THE SAN ANTONIO, CHILE, EARTHQUAKE OF 3 MARCH 1985

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ABSTRACT

This paper consists of the author's observations on the damage caused by the earthquake of 3 March 1985, centred near San Antonio, Chile. The damage was inspected approximately four weeks after the earthquake. At that time the overall cost of the damage caused by the earthquake was estimated at NZ$1,750 million.

INTRODUCTION**

The main earthquake occurred at 1947 hours local time on 3 March 1985.

Seismological records indicate that the earthquake actually consisted of two main shocks occurring within eleven seconds of each other. The first was of surface wave magnitude 5.2 on the Richter scale and was centred offshore of the town of Algarrobo. The second was of magnitude 7.8 and was centred offshore of the city of San Antonio approximately 40 kilometres south of Algarrobo. Both epicentres were at 33 kilometres depth.

The shocks combined to give violent shaking, reported for two minutes duration and were unable to be felt separately.

Although the effects were felt over a 1500 kilometre length of Chile, surface damage was minimal even in the immediate vicinity of San Antonio. There the major surface damage was confined to a subsidence on a large port reclamation, minor landslides at road cuttings and swamping where masonry retaining walls collapsed.

In the major earthquake(s) of 3 March approximately 170 people were killed, 2000 to 3000 injured and 200,000 made homeless. Casualty figures would have been much higher had the earthquake occurred during a weekday rather than at 8.30 on a Sunday evening. Some 200 schools were severely damaged and had to be closed along with numerous industrial structures.

Approximately 500 recorded aftershocks occurred in the month following the main shock. Some of these had a Richter magnitude of 4.5. The author experienced one of MM5 intensity on 8 April in which the shaking of tall buildings and the rolling motion of streets was clearly visible. As of 8 April 1985 the damage was estimated at NZ$1,750 million but the final cost was expected to be three to four times that amount.

GENERAL OBSERVATIONS

Overall damage in the metropolitan area of Santiago and region surrounding San Antonio-Santiago from Concon (135 kilometres north) to Carico (195 kilometres south) indicated that the earthquake had an intensity of MM7 to 8. Many residential, small commercial and industrial buildings suffered major structural damage and some large buildings, generally ten to fifteen years old or older, were badly damaged. Generally, new structures, both large and small, suffered very little, if any damage.

Residential areas suffered very badly. Most homes, particularly outside the wealthier areas of Santiago, are constructed from adobe bricks and mortar and as expected, these buildings suffered badly. Fallen parapets and collapsed walls resulted from inadequately tied roof structures and adjoining walls. The author visited the much publicised villages of Talagante and Melipilla and the port city of San Antonio, as well as the suburbs of Santiago, and in all areas 70-80 percent of adobe buildings suffered some damage ranging from cracks at wall junctions and parapets to complete collapse of walls and roof. The extent of damage in most cases was hard to assess accurately as much demolition, and often reconstruction in the same materials, had taken place in the month after the earthquake.

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** Full statistics for the earthquake are being sought from Chile and from the US Geological Survey.

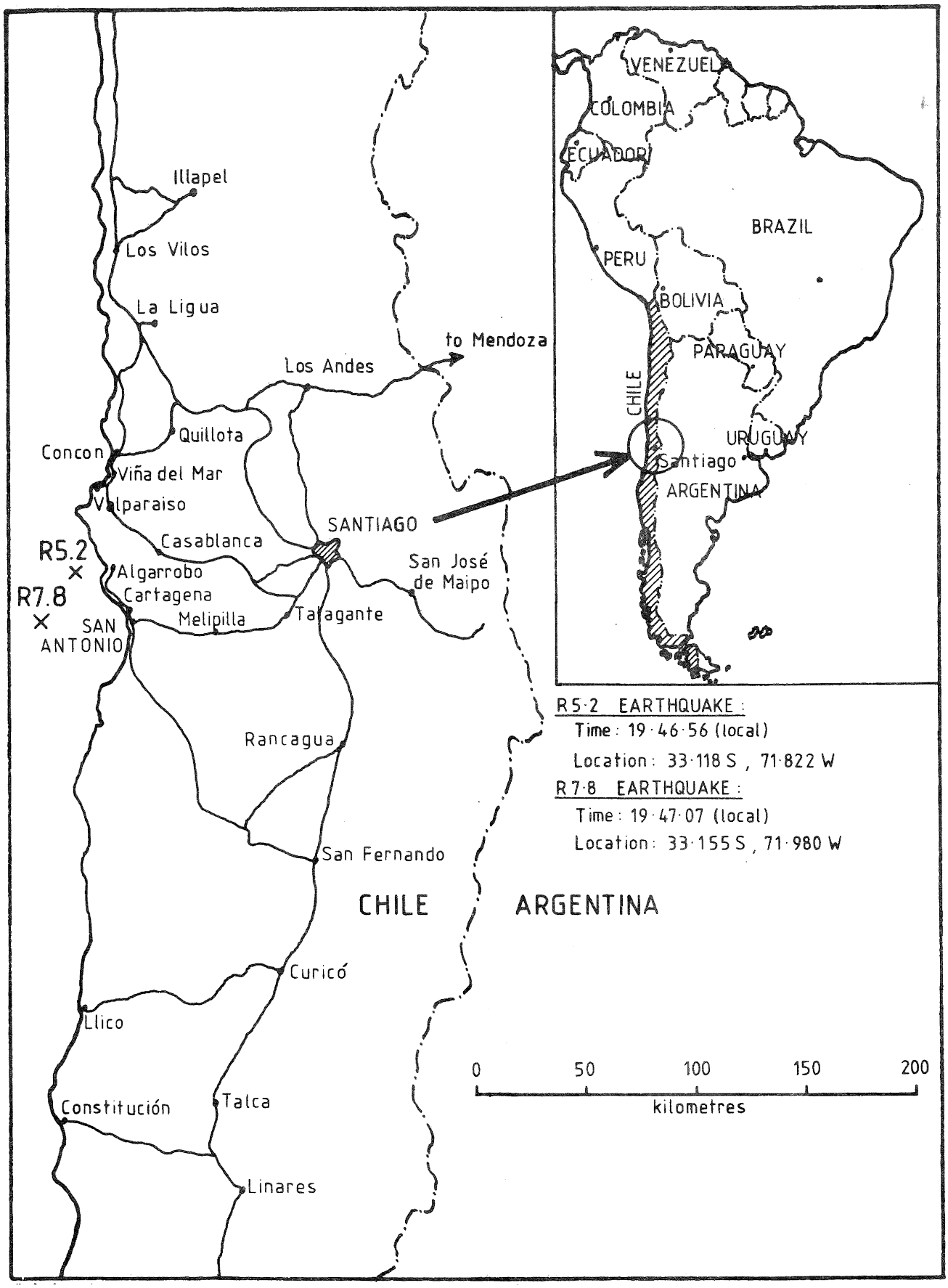

Figure 1. Area affected by the San Antonio Earthquake, Chile, 3rd March 1985. (see inset for location).

Reportedly almost all villages around the San Antonio-Santiago region suffered the same high level of destruction, mainly because of the predominant, poor earthquake resistant adobe construction.

Surface damage in general was not as severe as one would expect from the level of building damage. Many mountain roads (the main Andes range is less than 150 kilometres from San Antonio) suffered from rock falls. Apart from this, the only other surface damage the author was able to note was landslides resulting from masonry retaining wall failures and large-scale subsidence in the reclaimed areas of the port at San Antonio. The damage reduced the port's operating capacity by more than 50 percent. The subsidence appeared to be largely due to liquefaction of the failure of wharf foundations and retaining walls.

General structural damage to many commercial and industrial buildings was inspected. Most buildings in this region of Chile are constructed from in-situ reinforced concrete or masonry and there are very few structures of steel or timber. The main reasons for this are the local availability of materials for concrete and masonry, the low cost of labour, the high cost of imported steel from overseas and the high cost of timber from the south of the country.

Most damage occurred to buildings of 15 years or older due in the main to faulty detailing. Judging from the typical failure of shear panels, shear failure at column/slab joints and similar damage the faults could probably be put down to a lack of understanding at the time of ductility requirements and the differing behaviour of structural elements. More modern buildings behaved very well with hardly even a broken window in most buildings. One main road bridge south of San Antonio collapsed during the earthquake but the author was unable to visit it.

Most of the medium-rise, heavy, reinforced concrete buildings in the centre of Santiago dating from the 1930s to 1950s also showed very little damage. Many large masonry structures constructed in earlier years, typical of the sort of masonry buildings found in the centre of New Zealand cities, showed severe damage at corners, parapets and at large wall openings.

INDIVIDUAL BUILDINGS

1. Block 72 - Villa Olimpica - built circa 1962. A five-storey apartment block with a combination frame/shear wall structure in which end wall columns failed completely at their connection to the shear wall causing complete collapse of one bay of all floors. Block 72 is part of a large housing estate of similar structures, many of which showed some damage but which was generally confined to infill block walls on higher floors and roof mounted water tanks in other blocks.

Failure appears to have been due to lack of capacity at the column/shear wall connection on the west end wall of the block. Column ties at this highly stressed joint are too widely spaced, too small and inadequately anchored into the core to provide either sufficient shear capacity or confinement to the main column bars and the columns appear to have suffered a shear-compression failure.

Both the failed columns showed extensive fine cracking in both main faces. Although a general pattern was hard to see, it did appear that the cracks were inclined and were shear cracks. There did not appear to be any disturbance of the ground at the base of the columns indicating that the columns failed in shear and compression long before flexural capacity was reached.

From the overall orientation of the building (see sketch) the presence of cracks at the connection of the bridges to the adjacent block (see photograph 3) and the apparent east-west direction of the general ground motion, it seems that torsionally induced shear loading of the end wall initiated the failure. The relatively stiff stairwell structure at the east end of the block and the eccentric effect of the bridge restraints would have accentuated the torsional effect of basic eccentricity of the structure.

Some cracking of the east-west walls of the stairwell was visible, but only along the horizontal lines of construction joints and there was no apparent spalling at wall extremeties or floor connections. The sliding joints at the east end of the floors where they connected to the adjacent east-west block had moved - by approximately 100 mm at the roof slab connection - although it is difficult to say how much of this would have been due to the "dragging" effect of the collapsed slabs as they fell.

The connection of the bridges to the adjacent north-south block would have been expected to have a sliding joint at the end of their connection but this had not been constructed as such, and spalling on the bridge support beam (see photograph 3) confirms this. The internal columns of the adjacent north-south block were partially constructed of masonry and suffered severe damage, indicating significant deformation of this block, also five storeys.

Similar lack of adequate column ties is evident in the photo showing damage to the base of the south column in the second row from the west face. The failure of this column appeared to have been in compression, not in shear-compression, and possibly occurred at the moment when the west end columns failed and the end wall and upper floors collapsed, momentarily transferring excessive axial load on to these columns. No clearly identifiable shear cracks

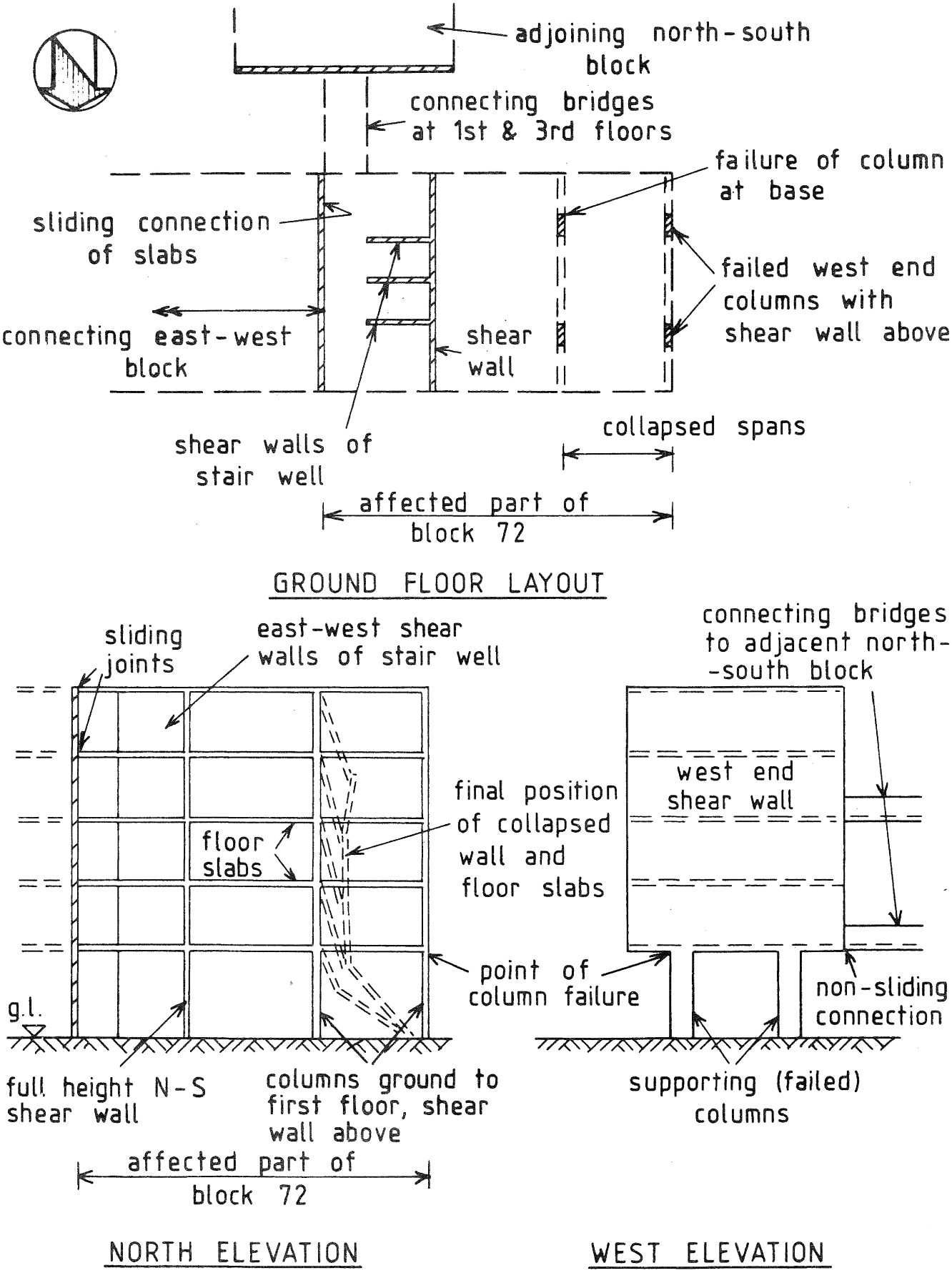

Figure 2. Villa Olimpica — Block 72 Layout.