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THE 1985 CHILE EARTHQUAKE

OBSERVATIONS ON EARTHQUAKE-RESISTANT CONSTRUCTION IN VIÑA DEL MAR

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A Report to the
NATIONAL SCIENCE FOUNDATION
Research Grants ECE 86-03789, ECE 86-03624,
and ECE 86-06089

UNIVERSITY OF ILLINOIS
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On 3 March 1985, a major earthquake occurred near the coast of central Chile. The resort city of Viña del Mar is of particular interest from an engineering standpoint because approximately 400 moderate-rise reinforced concrete buildings and a strong-motion instrument were located in the city.

The geology and seismicity of the Viña del Mar region, the development of seismic design provisions in Chile, and the ground motion records from the 1985 earthquake are described in the report. The structural characteristics of and damage in thirteen reinforced concrete buildings in Viña del Mar and Valparaiso are also described.

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CHAPTER 1
INTRODUCTION

On 3 March 1985, a major earthquake occurred near the coast of central Chile. Modified Mercalli Intensities of VII-VIII were observed in the region from Viña del Mar to Llolleo (Fig. 1.1) [61]. The resort city of Viña del Mar is of particular interest from an engineering standpoint because approximately 400 moderate-rise reinforced concrete buildings and a strong-motion instrument were located in the city at the time of the earthquake.

1.1 The Earthquake of 3 March 1985

The earthquake occurred at 7:47 p.m. local time on Sunday, 3 March 1985. The epicenter was located approximately 80 km (50 mi) southwest of Viña del Mar (Fig. 1.1), and at an estimated focal depth of 33 km (20 mi) [61]. Two separate events have been identified. The first, having a P-wave magnitude $m_b = 5.2$, was followed ten seconds later by a main event having a surface wave magnitude $M_s = 7.8$ and a P-wave magnitude $m_b = 6.9$ [68]. Modified Mercalli Intensities (Fig. 1.1) were as high as VIII in the epicentral region, including portions of Viña del Mar. The distribution of peak accelerations obtained from strong-motion instruments in central Chile is shown in Fig. 1.2 [61].

The earthquake resulted in extensive damage and casualties in the central region of Chile. Approximately 66,000 homes were destroyed and another 127,000 were damaged throughout the region [59]. During the earthquake, approximately 150 people were killed, and another 2,000 were injured [59]. In the towns of Viña del Mar and Valparaíso, the number of dead was approximately 40 and approximately 500 were injured, from a population of 550,000 [68].

Damage was not uniform for all construction types. Most adobe buildings in the epicentral region experienced some damage, with many in a state of partial or total collapse. Wood residential construction generally performed well, as did the majority of the low-rise masonry structures constructed with reinforced concrete confining elements. Steel structures, which are not common, generally did not sustain notable damage. Damage to industrial facilities and bridges was generally light and widely spread. Although numerous cases of damage and near collapse have been reported for modern reinforced concrete structures [12,13,44,68], the most significant observation may be that most such structures sustained little or no apparent structural damage.

1.2 Object and Scope

In July 1986, J. P. Moehle (University of California at Berkeley), J. K. Wight (University of Michigan), and S. L. Wood (University of Illinois at Urbana-Champaign) visited Chile to obtain information about earthquake-resistant design in the country. Discussions were held with the faculty from three universities, practicing engineers in Viña del Mar and Santiago, and officials at the Municipality of Viña del Mar. The group also visited damaged buildings in Santiago, Viña del Mar, and Valparaíso.

The objective of this report is to present the information obtained during that visit. Chapter 2 describes the seismicity, geology, and foundation conditions of Viña del Mar and Valparaíso. The development of seismic design provisions in Chile is presented in Chapter 3. Chapter 4 contains a brief summary of structural engineering practice in Chile. Chapter 5 deals with the ground motion records and the strong-motion arrays. Information about thirteen specific buildings in Viña del Mar and Valparaíso is presented in Chapters 6 and 7. Six buildings that sustained structural damage are described in Chapter 6, and seven undamaged or lightly damaged buildings are discussed in Chapter 7. Concluding remarks are presented in Chapter 8.

1.3 Acknowledgments

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University of California at Berkeley
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The writers are indebted to the many university faculty members in the departments of Civil Engineering, Geology, and Geophysics at Universidad de Chile, in Structural Engineering and Construction Engineering at Pontificia Universidad Católica de Chile, and in Civil Engineering at Universidad Técnica Federico Santa María for their valuable discussions.

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CHAPTER 2
ENGINEERING FEATURES OF THE VIÑA DEL MAR REGION

The municipality of Viña del Mar is located in central Chile (33°02′ S, 71°34′ W), adjacent to the city of Valparaíso and 120 km (75 mi) northwest of Santiago. At the time of the 1985 earthquake, the population of Viña del Mar was approximately 365,000.

This chapter briefly describes the history of the city, the seismicity and geology of the region, typical foundation conditions, and the distribution of engineered structures.

2.1 History of Viña del Mar

The history of Viña del Mar is linked closely to the history of the adjacent port of Valparaíso. The city of Valparaíso is located on a large bay of the Pacific Ocean and was the most important port on the west coast of South America before the opening of the Panama Canal [67].

The first inhabitants are believed to be native indians who migrated to the region in the early 1400's. Europeans arrived in 1536 as part of an expedition from Peru. Commercial ships arrived in Valparaíso in 1536, and in 1544 it was named the official port of Santiago [57,67].

During the sixteenth and seventeenth centuries, all imports from Europe entered through Valparaíso and then were distributed to the other parts of Chile. Italian, French, and British merchants established trade in Valparaíso in the early 1700's. By the middle of the nineteenth century, 96% of all imports and 50% of all exports passed through the port [67]. There was little industrial activity in the region until well into the nineteenth century.

The region now incorporated into Viña del Mar was divided into two large estates in 1543 and the municipality was not established until 1874. Although the land was used primarily for farming during this period, the beach front had already acquired acclaim as a resort area [66].

A railway between the Viña del Mar region and Santiago was completed in 1855, and a sugar refinery was established in Viña del Mar in 1873. In 1883, the English firm Lever and Murphy constructed a metal processing plant on the beach, and in 1887, the first steam locomotive in South America was built there [47,66].

Large hotels were constructed in 1874 and 1882, and the first public bathing area was opened in 1884. In 1910, the beach was leveled to provide a better swimming area and a casino was opened in 1930 [66].

The population of Viña del Mar grew from 28,500 in 1907 to more than 126,000 in 1960 [66]. Construction activity also increased throughout this period and reached a peak between 1960 and 1965 [56]. The number of planned hotels and condominiums was so great that in 1963 the Municipality passed a law forbidding construction on the beach [66].

A second period of peak construction activity occurred between 1977 and 1982 [56].

2.2 Seismicity of the Region

The Chilean coast parallels the boundary between the Nazca and South American Plates, and the seismic activity of the region has been attributed to large-scale crustal movements along the boundary (Fig. 2.1) [39]. A nearly uniform zone of seismic activity occurred in the region between latitudes 18° and 45° south (Fig. 2.2) during the period of 1963 to 1978 [55]. Geophysical evidence indicates the eastward subduction of the Nazca plate along the coast of Chile and Peru is at a rate of approximately 10 cm per year [55].

Two types of mechanisms have been suggested as the source of seismic activity along the western coast of South America [41,42]:

1. Interplate events at the interface between the descending slab and the overriding South American plate.
2. Intraplate events located 20-30 km inside the descending slab.

Earthquakes of magnitude 8 are likely to result from the first mechanism and magnitudes up to 7.5 may be expected from the second [41,42].

The inclination of the Nazca plate is not constant in this subduction zone [8]. As listed in Table 2.1, the region has been divided into five zones with alternating steep and gentle slopes. Present volcanic activity is high in regions of steep inclination and absent in the gently sloping regions. Viña del Mar is located at one of the boundaries between these zones.

Records of seismic activity in Chile date back to the arrival of Europeans in the area. A summary of major earthquakes is presented in Table 2.2 [40]. The apparent epicenters of approximately 80% of these earthquakes were located near the coast or offshore. The cities of Valparaiso and Viña del Mar were destroyed in the 1647, 1730, 1822, and 1906 events and serious damage was caused by the 1575, 1751, 1851, 1880, 1965, 1971, and 1985 earthquakes [40,23,57].

The 1985 earthquake appeared to be part of a cycle of damaging earthquakes in the central region of Chile. Historical evidence indicates a complete rupture of the plate boundaries from latitudes 31° to 36° south at approximately 80 year intervals (Fig. 2.3) [23,27]. One hypothesis suggests that another large earthquake is likely to occur in the near future south of the 1985 earthquake to complete this cycle [23].

Earthquakes of magnitude 6 and greater which occurred along the Chilean coast during the period of 1965 to 1985 are listed in Table 2.3 [65]. Modified Mercalli Intensities in the Valparaíso/Viña del Mar region are also noted. Relatively high accelerations have been recorded in this area during previous earthquakes. During the 7 November 1981 earthquake (M_s=6.8), a peak horizontal acceleration of 0.605g and a peak vertical acceleration of 0.637g were recorded in Papudo and the peak horizontal accelerations in La Ligua and Ventanas exceeded 0.35g (Fig. 2.4) [27]. However, the Modified Mercalli Intensity was VII in the epicentral region, indicating that peak acceleration is not necessarily an indication of damage from the Chilean events [27].

2.3 Geology of the Region

The orographic characteristics of Chile vary in the east-west and north-south directions. In general, three features dominate the terrain in the east-west direction: the coastal mountain range, the central valley, and the Andes. The central valley is not present along the entire length of the country [28,46].

The cities of Valparaíso and Viña del Mar are located on the western side of the coastal range. Outcrops of the Precambrian basement (primarily amphibolite) surround the port of Valparaíso. The geosyncline which led to the formation of the central valley developed during the Paleozoic Era, and during that time there was an intrusion of granite into the Valparaíso/Viña del Mar region. The present coast from Valparaíso to San Antonio emerged from the ocean during the Triassic period. During the Tertiary period, the ocean covered the region of Viña del Mar and sedimentary deposits were laid down which developed into sandstones [28,46].

The Quaternary period was one of great geological activity in Chile, with glaciation, volcanism, and tectonic movements combining to form the present coast and relief features. The floor of the central valley is almost exclusively morainic materials, but there is no evidence of these materials along the coast. Volcanic materials are not found in the Valparaíso/Viña del Mar region [28,46].

The present topography of Viña del Mar is typical of a receding coastline whose most notable features are marine terraces and sedimentary deposits (Fig. 2.5) [3,30]. Viña del Mar was founded on the marine-alluvial deposits at the mouth of the Marga-Marga River. Recently, the city has expanded onto the adjacent marine terraces and slopes [3]. The terraces on either side of the Marga-Marga River are at different elevations and unmatched sedimentary layers in the region suggest vertical motion along the fault which follows the river [3,30].

A geological map of the Valparaíso/Viña del Mar region is shown in Fig. 2.6.

2.4 Foundation Conditions in the Region

The foundation conditions in the Valparaíso/Viña del Mar region have been divided into five main categories [30]:

1. Fresh rock
2. Weathered rock
3. Cemented sand and gravel
4. Uncemented sand and gravel
5. Artificial fill

The fresh rock is thought to be the best foundation material; however, it is found primarily on vertical cliffs and steep slopes. The weathered rock is a good foundation material if the moisture content is low. In the presence of water, the weathered rock is very compressible. Weathered rock covers approximately 80% of the Valparaíso/Viña del Mar region. The cemented sands and gravels are highly compacted and are an excellent foundation material. The uncemented sands and gravels provide an adequate foundation material under normal conditions, but they become unstable if saturated or shaken. (Ground water is reported to have been expelled through cracks in the ground in Viña del Mar during the 1851 and 1906 earthquakes [40,57]). Artificial fill has been found to be the worst foundation material in the region, especially in regions with high organic content [30].

Four of these foundation materials are found in Viña del Mar. Unsaturated weathered rock, known as "maicillos", is found in the high regions surrounding the city. The majority of Viña del Mar is founded on uncemented sand and gravel deposits, and cemented sands are found in the dunes around Reñaca. Artificial fill was deposited along the beach in thicknesses ranging from 2 to 4 m. The typical characteristics of these foundation materials are summarized in Table 2.4 [3].

Extensive soil exploration programs are generally not conducted for residential and commercial construction projects in Viña del Mar. Available information from bore holes drilled near buildings of interest are documented in Appendix A. Structures in the compacted sand region are typically founded on mat or continuous footing foundations. Some pockets of saturated muddy clay have been identified along the river, and pile foundations are required for structures located in these areas. The mean water level is approximately 4 m below ground in the downtown region of Viña del Mar [3].

A notable geological feature of Valparaíso is the large portion of the city which is founded on artificial fill (Fig. 2.7) [2,21]. The depth of this fill is 10 m at the coast and tapers to zero approximately 2 km inland.

The majority of this material was deposited during the construction of the railroad which linked Valparaíso to Santiago in the 1880's [30]. Damage in this region is consistently greater than the damage in the surrounding hills [40,57].

The strong-motion instrument in Viña del Mar was located in the basement of a bank on the town square (Fig. 6.1). Most of the buildings examined in this report are located along the beach, north of the river. The soil on the south side of the Marga-Marga River is a fluvial sand whereas the sands on the north side of the river are generally marine deposits. Site response spectrum tests conducted during aftershocks in July and August 1985 [17] indicated almost identical amplification on each side of the river. Amplification may have been slightly higher on the hills in Reñaca at the site of El Faro [17], an eight-story R/C building that collapsed during the earthquake. More important amplification occurred at the site of the Canal Beagle housing complex on the ridges along the eastern boundary of Viña del Mar (Fig. 2.8) [17].

The Valparaíso strong-motion instrument was located on rock at Universidad Técnica Federico Santa María, and the El Almendral instrument was located in the Church of the Twelve Apostles which is founded on approximately 5 m of artificial fill.

After the 1985 earthquake, the municipality of Viña del Mar began a microzonation study to identify the parts of the city in which damage is likely to be highest. Preliminary results indicate that if an earthquake causes an intensity I in the surrounding marine terraces, the intensity is likely to be I+1 and I+2 on the slopes leading down to the city and I+3 in the alluvial, fluvial, and colluvial deposits along the Marga-Marga River (Fig. 2.9) [3].

2.5 Distribution of Engineered Structures

As part of the microzonation study, the Municipality of Viña del Mar developed a variety of maps that identify the location of all existing buildings, the distribution of damage, and the location of buildings that were demolished as a result of the damage. According to the study, approximately 400 buildings ranging in height from 5 to 23 stories were located in the city at the time of the 1985 earthquake (Fig. 2.10). All buildings in this height range were constructed from reinforced concrete. The majority of these buildings are concentrated in the downtown area of Viña del Mar, at the mouth of the Marga-Marga River (Fig. 2.11).

Although the buildings identified by the microzonation study were not classified according to the structural framing system, walls comprised the lateral-load resisting system for nearly all the reinforced concrete buildings. Only three buildings in the downtown area were identified as having moment-resisting frames: Emporium, a four-story retail building; Nuevo Centro 2, a six-story commercial center; and Arcadia, a nine-story commercial center and apartment building. A fourth building with moment-resisting frames, Eurosol (a six-story apartment building), is located in Reñaca. Reference 56 contains a detailed description of the building inventory in Viña del Mar.

CHAPTER 3
DEVELOPMENT OF SEISMIC DESIGN PROVISIONS IN CHILE

The Talca earthquake of 1 December 1928 (Table 2.2) initiated the development of seismic design regulations in Chile [10,26,63,64]. The first official regulation that included seismic design provisions, La Ordenanza General de Construcciones y Urbanización, was adopted in 1935. The current seismic code, NCh 433-Of72: Cálculo Antisísmico de Edificios [16], was adopted in 1972. The development of these codes is described in this chapter.

3.1 La Ordenanza General de Construcciones y Urbanización

Flores and Jimenez [26] describe the development of La Ordenanza General de Construcciones y Urbanización (referred to as the General Ordinance in this report) as follows:

> "As a consequence of [the Talca earthquake of 1928], the government named a committee of three engineers to study the subject [of seismic regulations for structural design]. The work of this committee was highly influenced by the theoretical literature of the epoch [1927-1932] in which the concept of resonance prevailed, so that the recommendations which were adopted were intended to avoid it.
>
> They prescribed that brick masonry be built with reinforced concrete columns and beams, according to a system devised by Italian engineers after the great Sicilian earthquake of 1908 [confined masonry construction]. This construction system proved to be quite effective during the Chillán earthquake of January 1939.
>
> A General Ordinance based on these recommendations was finally endorsed in 1931 and approved as law in 1935.
>
> This Ordinance applied to the whole country. It not only gave rules for construction, but also for the design of structures. In its first version, the text provided a series of specifications with the purpose of obtaining monolithic action or continuity of

structures, whenever possible. It also limited the use of certain materials and forbid some kinds of construction which up to that date had been widely used.

With reference to seismic forces, the Ordinance required that all structures should be checked for lateral forces of 1/10 to 1/20 of the weight of the structure, as a function of ground conditions and other factors.

This General Ordinance initiated modern seismic engineering in Chile. A significant number of minor and medium-sized structures built in accordance with its provisions behaved satisfactorily during recent earthquakes.

This code began to be applied in a provisional way in 1930 and ruled construction in Chile up to the devastating Chillán earthquake of 1939, when it became apparent that revision of this Ordinance was due."

A new General Ordinance was quickly approved in 1939 [10]. Building height limits were established:

Reinforced Concrete	12 stories or 40 m
Structural Steel	13 stories or 40 m
Timber	3 stories or 12 m
Masonry	2 stories or 8 m
Adobe	1 story or 3.5 m

The Ordinance did not permit construction with unreinforced masonry, and strict provisions were specified for adobe construction [10].

The seismic base-shear coefficient established in the General Ordinance of 1939 was dependent upon the period of the structure, as well as the foundation conditions. For buildings with calculated periods less than 0.4 sec, the base-shear coefficient ranged between 0.1 and 0.2 depending upon the quality of the soil [10,63]. Lateral forces were distributed uniformly over the height of the structure [10].

For buildings with calculated periods greater than 0.4 sec, the seismic response was considered to be equivalent to forced vibration with a peak acceleration between 0.1 and 0.2 g, a displacement amplitude between 4 and 6 cm, and a period between 1 and 2 sec [10]. The magnitude of the forcing acceleration was a function of the soil conditions. Buildings with calculated natural periods between 1 and 2 sec were not permitted [10].

Immediately after the revised General Ordinance was adopted in 1939, engineers, architects, and contractors expressed opposition because the new requirements led to an increase in building costs [10]. In 1945, the government appointed a committee to write a more economical building code [10,26].

The next revision to the General Ordinance [38], which contained less severe seismic requirements, was adopted in 1949. Height limitations were eliminated for reinforced concrete and structural steel buildings and the maximum height of confined masonry construction was increased to four stories [10].

The total lateral force for which a building was designed using the provisions of the General Ordinance of 1949 [38] was defined as:

$$V = CP$$

where

P = the total weight of the building plus a percentage of the live load. This percentage was not less than 25% for all buildings and was at least 50% for buildings with high occupancies.

T = fundamental period of the building, sec.

C = the seismic coefficient which ranged from 0.05 to 0.15g depending upon the period of the building, the quality of the soil, and the type of foundation.

The seismic coefficients are listed below:

Seismic Coefficient

	$T < 0.4$	$0.4 < T < 0.75$
Rock	0.08	0.05
Firm soil	0.12	0.10
Sand or Fill		
(Mat foundation)	0.10	0.12
(other foundation)	0.12	0.15

The forces were distributed over the height as follows:

$$F_k = \frac{P_k \; Z_k}{\sum(P_i \; Z_i)} V$$

where

k = story level

F_k = lateral force at level k

P_k = weight of level k

Z_k = height of level k above base

The General Ordinance of 1949, with minor modifications, is essentially still in force today. However, the sections governing seismic design of buildings have been replaced by the current building code, NCh 433, which was adopted in 1972 [26,64].

3.2 La Norma Chilena NCh 433: Cálculo Antisísmico de Edificios

Flores and Jimenez [26] described the development of the current seismic design provisions as follows:

> "In the year 1960, INDITECNOR (Instituto Nacional de Investigaciones Technológicas y Normalización) was studying a new code to incorporate the experience and knowledge gained in Chile and abroad until that time in the field of earthquake engineering. This project was left aside when new information on the behavior of buildings during the major earthquakes of May 1960, that shook the southern part of Chile, became available.
>
> A tentative proposal for a new code which was accepted, was submitted to the seismic studies committee of INDITECNOR in 1962... In May 1966, a provisional version of the NCh 433 code was approved to replace the chapters of the General Ordinance referring to seismic design of buildings. The committee appointed to study the Chilean seismic code took more than thirteen years to complete its work, until approval in 1972. The NCh 433 code became law on April 23, 1974.
>
> ...It is interesting to bear in mind that the concepts involved in the NCh 433 code were applied long before its becoming law. Many buildings which successfully withstood the 1960, 1965, and 1971

earthquakes, were designed in accordance with its prescriptions. Reciprocally, this experience influenced the code under development."

The new code classified buildings according to the occupancy importance and structural characteristics. Static and dynamic methods for determining lateral forces were also included [5]. NCh 433 [16] is summarized below*.

3.2.1 Static Analysis

This method of analysis is not permitted for buildings more than 15 stories or 45 m in height with irregular distributions of mass or stiffness along the height.

The total base shear is defined as:

$$Q = K_1 \ K_2 \ C \ P$$

where

Q = base shear

K_1 = coefficient related to occupancy importance
= 1.2 for essential facilities
= 1.0 for buildings for general use
= 0.8 for provisionary construction and isolated buildings not intended to be used as dwellings

K_2 = coefficient related to structural system
= 0.8 for ductile, moment-resisting frames
= 1.0 for buildings with rigid floor diaphragms
= 1.2 for all other structures

C = seismic coefficient
= 0.10 for $T \leq T_o$

$$= 0.10 \ \frac{2T \ T_o}{T^2 + T_o^2} \quad \text{for } T > T_o$$

T = fundamental period of vibration of the structure

*Summarized and translated by R. Riddell.

T_o = soil parameter which varies between 0.2 for rock and 0.9 for soft ground

P = total weight of the building plus a percentage of the live load. The percentage may not be less than 25% for all buildings and at least 50% for buildings with high occupancies.

The total base shear may not be less than 0.12P for one-story buildings, and may not be less than 0.18P for one-story, unreinforced masonry buildings.

The total base shear, Q, is distributed over the height of the building according to the formula:

$$F_k = \frac{P_k \ A_k}{\sum_{j=k}^{n}(P_j \ A_j)} Q$$

where

$$A = \sqrt{1 - \frac{Z_{k-1}}{H}} - \sqrt{1 - \frac{Z_k}{H}}$$

j,k = story level (numbered from the base to the top)

F_k = lateral force at level k

P_k = weight of level k

Z_k = height of level k above base

H = total height of the building

n = total number of stories

The overturning moment obtained from the forces defined above may be reduced by a factor $J_k = 0.8 + 0.2\ Z_k/H$, for buildings with 4 or more stories.

The total story shear is distributed to the various resisting elements in proportion to their stiffnesses, considering the rigidity of the horizontal diaphragm. In addition to the direct shears, the resisting elements are required to carry a torsional moment at each floor given by:

$$M_k = 1.5\, Q_k e_k \pm 0.05\, I_k \sum_{j=k}^{n} F_j b_j$$

where

M_k = torsional moment at level k

Q_k = shear at level k

e_k = distance between the center of rigidity of level k and the line of action of the story shear Q_k

F_j = lateral force at level j

b_j = plan dimension at level j in a direction perpendicular to F_j

I_k = $0.7 + 0.3\, Z_k/H$

3.2.2 Dynamic Analysis

A modal superposition type of analysis is permitted with an acceleration spectrum (Fig. 3.1) given by:

$$\frac{S_a}{g} = \begin{cases} 0.10\, K_1\, K_2 & \text{for } T \le T_o \\ 0.10\, K_1\, K_2 \dfrac{2T\, T_o}{T^2 + T_o^2} & \text{for } T > T_o \end{cases}$$

The ordinates of the acceleration spectrum are equal to the product of the seismic coefficient, C, the occupancy factor, K_1, and the structural system factor, K_2. The total base shear may not be less than $0.06\, K_1\, K_2\, P$.

Each effect S shall be estimated by the superposition of the values for each mode, S_i, with the formula:

$$S = \frac{1}{2} \left(\sum |S_i| + \sqrt{\sum S_i^2} \right)$$

A complete English translation of NCh 433 is contained in Reference 22.

CHAPTER 4
STRUCTURAL ENGINEERING PRACTICE IN CHILE

This chapter presents detailed information on typical building types, structural and nonstructural details, construction, and material properties. Much of the information was obtained during discussions with practicing engineers and from observations of existing buildings.

4.1 Building Types and Design Concepts

Most buildings in Chile are constructed using masonry (reinforced and unreinforced) or reinforced concrete. The cost of structural steel buildings is high because the nonstructural finishing materials are not readily available and heavy structural steel sections are not rolled in the country. Structural steel is commonly used for bridges and industrial structures, however. Prestressed/precast concrete construction is not typically used for buildings, but it is used for bridges and underground parking structures.

4.1.1 Masonry Construction

Three types of masonry construction are commonly used in Chile: unreinforced masonry, confined masonry, and reinforced masonry. The use of unreinforced brick masonry was prohibited in the General Ordinance of 1939; however, unreinforced brick partition walls are common. Adobe construction is limited to one-story buildings. Adobe and unreinforced masonry construction suffered heavy damage during the 1985 earthquake, especially in Melipilla where hundreds of houses were destroyed [7]. Adobe structures in Valparaíso also suffered serious damage.

Confined masonry was introduced in Chile in the late 1920's following the 1928 Talca earthquake [26]. This type of construction is characterized by reinforced concrete columns that are cast directly against previously constructed unreinforced brick panels (Fig. 4.1). A "chain beam" is then cast around the perimeter of the structure. This type of construction provides a confined brick panel and a keyed surface that facilitates composite action between the brick and surrounding reinforced concrete members. It also provides a means of connecting perpendicular walls (Fig. 4.1). Structures of this type are limited to four stories in height and reinforced concrete floor slabs are required [38]. In general, confined masonry structures performed well during the 1985 earthquake; however, columns were observed to separate from the masonry panels in a number of buildings. Problems were also observed in buildings with unsymmetrical arrangements of walls [7].

There are two types of blocks used in reinforced masonry: concrete blocks and machine fabricated clay bricks. The clay bricks are more common and are fabricated with perforations to allow placement of the reinforcing bars. Reinforced masonry construction with clay bricks is very lightly reinforced. The average vertical reinforcement in walls is 8-mm diameter bars spaced at 1-m intervals. More reinforcement is placed in the corners (Fig. 4.2). One 8-mm diameter bar is typically placed horizontally above doors and windows. Reinforced concrete slabs are also used for this type of construction.

Reinforced masonry construction with machine fabricated clay bricks did not perform well during the 1985 earthquake. Extensive cracking of the masonry walls, around openings, and at the connection with the floor slabs was observed [7]. The General Ordinance does not have specifications for this type of construction, and foreign codes were unofficially adopted.

Confined masonry and reinforced masonry is used for most of the social housing constructed by the government.

4.1.2 Reinforced Concrete Construction

Both frame and wall construction philosophies developed in Chile during the early 1900's. However, according to engineers at Arze, Recine y Asociados, Ingenieros Consultores LTDA, in Santiago, construction of frame buildings declined following failures of some early frame buildings during earthquakes in the 1930's. The typical Chilean building constructed since that time relies on structural walls for lateral load resistance.

The use of structural walls became a significant part of architectural practice in Chile. Thus, when high-rise construction began to evolve in the early 1960's, liberal use of structural walls continued. Ratios between wall and floor areas are typically much higher in Chilean construction than in construction in seismic zones in the United States. The moderate-rise buildings examined in this report relied primarily on structural walls to resist lateral loads.

Detailed design of reinforced concrete construction has been influenced by procedures in the U.S., Germany, and Japan. The current building code for reinforced concrete [32,33] is based on the 1952 German DIN standard [11]; however, many designers have adopted portions of the strength design provisions of the ACI building code [14]. In general, modern reinforced concrete construction in Chile does not follow the ductile detailing requirements in Appendix A of the ACI building code [14].

4.1.3 Design Concepts

The Chilean philosophy with respect to acceptable damage and safety is the same as that commonly expressed in the U.S.: minor damage is acceptable in moderate earthquakes and structural failure should be avoided in severe earthquakes [53]. However, the scales of earthquake intensity are not the same in the U.S. and Chile. Although bounds are not explicitly established, earthquakes with magnitudes between 6.5 and 7.0 are considered to be minor in Chile, and structural damage is not expected during such events. A magnitude on the order of 7.5 corresponds to a moderate earthquake.

The Chilean experience with frequent strong earthquakes has led to a building construction form that differs from that of the U.S. Chilean engineers, architects, and occupants prefer "rigid" structures. The rigidity is achieved by the use of relatively large proportions of structural walls for lateral load resistance. Experience with these structures has led to the conclusion that careful detailing and construction inspection are, in most cases, unnecessary. A more detailed discussion of building design and construction practice is presented in the following section of this chapter.

4.2 Structural and Nonstructural Details in Reinforced Concrete Buildings

A brief description of typical structural and nonstructural details in reinforced concrete construction is presented in this section. Emphasis is placed on reinforced concrete construction because all buildings having more than four stories in Viña del Mar were constructed from reinforced concrete.

4.2.1 Structural Details

The detailing requirements for reinforced concrete elements in the current Chilean code, NCh 429 [32] and NCh 430 [33], were not developed for construction in seismic areas. There are no special detailing requirements for ductility, and some requirements are specific to gravity loads. Although not required, many engineers follow some of the detailing provisions in Appendix A of the 1977 edition of the ACI building code [14]. The Chilean reinforced concrete code is described in greater detail in Reference 56. A few provisions are summarized below.

No limits are placed on the amount of flexural reinforcement placed in beams and walls. The flexural reinforcement ratio in columns must be between 0.8 and 6%. Shear reinforcement is required in beams if the average shear stress exceeds $1.9 \sqrt{f'_c}$ where f'_c is in psi*. Bending of longitudinal reinforcement in beams to provide shear reinforcement is recommended; however, vertical stirrups are permitted. The minimum wall thickness is 20 cm (8 in.) and the reinforcement ratio for shear reinforcement in the walls must not be less than 0.2%. Hooks with 180° bends are specified for anchorage of reinforcement and 90° hooks are permitted for column steel. The inside diameter of the bend may not be less than 2.5 bar diameters and the bar must extend at least 4 bar diameters past the bend.

Because these detailing provisions for anchorage and transverse reinforcement are not practical for construction in Chile, the manufacturer of reinforcing bars, CAP, publishes a set of recommended details [9]. The recommended details for development length, lap splices, hooks, and spacing between bars are the same as the ACI requirements [14] for regions with low seismic risk. One notable exception to the ACI code is the reduction of

*The Chilean reinforced concrete code [38,39] is based on working stresses.

cover requirements. The Chileans interpret the ACI provisions in cm rather than in.

Because a single set of detailing recommendations is not followed closely, it is not possible to characterize detailing practice in Chile. However, some perspective may be gained by reviewing the structural drawings of buildings in Chile. The following observations were made from buildings located in Viña del Mar. The structural characteristics of these buildings are described in Chapters 6 and 7.

Each structure considered had structural walls occupying a large portion of the floor area. Wall thicknesses ranged from 20 cm (8 in.) in low-rise buildings and the upper stories of moderate-rise buildings to 70 cm (28 in.) at the base of a 22-story building. The walls were lightly reinforced, longitudinal reinforcement ratios were typically less than 1%. In general, there was no confinement of the boundary reinforcement in the walls. One exception was Festival where a single hoop, formed from a 10-mm (0.4 in.) diameter bar, spaced every 20 cm (8 in.) was used around sixteen 26-mm (1 in.) diameter bars.

The shear reinforcement in walls was also light, and typically provided by two layers of welded wire fabric. In some cases, the shear reinforcement was oriented at 45° from vertical. This arrangement was used primarily in portions of the walls which were used as transfer girders where the locations of openings in the shear walls changed near the base of the structure.

Lap splices were used for the longitudinal bars in walls and were typically located immediately above the level of the slab.

Transverse reinforcement was carefully detailed for columns. The location of each tie was shown on the structural drawings. The use of spiral reinforcement in the columns was not observed. The locations of lap splices for longitudinal bars in the columns occurred generally near the beam-column joint, and through the joint in some cases. Details of transverse reinforcement in the joint region were not specified on the drawings examined in this study, although such reinforcement is recommended in the CAP manual [9].

Bent longitudinal beam reinforcement was used for shear reinforcement in approximately one-third of the structures. Bent bars were also observed in slabs, but were less common. Slabs were often used as coupling beams between structural walls. Additional longitudinal reinforcement was usually placed in these areas.

Detailed drawings of the reinforcing bars are not prepared in Chile. Reinforcement is fabricated at the construction site directly from the structural drawings.

The location of construction joints are not identified on the drawings, but left to the contractors' discretion.

4.2.2 Nonstructural Details

Seismic requirements in Chile do not cover nonstructural elements, such as partition walls, ceilings, light fixtures, piping, elevators, stairs, and windows. Windows are currently regarded with some concern because there are no wind design requirements in Chile which would require special detailing of window frames.

Structural engineers and architects agree that partitions walls should be isolated from structural elements to avoid damage during earthquakes. However, isolation details are not given in structural drawings so instructions are typically given orally in the field. A considerable amount of partition damage was observed during the 3 March 1985 earthquake. Poorly designed details and improper construction are believed to be the cause of this damage.

The selection of partition type is made by the architect and described in the technical specifications. Heavy masonry infills are preferred for sound insulation even in moderate-rise apartment buildings. Experience indicates that the use of lighter partitions makes the sale or rental of apartment units more difficult. Common types of nonstructural partitions are listed below:

Volcanita: thin gypsum sheets over wood studs (sound insulation may be added between the gypsum sheets).

Bepolita: mortar with styrofoam balls sandwiched between thin gypsum sheets.

Gypsum: a single gypsum sheet, thickness of 5 cm. (2 in.).

Pandereta: 5-cm bricks placed on their narrow edge and covered with a mortar coating forming a 10-cm thick (4 in.) partition.

Masonry: 10 or 15-cm bricks placed normally and covered with a mortar coating forming a 15 to 20-cm thick (6-8 in.) partition.

Lightweight Concrete: mortar with styrofoam balls reinforced with wire mesh.

Asbestos: mortar with asbestos mesh.

Particle Board: pressed wood with mortar impregnation.

Concrete slabs are usually topped with 5 to 12 cm. (2 to 5 in.) of lightweight concrete. The topping material is a cement mortar with styrofoam balls mixed in to reduce weight and provide sound insulation. Various floor finishes, including tile and carpeting, are used.

Because most forms for reinforced concrete members are constructed from 4-in. wide boards, the concrete surface is rough. Almost all structural elements are covered with a minimum of 2 cm. (0.8 in.) of plaster to create a finished surface. This coating is typically painted for interior surfaces and ceramic tile is commonly affixed to exterior walls to create a more decorative surface.

4.3 Construction and Inspection Practices

Construction and inspection practices in Chile appear to be less sophisticated than the design practices. Flores and Jimenez [26] summarized the Chilean construction industry as follows:

> "The design and construction of the most important projects are carried out in Chile under the best conditions, resembling very closely those being used in developed countries. Construction is performed with the most appropriate types of equipment, under well controlled conditions, [and] complying with international standards...On the other hand, in small projects, the quality obtained is deficient, in many cases, for lack of adequate quality control and on account of mediocre designs...Between the large and the small projects, lie the medium sized ones, which are really the most numerous and important, because of their socio-economic significance...Many engineers maintain that [the damage from the 1985 earthquake] could have been partially avoided if adequate regulations concerning design review by qualified engineers and construction inspection by independent parties had been in force."

The following discussion of inspection practice is specific to Viña del Mar, but may be considered representative of most of Chile.

Before 1955, inspection of construction was closely monitored by city inspectors. Site visits were scheduled to inspect the project at three critical times during construction.

The construction industry grew dramatically in the early 1960's and the municipality was unable to maintain the on-site inspection. However, all drawings and design calculations were checked before the construction permit was issued. This procedure was also discontinued in 1981 due to a lack of technical support at the municipality and the pressure to begin construction projects as soon as possible. The only current requirement is that the drawings for buildings over two stories be presented to the municipal office with a statement that an engineer has reviewed and stamped the drawings.

The architect is responsible for building construction and inspection; however, the structural engineer typically visits the site also. The progress of the project is recorded in a log book, and all problems are documented in that book. The book serves as the contractor's proof that the building was constructed according to the drawings and any deviations were approved by the architect or engineer.

The municipality requires that both the architect and structural engineer accept the building before a certificate of occupancy is issued. The municipality inspects the building at that time to insure that it agrees with the drawings. The municipality keeps a file for all engineered structures. The file contains the structural drawings, architectural drawings, maps of utility lines, material test reports, and construction specifications.

4.4 Materials Used in Building Construction

4.4.1 Concrete

Specifications covering cement and aggregates are given in Chilean codes, NCh 148 [18] and NCh 163 [19], respectively. The current specification for concrete, NCh 170 [34], defines five grades of concrete which are classified by the minimum resistance of a 20x20x20 cm cube at an age of 28 days, R_{28}. A factor of 0.85 may be used to convert from the resistance of a cube to the compressive strength of a cylinder, f'_c.

CLASS	R_{28} kg/cm^2	f'_c kg/cm^2	f'_c ksi
A	120	100	1.45
B	160	140	1.90
C	180	150	2.20
D	225	190	2.75
E	≥300	255	3.60

The lower strength concretes are used for slabs on grade and low-rise structures. The higher strengths are used for taller buildings. Classes D and E were typically specified for the moderate-rise buildings in Viña del Mar. The concrete code is currently being revised, and a statistical approach for acceptance of concrete strength, similar to the approach described in Reference 54, has been proposed.

River aggregates are typically used in Chilean concretes. They are usually rounded and of high quality. However, as gathered and stockpiled, the properties are not consistent with respect to size and gradation. Aggregate properties are not checked daily, and changes are not incorporated into the mix design. However, concrete is usually made with a high margin of safety, so that most contractors get concrete strengths well above the specified minimum resistance. Some concrete companies have begun to use crushed aggregates for more uniform quality. Typical Chilean concretes are relatively impermeable. The average unit weight of concrete is approximately 2400 kg/m^3 (150 pcf).

Chilean cements are not pure Portland cement. They contain 20 to 30% pozzolan. The cement grains are usually very fine and result in high early strengths. Concretes made using this cement exhibit problems with shrinkage cracking. The fine grain sands contribute to this problem.

Admixtures are frequently added to reduce costs. For example, a water reducer may be added so the cement content could be reduced without lowering the concrete strength.

Quality control laboratories that monitor the concrete strength are usually associated with a university. Concrete prisms are normally cast at the job site and tested in the laboratory. Specifications require that one sample be taken per 100 m^3 of concrete cast or at least once a day. However, time and location within the building are the choice of the contractor. The concrete testing laboratory personnel are involved only with material testing, not with general building inspection. Among the Chilean engineers that were interviewed, there was a general feeling that although the testing laboratories enforce good quality control on the concrete, casting practices could be improved.

Concrete tensile strength, as measured by a flexural rupture test, is approximately 15% of the compressive strength obtained from cube tests. The splitting strength is approximately 70% of the flexural modulus of rupture.

4.4.2 Reinforcing Steel

Specifications regarding properties of reinforcing bars are given in NCh 204 [1]. There are three general grades for deformed bar and one for plain bar. The plain bars are available only in 6-mm diameter, all bars with diameters of 8 mm and larger are deformed bars.

Grade	Type	f_y kg/cm^2	f_u kg/cm^2	f_y ksi	f_u ksi
A44-28H	plain	2,800	4,400	40	62
A44-28H	deformed	2,800	4,400	40	62
A56-35H	deformed	3,500	5,600	50	80
A63-42H	deformed	4,200	6,300	60	90

The first number in the grade specification is the tensile strength of the steel in hundreds of kg/cm^2, and the second number is the yield stress in the same units.

Ninety-five percent of the bars are rolled within Chile and are of high quality. All iron ore comes from the same mine and only one company produces rebars. The deformed rebars generally exhibit good ductility. Some problems with the quality of imported steel have been observed, primarily with the ductility of high strength bars imported from other countries in South America. In the early 1980's, the army produced some high strength bars that were very brittle and were subsequently discontinued.

Cold-twisted steel imported from Europe was used until 1960. Cold-drawn bars may also be found in some older buildings, but the use of both types of steel is prohibited now.

CHAPTER 5
GROUND MOTION DURING THE 3 MARCH 1985 EARTHQUAKE

The Chilean earthquake is notable in part because an array of strong-motion instruments was in place to record the event. The network of instruments will enable important observations to be made on source and transmission mechanisms of the earthquake. Although studies of these mechanisms are of great importance, no attempt will be made in this chapter to address these issues. Rather, the chapter will be limited to analysis and discussion of engineering features of strong motion recorded in the epicentral region.

5.1 Strong-Motion Instrumentation

A strong-motion accelerometer has been in operation in Santiago since approximately 1940, and has recorded seven earthquakes between 1945 and 1971 [68]. More recently, the departments of Geology and Geophysics and Civil Engineering at the University of Chile have assembled a strong-motion instrumentation network comprising over 60 analog strong-motion instruments [58,68]. Most of the instruments in the network are SMA-1 accelerometers. The instruments are typically located in small buildings and the recorded motion closely resembles free-field motion.

Recognizing the likelihood of a strong earthquake in central Chile, several of the instruments were positioned and in operation in the epicentral region during the 3 March 1985 event. Approximately 15 high-quality records were obtained from stations located within 150 km of the epicenter (Fig. 1.2). Peak accelerations recorded during the earthquake are listed in Table 5.1.

The Santiago instrument that had recorded previous earthquakes did not function during the 1985 event, so no direct comparison with previously recorded earthquakes is possible. However, practicing engineers in Santiago generally agreed that the motion in 1985 was stronger than the motion recorded during the 1971 Valparaíso earthquake (0.16g peak acceleration in Santiago).

5.2 The Records

Records of the main event, obtained at five locations (Llolleo, Melipilla, Viña del Mar, Valparaíso, and El Almendral), are plotted in Fig. 5.1. No attempt was made to synchronize the records shown in Fig. 5.1. The NS component of the record obtained at El Centro during the 1940 Imperial Valley earthquake is plotted as the last record in the figure for comparison. Table 5.2 describes relevant data on the instruments and soil conditions. The Llolleo, Viña del Mar, and Valparaíso records include two horizontal and one vertical component. The vertical component is missing for El Almendral and Melipilla.

The recorded acceleration histories indicate that the ground motion was strong throughout the epicentral region. The duration of strong shaking (horizontal accelerations exceeding 0.1g) was on the order of 60 seconds for the five records considered, and peak horizontal accelerations were 0.67g in Melipilla and Llolleo.

Compared with the 1940 El Centro NS record, the Llolleo, Melipilla, and Viña del Mar records obtained during the 3 March 1985 Chilean earthquake reveal equal or higher accelerations, with significantly longer durations of strong shaking. It should be noted, however, that the CalTech digitization procedure [36] is not used in Chile. The Chilean records are digitized with unequal time increments. Therefore, peak accelerations for the Chilean and U.S. records may not be equivalent.

The Llolleo, Valparaíso, and Melipilla records were dominated by high frequency motion. The frequency content of the Viña del Mar and El Almendral records resembled more closely that of the El Centro record.

The motion recorded at stations in Viña del Mar and Valparaíso (Viña del Mar, Valparaíso, and El Almendral) is of particular interest because numerous engineered structures are located near the stations.

5.3 Response Spectra

Elastic and inelastic response spectra were computed for the acceleration records* shown in Fig. 5.1. Viscous damping ratios of 0.02, 0.05, 0.10, and 0.20 were considered for the elastic spectra. For the inelastic spectra, viscous damping in the elastic structure was 5% of critical. One set of inelastic spectra was calculated assuming an elasto-perfectly plastic response function and yield strengths, C_y, ranging from 0.1W to 0.5W, where W is the weight of the single-degree-of-freedom oscillator. A second set of inelastic response spectra was calculated for the same range of yield strengths with a strain hardening stiffness after yield equal to 20% of the elastic stiffness.

The Newmark Beta integration scheme [48] with $\beta = \frac{1}{4}$ was used for all analyses. For the inelastic response calculations, iterations were made in the duration of the time step to ensure that the response did not overshoot yield. Similar iterations were not made to determine the unloading points. No limits were placed on the maximum resistance of the strain-hardening systems. Therefore, unrealistic responses are likely for cases where large ductilities and resistances several times the yield resistance are computed.

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* The records used for response spectra calculations were corrected at the Universidad Católica de Chile using the segmental baseline adjustment procedure described in Reference 43.

5.3.1 Elastic Response Spectra

Absolute acceleration and relative displacement spectra for the horizontal components of the five records are shown in Fig. 5.2. For comparison, response spectra for El Centro NS 1940 with a damping factor of 0.02 are plotted with the Viña del Mar spectra in Fig. 5.3.

The shapes of the elastic spectra for the Viña del Mar and El Almendral records are notably similar, and reveal a characteristic ground period of approximately 0.7 sec (Fig. 5.2). Spectra for the two locations resemble the spectra calculated for the El Centro NS 1940 record for periods less than 1.5 sec (Fig. 5.3). Both displacement and acceleration ordinates of the Chilean records exceed those for the El Centro record for that period range.

Acceleration response spectra for the N10E Llolleo record reveal ordinates significantly higher than the other records in the period range less than 0.5 sec. In the period range above 0.6 sec, spectral accelerations for the N10E Llolleo record are only moderately higher than those for the S20W Viña del Mar and transverse Melipilla records. Spectral accelerations for the El Almendral and Valparaíso records are lower still, with the Valparaíso record being the lowest of the records considered.

5.3.2 Inelastic Response Spectra

Elasto-perfectly plastic response spectra for the Viña del Mar and El Almendral records are plotted in Fig. 5.4. Spectra for the strain-hardening response functions are shown in Fig. 5.5 for the same records. Figure 5.6 contains the inelastic response spectra for the El Centro NS 1940 record.

The elasto-perfectly plastic response spectra (Fig. 5.4) suggest that significant inelastic responses should be generated by the records for periods less than 1 sec. In some period ranges, large ductility demands are indicated even for systems having yield strengths equal to half the weight of the oscillator. However, as is generally accepted, spurious ductility demands often result for perfectly plastic, yielding systems.

Computed spectra for the bilinear strain-hardening systems (Fig. 5.5) indicate ductility demands markedly lower than those for the elasto-plastic response functions. However, required ductilities are substantial over a broad range of periods and strength levels.

Comparison between inelastic spectra for the Chilean records (Fig. 5.4 and 5.5) and the 1940 NS El Centro record (Fig. 5.6) suggests that ductility demands were comparable for the Viña del Mar, El Almendral, and El Centro records.

5.4 Intensity Measures

As described in Chapter 1, Modified Mercalli intensities of VII and VIII have been assigned to the Viña del Mar and Valparaíso region. Less subjective measures, based on mathematical manipulations of the measured ground motions, were computed for the present study and are listed in Table 5.3.

Housner Spectrum Intensities were computed as the area under the 5% damped response spectra between periods of 0.1 and 2.5 sec [35]. The intensities for the S20E Viña del Mar and N50E El Almendral records are similar, and compare closely with that obtained for the 1940 NS El Centro record. The intensity for the N10E Llolleo record is approximately 40 percent higher. It is noted that this intensity measure does not explicitly account for differences in duration of strong shaking.

The Arias Intensity [4] attempts to incorporate the effects of duration of strong motion into the intensity measure. The intensity is computed effectively as a constant times the integral over the duration of the earthquake of the ground accelerations squared. A trapezoidal rule was used to integrate the accelerations. As is apparent from the values in Table 5.3, this intensity measure suggests that many of the Chilean records were significantly more intense than the 1940 NS El Centro record. Figure 5.7 plots the Arias intensity versus time for the Viña del Mar and El Centro records. Similarity in slope for the plots suggests that intensities were comparable for any given length of time, but that intense ground motion lasted for a much longer period of time in the Chilean earthquake than in the 1940 Imperial Valley earthquake.

5.5 Summary

The preceding analysis suggests that, from a structural engineering point of view, the ground motions measured during the 3 March 1985 Chilean earthquake are similar to those measured during previous strong earthquakes on the west coast of the United states. Frequency content varied from station to station, but at the Viña del Mar and El Almendral stations, frequency content was observed to be similar to that observed in El Centro during the 1940 Imperial Valley earthquake. Spectral ordinates are comparable for these two Chilean records and the El Centro record. Intensities for given durations of shaking were similar for the Viña del Mar, El Almendral, and El Centro records, but duration of strong shaking was significantly longer for the Chilean records.

CHAPTER 6
AVAILABLE INFORMATION ON BUILDINGS WITH STRUCTURAL DAMAGE

Early newspaper reports identified seven seriously damaged buildings in Viña del Mar after the March 1985 earthquake. El Faro, an eight-story apartment building in Reñaca, and Barrios, a four-story building one block north of the Marga-Marga River in downtown Viña del Mar, were immediately identified for demolition. The damage in Acapulco, Hanga-Roa, Festival, Tahiti, and Coral was significant, and the buildings were considered to be uninhabitable [49]. Later investigations indicated that the damage in Festival, Tahiti, and Coral was not as heavy as first diagnosed.

Six of the buildings with structural damage have been selected for further study. Five buildings, Acapulco, Hanga-Roa, Coral, Tahiti, and Festival, are located near the beach in Viña del Mar (Fig. 6.1). The sixth structure, El Faro, was located atop a hill in Reñaca overlooking the Pacific Ocean (Fig. 6.2).

A brief description of each building is provided in this chapter. Information was obtained from the structural and architectural drawings, discussions with some of the engineers, and data on file at the Municipality*. Numerical information is summarized in Tables 6.1 and 6.2.

6.1 Acapulco

Acapulco was constructed in 1962 and was the first high-rise building in Viña del Mar (Fig. 6.3). The fifteen-story structure is located on the beach along Avenida San Martin.

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* In 1984, the basement of the Viña del Mar municipal building was flooded. A great deal of the local construction information was damaged or destroyed by the water.

A typical floor plan is shown in Fig. 6.4. The building is almost symmetrical about the longitudinal axis, Y. In the transverse direction, shear walls are oriented at 60° with respect to the central walls and corridor, forming a herringbone pattern. This arrangement of walls provided an unobstructed view of the ocean from each balcony. Most of the transverse walls are coupled to the central walls with lintel beams. However, the transverse and longitudinal walls meet to form the elevator core. The typical wall thickness is 20 to 25 cm in the first two stories and is reduced to 20 cm above level 2. The slab thickness is 12 cm, and the typical story height is 2.75 m (Fig. 6.5). The structure is supported by a 1.0-m thick mat foundation which extends to a depth of 4 m. A two-story water tank was located on the roof of the structure.

Damage was observed in the building after the 1965 and 1971 earthquakes in the region [37,45,50,62]; however, cracks in the walls were considered to be nonstructural damage. Lightweight particle-board was mounted on the walls to cover the diagonal cracks [13,68]. No structural repairs were made.

Major damage during the 1985 earthquake occurred at the northeast end of the building, in the wall along axis M′ (Fig. 6.6). Large shear cracks, buckling of the boundary reinforcement, and crushing of the concrete were observed in this region in the fourth story. Wide longitudinal cracks developed in the slabs around the elevator core (between axes E and F), where both the longitudinal and transverse walls were discontinuous and the slab was the only coupling element. Significant shear cracking was also observed in the lintel beams and diagonal cracks were common in the walls. The roof slab suffered serious damage around the water tank.

Both plain and twisted bars were used throughout the building. The twisted bars were typically used at the boundary of the wall elements and the plain bars were used inside the wall. Diagonal reinforcement, in addition to vertical and horizontal bars, was used in the shear walls (Fig. 6.6).

The total repair cost of 180,000,000 pesos was estimated to be one-third the commercial value of the building.

6.2 Hanga-Roa

Hanga-Roa is located immediately north of Acapulco along Avenida San Martin (Fig. 6.7). Construction of the building began in January 1969 and the building was officially accepted by the Municipality in June 1971, two weeks before the July 1971 earthquake.

The building is characterized by its unusual plan of two curved walls forming a 'Y' (Fig. 6.8). The open end of the 'Y' faces the street. Balconies running along the other sides of the building provide views of the ocean from all locations. The central corridor is bounded by the two main longitudinal walls, C and I, and transverse walls run radially. The elevators and stairs are located at the intersection of the two legs with the stem of the 'Y'.

Continuous footings support the structure at a depth of 4.15 m. The wall thicknesses are 30 cm in the first two stories, 25 cm in stories three and four, and 20 cm in the upper stories. Typical slab thicknesses are 20 cm in the corridors and 12 cm in the apartments. All stories have a height of 2.75 m. The locations of some doors are staggered in adjacent stories (Fig. 6.9).

After the 1971 earthquake, damage was concentrated near the intersection of longitudinal walls C and I with radial walls 4 and 18. Lintel beams were severely damaged and cracks were observed in the longitudinal walls [37,50,62]. The damage was concealed, but not repaired.

In 1985, the major damage occurred near the intersection of walls I and 10 [6]. Apartment doors were staggered in this area, and wide vertical cracks connected the doors on alternate floors (Fig. 6.10). Damage was also heavy at the intersection of walls C, I, 4, and 18.

Cracks were identified and measured during an inspection of the building shortly after the earthquake [6]. Approximate lengths are listed below:

Structural Elements:	
Horizontal and inclined cracks in R/C walls	1500 m
Vertical cracks in R/C walls	50 m
Cracks in the slabs	400 m
Vertical or inclined cracks in lintel beams	70 m
Multiple inclined cracks in the lintel beams	100 m
Vertical or inclined cracks in beams	70 m
Nonstructural Elements:	
Horizontal and inclined cracks in masonry partitions	1200 m
Horizontal cracks through masonry partitions and R/C beams or slabs	300 m
Vertical cracks through masonry partitions and R/C columns or walls	200 m

Although the building is nearly symmetric about axis F, damage was noticeably more severe in wall I than in wall C. The total cost of repair was 225,000,000 pesos.

6.3 Coral

Coral (Fig. 6.11) is a twelve-story apartment building located across the street from Hanga-Roa. The building was constructed in 1968.

A typical floor plan is shown in Fig. 6.12. The building is nearly rectangular with two small wings extending to the east. The typical story height is 2.8 m and floor slabs are 16-cm thick. Structural walls are 20-cm thick throughout the building.

Coral suffered moderate to heavy structural damage. Cracks were observed throughout the walls and floor slabs. A column was severely damaged in the first story, and lintel beams were cracked at all levels. The damage in one lintel beam in the northeast corner of the building sustained such large deformations that the location of the door was changed in stories 1 through 11. The damaged beams and doorways were incorporated into the adjoining walls.

Masonry partition walls also collapsed during the earthquake. The engineer recommended replacing the masonry partitions with lightweight materials, such as volcanita. The cost to repair the structural elements and replace the nonstructural partitions was less than 16,000,000 pesos.

6.4 Tahiti

Tahiti (Fig. 6.13) is a fifteen-story building and was constructed in 1970. The building is located immediately north of Coral. Drawings were not available for the structure.

The damage caused by the 1985 earthquake was described in three main categories: cracking of the central north-south wall in the first story, lintel beams were severely cracked throughout the building, and masonry partitions collapsed in the upper stories. The repairs to the building included grouting the cracks in the structural walls, restoring the lintel beams to their original size, and replacing the damaged masonry partitions in the common areas. The cost associated with these repairs was less than 3,000,000 pesos. Nonstructural damage in the individual apartments was repaired by the owner.

6.5 Festival

Festival is a 14-story apartment building located at Calle 9 Norte No. 450 (Fig. 6.14-6.16). The building was designed in 1978. The building plan has what may be described loosely as an "H" shape (Fig. 6.15). The structural walls are arranged almost symmetrically in plan, and, with the exception of some wall openings that are staggered from floor to floor, the walls are effectively continuous over height.

The foundation is a mat founded 7 m below grade. The typical story height is 2.65 m (Fig. 6.16). Slabs are typically 13-cm thick. Structural walls are coupled by relatively deep beams at several locations. Wall thicknesses are 30 cm for stories 1-4, 25 cm for stories 5-9, and 20 cm above the ninth story.

Structural damage consisted of cracking in lintels along the major longitudinal axes of the building, diagonal cracks in several shear walls aligned parallel to the transverse direction, and crushing of a wall boundary (axis I in Fig. 6.15) where it intersected a perimeter reinforced concrete retaining wall. Masonry walls in bathrooms along the north side of the building collapsed during the earthquake.

Crack lengths were measured to be:

Structural Elements:	
Walls	1920 m
Beams	330 m
Slabs	110 m
Nonstructural Elements:	
Partition walls	900 m
Masonry walls	560 m

The cost of the structural repairs was approximately 41,000,000 pesos.

6.6 El Faro

El Faro was an eight-story apartment building located in Reñaca (Fig. 6.17). The building was designed in 1979 and constructed in 1980. The structure was severely damaged during the 1985 earthquake and was demolished on 8 March 1985.

Structural plans for the foundation and a typical story are shown in Fig. 6.18. All levels were nearly square in plan; however, the basement and upper stories were not aligned vertically. The typical story height was 2.63 m (Fig. 6.19) and the slab thickness was 14 cm. Structural walls in the upper stories were 20-cm thick and 30-cm thick in the basement.

The architectural plans (Fig. 6.20) identify three bay windows at each story level that are not shown on the structural drawings. Minor changes in the locations of some interior walls are also found on the architectural drawings. The as-built configuration of El Faro more closely resembled the architectural drawings than the structural drawings. The roof of El Faro was constructed of wood and formed a one-story penthouse. Details for the penthouse were not shown on either the architectural or structural drawings, and therefore, the total height of the building is not known.

The structure was supported by a continuous footing foundation. Because the building was located on the top of a steep hill, the base of the foundation was 3 m below grade on the southeast side and 2 m below grade on the northwest side which overlooked the ocean.

During the earthquake, the building leaned approximately 15° to the northwest, crushing walls A, K, and M below the first-floor slab (Fig. 6.21). The building was demolished before an extensive investigation could be conducted [20]. Honeycombed concrete and poorly executed construction joints were observed in the foundation of the structure which remains at the site (Fig. 6.22).

Concrete and steel samples were removed from the building after the collapse. Test indicated that material strengths were above those specified on the structural drawings.

6.7 Summary

Extensive damage was observed in Acapulco, Hanga-Roa, and El Faro after the 3 March 1985 earthquake. The repair of Acapulco and Hanga-Roa was in progress in July 1986 and focused on making the buildings stiffer and stronger. The thicknesses of damaged walls was increased in both buildings and lintel beams were removed, presumably to avoid coupling problems. Pressurized grout was used to fill cracks in areas with less extensive damage. An effort was made to reduce the weight of Acapulco by replacing all the masonry partition walls with volcanita. The water tanks were also relocated from the roof to the basement of Acapulco.

The repair of Coral, Tahiti, and Festival had been completed by the summer of 1986. The repair projects in these buildings focused on returning each building to its condition before the earthquake.

CHAPTER 7
AVAILABLE INFORMATION ON UNDAMAGED OR LIGHTLY DAMAGED BUILDINGS

Seven buildings that were undamaged or lightly damaged were selected for study in this report. Six structures are located in the downtown area of Viña del Mar within 1.5 km of the strong-motion instrument that was located in the city (Fig. 6.1 and 7.1). The other building is located in Valparaíso, approximately 300 m from the site of the El Almendral record. All of the buildings were less than 15 years old and the 1985 earthquake represented the first significant ground motion.

The structural framing system, observed damage, and history of each building is described briefly in this chapter. Numerical information is summarized in Tables 7.1 and 7.2.

7.1 Plaza del Mar

Plaza del Mar is a 23-story building located approximately 100 m south of Acapulco on Avenida San Martin (Fig. 7.2- 7.4). The building design was completed during 1980-81. Construction was completed in 1983 and the building in service at the time of the 1985 earthquake. The building plan (Fig. 7.3) comprises two similar portions connected by a relatively narrow slab. Structural walls are aligned in pairs to form a central corridor parallel to the longitudinal axis of the building. The majority of the remaining walls are "L" shaped, and frame into the central corridor (Fig. 7.3). The plan of the building is effectively unchanged over the building height.

A 1.5-m thick cellular mat foundation is founded at 7 m below grade. The typical story height is 2.68 m, with first story height of 4.69 m (Fig. 7.4). The floor system comprises beams framing between columns and walls, with a floor slab having typical thickness of 14 cm. Wall thicknesses vary, but are typically 30 cm in the lower 12 stories, and 25 cm above the twelfth floor.

Damage was reported by the structural engineer to be light. Typical damage comprised horizontal hairline cracks in the floor slab located at the middle of the building plan, apparently occurring as a consequence of slab coupling of the two "halves" of the building. Cracks were also observed at stair landings, in the nonstructural lintel beams, and in some walls. A water tank located on the roof straddling the central corridor walls sustained minor damage, apparently due to coupling of the walls not anticipated during design. Damage was sufficiently light that the tank was holding water following the earthquake.

7.2 Marina Real

Marina Real is a 20-story building located at Avenida San Martin 890 (Fig. 7.5-7.7). According to dates on the structural drawings, design was underway in 1980. The building was constructed during 1983. The building plan is approximately rectangular (Fig. 7.6). Lateral load resistance is apparently provided primarily by structural walls. The plan changes between the first and fourth stories, primarily due to a gradual reduction in plan of some of the walls at the perimeter of the building. Above the fourth floor, changes in plan are minor.

The foundation is a cellular mat having thickness of 200 cm at a depth of approximately 740 cm. The typical story height is 2.7 m, with a first story height of 3 m (Fig. 7.7). Floor framing comprises beams and slabs. Wall thicknesses are typically 50 cm for the basement through third story, 40 cm through the eighth story, 30 cm through the 13th story, 25 cm through the 17th story, and 20 cm through the 20th story.

According to the structural engineer, the only major damage occurred in the curved portion of the perimeter walls. Examination of the damage and of the reinforcement details suggests that this damage was initiated by straightening, under tension, of the wall boundary reinforcement that had been placed to follow the curve of the wall. In addition, minor damage was observed in some of the lintels. The total cost of repairs was estimated by the Municipality to be 5,000,000 pesos.

7.3 Torres de Miramar

The two triangular structures composing the condominium complex Torres de Miramar are located at Avenida San Martin 1020 and 1080 (Fig. 7.8). The buildings are one block north of Hanga-Roa on the east side of the street. Design of the structures was completed in 1973 and the buildings were constructed in 1975 and 1976. Each building is 21 stories in height (Fig. 7.10). The plan is in the shape of an equilateral triangle with sides of 34.9 m (Fig. 7.9). Four exterior columns are located along each edge and the corners cantilever approximately 4.5 m beyond the columns. The typical story height is 2.55 m, the first and second stories are slightly taller.

A continuous footing foundation supports the structure with the base of the foundation located 6 m below grade. The first and second stories are enclosed only in the core area, forming a small pedestal at the base of the structures (Fig. 7.9). Above the second level, the floor slabs span from the core region to the external columns and the shape of the core is changed so that the radial walls extend to the exterior columns (Fig. 7.9). The shape of the columns changes above the second level from a rhomboid to an equilateral triangle. The locations of apartment doors leading into the central core area are staggered at every level (Fig. 7.10). The thickness of the central core walls varies from 40 cm at the base to 20 cm above the 15th floor. The typical slab thickness is 20 cm.

No structural damage was reported in these buildings. The nonstructural damage was limited to partition walls which separated from the structure in the cantilevered corners of the plan. The soil around the perimeter of both structures was observed to have settled as much as 30 cm following the earthquake [68], and was attributed to settlement of the backfilled material.

7.4 Torres del Pacifico

The three identical 17-story buildings that compose Torres del Pacifico are located one block north of Torres de Miramar and two blocks from Hanga-Roa. Construction of the buildings began in 1979 and extended through 1981. Two of the towers, located at San Martin Avenida 1130 and 1206, were completed at the time of the earthquake. However, economic conditions forced the construction of the third building to be halted before any nonstructural elements were added (Fig. 7.11). The plan of each building is a modified equilateral triangle (Fig. 7.12). One corner of the plan is truncated to form a 5-m edge that faces the ocean. The rear of each building cuts back into the main portion forming a 'V' shape. A two-story entrance foyer projects from the top of the 'V' (Fig. 7.11 and 7.12).

Continuous footings at a depth of 6 m support the structures. Four trapezoidal columns are located along the edges which point toward the ocean and are connected by beams. Walls run perpendicular to the exterior edges toward a central wall which bisects the angle between the two column lines. The typical slab thickness is 16 cm. Additional reinforcement was placed in the slab to form lintel beams to couple the walls over most of door openings. The exterior columns remain the same size over the entire height (Fig. 7.13), and the thicknesses of the main walls vary from 30 to 20 cm. The locations of the apartment doors are not staggered (Fig. 7.13).

Small cracks were observed in the masonry walls of the upper stories and small diagonal cracks were observed in the edge beams. Cracks were observed in the lintel beams and in the slabs at all levels. Compaction of the fill material surrounding the foundation was also observed.

7.5 Villa Real

The Villa Real complex consists of a ten-story residential building, a subterranean parking structure that surrounds the building on three sides, and a small commercial center adjacent to the fourth side of the building. Villa Real (Fig. 7.14) is located at 5 Norte/4 Poniente close to the downtown area of Viña del Mar. The building design was finished in 1981 and construction was completed in 1983.

The main structure is a slab-girder-shear wall system, the shear walls being irregularly spaced and varying dramatically in plan from the first to second story (Fig. 7.15). Stories two through eight are similar in plan, while stories nine and ten again differ. Shear wall thicknesses range from 20 to 30 cm at the base of the structure. The majority of the walls in the upper stories are 20-cm thick. Floor slabs are 12 and 16-cm thick. Typical slab-girder-column framing systems are used for both the parking structure and the commercial center.

The total building complex plan area is approximately 800 square meters at the foundation level. The main building, the parking structure, and the commercial center are connected at the foundation level by a network of foundation beams. Continuous wall footings on a 70-cm mat-type foundation support the main building, while the parking structure rests on a continuous wall footing along its outer edges and on spread column footings within its perimeter. The commercial center rests on square footings located at varying depths. At the ground level, the parking structure girders are supported by corbels extending out from the main building in order to provide expansion relief.

Damage was concentrated near the base of the structure. Diagonal cracks were observed in the structural walls in the first, second, and third stories. Most cracks were less than 0.2-mm wide. The concrete cover spalled from the lintel beams along axes D and E.

7.6 Torres del Sol

Torres del Sol (Fig. 7.17) is a 22-story building located three blocks south of Hanga-Roa at 8 Norte/4 Poniente. Construction was completed in 1982. The plan of the building is a 24.3 m square above the first level (Fig. 7.18). At the first level, the four exterior columns are connected to the central core and to each other by beams of varying depths. Above the first level, the core walls are extended and incorporate the exterior columns in the transverse direction. This change in wall location creates a significant discontinuity in the main transverse shear walls in the first story. The second-story walls act as a transfer girder to carry the lateral load from the upper stories to the first-story walls.

The foundation is a 2-m thick cellular mat located 4.85 m below grade. The typical story height is 2.65 m with the first and second stories 5.30 and 2.99 m respectively. Balconies cantilever 1.8 m past the exterior columns in the direction of column lines A and D (Fig. 7.19). The typical slab thickness is 16 cm. The thickness of the core walls varies from 50 cm at the second level to 20 cm above level 18. The locations of apartment doors leading into the central core at each level are staggered in walls B and C, and remain at the same locations in walls F and G (Fig. 7.19).

Damage in the structure was concentrated in the second floor. Diagonal cracks were observed in the walls and lintel beams. Cracks in the floor slabs were also observed. Cracks were observed in the lintel beams at other levels and there were some failures at the connection between the stairs and the slab. The estimated cost of repair to the structure was 1,000,000 pesos.

7.7 Torre del Almendral

The 23-story Torre del Almendral rises to a height of 60.2 m above ground (Fig. 7.20). Almendral is the only structure considered which is located in Valparaíso. The date of construction was not determined. However, the structural drawings are dated January 1972. The building's plan is almost square (Fig. 7.21) and is made up of nine panels. The structural system is composed of two back-to-back, channel-shaped shear walls that are coupled with two three-bay frames through girders and the floor slabs. The channel-shaped walls extend from the foundation level to the 21st story (Fig. 7.22). The web portions of the walls continue to the 23rd floor, where they are joined together forming a sloping roof above the 23rd floor (Fig. 7.22). The web portion varies in thickness from 45 cm at the first story to 30 cm above the seventh and has staggered door openings along its elevation.

The building rests on a 1-m thick mat foundation with its base located 6.18 m below grade. The typical story height is 2.55 m with the first story being 3.87 m. The plan view of the floors above the third level is fairly typical, although openings in the central panel of the 18-cm thick slab are staggered. Column cross-sections remain rectangular and uniform along the height.

No information was available about damage sustained during the earthquake.

7.8 Summary

Investigation of the seven buildings described in this chapter and the six buildings in Chapter 5 indicates that structural walls are common in Chilean buildings and are the primary lateral-load resisting system in the buildings. As a result, Chilean buildings are much stiffer than buildings of the same height located in seismic regions of the U.S. The fundamental natural periods for ten of the buildings described in this report were determined from forced vibration tests of the buildings [15] and from measured response during aftershocks. These data are plotted in Fig. 7.23 as a function of the number of stories. The measured periods during low-amplitude excitation after the 1985 earthquake may be approximated as N/20, where N is the number of stories.

CHAPTER 8
SUMMARY

The western coast of Chile lies along the boundary between the Nazca and South American Plates. As a result, Chile is located in one of the most active seismic regions of the world. The seismicity of the region has been studied since the mid-1500's and more than twenty significant earthquakes, $M \geq 8$, have been documented [40].

In the central region, the return period for an earthquake of magnitude 7.5 or greater is approximately 80 years [23]. The earthquake of 3 March 1985 was not unexpected and a network of accelerometers had been located in the region. Thirty-one strong-motion records (Fig. 1.2) representing many soil conditions were obtained during the earthquake [61].

The motion recorded in the coastal city of Viña del Mar was similar in frequency content to the motion obtained during the 1940 Imperial Valley earthquake. However, the peak ground acceleration and duration of the Viña del Mar record exceeded that of the 1940 El Centro motion. The majority of the reinforced concrete buildings considered in this report were located within 1.5 km of the site of the strong-motion instrument in Viña del Mar.

The soil in this region comprises consolidated sand and gravel. Foundation failures were not observed in this material. Damage was high in the area surrounding the harbor in Valparaíso where artificial fill extended to a depth of 10 m near the water. Heavy damage was also observed in the saturated, weathered rock on the slopes around Viña del Mar and Valparaíso.

Of the 415 buildings in Viña del Mar ranging in height from 5 to 23 stories, six may be identified as having suffered substantial structural damage (excluding Canal Beagle). In the downtown area, the two severely damaged buildings, Acapulco and Hanga-Roa, were among the oldest buildings above 12 stories in height and had suffered damage during previous earthquakes. Tests conducted during aftershocks indicated that site amplification may have been a problem for El Faro [17], which was located on top of a hill in Reñaca, overlooking the ocean. However, controversy surrounds the design and construction of the building, and the collapsed structure was destroyed a few days after the earthquake before any investigations of the as-built conditions could be conducted. Although structural damage did occur in Coral, Tahiti, and Festival, the damage was concentrated in specific areas of the buildings.

In addition to the six buildings mentioned above, this report documents conditions of six other buildings located near the strong-motion instrument in Viña del Mar and one near the El Almendral instrument in Valparaíso. These buildings sustained little or no structural damage during the 1985 earthquake.

A common characteristic of buildings in Chile is the relatively high proportion of structural walls compared with typical U.S. buildings in seismic regions. This characteristic has evolved as a consequence of good performance of this structural system during Chile's history of frequent, strong ground motions. Because of the large number of walls, the buildings are comparatively stiff and redundant, and apparently develop high lateral-load strengths with relatively low amounts of reinforcement.

Although apparent deficiencies in detailing, construction, and inspection have been identified, reinforced concrete buildings generally performed well. An implication is that the Chilean practice in detailing, construction, and inspection is adequate for the type of structural system prevalent in the country. Research is continuing to determine the relationships between the Chilean practice and the observed good performance.

TABLES

Table 2.1 Segments of Inclined Seismic Zones Along the West Coast of Central and South America [8]

Geographical Location	Latitude (°S)	Dip Angle (°)
Southern Ecuador	0 - 2	25 - 30
Northern and Central Peru	2 - 15	10
Southern Peru and Northern Chile	15 - 27	25 - 30
Central Chile	27 - 33	10
Southern Chile	33 - 45	25 - 30

Table 2.2 Summary of Major Earthquakes in Chile [40]

Date	Epicentral Region	Magnitude
1570 February 8	Concepción	8 - 8½
1575 March 17	Santiago	7 - 7½
1757 December 16	Valdivia	8½
1604 November 24	Arica	8¼ - 8½
1615 September 16	Arica	7½
1647 May 13	Santiago	8½
1657 March 15	Concepción	8
1681 March 10	Arica	7 - 7½
1687 July 12	San Felipe	7 - 7½
1715 August 22	Arica	7½
1730 July 8	Valparaíso	8 3/4
1737 December 24	Valdivia	7½ - 8
1751 May 25	Concepción	8 - 8¼
1796 March 30	Copiapó	8½ - 8
1819 April 3,4,11	Copiapó	8¼ - 8½
1822 November 19	Valparaíso	8½
1829 September 26	Valparaíso	7
1835 February 20	Concepción	8 - 8¼
1837 November 7	Valdivia	8+
1847 October 8	Illapel	7 - 7¼
1849 November 17	Coquimbo	7½
1850 December 6	Valle de Maipo	7 - 7½
1851 April 2	Casablanca	7 - 7½
1859 October 5	Copiapó	7½ - 7 3/4
1868 August 13	Arica	8½
1869 August 24	Pisagua	7 - 7 3/4
1871 October 5	Iquique	7 - 7½
1877 May 9	Pisagua	8 - 8½
1879 February 2	Estrecho de Magallanes	7 - 7½
1880 August 15	Illapel	7½ - 8
1906 August 16	Valparaíso	8.6
1918 December 4	Copiapó	7½+
1922 November 10	Huasco	8.4
1928 December 1	Talca	8.4
1939 January 24	Chillán	8.3
1943 April 6	Illapel	8.3
1949 December 17	Punta Arenas	7½
1950 December 9	Chillán	8.3
1953 May 6	Chillán	7½
1960 May 21	Concepción	7.5
1960 May 22	Valdivia	8.5
1965 March 28	Santiago	7.3
1971 July 8	Valparaíso	7.5
1975 May 10	Lebu	7.8
1985 March 3	San Antonio	7.8

Table 2.3 Summary of Earthquakes Along the Chilean Coast from 1965 to 1985 [65]

Date	Coordinates °S	°W	Magnitude	Intensity in Valparaíso/Viña
12 Jun 65	20.3	68.9	6.5	
18 Jan 65	37.7	72.9	5.3	
23 Feb 65	25.7	70.5	7.25	VII
22 Mar 65	31.9	71.5	6.0	
28 Mar 65	32.4	71.2	7.0	
20 Aug 65	19.0	69.1	6.5	
3 Oct 65	42.9	75.4	6.0	
28 Nov 65	45.6	72.4	6.0	
10 Apr 66	31.5	71.2	6.0	
12 Apr 66	38.1	83.0	6.0	
22 Dec 66	25.5	70.7	7.6	
13 Mar 67	40.1	74.5	7.3	
11 May 67	20.3	68.5	6.75	
4 Jul 67	38.1	73.4	6.5	
26 Sep 67	30.0	71.5	6.75	
15 Nov 67	28.7	71.2	6.2	
21 Dec 67	21.8	70.0	7.0	
25 Dec 67	21.5	70.4	6.75	
6 Jan 68	27.8	71.1	6.3	
26 Apr 69	30.6	71.4	6.0	
26 Apr 69	30.6	71.5	6.3	
13 Nov 69	27.8	71.6	6.0	
14 Jun 70	52.0	73.8	6.6	
18 Nov 70	20.9	69.8	6.0	
4 Dec 70	23.1	70.1	6.7	
8 Dec 70	30.7	71.2	6.7	V
17 Jun 71	25.5	69.2	7.0	
9 Jul 71	32.5	71.2	7.5	VIII-IX
11 Jul 71	32.3	71.8	6.3	
8 Dec 71	22.9	70.8	5.6	
8 Jun 72	30.5	71.8	6.6	VI
29 Dec 72	30.6	71.0	6.0	
31 Jul 73	27.1	71.5	6.3	
7 Aug 73	26.8	70.9	5.8	
7 Aug 73	26.8	70.9	6.2	
5 Oct 73	33.0	71.9	6.5	VII
5 Oct 73	32.5	71.5	6.3	VI
2 Jan 74	22.5	68.5	6.8	
18 Aug 74	38.5	73.4	7.0	
13 Mar 75	29.9	71.3	6.9	
10 May 75	38.1	73.2	7.8	
28 Oct 75	22.9	70.5	6.3	
28 Feb 76	40.00	74.73	6.0	
30 May 76	41.64	75.41	6.0	
20 Aug 76	20.41	69.99	6.0	
24 Aug 76	25.33	70.69	5.6	

Table 2.3 (cont.) Summary of Earthquakes Along the Chilean Coast from 1965 to 1985 [65]

Date	Coordinates °S	Coordinates °W	Magnitude	Intensity in Valparaíso/Viña
29 May 78	44.85	79.41	6.0	
3 Aug 78	26.51	70.54	7.0	
14 May 79	22.81	69.12	6.5	
26 May 80	19.36	69.20	6.1	
23 Mar 81	33.66	71.80	6.2	VI
16 Oct 81	33.13	73.07	7.2	VI
7 Nov 81	32.20	71.37	6.8	VII
4 Oct 83	26.54	70.56	7.4	
9 Oct 83	26.14	70.52	6.2	
11 Jun 84	30.71	71.18	6.3	
18 Jan 85	29.37	70.79	6.0	
3 Mar 85	33.14	71.87	7.8	VIII
3 Mar 85	32.74	71.22	7.0	
4 Mar 85	33.21	71.66	6.7	
4 Mar 85	34.12	71.91	6.2	
4 Mar 85	32.93	71.79	6.6	
4 Mar 85	33.83	71.93	6.0	
4 Mar 85	33.14	72.00	6.3	
4 Mar 85	33.84	71.25	6.0	
4 Mar 85	32.88	71.82	6.0	
17 Mar 85	32.63	71.55	6.6	VII
19 Mar 85	33.20	71.65	6.6	VI
25 Mar 85	34.25	72.19	6.4	
3 Apr 85	32.58	71.66	6.3	
9 Apr 85	34.13	71.62	7.5	VI
28 Apr 85	39.73	75.66	6.1	V
12 Aug 85	38.38	73.5	6.0	III

Table 2.4 Typical Engineering Properties of Foundation Materials in the Viña del Mar Region [3]

Maicillos:	
Classification	SM with some SC
Liquid Limit	20%
Plastic Limit	7%
Natural moisture content	7%
% fines passing a #200 sieve	12%
Internal angle of friction	33° - 54°
Effective cohesion	0.07 - 0.35 kg/cm^2
Uncemented Sands:	
Classification	SP
% fines passing a #200 sieve	1%
Relative density	70%
Internal angle of friction	38° - 41°
Effective cohesion	0
Cemented Sands:	
Classification	SP
% fines passing a #200 sieve	2%
Natural moisture content	7%
Internal angle of friction	39° - 43°
Effective cohesion	0.04 - 0.57 kg/cm^2

Table 5.1 Peak Accelerations Measured during the 1985 Earthquake [61]

Station	Coordinates S	Coordinates W	Component	Maximum Acceleration (g)
Illapel	31°38′	71°10′	N20W	0.12
			S70W	0.10
Los Vilos	31°55′	71°30′	NS	0.25
			EW	0.04
La Ligua	32°30′	71°06′	N70W	0.19
			S20W	0.13
Papudo	32°31′	71°27′	N50E	0.13
			S40E	0.47
Zapallar	32°34′	71°28′	NS	0.32
			EW	0.33
San Pedro	32°36′	71°00′	NS	0.60
			EW	0.57
Ventanas	32°40′	71°37′	NS	0.18
			EW	0.18
San Felipe	32°45′	70°44′	S10E	0.35
			N80E	0.43
Llay-Llay	32°50′	70°58′	N80W	0.36
			S10W	0.49
Saladillo	32°55′	70°28′	NS	0.11
			EW	0.09
Valparaíso (El Almendral)	33°01′	71°38′	N50E	0.29
			S40E	0.16
Valparaíso	33°01′	71°38′	S20E	0.16
			N70E	0.18
Viña del Mar	33°02′	71°35′	N70W	0.23
			S20W	0.36
Peldehue	33°08′	70°41′	EW	0.64
			NS	-
Quintay	33°16′	71°19′	NS	0.20
			EW	0.18

Table 5.1 (cont.) Peak Accelerations Measured during the 1985 Earthquake [61]

Station	Coordinates S	Coordinates W	Component	Maximum Acceleration (g)
Santiago	33°27′	70°40′	NS	0.11
			EW	0.11
Llolleo	33°41′	71°36′	S80E	0.43
			N10E	0.67
Melipilla	33°41′	71°13′	EW	0.60
			NS	0.67
Rapel	34°01′	71°40′	NS	0.31
			EW	0.14
Pichilemu	34°23′	72°01′	NS	0.27
			EW	0.18
San Fernando	34°36′	71°00′	NS	0.23
			EW	0.34
Iloca	34°55′	72°13′	NS	0.22
			EW	0.28
Hualañé	34°58′	71°49′	NS	0.17
			EW	0.14
Constitución	35°18′	72°19′	NS	0.14
			EW	0.08
Talca	35°26′	70°40′	N80W	0.16
			N10E	0.17
Colbún	35°43′	71°26′	NS	0.04
			EW	0.03
El Colorado	35°43′	71°26′	NS	0.12
			EW	0.11
Pehuenche	35°44′	71°14′	NS	0.03
			EW	0.02
Cauquenes	36°00′	72°13′	NS	0.09
			EW	0.12
Chillán Viejo	36°36′	72°06′	N80E	0.06
			N10W	0.07
Chillán Nuevo	36°40′	72°09′	N20W	0.11
			N70E	0.07

Table 5.2 Strong-Motion Instrument Data [61,68]

Station	Coordinates S	Coordinates W	Peak Ground Acceleration (g) L	Peak Ground Acceleration (g) T	Peak Ground Acceleration (g) V	Site Geology	Structure	Instrument Location	Instrument Model
Vina del Mar	33°02'	71°35'	0.36	0.22	0.19	Sand	10-story building	Basement	SMA-1
Valparaiso	33°01'	71°38'	0.17	0.19	0.12	Rock	1-story building	Ground level	SMA-1
El Almendral	33°01'	71°38'	0.29	0.16	-	Artificial Fill	Church		SMA-1
Llolleo	33°41'	71°36'	0.67	0.43	0.86	Sand	1-story building	Basement	SMA-1
Melipilla	33°41'	71°13'	0.67	0.60	0.59	Rock	1-story building	Ground level	SMA-1

Table 5.3 Housner and Arias Intensities

Record	Housner SI (m)	Arias Intensity (g^2-sec)
Llolleo N10E	1.96	1.64
Llolleo S80E	1.07	0.75
Viña del Mar S20W	1.46	0.58
Viña del Mar N70W	0.98	0.34
Valparaíso S20E	0.26	0.08
Valparaíso N70E	0.71	0.12
El Almendral S40E	0.77	0.24
El Almendral N50E	1.33	0.35
El Centro NS	1.34	0.16

Table 6.1 Physical Characteristics of Buildings with Structural Damage

Building	Address	Year Designed	Year Constructed	Number of Stories	Typical Story Height	Total Height	Wall Area* / Floor Area	Slab Thickness
Acapulco	San Martin 821	1961	1964	15	2.75 m (9'-0")	41.25 m (135.4')	6.8% (1) 5.8% (2)	12 cm
Hanga-Roa	San Martin 925	1968	1970	15 +2 mech	2.75 m (9'-0")	46.75 m (153.4')	9.6% (1) 6.3% (2)	12&20 cm
Coral	San Martin 928		1969	12	2.80 m (9'-2")	34.80 m (114.2')	5.2% (11)	16 cm
Tahiti	San Martin 972		1969	15				
Festival	9 Norte 450	1978	1979	14	2.65 m (8'-8")	37.25 m (122.2')	5.2% (1) 4.6% (5)	13-14 cm
El Faro	(Renaca)	1979	1980	8 + pent	2.63 m (8'-7")		6.8% (1)	14 cm

* The floor level is given within parentheses.

Table 6.2 Material Properties and Description of Foundations of Buildings with Structural Damage

Building	Concrete* Strength (kg/cm^2)	Steel** Specification	Type of Foundation	Depth of Foundation (m)
Acapulco	225	(f_y = 4200 kg/cm^2) twisted	Cellular Mat	6.45
Hanga-Roa	300	A63-42H	Continuous Footing	4.15
Coral	300	(f_y = 4000 kg/cm^2)		
Tahiti	300		Mat	
Festival	300	A63-42H	Mat	7.0
El Faro	225	A63-42H	Continuous Footing	2.0 - 3.0

* Specified 28-day cubic resistance (R_{28}).

** A63-H42 corresponds to Grade 60 steel (See Section 4.4.2).

Table 7.1 Physical Characteristics of Undamaged or Lightly Damaged Buildings

Building	Address	Year Designed	Year Constructed	Number of Stories	Typical Story Height	Total Height	Wall Area* / Floor Area	Slab Thickness
Plaza del Mar	San Martin 821	1980	1982	23 +2 mech	2.68 m (8'-9")	68.42 m (224.5')	6.5% (3)	14 cm
Marina Real	San Martin 890	1980	1983	20 + mech	2.70 m (8'-10")	51.70 m (169.6')	9.1% (1) 7.1% (7)	12-16 cm
Torres de Miramar	San Martin 1020 San Martin 1080	1973	1975 1976	21 +3 mech	2.55 m (8'-4")	63.71 m (209.0')	4.7% (3)	20 cm
Torres del Pacifico	San Martin 1130 San Martin 1206 San Martin	1979	1980 1981	17 +2 mech	2.62 m (8'-7")	51.06 m (167.5')	8.0% (3)	16 cm
Villa Real	5 Norte/4 Poniente	1981	1983	10	2.55 m (8'-4")	28.20 m (92.5')	7.2% (1) 5.9% (2)	12-16 cm
Torres del Sol	8 Norte/4 Poniente	1979	1982	22 +2 mech	2.65 m (8'-8")	67.15 m (220.3')	6.5% (1) 5.9% (3)	16 cm
Almendral	(Valparaiso)	1972		22 +1 mech	2.55 m (8'-4")	60.21 m (197.5')	4.3% (1) 3.4% (2)	18 cm

* The floor level is given within parentheses.

Table 7.2 Material Properties and Description of Foundations of Undamaged or Lightly Damaged Buildings

Building	Concrete* Strength (kg/cm^2)	Steel** Specification	Type of*** Foundation	Depth of Foundation (m)
Plaza del Mar	300	A63-42H	Cellular Mat (1.5 m)	7.0
Marina Real	350	A63-42H	Cellular Mat (2 m)	7.4
Torres de Miramar	300	A63-42H	Continuous Footing (1 m)	6.0
Torres del Pacifico	300	A63-42H	Mat (1 m)	5.5
Villa Real	300	A63-42H	Continuous Footing	3.8
Torres del Sol	300	A63-42H	Cellular Mat (2 m)	5.85
Almendral	300		Mat (1 m)	6.18

* Specified 28-day cubic resistance (R_{28}).

** A63-H42 corresponds to Grade 60 steel (See Section 4.4.2).

*** The mat and footing thicknesses are given within parentheses.

FIGURES

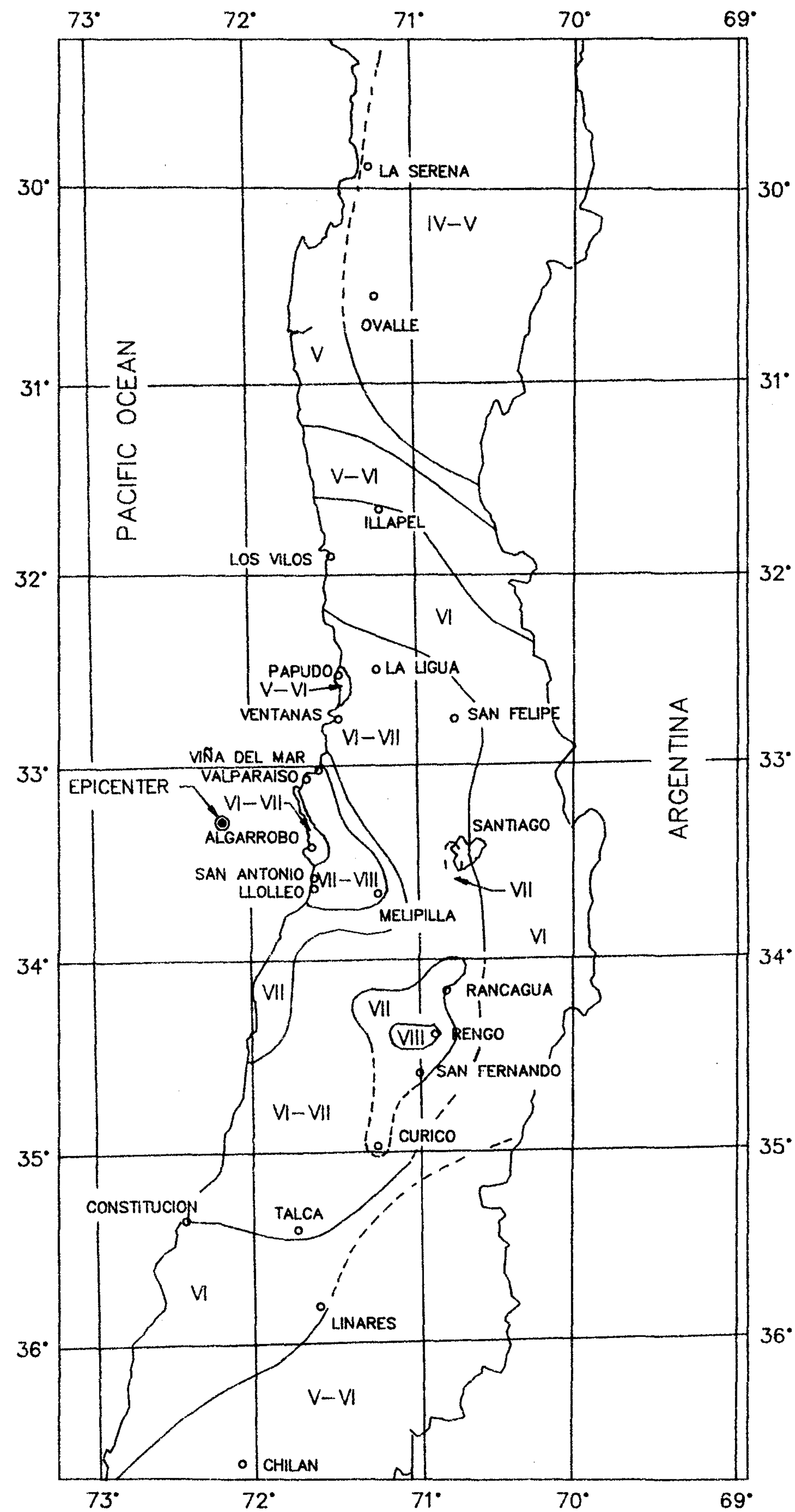


Fig. 1.1 Intensity Map from the 3 March 1985 Earthquake [61]

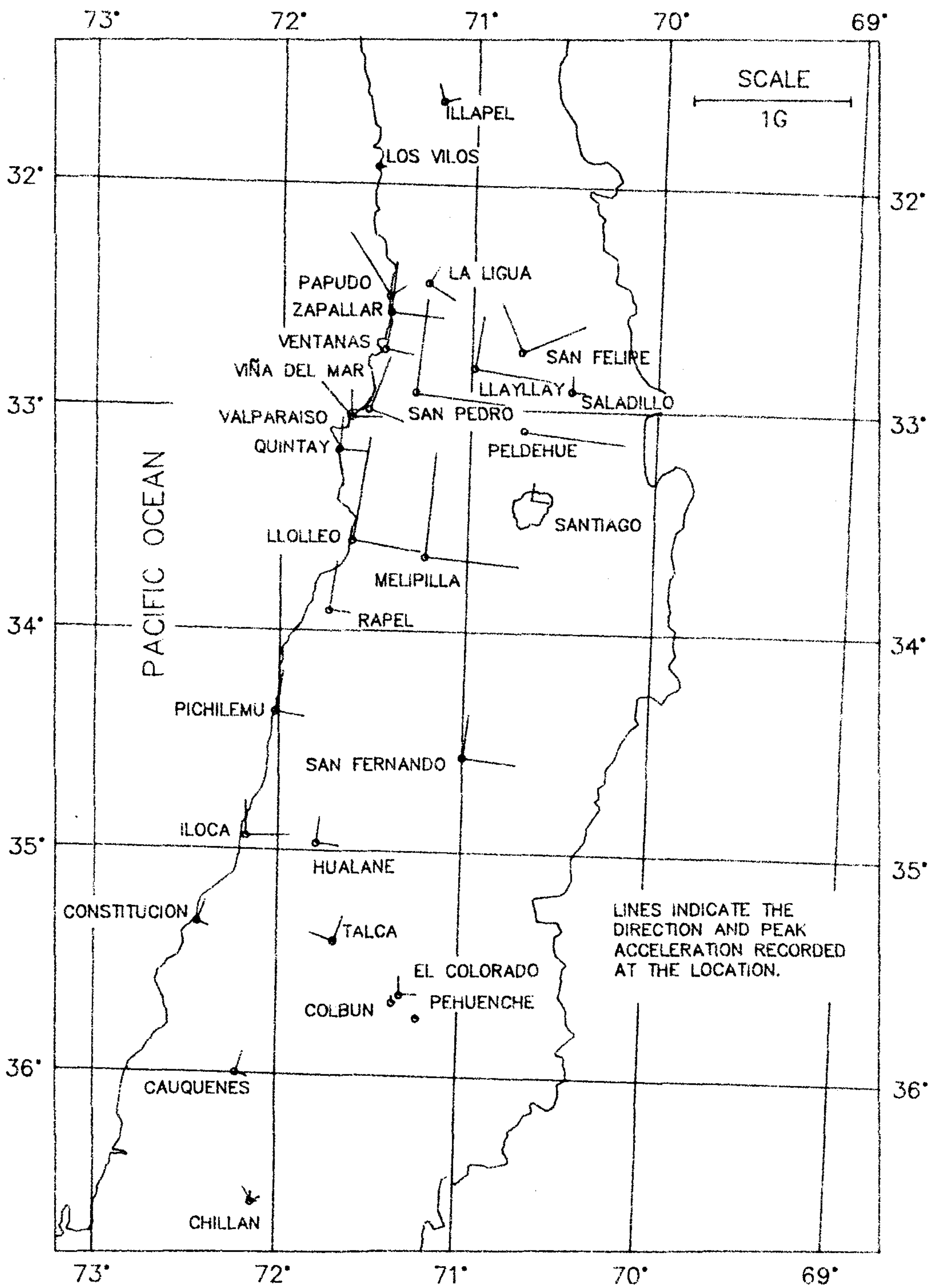


Fig. 1.2 Location of Strong-Motion Instruments during the 3 March 1985 Earthquake [61]

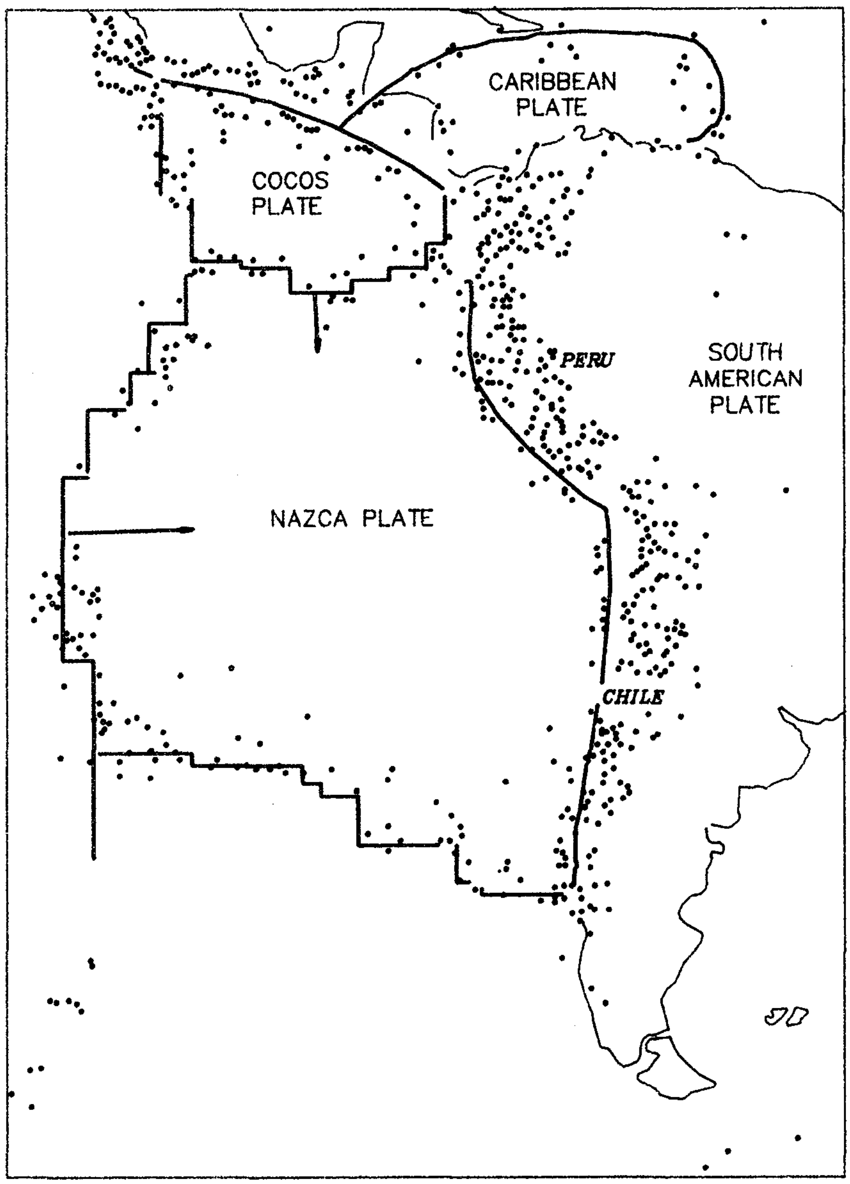


Fig. 2.1 Movement of the Crustal Plates along the West Coast of South America [60]

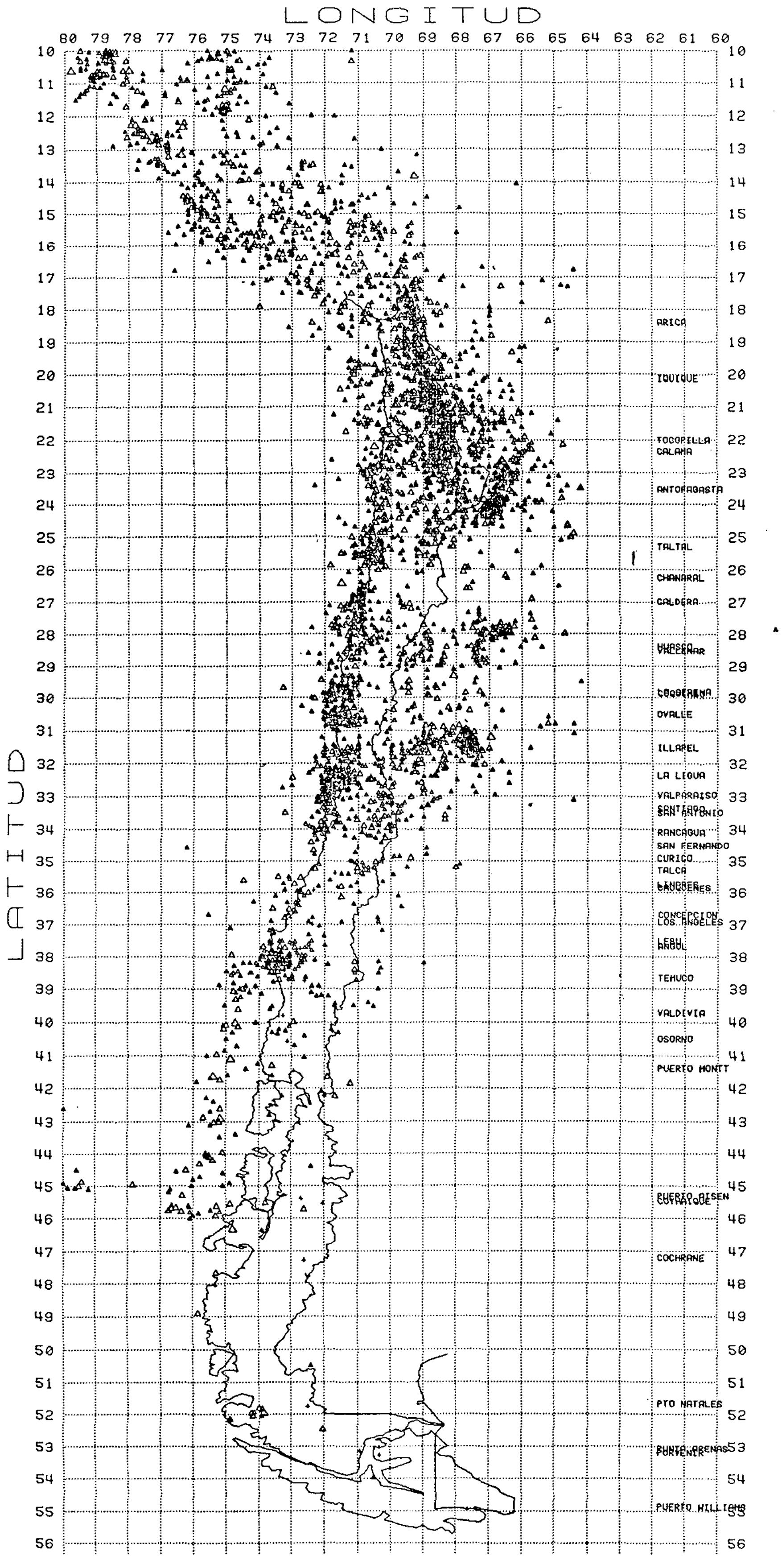


Fig. 2.2 Distribution of Epicenters in Chile between 1963 and 1978 [55]

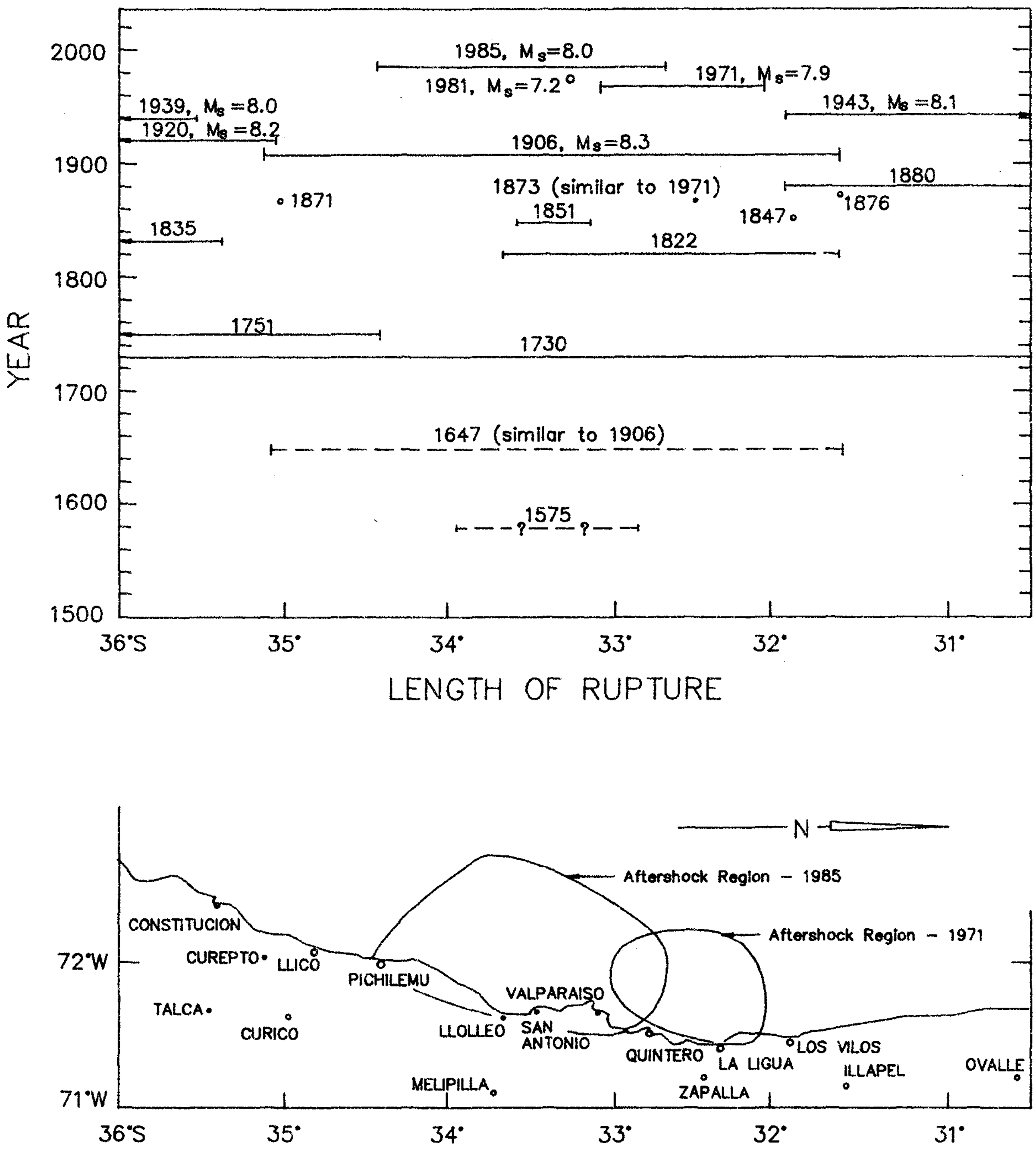


Fig. 2.3 Rupture Lengths Attributed to Great Earthquakes in Central Chile [23]

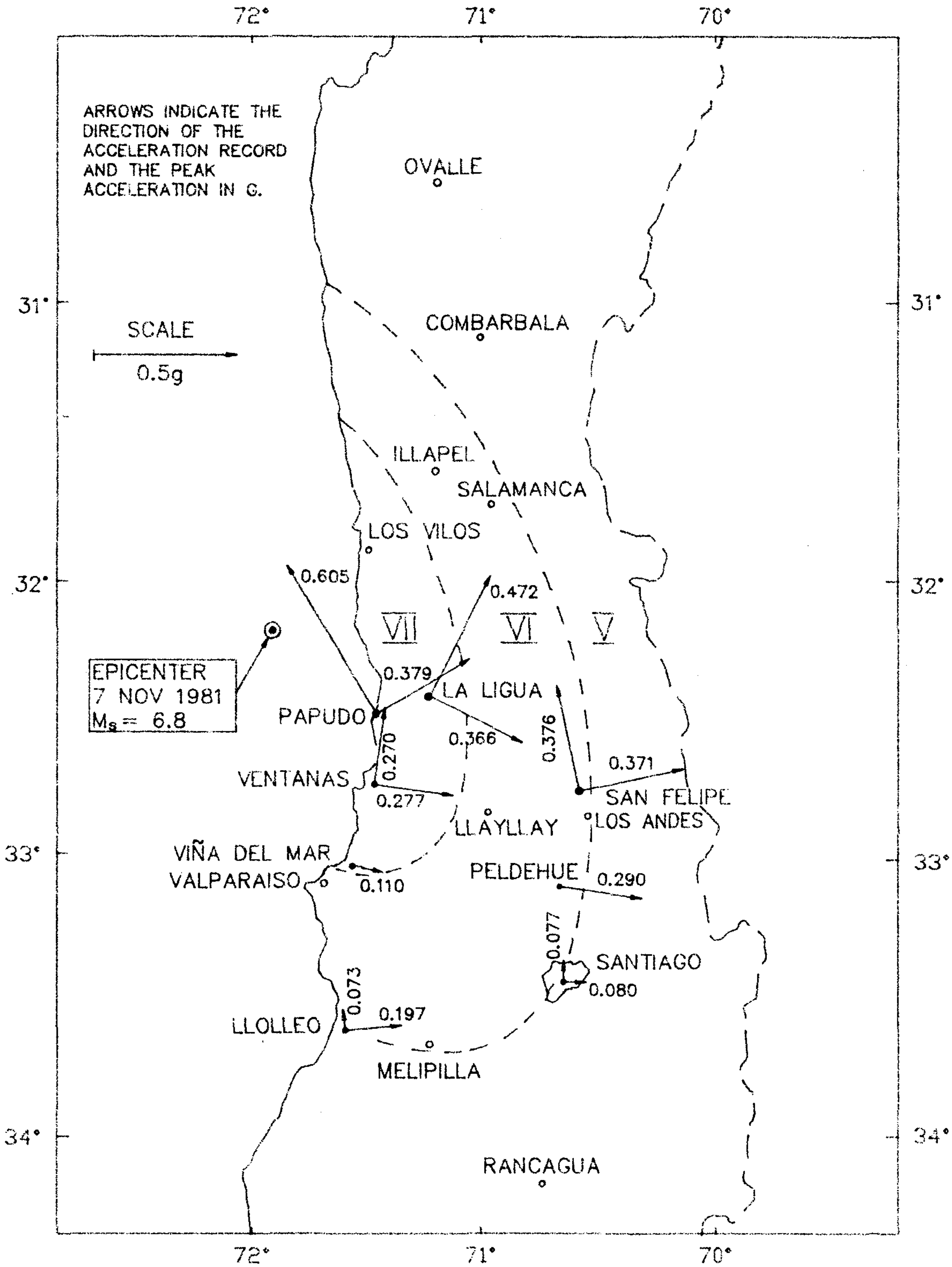


Fig. 2.4 Intensity Map from the 7 November 1981 Earthquake [27]

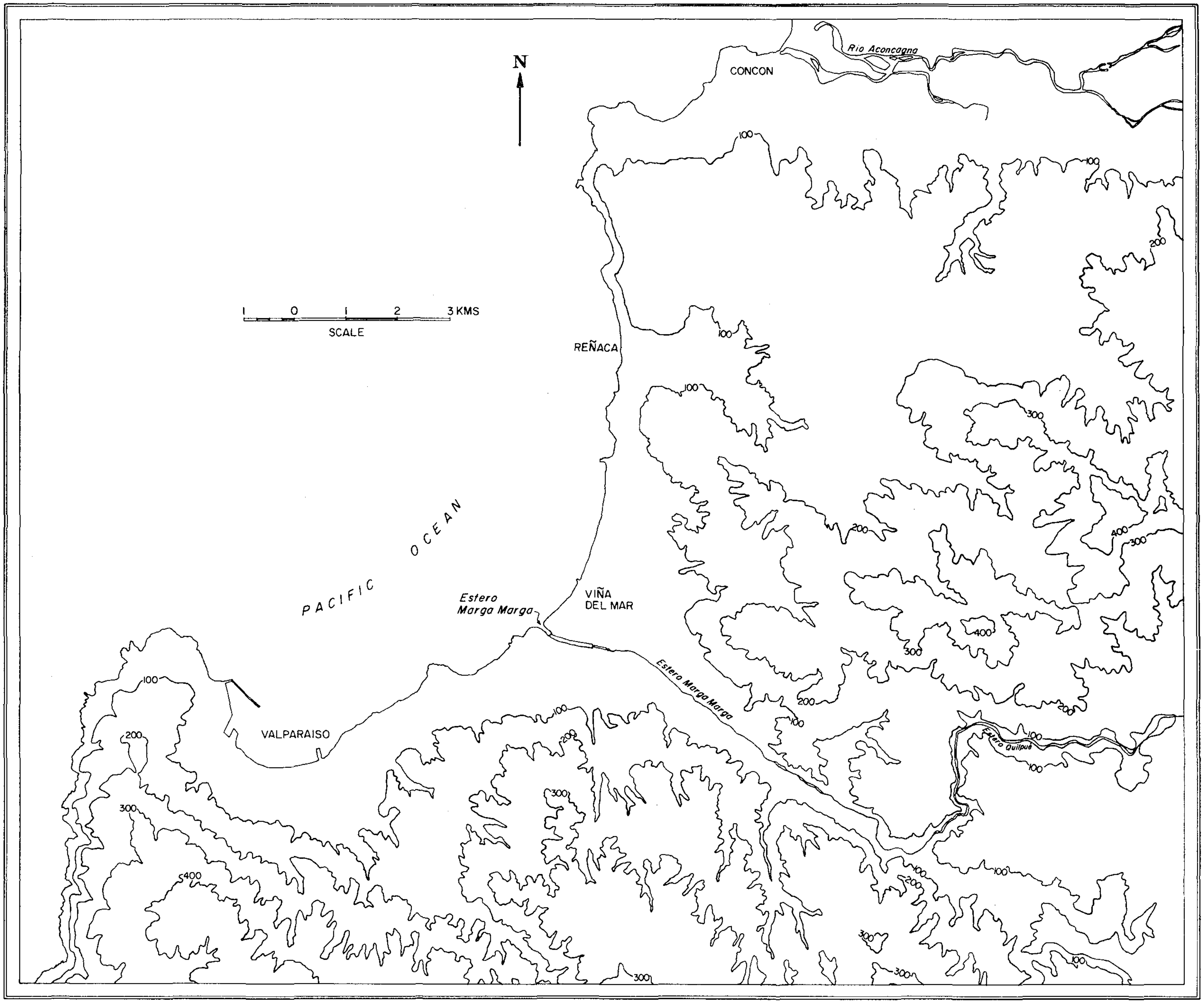


Fig. 2.5 Topological Map of Viña del Mar and Valparaíso [30]

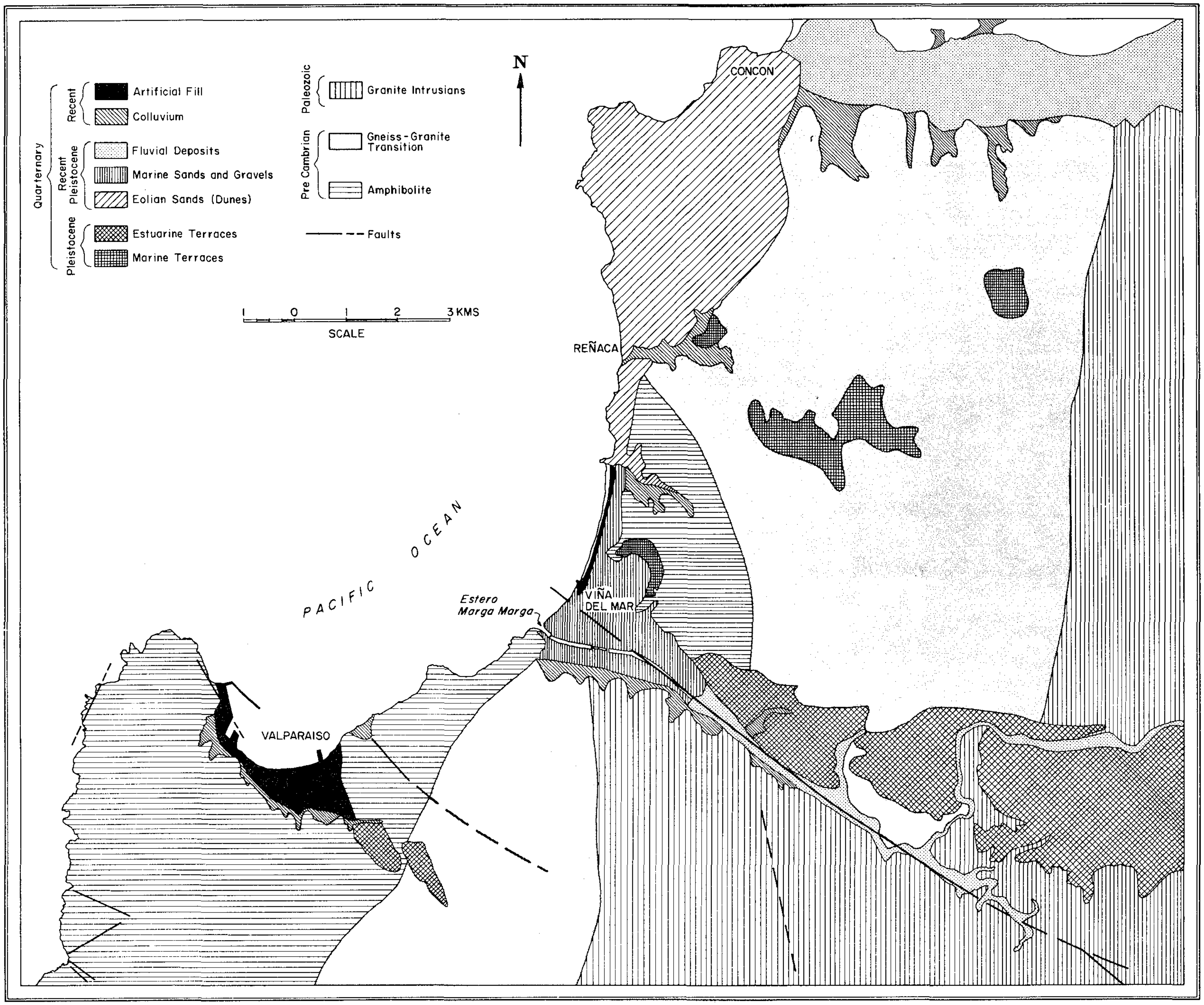


Fig. 2.6 Geology of Viña del Mar and Valparaíso [30]

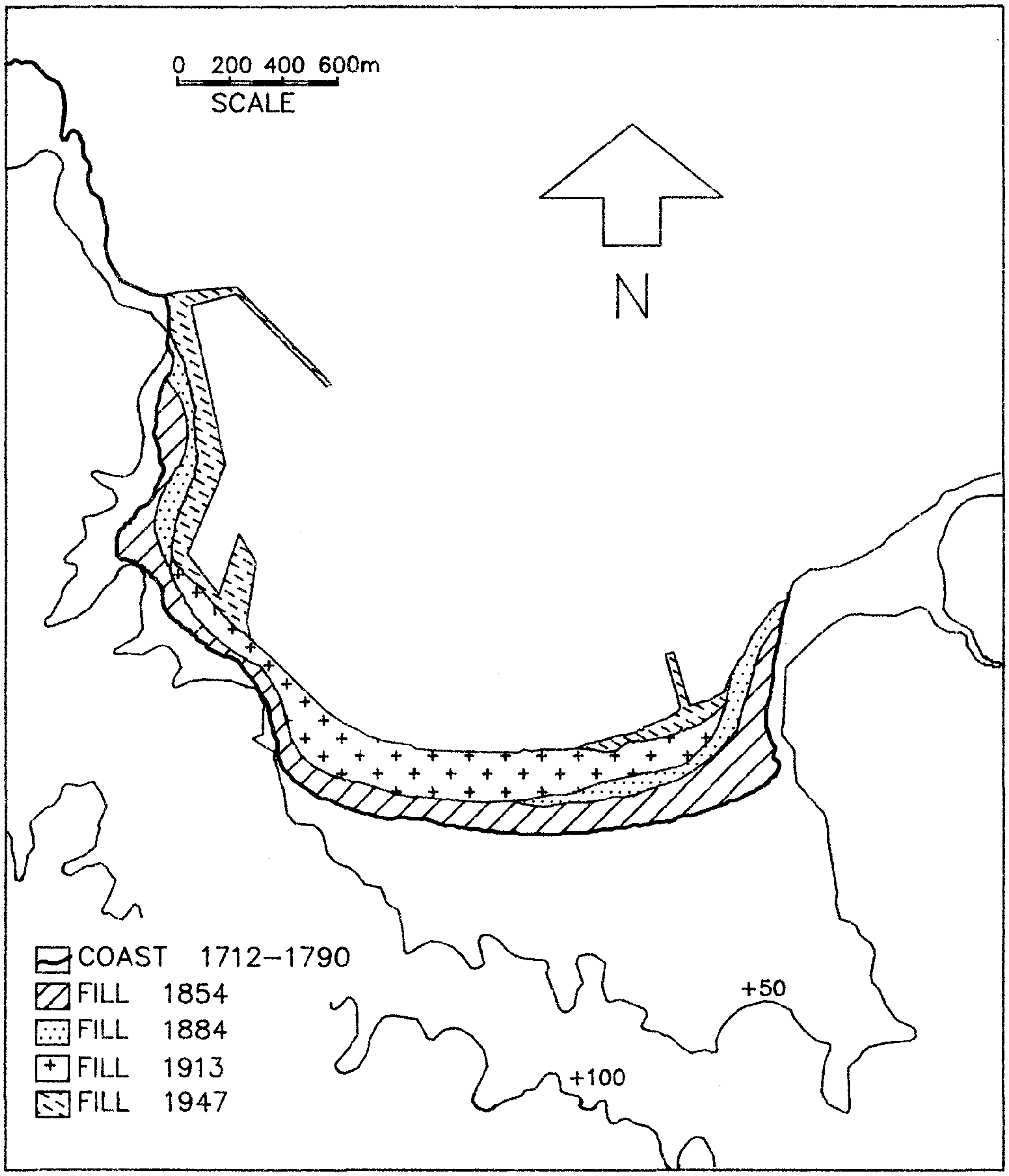


Fig. 2.7 Development of Valparaíso Harbor [2]

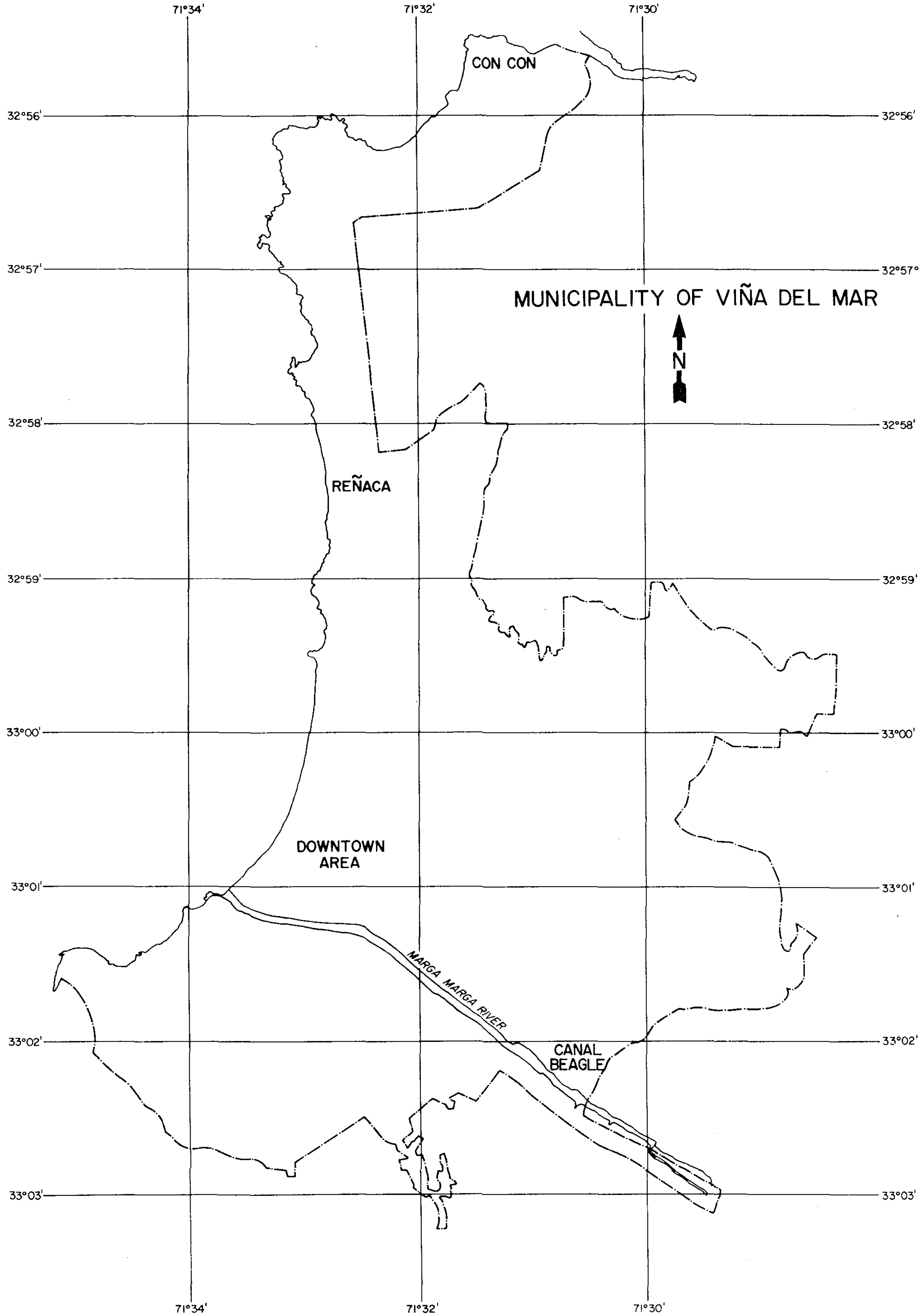


Fig. 2.8 Viña del Mar

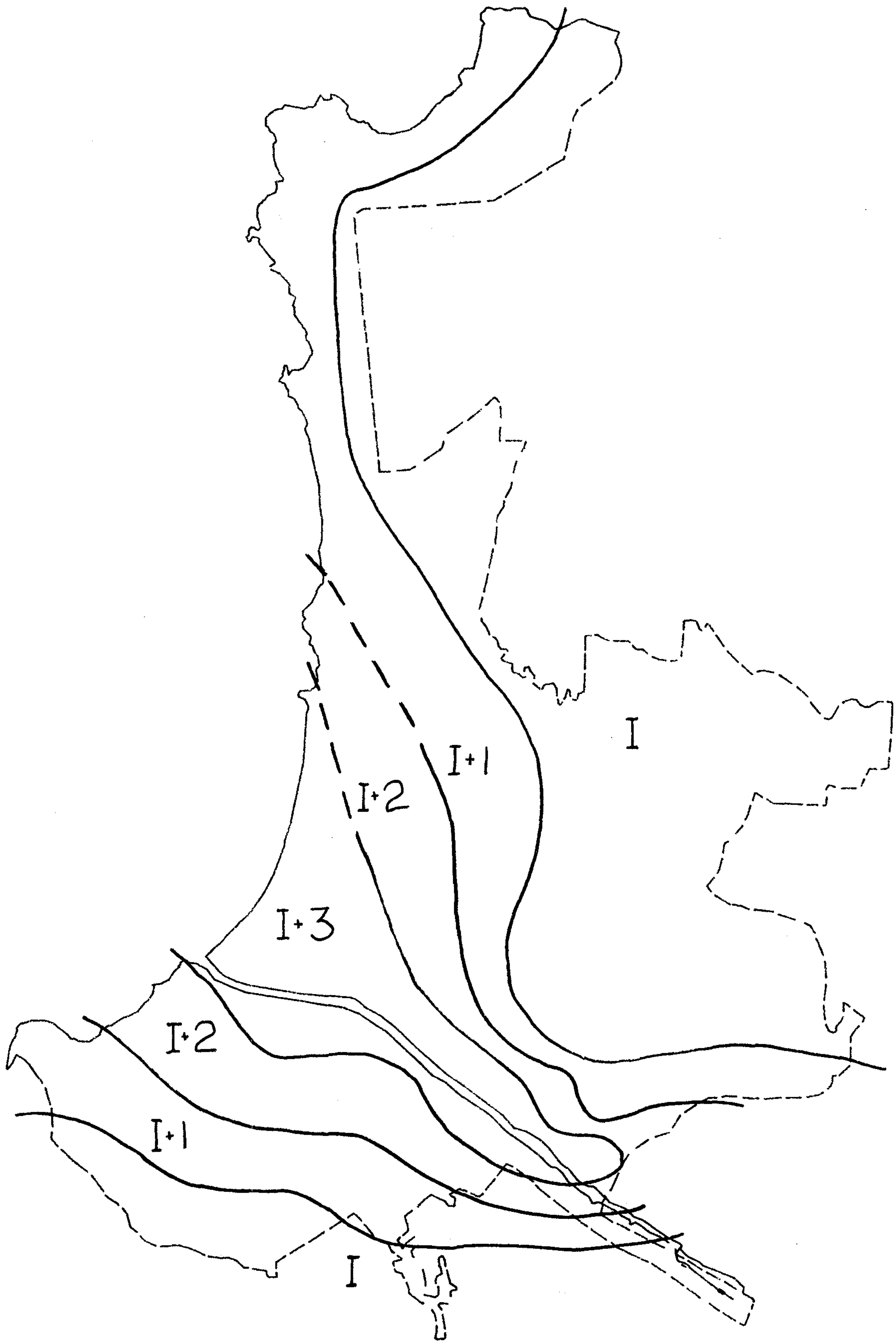


Fig. 2.9 Expected Distribution of Modified Mercalli Intensities within Viña del Mar [3]

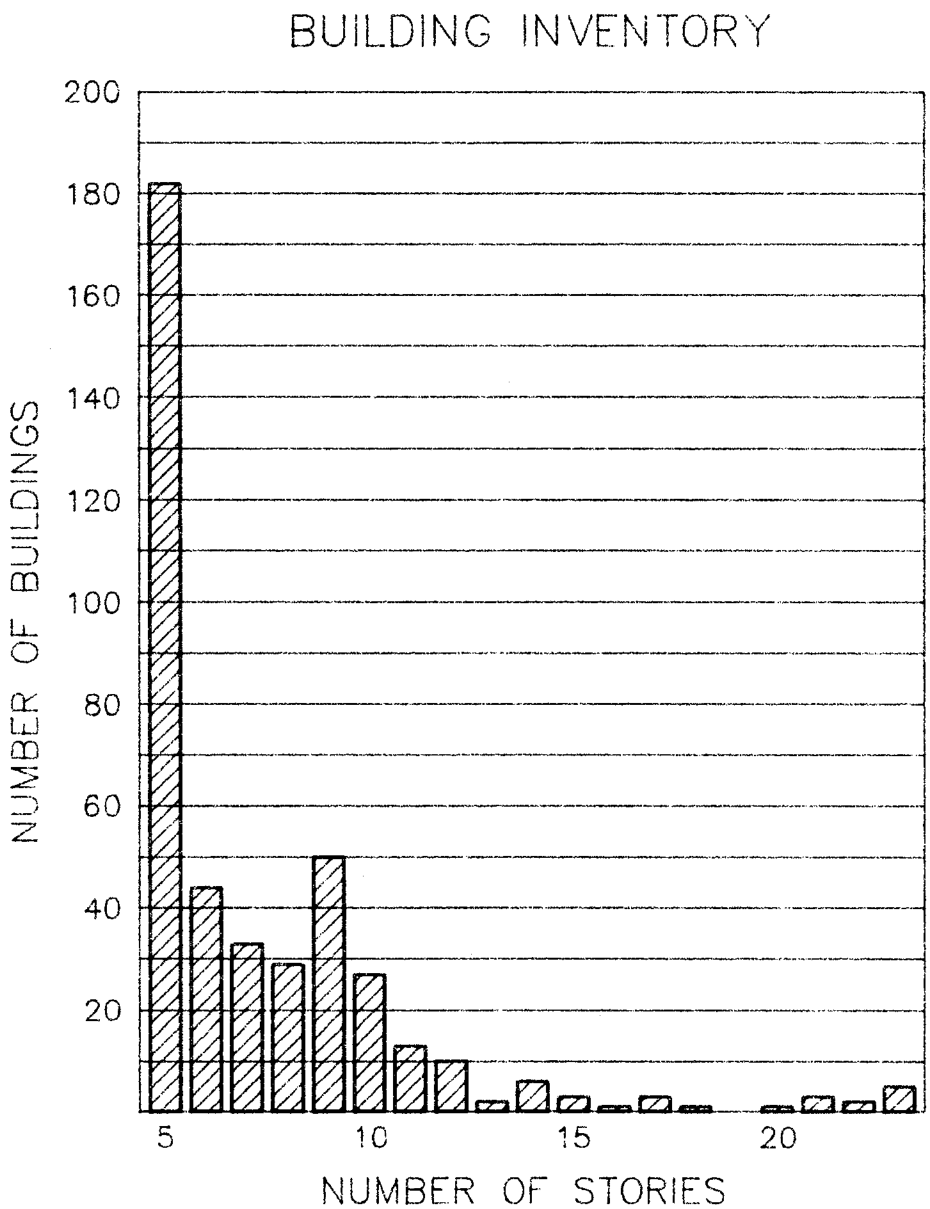


Fig. 2.10 Distribution of Buildings in Viña del Mar with Respect to Height

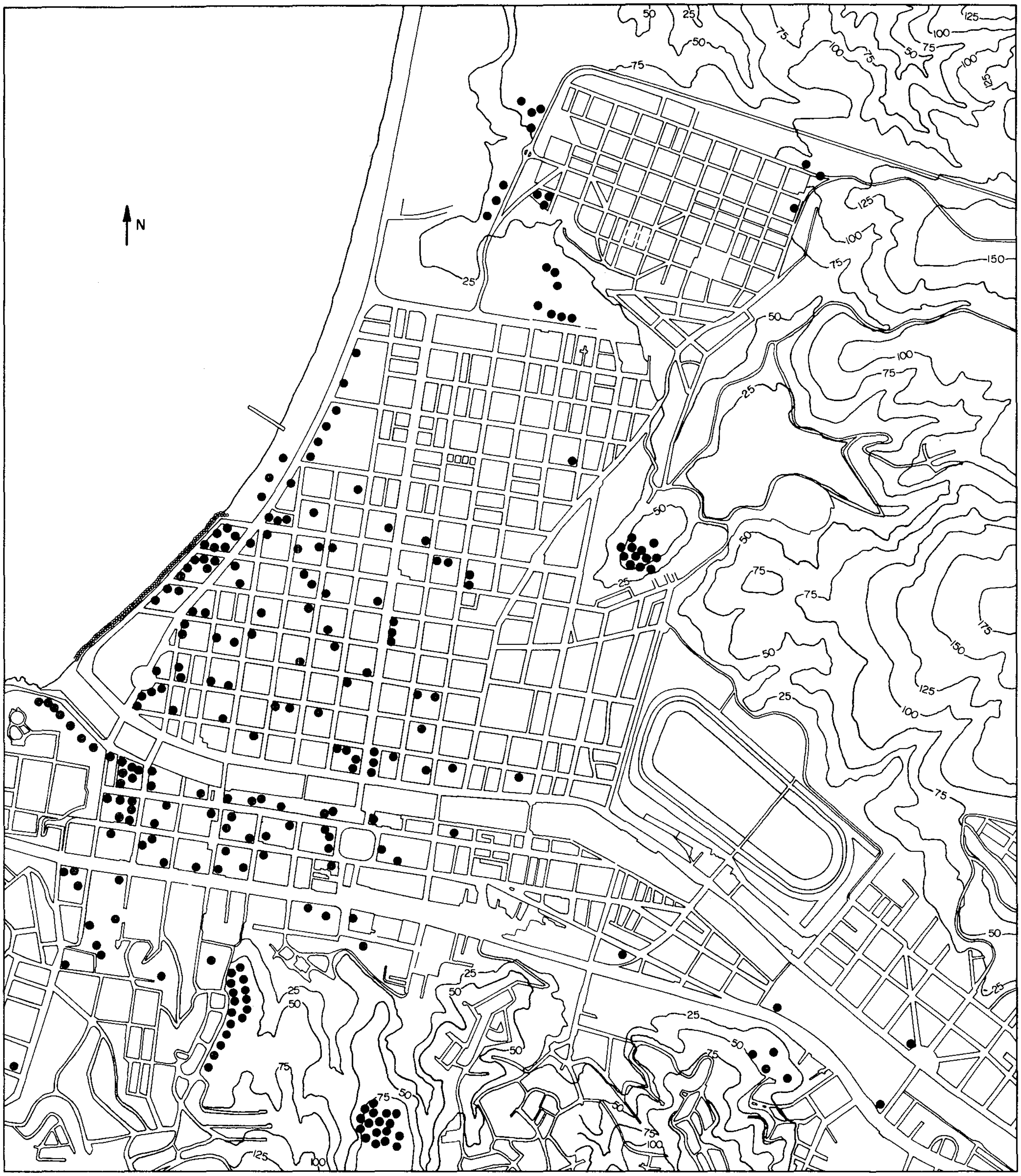


Fig. 2.11 Locations of Buildings with Five or More Stories in Downtown Viña del Mar

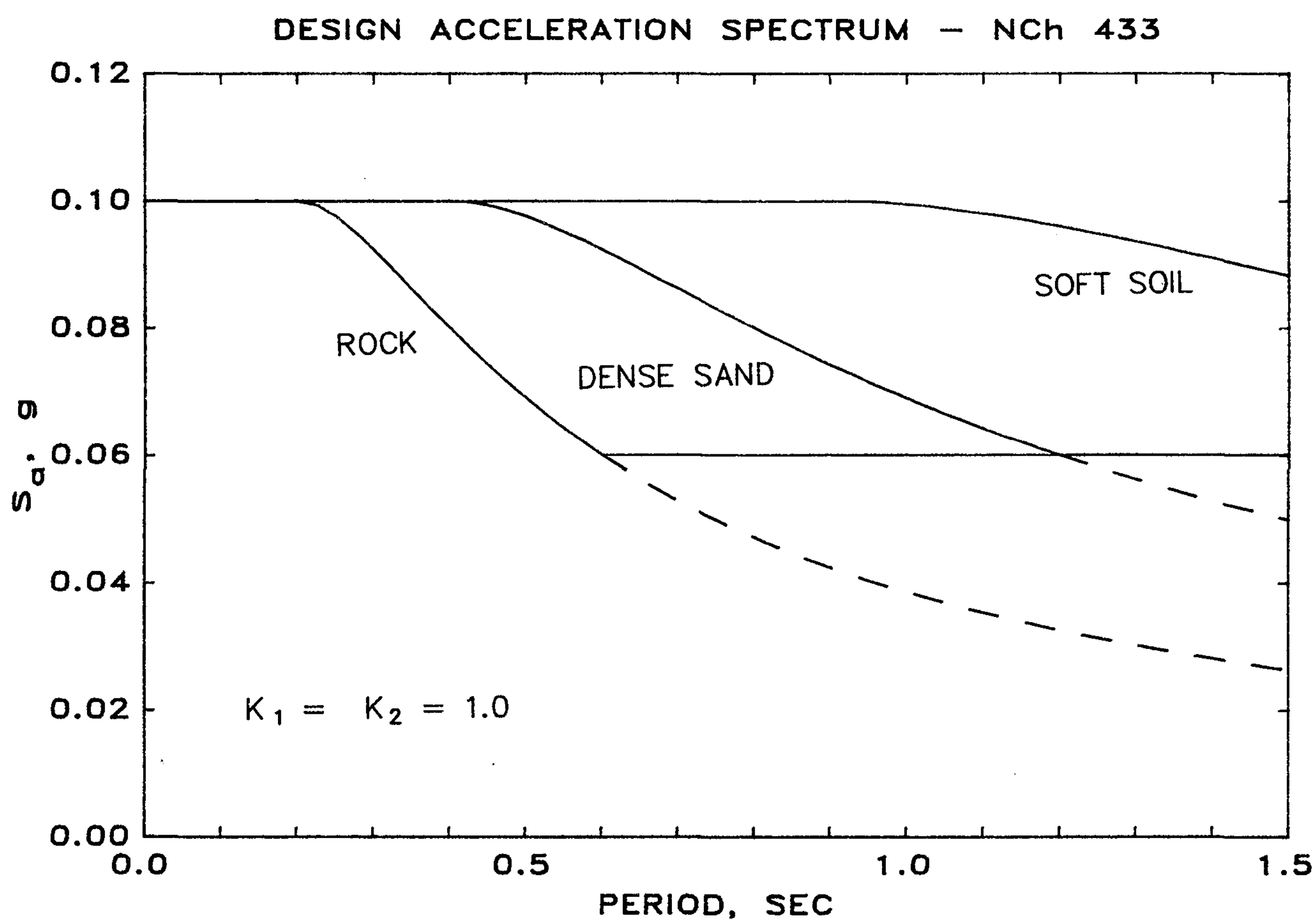
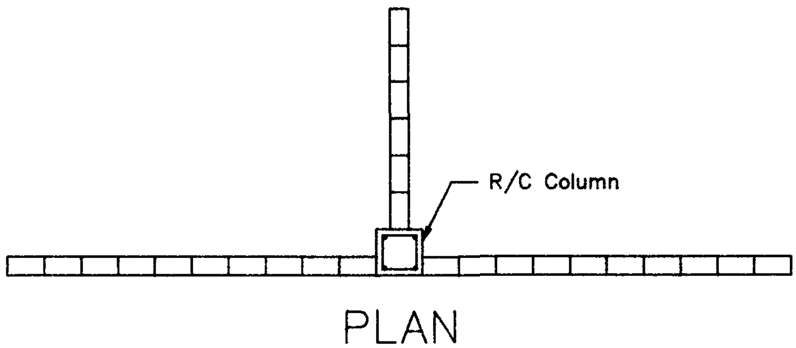


Fig. 3.1 Design Acceleration Spectrum [16]



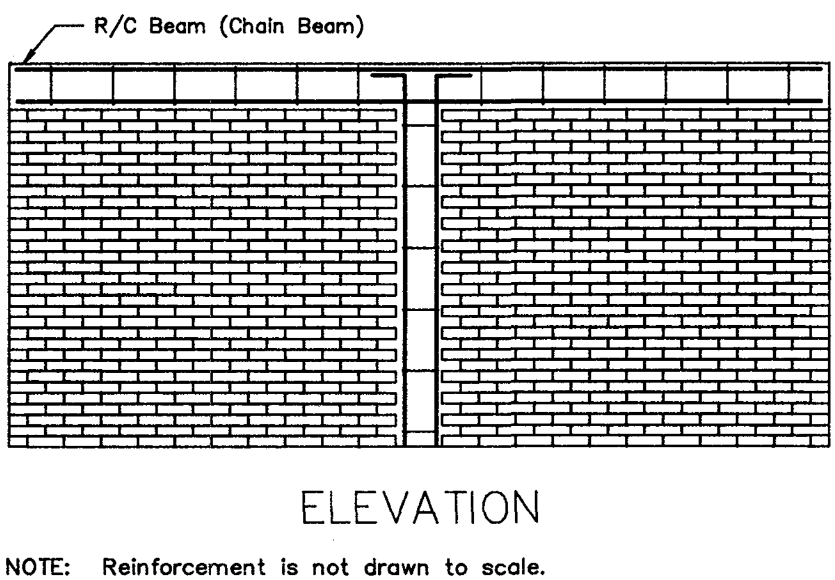


Fig. 4.1 Confined Masonry Construction

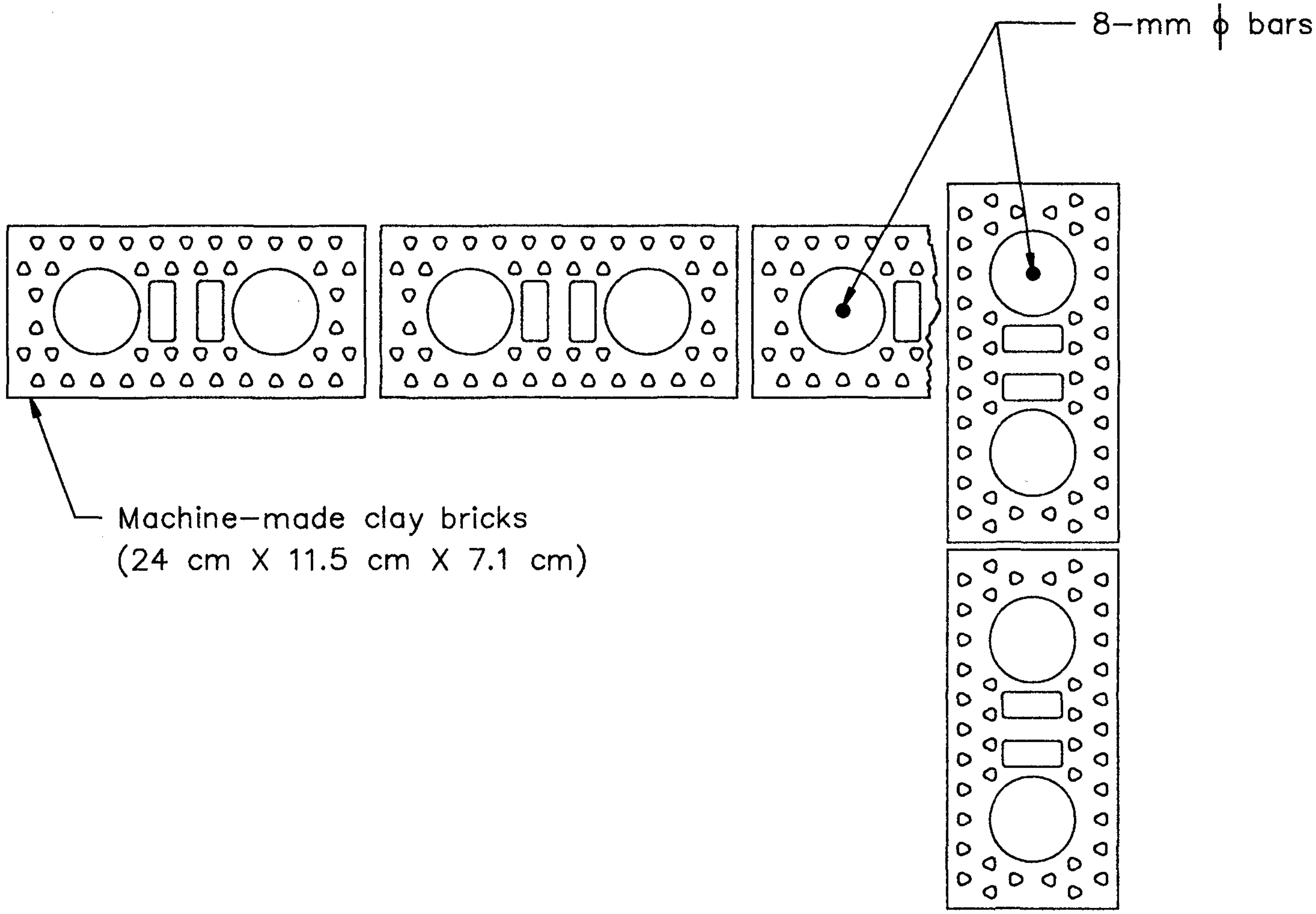


Fig. 4.2 Reinforced Masonry Construction

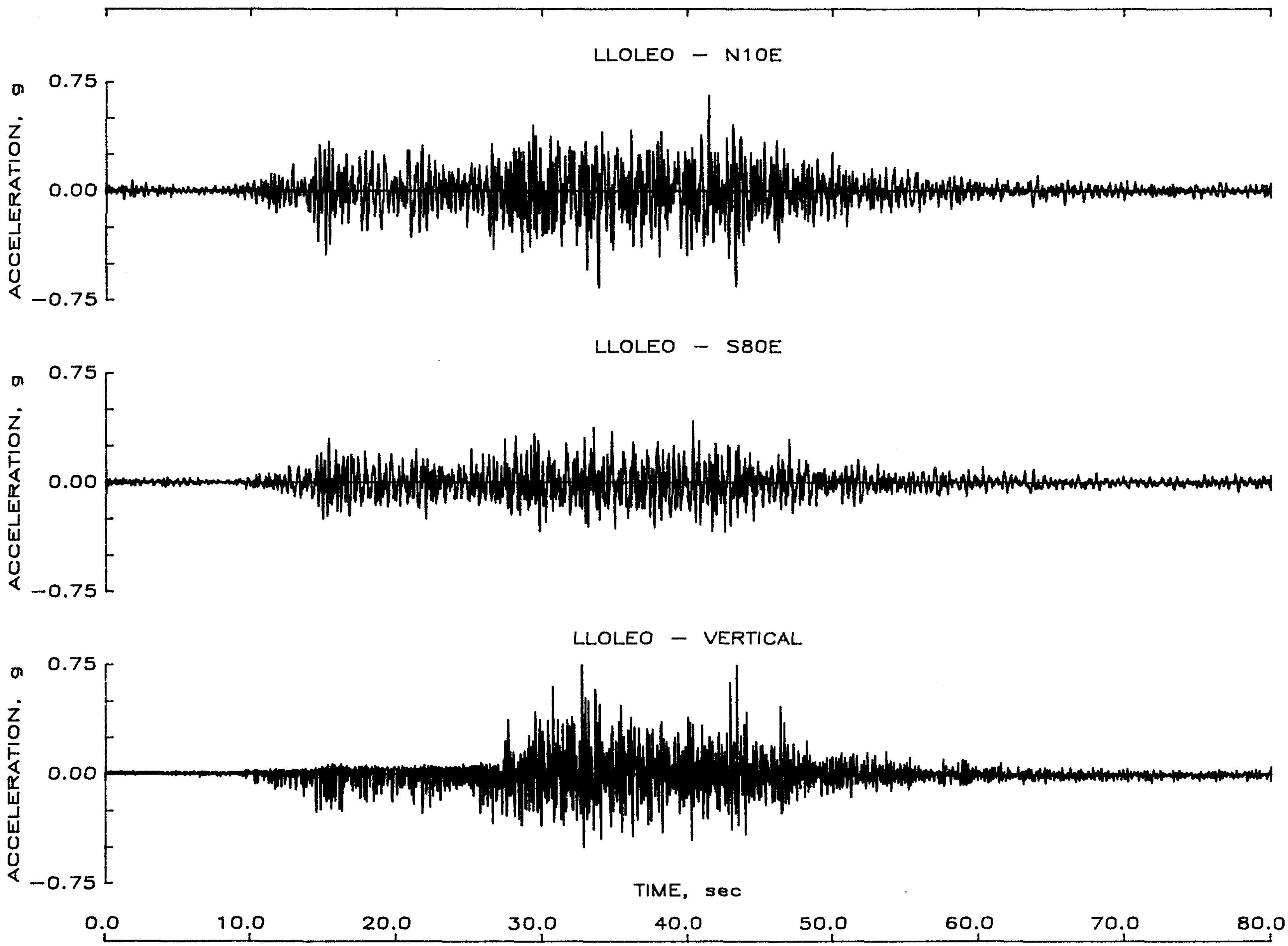


Fig. 5.1 Strong-Motion Records Obtained during the 3 March 1985 Earthquake

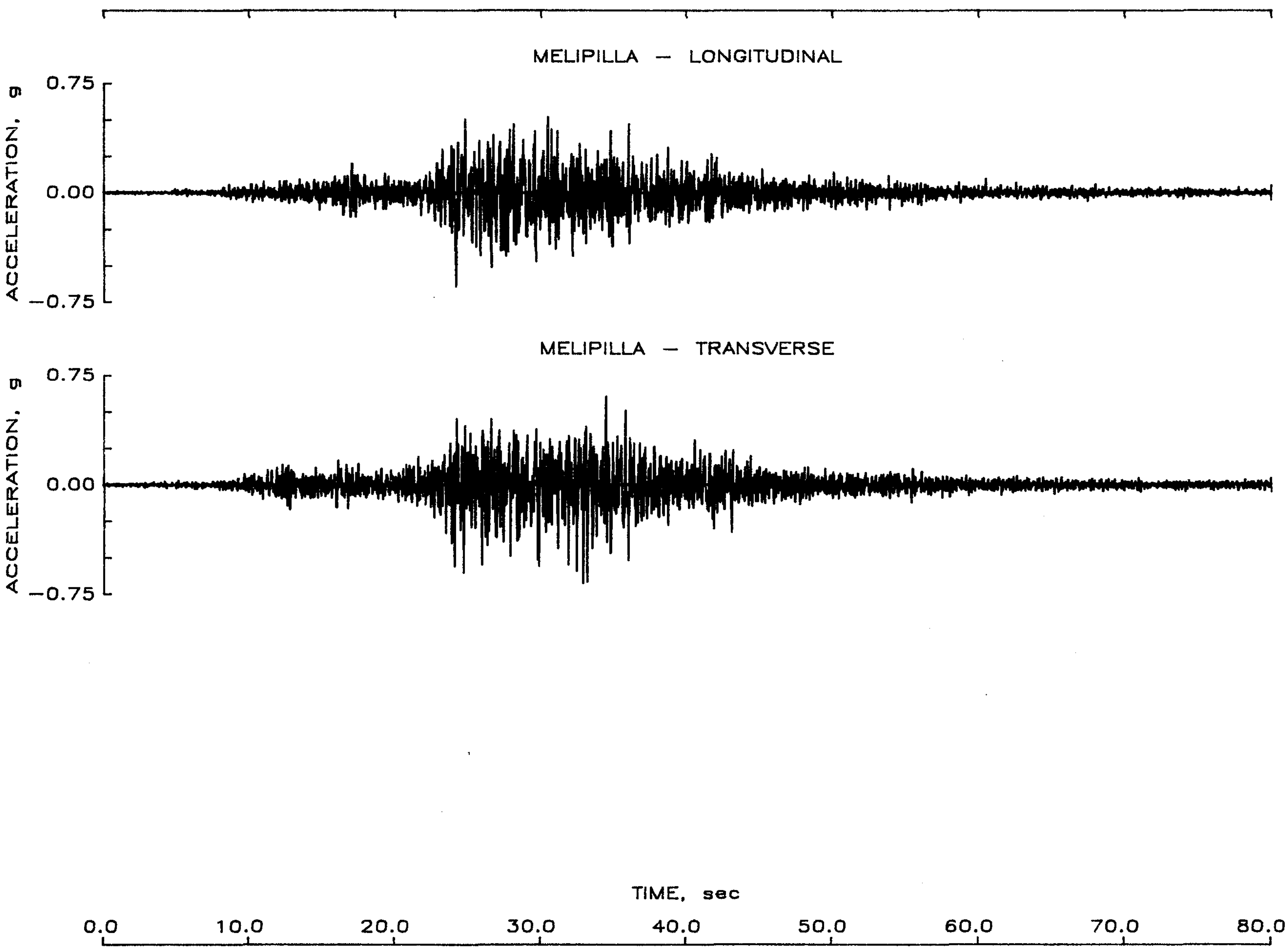
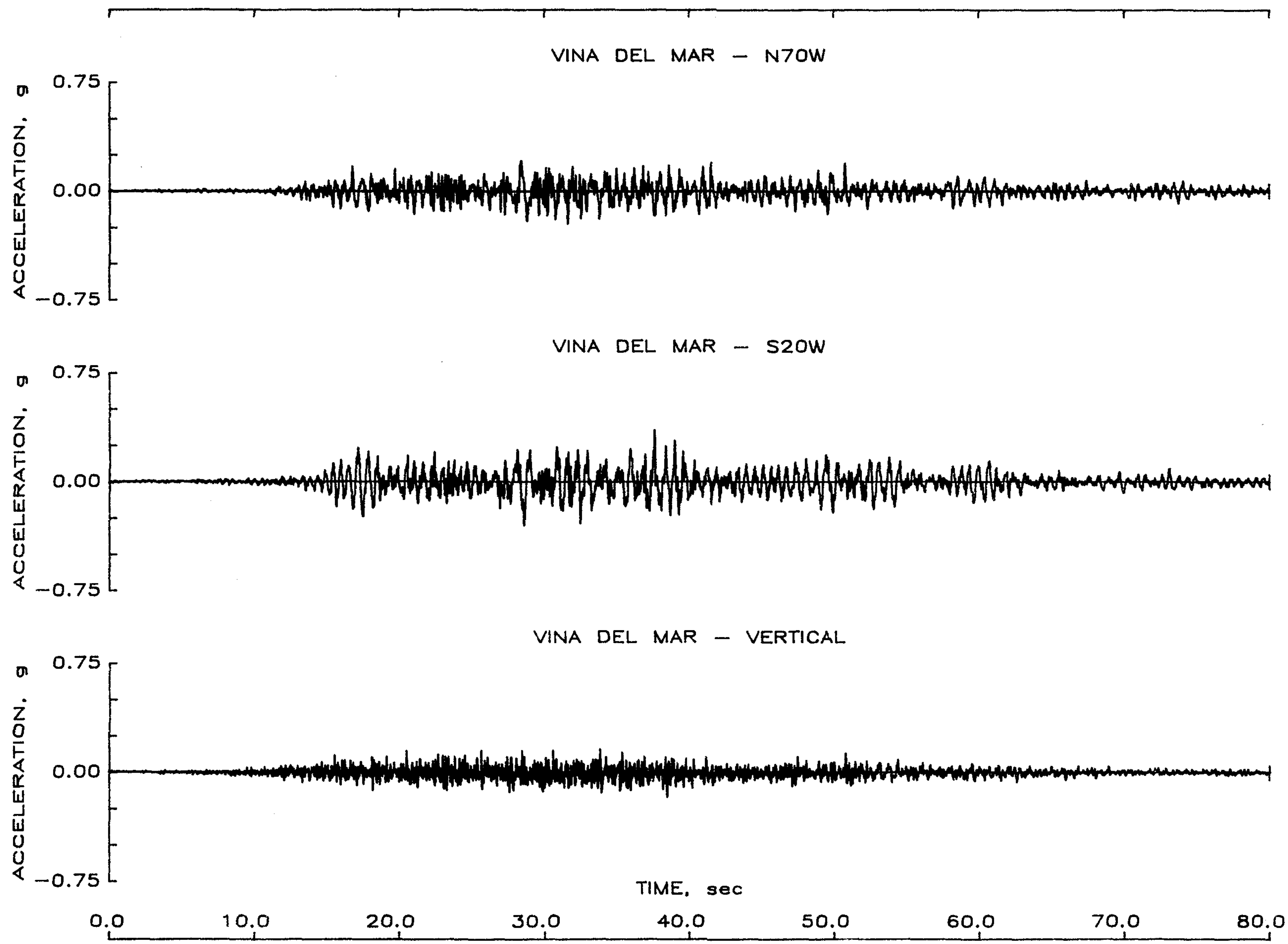


Fig. 5.1 (cont.) Strong-Motion Records Obtained during the 3 March 1985 Earthquake



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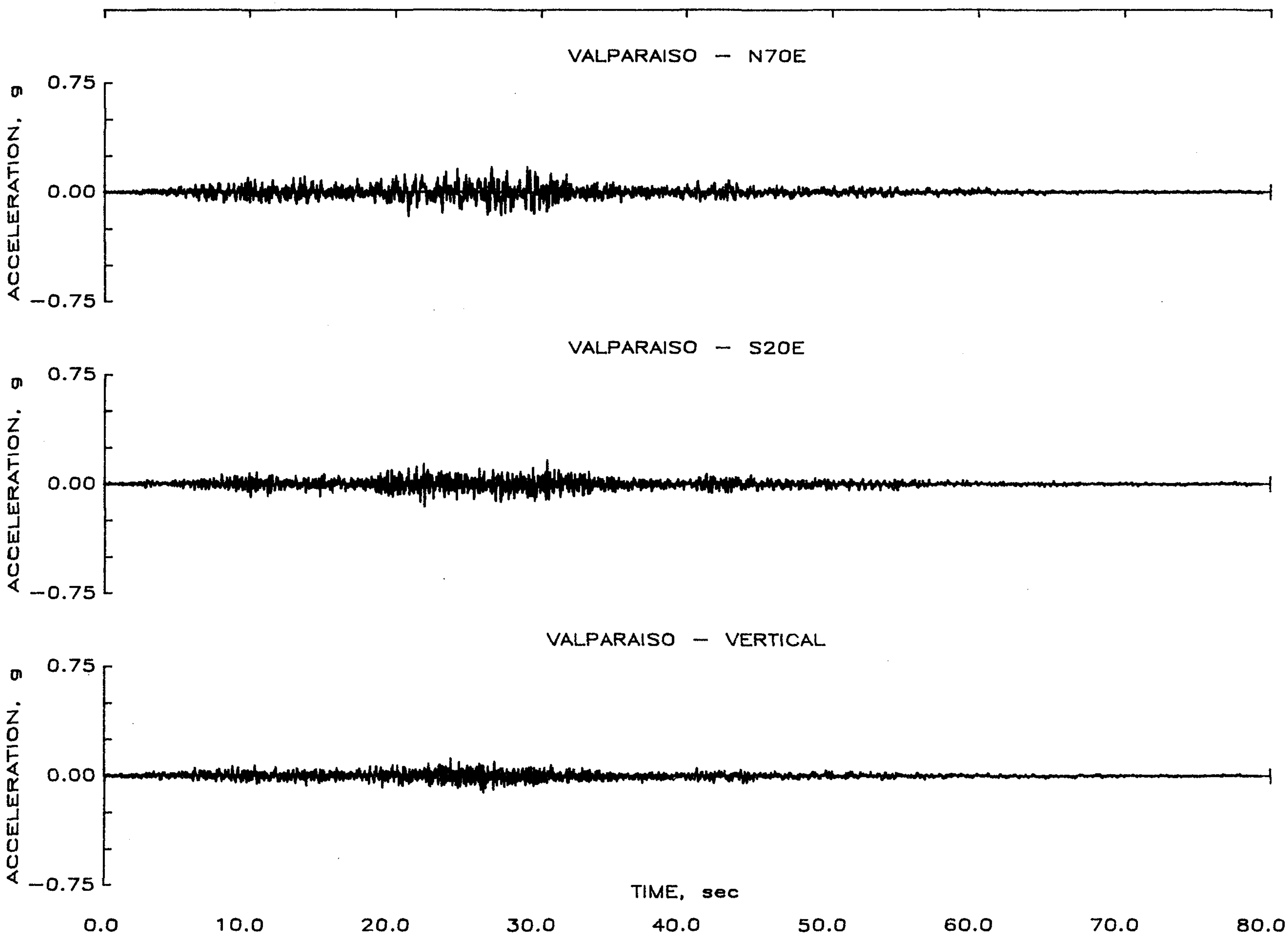

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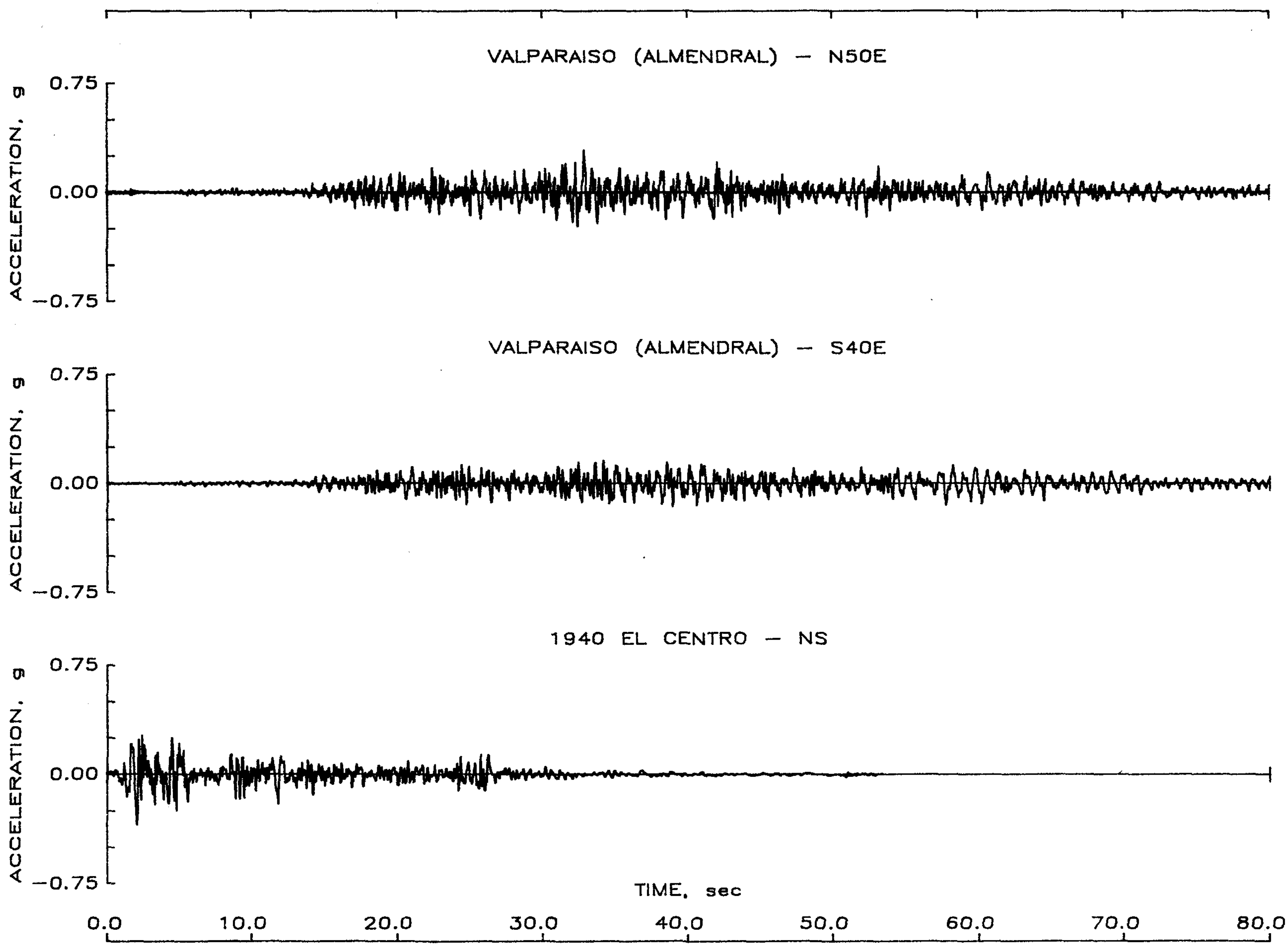

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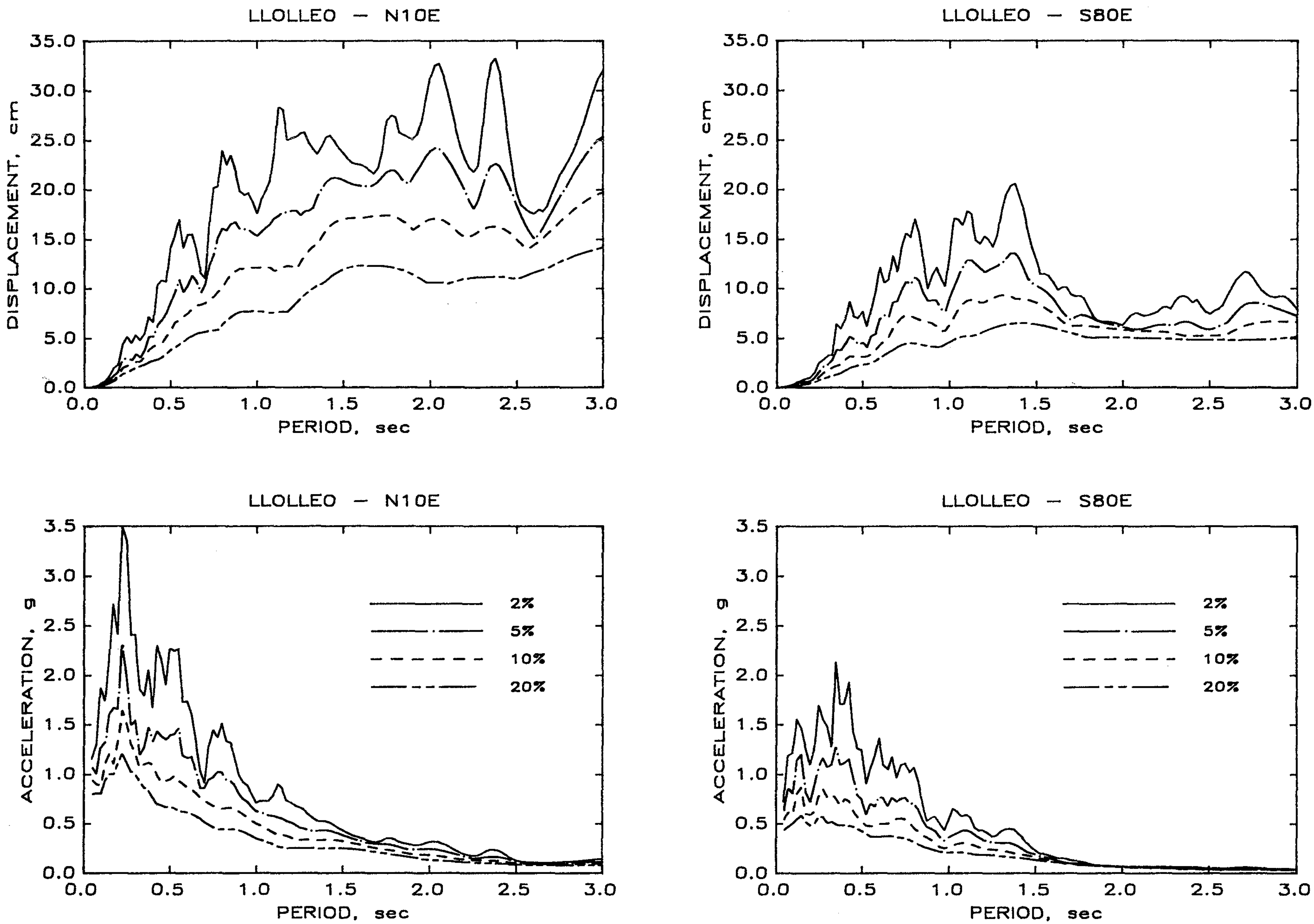
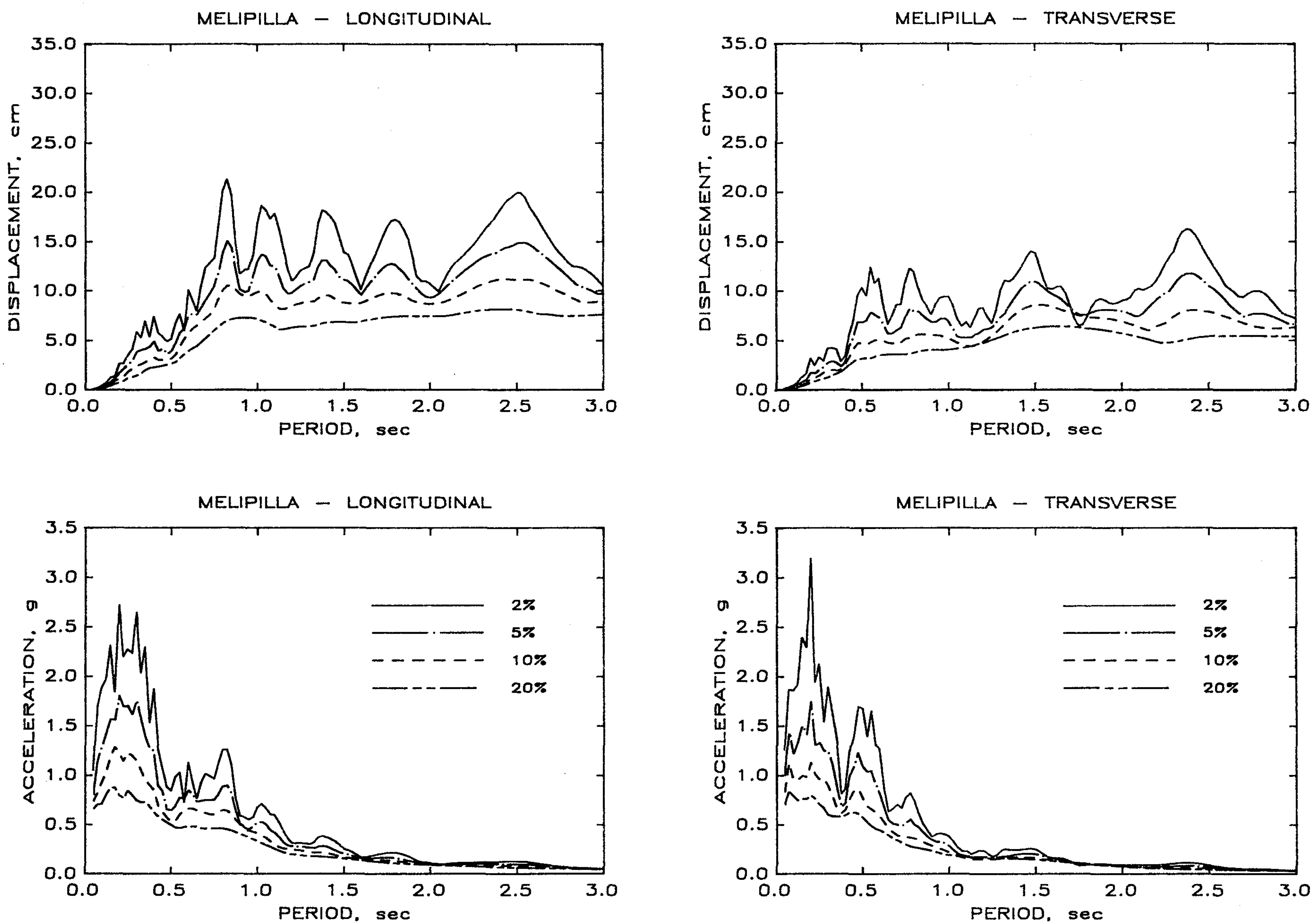


Fig. 5.2 Elastic Response Spectra



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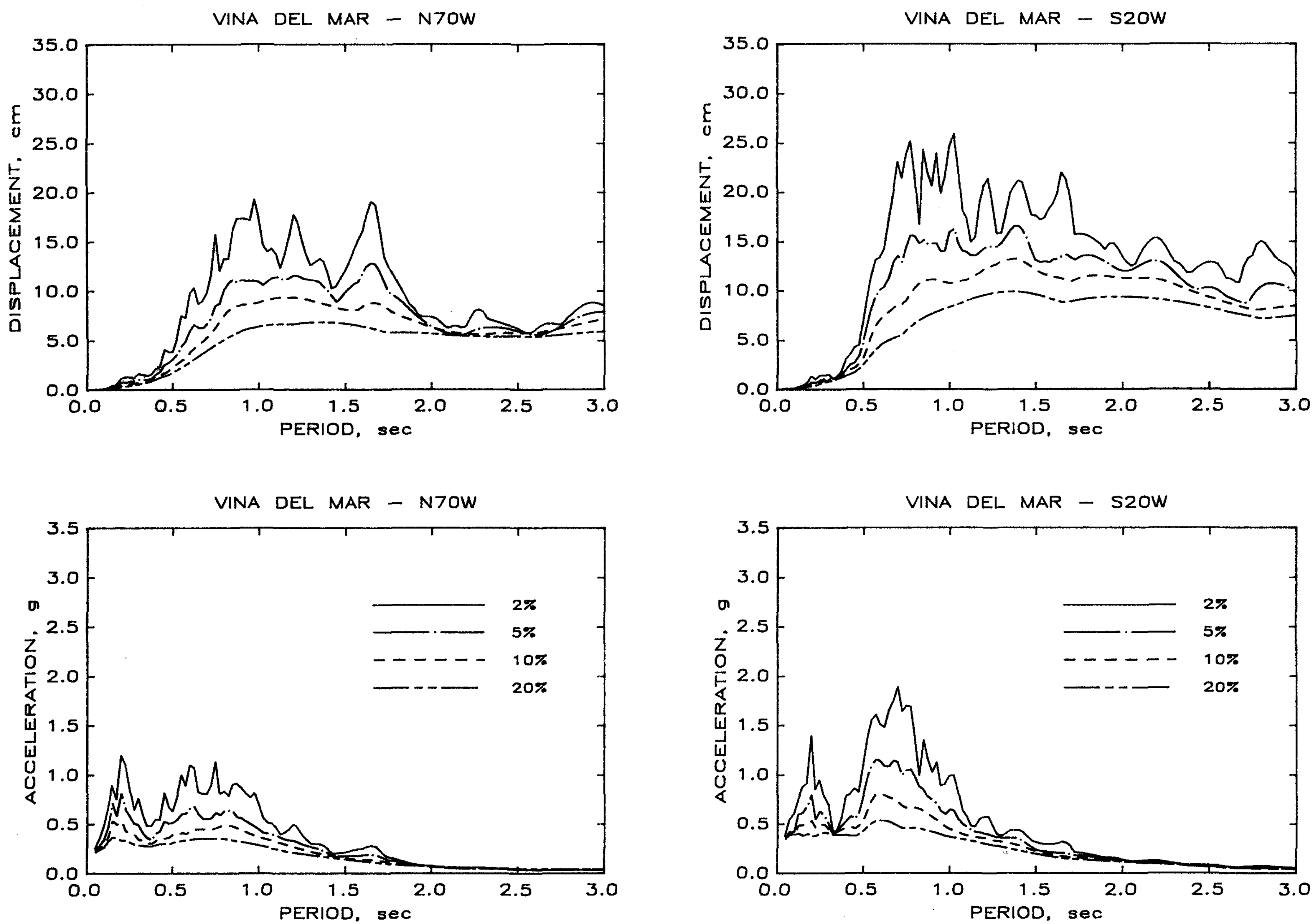

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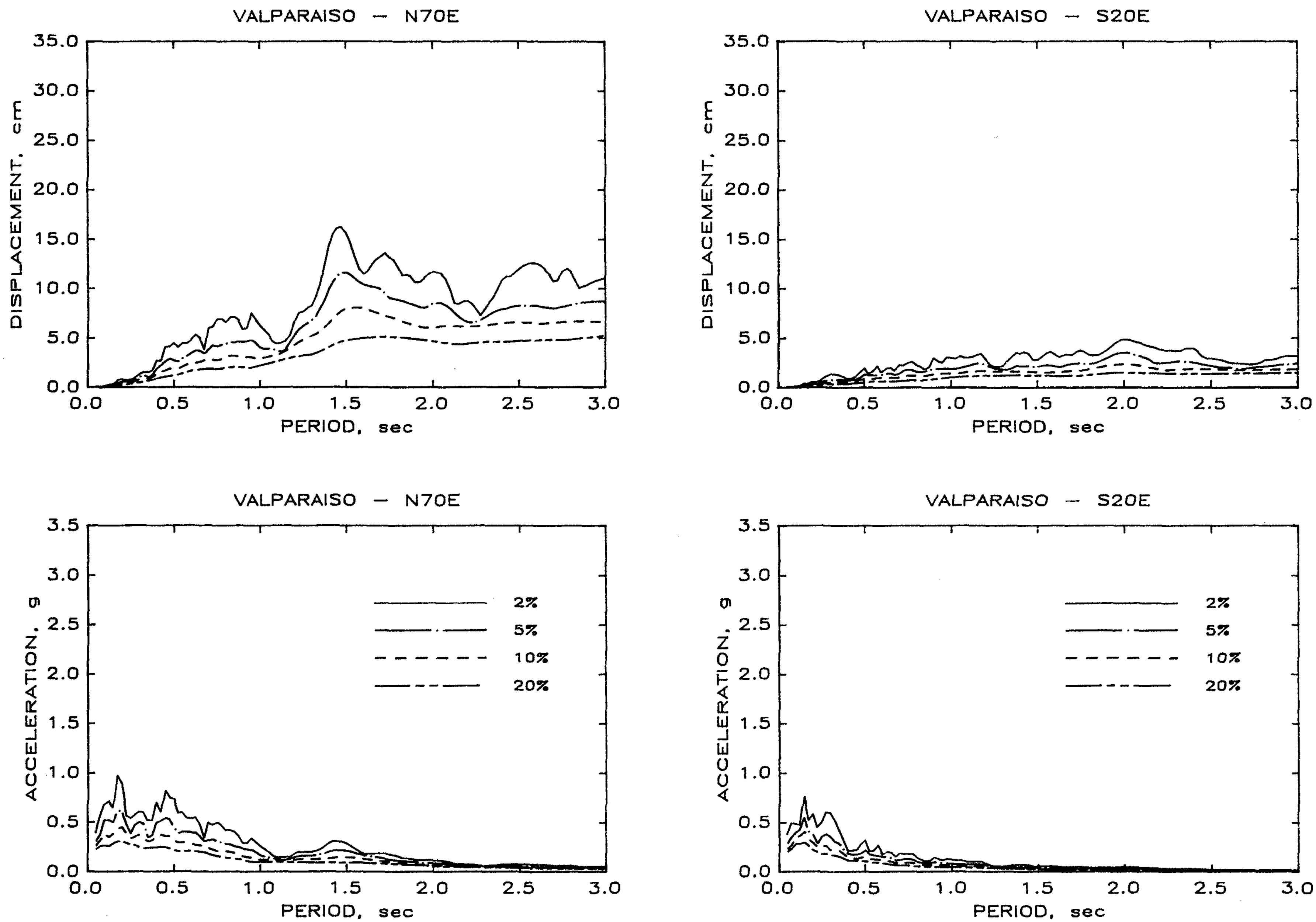

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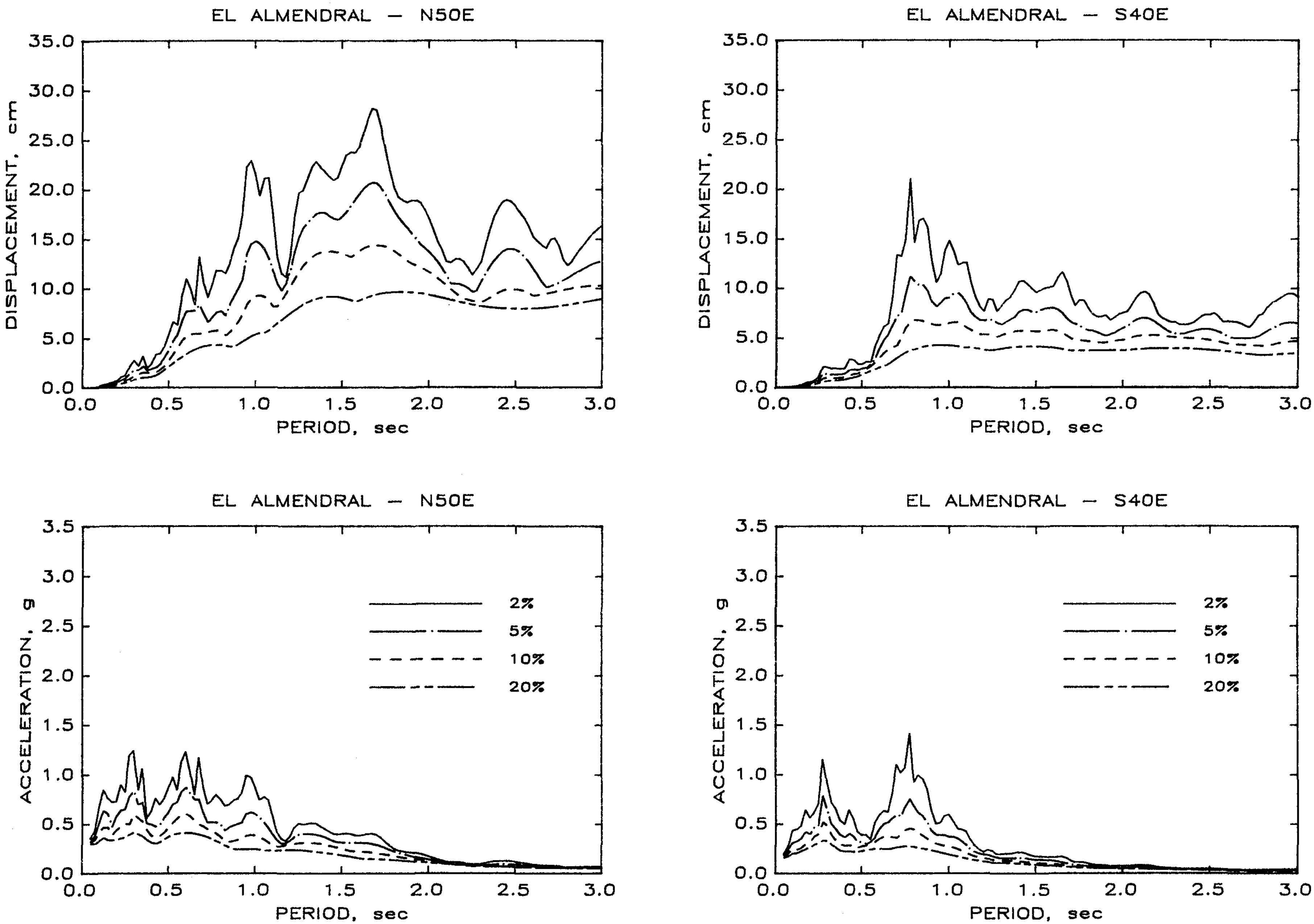

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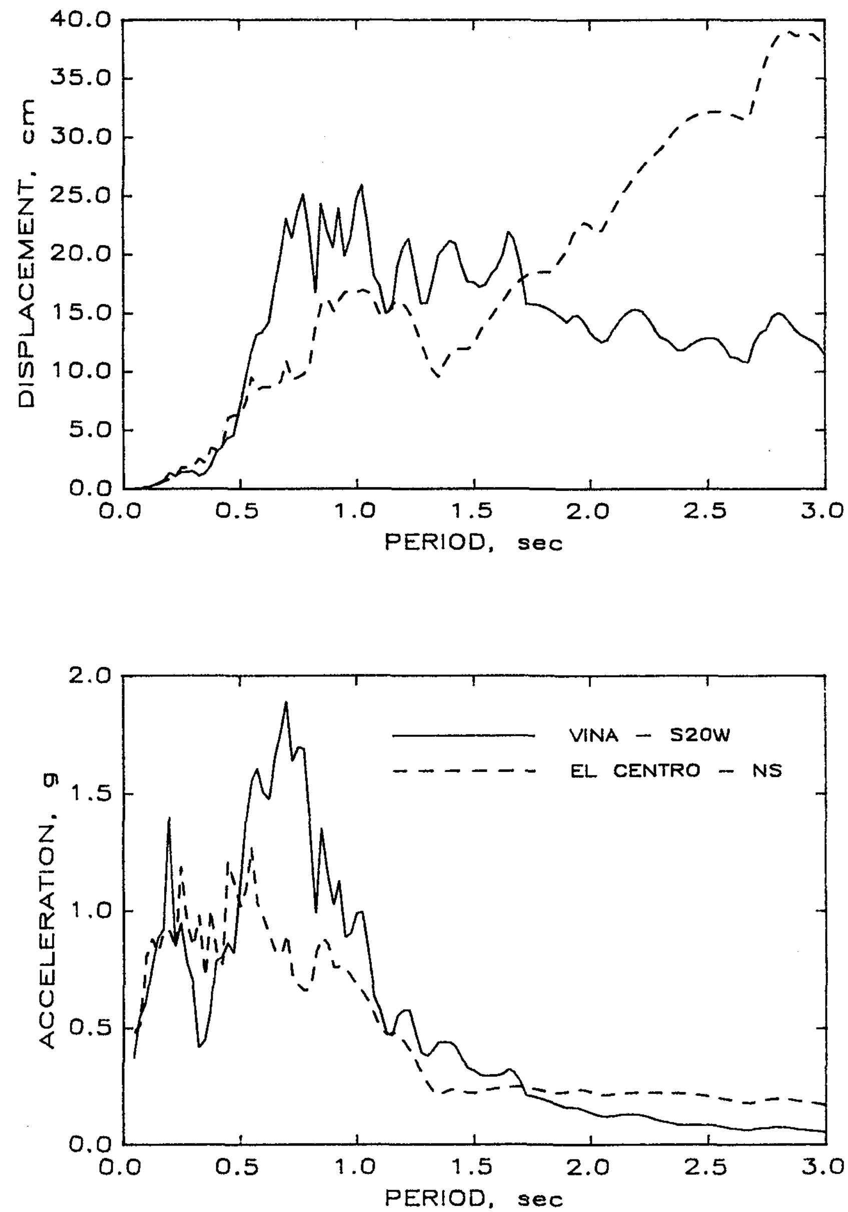


Fig. 5.3 Comparison of Elastic Response Spectra from the 1940 El Centro and 1985 Viña del Mar Records (Damping Factor = 0.02)

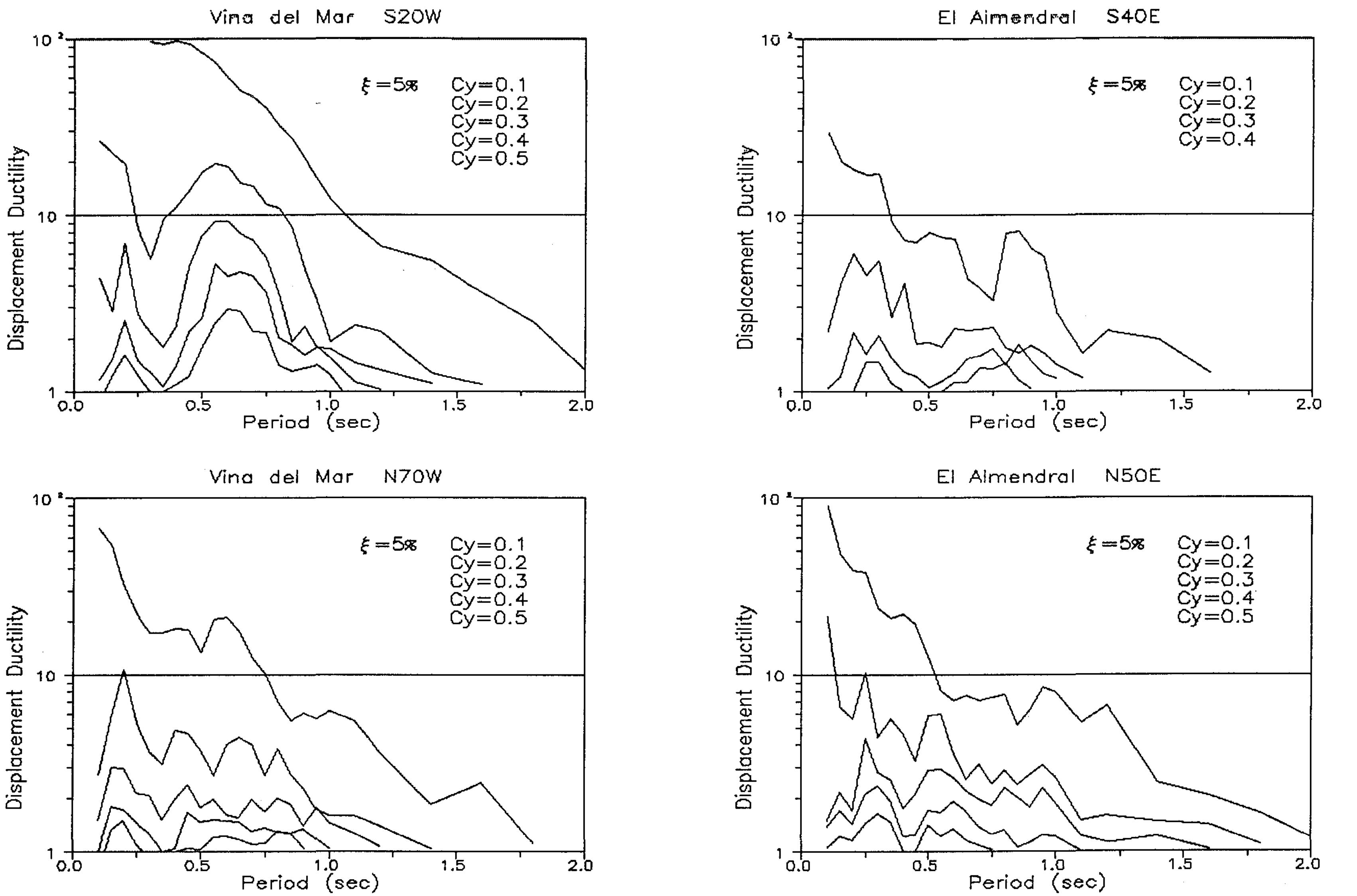


Fig. 5.4 Elasto-Perfectly Plastic Response Spectra

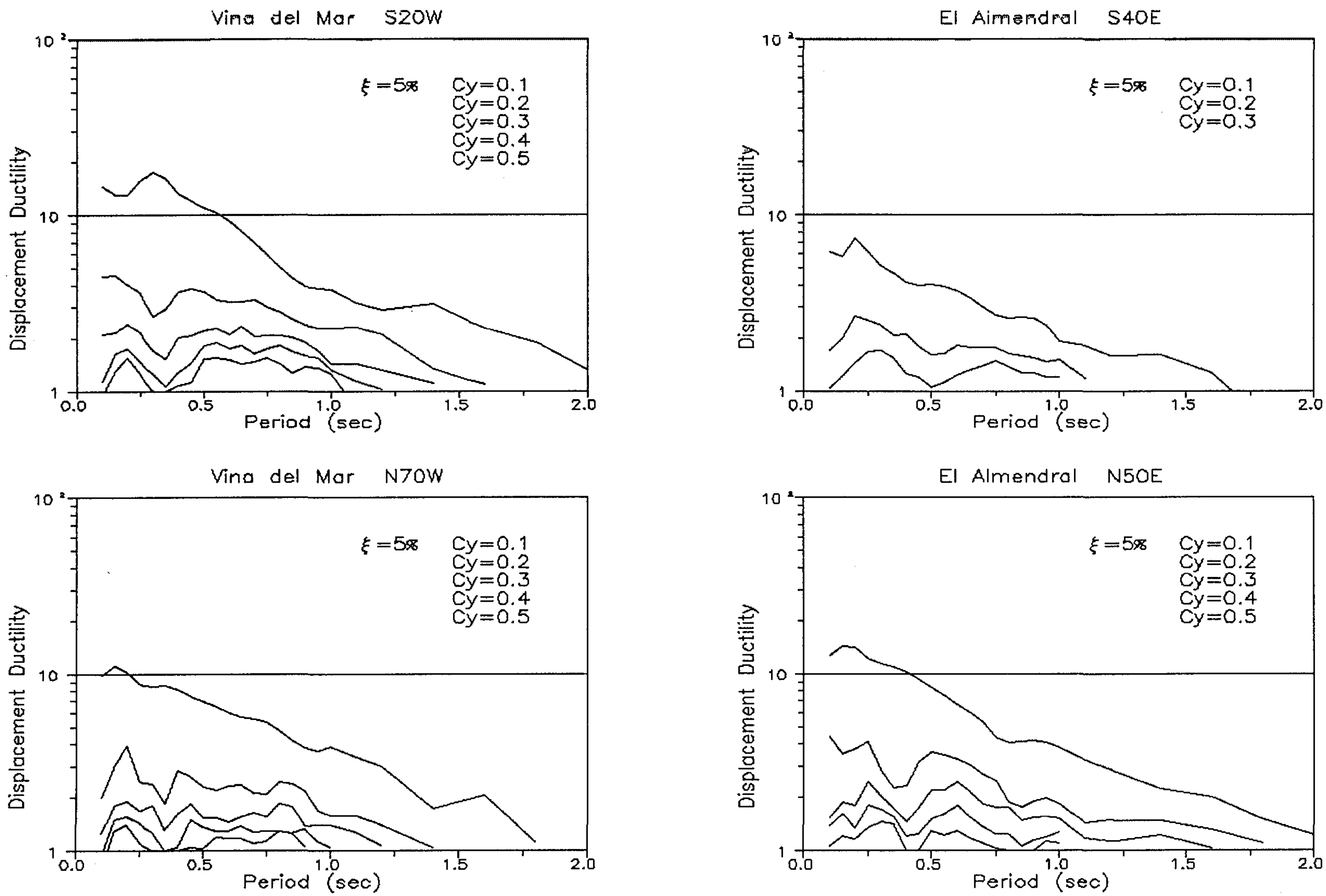


Fig. 5.5 Bilinear Response Spectra

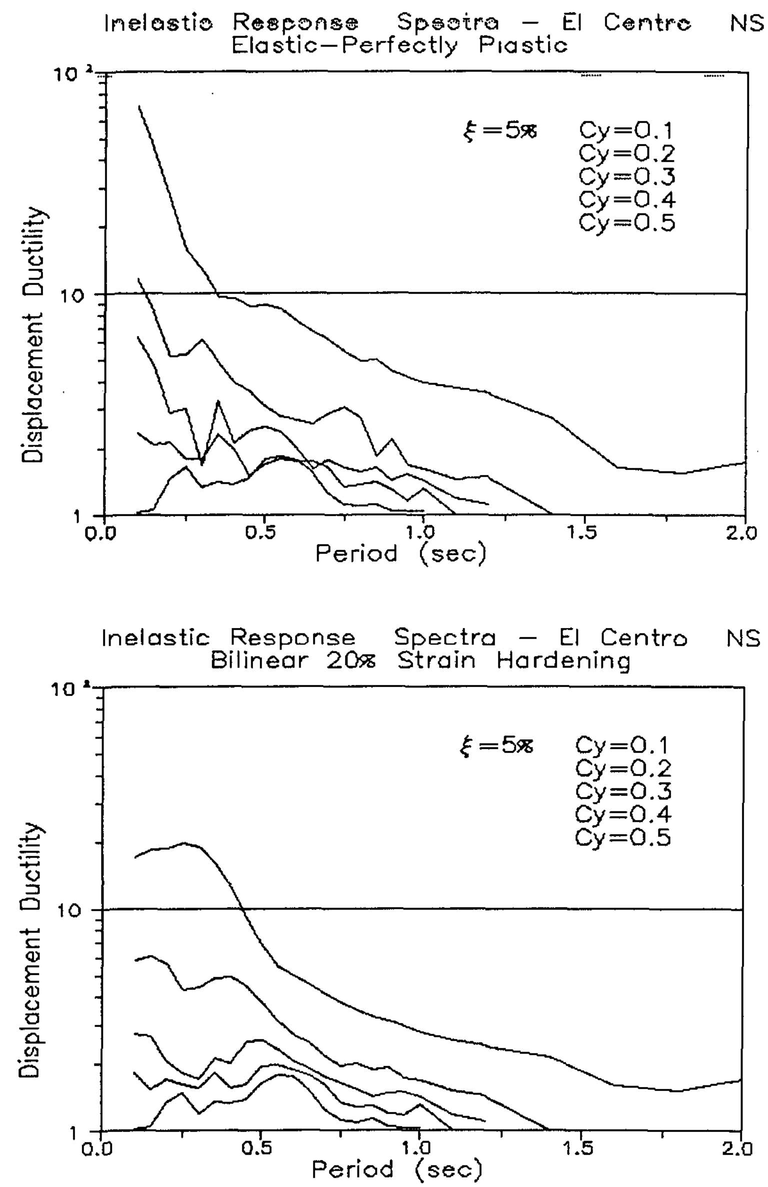


Fig. 5.6 Inelastic Response Spectra from the 1940 El Centro Record

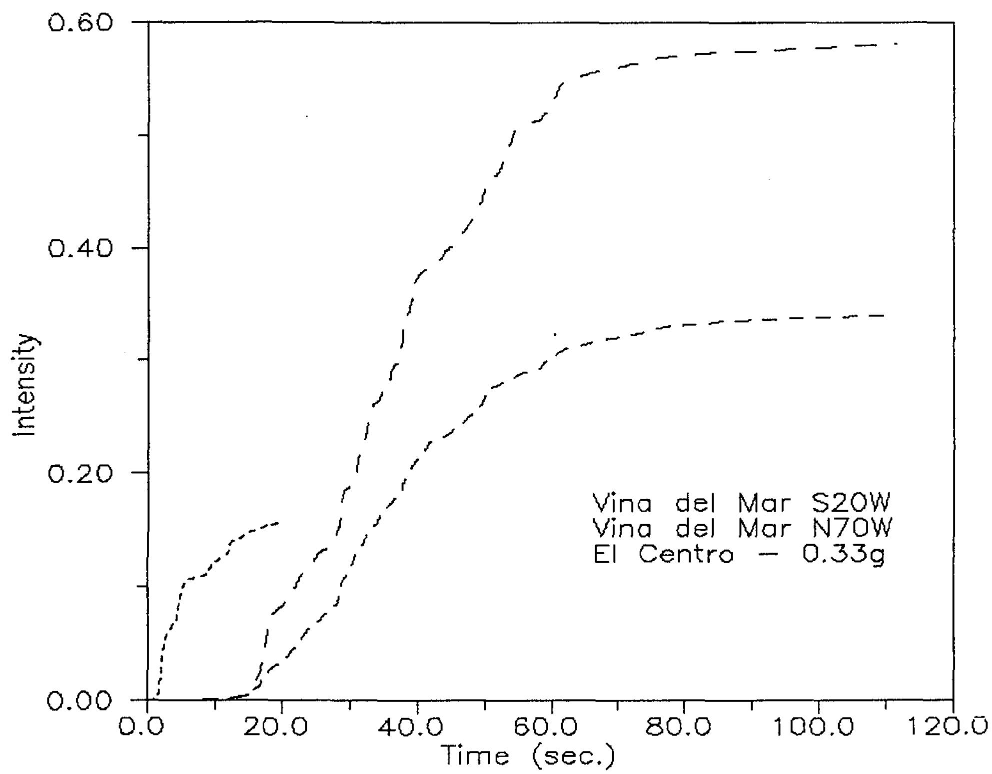


Fig. 5.7 Arias Intensity versus Time

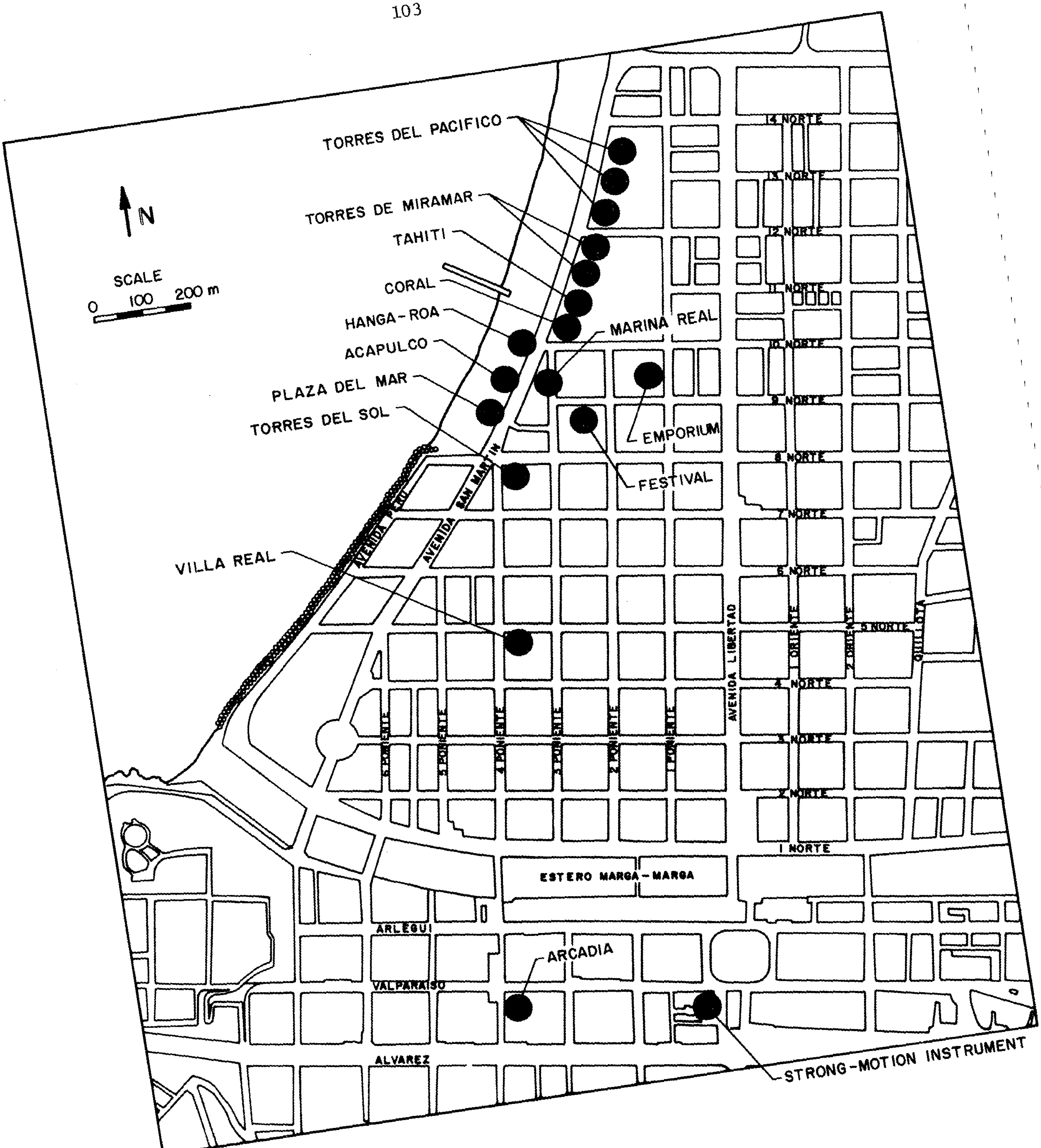


Fig. 6.1 Locations of Buildings in Viña del Mar

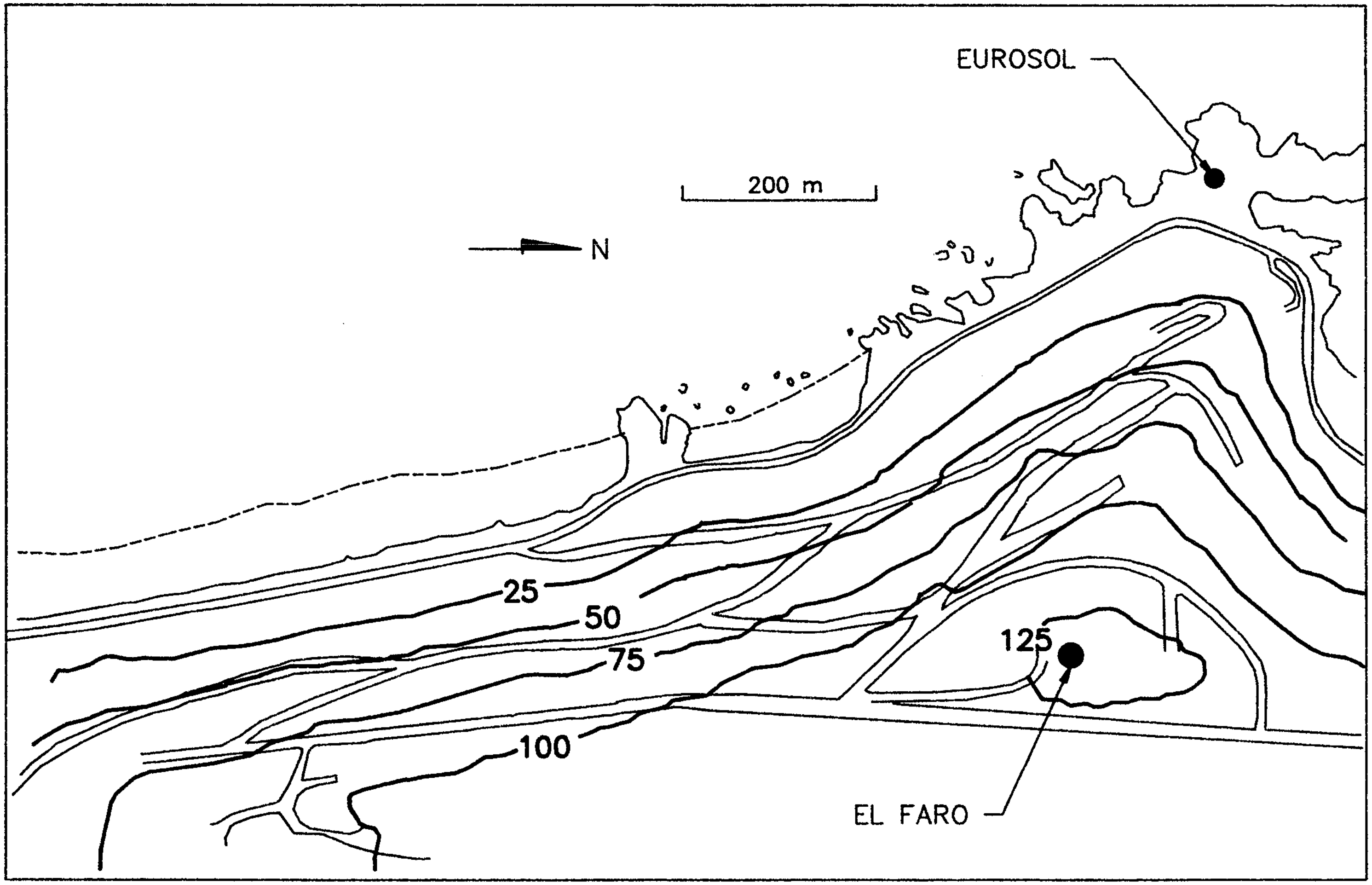


Fig. 6.2 Locations of Buildings in Reñaca

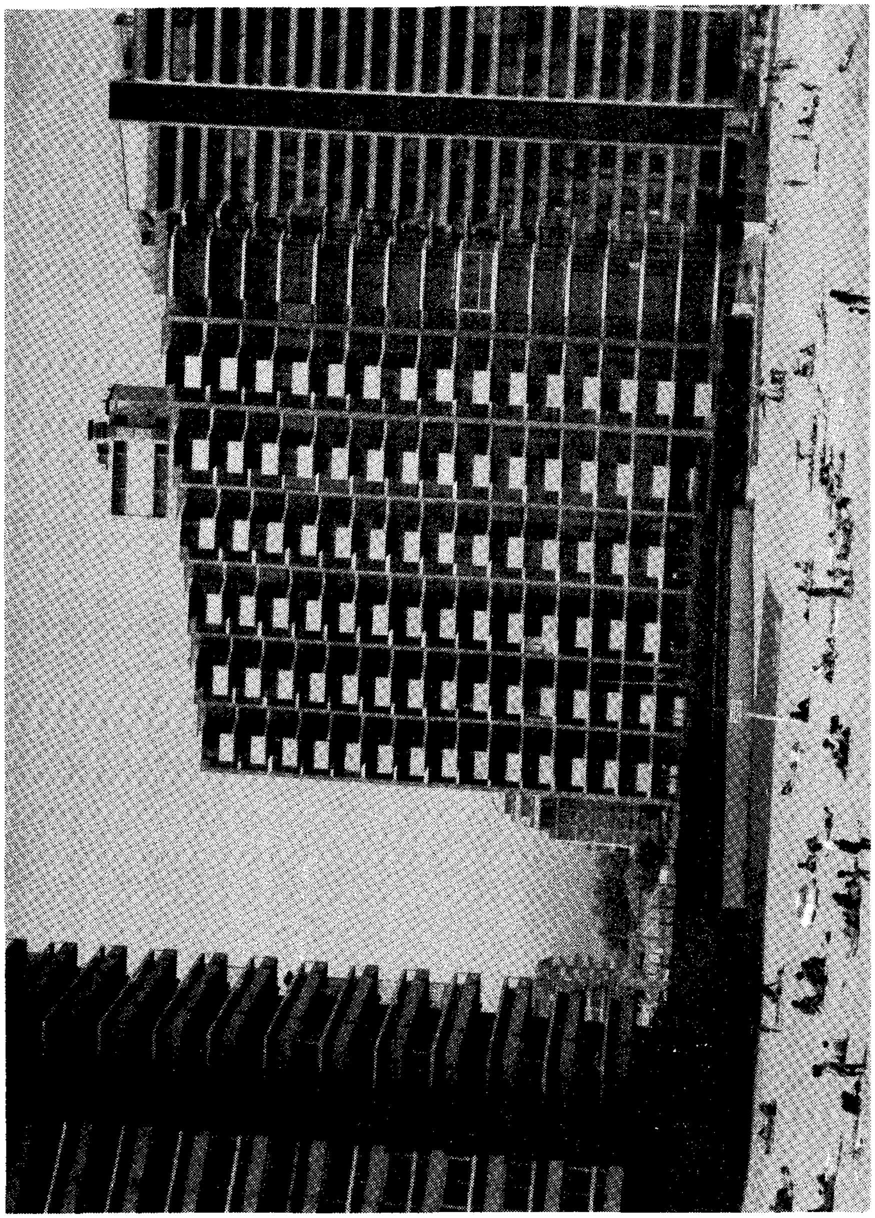


Fig. 6.3 Acapulco

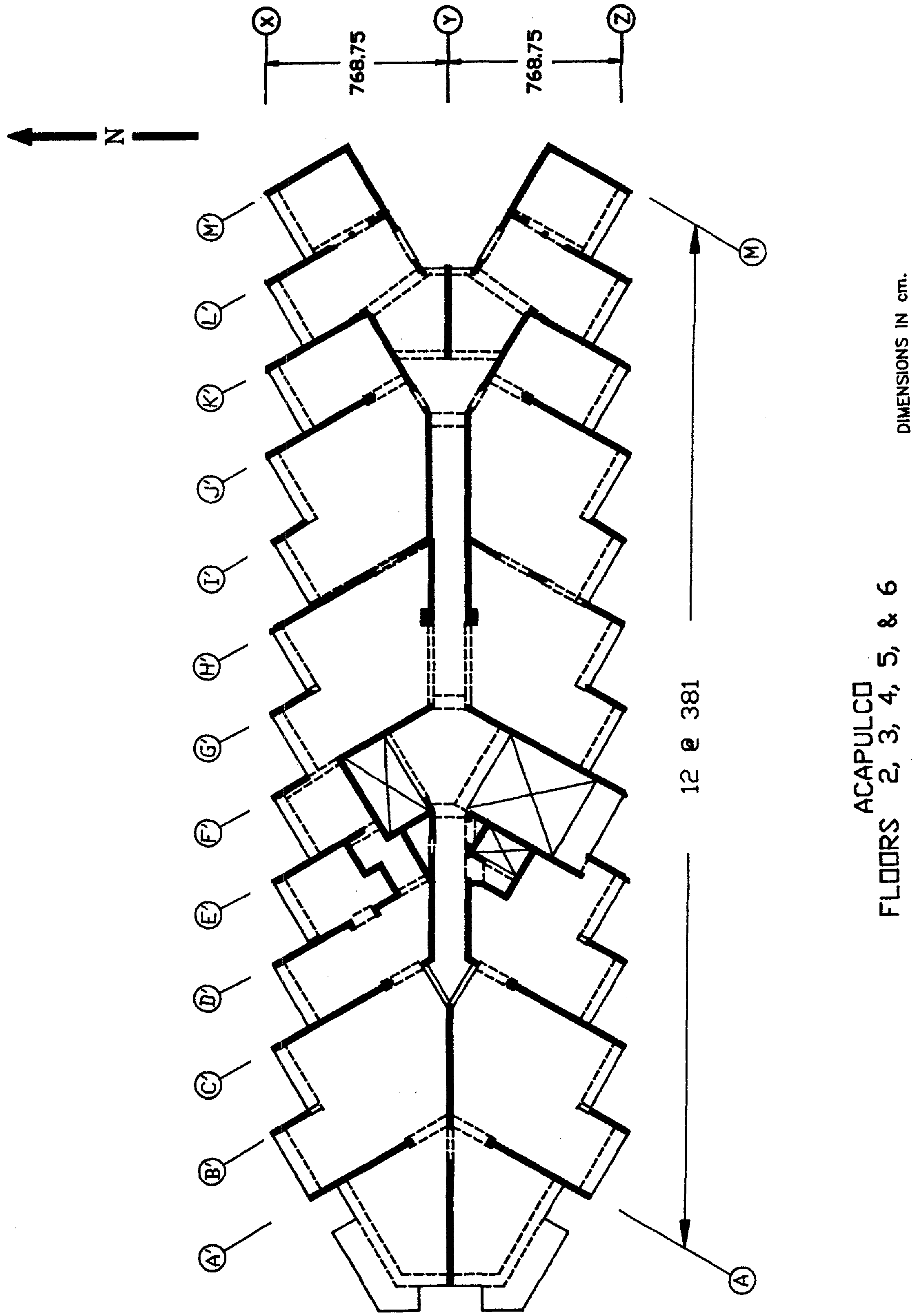


Fig. 6.4 Floor Plan - Acapulco

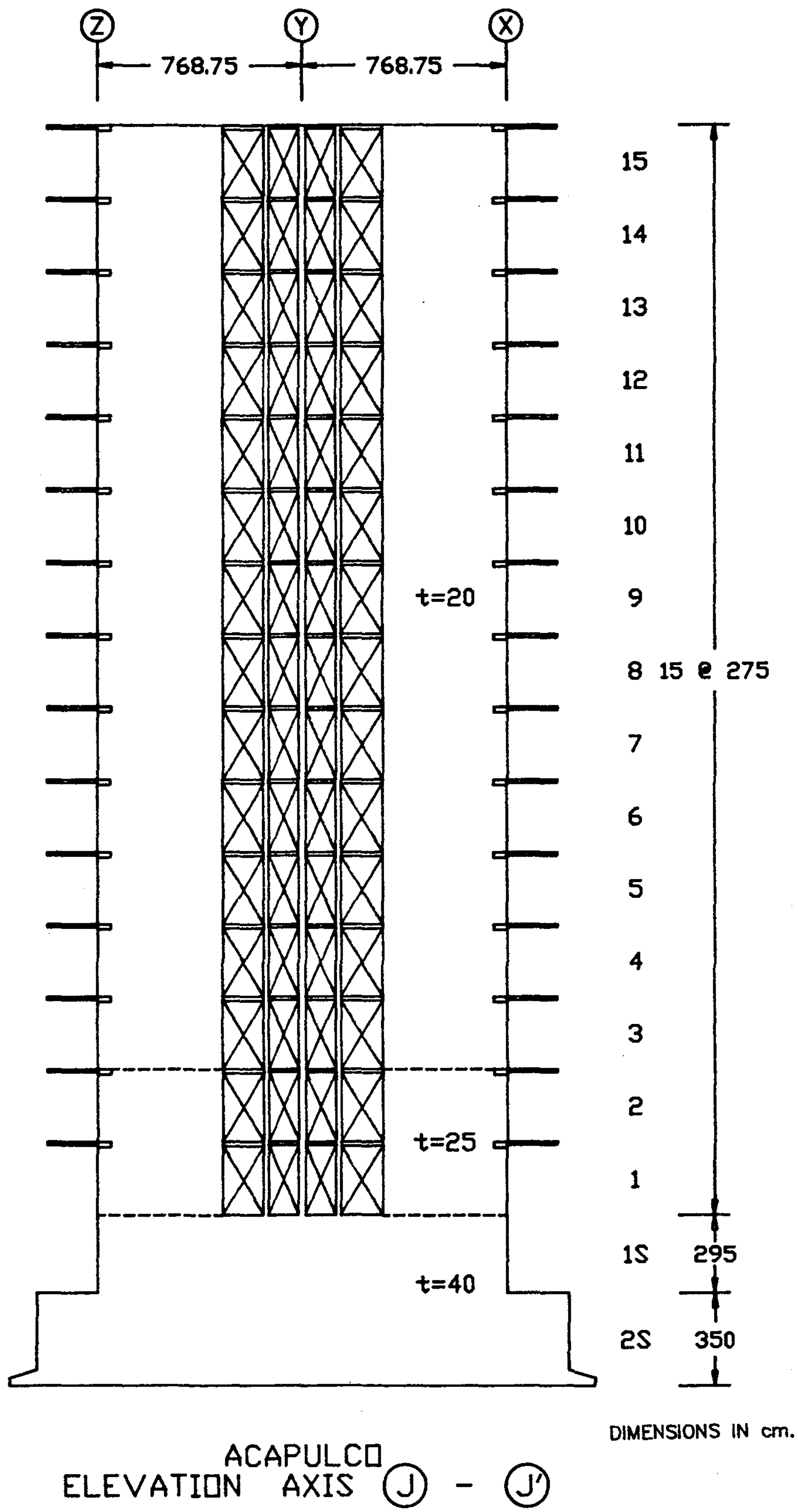


Fig. 6.5 Elevation - Acapulco

Fig. 6.6 Observed Damage - Acapulco

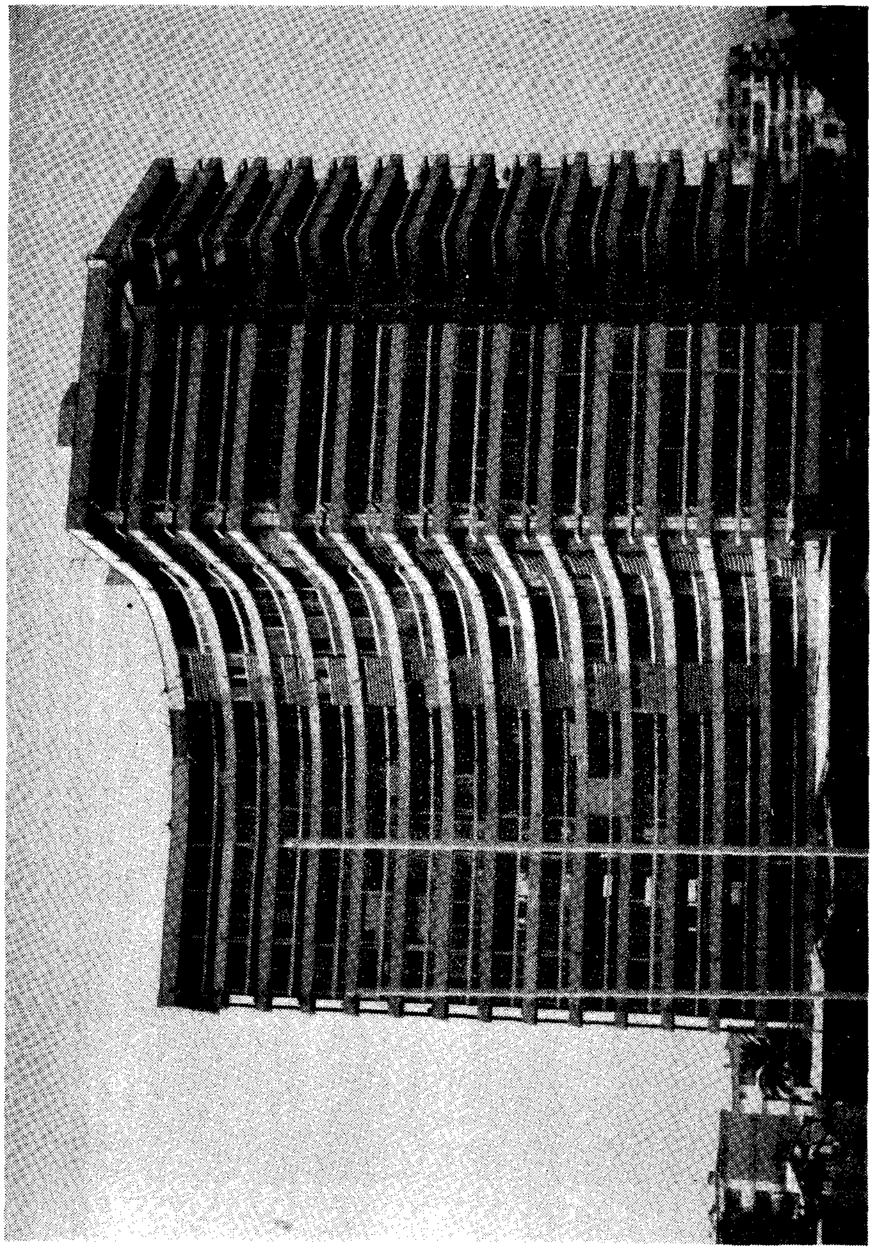


Fig. 6.7 Hanga-Roa

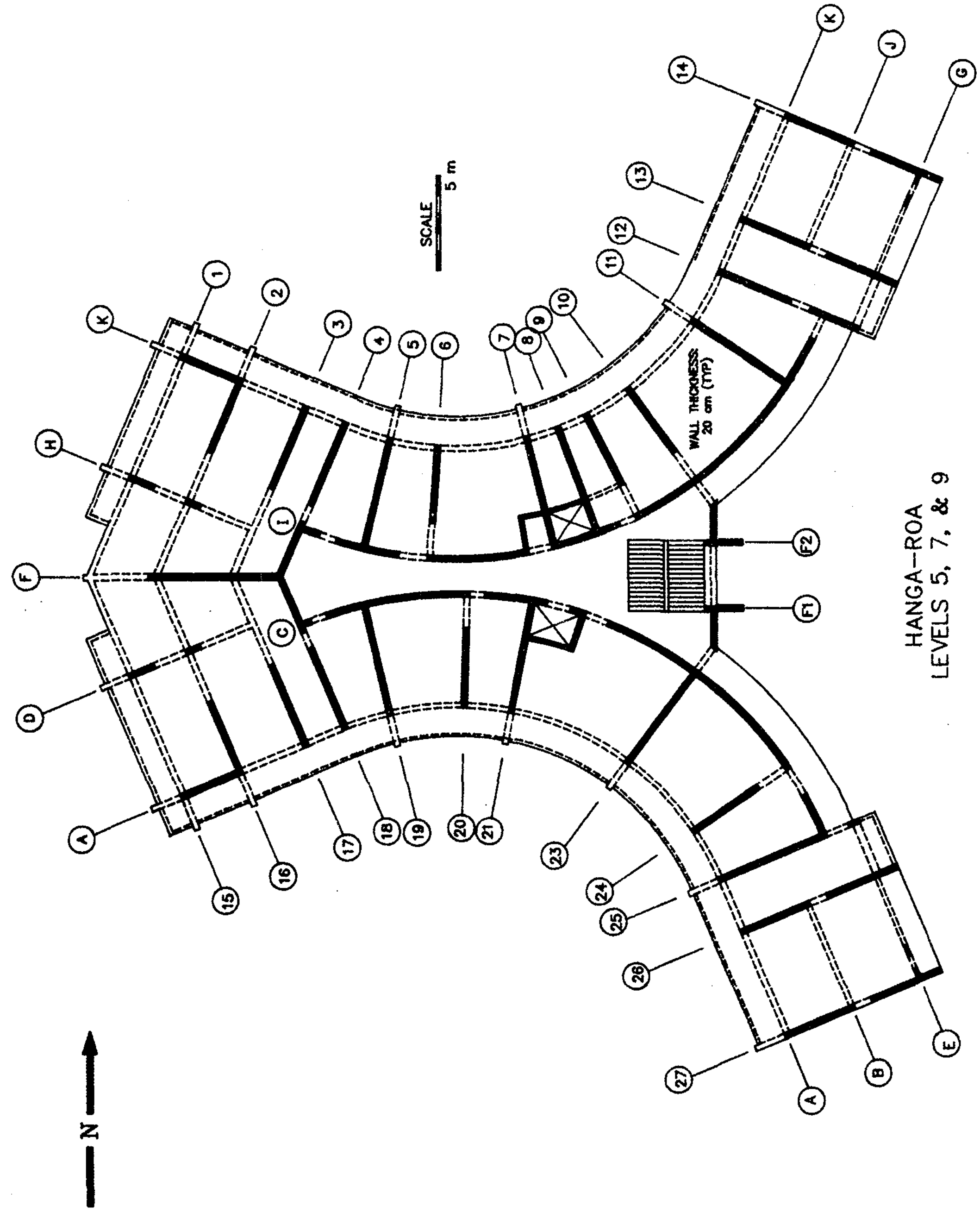


Fig. 6.8 Floor Plan - Hanga-Roa

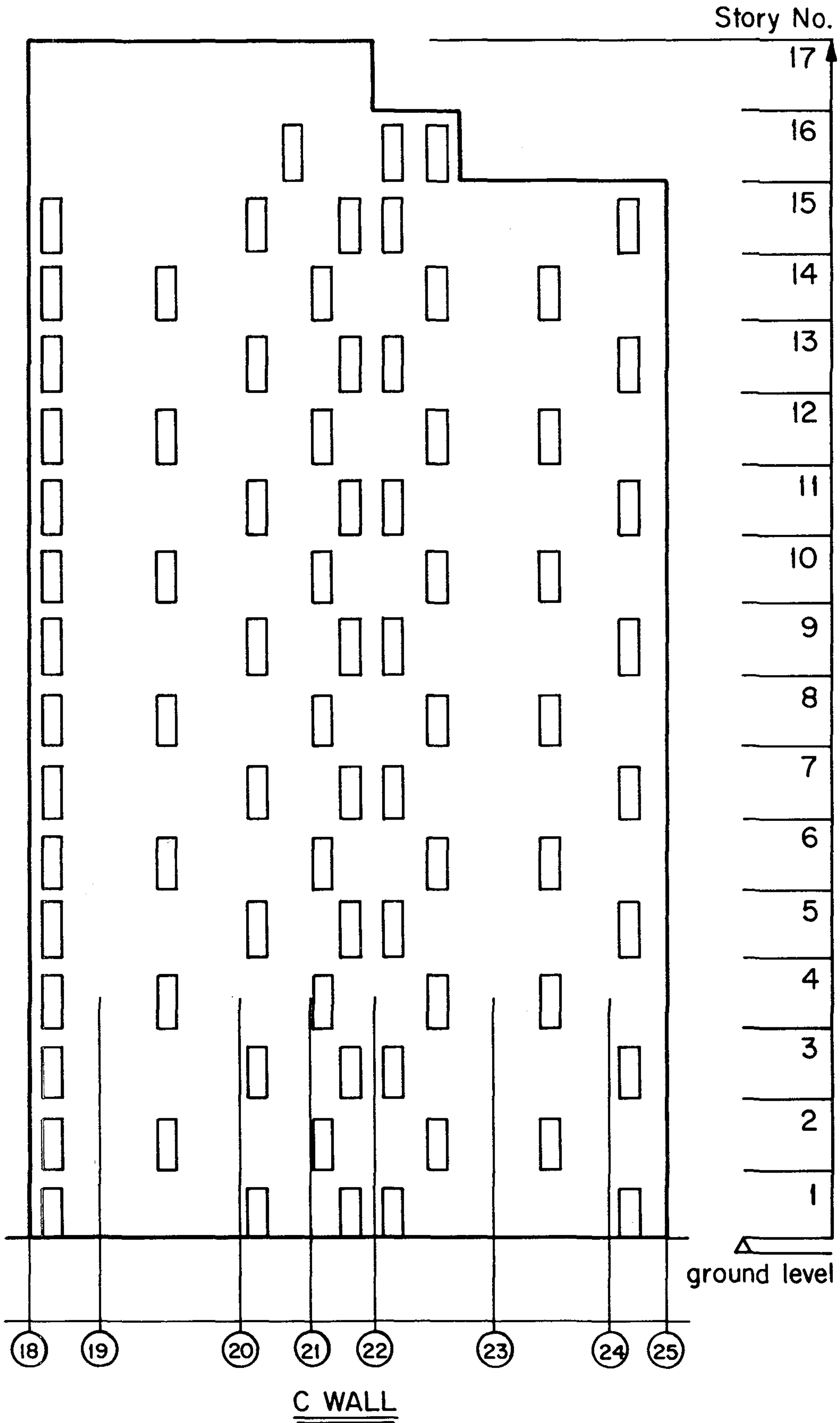


Fig. 6.9 Elevations - Hanga-Roa

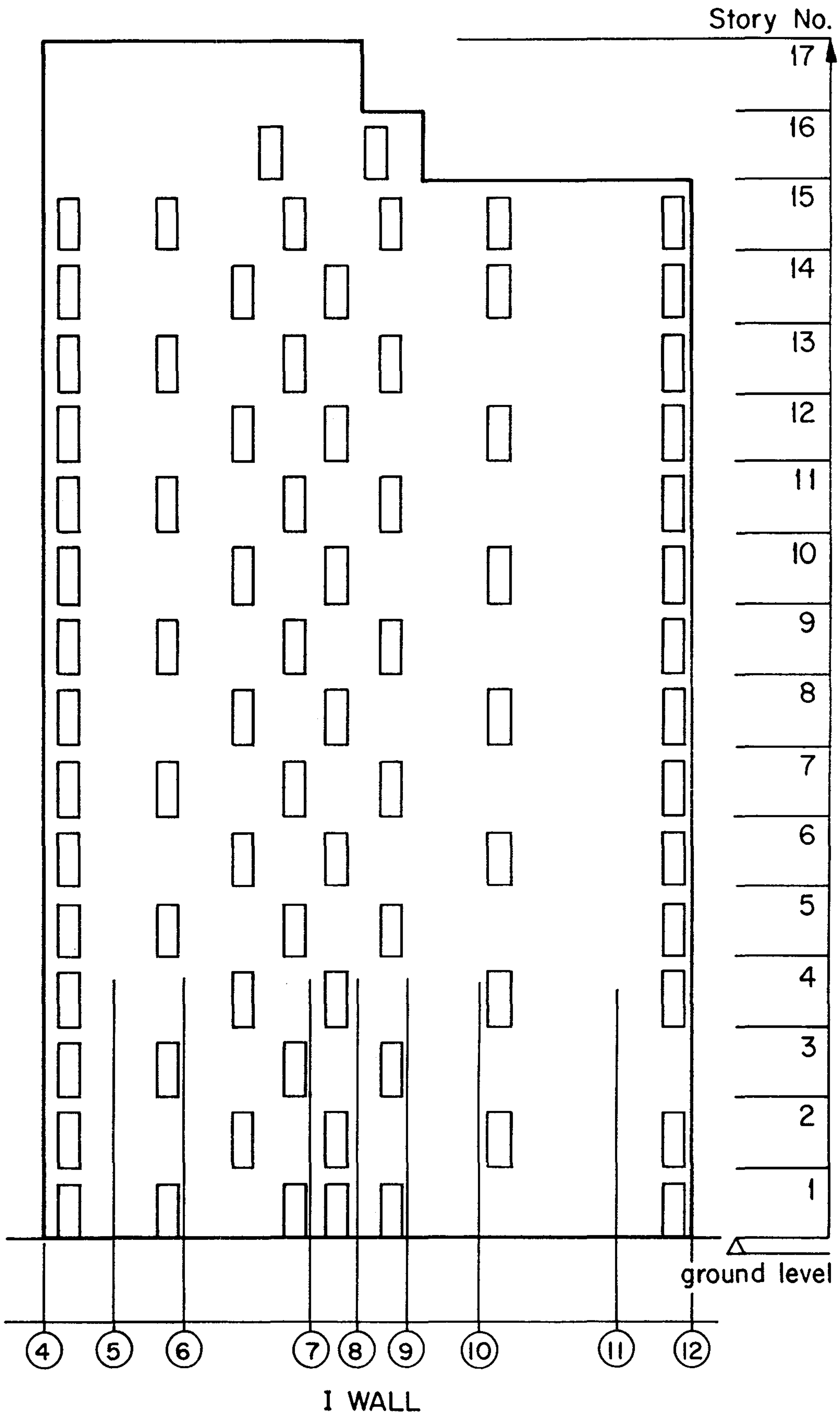


Fig. 6.9 (cont.) Elevations - Hanga-Roa

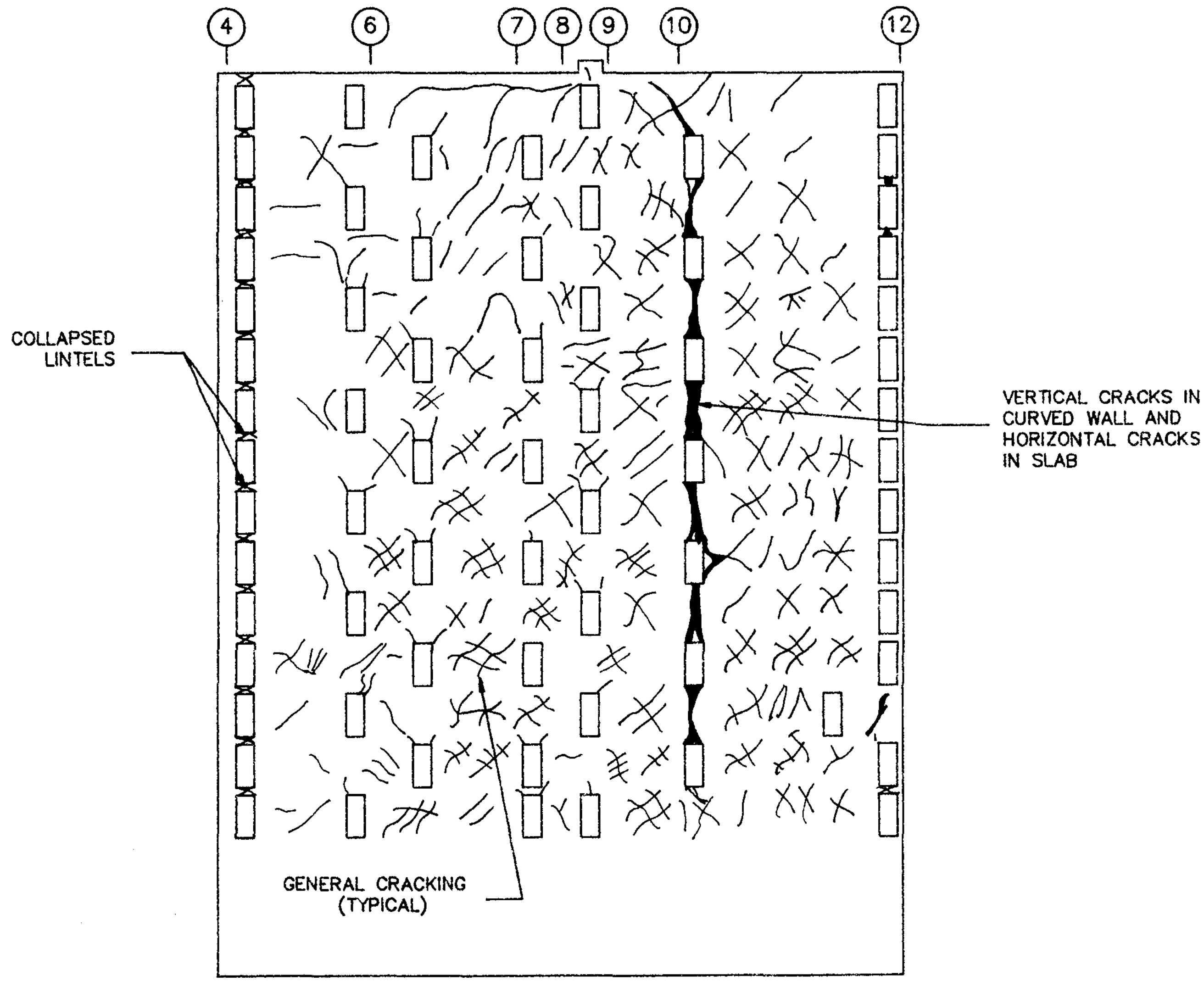


Fig. 6.10 Observed Damage - Hanga-Roa [6]

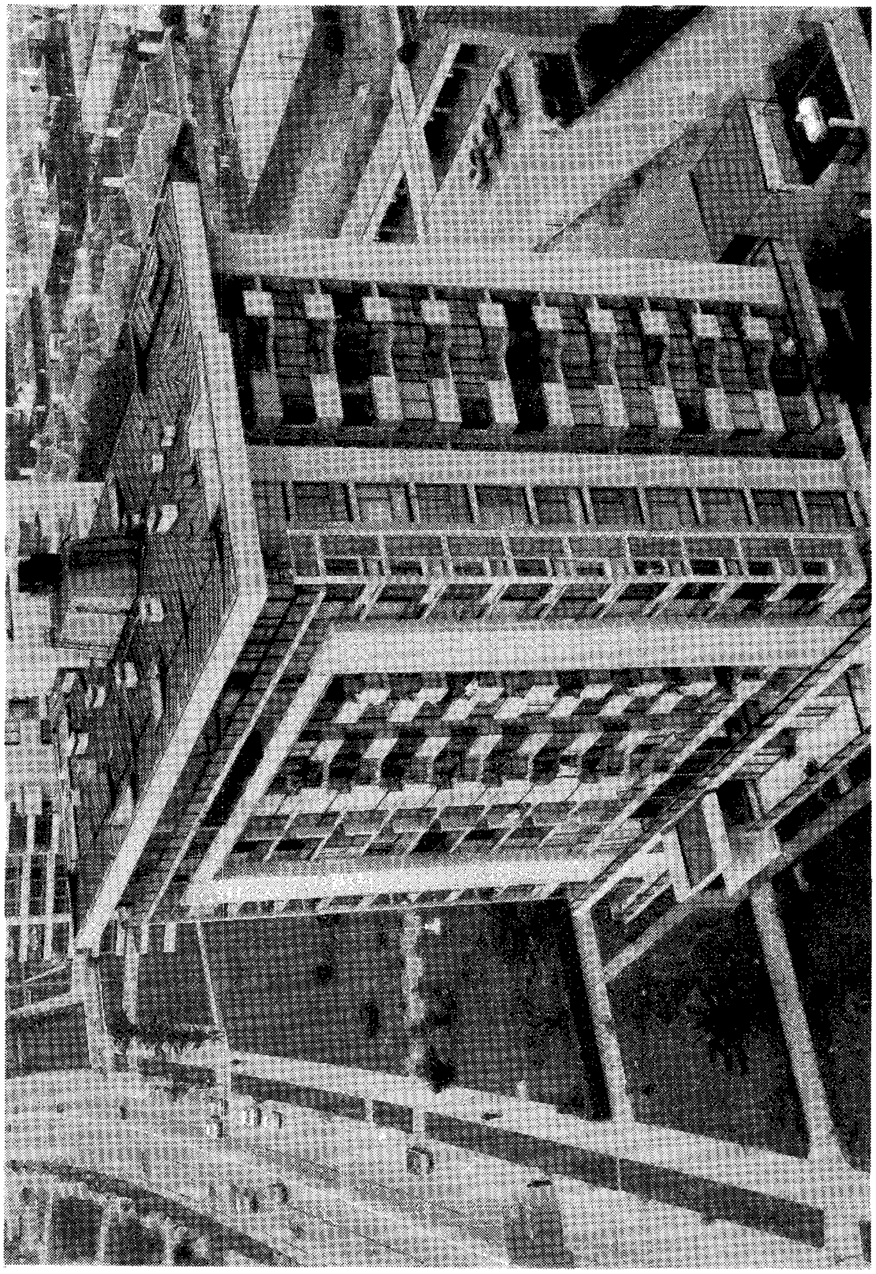


Fig. 6.11 Coral

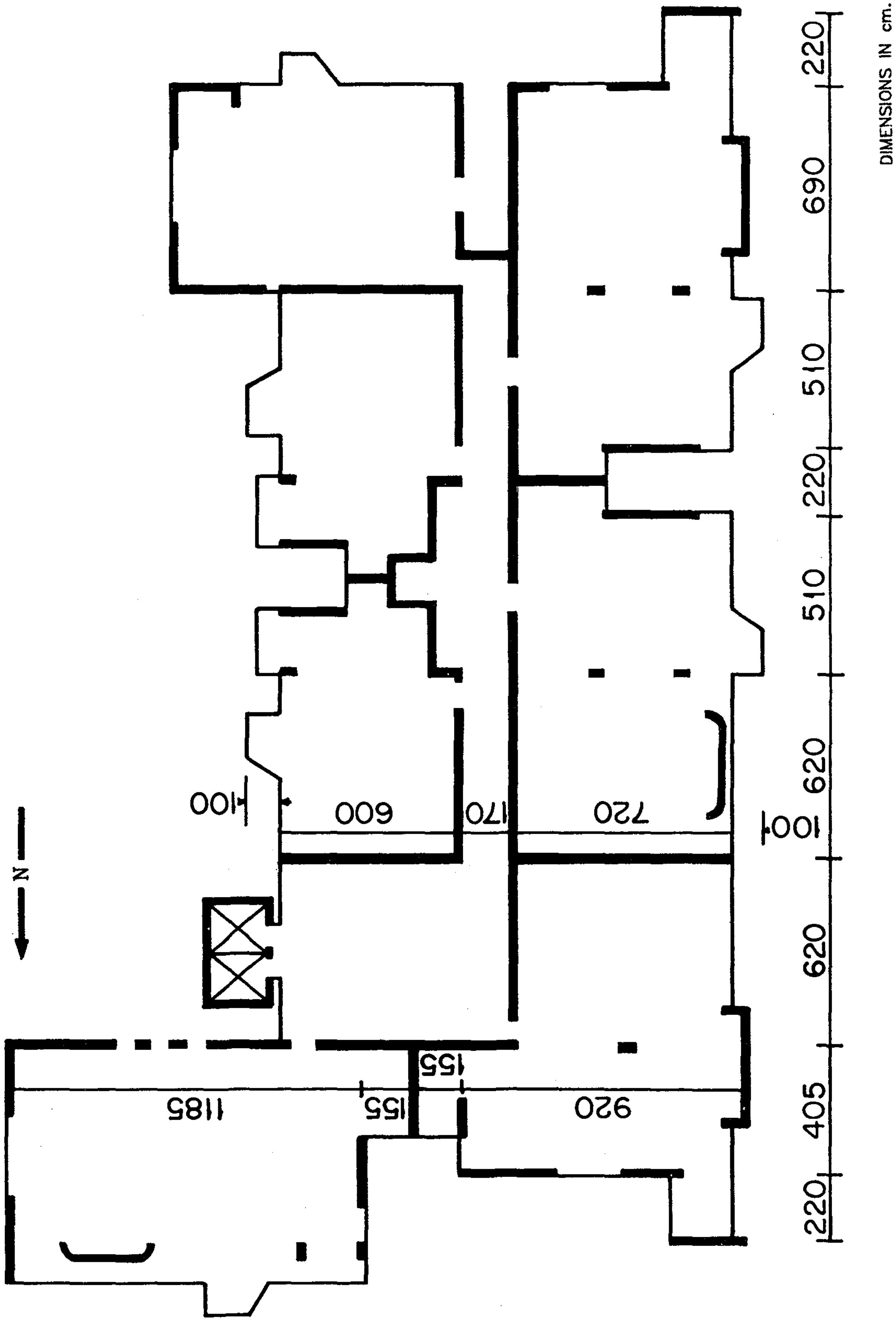


Fig. 6.12 Floor Plan - Coral

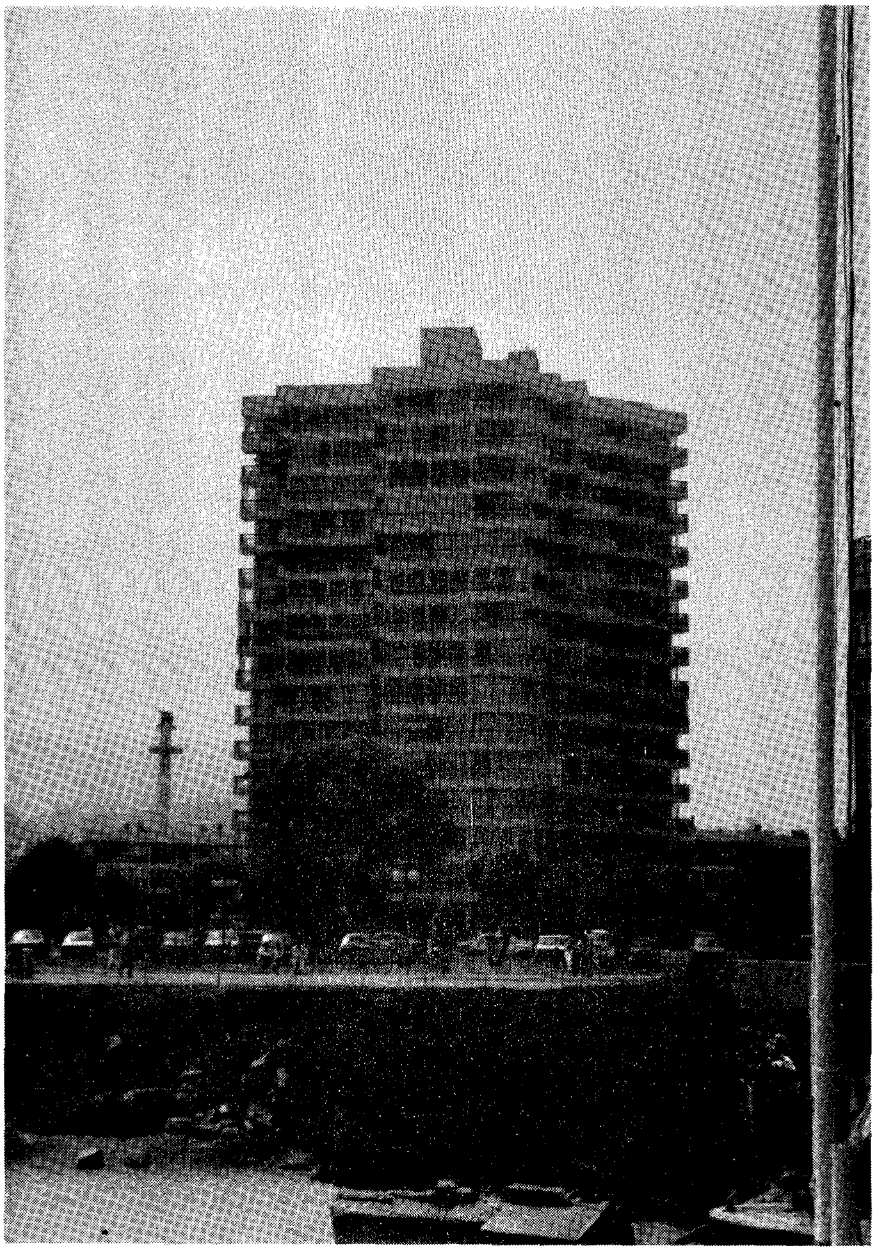


Fig. 6.13 Tahiti

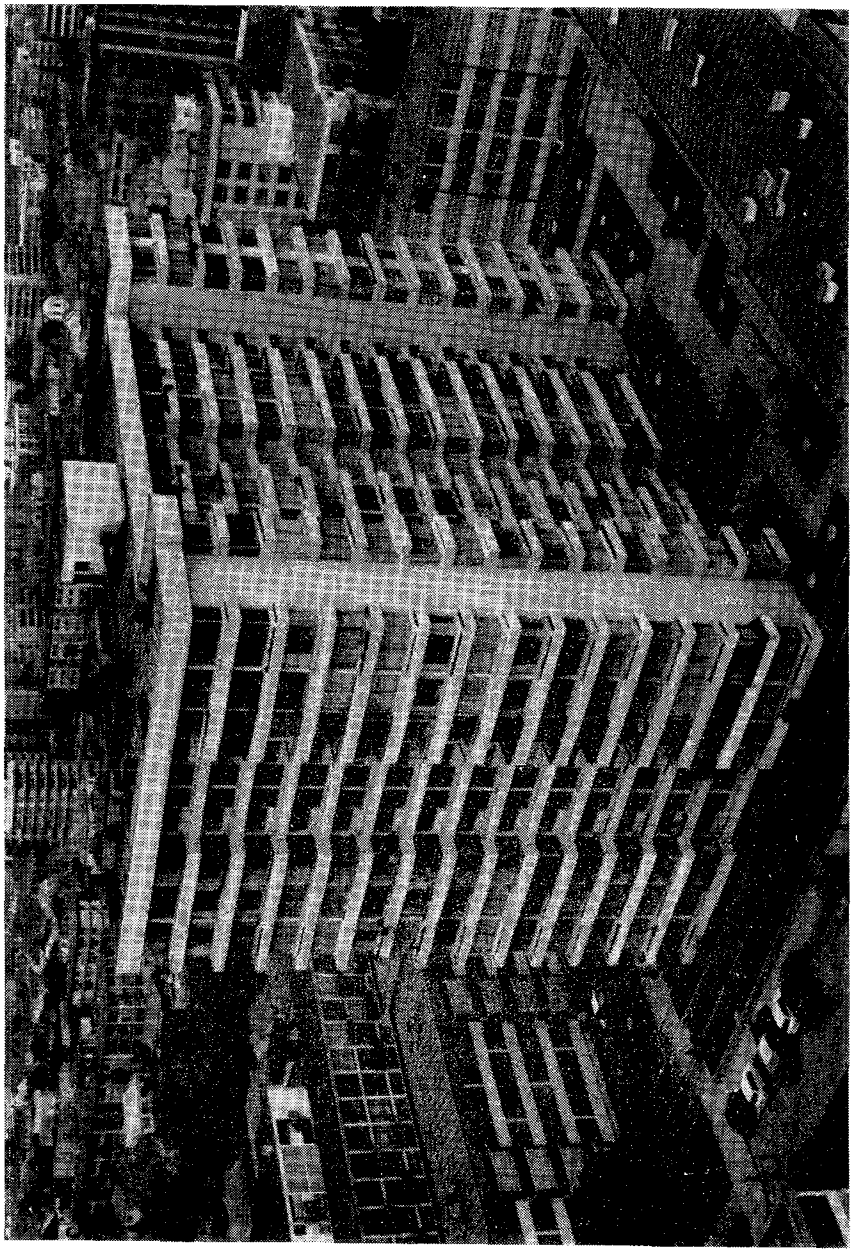


Fig. 6.14 Festival

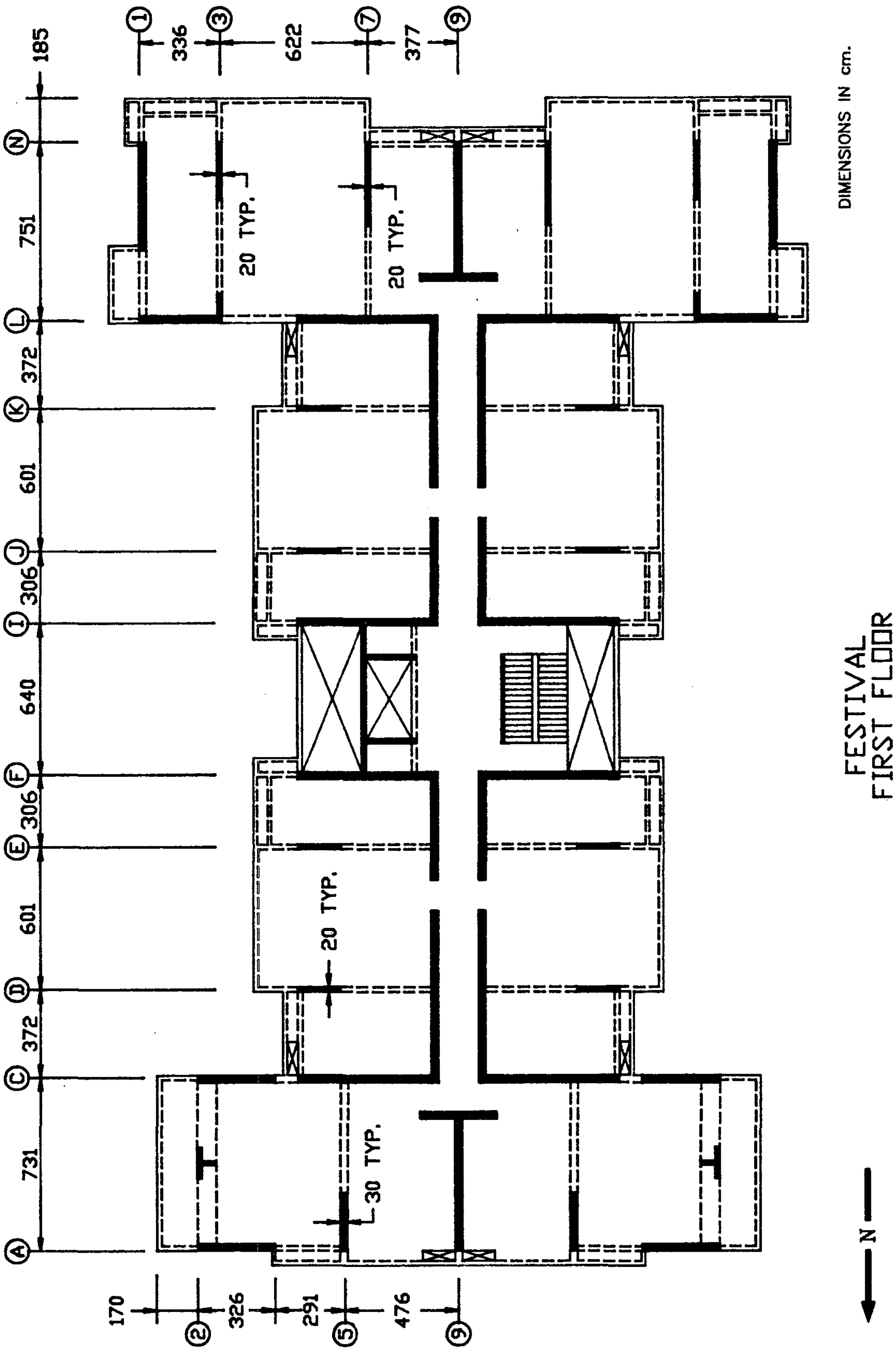


Fig. 6.15 Floor Plan - Festival

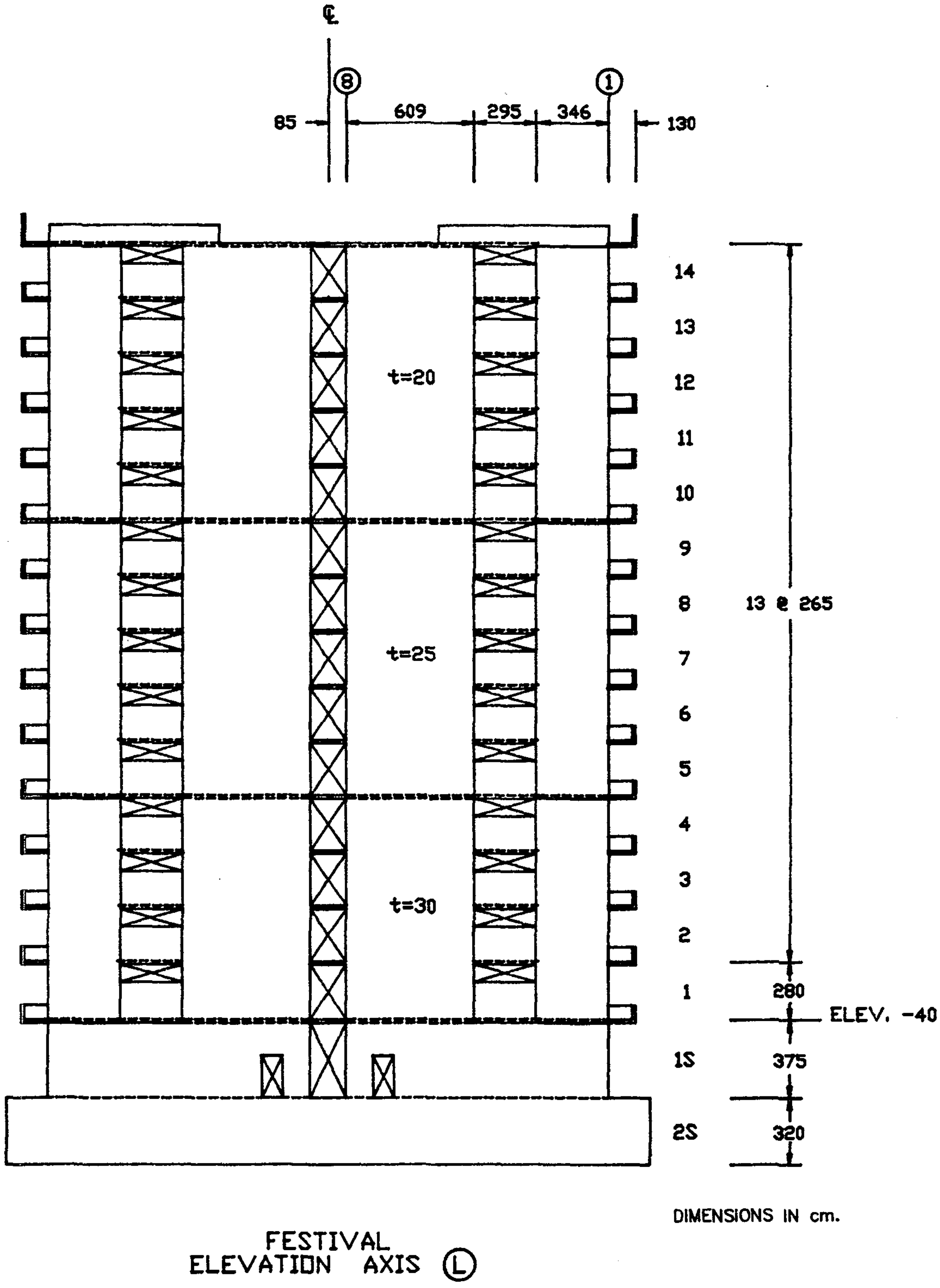


Fig. 6.16 Elevation - Festival

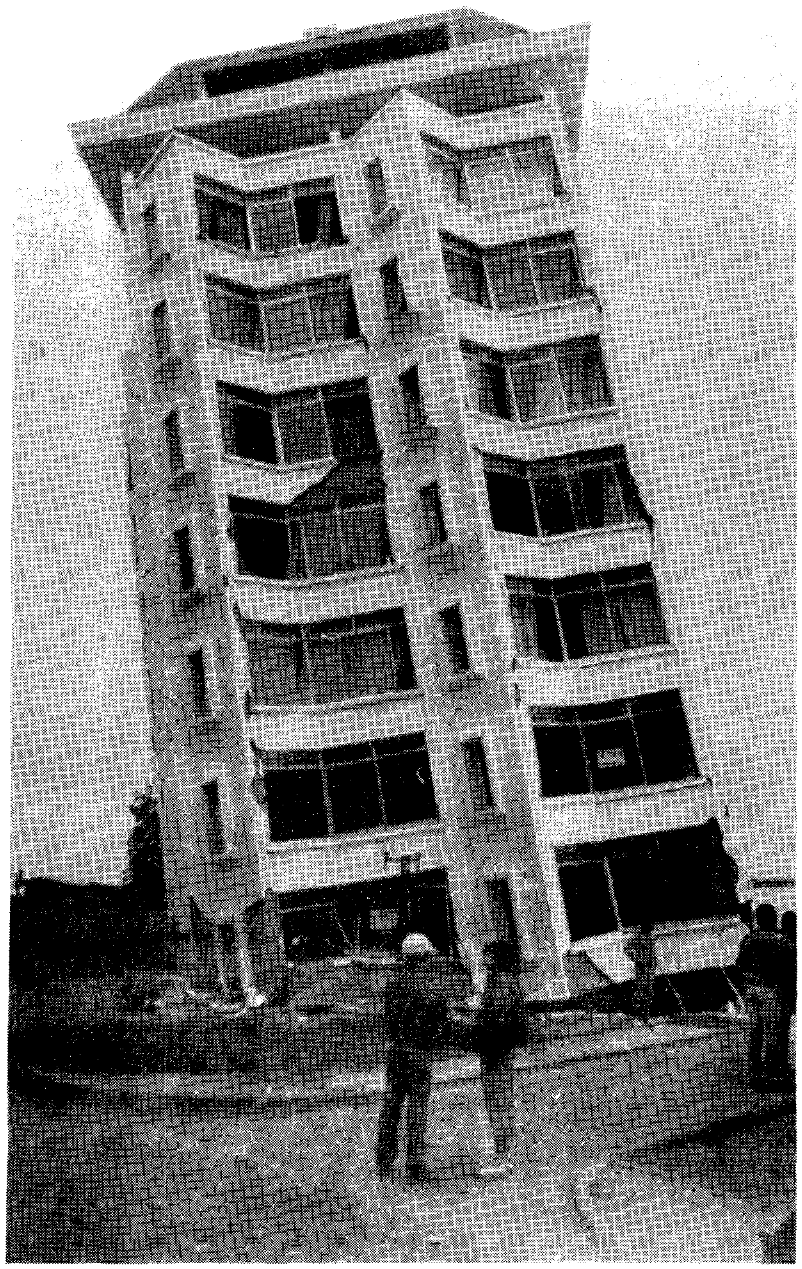


Fig. 6.17 El Faro
(a) After the Earthquake [49]

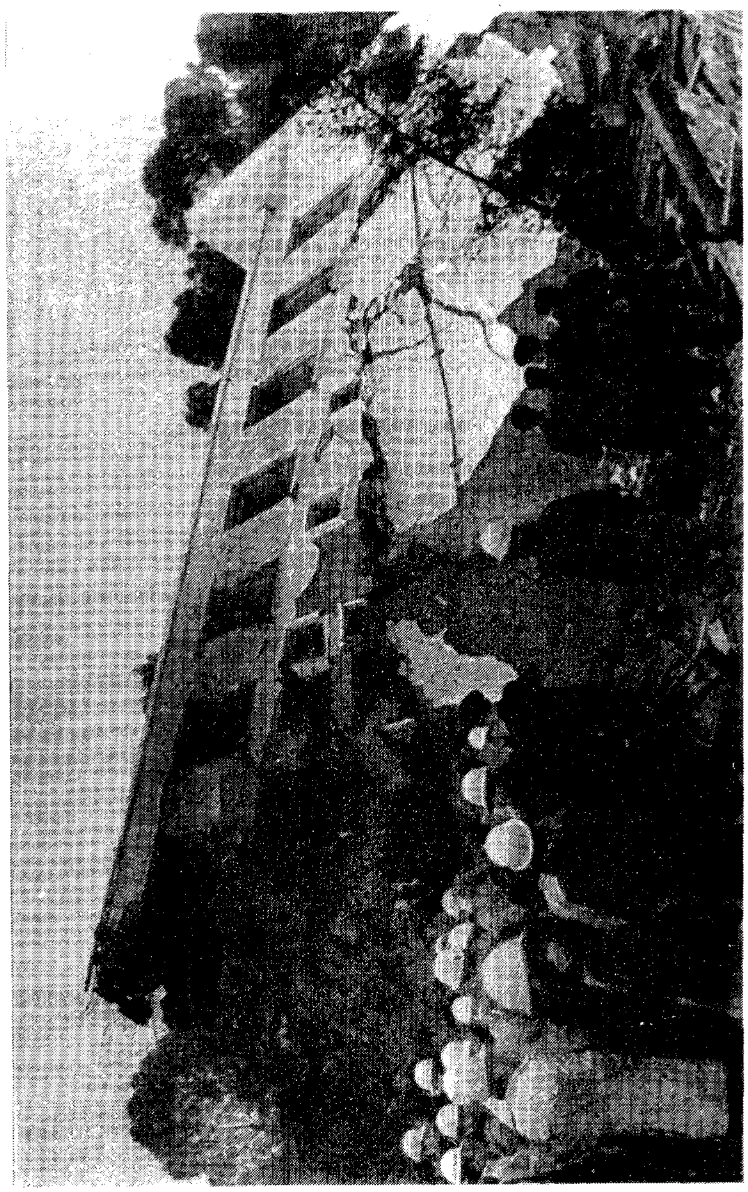
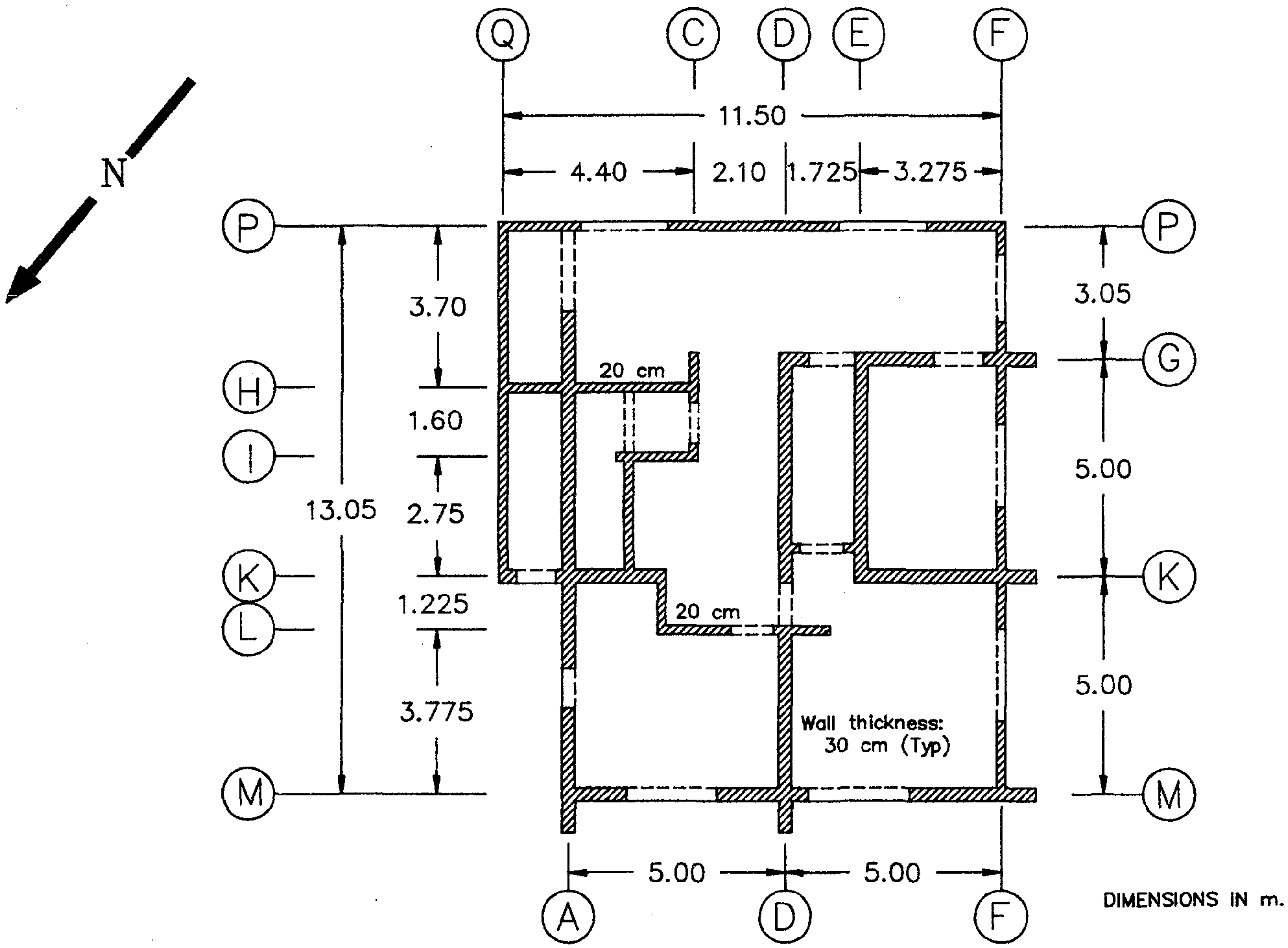


Fig. 6.17 (cont.) El Faro
(b) After Demolition [20]



EL FARO
Basement Plan

Fig. 6.18 Floor Plans - El Faro

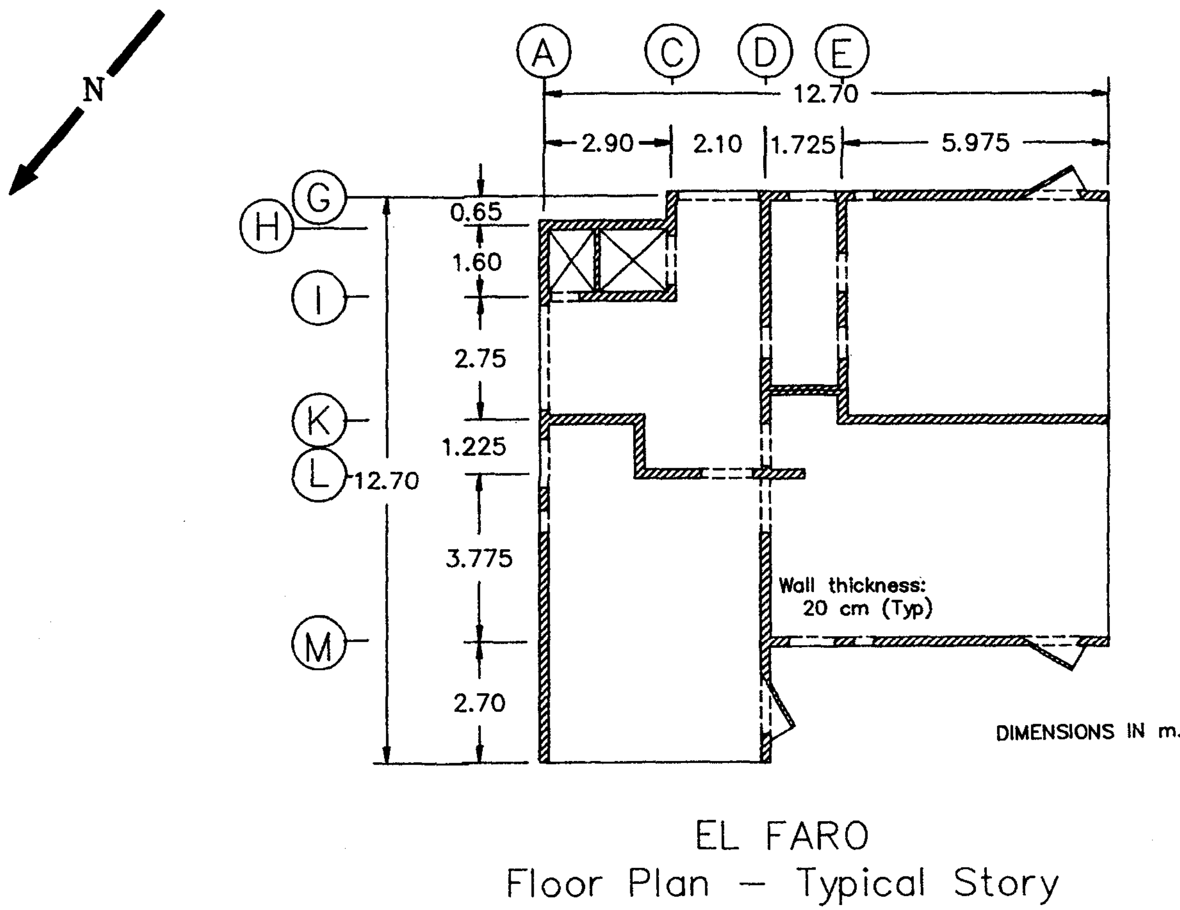


Fig. 6.18 (cont.) Floor Plans - El Faro

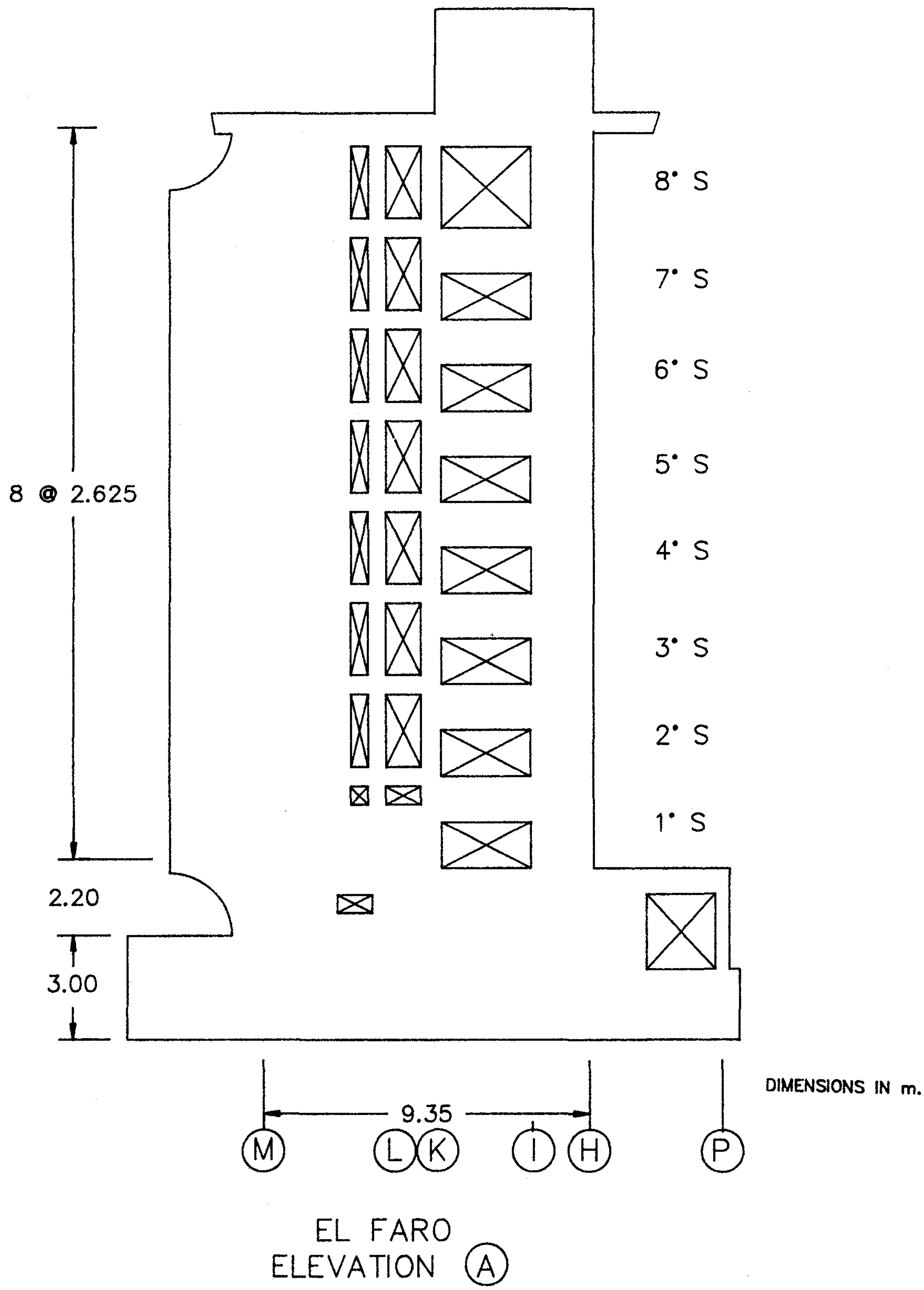


Fig. 6.19 Elevations - El Faro

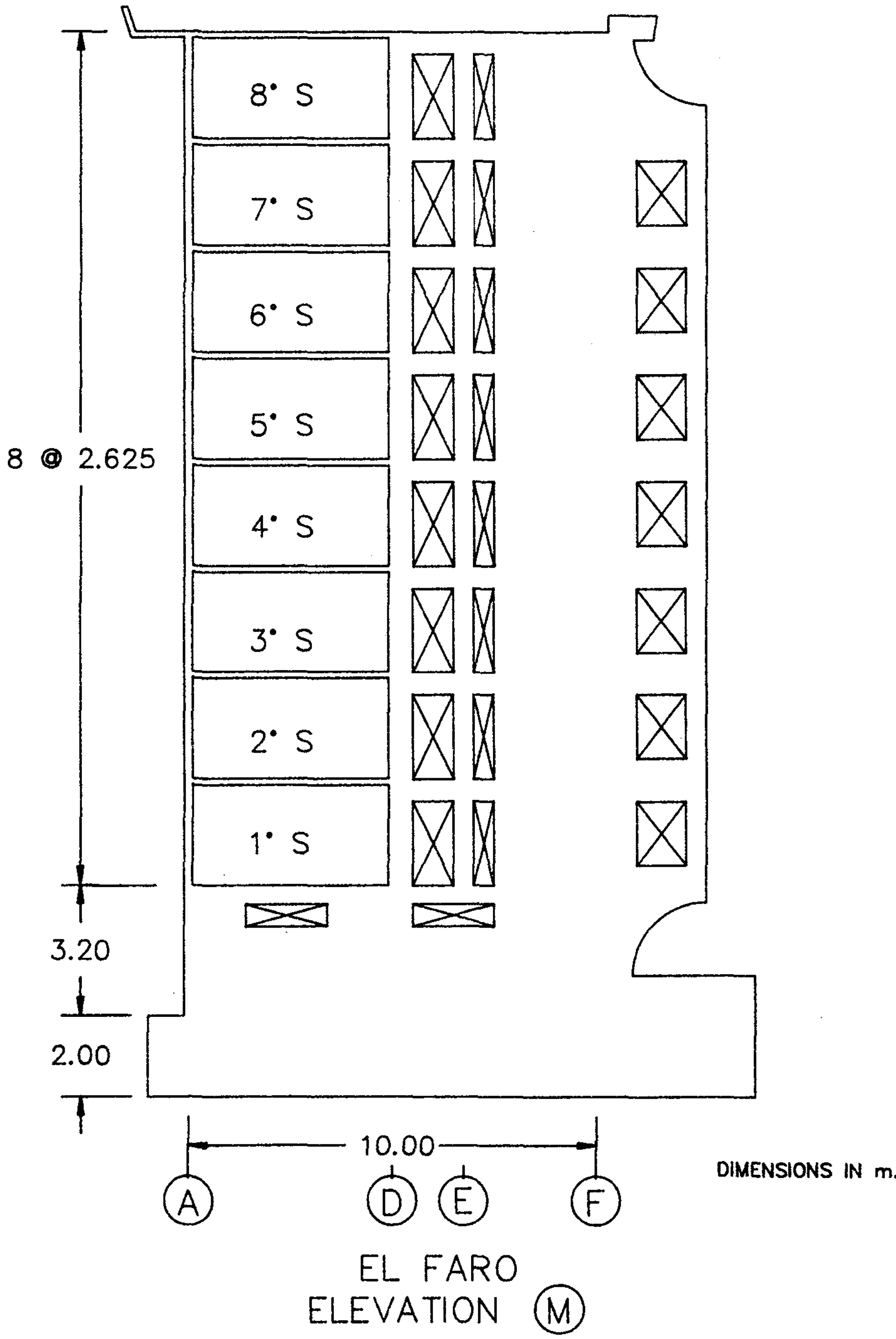
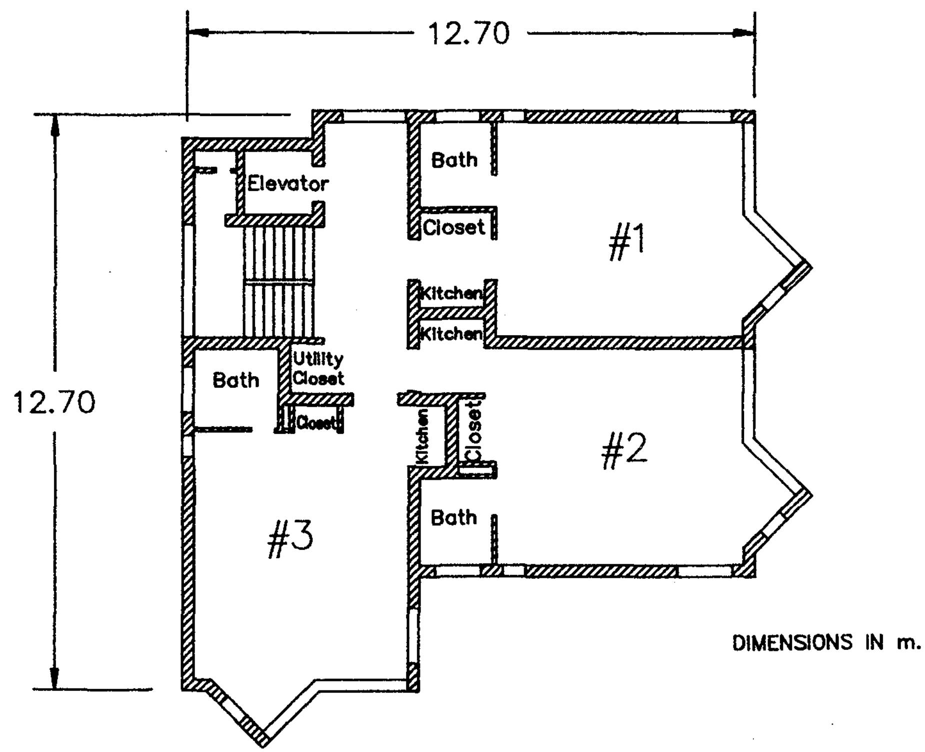


Fig. 6.19 (cont.) Elevations - El Faro



EL FARO
Typical Architectural Plan

Fig. 6.20 Architectural Plan - El Faro

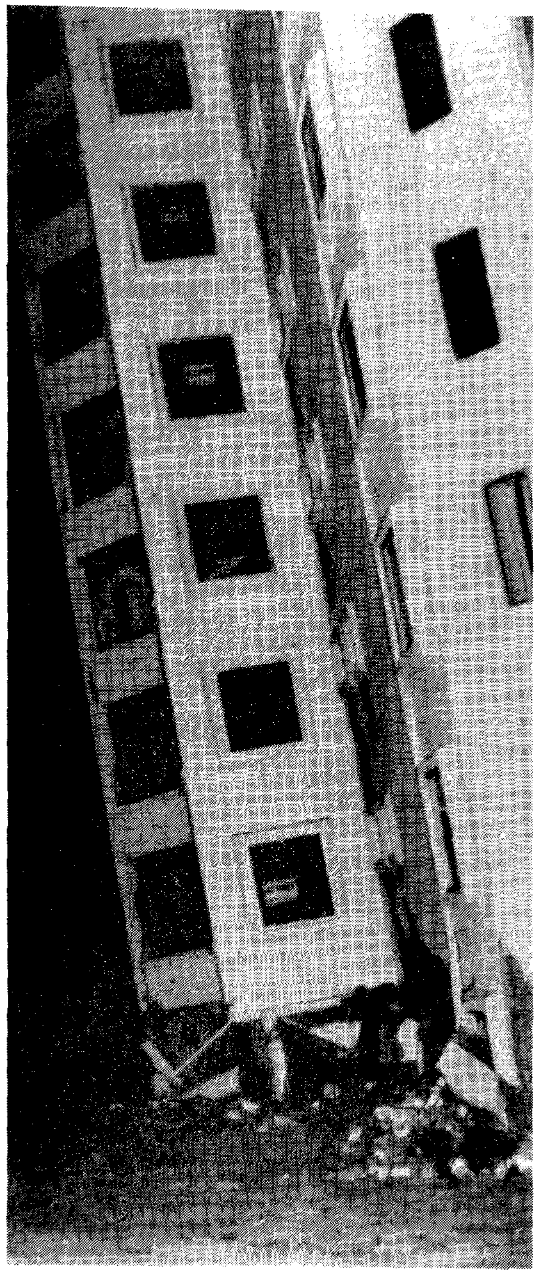


Fig. 6.21 Observed Damage - El Faro [49]

Fig. 6.22 Construction Joint - El Faro

Fig. 7.1 Reinforced Concrete Buildings along the Beach in Viña del Mar (left to right: Torres de Miramar (2), Tahiti, Coral, Marina Real, Torres del Sol, Hanga-Roa, Acapulco, Plaza del Mar)

Fig. 7.2 Plaza del Mar

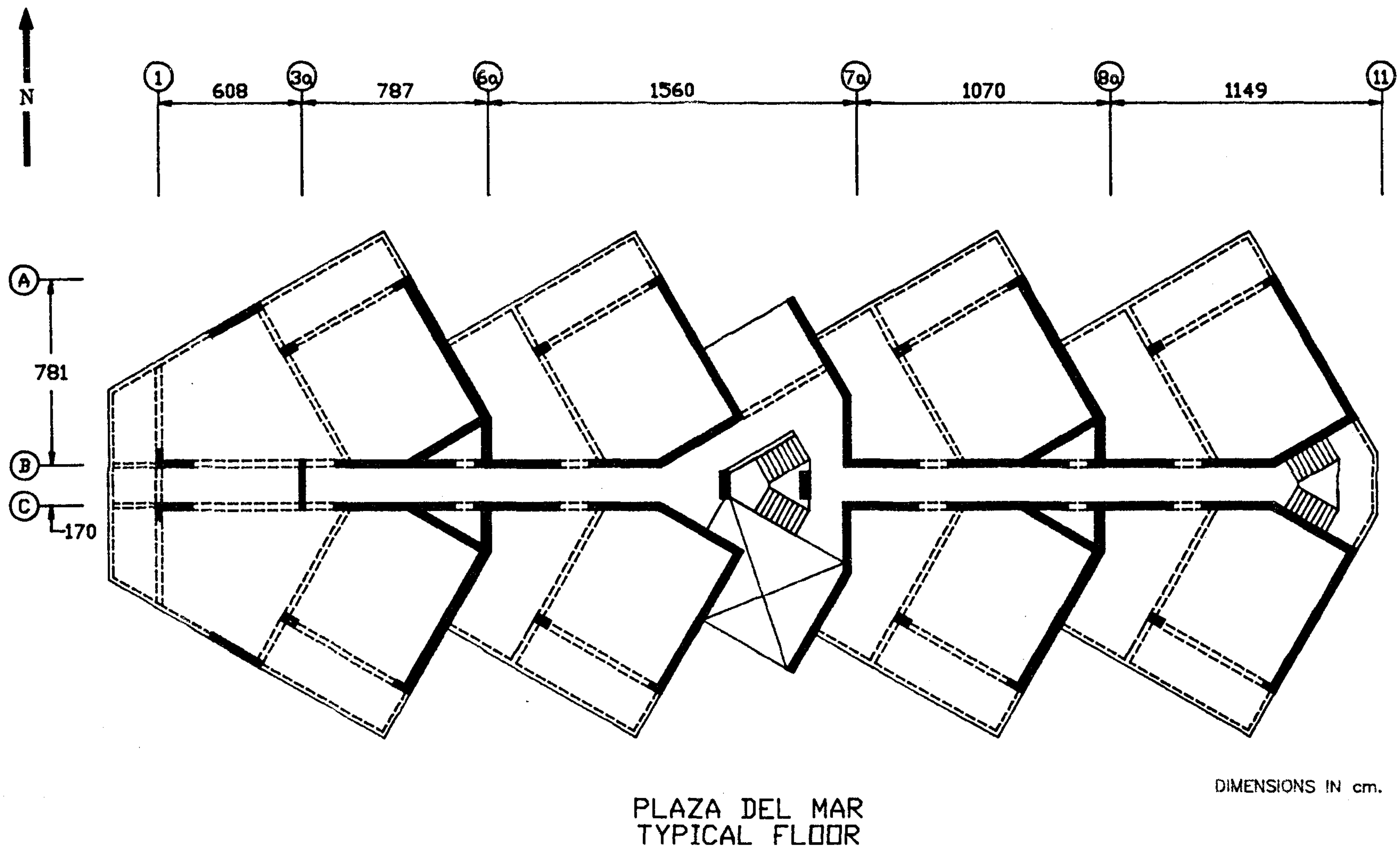


Fig. 7.3 Floor Plan - Plaza del Mar

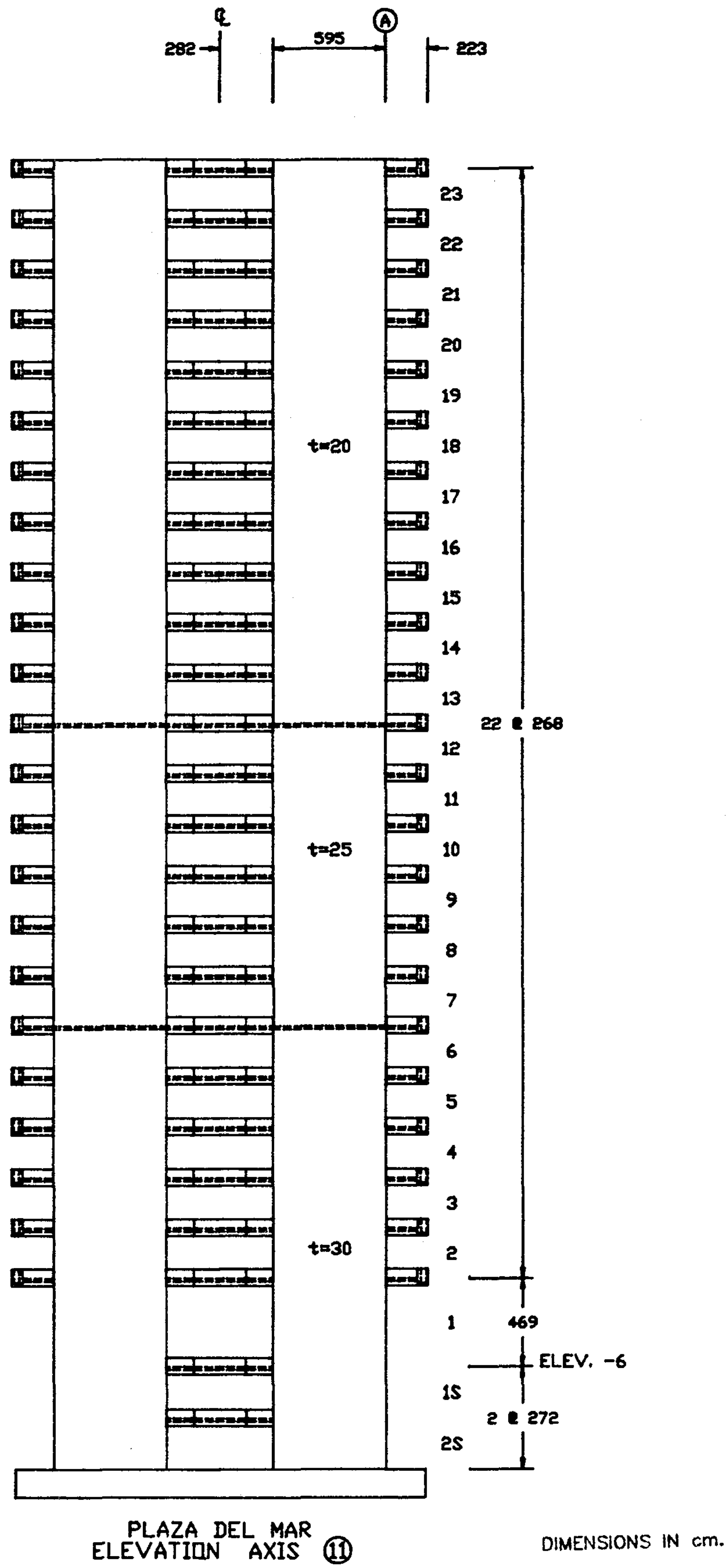


Fig. 7.4 Elevation - Plaza del Mar

Fig. 7.5 Marina Real

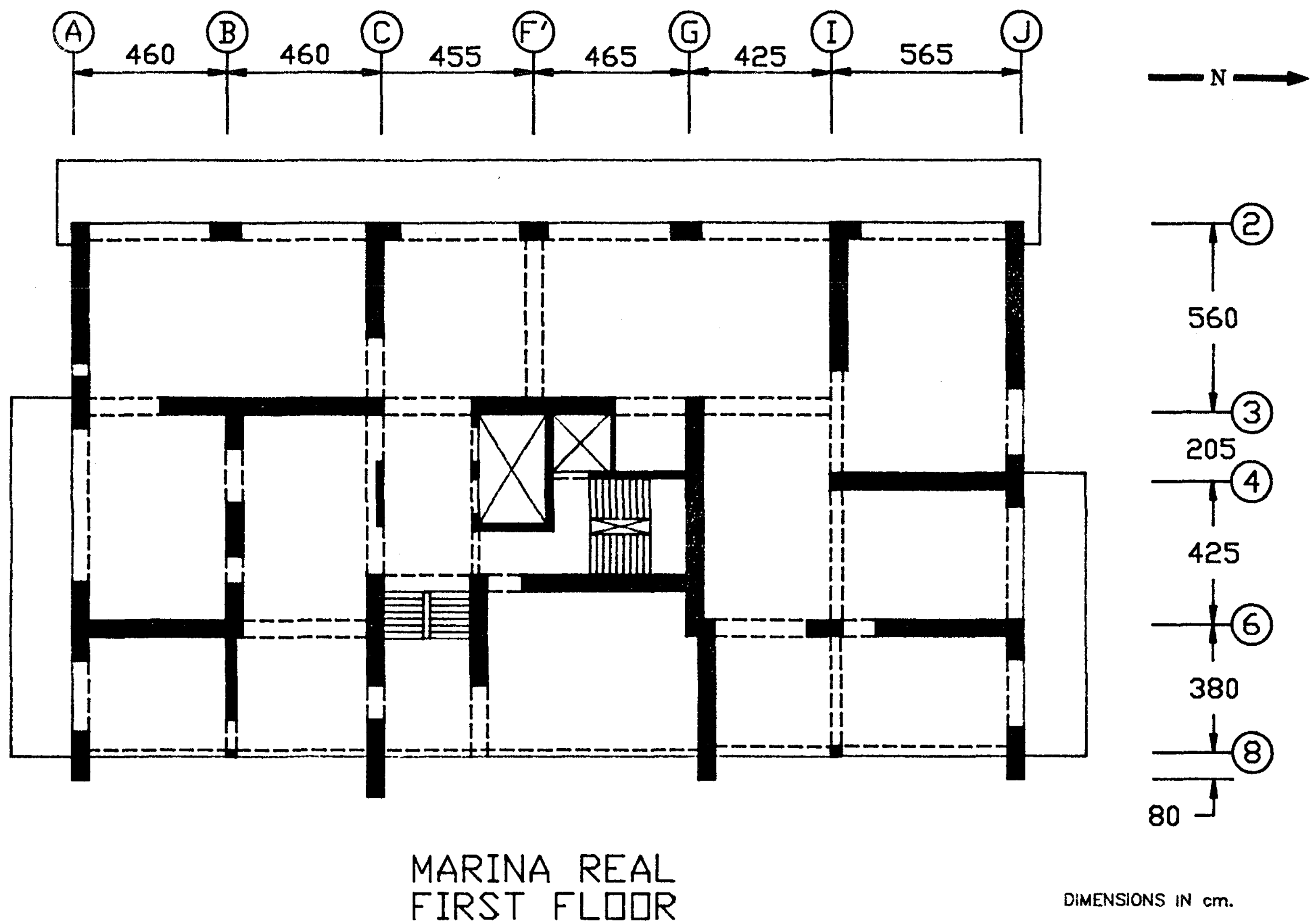


Fig. 7.6 Floor Plans - Marina Real

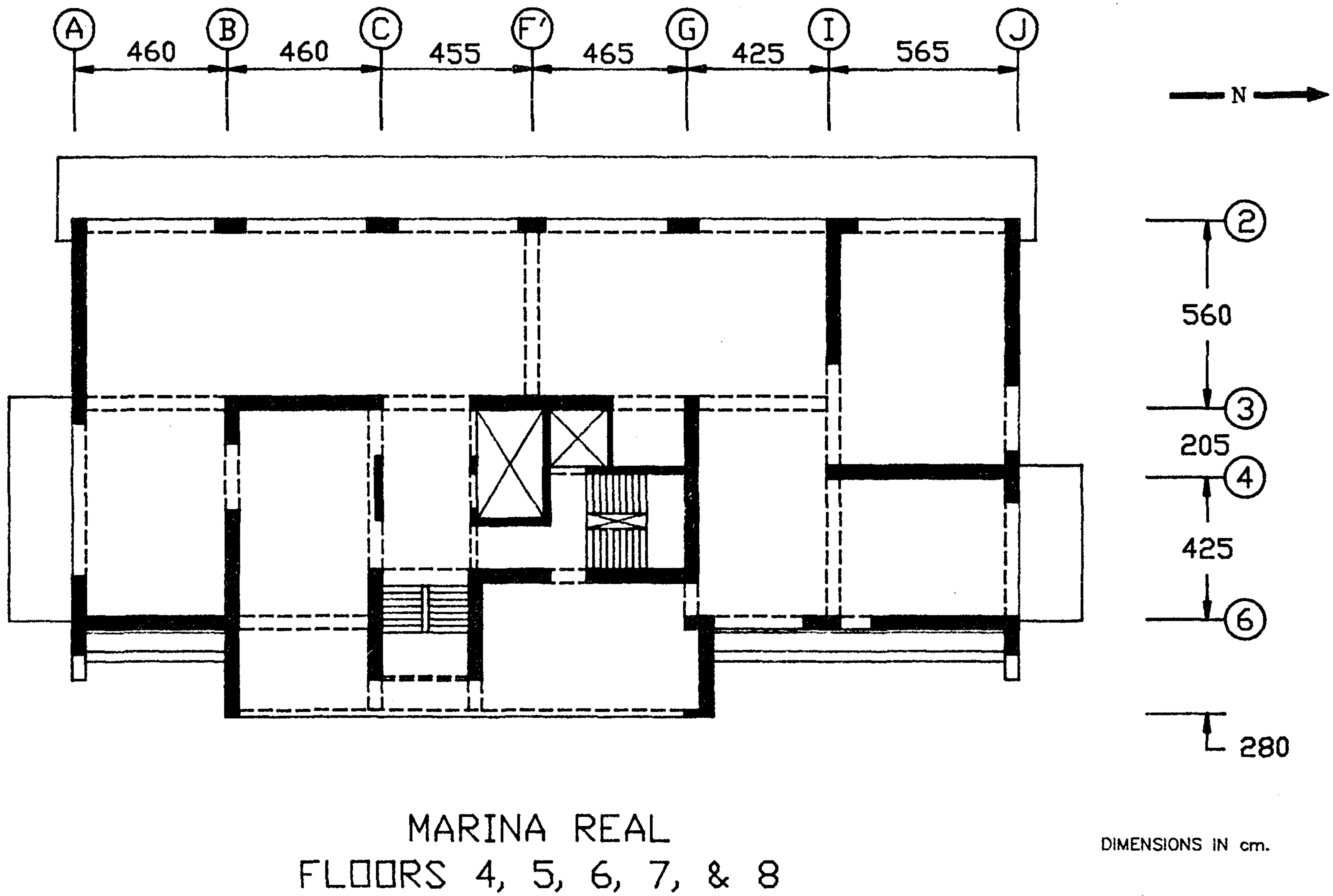


Fig. 7.6 (cont.) Floor Plans - Marina Real

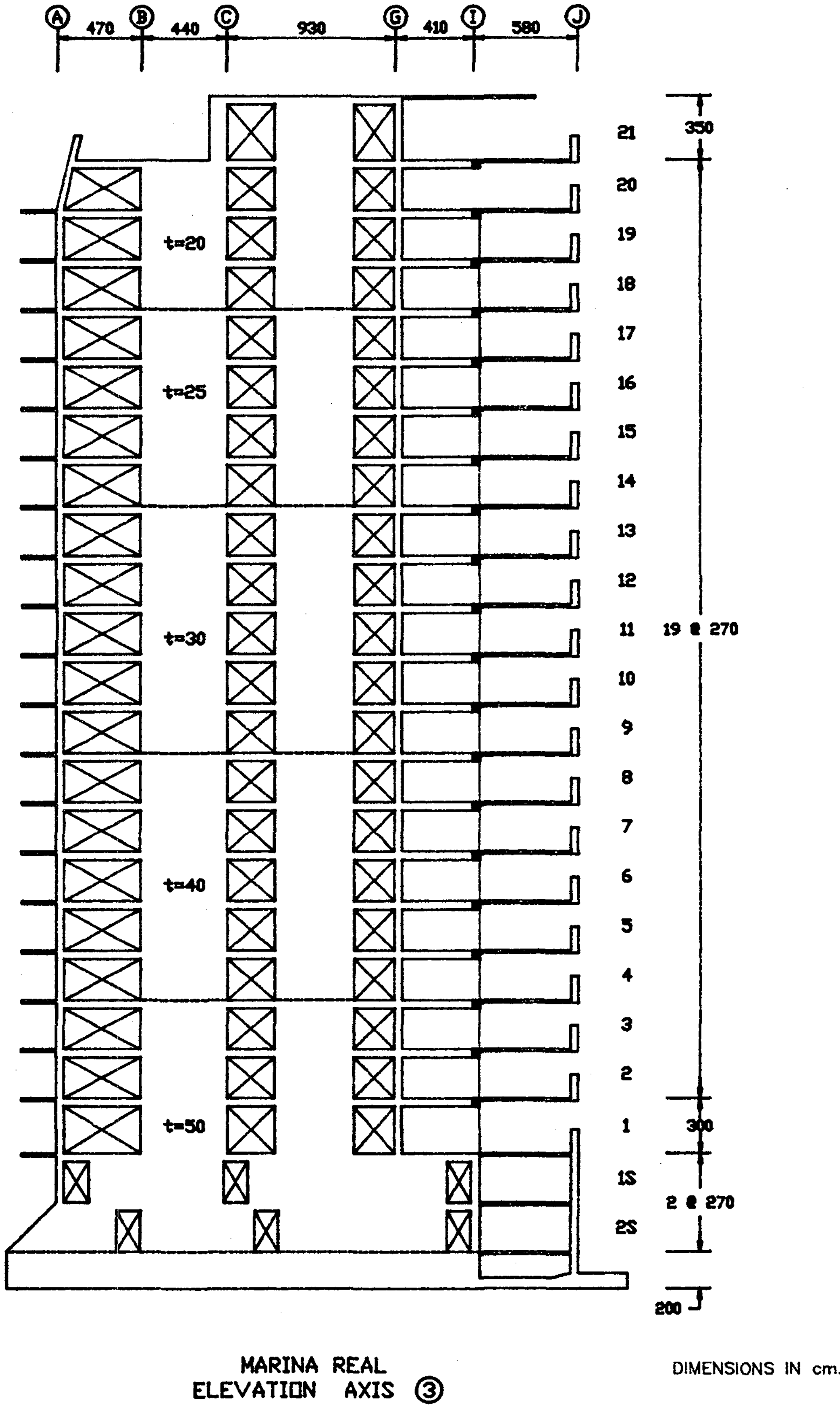


Fig. 7.7 Elevation - Marina Real

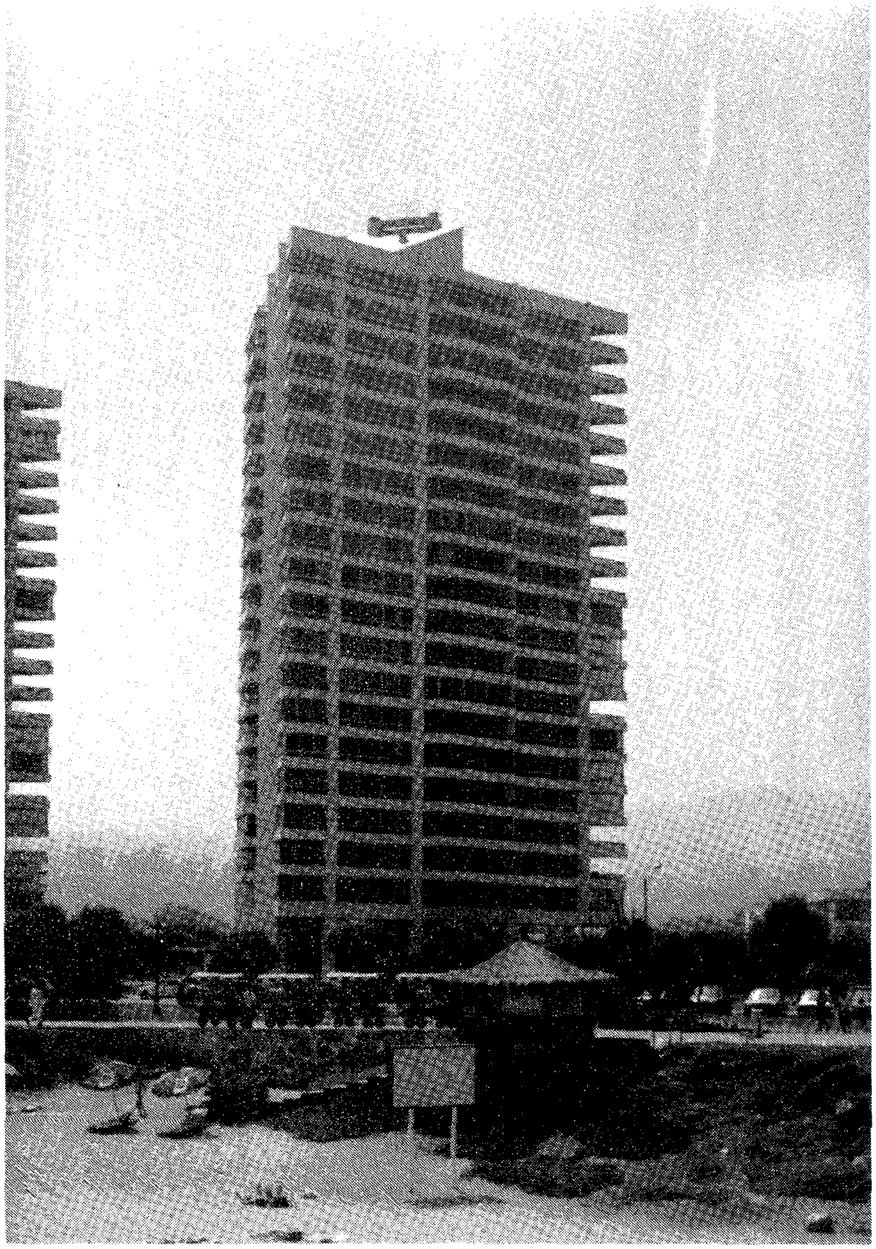
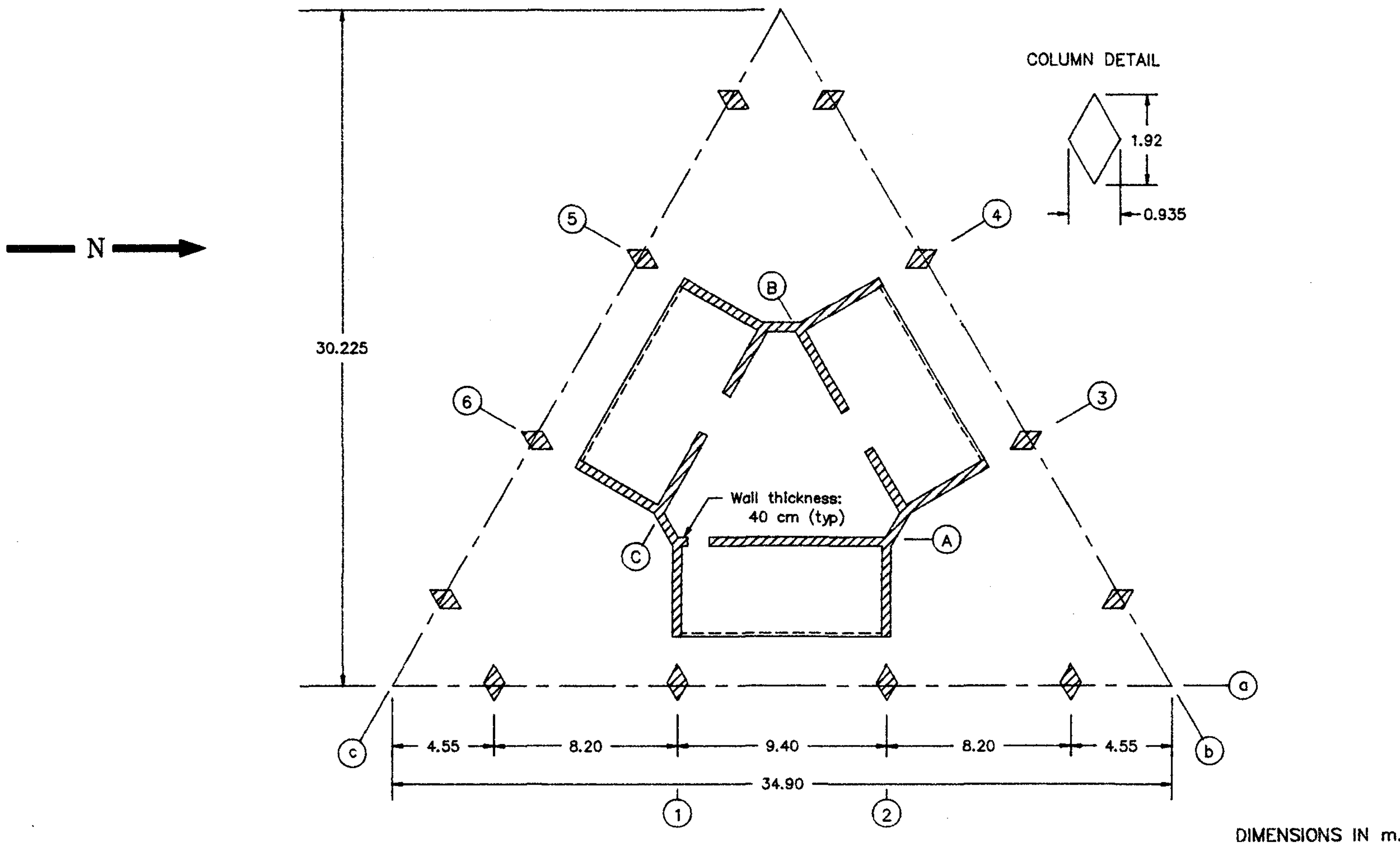


Fig. 7.8 Torres de Miramar



TORRES DE MIRAMAR
FIRST FLOOR

Fig. 7.9 Floor Plans - Torres de Miramar

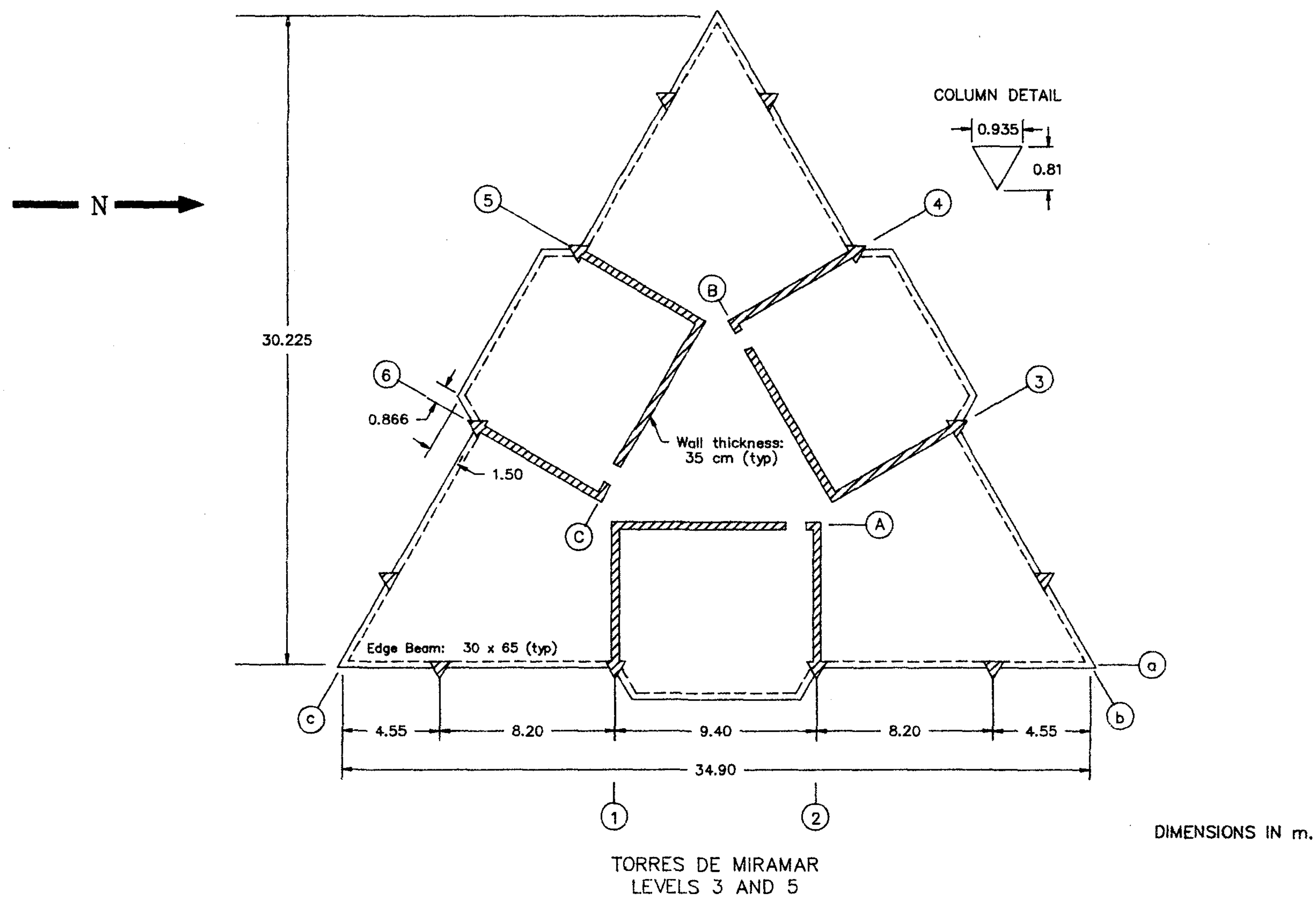


Fig. 7.9 (cont.) Floor Plans - Torres de Miramar

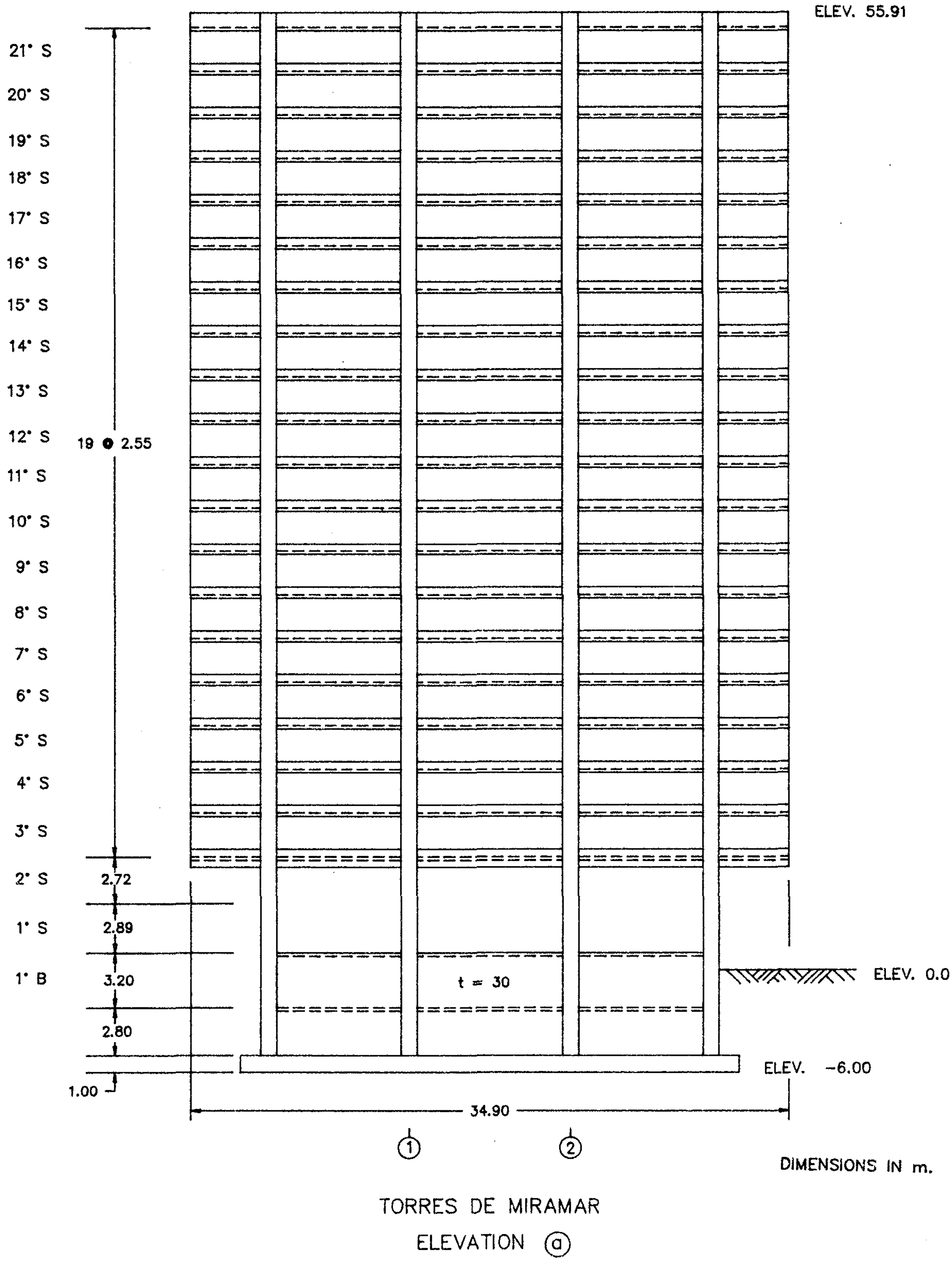


Fig. 7.10 Elevations - Torres de Miramar

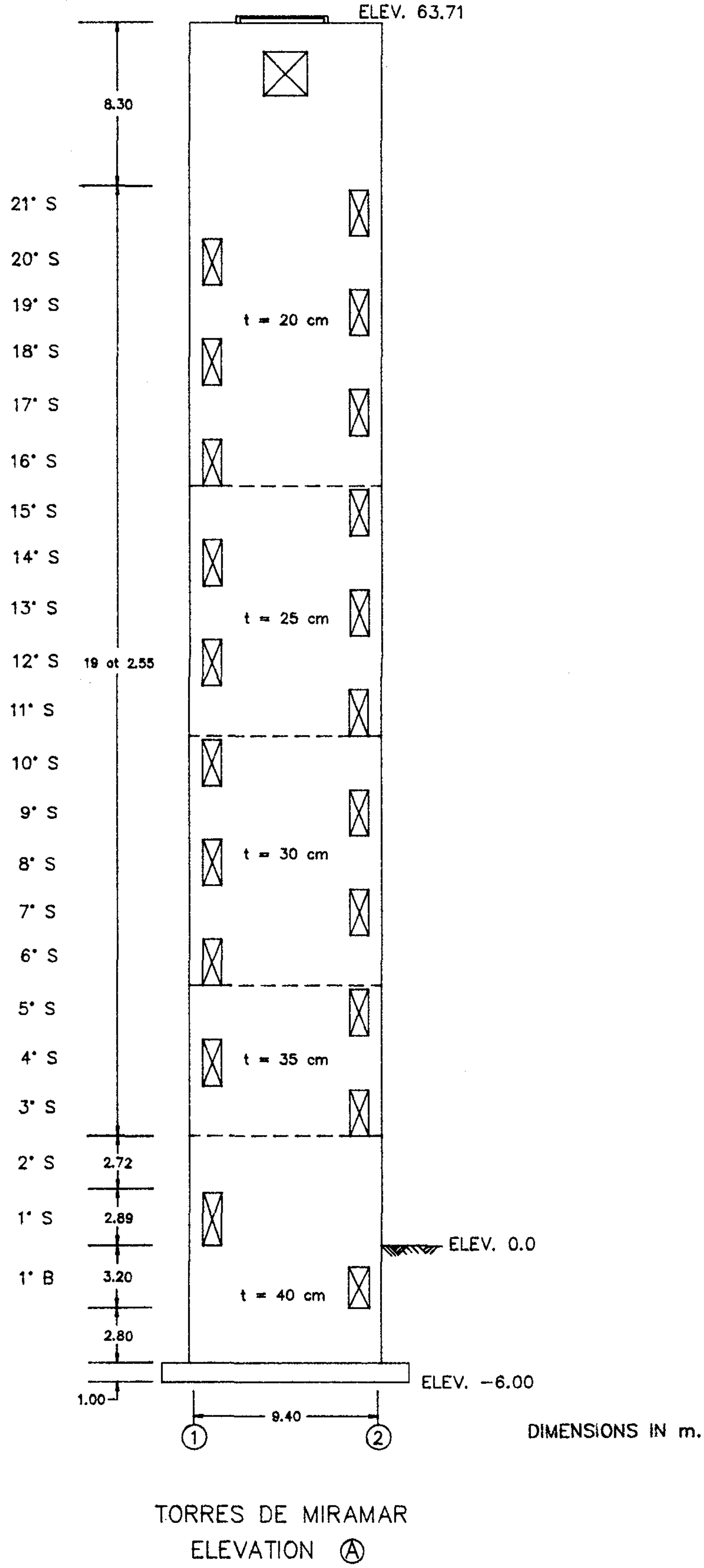
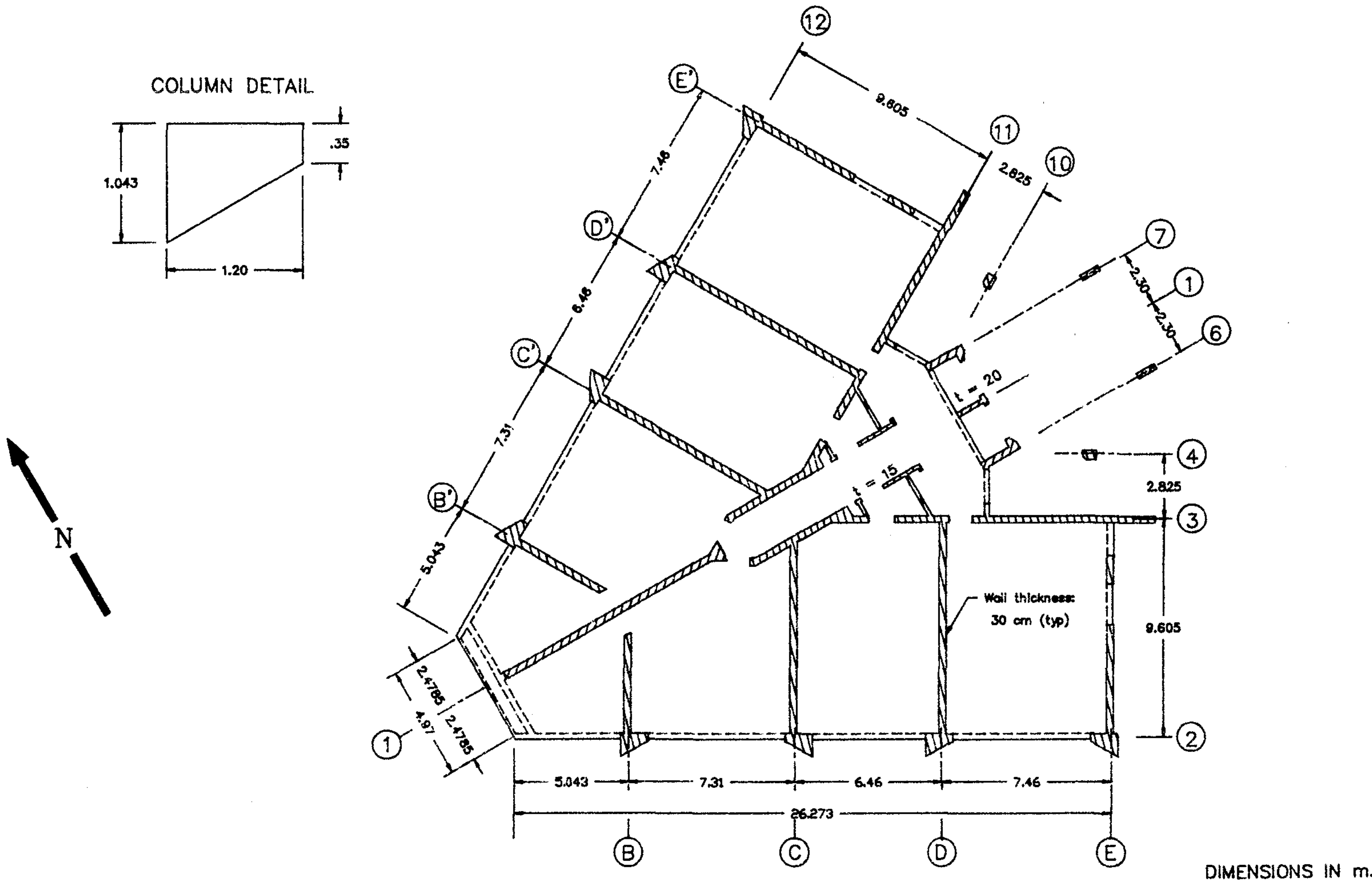


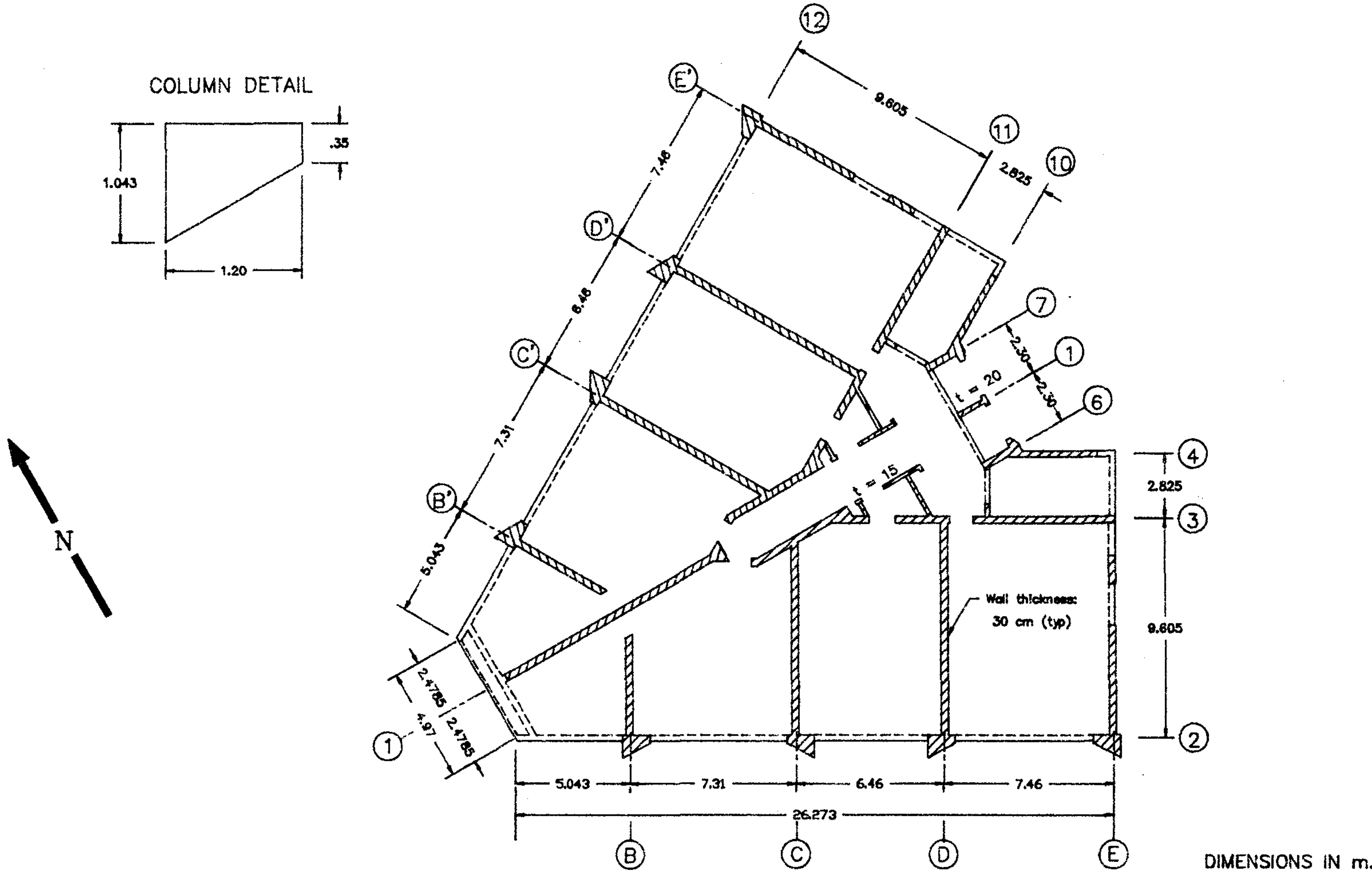
Fig. 7.10 (cont.) Elevations - Torres de Miramar

Fig. 7.11 Torres del Pacifico (Towers 2 & 3)



TORRES DEL PACIFICO
FIRST FLOOR

Fig. 7.12 Floor Plans - Torres del Pacifico



TORRES DEL PACIFICO
FLOORS 3, 4, & 5

Fig. 7.12 (cont.) Floor Plans - Torres del Pacifico

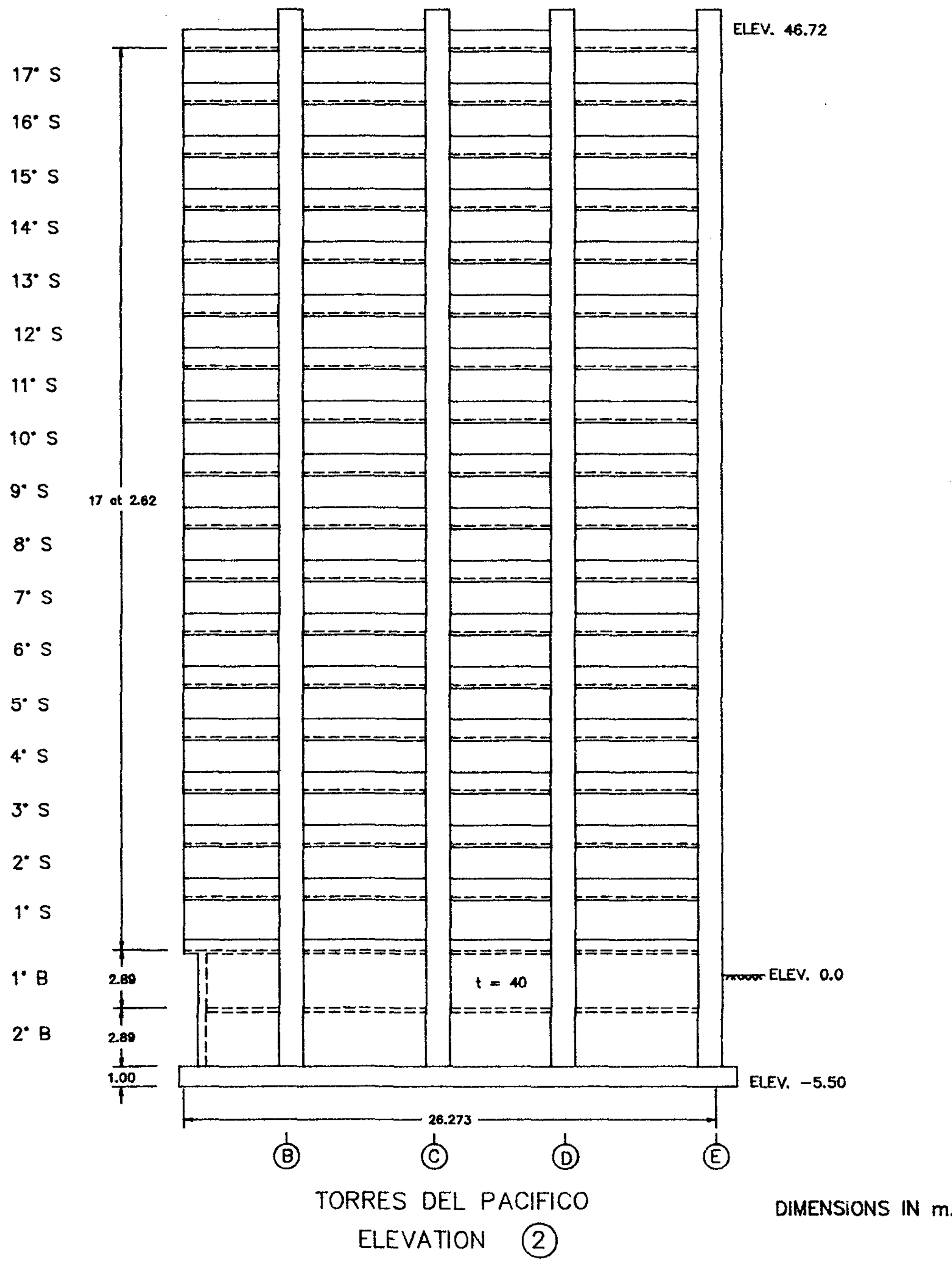


Fig. 7.13 Elevations - Torres del Pacifico

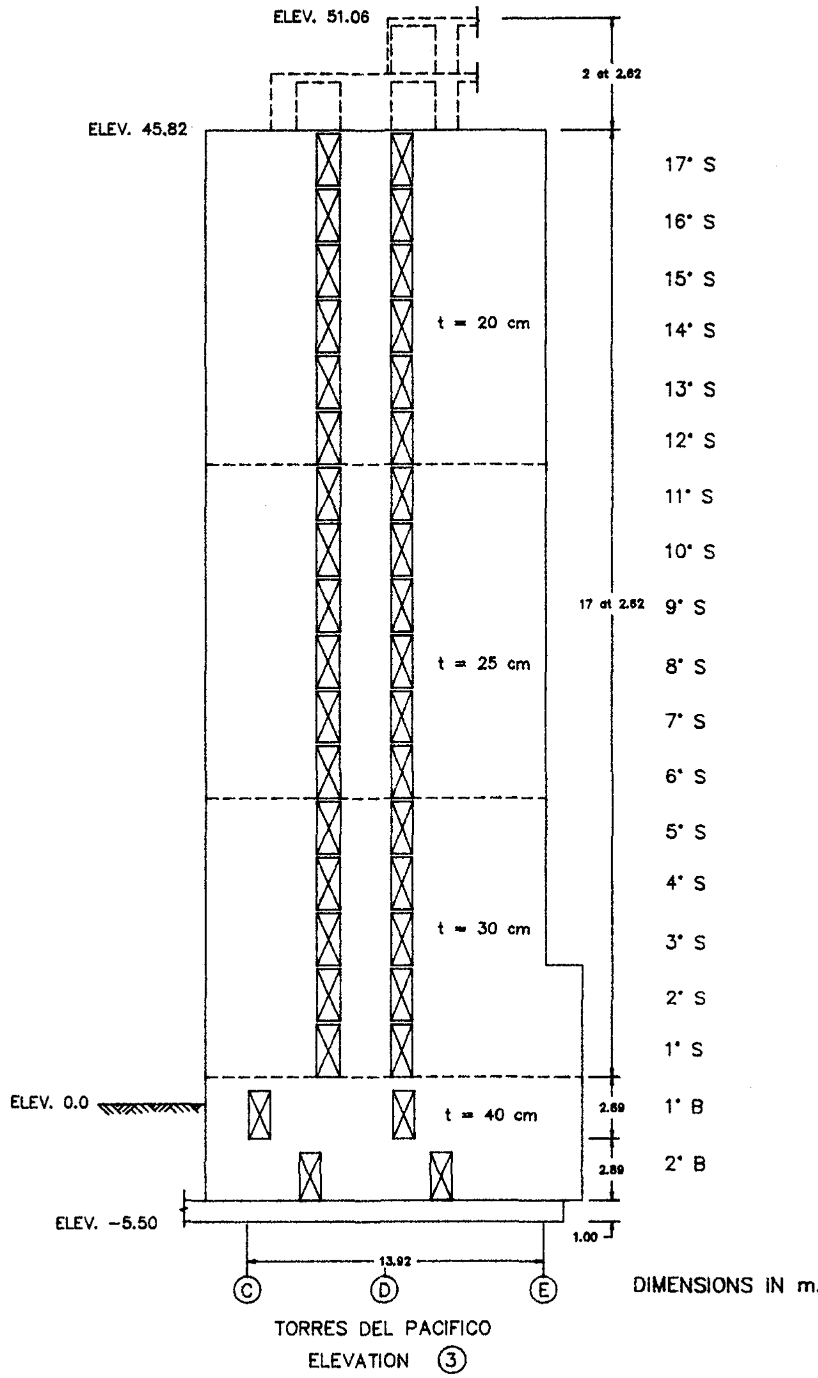


Fig. 7.13 (cont.) Elevations - Torres del Pacifico

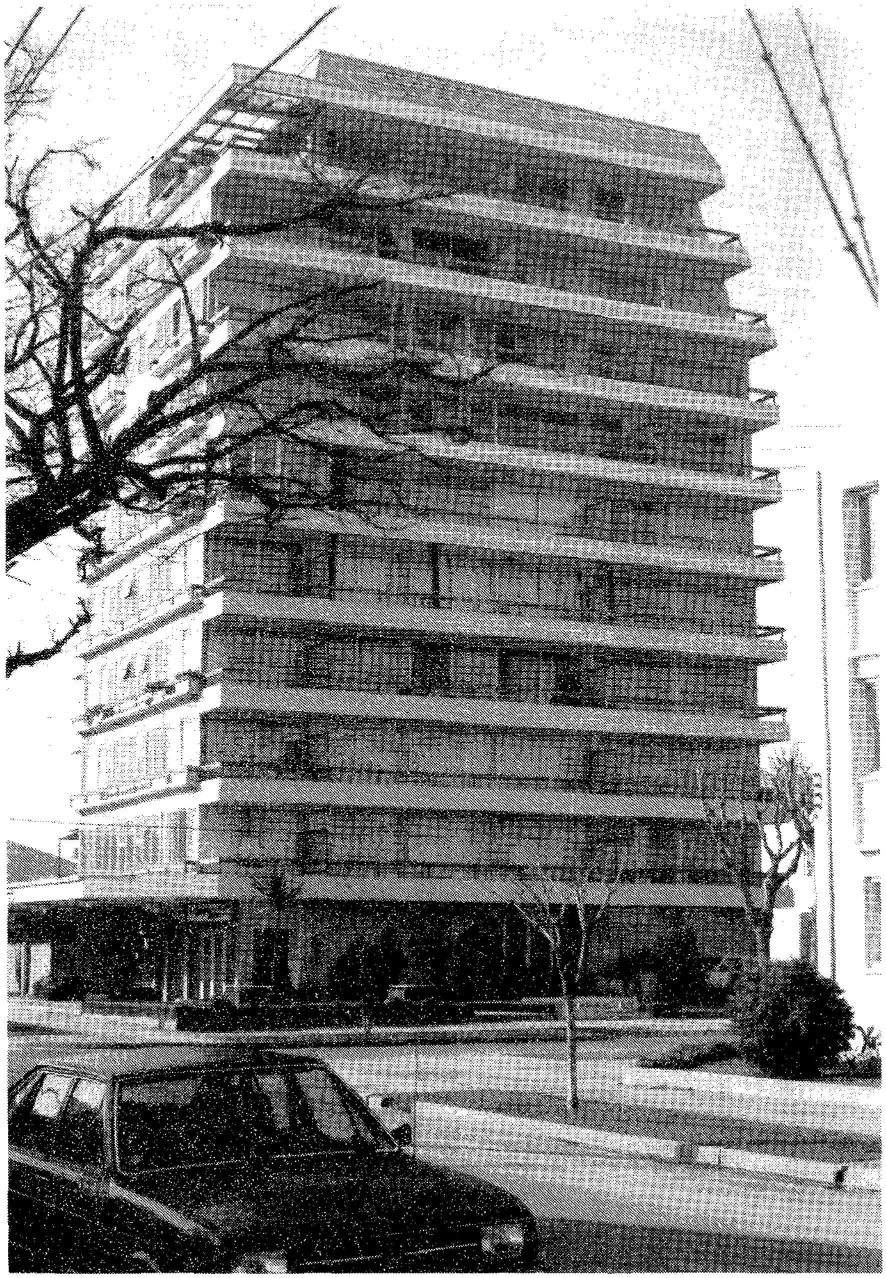
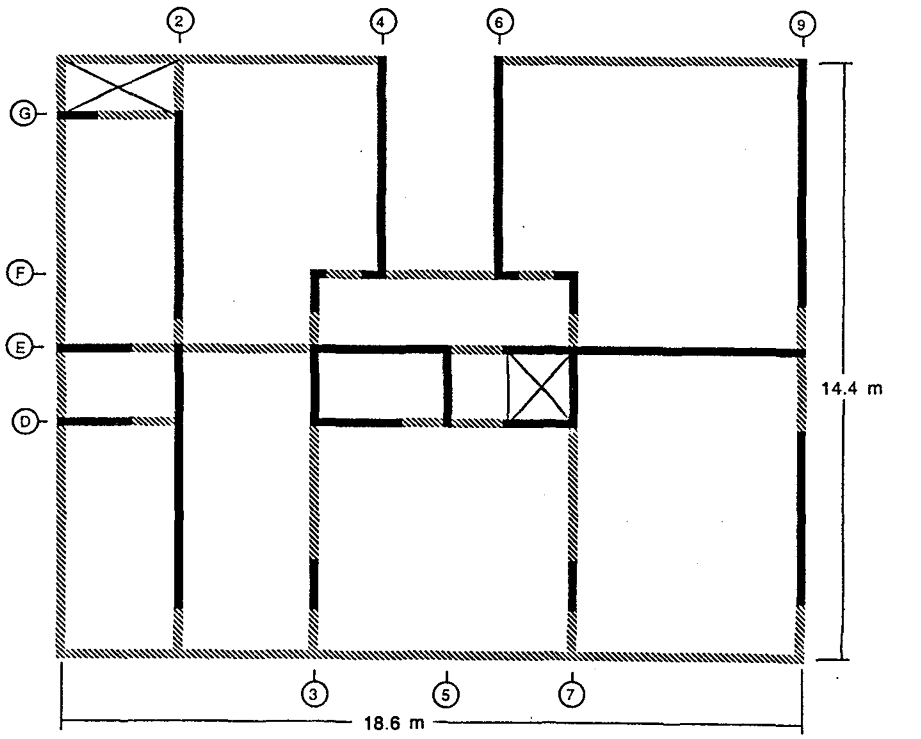


Fig. 7.14 Villa Real

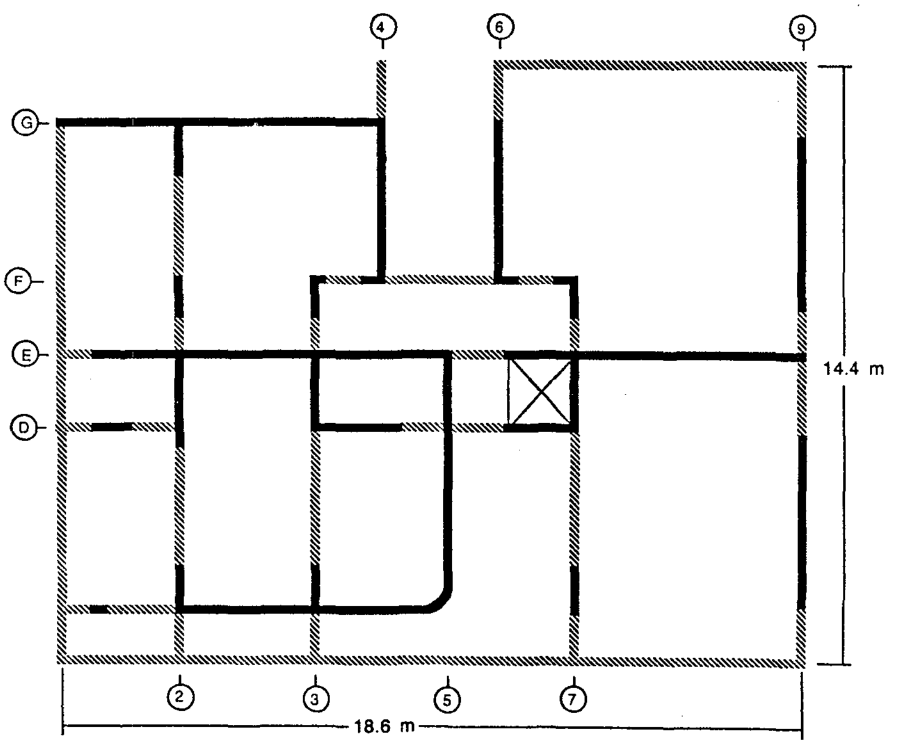


Second Story Plan

N

Indicates shear wall over story height

Indicates beam at second level



First Story Plan

Indicates shear wall over story height

Indicates beam at first level

Fig. 7.15 Floor Plans - Villa Real

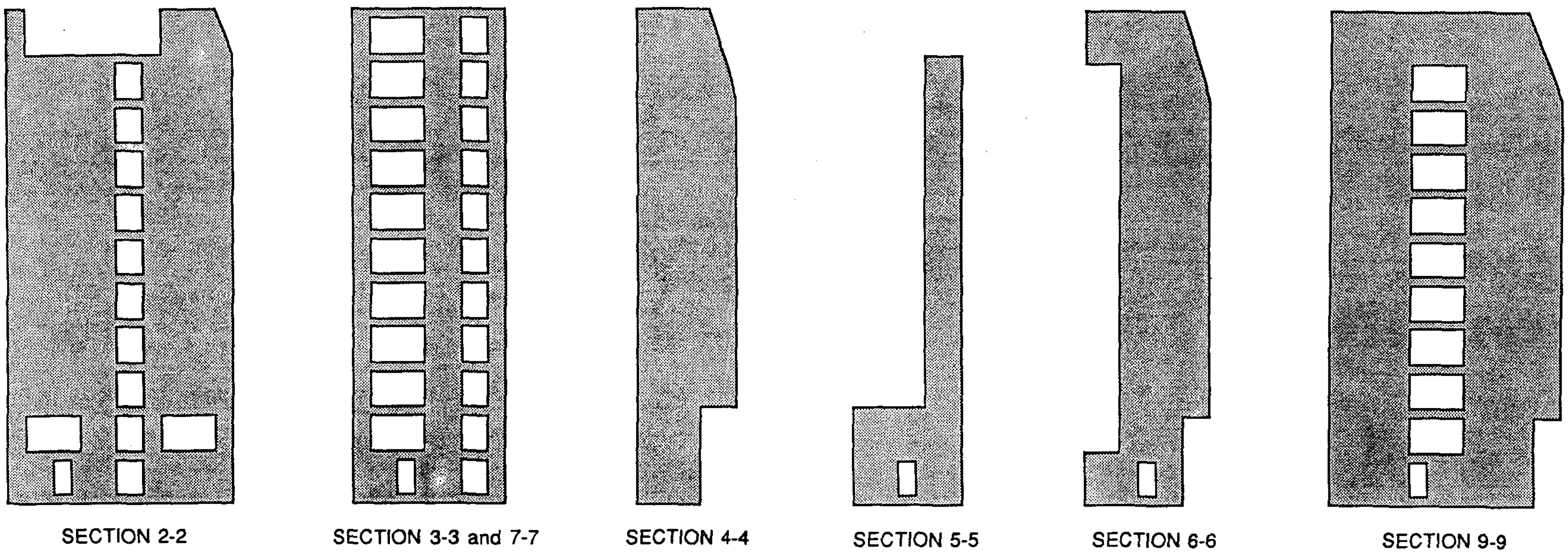


Fig. 7.16 Elevations - Villa Real

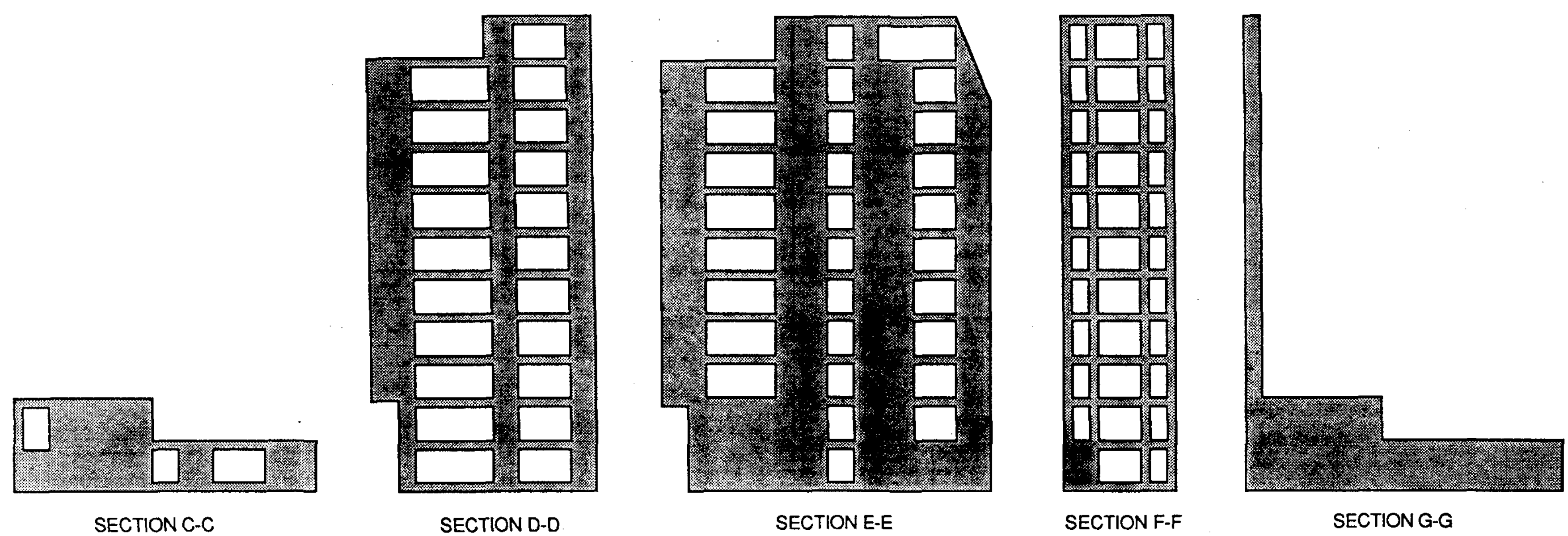


Fig. 7.16 (cont.) Elevations - Villa Real

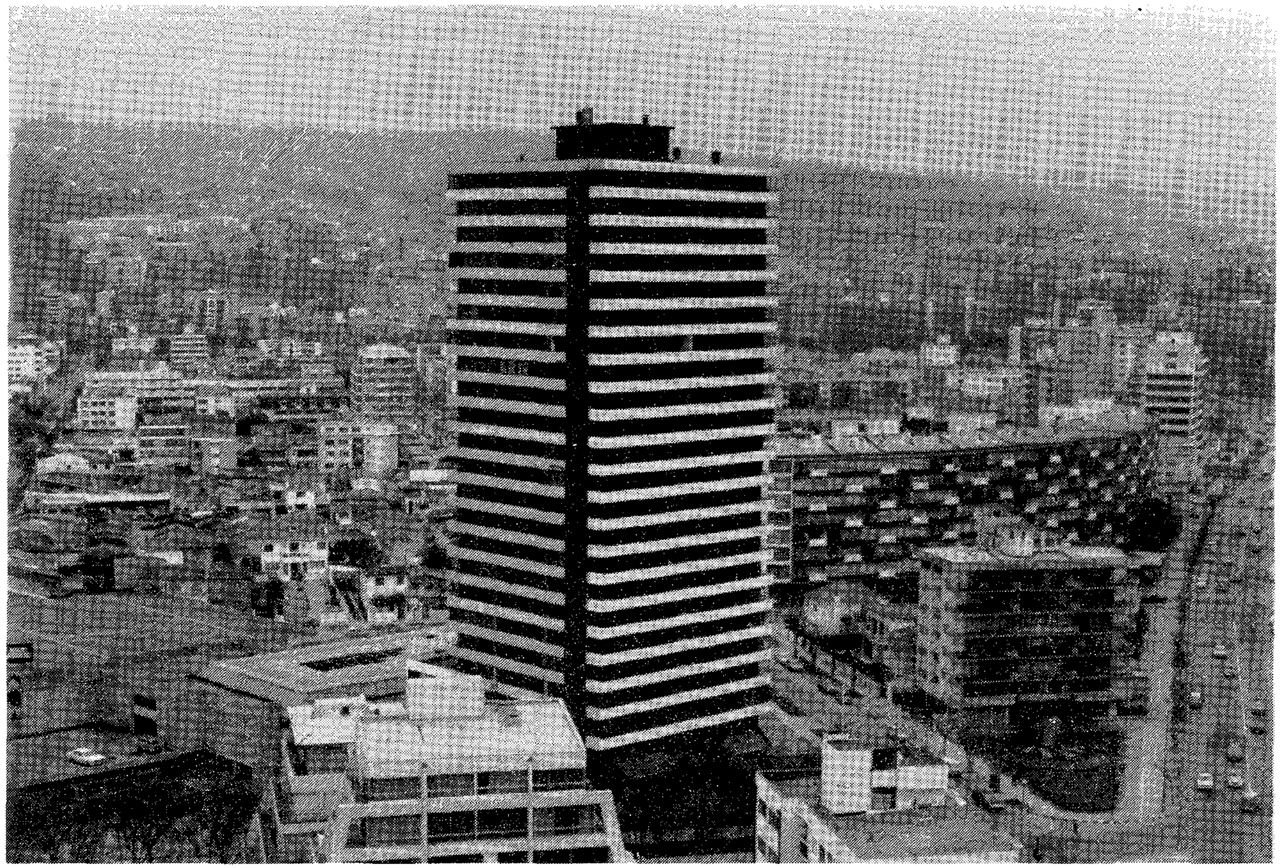
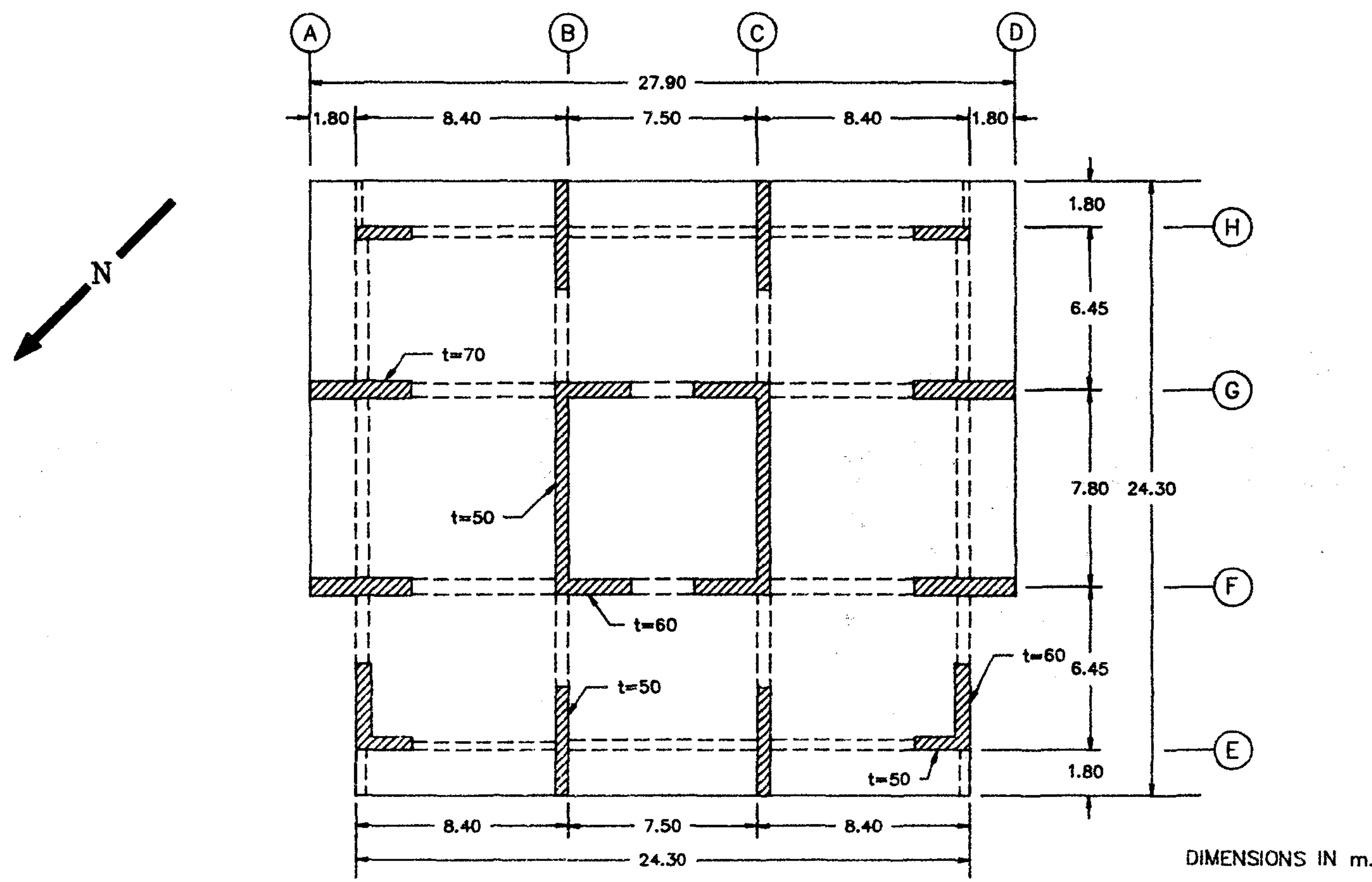


Fig. 7.17 Torres del Sol



TORRES DEL SOL
FIRST FLOOR

Fig. 7.18 Floor Plans - Torres del Sol

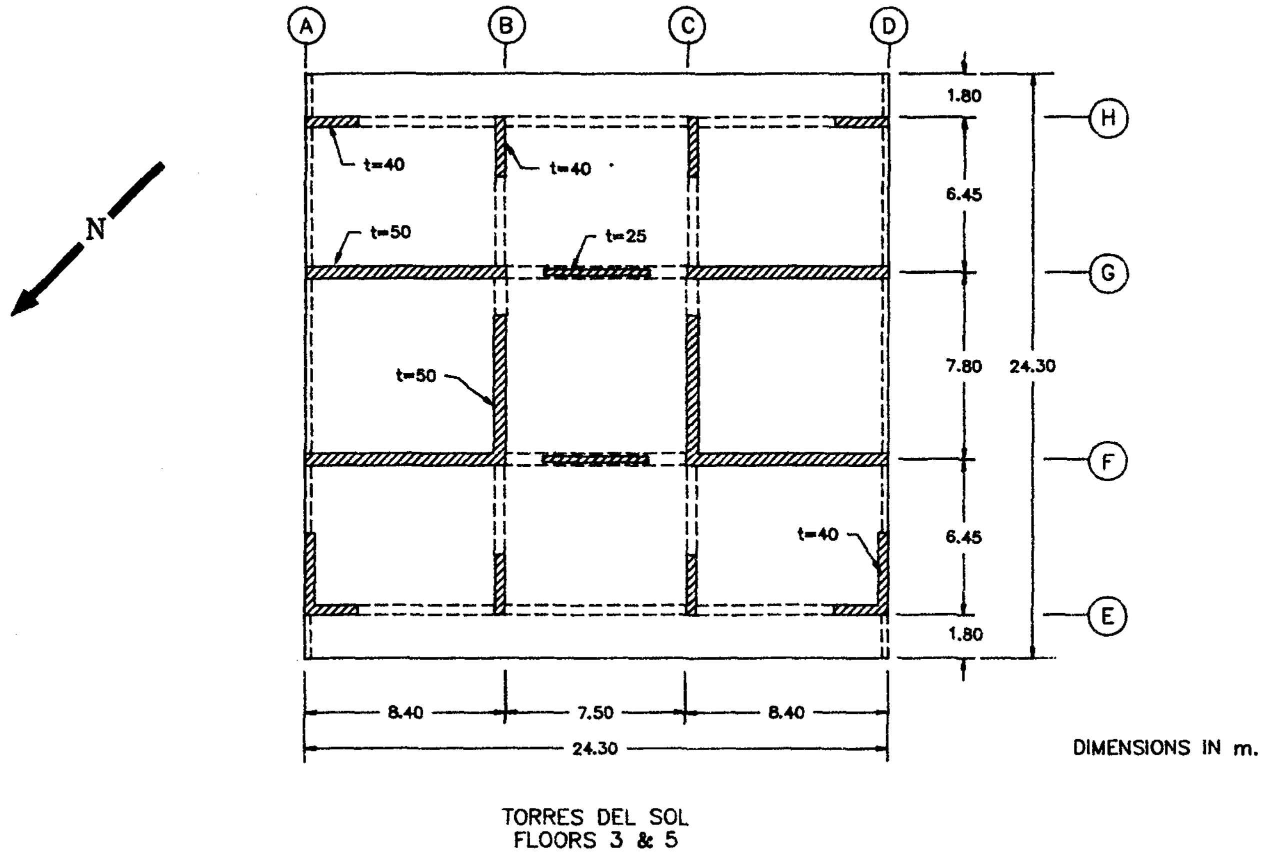


Fig. 7.18 (cont.) Floor Plans - Torres del Sol

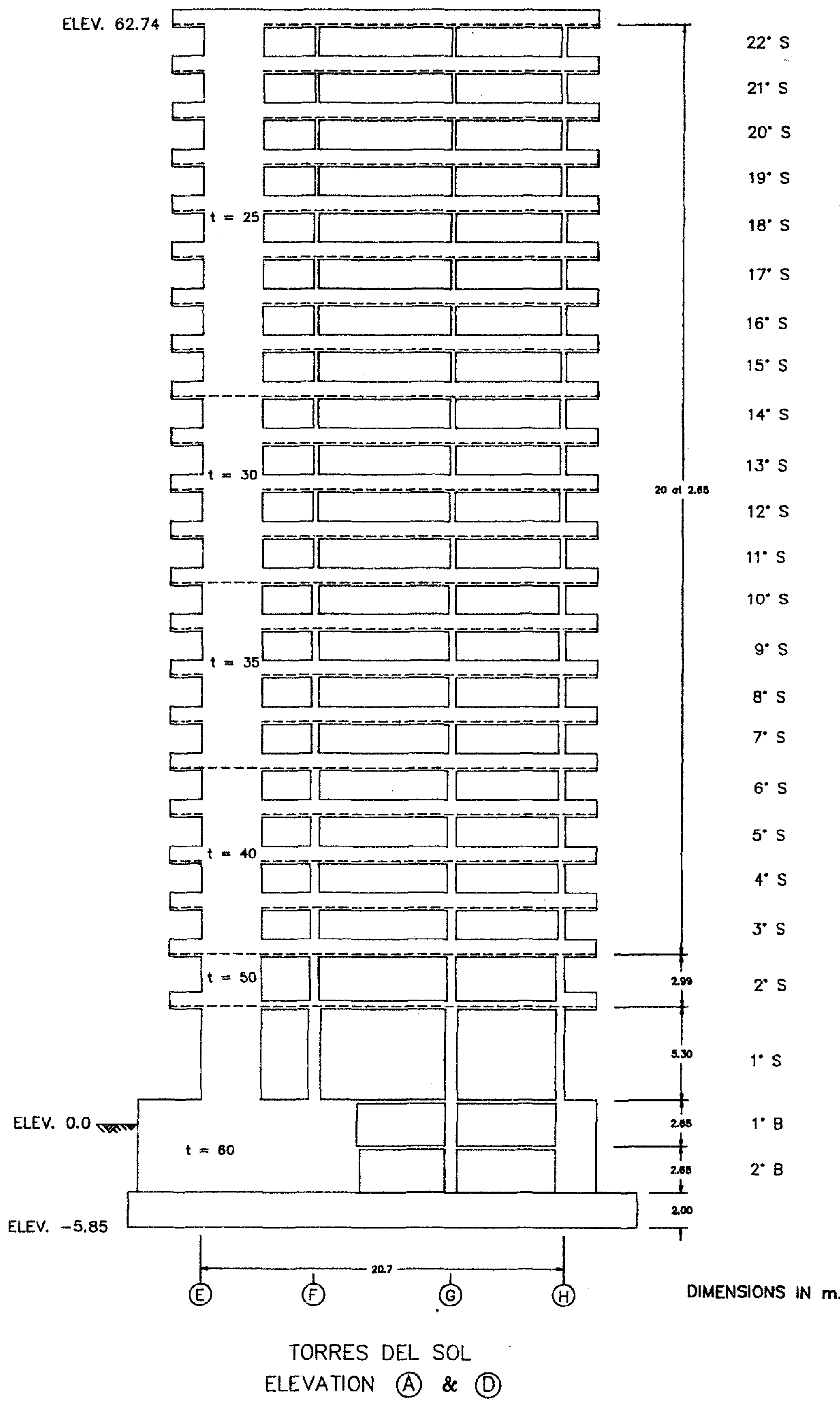


Fig. 7.19 Elevations - Torres del Sol

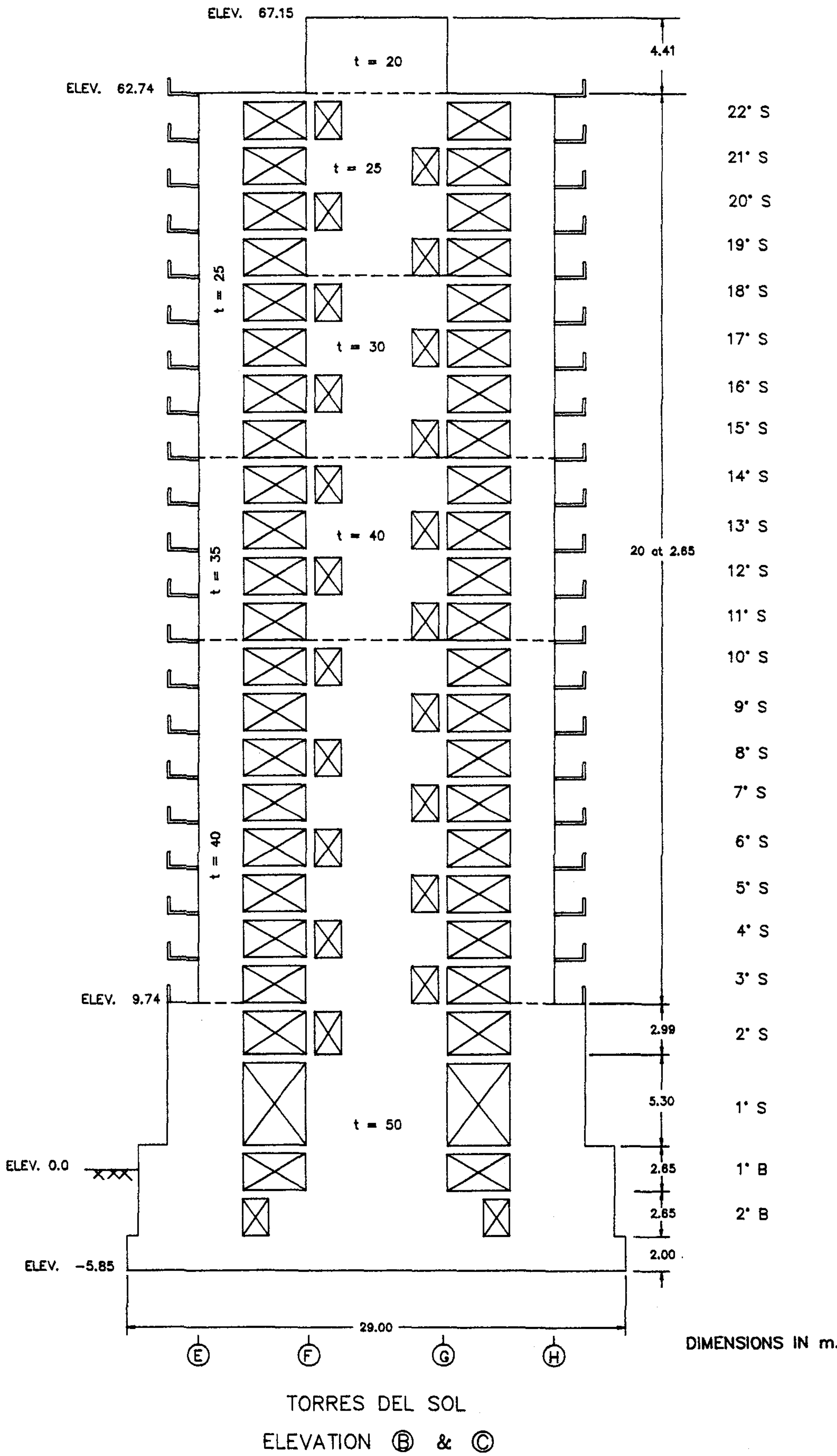


Fig. 7.19 (cont.) Elevations - Torres del Sol

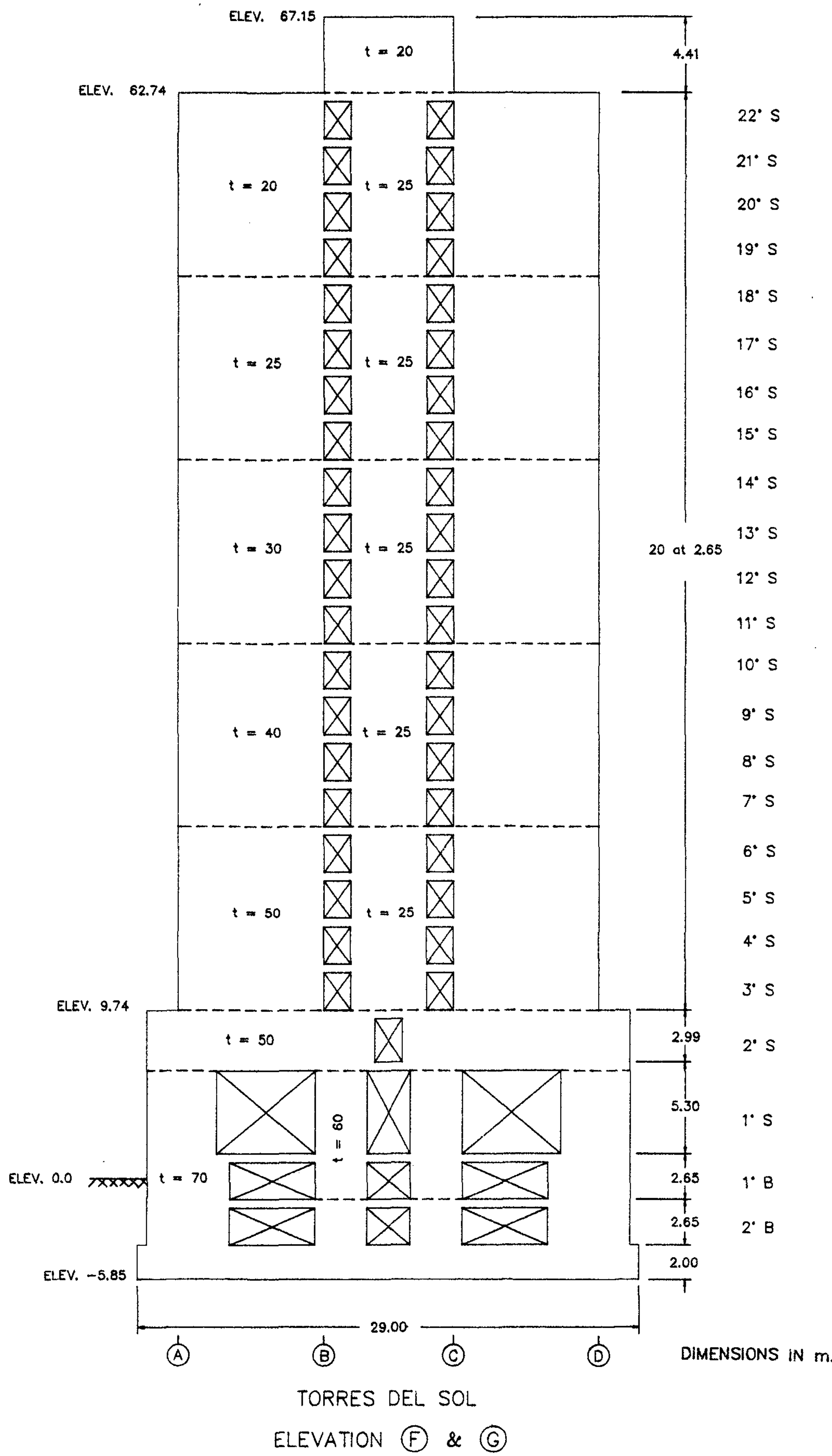


Fig. 7.19 (cont.) Elevations - Torres del Sol

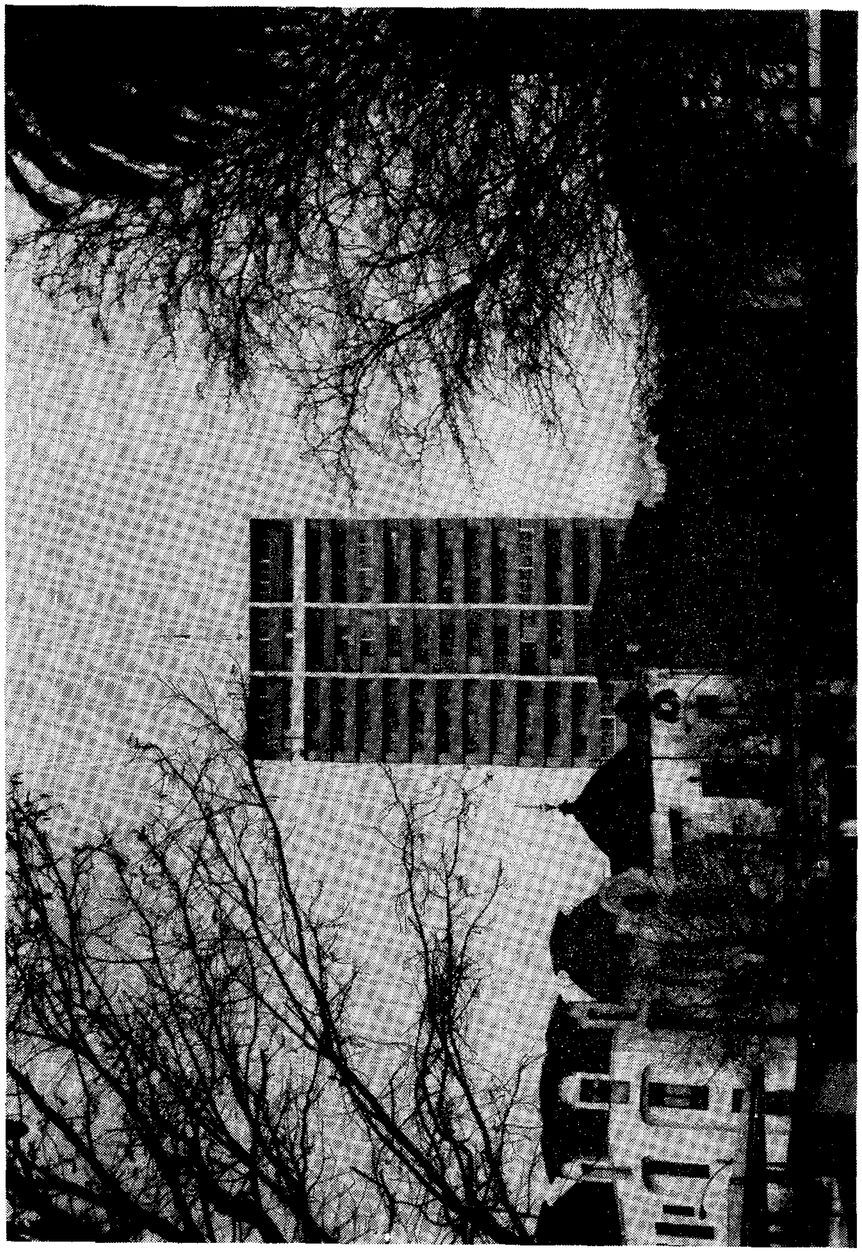


Fig. 7.20 Torre del Almendral

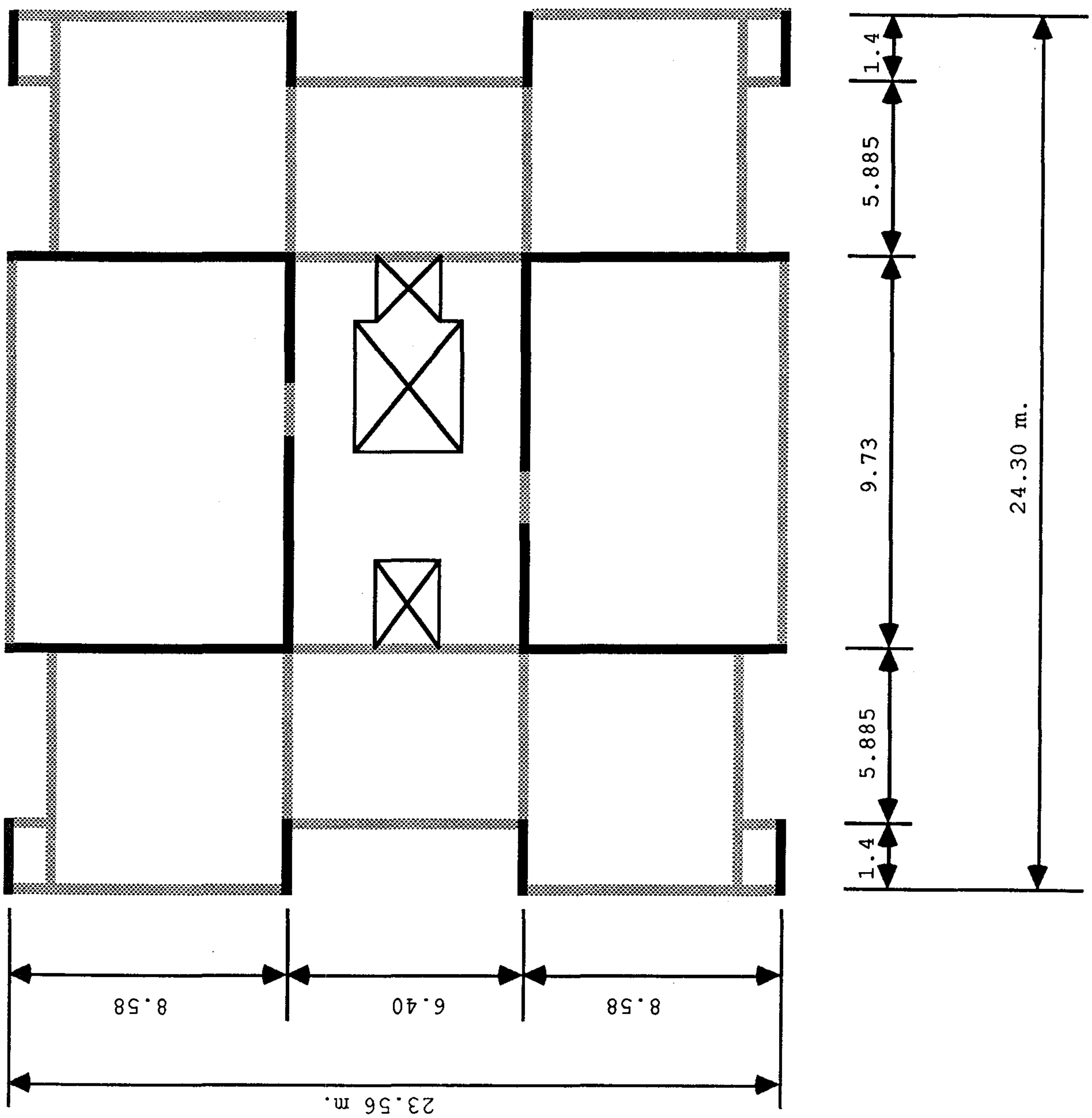


Fig. 7.21 Floor Plan - Torre del Almendral

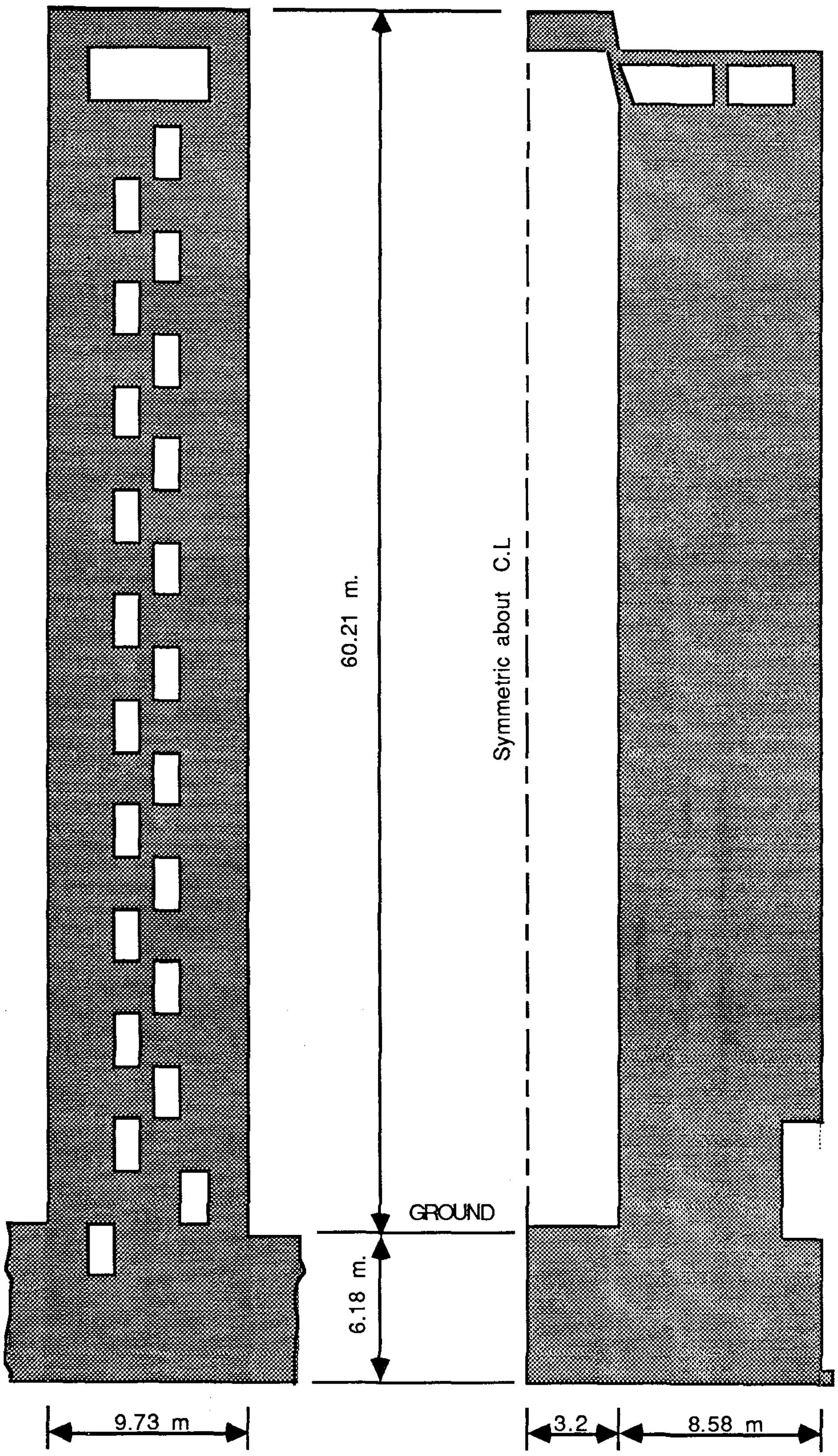


Fig. 7.22 Elevations - Torre del Almendral

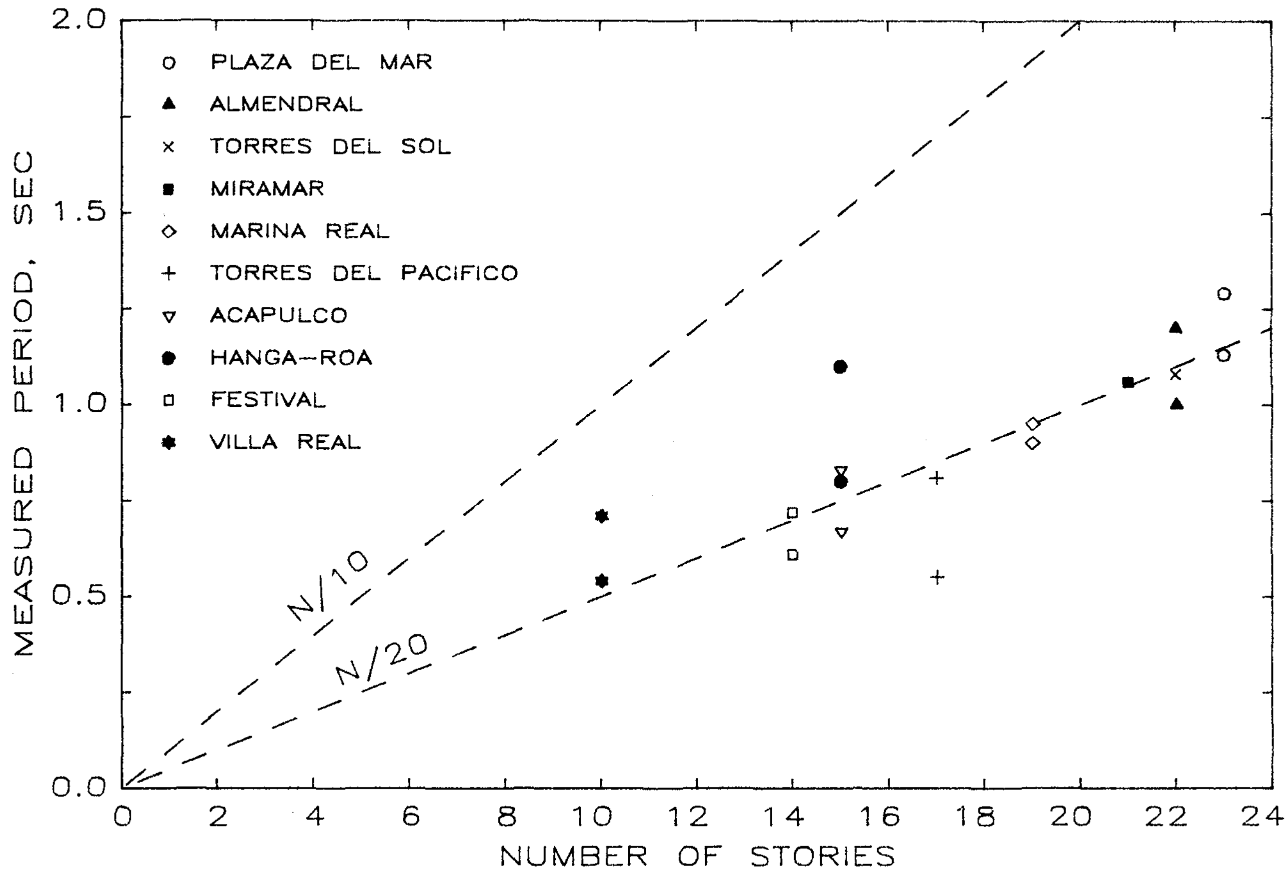


Fig. 7.23 Measured Periods of Buildings [15]

APPENDIX A
GEOTECHNICAL CHARACTERISTICS OF THE SOIL IN VIÑA DEL MAR

Detailed geotechnical investigations are generally not conducted at the site of a new buildings in Viña del Mar because the soil is considered to be a good foundation material and fairly uniform throughout the downtown area. Available data indicate that the following information is obtained when a geotechnical investigation is required:

Bore holes - average depth between 10 and 20 m.
Standard penetration tests - average depth between 8 and 20 m.
Dynamic cone tests - average depth between 8 and 12 m.
Grain size analysis.

The results from bore holes and standard penetration tests conducted at eight sites in Viña del Mar are summarized in this Appendix. More detailed descriptions of the soil using data from 46 borings in Viña del Mar are presented in Reference 31.

The locations of the eight bore holes in Viña del Mar are shown in Fig. A.1. Data obtained at these locations are presented in Fig. A.2 - A.9.

Test results were available for 4 buildings that were described in Chapters 6 and 7: Torres del Pacifico (boring #1), Torres de Miramar (boring #2), Hanga-Roa (boring #3), and Marina Real (boring #5). All four buildings are located on Avenida San Martin near the beach. The results of the standard penetration tests indicated the presence of sand with medium to dense relative densities for depths below 2 m [51]. The elevation of the water table varied significantly at the four locations, from 1.9 to 8.5 m below grade. However, no information was available about tidal fluctuations at the time of the borings.

The soil profiles were reasonably consistent at the four sites with thin layers of artificial fill and fine sand near the surface. Layers of coarse to medium sand and fine gravel extended to depths of approximately 20 m. The deepest boring, #1 at Torres del Pacifico, identified marine deposits (shells in the sand layers) below 32 m.

The results of the standard penetration tests performed at locations further inland (#4, #6, #7, and #8) also indicated sand with medium to dense relative densities for depths below 2 m. The soil profiles identified a thin layer of artificial fill at the surface and layers of medium and coarse sand extending to depths of approximately 10 m. A higher proportion of silt was observed in the soil samples obtained at the inland locations compared with the locations near the beach.

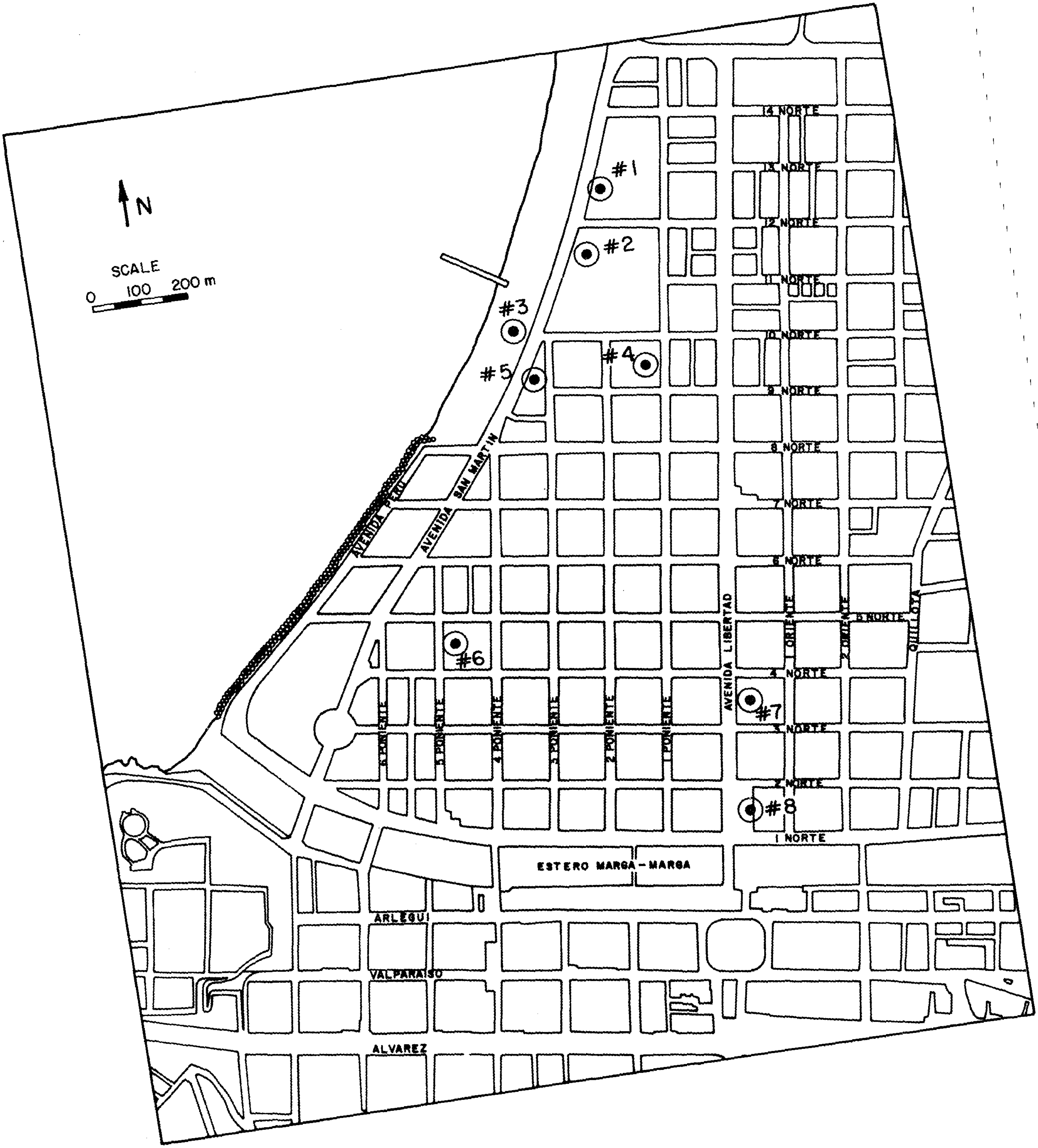


Fig. A.1 Locations of Soil Borings

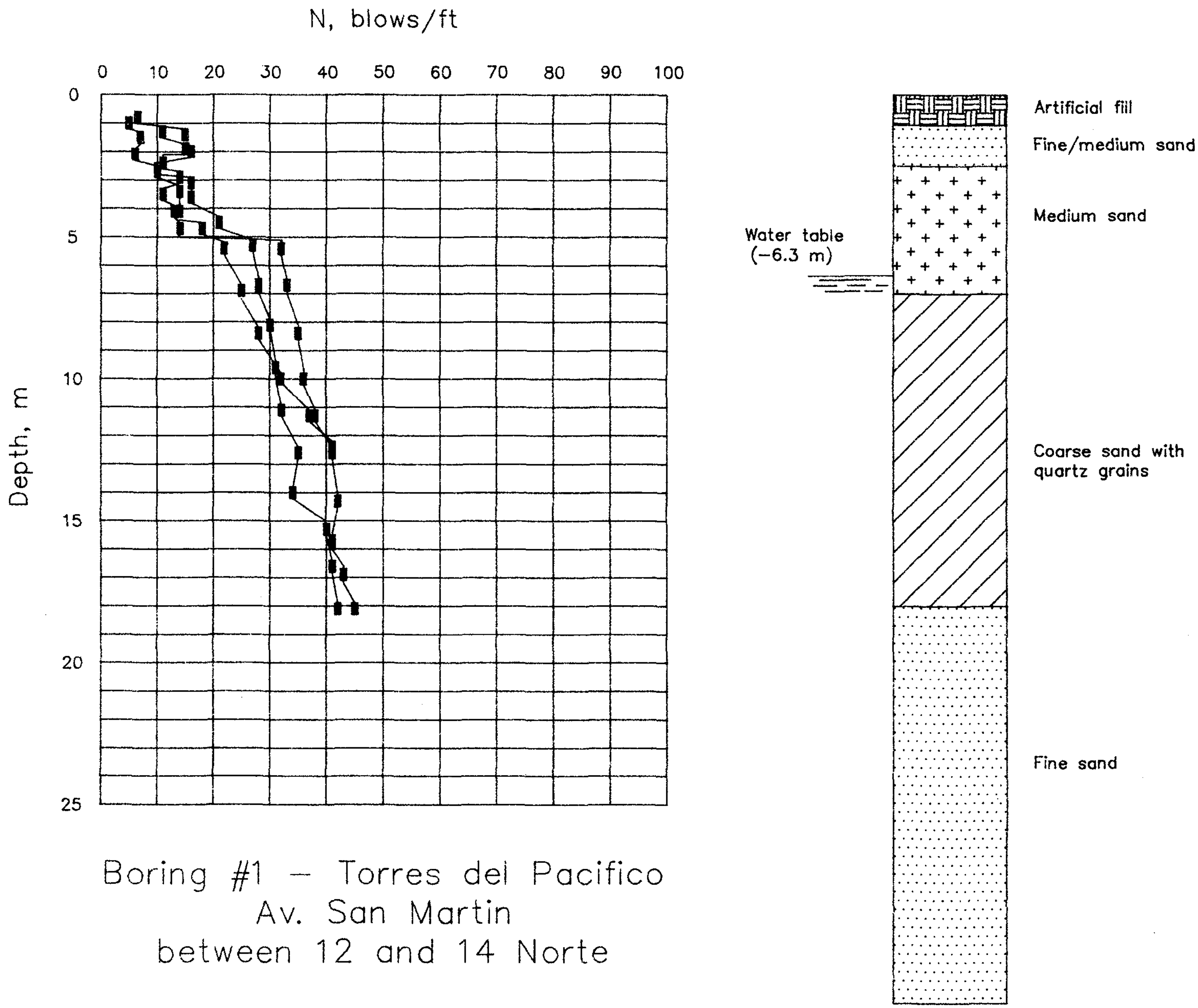


Fig. A.2 Soil Profile from Boring #1 [31]

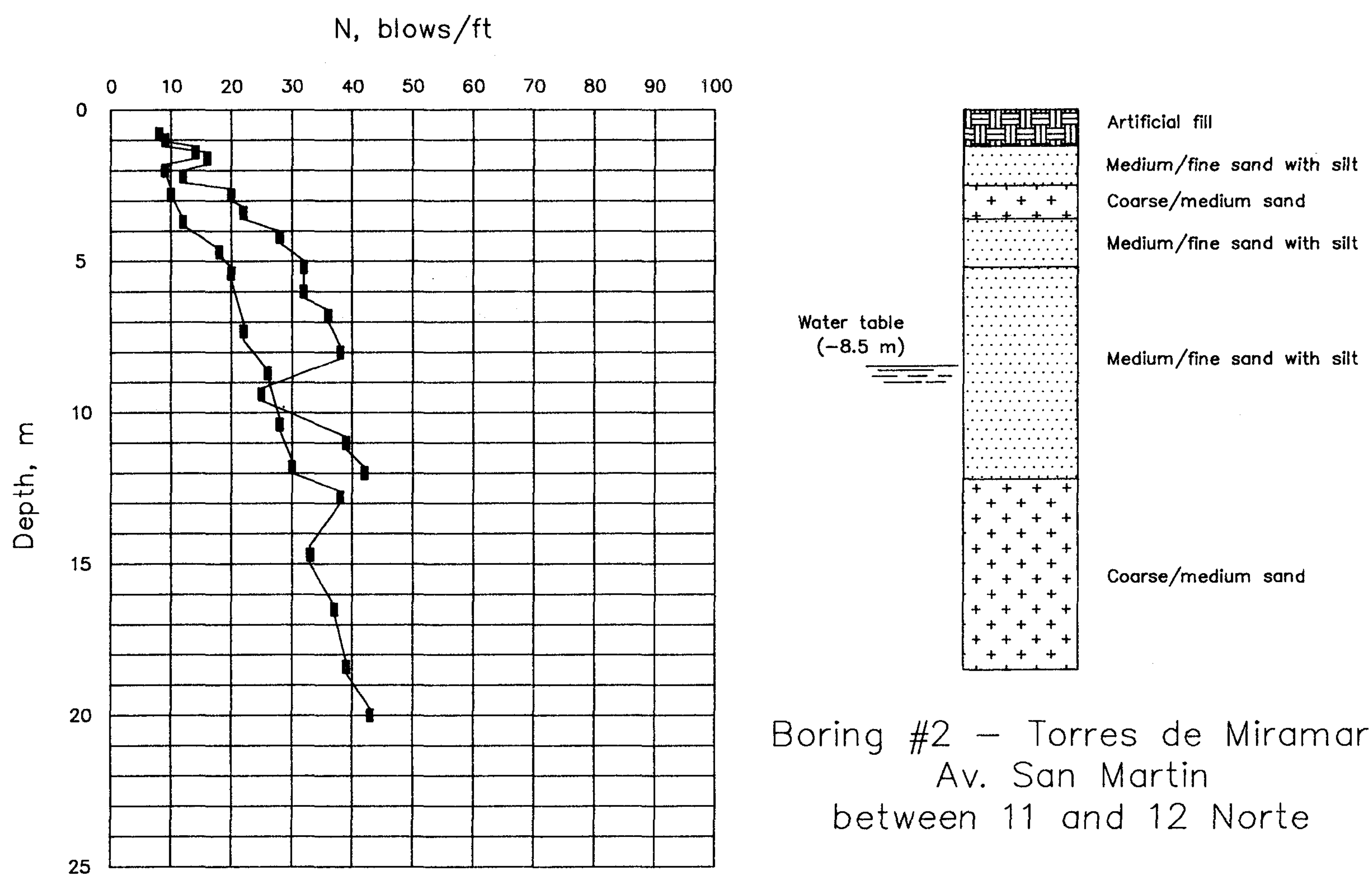


Fig. A.3 Soil Profile from Boring #2 [31]

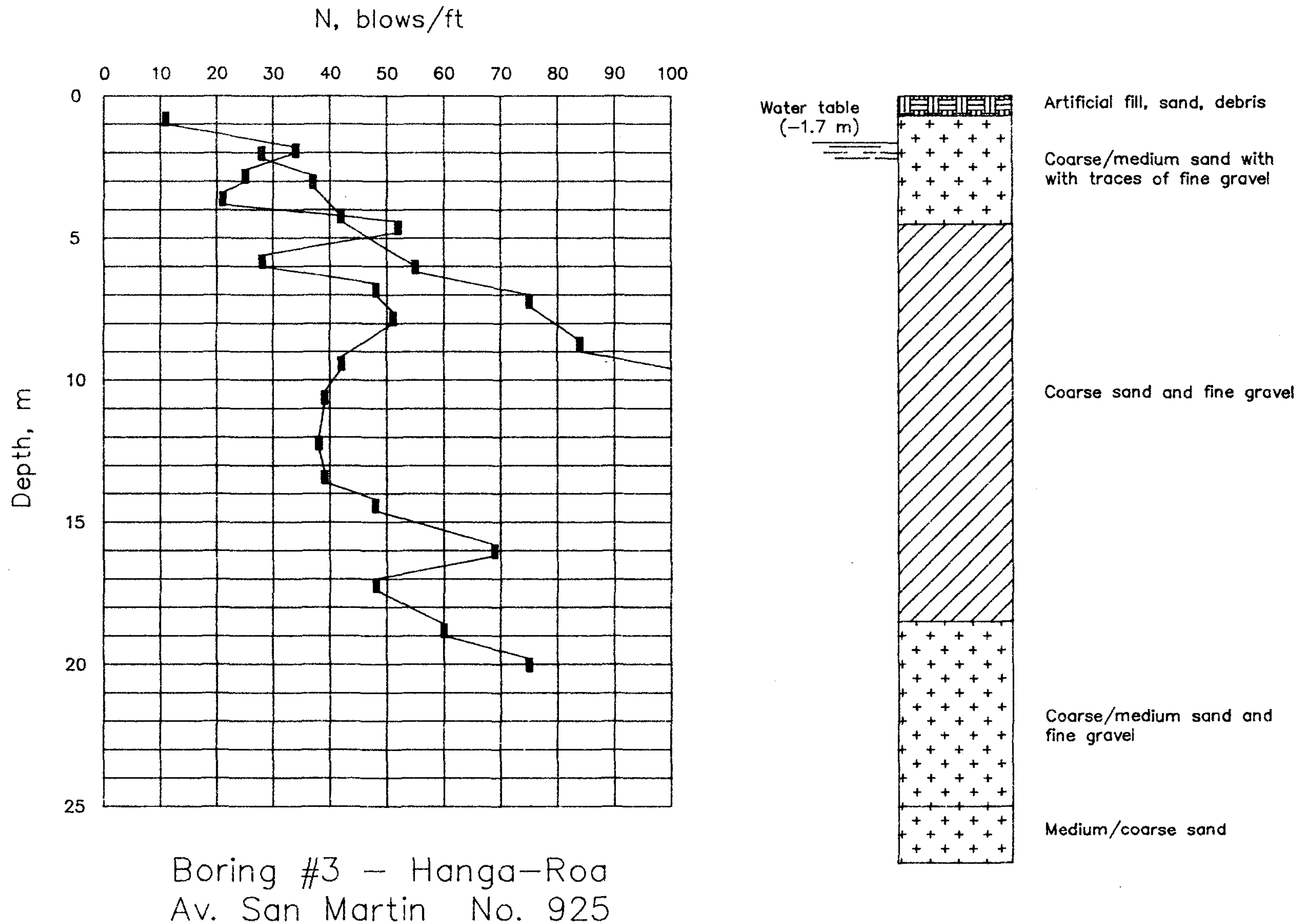


Fig. A.4 Soil Profile from Boring #3 [31]

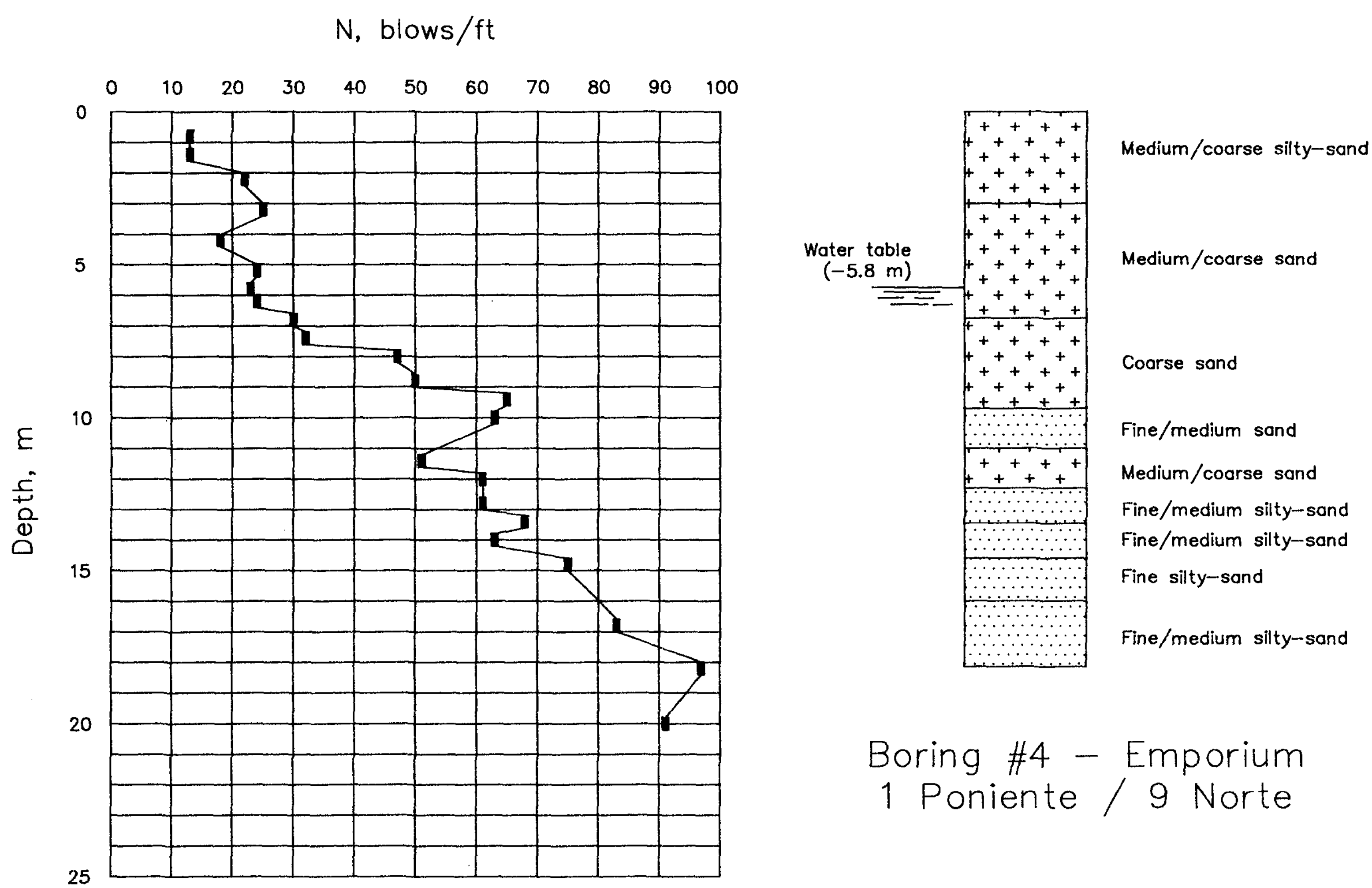


Fig. A.5 Soil Profile from Boring #4 [31]

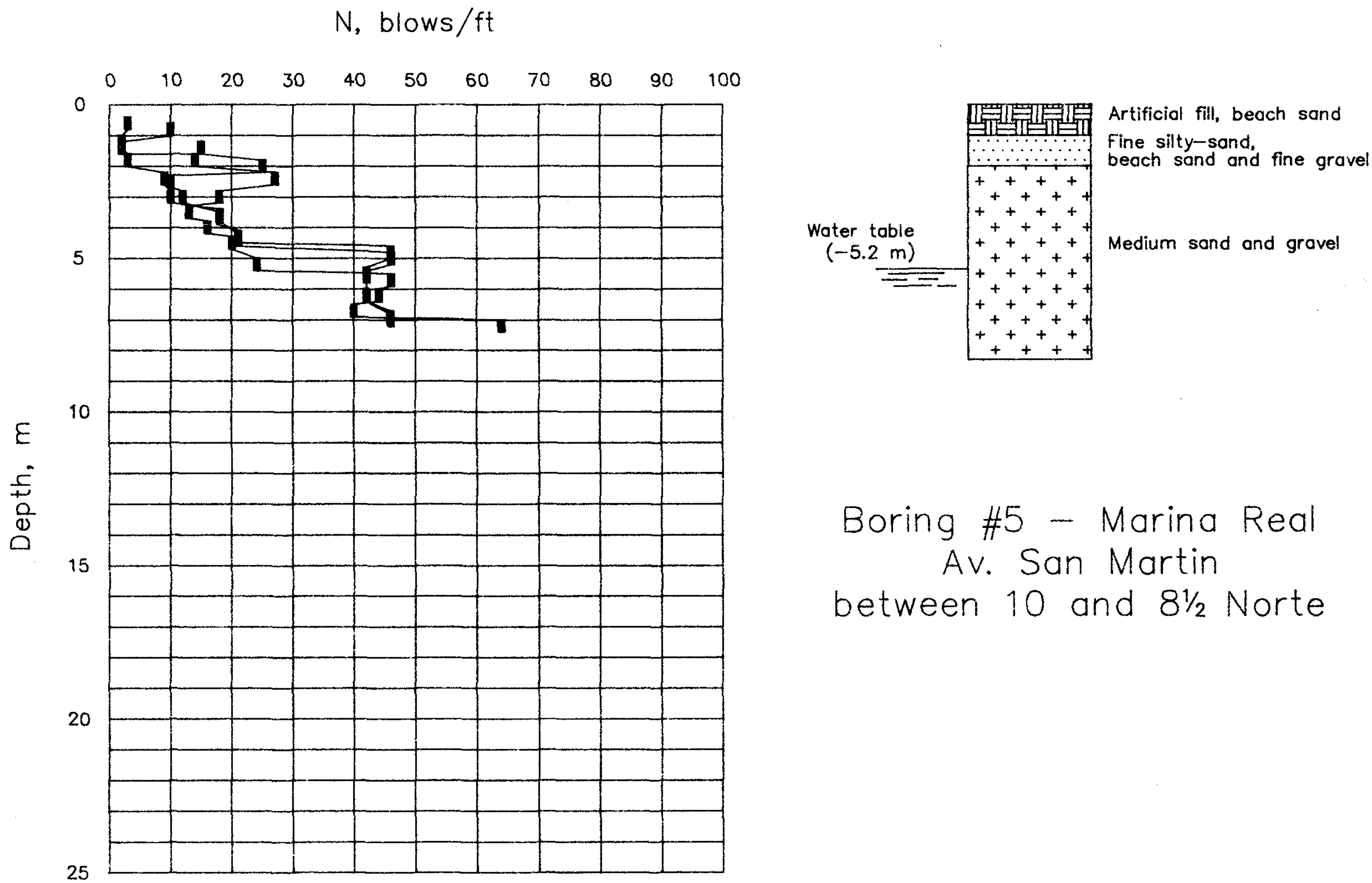


Fig. A.6 Soil Profile from Boring #5 [31]

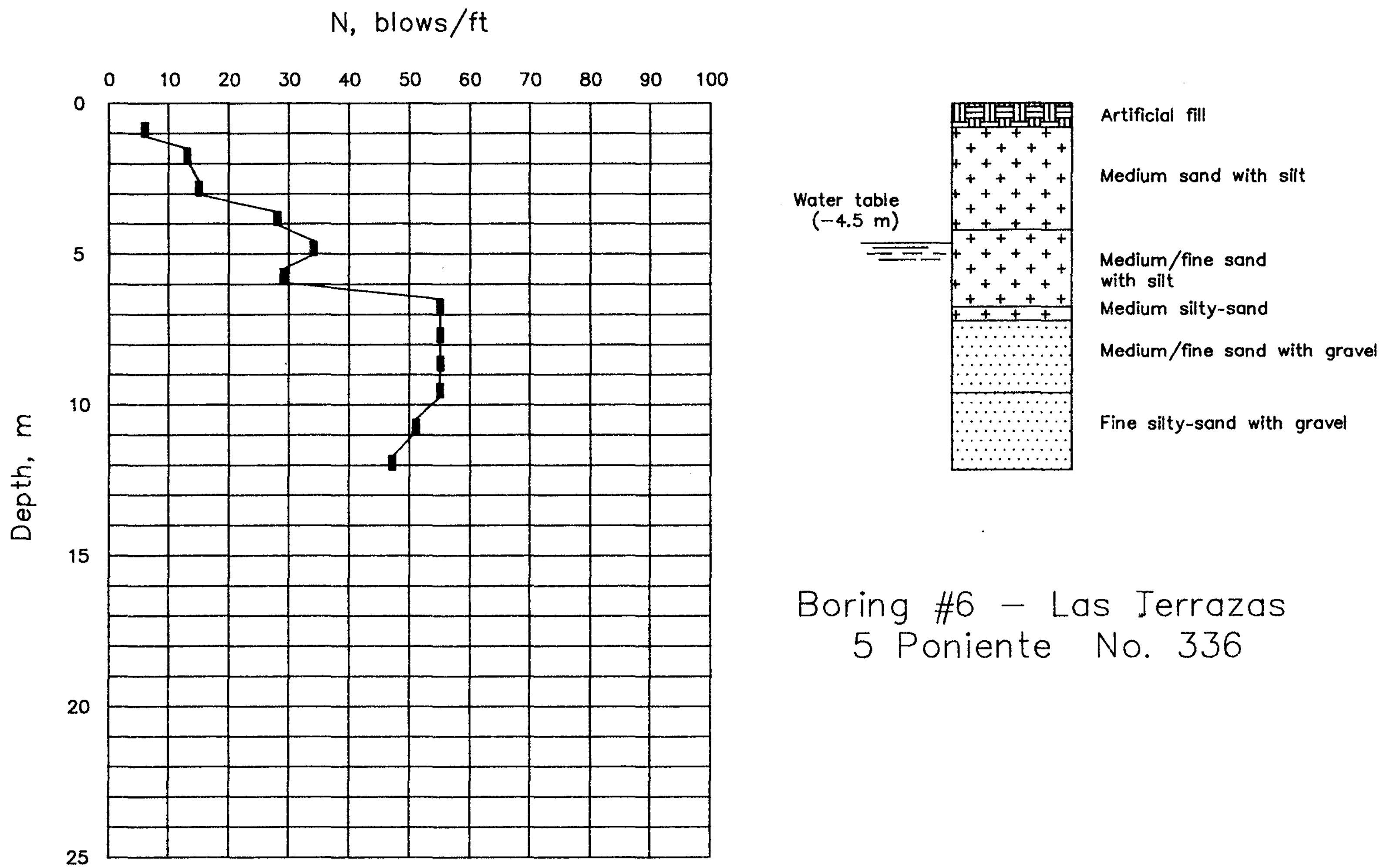


Fig. A.7 Soil Profile from Boring #6 [52]

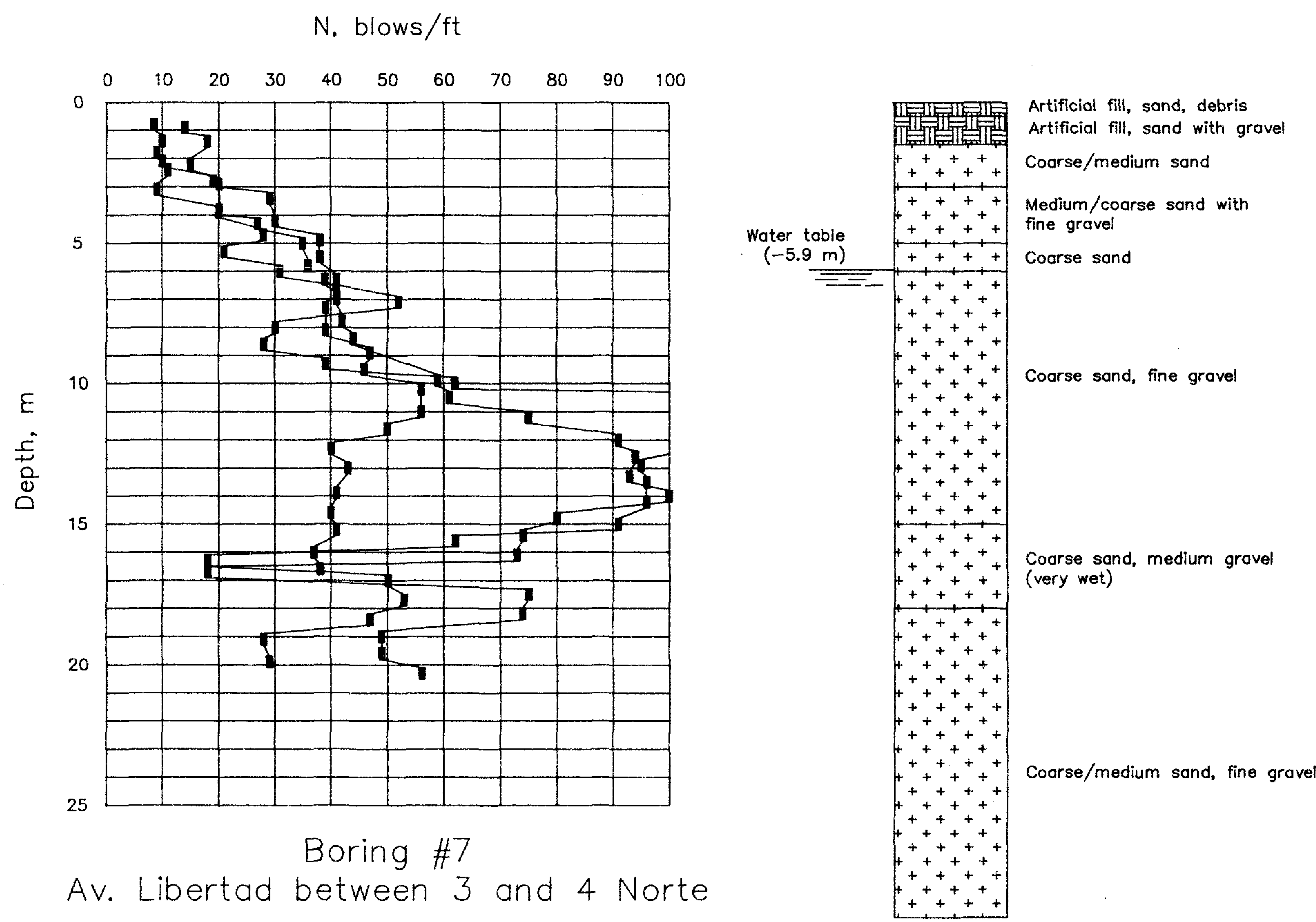


Fig. A.8 Soil Profile from Boring #7 [31]

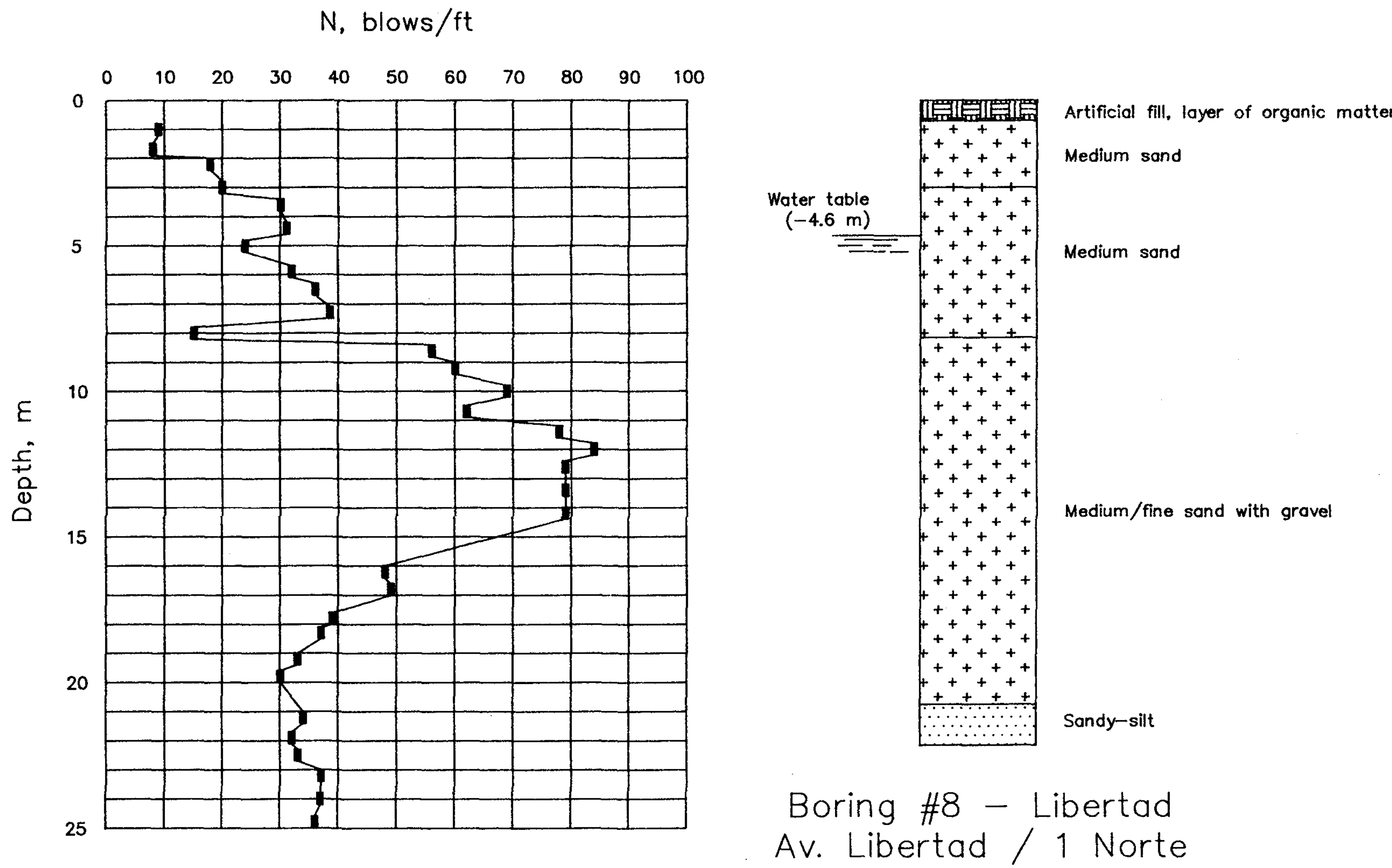


Fig. A.9 Soil Profile from Boring #8 [31]

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