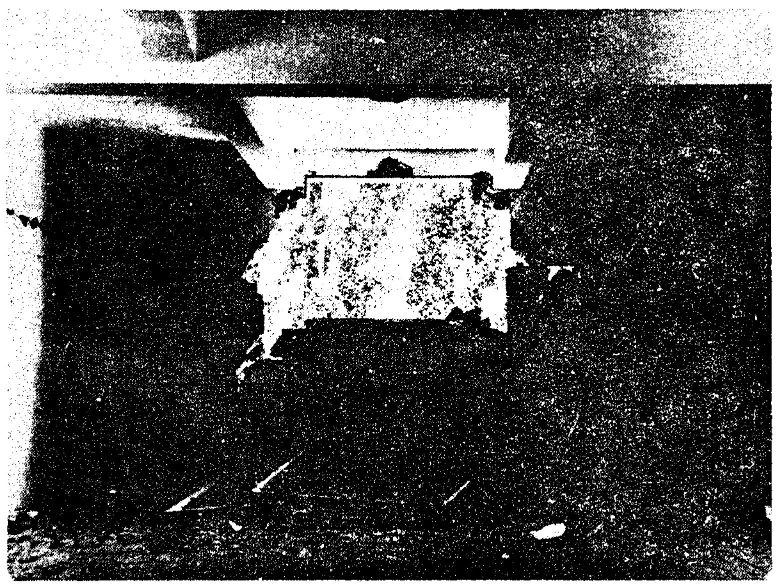

Fig.21.- Melosas Summer School Camp. Second floor wall destroyed by shear stresses.