THE CHILEAN EARTHQUAKE OF 3 MARCH 1985

A FIELD REPORT BY EEFIT

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PREFACE
by SECED-EEFIT Editorial Panel

EEFIT is a non-profit organisation which enables the engineering and scientific communities in the UK and abroad to learn from major destructive earthquakes. The most important link with these communities is established through the publication of field reports. These reports serve many purposes and are unique in conveying the achievements of the particular investigating team within the specific EEFIT set-up. In this respect, the field reports have a long term value as reference material for colleagues sharing the EEFIT interests as well as for future participants to post-earthquake field missions.

The 1985 Chile mission was mounted only six months before the destructive Mexican earthquake, which was also visited by EEFIT; the short interval between the two major earthquakes is primarily responsible for the delayed publication of the Chile report.

Chile has experienced among the largest earthquakes in the world. The 1835 Concepcion earthquake produced spectacular changes to the landscape; this earthquake was experienced and described by Charles Darwin in his Beagle scientific voyage. The 1985 earthquake, although large by European standards, was short of the expected 'big bang' in Chile. Still, the earthquake generated a large epicentral region containing a variety of structures and local geological/topographical conditions. It is unfortunate that the seismotectonic conditions prevailing in Chile have not been adequately exploited in deploying extensive strong motion arrays (synchronised free-field networks, special 3-D arrays, special instrumentation of structures etc.) However, a number of important strong motion records were obtained in 1985.

The EEFIT mission was restricted, due to the adverse logistics involved, to a minimum team of structural engineers for just over a week in the field. The team was very effective in coordinating with Chilean and North American investigators. This coordination is reflected in the overall picture conveyed in the report. Detailed reports from other investigators have already appeared. However, the wealth of information generated by an earthquake, particularly a large one, cannot be contained in any single contribution. In this respect, the EEFIT report has the lasting value of recording the observations of a team from outside the Americas; this 'outside' view of an earthquake scene has proved valuable in the past.

The minimum scope of the EEFIT investigation restricts the report to an appraisal of the post-earthquake conditions, rather than the local implications of earthquake resistance. The report is well presented and contains the salient features of the earthquake that attracted the attention of European earthquake engineers. The observations are often put within the perspective of European standards and practices. Moreover, the EEFIT report poses some interesting questions that have not yet been given a satisfactory answer.

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1.0 INTRODUCTION

1.1 General

On Sunday 3rd March 1985, at 19.47 local time, the central region of Chile (Figure 1.1) was shaken by a major earthquake of surface wave magnitude M_s=7.4 which caused heavy damage to a wide range of structures and left over 170 people dead and 1,000,000 people homeless. Many types of structures were damaged, ranging from adobe buildings to engineered bridges and harbour facilities. One week after the earthquake, the UK-based Earthquake Engineering Field Investigation Team (EEFIT) mounted a two-man, eight-day field mission to the affected zone. This report presents the findings of the EEFIT team.

The EEFIT team consisted of Edmund Booth, a structural engineer with Ove Arup and Partners, London and Colin Taylor, a lecturer in structural dynamics at Bristol University. They were accompanied by David Dowrick, a consulting engineer from New Zealand, who was representing the New Zealand National Society for Earthquake Engineering. All three are specialists in earthquake engineering.

EEFIT spent a total of eight days in Chile, visiting many parts of the affected zone. The team received considerable assistance from the University of Chile, in Santiago, and the Frederico Santa Maria Technical University at Valparaiso which arranged visits to the damaged areas and facilities. In addition, local government officers in Valparaiso provided helpful assistance, giving permission to visit public facilities and providing much detailed information. It was therefore possible to make a good, overall assessment of the damage and to study in more detail a small number of structures. The time available limited the scope of the detailed studies of structures to superficial assessments of the reasons for failure and, equally important, non-failure.

The principal aim of the report is to describe and discuss the observations of damage made by EEFIT team members.

1.2 Contents of the Report

Chapters 2, 3 and 4 provide a brief introduction to the main part of the report, as follows. Chapter 2 outlines the methodology adopted by EEFIT in the field, while Chapter 3 gives a short description of the geography, geology, tectonics, population and economy of the affected region. Chapter 4 briefly describes the seismological aspects of the earthquake.

Chapter 5 contains the main material of the report, and describes the response of buildings and civil engineering structures to the earthquake, as observed by the EEFIT team or reported to them. It includes a short description of the Chilean seismic design code.

The principal conclusions of the team are presented in Chapter 6.

1.3 Background to EEFIT

EEFIT is a group of British earthquake engineers, architects and scientists who seek to collaborate with colleagues in earthquake prone countries in the task of improving the earthquake resistance of traditional and engineered structures.

The principal activity of EEFIT is conducting field investigations following major damaging earthquakes, and reporting to the local and international engineering community on the performance of civil engineering and building structures under seismic loading. It carries out a preliminary reconnaissance mission within a few days of an earthquake. For major European events, it is the intention to have a six-man survey team in the field within a few weeks.

EEFIT was formed in 1982 as a joint venture between universities and industry. It has the support of the Institution of Civil Engineers through its society SECED (the British National Section of the International Association for Earthquake Engineering) and of the Institution of Structural Engineers. It is advised by a number of British engineers experienced in the field of earthquake engineering.

EEFIT members have investigated earthquakes in Italy (1980), Turkey (1983), N. Yemen (1983), Liege (1983), Chile (1985), Mexico (1985), Kalamata (1987) and San Salvador (1987). EEFIT reports have been published on the Liege, Chile, Mexico and San Salvador earthquakes.

2.0 METHODOLOGY

2.1 Principal Aims

The principal aims of the EEFIT team were to make an overall assessment of the damage that was as general as possible in the time available and to study particular failures, with an emphasis on engineered structures. Also of interest were the effects of the earthquake on the local, regional and national infrastructure, its sociological impact and the nature and effectiveness of the relief operations.

2.2 Factors influencing methodology

The following factors, which applied to the situation in the epicentral area immediately after the earthquake, influenced the methodology adopted by the EEFIT team:

- Road access was generally unhindered.

- There were no officially restricted areas, but entry to buildings often needed official help.

- The reactions of the local populace to the EEFIT team were friendly, not hostile.

- EEFIT was fortunate to have the help of Professor R. Saragoni, of the Civil Engineering Department of the University of Chile in Santiago, who arranged access and visits to many sites and provided much background information.

- Although the team members spoke little Spanish, many of the educated Chileans spoke English. In addition, the team was assisted on several days by postgraduate students from Prof. Saragoni's Department who acted as guides and as interpreters.

- Clearing-up was proceeding quickly in all parts of the affected area. Many damaged structures were demolished and sites cleared within 10 days of the event.

- The EEFIT team was operating within a low budget. Useful but expensive facilities, such as the hire of a helicopter, were not available.

2.3 Approach

EEFIT cooperated as much as possible with several other groups investigating the damage. These included Chilean researchers from the University of Chile in Santiago (Prof. Saragoni), a group led by Prof. Bonelli of the Santa Maria Technical University in Valparaiso, the field investigation team from the Earthquake Engineering Research Institute (EERI) in California, and Mr David Dowrick representing the New Zealand National Society for Earthquake Engineering. By sharing information, it was possible to achieve a wide scope to the investigations. The team also held discussions with the British Embassy in Santiago, and provided it with advice on engineering aspects of earthquake relief aid.

The team used little equipment, the major items being a hire car, 3m tape measure, spirit level, compass, large notebook, pencils, good camera equipment and, most importantly, sharp eyes and enquiring minds.

2.4 Brief Travelogue

Day 1 Booth and Dowrick arrive in Santiago

Day 2 Booth and Dowrick by EEFIT hire car to Melipilla and San Antonio.

Day 3 Booth and Dowrick, with EERI team members, by EEFIT hire car to Vina del Mar and Concon.

Day 4 Booth and Dowrick, with Prof. Saragoni's students from University of Chile, to Santiago.

Day 5 Booth and Dowrick, with Prof. Saragoni's minibus party, to Vina. Taylor to Vina with EERI team.

Day 6 Booth and Taylor by EEFIT hire car around Santiago.

Day 7 Booth and Taylor by EEFIT hire car on intensity survey south of Santiago.

Day 8 Booth, Taylor and Dowrick, with Prof. Saragoni's minibus party, to San Antonio.

Day 9 Team depart from Chile.

3.0 THE EARTHQUAKE AFFECTED REGION

3.1 Population and economy

The affected region comprised approximately 40,000 square kilometres in the Central Region of Chile, around the capital Santiago (see Figure 1.1). The affected area was roughly bounded to the north and south by latitudes S32 and S35 respectively, to the east by Santiago and to the west by the Pacific Ocean. This region is the most prosperous in Chile and, in addition to Santiago (population including suburbs c. 4.3 million), encompasses the cities of Vina del Mar and Valparaiso (combined population c. 600,000) and San Antonio (population c. 100,000).

The population covers a wide range of socioeconomic classes, from the wealthy in parts of the major cities, through the reasonably well-off middle and upper working classes, to the poor living in primitive conditions in the countryside. Living standards are similar to those in Southern Europe as is the climate. The earthquake seriously affected all the socioeconomic classes, with the upper working and middle classes, who live in the larger but older masonry and better quality adobe houses which fared badly in the earthquake, perhaps being the worst hit.

The fortitude and warm hospitality of the Chilean people, despite the disaster that had occurred, was remarkable and was in evidence wherever the EEFIT team travelled in the damage zone.

The local economy is largely based on agriculture, with some light and heavy industry, including an oil refinery at Concon and a copper refinery and foundry at Las Ventanas. Valparaiso and San Antonio are the major commercial ports of Chile. Parts of the coast in and around Vina del Mar are popular holiday resorts and the main summer vacation period had finished about a week before the earthquake.

3.2 Topography and Geology

The following brief review is based on Ziel (Refs. 20 and 21) from which further details may be obtained. The topography and geology of the region are dominated by three, north-south trending features (see Figure 3.1). To the west are the Coastal Cordilleras, a range of eroded mountains rising up to 2,000m above sea level, and to the east are the High Cordilleras, forming part of the Andean mountain range, which rise to over 5,200m above sea level. Between the two Cordilleras, at an elevation of 520m around Santiago, is a relatively narrow plain, known as the "Longitudinal Valley". This stretches over 1,100km south from Santiago.

The Coastal Cordilleras consist of a strongly folded and generally metamorphic Pre-Cambrian to Paleozoic basement covered by Cretaceous granites and diorites and younger sedimentary rocks. The High Cordilleras are predominantly sediments and volcanics from the late Jurassic and the Cretaceous. The Longitudinal Valley is a graben-like depression, dating from the Pliocene/Pleistocene to Recent times.

Little information on the thickness of the Quarternary deposits in the Valley appears to be available, but some measurements have shown that Santiago is founded on sediments that are about 500m thick.

3.3 Tectonics and previous earthquakes

Nearly all of Chile's 3,000km length lies on the boundary between the Nazca plate of the South Pacific and the South American continental plate. The boundary marks a subduction zone, where the Nazca plate is thrusting under the South American plate (see Figure 3.1). It is one of the most seismically active regions in the world and forms part of the circum-Pacific "Ring of Fire". A list of large earthquakes in the central region of Chile during the last 400 years is given in Table 3.1; they are plotted in Figure 3.1. It can be seen that on average, a magnitude 8 earthquake or greater (classified as a great event) occurs in the region about once every 25 years.

4.0 SEISMOLOGICAL ASPECTS

4.1 General Description

According to the determination by the International Seismological Centre (ISC) given in their Bulletin and Regional Catalogue (Ref. 12), the principal shock had a surface wave magnitude M_s of 7.4, and a body wave magnitude m_b of 6.0, based on the average of worldwide station reports. There was a strong foreshock about ten seconds earlier, for which ISC gives m_b = 5.9. The National Earthquake Information Centre, Denver, (NEIC) magnitudes for the main event are significantly higher at M_s = 7.8 and m_b = 6.7. According to ISC (R. Adams, private communication) the body wave magnitude of the main event is difficult to determine because of the short time difference from the large foreshock; the NEIC value of m_b = 6.7 is likely to be more reliable for the mainshock than the ISC value.

The main shock occurred at 22h47m07.9s GMT (19:47 local time). ISC gives the epicentre at 33.08S $\pm$ 0.036, 071.72W$\pm$0.044, placing it near the coast just southwest of Valparaiso (see Figure 1.1) and the depth as 36$\pm$3km, which classifies it as a shallow event. The NEIC location is about 20km further West. The foreshock occurred 10 seconds earlier, with its epicentre a few kilometers northwest.

The NEIC fault plane solution (see Ref. 12) indicates thrusting on a reverse fault striking 20 degrees east of south. The centroid moment tensor position given by Harvard (see Ref. 12) will more closely define the centre of energy release. This is on land due south of the epicentre at a distance of 80km (see Figure 1.1). The magnitude of M_s = 7.4 would be consistent with a fault break length of 100 $\pm$ 50km and a displacement of 3.0 $\pm$ 2m (Bonilla et al, Ref. 18).

Based on these data, an approximate probable source area for the main event has been added by EEFIT to Figures 1.1 and 4.1.

A month-long swarm of foreshocks, with magnitudes of up to 5, had been recorded prior to the main event (Ref. 4). An intensive sequence of aftershocks with magnitude up to 7 were recorded (Ref. 4). There were no reports of surface faulting. Small tsunami were recorded with a maximum amplitude of 1.1m at Valparaiso (Ref. 4).

By comparison with previous events listed in Table 3.1 it can be seen that the 1985 earthquake, although large, is not exceptional and is likely to be exceeded in the next few years.

4.2 Strong Motion Records

The event triggered an extensive network of strong motion instruments operated by the Department of Geology and Geophysics (D.G.G) and the Department of Civil Engineering (D.I.C) at the University of Chile, Santiago. Instruments were also kept by the Chilean Nuclear Energy Commission (C.CH.E.N), the Electricity Company (CHILECTRA) and others. Table 4.1, taken from Saragoni (Ref. 1), tabulates the maximum accelerations recorded, and they are plotted on Figure 4.1. The information in Table 4.1 on site geology and instrument location is taken from EERI (Ref. 4). The duration of strong motion exceeding 0.1g was at least 40 seconds (Ref. 4).

The instrument at Llolleo, sited on rock within the source area, recorded the maximum accelerations, namely 0.67g horizontally and 0.86g vertically.

Figure 4.2 (based on Saragoni, Ref. 3) shows the absolute acceleration response spectrum for 5% damping of the N10E horizontal component of ground acceleration measured at Llolleo, plotted over a similar spectrum for ground motions measured at Zacatula during the 1985 Mexican earthquakes. Both recordings were on rock in the epicentral areas and resulted from the same tectonic mechanism - ie. thrust in a subduction zone.

The two recordings have similar frequency content, with peak response in the 2 to 7Hz range, but the Llolleo recording, within the source area of an M_s=7.4 earthquake, has amplitudes about 120% greater than the Zacatula recording, taken within the source area of an M_s=8.2 event.

The presence of very high accelerations some distance from the source area is noteworthy. At Melipilla, some 40km from the source area, horizontal and vertical accelerations of 0.52g and 0.23g respectively were recorded on rock. Large horizontal accelerations were recorded on alluvium at considerably greater distances from source area; for example, 0.34g was recorded at San Fernando to the south, and 0.43g was recorded at San Felipe to the north, both of which are likely to have been 100km from the source area.

4.3 Intensity

Figure 1.1 shows the Modified Mercalli intensities reported by Saragoni (Ref. 1), which are generally consistent with those observed by the EEFIT team. The area of high intensity around San Fernando, and extending south to Curico is noteworthy, and consistent with the accelerations reported above. This area is characterised by deep sediments in the Longitudinal Valley (Section 3.2) which may have amplified the earthquake motions. Damage was observed by the EEFIT team to be greater in the Valley (e.g. in Rengo, Curico and Puquillay Bajo) than on the surrounding slopes. Celebi (Ref. 10) has confirmed the amplification of motions on alluvial deposits, compared with rock, in the epicentral area at Vina del Mar.

The area of low intensities, shown shaded in Figure 1.1., recorded at the north end of the source area around the epicentre is noteworthy and unusual. It is consistent with the ground motion data shown in Figure 4.1. Since it occurred in an area of high population, it is based on a large database and hence can be regarded as reliable.

5.0 RESPONSE OF STRUCTURES TO THE EARTHQUAKE

5.1 Chilean regulations for earthquake resistant design of buildings

It is likely that the most recently constructed buildings comply with the Chilean Provisional Standard for Earthquake Resistant Design of Buildings, introduced in 1972, as detailed in the World List (1984) (Ref. 13). This standard covers all types of buildings and makes specific reference to structures in unreinforced and reinforced masonry, reinforced concrete and steel. It prescribes the types of seismic analysis that may be used, classifies buildings in accordance with their use and structural form and establishes general requirements with respect to structural layout, construction techniques, design of non-structural elements, quality of repairs and instrumentation of buildings. The ultimate base shears provided for in the code for short period buildings of normal importance vary between 8% and 12% of the building weight, depending on structural form.

Many recently constructed buildings performed very well and this may be an indication of the effectiveness of the Standard.

5.2 Reinforced Concrete Housing

The overall impression gained by the EEFIT team of engineered reinforced concrete housing in the areas visited was firstly that the standard of design and construction was generally satisfactory, and secondly that the structure of the great majority performed well. There were, however, some spectacular failures and in a number of cases these could be attributed to the harmful effects of irregular form leading to torsional effects. The extent of damage in at least two cases (Claudio Vicuna Hospital in San Antonio, and Canal Beagle housing development in Vina del Mar), appeared anomalously high, and merit further study.

The reinforced concrete buildings inspected by the EEFIT team are now described, area by area, with particular emphasis on damaged buildings. For a map of the epicentral area, see Figure 1.1. The rupture zone and epicentral distances given in each section are approximate.

5.2.1 San Antonio (MMI VIII; in rupture zone; 50km S of epicentre)

a) The Claudio Vicuna hospital is set on high ground above the harbour to the southern side of the town. It is a 5 storey building, completed in about 1981 with an r.c. frame, and split into two almost symmetrical halves by an expansion joint. The major difference was that the southern half had a basement whereas the northern half, which was very much more severely damaged, did not.

The hospital was evacuated after the earthquake, and was still unoccupied two weeks later, with no plans for reoccupation. Apart from a fascia beam which bridged the separation joint between the two structures, there were no structural collapses. The main structural failures were in the columns. The most dramatic damage was to an external line of columns at 2nd storey level, on the east facade of the northern half (Plate 5.1). All these columns had extensive spalling around their perimeter at mid height suggesting a compression failure mechanism. The concrete cores appeared intact. Figure 5.1 shows the reinforcement based on visual inspection; although the percentage of vertical reinforcement is high (6%) the lack of restraint by links at right angles to most of the main bars means that modern codes would not be satisfied.

The non failures at the hospital were equally interesting and puzzling. One external infill panel at 3rd storey level at the north end of the northern half had fallen out (Plate 5.2). There appeared to have been no positive connection between panel and structural frame, and if the same applied to the other panels, it was surprising they had not also fallen out. There was no buffeting damage across the joint between the two halves of the structure (the joint being about 20mm wide). Some internal shear walls were slightly cracked, but generally non-structural damage appeared slight (Plate 5.3). By eye, the two halves of the building lined through well, though there was possibly some settlement to the north end of the northern half.

b) <u>4 Storey r.c. apartment blocks</u> on the slope below the hospital appeared from the outside to be undamaged, and inhabited, though some damage was reported in one block.

c) <u>Foundation settlements</u> affected a number of r.c. structures in the slopes above the harbour to the north side of the town, where there was evidence of extensive ground movement. Plate 5.4 shows a 2 storey r.c. frame shop where very large settlements of edge and internal columns occurred, though there was little evidence of distress in the r.c. frame, suggesting that the differential settlements were small. There was direct evidence for the mechanism of failure. A similar building next to that pictured was undamaged. An r.c. frame school north of the harbour was also severely damaged and was apparently affected by foundation movements.

d) <u>Undamaged r.c. structures</u> could be found throughout the worst affected areas of San Antonio. For example, the modern Banco de Chile building on flat, possibly made, ground adjacent to the harbour, was unaffected (Plate 5.5) as was an r.c. frame building with block infill on higher, sloping ground (Plate 5.6).

5.2.2 <u>Valparaiso</u> (MM intensity VI to VII; near rupture zone; 10km NE of epicentre)

A brief tour through Valparaiso revealed moderate damage to masonry structures, implying MMI of VI to VII, but none to engineered structures.

According to Professor Patricio Bonelli (Ref. 2), of the Technical University at Valparaiso, there were no serious failures in code designed buildings in Valparaiso, but 3 such buildings suffered some damage;-

a) <u>Sermena</u> A frame building with an eccentric core and strong beam/weak columns. This was a hospital, which was evacuated after the earthquake.

b) <u>Brazil</u> A coupled shear wall with a soft first storey, which suffered shear cracks in the base of the shear wall, and some compression failures in the external column forming the soft first storey.

c) <u>Van Buren</u> A concrete moment resisting frame with brick infill panels. There was no damage to the frame, but serious damage to the infill.

5.2.3 <u>Vina del Mar</u> (Intensity VI to VII; near rupture zone; 20km NE of epicentre)

Vina is a resort town immediately north of Valparaiso. There has been a great deal of high quality, high rise development in the last twenty five years, especially around the waterfront, and most of this survived the earthquake well. The damaged structures that were inspected are now described.

a) <u>Hangaroa Apartments</u>

This fifteen storey apartment block was built during 1970 and 1971. Bonnelli (Ref. 2) reports that it was damaged by an earthquake in 1971, but no sign of repairs were observed by the EEFIT team.

It is situated on the waterfront. A typical plan view is shown in Figure 5.2 and the elevation of the building is shown in Plate 5.7. It can be seen that lateral resistance is provided by curved shear walls linked by reinforced concrete bracing.

The main failure in the primary structure was a major vertical crack (Plate 5.8) at the junction of one of the shear wall arms with the core which had apparently racked extensively during the earthquake. The shear wall was perforated on alternate floors along the line of failure by door openings.

A comparison of the high yield steel provided in the failed area with minimum code requirements is given below:-

	Horizontal (each face)	Vertical (each face)	
Provided in Hangaroa	0.11%	0.11%	
UBC (Ref. 14)	0.10%	0.06%	(bars > 16mm dia)
BS8110 (Ref. 15)	0.10%	0.20%	(for "reinforced concrete walls")

In comparison with minimum requirements, though of course not necessarily imposed design stresses, the horizontal steel appears just adequate. Bonelli has postulated (Ref. 2) that the failure was due to the different mode shape between the x-braced twin shear walls in the central core, and the circular shear wall in the wing; a full dynamic back analysis would clearly be valuable to investigate this further.

There was no sign of failure in the corresponding place on the opposite side of the building, probably because there were no door perforations at this corresponding place. Extensive damage to the secondary external elements (fascia beams etc) was observed. The lifts apparently did not fail; the EERI team inspected the lift motor room and found no damage. A few windows were cracked, but breakage was not extensive.

b) <u>Acapulco Apartments</u>

This 15 storey r.c. shear wall apartment block was completed in 1964. It is adjacent to the Hangaroa. The shear walls were damaged during earthquakes in 1965 and 1971, and superficially patched with inadequate reinforced plaster (Plate 5.9). The building had extensive structural and non-structural damage, (Plate 5.10) though the lifts were still operational. Structural damage was observed in shear walls. Cracks a few millimetres wide were observed in the soffits of some floor slabs. The cracks ran parallel to the long axis of the building. The building was being emptied of its contents by the apartment owners during the EEFIT inspection. There was some broken glass but mainly the glazing stayed intact.

c) <u>Adjacent buildings on the waterfront</u>

Six high rise concrete buildings adjacent to the Hangaroa and Acapulco showed little superficial damage. The nearest one (Edificio Marina Real) had some spalling on one external shear wall (Plate 5.11).

Two others had some superficial damage to balconies, and ground settlements of the order of a few centimetres were apparent around these blocks. The two newest blocks (one incomplete) showed no damage. The lack of damage to other high rise buildings in the vicinity was confirmed by Prof Bonelli of Valparaiso University.

d) Canal Beagle estate

This is a complex of about 40 four and five storey buildings, with concrete cores, concrete columns and shear walls and lightly reinforced blockwork infill panels. They are built around a steep ridge (Plate 5.12). There was extensive structural and non-structural damage (Plate 5.13), with buildings at the top of the ridge being severely damaged, and those at the base experiencing little or no damage. Similar developments elsewhere were little affected. The estate had been partially evacuated, though the buildings near the base of the slope remained occupied.

In so far as it could be determined on site, there seemed no obvious correlation between the disposition of shear walls and extent of damage; some buildings appeared to have shear walls in one direction only; others equally affected, had well disposed walls. Generally, quality of materials and workmanship seemed adequate, though there was some evidence of poorly prepared construction joints and a wooden block was found cast into the section of one column which failed. The percentage of vertical reinforcement in the shear walls, around 0.08% each face, though rather light, was not negligible.

Overall, the extent of the damage appeared to be surprisingly high. Topographical effects, namely ridge amplification, were postulated as the cause, and other examples from around Vina in this earthquake were cited of correlation between damage and position on the tops of ridges. Subsequent research has confirmed the existence during small earthquakes of significant amplication of ground motions on top of the Canal Beagle Ridge compared with motions at the base (Celebi, Ref. 10).

e) Barrios Edificio

This 5-storey r.c. apartment building had been damaged in previous earthquakes and repaired but again suffered severe damage in the present earthquake. It was located on level ground in the centre of the city. The central staircase was flanked by concrete shear walls which were perforated by door openings. All the lintols above the doors had failed. Previous earthquakes had led to severe cracking of one end wall and the latest earthquake had reopened the old cracks which had simply been patched over. At the other end wall a new, narrow shear wall had been added following the previous earthquake. Although the new shear wall was undamaged, it did not prevent old cracks from reopening (see Plate 5.14).

There may not have been effective connectivity between the old and new structures, thus allowing the new shear wall, which appeared undamaged, to act independently of the old structure.

f) Other damage reports from Vina del Mar

Bridge links connecting some 5 storey r.c. housing blocks were damaged (Plate 5.15) though they were not visited by the EEFIT team. The buildings themselves were apparently unaffected. Professor Bonelli also reported extensive non-structural damage, but no structural damage, to a 50 storey building.

5.2.4 Coastal resorts north of Vina del Mar and Valparaiso

An eight storey shear wall block (Edificio El Faro) on high ground at Renaco, about 5km north of Vina, suffered a spectacular failure in the lower shear walls. This resulted in a rotation of the structure by some 15 degrees, but not total collapse. By the time the EEFIT team arrived, the building had been dynamited since it posed a safety hazard. There were suggestions that severe design and construction errors were the cause of failure, but EEFIT was not able to pursue this further.

One other modern r.c. structure on the coastline north of Vina was reported damaged due to differential settlement by EERI (Ref. 4), though EEFIT did not find this. Many other modern r.c. structures were seen which appeared superficially undamaged.

5.2.5 Towns in the hinterland between the Valparaiso/San Antonio coastline and Santiago

There are few engineered residential buildings in this area, and no reports of major damage. Some industrial structures were damaged, as reported in 5.9.

5.2.6 Santiago (MMI: generally VI to VII, in places VII to VIII; 80km from source area; 100km ESE of epicentre).

a) General

Downtown Santiago has the appearance of a prosperous modern capital, with extensive modern development, including high rise offices, hotel and public buildings, a metro system, and a developing road network. It has the character of belonging more to the developed than the developing world. As discussed in Section 3.2, Santiago is situated on flat alluvial deposits at the northern end of the Longitudinal Valley (Figure 3.1).

Although damage to masonry structures was widespread and superficially quite evident, this was not the case for engineered structures. However, there were isolated cases of collapses in engineered building structures, including the only total collapse observed, which are discussed in detail below. Moreover, there were reports of widespread minor structural and non-structural damage, which are further discussed at the end of this section.

b) Villa Olympia housing estate

This estate of r.c. five storey apartment buildings was completed in 1962 to house visitors to the World Cup. It is located next to the large National Stadium which was superficially undamaged. The Villa Olympia estate covers several hectares and is now used as low cost municipal housing.

The housing is formed of 5 storey linear blocks. The first storey is generally open, though in some places this space has been partitioned off with blockwork to form storage space. Walkways at first (ie. one above ground) floor level connect the blocks, and an apparently well thought out engineered detail had been provided to prevent horizontal forces being transmitted through the walkways. The vertical structure comprises r.c. columns and access cores, and the horizontal structure r.c. beams and slabs. Superficially, the standard of construction appeared high.

The only major damage that the EEFIT team observed, or that was reported to them, was the total collapse of an end section of one block - (Plate 5.16). A 240mm thick r.c. shear wall 4 storeys high supported by 2 r.c. columns from ground to first floor level formed the end elevation. The columns had failed at the junction with the shear wall, and the floors above had suffered a pancake collapse. The columns, originally 3.2m high, now stood at a maximum of 2m high. The framing of the block is sketched in Figure 5.3.

Three major causes of the collapse are indicated. Firstly, the lower relative horizontal stiffness of the column supported end bay, compared with the core supported internal bays, may have lead to torsional amplification of earthquake motions at the end of the building. Secondly, the sharp change in stiffness and strength on the end elevation between shear wall and column formed a classic 'soft storey', with its consequent high ductility demand on the ground floor columns. Thirdly, there was a lack of redundancy; the presence of only two columns meant that there was no scope for redistribution of loads, and incipient failure in either column meant almost inevitable collapse.

Plate 5.17 shows the similar end of an adjacent block at right angles, which however was supported by 4, not 2, columns and may have received some lateral restraint from the first floor walkway. It appeared undamaged. The remarkable feature was the lack of damage in surrounding blocks in the estate; there was some damage at the walkway connections, but no broken glass, cracked partitions or evidence of ground movements.

c) Villa Portales estate

This is located about 3km WSW of Villa Olympia, and forms a very large estate of 6 storey housing in long linear blocks, built by the state in the early 1960's. The main features noted were as follows:-

i) Damage had been suffered in earthquakes in 1965 and 1971, and the resulting cracks had been patched rather than repaired.

ii) Unlike the Villa Olympia estate, structural and non-structural damage was widespread, though there were no total collapses.

iii) The worst damage was at the end of Block-1 (Plate 5.18) which had some similar features to the failed Villa Olympia block. These features comprised a soft lower storey, with the probability of torsional amplification due to a lightly restrained end.

iv) A block parallel and adjacent to Block-1 with very similar layout appeared largely undamaged (Plate 5.19).

v) Poor detailing was associated with some cracking. Box outs for gas pipes in a shear wall at the end of Block-1 appeared to have been associated with a major failure (Plate 5.20) and lack of lateral restraint to main bars supporting the end of the same block probably were associated with the extensive spalling, (Plate 5.18 (b)).

vi) The extensive high level walkway system appeared undamaged.

vii) A number of roof water tanks supported on X shaped r.c. cross walls in turn supported on four stub columns were extensively damaged (Plate 5.21), though interestingly, that on the severely damaged Block-1 appeared undamaged.

viii) Water pipes crossing expansion joints had split in previous quakes. A U- loop had been subsequently provided (Plate 5.22) and seemed to have worked well in this earthquake.

d) <u>Other damage reports</u>

The 5 storey Sheraton hotel suffered extensive cracked plaster and partitions, and there was spalling due to buffetting across an expansion joint. Both lifts were reported to have failed during the earthquake; one was restored a few hours later and the other a few days later. However, there appeared to be no major structural damage.

The British Embassy located in a 4 storey r.c. frame building had some quite extensive non-structural damage, though it appeared undamaged from outside. The major problem was caused by the water tank spilling its contents during the earthquake, causing water damage, and blockwork partitions had also cracked.

The University of Chile engineering faculty is a 4 storey, r.c. frame building with massive columns allowing future provision of some 8 additional floors. No structural damage was reported, but there was considerable damage to metal false ceilings and books were thrown from shelves.

Similar damage to the metal false ceiling in the single storey airport terminal building was observed.

Similar types of non-structural damage were reported to be quite widespread in Santiago. Piles of debris from collapsed false ceilings, partitions etc. was a common sight on the pavements of Santiago.

5.2.7 Rancagua (MMI VI; 80km from source area; 150km SE of epicentre)

No evidence of damage to engineered structures was found in a brief visit to this town, with a population of c. 150,000, to the south of Santiago.

5.2.8 Curico (MMI VI to VII; 100km from source area; 200km SSE of epicentre)

Initial reports were that the modern, 5 storey r.c. hospital at Curico was severely damaged. However, an external inspection, although revealing some broken glass, cracked infill panels and local spalling, suggested that damage was not serious, and this was confirmed both by the fact that the hospital was operational, and by the pastor of the main church in Curico. The pastor also confirmed the impression that no other engineered structures in Curico were damaged.

The same pastor reported that there was little damage to any sort of building south of Curico.

5.3 Reinforced Blockwork Housing

The Villa Santa Carolina estate in Santiago is a recently completed (1984) estate of 3 storey housing. It consists of load bearing walls, with reinforced concrete slabs and perimeter beams. There were no r.c. columns. All the shear resistance appeared to come from the load bearing walls. The walls were formed of fired bricks containing voids filled with grout and occasional reinforcement. This reinforcement appeared to comprise about 2 No 8mm diameter mild steel bars at corners and framing window and door openings. Single 4mm diameter mild steel bars laid in about every other course formed the horizontal reinforcement. The bricks appeared of high quality; they were hard and non friable, as was the mortar. Shear cracks ran non-preferentially through both brick and mortar. There was some evidence that mortar filling of voids in the brickwork was incomplete in places.

The estate is sited on flat ground, about 1km SE of the Villa Olympia which was damaged (Section 5.2.6 (b)).

The ground floor walls of most of the blocks suffered extensive cracks to walls running in both directions (Plate 5.23). Upper floors appeared little damaged. The estate had been largely evacuated. There were no obvious weaknesses in the layout of shear walls, which appeared well distributed. The undamaged concrete floors, though heavy, formed an adequate horizontal tie around the building, and there was no sign of differential movement between slab and walls. The concentration of damage in the ground floor suggests that dynamically induced inertial shears caused the cracking, not differential settlement, and there was no sign of the latter.

The extent of damage 80km from the source area in a series of buildings without any readily apparent defects seems anomalous. This fact, the simple form of the structure, and the importance of such low cost housing to the Chileans makes this a particularly good candidate for follow up studies.

No other buildings entirely of reinforced blockwork were inspected, though a number of r.c. buildings had reinforced blockwork partitions which suffered widespread damage.

5.4 Adobe and Unreinforced Masonry Housing

Damage to these non-engineered structures was very widespread, both on the towns and villages on the Santiago/San Antonio axis and also those that the EFFIT team visited south of Santiago down to Curico. On this latter axis, there appeared to be concentrations of damage at Rengo and Curico and this is confirmed by Saragoni (Ref. 1) - see Figure 1.1. Both towns are located on alluvial deposits in the Longitudinal Valley, and soil amplification effects may have been present, though it also seemed possible that the extensive damage at Rengo was due to very poor construction. (See also Section 5.6f).

5.5 Wooden Housing

One and two storey wooden houses, even those of poor construction, escaped the earthquake largely unscathed. A dramatic example was seen in San Antonio, where a street had masonry buildings on one side and wooden ones on the other. Local people confirmed the immediate impression that the wooden house had been almost undamaged while the masonry ones were badly damaged (Plate 5.24).

A five storey hotel just above the dock at San Antonio (Hotel Jockey Club) had one facade collapse. The timber frame appeared rotten and inadequate, and the building had been extended at some time in the past.

Chile has large timber resources and researchers at Concepcion are actively engaged in developing low-cost earthquake resistant timber dwellings.

5.6 Churches and Other Monumental Buildings

a) General

A large number of churches were visited. Almost all were damaged and in many cases the damage was severe. Another common feature was that many of the churches were reported to have been damaged in previous earthquakes. The construction was mainly unreinforced block; both adobe (mud) and fired blocks were encountered. Some notes on some of the structures visited are given below.

b) Vina del Mar (near source area; 20km NE of epicentre)

Part of the front facade of the Archaeology and Natural History museum collapsed (Plate 5.25).

c) Llolleo (in source area; 50km S of epicentre)

Some of the structure of the modern looking church appeared to be in reinforced concrete. It suffered damage to roof and campanile (Plate 5.26). Llolleo is just south of San Antonio, and had a strong motion instrument which recorded 0.86g. A smaller Spanish style unreinforced religious building adjacent to the strong motion instrument appeared extensively cracked (Plate 5.27), and it was difficult to understand why it had not collapsed.

d) Melipilla (40km from source area; 70km SE of epicentre)

Extensive cracking was observed in both the unreinforced churches. (Plate 5.28)

e) Santiago (80km from source area; 100km ESE of epicentre)

The Basilica del Salvador, (Plate 5.29) started in 1870 and completed in 1920, was built in unreinforced, but good quality fired brick with mortar joints. Two of the internal columns and parts of the external walls collapsed and there was extensive cracking.

f) Rengo (80km from source area; 150km SE of epicentre)

A large railway shed, about 10m high with adobe walls strengthened by adobe buttresses, and a timber truss, corrugated iron roof, suffered a partial collapse (Plate 5.30). This type of structure would be expected to be susceptible to earthquake damage. The fact that the collapse was not more extensive, and the lack of damage in the adjacent unreinforced masonry signal box (Plate 5.31) and some other adobe buildings in Rengo reinforces the view (Section 5.4) that the concentration of damage in Rengo may owe as much to poor construction as to soil effects causing amplified motion. The undamaged steel frame shop (Section 5.7c) also supports this view.

g) Pelequen (80km from source area; 150km SE of epicentre)

Extensive cracking and collapse of the false ceiling were found in a church building, the Sanctuar Santa Rosa (Plate 5.32).

h) Curico (100km from source area; 200km SSE of epicentre)

Two large churches were visited. The older was severely cracked. The newer church, built in 1914 with a concrete dome added in 1956 was in fired brick and suffered little structural damage. The significant feature was that the copper cross on top of the dome was tilted to the west by the earthquake (Plate 5.33), suggesting that strong motions were present in Curico, 100km from the source area. Crosses on other adjacent churches were also tilted, though not necessarily to the west. The pastor of the church confirmed that he had felt strong enough vertical and horizontal motion in the ground floor of his house to necessitate clutching for support.

5.7 Steel Frame Structures

The number of steel frame structures observed, compared with reinforced concrete structures, was small, and in all cases good performance was observed as described below. It is understood that there are no high rise steel buildings in Chile.

a) San Antonio (in source area; 50km S of epicentre)

A 30m span steel portal frame shed, (Plate 5.34) in the form of a 3 pinned arch, built in 1971 on the outer mole of the docks which failed (see Section 5.12) not only withstood without collapse several metres of differential movement across the span due to failure of the wharf, but also absorbed the impact from one of the pedestal cranes which fell on its roof. It subsequently supported the crane.

b) Melipilla (40km from source area; 70km SE of epicentre)

A 3 storey steel frame building under construction, of which only the frame had been built, was apparently undamaged.

c) Rengo (80km from source area; 150km SE of epicentre)

A single storey supermarket (Plate 5.35) with a steel truss roof covered by asbestos, cement sheets and supported on steel columns, was reported by the inhabitants as undamaged. Three sides were stiffened partially by concrete shear walls, the fourth long side contained a glass curtain wall. There was also considerable eccentricity in the structure. No special earthquake resistant provisions appeared to have been made in the glass facade, which was, however, undamaged.

d) Puquillay Bajo (80km from source area; 100km SSE of epicentre)

An X braced water tower, approximately 20m high and square in plan, made of bolted steel angles, appeared to have suffered minor buckling to the rod braces on at least one face (Plate 5.36). However, the bracing was probably designed on the basis of the tension strength of the bracing, so buckling of the compression members would be expected well before failure, and the structure was evidently well able to perform its function after the earthquake. The small masonry pump house next to the tower was undamaged.

Puquillay Bajo is situated on the Longitudinal Valley. At an epicentral distance of 180km, long period motions would be expected to be more significant than short period ones, especially where soil amplification effects might be present. The relative response of the long period water tower and short period pump house is consistent with this expectation.

e) Vina del Mar (near source area; 20km NE of epicentre)

A number of steel microwave towers on high ground above Vina appeared undamaged.

5.8 Power Stations

No reports of major damage to power stations were received, and it is understood that all power plants were back on line within a few hours of the main shock. The EERI report (Ref. 4) reports quite widespread minor damage; pipe thermal insulation was damaged at a plant at Las Ventanas, 65km NE of the epicentre, and lateral restraints to rod hung boilers were damaged at Renca. The EEFIT team did not encounter any instances of power loss, in buildings, street lighting, telecommunications, or transport.

It is understood that two experimental nuclear power stations are located in the Santiago area. No official details were available, but it appears that neither was seriously affected.

5.9 Industrial Facilities

Response of facilities connected with some of the major industries of the area are discussed below.

5.9.1 Agriculture

A number of flour mills and particularly their grain silos, were damaged in the earthquake.

The central reinforced concrete elevator tower to a flour mill in Melipilla was damaged, apparently due to interaction with connecting links to the adjacent main mill building (Plate 5.37). The mill was however operational at the time the photo was taken, and a 5 storey steel mullion supported glass curtain wall in the main mill building (out of view to right, Plate 5.37) was apparently undamaged.

Another flour mill, just outside Melipilla, was reported to be much more severely damaged, with a number of r.c. silos affected, but this installation was not visited.

Dowrick visited a flour mill at Casablanca, where a number of silos, both in concrete and corrugated steel, were either badly damaged or completely collapsed (Plate 5.38). Dowrick reports (Ref. 5) that some of the concrete silos, dating from the sixties, had been damaged by the 1965 and 1971 events. They had been repaired by thickening their walls at the base, and had subsequently failed at the change between thickened and original wall section.

5.9.2 Petrochemical Installations

A large refinery at Concon, 30km north east of Valparaiso, was damaged by the earthquake. One steel tank appeared to be leaning significantly and another (Plate 5.39) could be seen to be buckled at the top, and to be stained by an oil spill. Although no information could be obtained from the authorities, the refinery was apparently working normally and no sign of serious fires or oil spills could be seen. Further information is given by the EERI report (Ref. 4).

Some steel oil storage tanks near San Antonio docks appeared undamaged, and no reports of other damage to petrochemical facilities were received.

5.9.3 Copper mines and refineries

Damage to tailings dams is discussed in 5.11. Some damage to Las Ventanas copper refinery and foundry is reported by EERI (Ref. 4), but this installation was not visited by the EEFIT team.

5.10 Roads and Bridges

13 major road crossings were closed immediately following the earthquake. In addition, numerous minor slips in bridge approach embankments were observed in the roads running south and east from Santiago, and at least one wingwall failure in a short span bridge. Nevertheless, quick action by the authorities in providing alternative routes meant that the road network was largely operational at the start of the EEFIT mission, 7 days after the event, and there were no serious obstacles to relief distribution. Particular cases are now described in detail.

5.10.1 River Maipo crossings, Llolleo (in source area; 60km S of epicentre)

Two reinforced concrete bridges cross the mouth of this major river, about 2km south of where motions of 0.67g horizontally and 0.86g vertically were recorded, and about 5km south of San Antonio.

The newer bridge, the Ponte lo Gallardo, (Plate 5.41) dates from 1956. It consists of 28 simple prestressed concrete spans of 30m length, supported on solid concrete piers and abutments, founded on 1m diameter concrete piles. One river pier was complete lost, and another had both sunk and shifted downstream by several metres. Two spans over the extensive flood plain to the south of the bridge had dropped on one side (see left of plate 5.41(a)), perhaps due to spreading of the bridge following loss of the river spans.

The older bridge (Plate 5.40), unused since completion of the newer one, was sited about 0.5km upstream. The EERI team (Ref. 4) reports that it had been used for target practice by the army, and it may have been weakened both by this and by previous flood damage. A river pier was lost, another had sunk, but there was no sign of any lateral movement.

The EERI team (Ref. 4) was informed that substantial scour around the piles may have been present before the quake, and so may have been the prime cause of failure. The fact that there was no evidence of lateral movement in the older bridge, and the limitation of movement in the newer bridge casts some doubt on this interpretation, though without more data no conclusions are possible. Given the location in the earthquake source area on an alluvial plain with high water table, liquefaction is an alternative possible cause. The nearby strong motion recording would add to the fruitfulness of follow up studies on this failure.

Interestingly, an unreinforced masonry, single storey shed on the north embankment by the newer bridge appeared undamaged.

The army had provided a pedestrian pontoon bridge within a few days of the earthquake (Plate 5.41(b), foreground), and appeared almost ready to launch a Bailey type vehicular bridge over the 250m crossing 2 weeks after the event.

5.10.2 Liheimo River - Near Peralillo (30km from source area; 150km SSE of epicentre)

Gates (Ref. 4) reports an interesting failure in this 50m long bridge. Eight days after the earthquake, one span dropped at least 3m, reportedly after passage of a laden cement lorry. This bridge was not however inspected by the EEFIT team.

5.10.3 Santiago bridges (80km from source area; 100km ESE of epicentre)

There are numerous bridges in Santiago, both river bridges of a variety of ages crossing the river Mapocho (the major river in the city), and also the bridges of the modern road interchanges. No damage was observed, nor were reports received.

5.11 Dams

The EEFIT team did not inspect any dams but obtained reports from the EERI team and Chilean engineers.

Two tailings dams at copper mines, at Veta de Agua near El Cobre (35km due east of Ventanas, a coastal town north of Valparaiso) and at Cerro Negro (65km due east of Ventanas), were reported to have failed, apparently as a result of liquefaction (EERI, Ref. 4). A small embankment dam, used for irrigation purposes, was said to have suffered significant slumping, but the reservoir had not been released.

An 18m high earthfill embankment at Lago Penuelas, south of Vina del Mar, suffered no significant damage. This embankment, built in 1903 and having steep slopes and a central clay core, supplies water to Vina and Valparaiso. It suffered settlements in the 1906, 1965 and 1971 earthquakes.

At Central Rapel, 100 km south of the epicentre, a concrete arch dam suffered no structural damage to the arch but drop gates to the low level sluices were dislodged. One gate fell onto a grid screen. There was also some cracking to the concrete guide walls of the drop gates at the top of the walls where an expansion joint had not been properly formed. Switch gear and ceramic insulators in the adjoining power station had been damaged. Several small battery packs had been dislodged because the packing between the batteries was inadequate. Copper conductors joining the batteries, 25mm wide and about 1mm thick, had buckled.

5.12 Docks

Harbour facilities at Valparaiso and San Antonio, the two main commercial ports of Chile, were damaged. The EEFIT team spent several hours examining the failures at San Antonio, which included a major embankment collapse, as reported below. The damage at Valparaiso was less serious, involving more minor settlements and horizontal movements, and was not studied in detail by EEFIT.

The main feature of the San Antonio harbour facilities is a long embankment mole (Figure 5.4) which has three berths for ships up to 18,000 tons. The part of the mole which collapsed (Figure 5.5) is constructed from hydraulic fill behind a massive block quay wall, founded on clay and dense sand. The southern end of the mole is formed from an anchored sheet pile, and did not fail. A number of reinforced concrete frame and steel frame warehouses are located on the mole.

The 1971 earthquake (Lastrico et al, Ref. 16) caused the top of the quay wall to tilt outwards by up to 500 mm and the backfill to settle by a similar amount, while a longitudinal crack running almost the full length of the mole, was observed. The 1985 earthquake caused similar and more extensive damage (Plate 5.42). Nearly all the gravity quay wall collapsed due to tilting, and multiple longitudinal cracks developed along the embankment as a result of movement of the wall. The primary cause of failure was probably liquefaction of the hydraulic fill in the centre of the embankment giving rise to increased pressures against the wall. The sea defence wall along the western edge of the quay was formed from massive concrete blocks laid to a shallow slope (see left of Plate 5.42(a) and Figure 5.5) and was undamaged.

A sand volcano about 1 m diameter and 1 m deep, (Plate 5.43) was found under a section of the concrete paving slab adjacent to the quay wall. The hole was surrounded by fine sand, suggesting that water had first been forced out of the hole and had then drained back in, probably as the wall began to move. One of the warehouses contained a large number of copper ingots and consequently there would have been a considerable surcharge close to the quay wall.

Several dock-side cranes toppled on to the steel framed warehouses as the quay wall failed (Plate 5.34). These warehouses use three pinned arch frames rather than rigid portal frames and performed very well, despite spreading of the arches by more than a metre due to the movements of the embankment.

The adjacent concrete framed warehouse suffered severe damage due to spreading of the foundations across the cracks in the embankment (Plate 5.44). Damage to all the dock-side structures appeared to be due to earthquake induced differential foundation movements rather than earthquake induced inertia forces.

5.13 Water supply and sewerage systems in Greater Valparaiso

Engineers from the Government Autonomous Enterprise responsible for the water supply to Greater Valparaiso, including Vina del Mar, La Calera and Limache, provided much useful information on the performance of the water supply and sewerage systems in these areas and arranged visits to a few works. The water supply systems had been badly damaged in the 1965 and 1971 earthquakes (Lastrico et al, Ref. 16). The following sections are mainly based on information supplied by the local engineers.

5.13.1 Water Sources

The water utility serves a population of over 750,000 in Greater Valparaiso, 40,000 in La Calera and 20,000 in Limache. Its three main water sources are a system of underground galleries at Las Vegas, near Llay-llay, a pumping station at Concon on the Aconcaqua River, and a 90M.cu.m reservoir at Las Penuelas. These are now described.

a) Las Vegas aqueduct

The Las Vegas galleries are about 80m long and supply about 2,200 litres/sec to a 80 km long, 1.3 m diameter, reinforced concrete aqueduct that supplies Greater Valparaiso. According to Lastrico et al, (Ref. 16) the walls of the aqueduct are 130 mm thick and are reinforced circumferentially by 10 mm bars at 80 mm centres and longitudinally by 6 mm bars at 115 mm centres.

Water flows through the aqueduct under gravity. The aqueduct had been inspected internally and all breaks identified. However, lack of personnel had prevented repairs from being started and the aqueduct was still out of service.

b) Pumping Station at Concon

The pumping station at Concon, built around 1925, supplies 400 litres/sec and includes settlement tanks, filtration and flocculation systems. Damage caused by the 1971 earthquake was repaired by epoxy injection. No significant damage was reported as a result of the 1985 earthquake.

c) Reservoir at Las Penuelas

Details of the 18 m high embankment dam have been given in section 5.11. No significant damage to the dam or associated aqueduct, which supply 400 litres/sec to Valparaiso, was reported.

5.13.2 Distribution network in Greater Valparaiso

The distribution network in Greater Valparaiso has developed with little overall planning and is therefore rather complex. It does not follow a logical tree system and operates at about 60 m head. In the main, water from the three sources at Las Vegas, Concon and Las Penuelas is distributed via 1.2 m and 600mm diameter underground pipe networks to large elevated tanks. Secondary networks distribute the water from the elevated tanks to consumers via 300 mm and 75 mm asbestos-cement pipes. Not all consumers are supplied from the network and the water utility has its own fleet of supply trucks serving these consumers.

Immediately after the earthquake, all water supply systems were shut down to conserve water. Sections were then gradually returned to service when repairs were completed. Ten days after the earthquake, about 80% of the network had been restored. An additional 20 water tankers were hired to supplement the water utility's own vehicles and service small distribution stations that were set up on street corners in areas where there was no operating piped supply. The repair programme appeared to be well planned and executed, despite the difficulties imposed by the nature of the network and lack of resources.

Problems were experienced with the asbestos-cement pipes which tended to fail at the joints or by shearing through. The joints consist of a plastic collar which fits over the ends of the pipe sections and is sealed by a pair of rubber O-rings. These joints have little tensile strength and tended to fail by pulling apart as the pipes were put into tension due to differential earth movements.

No problems with elevated water tanks in the area were reported, or observed by the EEFIT team.

5.13.3 Sewerage system in Greater Valparaiso

The sewerage system in Greater Valparaiso is very simple and suffered no significant damage in the earthquake. Sewage undergoes little or no treatment before being discharged into the sea. An old pumping station in the centre of Vina was visited and found to be in operation. The station consisted of a 8 m deep, 6 m diameter underground tank from which raw sewage was pumped into the discharge pipeline.

5.13.4 Pumping Station at Llolleo

A water supply pumping station by the River Maipo, near Llolleo where peak ground accelerations of up to 0.86g were measured, was visited by the EEFIT team following reports that it had been seriously damaged. This station was reported to have suffered significant damage in the 1971 earthquake.

The pumping station consisted of a pumphouse feeding a large diameter watermain supplying Valparaiso and some water treatment and storage tanks. The station was fed from about 5 well points, located about 0.5km away in the River Maipo flood plane (Plate 5.45) near the edge of fill about 1.5m high, some 15m from the river bank. The structure of the pumphouse had been damaged by the 1985 earthquake but at the time of the EEFIT visit was operational and supplying water to Valparaiso.

The cause of the original failure in supply was that all the wellpoints had failed. Movement in the fill may have been associated with the failure, but this could not be established. At the time of the EEFIT visit, 2 weeks after the earthquake, about 3 new boreholes had been installed about 20m back from the edge of the fill and at least one was supplying water.

5.14 Electricity Supply in Greater Valparaiso

Discussions were held with engineers from Chilectra, the company responsible for electricity supply to Valparaiso and parts of Vina del Mar. The following information was supplied by the Chilectra engineers.

Chilectra is mainly concerned with electricity distribution and does not generate much of its own power. It has about 300,000 customers. The distribution system consists of 12 kV feeder lines supplying a system that is generally carried on reinforced concrete poles.

Chilectra has had emergency plans since 1968, mainly as a result of previous storm damage rather than earthquake. The plans are not co-ordinated with the police, fire or other public emergency services. One engineer is nominated as emergency manager and is responsible for the co-ordination of the emergency procedures. The procedures involve closing down the whole system followed by staged reconnection of the feeder lines, sector by sector. In this way, faults in the primary network can be readily located. When the primary network is functioning satisfactorily, the associated secondary network is reconnected in a similar manner. Any sectors of the system not functioning are left until any attempts to reconnect the remainder of the network have been completed. The non-functioning sectors are then repaired and reconnected. Priority is given to hospital and water supply systems.

No transformers dropped during the earthquake and little power was lost initially. Most of the failures occurred when the system was reconnected after the planned post earthquake shutdown, about 16% of the network being affected.

One primary feeder line failed due to ingress of moisture through cracked insulation, although this damage may have occurred before the earthquake. Reconnection of the transformers commenced on the Tuesday following the earthquake (a delay of between 36 and 48 hours) and 84% of the network was functioning by the evening of the following day, at the end of the first phase of the reconnection sequence. The remaining 16% of the network was functioning by the following Sunday, 7 days after the earthquake.

In the secondary system, about 15% of the network was affected by cracking to ceramic insulators as a result of seismic stresses, and a further 15% of the network was affected by wires falling from poles. One hundred poles fell down out of a total of 30,000, with six being left on a slant at the end of the earthquake.

The main problems encountered by Chilectra were in the civil works, particularly underground masonry vaults of which a number collapsed or were badly cracked leading to water intrusion and shorts. Further problems were encountered with underground asbestos-cement conduits which failed due to differential ground settlements. The conduits failed in shear or by pulling apart of the plastic collar joints between sections.

Street lighting also suffered significant damage. The sodium lighting units were of two types, one mounted at the end of a 5 m cantilever on top of 9 to 12m high posts, the other mounted at the end of a cantilevered bracket attached to the wall of a building. There were 20,000 of the first type and 4,000 of the second. Approximately 1,200 units were damaged in the earthquake, mainly due to lost heads. A further 11,000 units were not operational due to other reasons (e.g. failed power lines) in the first week after the earthquake. Repairs to the lighting system commenced one week after the earthquake and were completed in three days.

The emergency procedures appeared to be well planned and competently executed.

5.15 Geotechnical Failures

The major geotechnical failure observed was at the San Antonio wharf described in Section 5.12. Dam failures are discussed in Section 5.11.

A number of other minor failures were observed by the EEFIT team. Cracks in road bridge embankments appeared quite common and were noticed up to 100km south of Santiago. A minor fall from a road cutting just east of San Antonio was observed, which partially blocked the road (Plate 5.46) and more major rock falls in the mountains on the main road between Santiago and Valparaiso were also reported, though not seen by the EEFIT team. Embankment failures on the coast north of Vina del Mar were also observed in partially cemented silty sand.

San Antonio is situated on the slopes of a narrow valley leading to the sea, although no surface river flows through this valley. At the mouth of the valley, in front of the harbour, there is an area of flat ground. Excavation for water mains throughout the city (including on sloping ground) indicated that the superficials deposits were a fine grained, black sand. A number of streets of around 4% slope showed tension cracks at the top, and compression at their base. These were probably associated with small slips caused by liquefaction.

Throughout the sloping areas of San Antonio, there was extensive evidence of ground movement, which in some cases led to building collapse (see front cover). Foundation settlements were reported to be continuing some days after the earthquake. Foundation settlements are further discussed in Section 5.2.1(c).

5.16 Fire

A timber frame single storey house was observed destroyed by fire on the coast a few kilometres north of Vina del Mar, and this was reported to be related to the earthquake. However, no major outbreaks of fire were observed or reported. It is understood that there is no extensive pipeline network supplying gas, and that liquefied gas is delivered by tanker to individual dwellings and community tanks.

5.17 Metro

A modern cut and cover Metro, consisting of some 10kms of line, runs through Santiago. The system was operating normally at the time of the EEFIT visit, and there was no evidence or reports of damage.

6.0 Conclusions

6.1 General

The Chilean earthquake of 3rd March 1985 was a major event with many features of interest to the earthquake engineering community. Although damage was widespread, it was not, on the whole, as severe as might have been expected for a magnitude M_s = 7.4 earthquake. The spread and variability of damage to engineered and non-engineered structures seems to have been considerably influenced by topograpy, local soil deposits and quality of construction.

Many well built structures within the source area suffered little or no damage, while serious damage was experienced up to 100 km from the source area. An extensive network of strong motion instruments has yielded a large number of good quality ground motion records which will form a valuable basis for the study of these effects.

The earthquake confirmed many old lessons in earthquake engineering and taught several new ones. With the spread of strong motion data and the range of engineered structures in the affected area, the earthquake has presented a rare opportunity for detailed study.

6.2 Old Lessons Confirmed

- Structures which observe the basic rules of earthquake engineering will generally perform much better than those that do not. These rules include choice of regular form, good ductility, adequate lateral stiffness, careful detailing and construction and avoidance of unstable slopes (see front cover). The effort and cost involved in observing these rules can be small in comparison to the benefits accrued. Robert Mallet (Ref. 17) was giving similar rules in 1862, so some of the lessons are over a century old.

- Dynamic response to strong ground shaking is crucial in the assessment of structural behaviour and strength. The pseudo-static approach, in which a lateral resistance of say 10% of the structure's weight is provided, is therefore often inadequate, particularly for tall structures and those of irregular form.

- Well designed buildings which suffer little or no structural damage can still suffer very extensive non structural damage. Many fatalities and injuries are due to falling non structural debris and repair of non structural damage may involve considerable expense. Great care needs to be taken in detailing such parts as infill panels, partitions glazing, parapets and services. Small components often cause the most problems because they are neglected in the overall seismic design.

- Damage accumulated in past earthquakes can have a significant effect on performance in subsequent earthquakes. Repairs need to be well thought out and carefully implemented. It is difficult to make a reliable repair to a damaged structure which will restore it to its undamaged strength. Careful consideration must be given to the question of whether to repair or demolish.

- Timber structures tend to perform well in earthquakes because they have a low mass to strength ratio, have high tensile strength and can be extensively tied together by nails or screws. These features made for good seismic resistance of the Chilean wooden buildings, although rotten timber and poor connections formed a seismic hazard. Use of more structural timber in the traditional building techniques would be a cost effective improvement and would utilise a major natural resource in Chile. This fact is recognised by the Chilean authorities, who are developing seismically resistant timber houses.

- Saturated, loose to medium dense sand readily loses its strength when shaken, and is a poor foundation or structural material. Liquefaction was apparent in parts of San Antonio where it was associated with a major embankment mole failure, and possibly with at least some of the slope failures, some of which damaged houses, and with gross foundation settlements. It may also have been associated with the River Maipo bridge failures.

- Well planned emergency and relief procedures, with an identified chain of command, can alleviate much suffering, despite any lack of resources. If relief work is seen to be well organised and rapid, public morale will be maintained for longer and the general situation will improve more rapidly. The good public morale and speed of repairs in Chile was in marked contrast to the situation prevalent in Southern Italy following the earthquake of 23rd November 1980. Having been through several major earthquakes in the last 25 years, the Chilean people obviously knew how to cope and what to expect. Education of the general public in earthquake preparedness and post-earthquake response, and some official guidance on earthquake resistant do-it-yourself repairs could be very beneficial and cost-effective in developing and developed countries.

- Road communication is an important factor in effective post-earthquake relief operations. The identification of potential earthquake induced slope failures and rock slides could do much to ensure unhindered post-earthquake access, and could be incorporated in normal maintenance programmes at reasonable cost.

6.3 Particular Lessons from the 1985 Chilean earthquake

6.3.1 Value of the Earthquake for Further Study

The 3rd March earthquake is an important one to study for three main reasons:

i. There was a good spread of strong motion recordings offering a data source, the scope of which is rare outside Japan and California.

ii. The epicentral area contained many well designed and constructed engineered structures, some of which performed well while others did not. The infrastructure in the epicentral area had many features of a developed country.

iii. The earthquake was a major event and affected a large area, providing a wealth of data.

6.3.2 Damage to Reinforced Concrete Shear Wall Structures

A number of modern shear wall structures were damaged, some seriously, though none collapsed, or as far as is known caused fatalities. Examples of shear wall failures are comparatively rare, and the data from this earthquake would repay further study.

6.3.3 Low Cost Housing

In regions like Chile, there is a serious need for low cost, engineered housing to improve living standards over traditional forms of housing, including improving earthquake resistance. There were several interesting failures in existing low cost housing, not always for obvious reasons, and further study would be beneficial.

6.3.4 Soil Liquefaction

Complete liquefaction of sandy soils (ie. where the effective stress falls to zero) may not have occurred, but increases in pore water pressures, during and immediately after the earthquake, certainly did, leading to significant loss of soil strength and failures (eg. San Antonio docks). Soil liquefaction is a very important phenomenon, not only for seismic but also wave loaded structures, but its investigation is hampered by the lack of full scale failure data to corroborate laboratory results. The failures experienced in Chile should provide an important source of field data, especially since there are extensive strong motion records available.

6.3.5 Dynamic Effects

Dynamic response effects clearly played an important part in many failures. There is an urgent need to validate and calibrate seismic design codes and analytical techniques against prototype behaviour. As many well engineered structures were affected and the input motions are well defined, the earthquake offers an important opportunity for such studies to be carried out.

6.3.6 Spread of Intensity and Acceleration data

A number of interesting features can be noted, as follows.

a) A significant feature of the earthquake was the relatively high intensities and accelerations noted at considerable distances from the epicentral region. For example, Curico, about 100km from the source area, experienced MM intensities of VI to VII, which compares with a maximum intensity of VIII in the epicentral region. Intensity data given by Lastrico and Monge (Ref. 16) for the 1971 earthquake indicate a similar spread (IX in the epicentral region, VI to VII near San Antonio about 200km to the south).

b) Amplification of peak ground motions in the alluvial deposits of the Longitudinal Valley (section 3.2) were almost certainly present, which would help to explain the high intensities at large distances from the source area noted above. The amplification of long period motions is likely to have been even greater. Similar, even more dramatic, amplification was noted in Mexico City, situated some 400km from the source area of the 1985 Mexican earthquake.

c) Local increases in intensity at the top of ridges were observed during the main earthquake at Vina del Mar and elsewhere, and confirmed at Vina by measurement of aftershocks (Celebi, Ref. 10). Celebi states that this is the first time that ridge amplification effects have been confirmed in an area of dense construction.

d) For this earthquake, with a source area some 130km long and an epicentre at one end, standard acceleration formulae based on epicentral, rather than source area, distances, would give totally misleading results.

e) An area of anomalously low ground motions was recorded in the north end of the source area. The authors are not aware of any published explanation for this phenomenon.

6.3.7 Sociological Impact and Response of Relief Organisations

The resiliance and high morale of the Chilean people were quite remarkable considering the extent of the damage. This may partly have been due to the low loss of life, their experience of several major earthquakes in the last 25 years and the fact that the March 3rd earthquake occurred in late summer when weather conditions were still good.

The EEFIT team was impressed with the speed of response of relief work organised by local and national government agencies, including the public utilities. Demolition of unsafe structures was progressing rapidly within a few days of the earthquake, water supply, sewerage, electricity and telephone systems were quickly being repaired, and temporary bridges over major crossings (eg. over the River Maipo) were under construction. The relief operations appeared to be well organised, which helped to offset any lack of resources.

6.4 Implications for Future Great Earthquakes in Chile

Although a large event, the 1985 earthquake is not as large as the great event (M_s > 8.0) to be expected in the near future, on the basis of the previous seismic history (see Section 3.3). A great earthquake would give rise to epicentral accelerations, which, while not necessarily of higher amplitude or different frequency content than in 1985 (see Figure 4.2), would be likely to last for longer. Moreover, the epicentral region of a great earthquake would be larger than in 1985 and would be prone to land deformations likely to damage facilities such as lifelines and dams. These are factors influencing relief operations after a great earthquake, which in these respects would be more severe than the 1985 event.

The 1985 earthquake provided much evidence supporting the notion that the latest Chilean code of practice for earthquake resistant design, if applied correctly, will ensure that buildings can withstand such motion safely. However, many structures constructed before the introduction of the current code performed badly and are likely to do so again in stronger earthquakes. The Chilean authorities and engineers were clearly trying to learn as much as possible from the disaster to improve design methods, to identify weaknesses and necessary repair and strengthening works in existing structures, and to improve the effectiveness of the relief operations.

Inevitably, a great earthquake in the near future will cause considerable damage. However, it is likely that the lessons learned from the 1985 event will significantly mitigate the effects of a great earthquake in the future.

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Table 3.1 Large Earthquakes in Central Chile Since 1570
(from Lomnitz, Ref. 19)

Date	Epicentre	M	Comments
1570 Feb 8	Concepcion	about 8	Tsunami
1647 May 13	Santiago	about 8.5	1000 dead
1657 Mar 15	Concepcion	about 8	Tsunami
1730 Jul 8	Valparaiso	about 8.5	Tsunami
1751 May 25	Concepcion	about 8.5	Tsunami
1822 Nov 19	Valparaiso	about 8	72 dead
1835 Feb 20	Concepcion	about 8.5	Tsunami
1847 Oct 8	Illapel		
1851 Apr 2	Casablanca		
1873 Jul 7	La Ligua		
1906 Aug 7	Valparaiso	8.6	
1928 Dec 1	Talca	8.3	
1939 Jan 25	Chillan	8.3	30,000 dead
1943 Apr 6	Illapel	8.3	11 dead
1965 Mar 28	La Ligua	7.3	
1971 Jul 8	La Ligua	7.5	
1985 Mar 3	Valparaiso	7.4	170 dead

Table 4.1 Uncorrected Peak acceleration from strong motion records, Chilean earthquake of 3rd March 1985 (based on Saragoni, Ref.1)

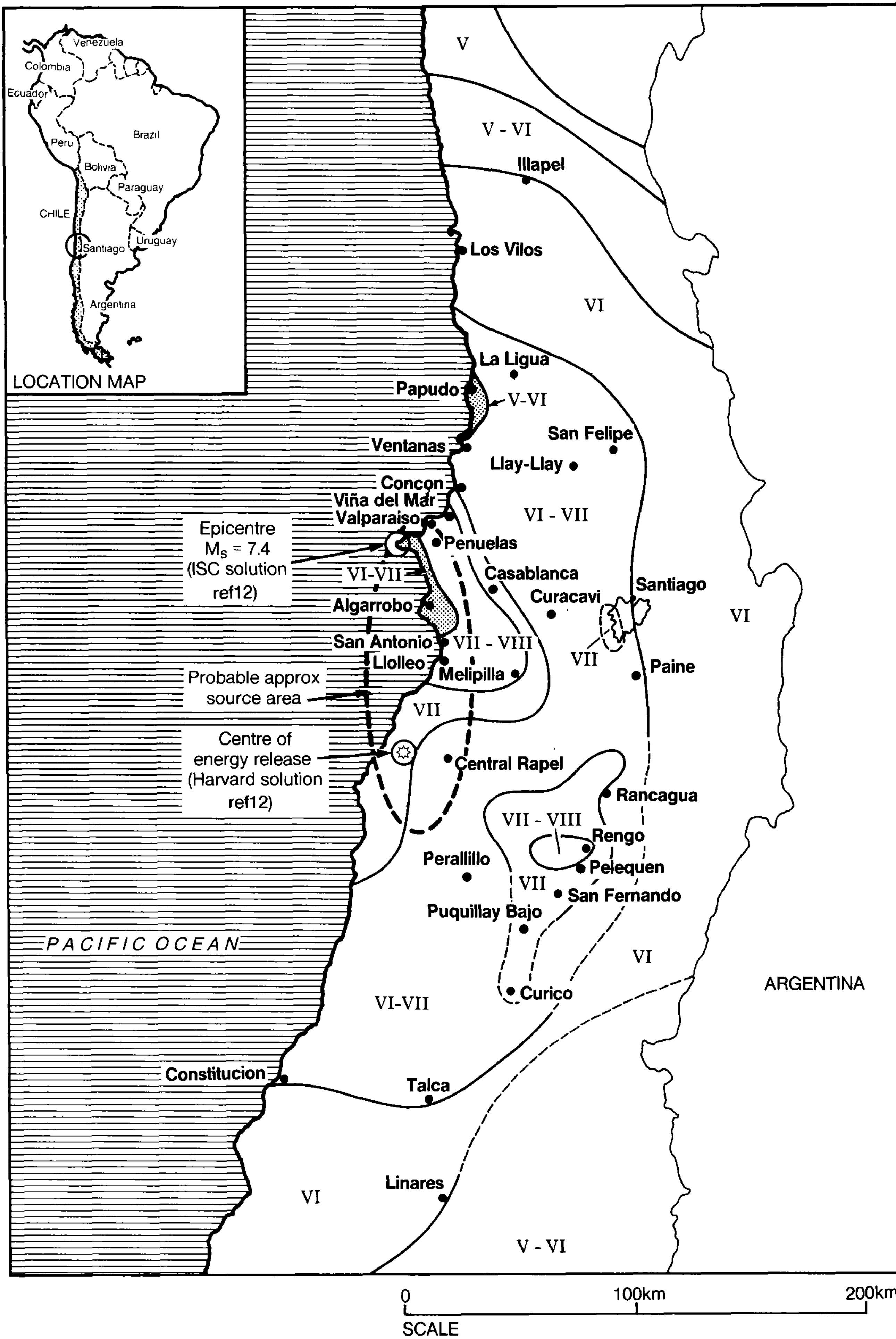
SITE	COORDINATES	MAXIMUM ACCELERATION (g)	SITE GEOLOGY	INSTRUMENT LOCATION	MODEL
Illapel (D.I.C.)	31°38'S 71°10'W	N20W: 0.120 S70W: 0.100 Vertical: 0.056	Soft Alluvium	1-storey bldg, Basement	SMA-1
Los Vilos (D.G.G.)	31°55'S 71°30'W	NS: (0.025)** EW: 0.036 Vertical: 0.021	Sedimentary Rock	1-storey bldg, Ground Level	SMA-1
La Ligua (D.I.C.)	32°30'S 71°06'W	N70W: 0.193 S20W: 0.126 Vertical	Alluvium -	1-storey bldg, Ground Level	SMA-1
Papudo (C.CH.E.N.)	32°31'S 71°27'W	N50E: 0.128 S40E: (0.469)** Vertical: 0.110	Granite	1-storey bldg, Ground Level	SMA-1
Zapallar (D.G.G.)	32°34'S 71°28'W	NS: 0.317 EW: 0.332 Vertical: 0.228	Granite	1-storey bldg, Ground Level	SMA-1
Ventanas (CHILECTRA)	33°02'S 71°37'W	NS: 0.180 EW: 0.180 Vertical: 0.140	-	-	-
San Felipe (D.I.C.)	32°45'S 70°44'W	S10E: 0.348 N80E: 0.430 Vertical: 0.208	Alluvium	1-storey bldg, Ground Level	SMA-1
Llay-Llay (D.I.C.)	32°50'S 70°58'W	N80W: 0.335 S10W: 0.486 Vertical: -	Soft Alluvium	1-storey bldg, Ground Level	SMA-1
Saladillo (ANDINA)	32°55'S 70°28'W	NS: 0.106 EW: 0.085 Vertical: 0.052	-	-	-
San Pedro (ENDESA)	32°55'S 71°30'W	NS: 0.600 EW: 0.570 Vertical: 0.375	-	-	SMA-1
Vina del Mar (D.I.C.)	33°02'S 71°35'W	N70W: 0.228(+) S20W: 0.356(+) Vertical: 0.171(+)	Sandstone and Volcanic Rock	10-storey bldg, Basement	SMA-1
Valparaiso (El Almendral) (D.I.C.)	33°01'S 71°38'W	N50E: 0.293(+) S40E: 0.163(+) Vertical: -	Soil		SMA-1
Valparaiso (University) (D.I.C.)	33°01'S 71°38'W	S20E: 0.164(+) N70E: 0.179(+) Vertical: 0.125(+)	Volcanic Rock	1-storey bldg, Ground Level	SMA-1
Peldehue (D.I.C.)	33°08'S 70°41'W	EW: (0.640)** NS:- Vertical:-	Volcanic Rock and Alluvium	1-storey bldg, Ground Level	DG-2
Quintay (ENDESA)	33°16'S 71°19'W	NS: 0.203 EW: 0.182 Vertical: 0.131	Loose Sand	-	SMA-1
Santiago (ENDESA)	33°27'S 70°40'W	NS: 0.114 EW: 0.114 Vertical: 0.073	-	-	Japanese

Table 4.1 (Cont'd)...

SITE	COORDINATES	MAXIMUM ACCELERATION (g)		SITE GEOLOGY	INSTRUMENT LOCATION	MODEL
Llolleo (D.I.C.)	33°41'S 71°36'W	S80E: N10E: Vertical:	0.426(+) 0.669(+) 0.852(+)	Sandstone and Volcanic Rock	1-storey bldg, Basement	SMA-1
Melipilla (D.G.G.)	33°41'S 71°13'W	EW: NS: Vertical:	0.599 0.671 0.593	Granite Rock	1-storey bldg, Ground Level	SMA-1
Rapel (ENDESA)	34°01'S 71°40'W	NS: EW: Vertical:	0.310 0.144 0.110	Marine and Continental Sediments	Tunnel, Ground Level	RFT-250
Pichilemu (D.G.G.)	34°23'S 72°01'W	NS: EW: Vertical:	0.266 0.180 0.132	Slates, Sandstone, Linestone	1-storey bldg, Ground Level	SMA-1
San Fernando (D.G.G.)	34°36'S 71°00'W	NS: EW: Vertical:	0.227 0.343 0.121	Alluvium	1-storey bldg, Ground Level	SMA-1
Iloca (D.G.G.)	34°55'S 72°13'W	NS: EW: Vertical:	0.216 0.282 0.087	Sandstone, Alluvium	1-storey bldg, Ground Level	SMA-1
Hualane (D.G.G.)	34°58'S 71°49'W	NS: EW: Vertical:	0.169 0.143 0.100	Alluvium	1-storey bldg, Ground Level	SMA-1
Constitucion (D.G.G.)	35°18'S 72°19'W	NS: EW: Vertical:	0.144 0.077 0.039	Granite Rock	2-storey bldg, Ground Level	SMA-1
Talca (D.I.C.)	35°26'S 71°40'W	N80W: N10E: Vertical:	0.161 0.168 0.077	Soft Alluvium	1-storey bldg, Ground Level	SMA-1
Colbun (ENDESA)	35°43'S 71°26'W	NS: EW: Vertical:	0.037 0.034 0.022	-	-	SMA-1
El Colorado (ENDESA)	35°43'S 71°26'W	NS: EW: Vertical:	0.120 0.113 0.060	-	-	-
Pehuenche (ENDESA)	35°44'S 71°14'W	NS: EW: Vertical:	0.031 0.023 0.020	-	-	SMA-1
Cauquenes (D.G.G.)	36°00'S 72°13'W	NS: EW: Vertical:	0.086 0.117 0.046	Alluvium	2-storey bldg. Ground Level	SMA-1
Chillan Viejo (D.I.C.)	36°36'S 72°06'W	N80E: N10W: Vertical:	0.057 0.065 0.041	Soft Alluvium	-	-
Chillan Nuevo (D.I.C.)	36°40'S 72°09'W	N20W: N70E: Vertical:	0.108 0.067 0.026	-	-	-

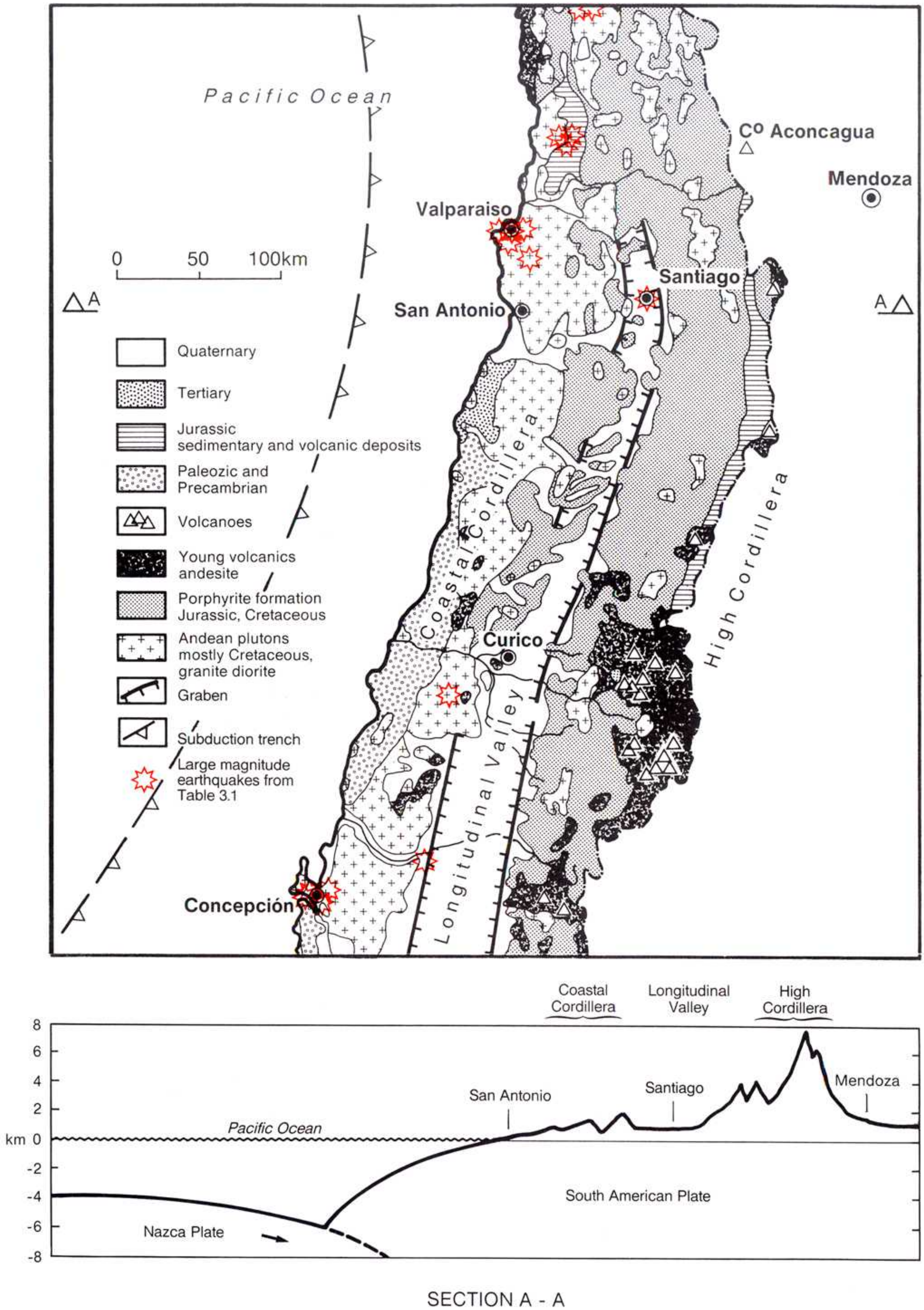
** Provisional value

(+) Value from digitised, corrected record



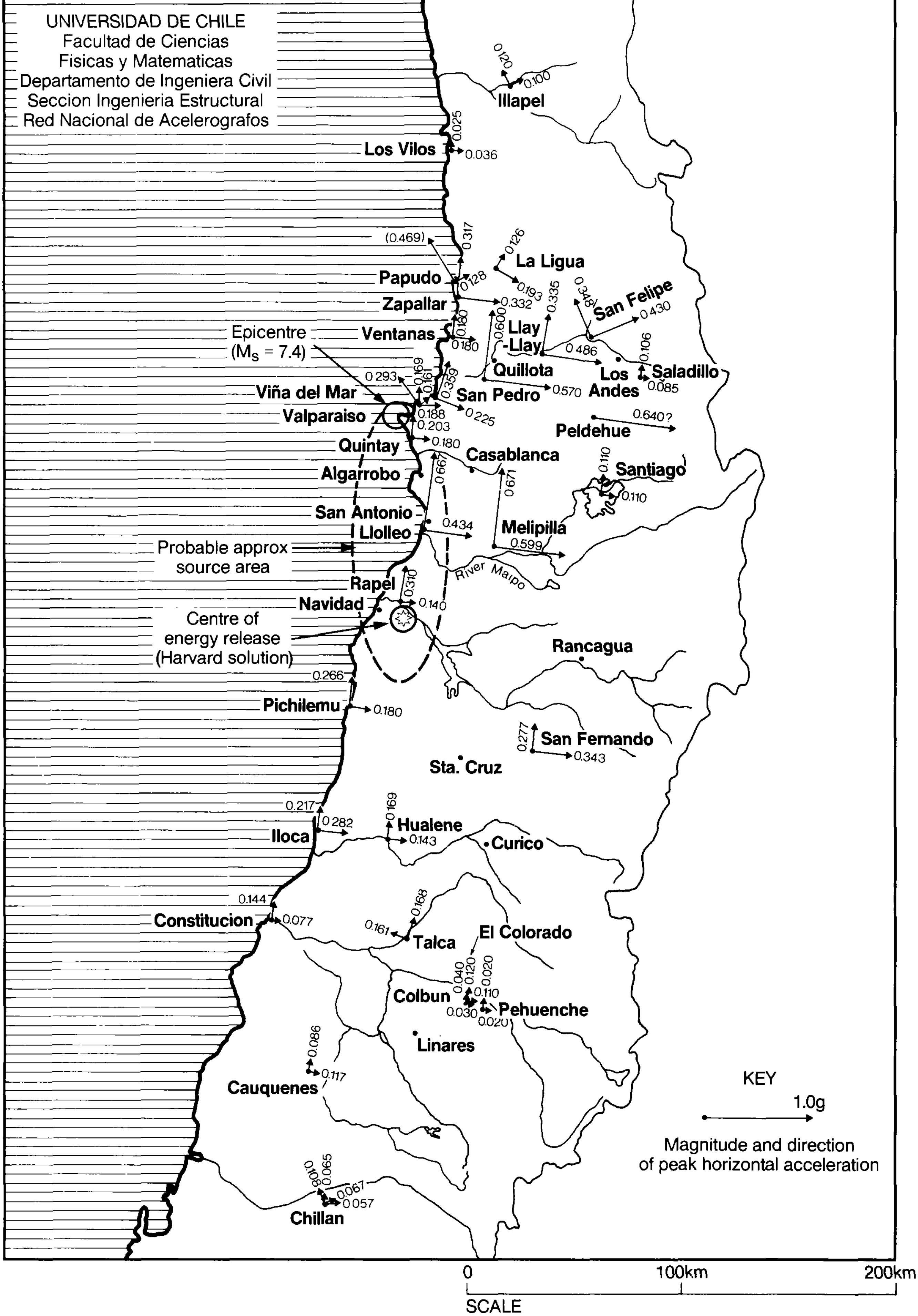
ISOSEISMAL MAP OF THE CHILE EARTHQUAKE
OF 3RD MARCH 1985 (after Saragoni, ref 1)

Figure 1.1



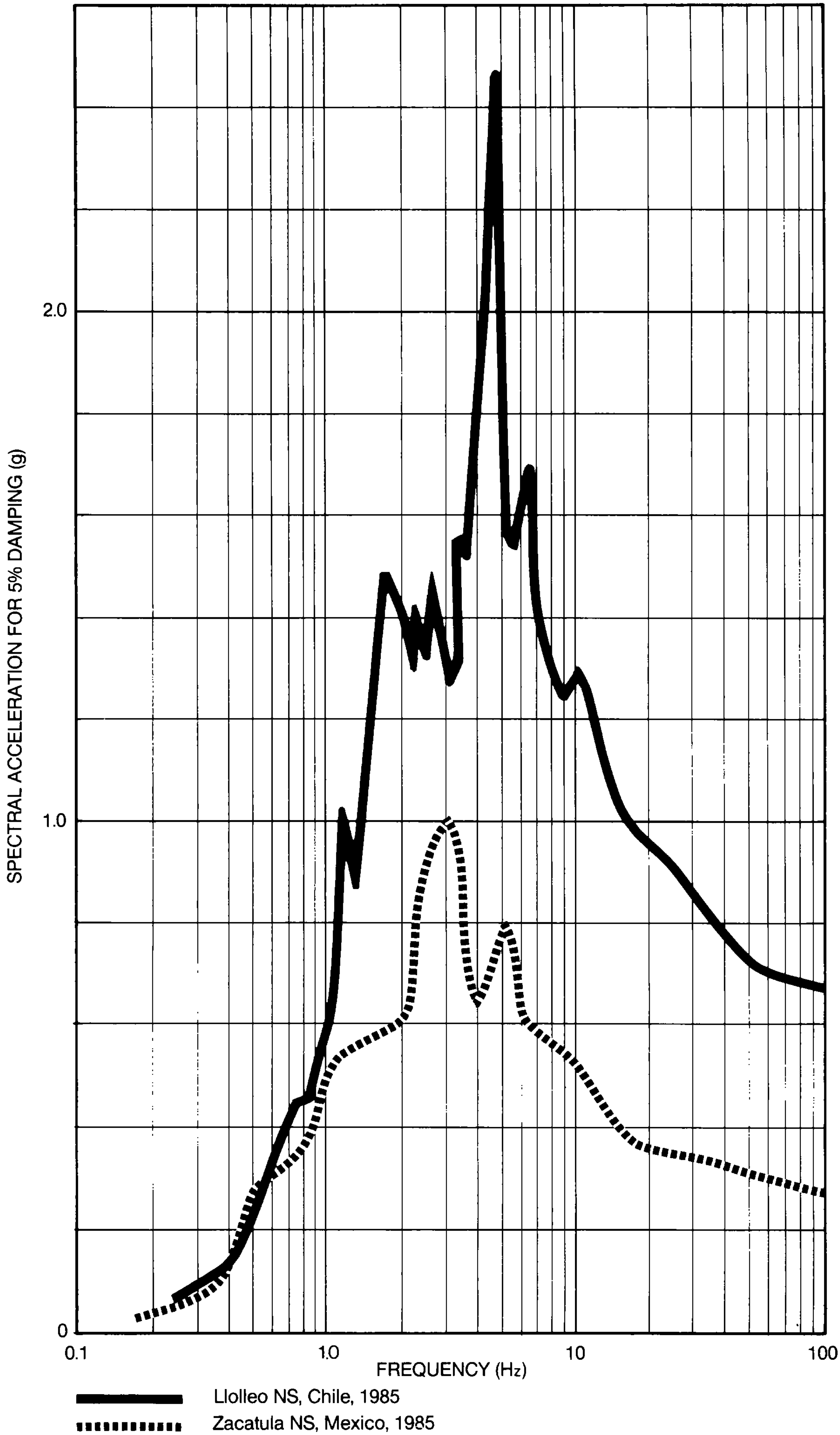
GEOLOGY OF CENTRAL CHILE
(after Ziel, ref 20)

Figure 3.1



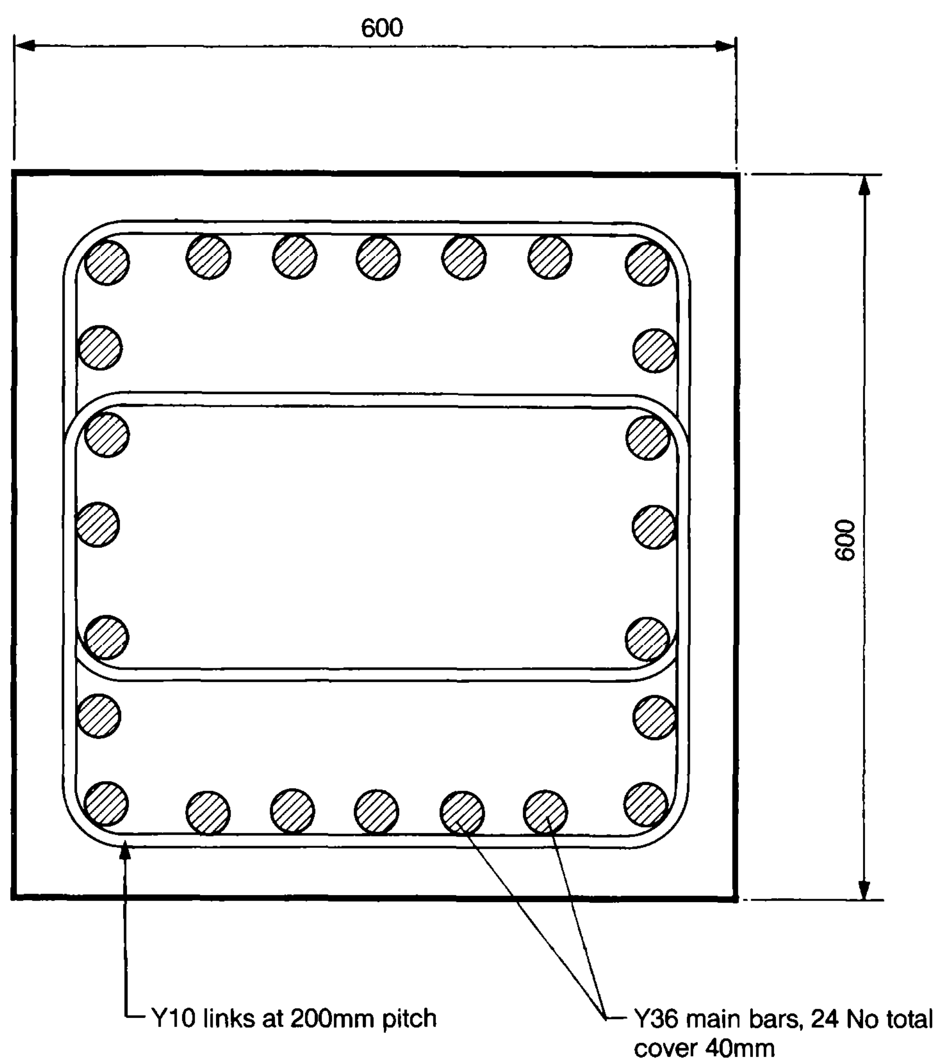
PEAK HORIZONTAL ACCELERATIONS RECORDED
IN THE CHILE EARTHQUAKE OF 3RD MARCH 1985
(after Saragoni, Ref. 1)

Figure 4.1



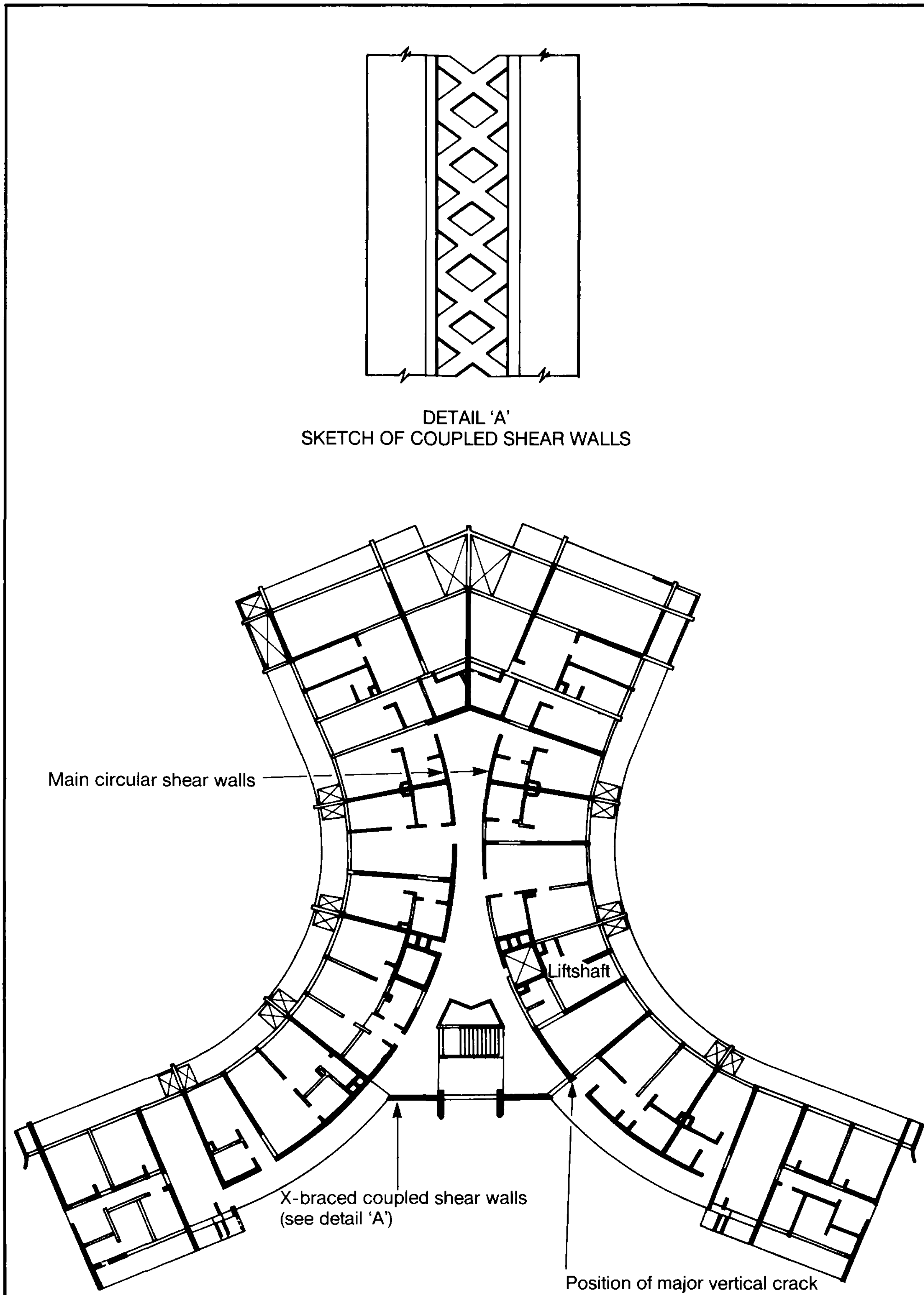
EPICENTRAL ACCELERATION SPECTRA

Figure 4.2



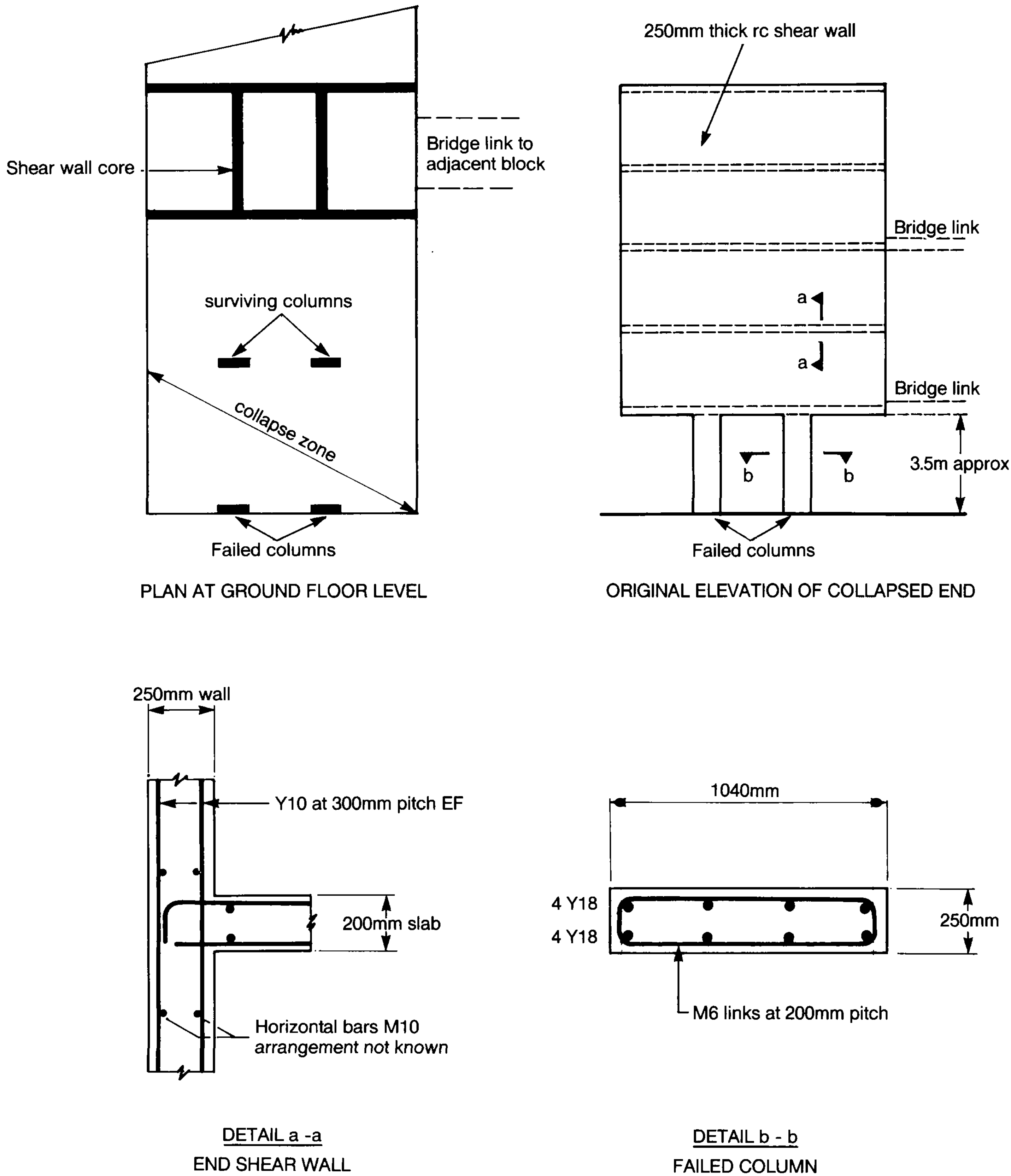
CLAUDIO VICUNA HOSPITAL, SAN ANTONIO
SKETCH OF REINFORCEMENT IN COLUMN
SHOWN IN PLATE 5.1

Figure 5.1



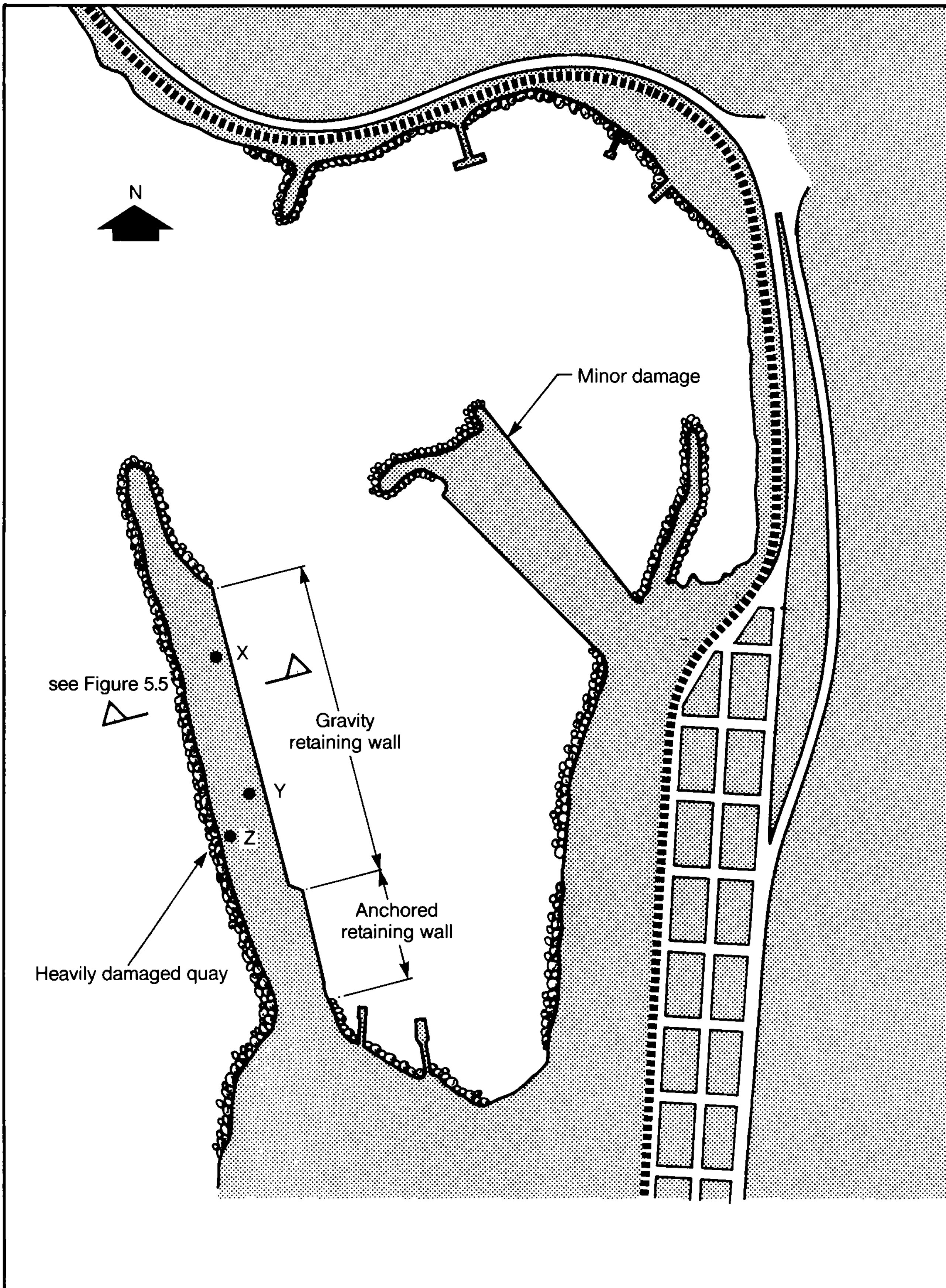
PLAN VIEW OF HANGAROA APARTMENTS,
VINA DEL MAR

Figure 5.2



VILLA OLYMPIA, SANTIAGO
SKETCH OF FRAMING OF BLOCK 72

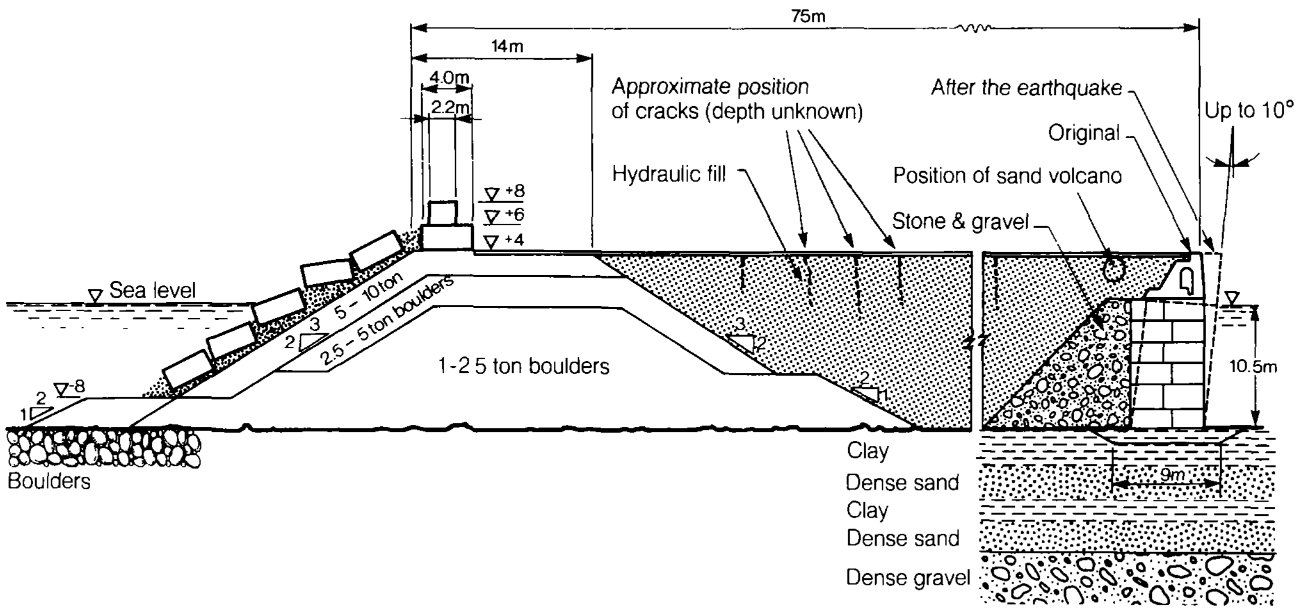
Figure 5.3



NOTE: For views at locations X, Y & Z, see plate 5.42

SAN ANTONIO DOCKS

Figure 5.4



CROSS SECTION THROUGH DAMAGED QUAY,
SAN ANTONIO

Figure 5.5

PLATE 5.1 Compression failure in external columns
Claudio Vicuna Hospital, San Antonio

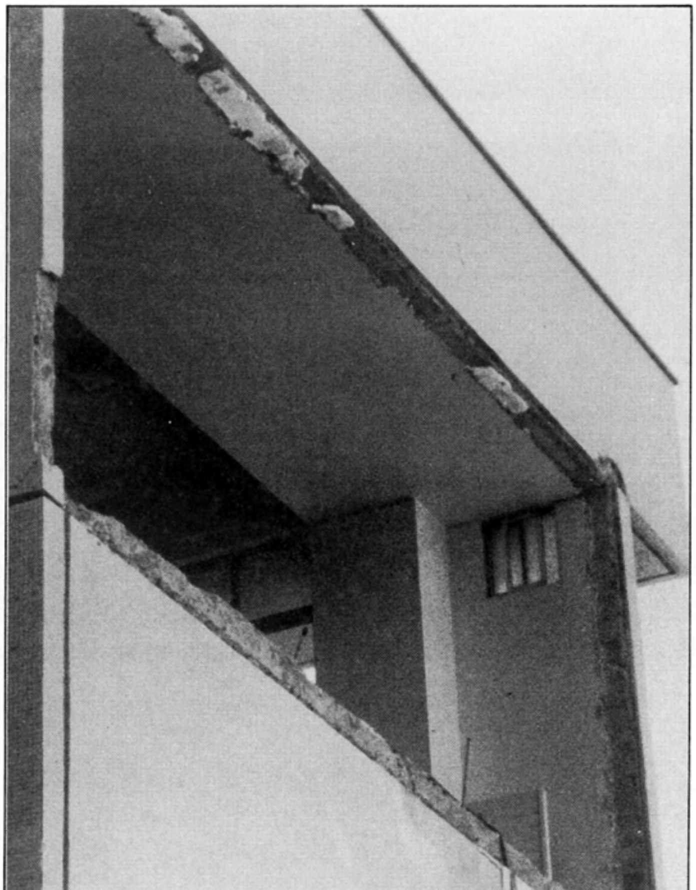


PLATE 5.2 Failed infill panel, Claudio Vicuna Hospital,
San Antonio

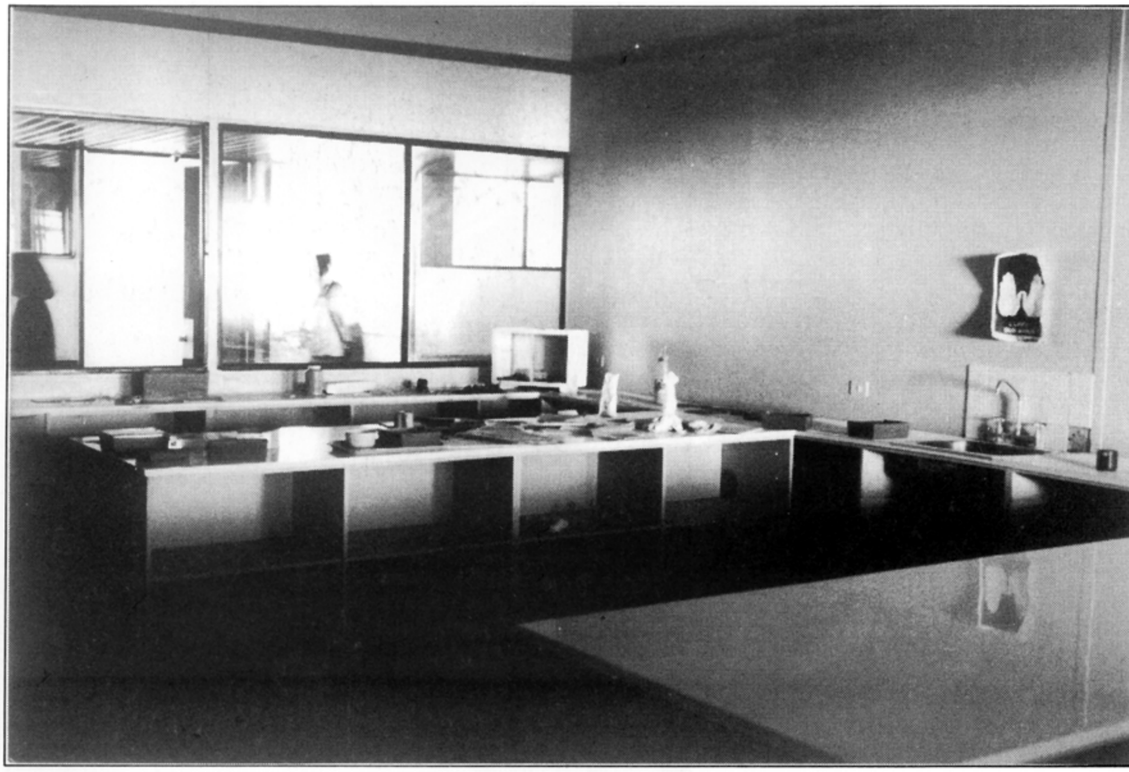


PLATE 5.3 Interior of Claudio Vicuna Hospital, San Antonio

PLATE 5.4 Large settlements in 2 storey building on sloping ground, San Antonio

PLATE 5.5 Modern undamaged buildings near San Antonio waterfront

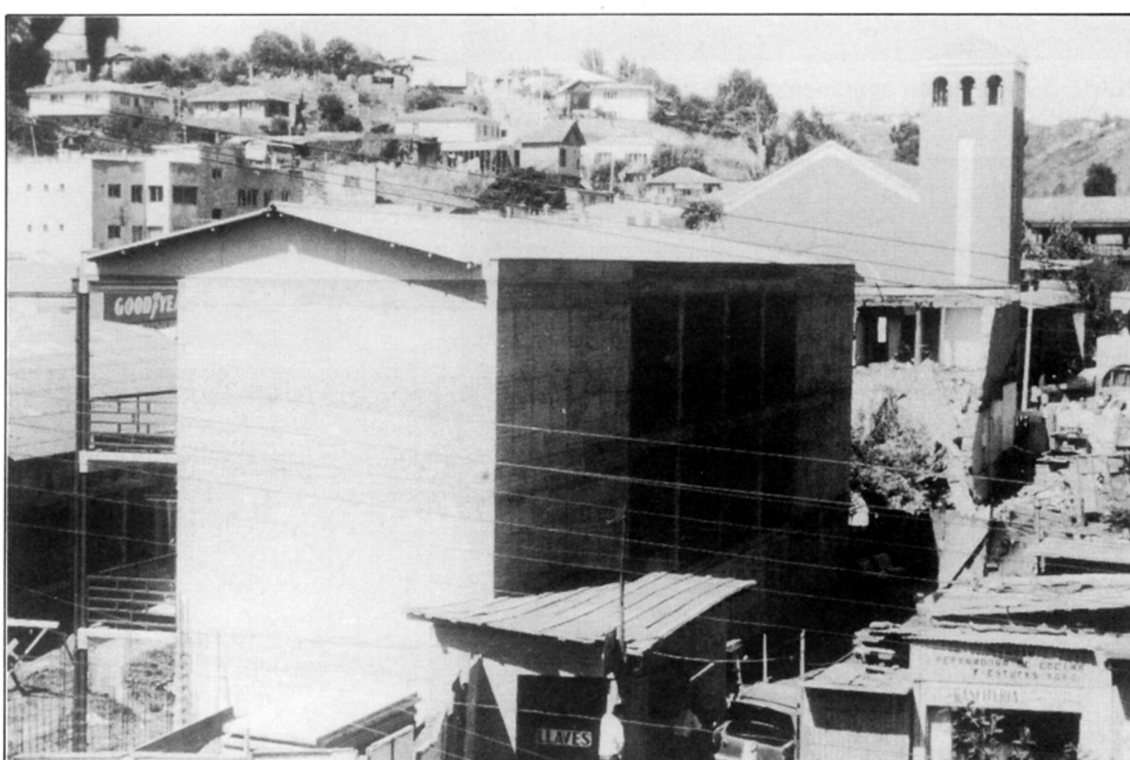


PLATE 5.6 Undamaged infill frame building on sloping ground, San Antonio

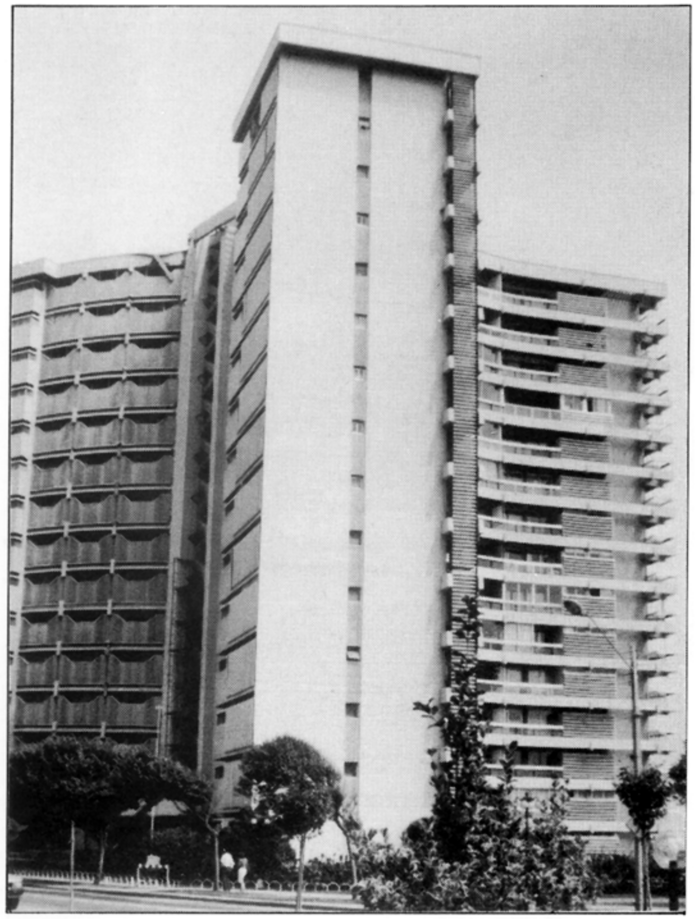


PLATE 5.7 Hangaroa apartments, Viña del Mar

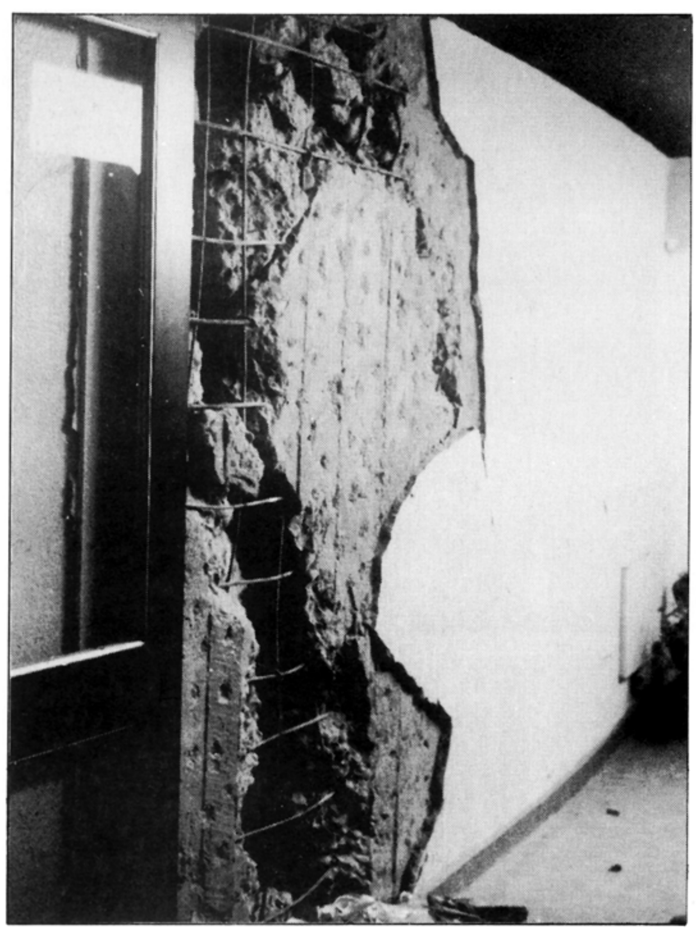


PLATE 5.8 Major vertical crack in Hangaroa apartments

PLATE 5.9 Patch repair to previous earthquake damage, Acapulco apartments, Viña del Mar

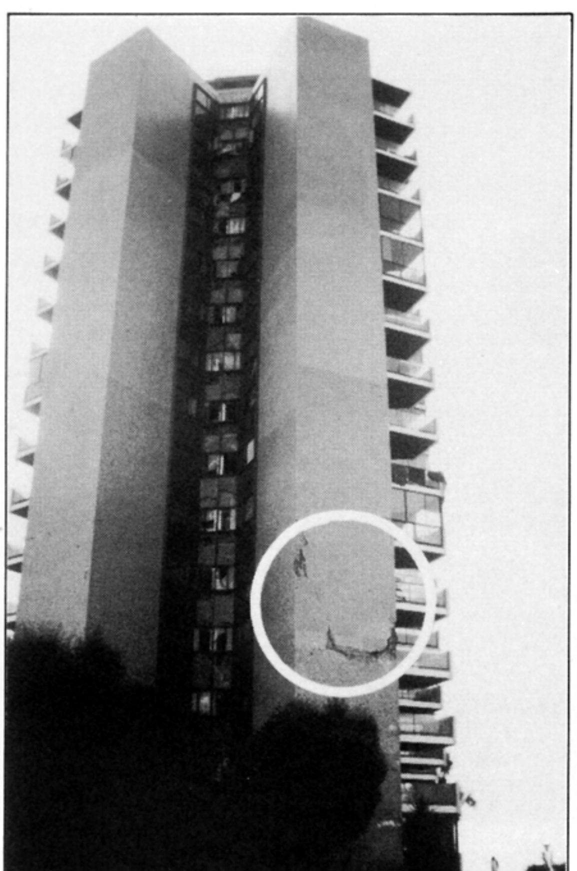
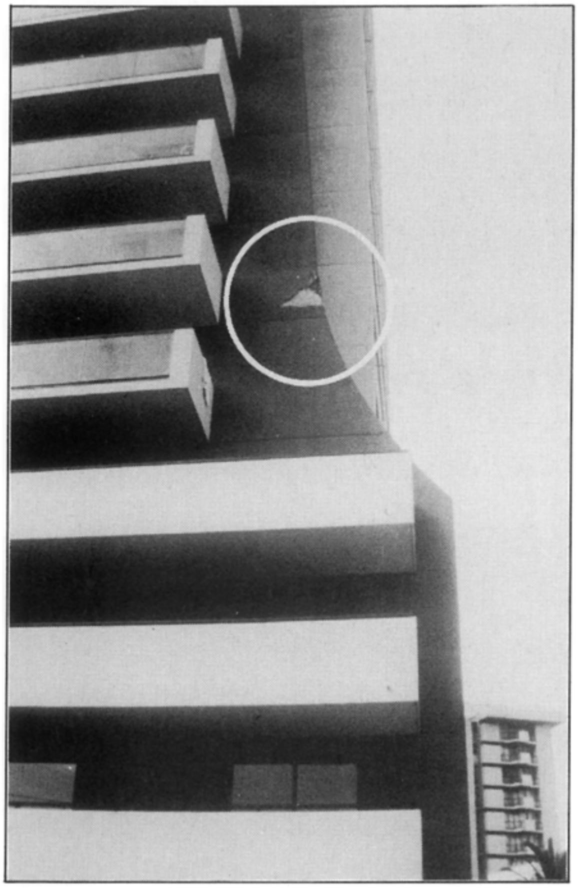


PLATE 5.10(a) Elevation
Damaged shear wall, Acapulco apartments

PLATE 5.10(b) Detail



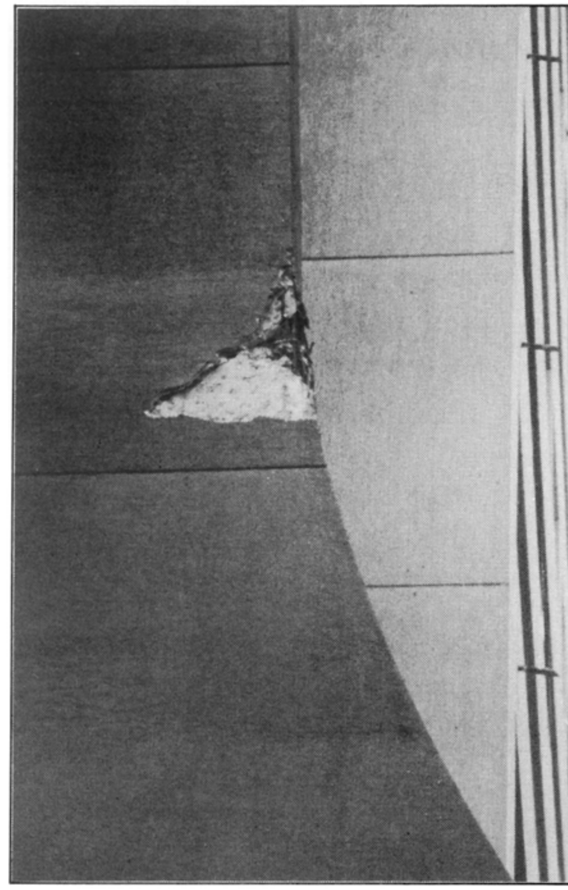


PLATE 5.11(a) Elevation PLATE 5.11(b) Detail
Spalling in shear wall of building adjacent to Acapulco apartments, Viña del Mar

PLATE 5.12 Canal Beagle estate, Viña del Mar

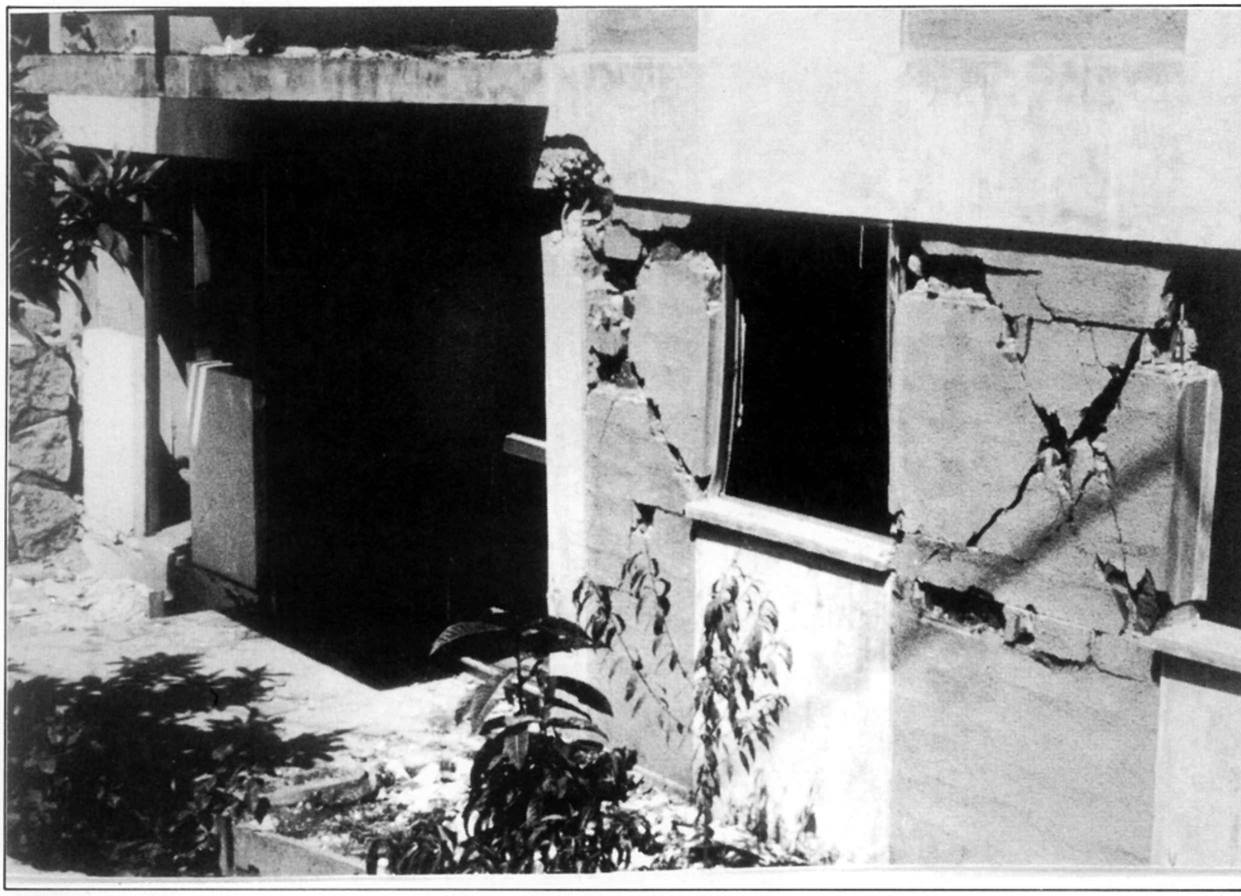


PLATE 5.13 Damage at Canal Beagle estate, Viña del Mar

Full height shear wall
added after earthquake
damage in 1971

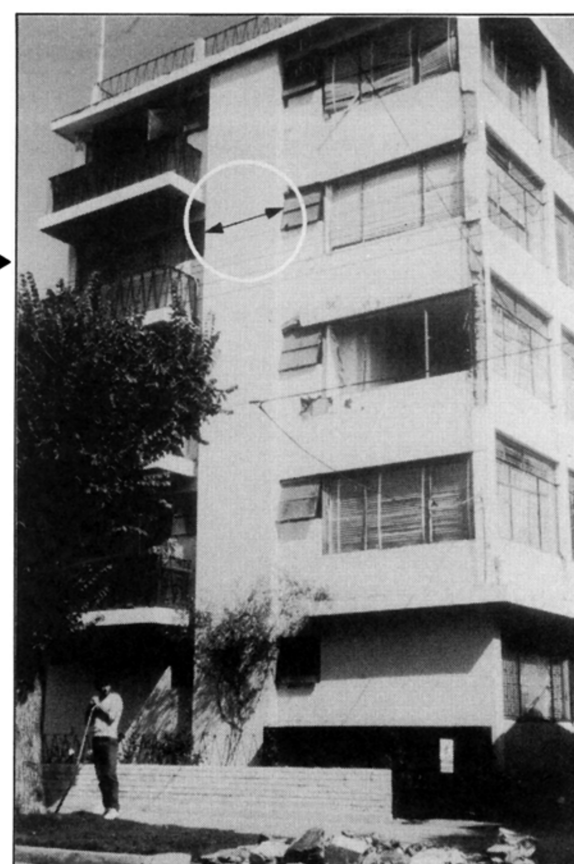
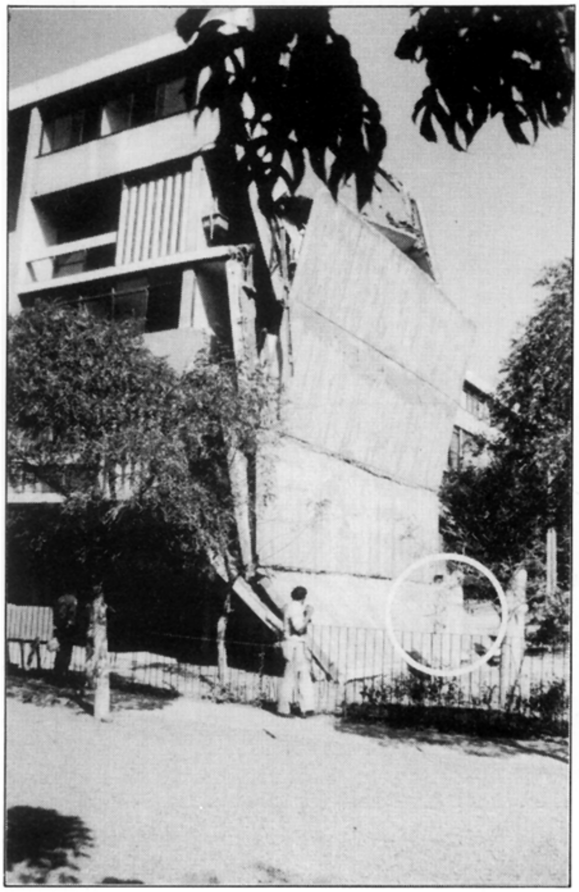


PLATE 5.14 Barrios Edificio, Viña del Mar

PLATE 5.15 Damaged bridge links in Viña del Mar building



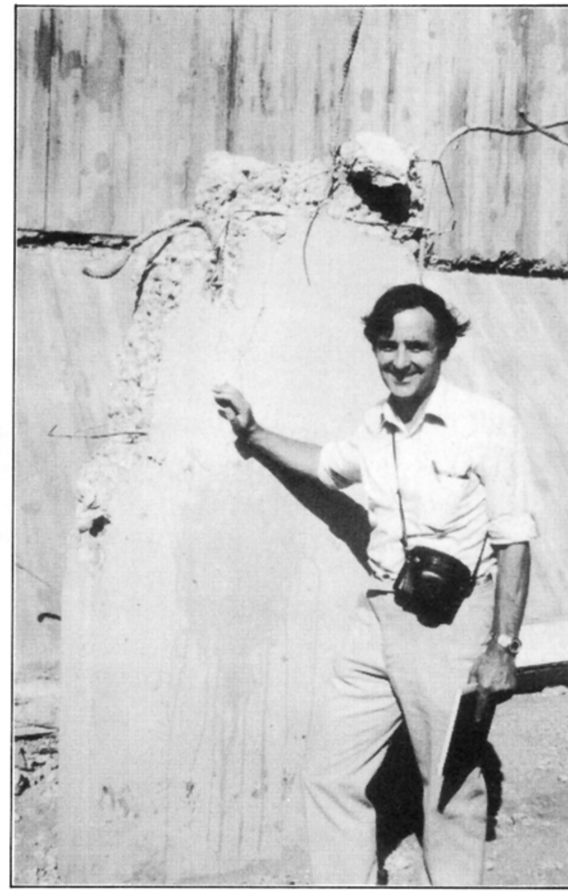


PLATE 5.16(a) Elevation
Collapsed end of block at Villa Olympia, Santiago

PLATE 5.16(b) Detail

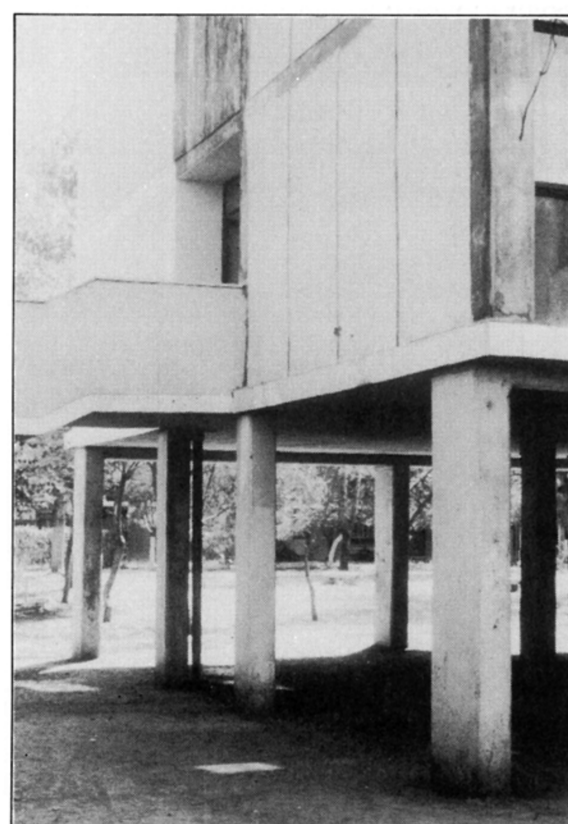


PLATE 5.17 Undamaged block at Villa Olympia with similarities to block in PLATE 5.16

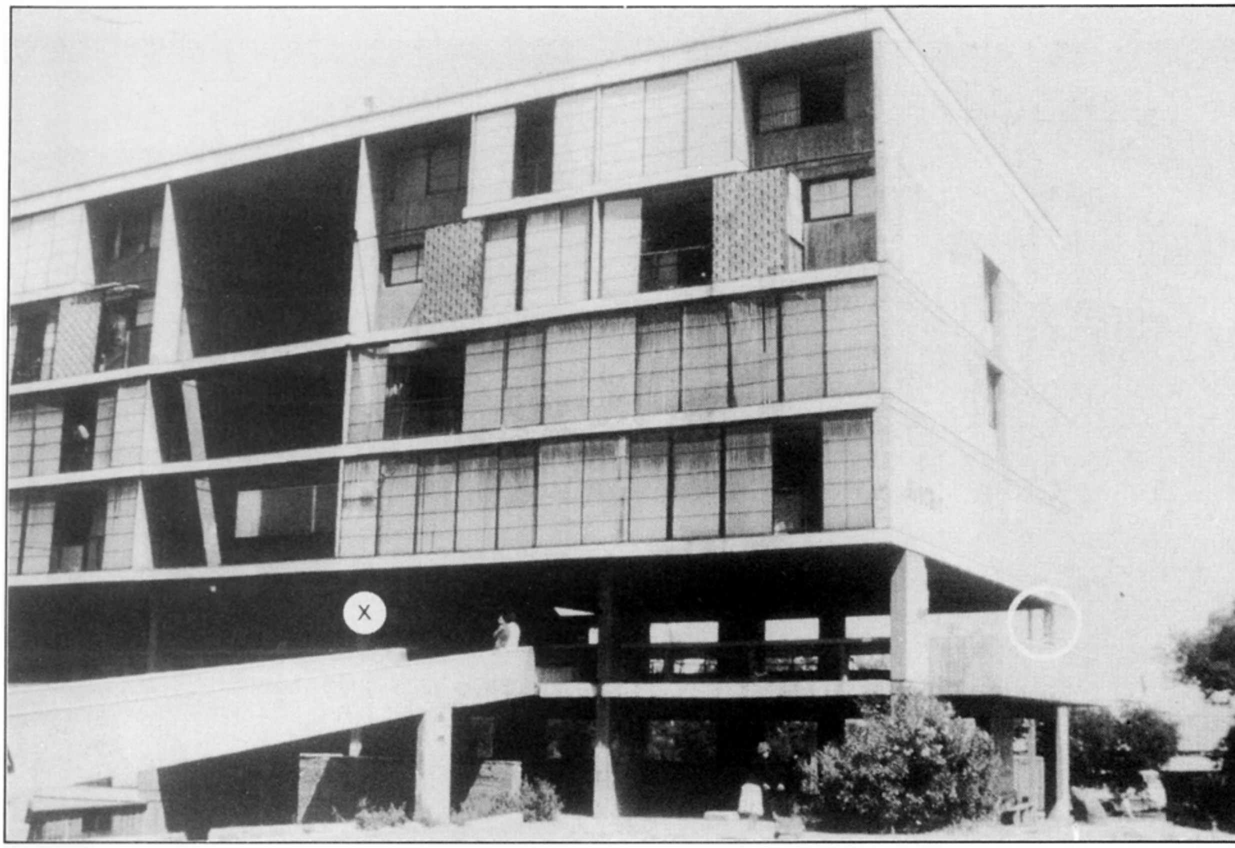


PLATE 5.18(a) Elevation

Location X
PLATE 5.20

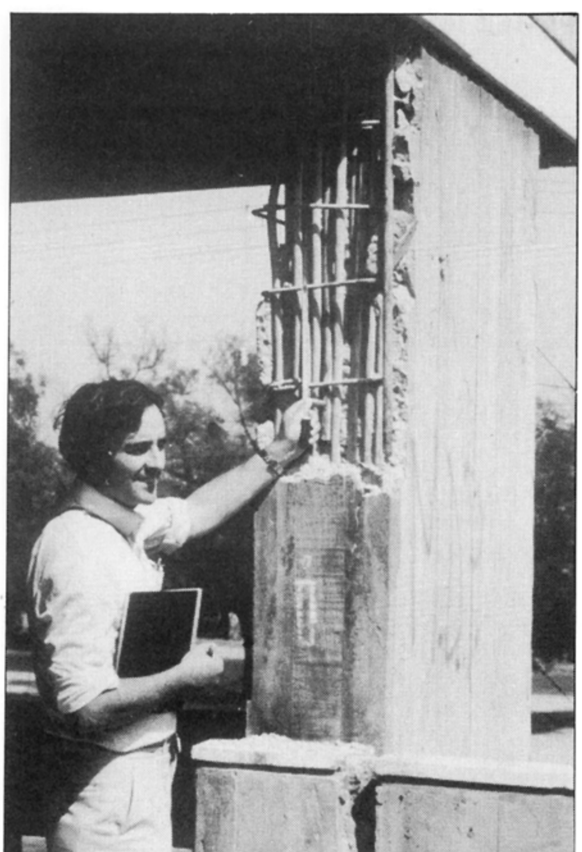


PLATE 5.18(b) Damage to end of Block 1,
Villa Portales, Santiago

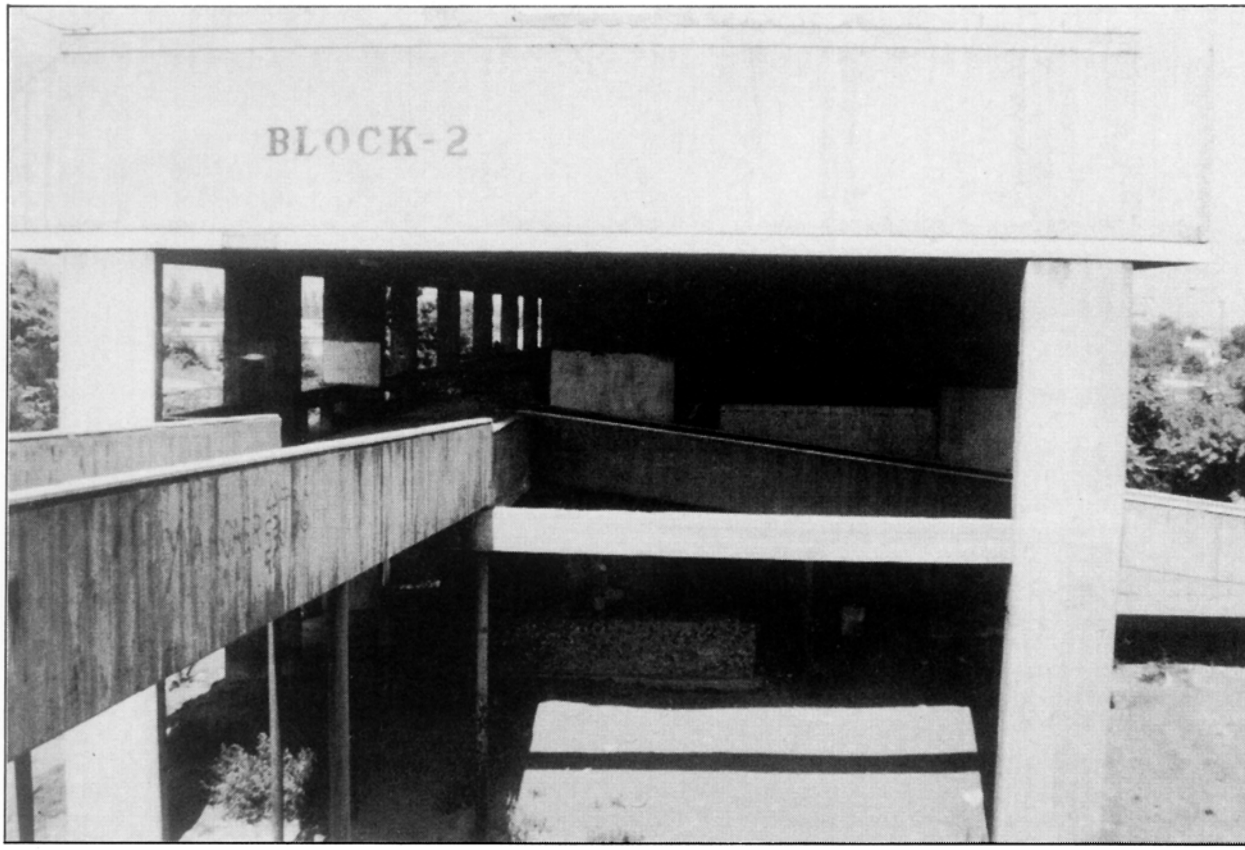


PLATE 5.19 Undamaged block parallel and adjacent to Block 1, with similar layout, Villa Portales

PLATE 5.20 Damage to internal shear wall in Villa Portales (location X, PLATE 5.18)

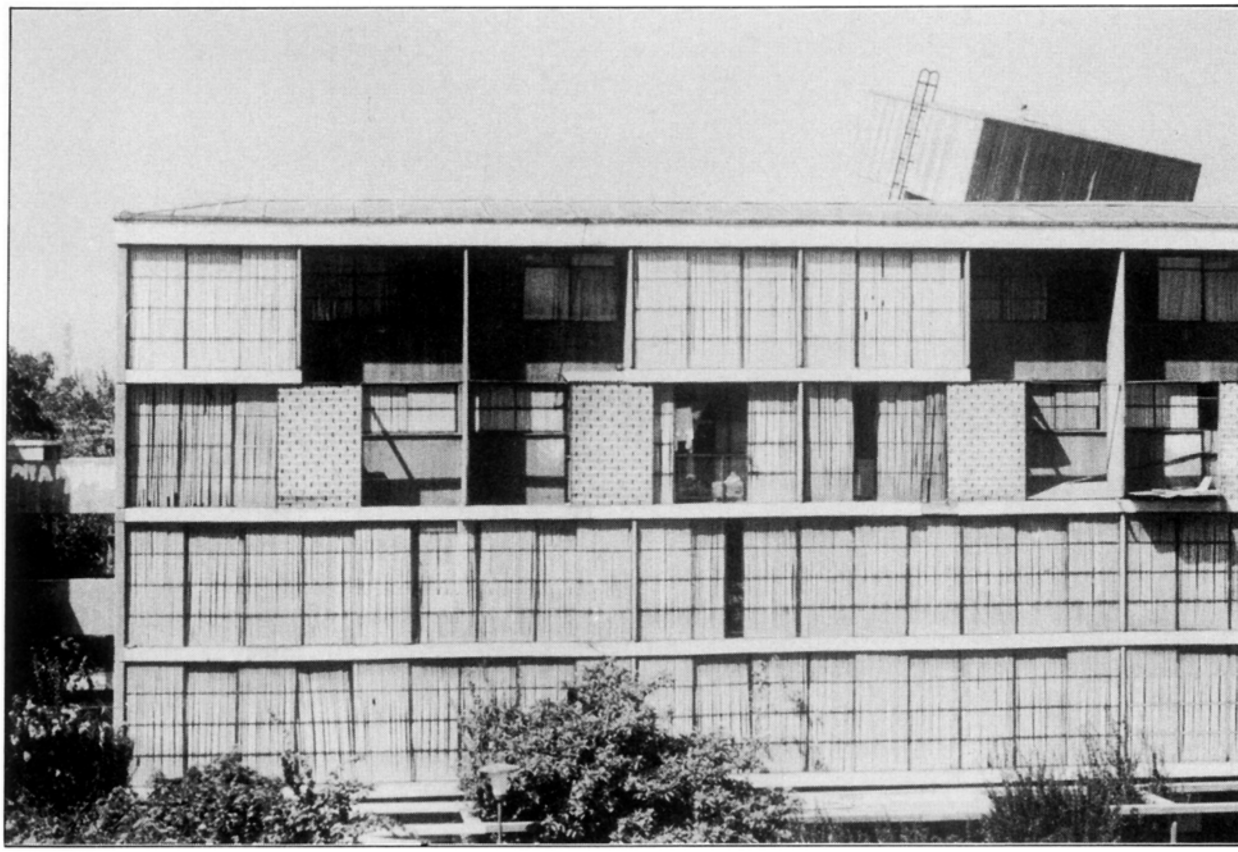


PLATE 5.21 Damaged roof tank, Villa Portales, Santiago

PLATE 5.22 Expansion loop in water pipe crossing expansion joint, Villa Portales, Santiago

PLATE 5.23(a) Elevation

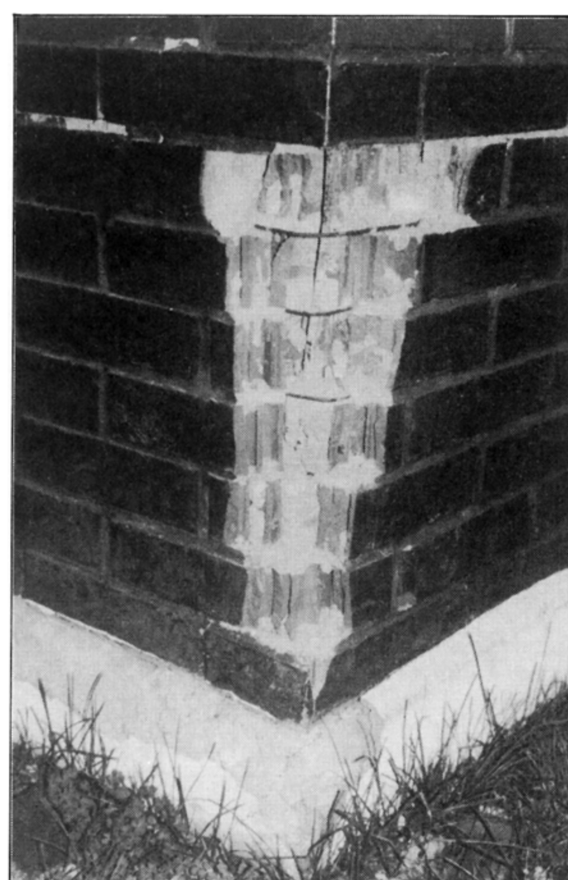


PLATE 5.23(b) Detail of damaged brickwork
Villa Santa Carolina, Santiago

PLATE 5.24(a) Heavily damaged masonry houses

PLATE 5.24(b) Undamaged wooden house
Comparitive damage in a San Antonio street

PLATE 5.25 Museum of archeology and natural history, Viña del Mar

PLATE 5.26(a) Elevation

PLATE 5.26(b) Detail of spire
Church at Llolleo

PLATE 5.27 Religious building, Llolleo

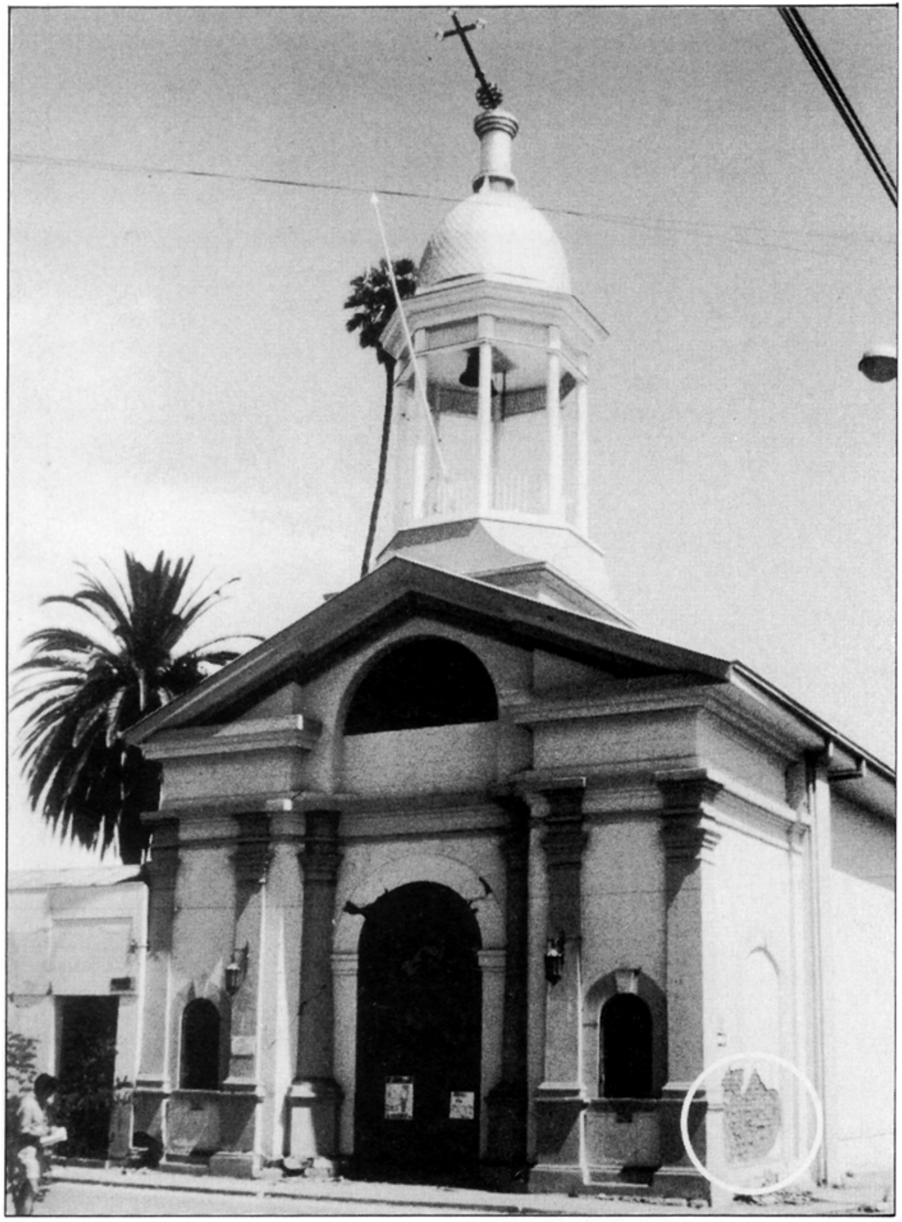


PLATE 5.28(a) Elevation. Note tilted cross

PLATE 5.28(b) Detail of cracked adobe bricks
Church at Melipilla

PLATE 5.29(a) External view

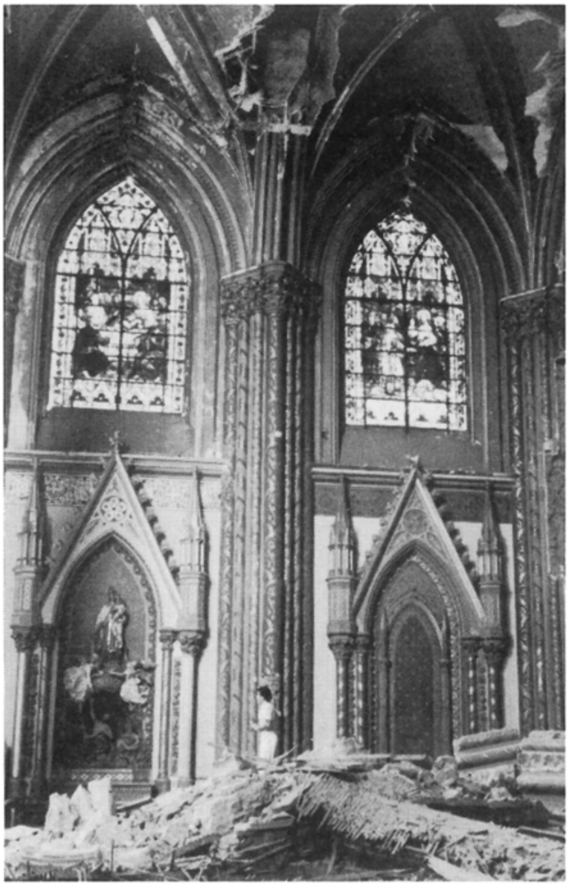


PLATE 5.29(b) Interior view
Basilica del Salvador, Santiago

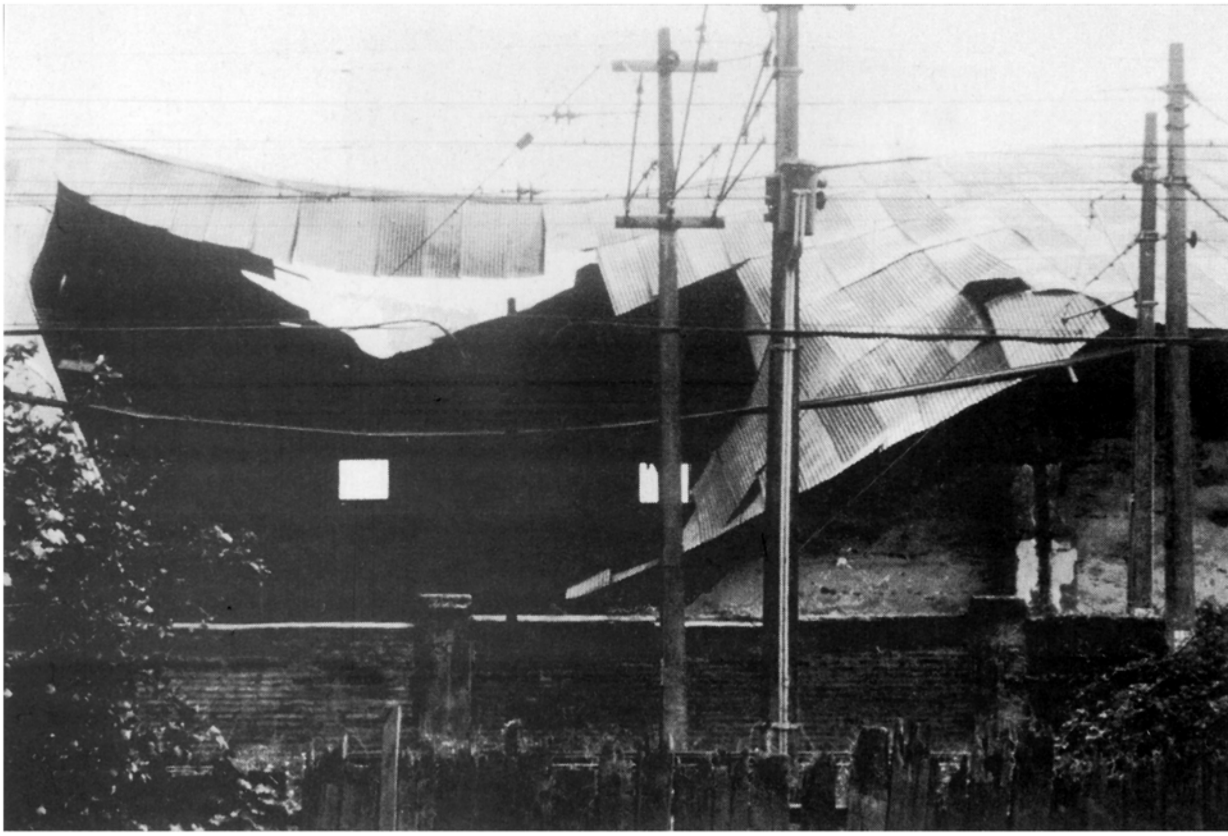


PLATE 5.30 Adobe warehouse, Rengo

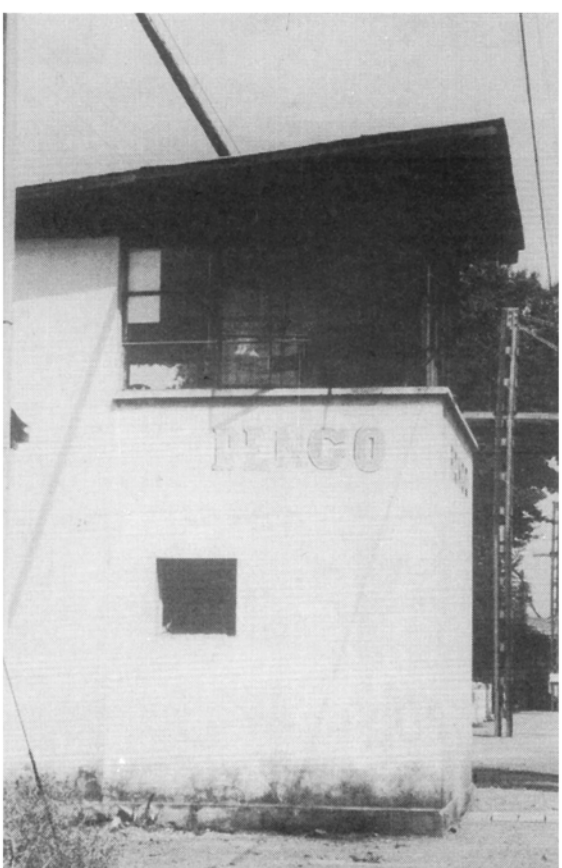


PLATE 5.31 Signal box, Rengo

PLATE 5.32 Santuar Santa Rosa, Pelequan

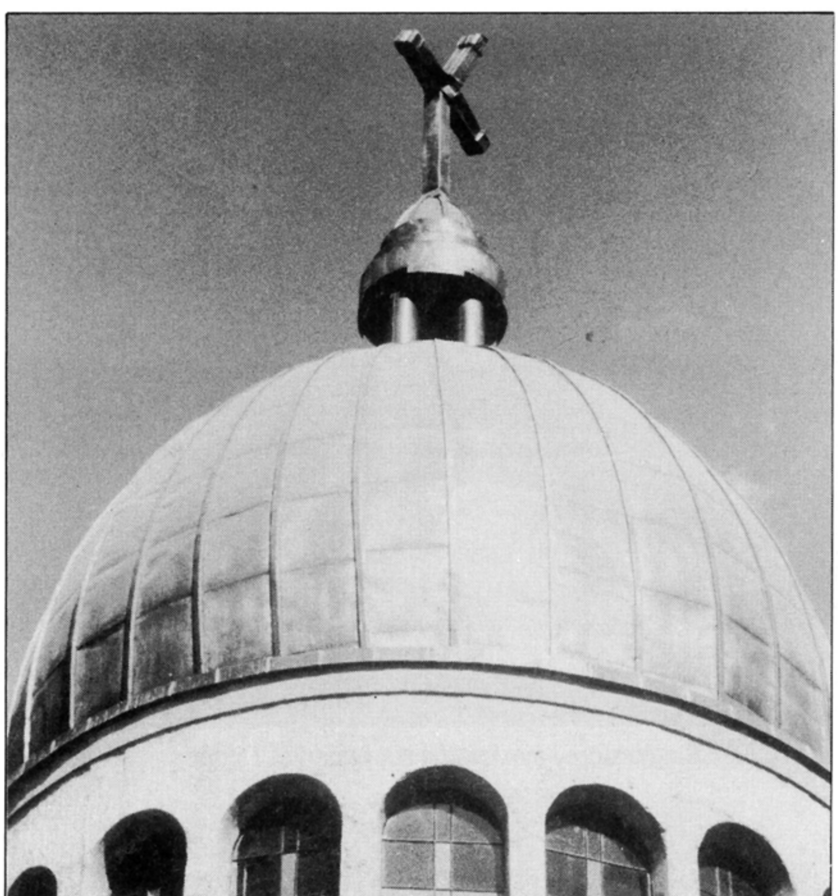


PLATE 5.33 Cross on newer church, Curico

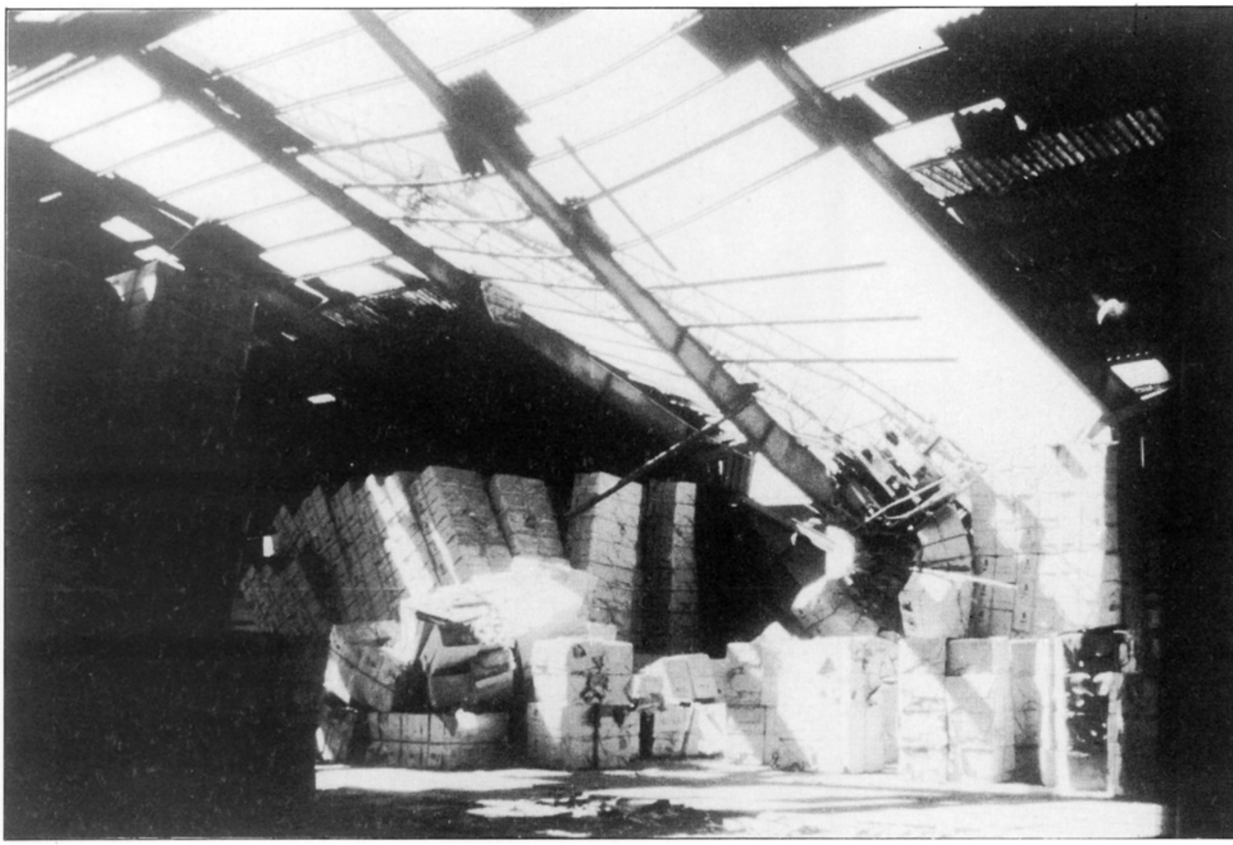


PLATE 5.34 Portal frame shed, San Antonio docks

PLATE 5.35 Single storey steel frame supermarket, Rengo

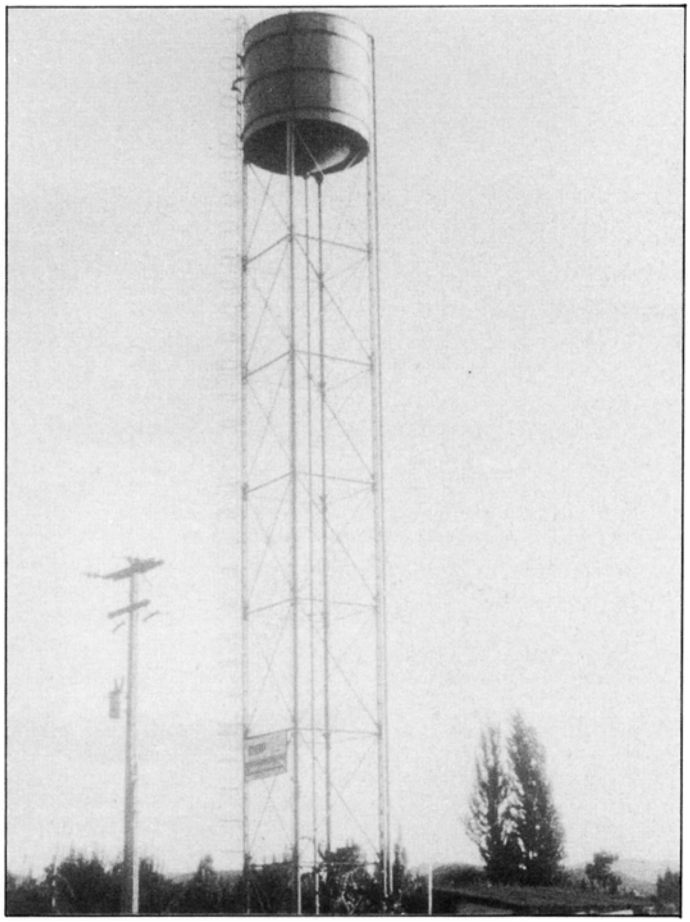


PLATE 5.36(a) Elevation

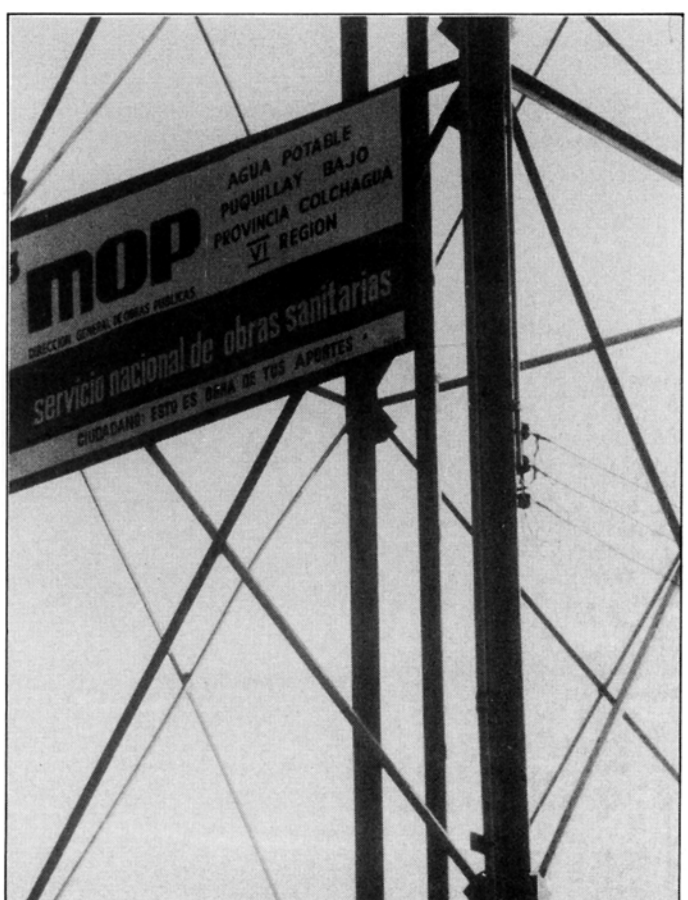


PLATE 5.36(b) Detail of buckled braces
Water tower, Puquillay Bajo

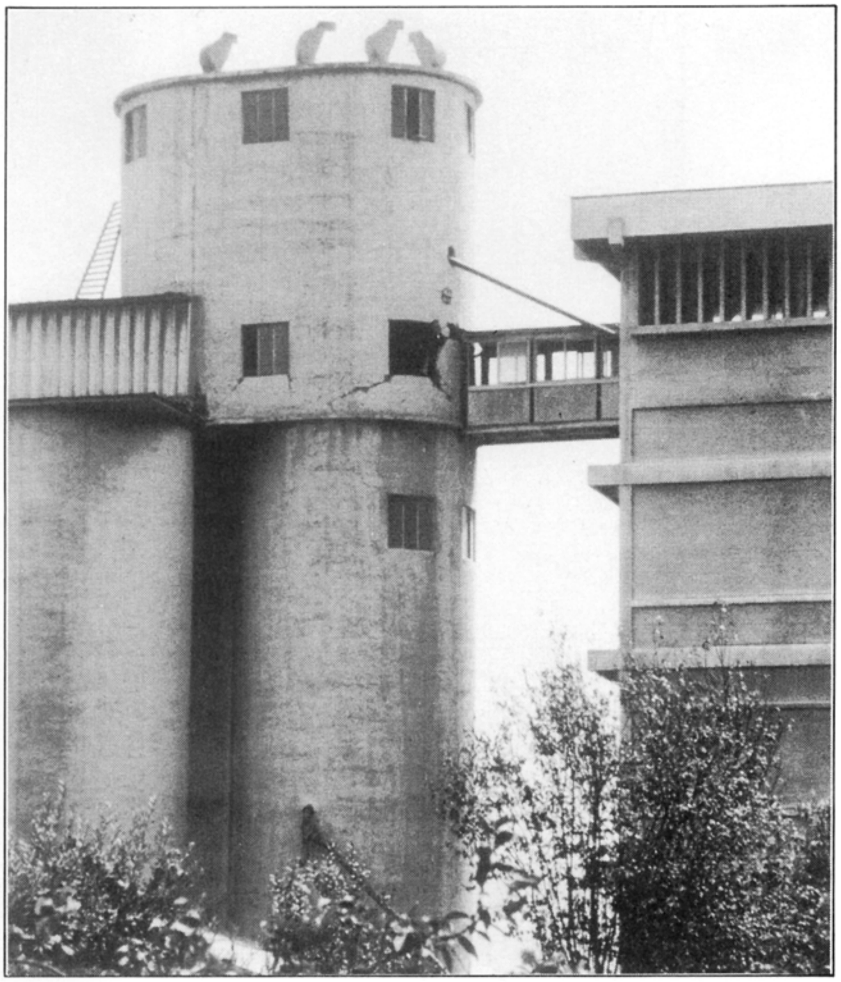


PLATE 5.37 Grain elevator, Melipilla

PLATE 5.38 Grain silos, Casablanca (photo: D Dowrick)

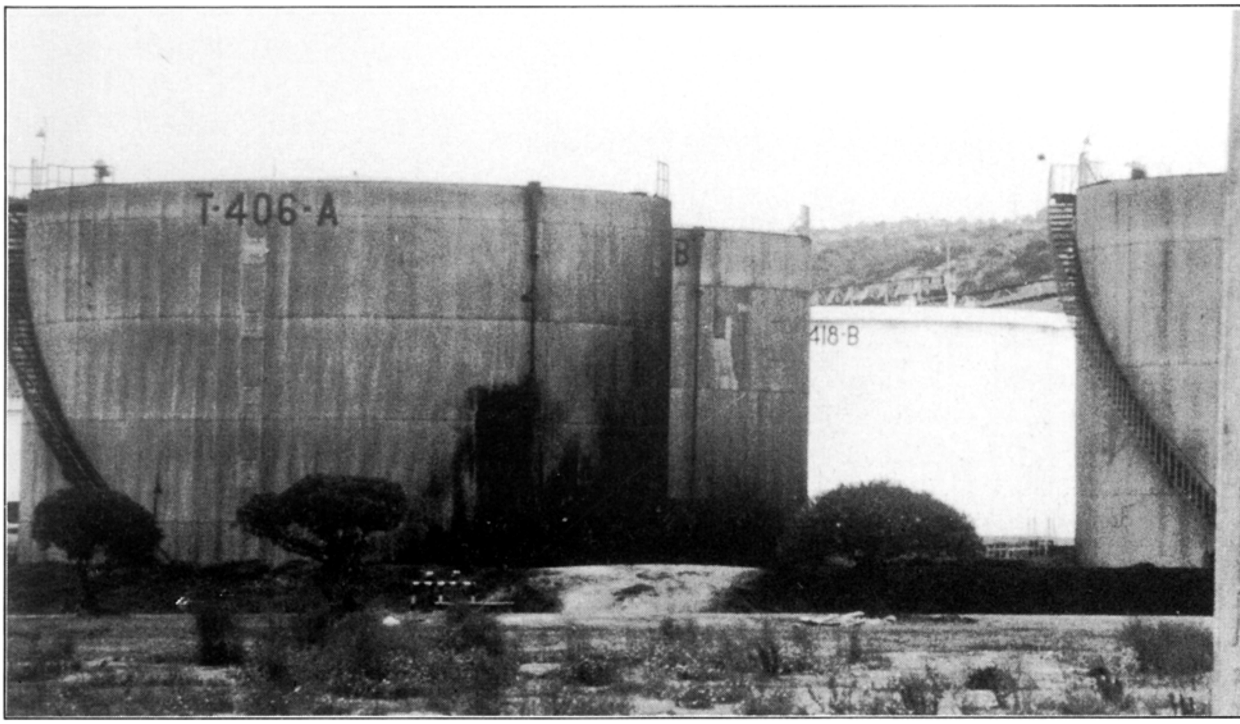


PLATE 5.39 Damaged oil storage tanks, Concon

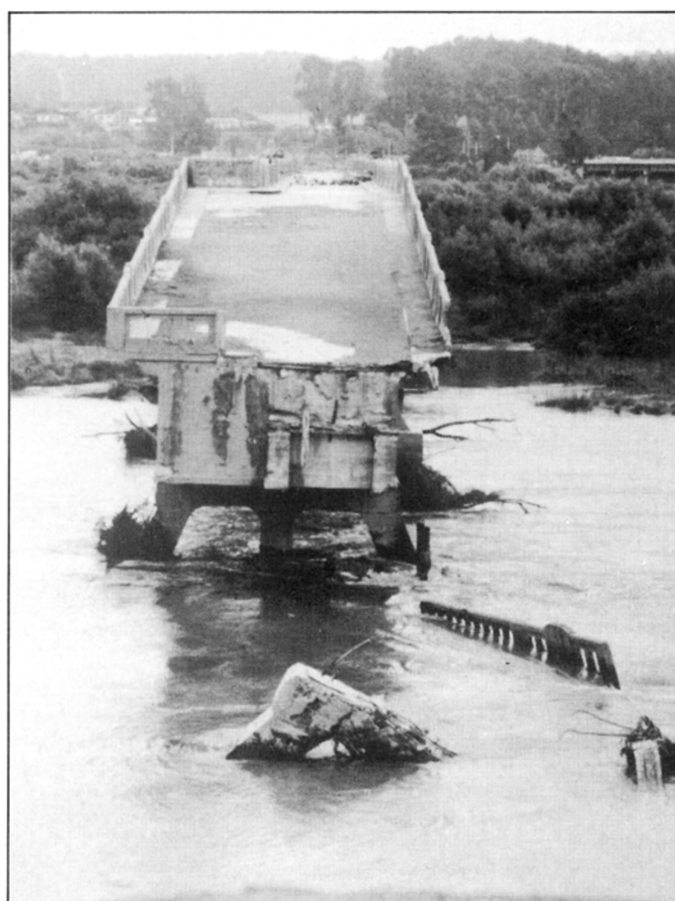


PLATE 5.40 Old bridge over River Maipo, Llolleo

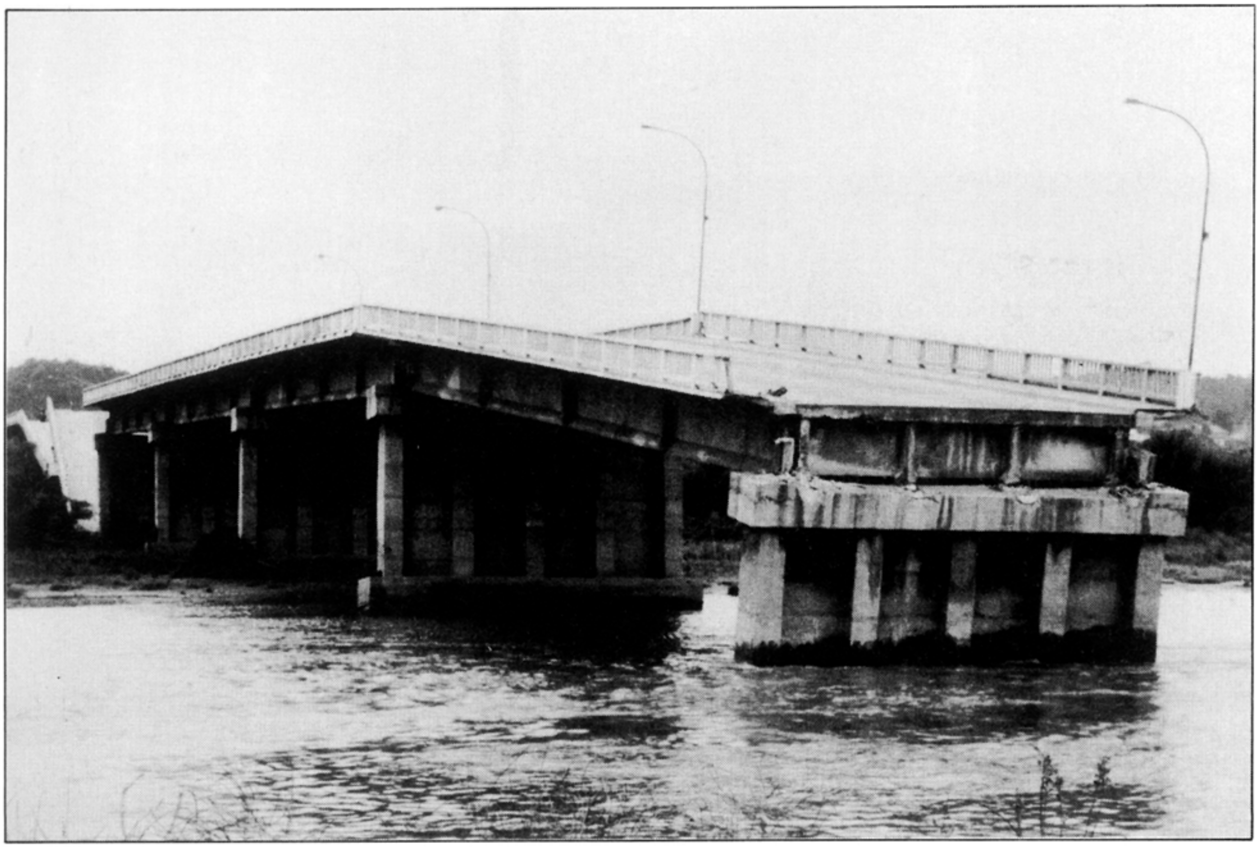


PLATE 5.41(a) Looking south

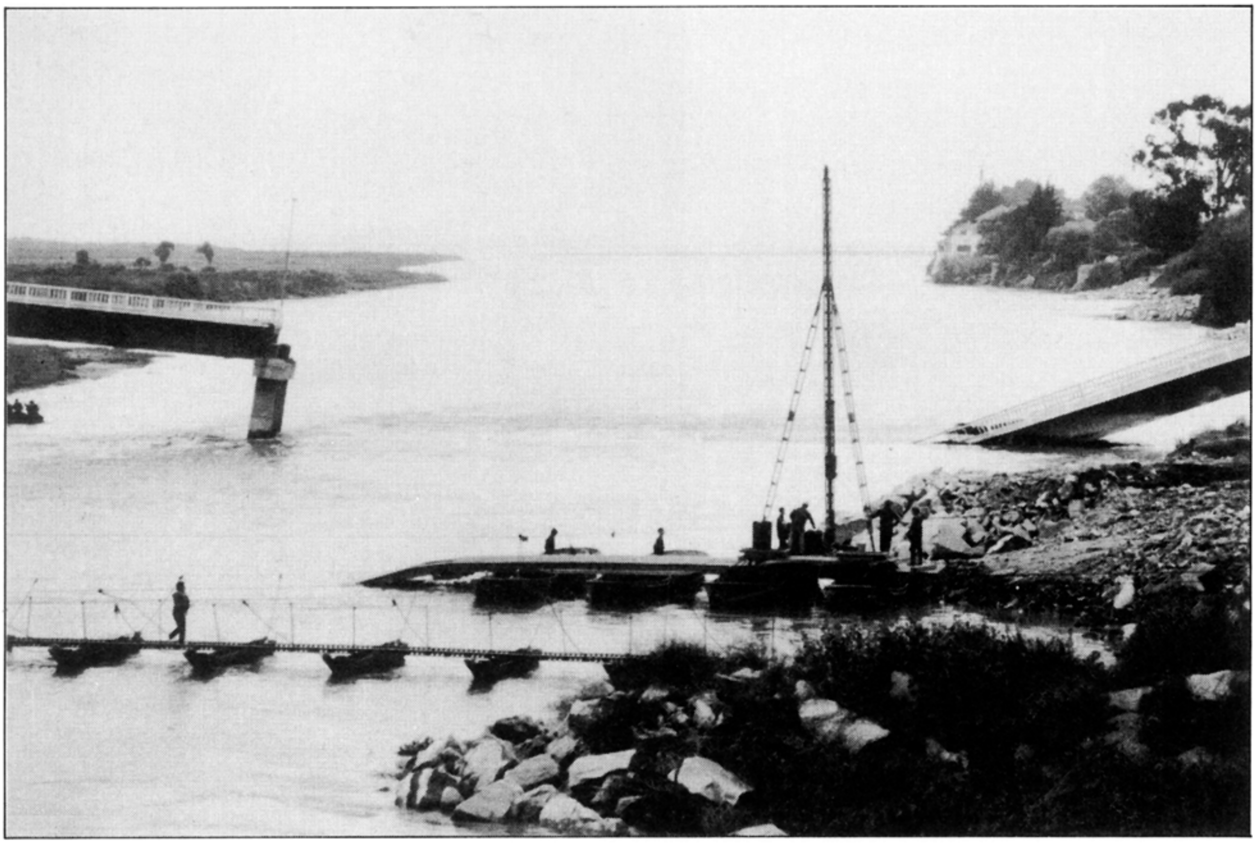


PLATE 5.41(b) Looking west (downstream) Note pontoon footbridge in foreground
Ponte lo Gallardo, River Maipo, Llolleo

For detail, see PLATE 5.44

PLATE 5.42(a)   From Point X, Fig 5.4, looking north

PLATE 5.42(b)   From Point Y, Fig 5.4, looking south

PLATE 5.42(c) From Point Z, Fig 5.4, looking south
Damaged quay at San Antonio docks

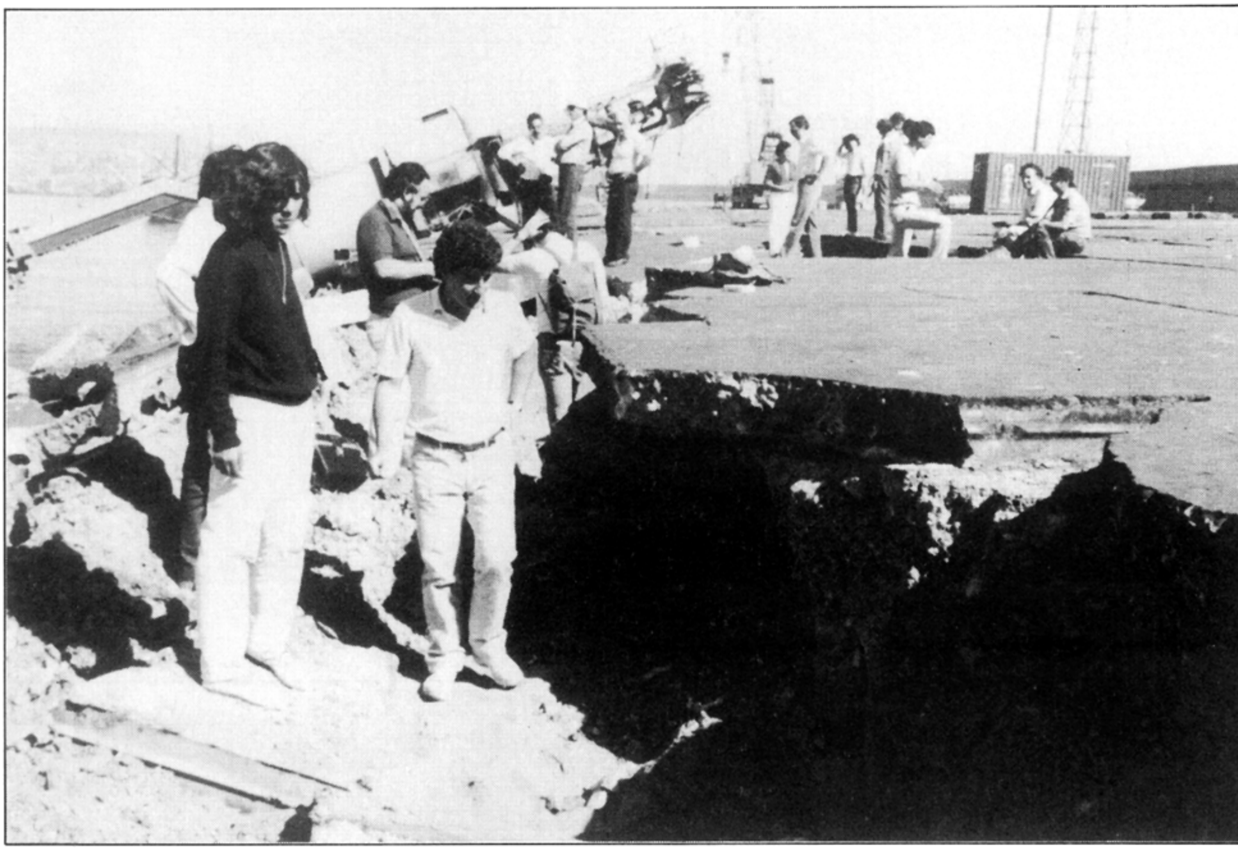


PLATE 5.43(a) Location of sand volcano

PLATE 5.43(b) Detail of sand volcano near point Z, Fig 5.4, San Antonio Docks

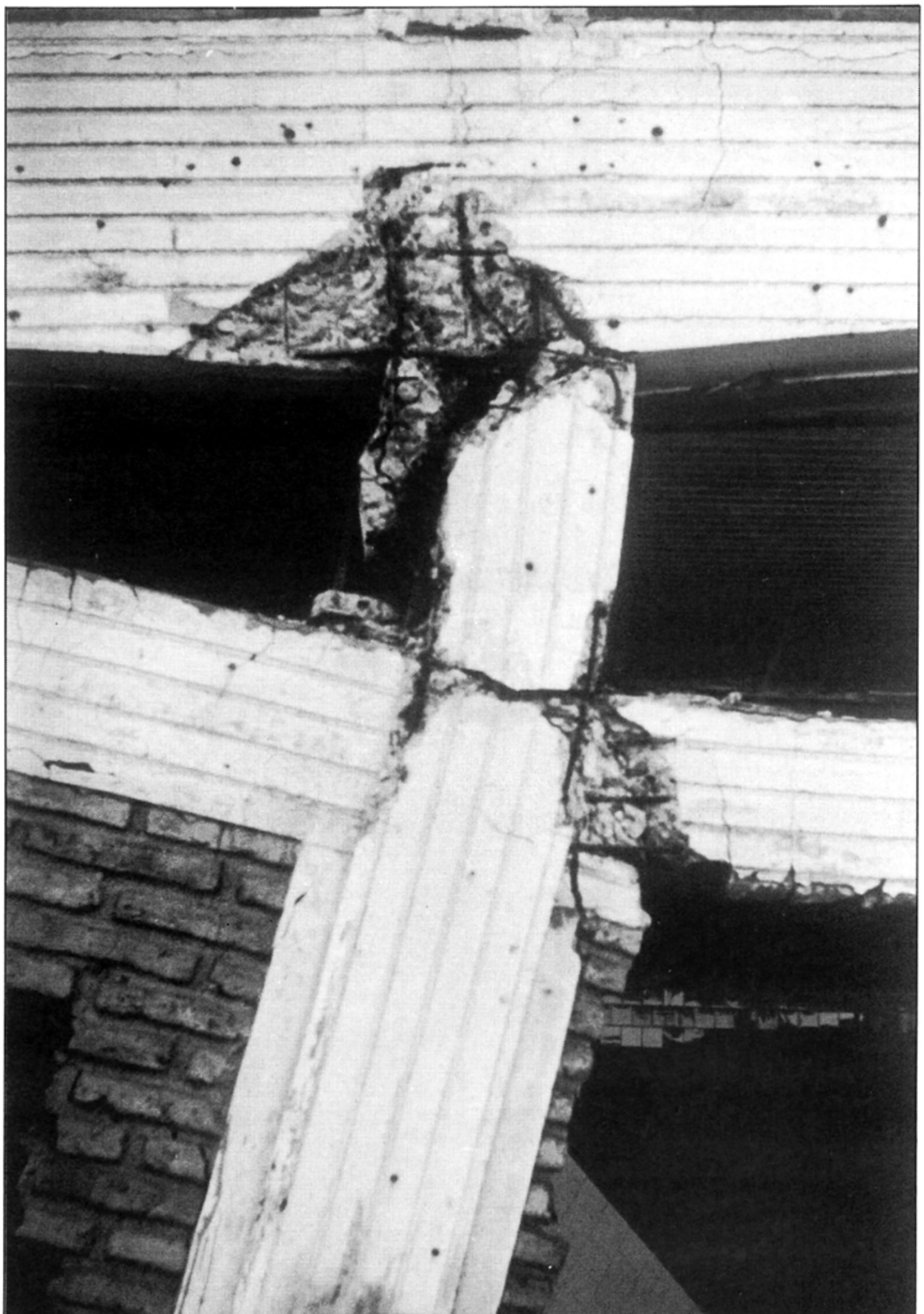


PLATE 5.44 Damaged to rc frame building due to lateral spread of quay,
San Antonio Docks [see PLATE 5.42 (a)]

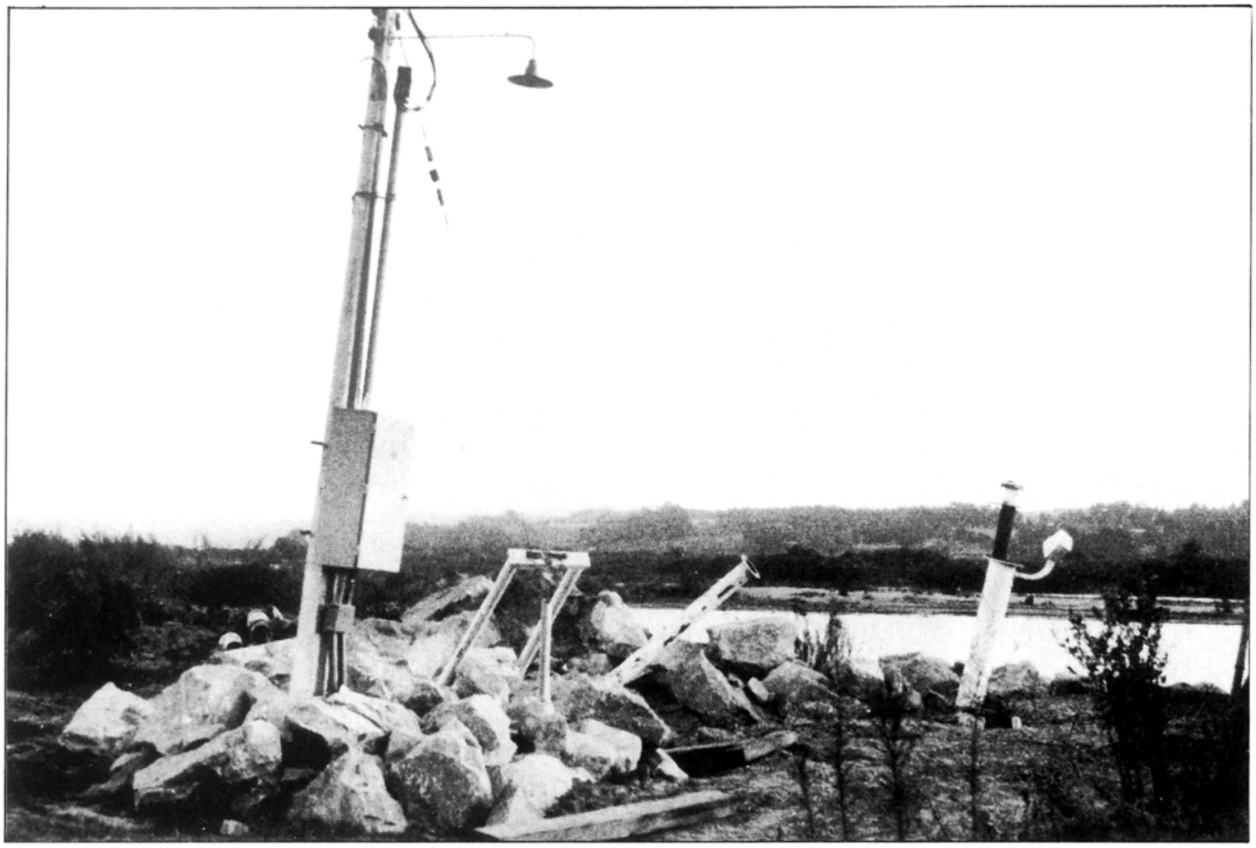


PLATE 5.45(a) Failed well point

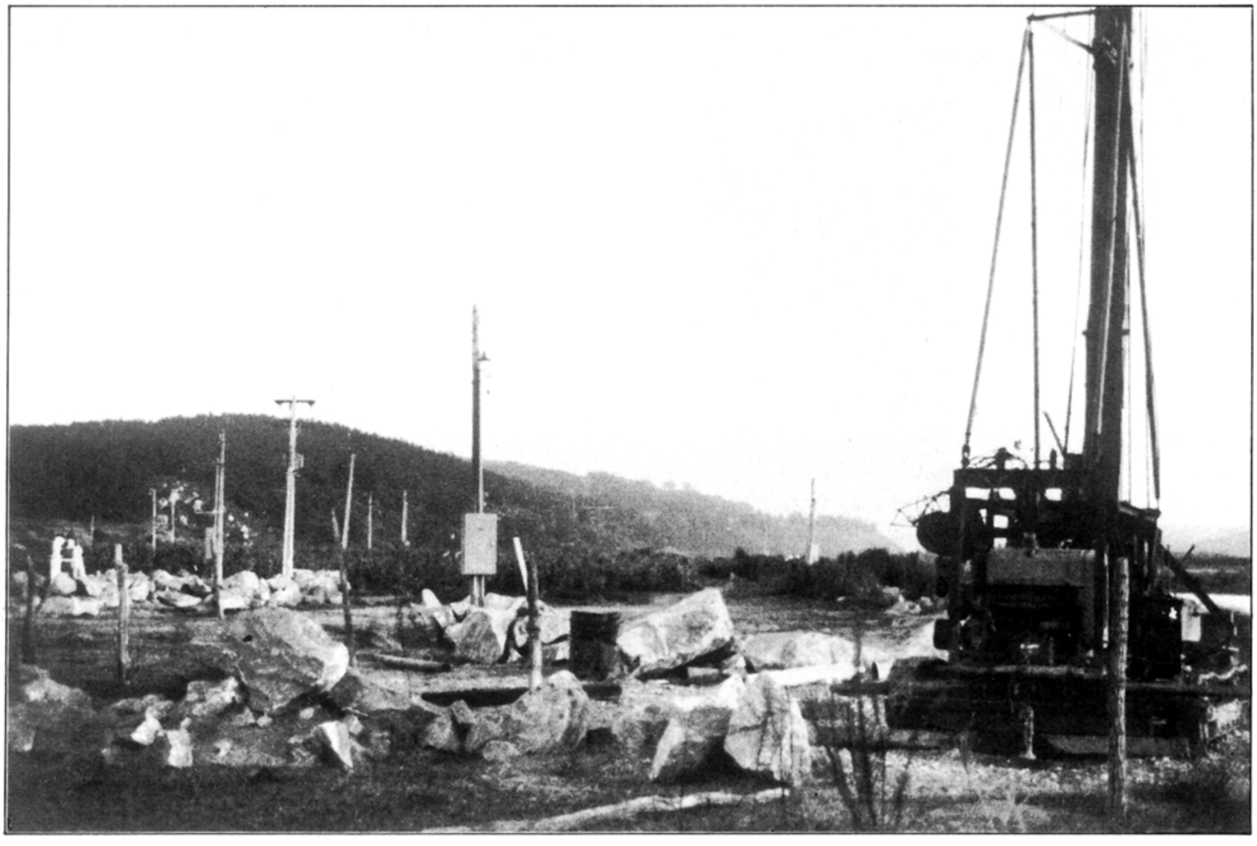


PLATE 5.45(b) Installation of new well point
Well points supplying Llolleo pumping station

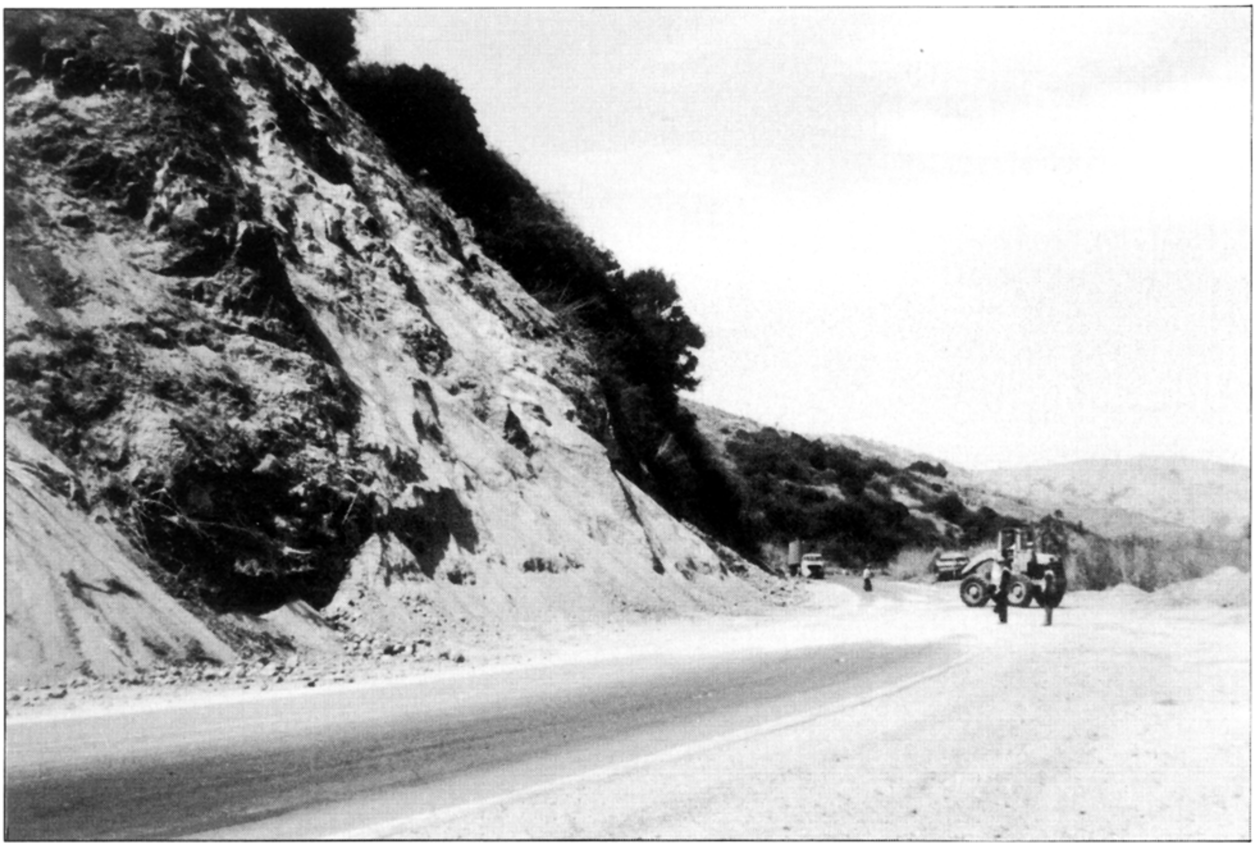


PLATE 5.46 Slip on road cutting near San Antonio.