# THE 1989 NEWCASTLE AUSTRALIAN EARTHQUAKE

## A FIELD REPORT BY EEFIT

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## 1.0 INTRODUCTION

### 1.1 General

On Thursday 28 December 1989 at 10:27 AM local time the City of Newcastle, a major industrial town situated on the east coast of Australia about 120km north of Sydney, was shaken by an earthquake measuring about 5.6 on the Richter scale. The earthquake caused considerable damage in Newcastle and twelve people were killed making this the first earthquake in Australia to cause casualties. Most of the damage was to unreinforced masonry and yet nine of the casualties were in one building, the Working Mens' Club, which is a reinforced concrete structure. The other three casualties were caused by falling masonry. It has been estimated that about 160 people were seriously injured as a result of the earthquake.

Ten days after the earthquake the UK-based Earthquake Engineering Field Investigation Team (EEFIT) mounted a two man, six day mission to the affected area. This report presents the findings of the EEFIT visit and subsequent follow-up studies.

The EEFIT team consisted of Jack Pappin, a geotechnical engineer from Ove Arup and Partners, London and Adrian Chandler, a lecturer in structural engineering at University College London who was supported by the Science and Engineering Research Council. The team spent four days in the Newcastle area carrying out damage surveys and one day in Canberra at the Australian Seismological Centre. Considerable assistance was received from Ove Arup and Partners Sydney who organised travel, accommodation and meetings with local authorities. In addition the New South Wales Public Works Department and the Newcastle City Council provided a great deal of useful information and allowed access to damaged buildings.

On returning to the UK a considerable effort has been made in evaluating and classifying the photographic and other data collected in the field. Andrew Coburn and other members of Cambridge Architectural Research have provided considerable assistance with this task, applying their established experience in this area. Finally a follow up visit has been made to the Newcastle area by Colin Taylor, a lecturer in structural dynamics at Bristol University, in early June 1990. During this visit he obtained damage statistics from the Newcastle City Council together with reports on the post-earthquake recovery and reconstruction programme.

### 1.2 Contents of the Report

Chapter 2 outlines the methodology adopted by EEFIT in the field and in the follow up studies. Chapter 3 gives a description of the geography, geology, population and economy of the affected region and Chapter 4 briefly discusses the seismological aspects of the earthquake.

Chapter 5 discusses the type of damage observed in the field primarily to the building stock and also to the infrastructure. Chapter 6 describes the distribution and extent of building damage and includes results from the detailed photographic survey and general street survey carried out in the field. Chapter 7 outlines the social response to the earthquake including the Disaster Management procedures adopted by the City Council.

The principal lessons from the earthquake and their implications to areas of low seismicity are discussed in Chapter 8.

### 1.3 Background to EEFIT

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

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

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

EEFIT members have investigated earthquakes in Italy (1980), Turkey (1983), N. Yemen (1983), Liege, Belgium (1983), Chile (1985), Mexico (1985), Kalamata, Greece (1986), San Salvador (1987), Loma Prieta, California (1989), Newcastle, Australia (1989), Romania (1990), Iran (1990) and the Philippines (1990). EEFIT reports have been published to date on the Liege, Chile, Mexico, San Salvador and Iran earthquakes.

## 2.0 METHODOLOGY

### 2.1 Principal Aims

The principal aims of the EEFIT team were to make an overall assessment of the distribution and scope of the damage and to study particular failures with an emphasis on engineered structures. Also of interest were the effect of the earthquake on the local infrastructure, its sociological impact and the methods by which the return to normal life was managed.

### 2.2 Factors Influencing Methodology

The following factors which applied to the epicentral area soon after the earthquake influenced the methodology:

- The area of damage was relatively small, being less than 100km$^2$
- The level of damage was generally small except in localised areas and access throughout the epicentral area by vehicle was generally unhindered
- Local engineers and officials were very helpful and informative
- A few badly damaged buildings had been demolished within one week of the earthquake.

### 2.3 Approach

The EEFIT team carried out most of the damage surveys by exterior visual inspection. This was done generally by vehicle with occasional photographs. The team used a 1:4000 scale aerial photograph montage of the town, dating from August 1986. A section of the montage is reproduced in Figure 2.1, and as can be seen individual buildings are readily identifiable. During the surveys the damage level was recorded directly onto the photographic montage. In two of the more heavily damaged areas a detailed photographic survey was undertaken on foot with photographs being taken of every building, thereby enabling reliable percentages of damage to be assessed at a later stage (see Section 6).

In addition to the general and detailed photographic surveys, several buildings were examined in detail and these are discussed in Section 5.

Meetings were also held with local authorities, Newcastle University and Coffey and Partners to obtain specific information on various aspects. These are detailed in the following travelogue.

### 2.4 Travelogue

**Sunday 7th January**
Arrive in Sydney

**Monday 8th January**
Driven by Ove Arup and Partners to Newcastle, collected hire car and met Jim Loke, senior structural engineer of the NSW Public Works Department, to discuss the type and extent of damage to Public Buildings (schools, hospitals, government buildings, colleges etc). Bruce Shephard of the New Zealand Works and Development Services Corporation who was assisting with the damage assessment and reconstruction also attended this meeting.

**Tuesday 9th January**

8.30am Meeting with Professor Moelle, Director of the Institute of Coal Research at the University of Newcastle. Discussions included information on coal seams that are being worked and foundation details.

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| 10.00am | Meeting with Professors Melchers and Page of the Department of Civil Engineering and Surveying to discuss types of damage. They provided the team with a copy of their Preliminary Report and Professor Page conducted the EEFIT team around several interesting damaged structures. |
| pm | Detailed photographic survey of Beaumont St and Lawson St in the suburb of Hamilton. General survey of the Hamilton area. |

**Wednesday 10th January**

| | |
|---|---|
| am | Detailed photographic survey of Hunter St. in the City centre area. |
| pm | General survey to south west and north of Newcastle followed by visit to Tighes Hill Technical College. |
| | Meeting with Geoff Padgett of Coffey Partners International Pty Ltd to discuss local soil conditions. |

**Thursday 11th January**

| | |
|---|---|
| am | General survey to south of Newcastle |
| 11.00am | Meeting with Harold Stuart, Director of Health and Building Services of the City of Newcastle Council, to discuss emergency procedures, building damage assessment and so forth. |
| 2.00pm | Meeting with Geoff Padgett to collect borehole data. |
| pm | General survey of areas west of Newcastle followed by return drive to Sydney. |

**Friday 12th January**

Flight to Canberra to meet Kevin McCue, Senior Research Scientist, and Marion Michael-Leiba of the Australian Seismological Centre, Bureau of Mineral Resources. Discussion included current thinking regarding Australian seismicity and the assessment of microseismic and instrumental records of the recent earthquake.

## 2.5 Follow Up Studies

Five days were spent at Cambridge Architectural Research with Andrew Coburn and with part time assistance from Shinobu Sakai, Robin Spence and Antonios Pomonis to study the detailed photographic survey. The photographs, together with field notes and maps referenced at the time, were used to compile a damage database by studying in some detail the construction types and damage patterns from the photographic evidence. Full details of this analysis are given in Section 6 of this report.

## 3.0 THE EARTHQUAKE AFFECTED AREA

### 3.1 Population and Economy

The City of Newcastle is situated at the mouth of the Hunter River and its development dates from the early 1800's. The Hunter Valley forms Newcastle's large and fertile hinterland, and the area has abundant coal resources. Newcastle soon became a centre for the export of coal and primary produce. Industry followed and in 1915 the Broken Hill Proprietary Company Ltd established its iron and steel plant. Now Newcastle is a major producer of iron and steel related products and produces about 25% of Australia's coal and steel. It is a major port and currently has a population of around 380,000 people.

### 3.2 Topography and Land Usage

Figure 3.1 shows the layout and topography of the Newcastle area. Generally it is between 5 and 10 metres above sea level in the vicinity of the Hunter River. There is a spit of land at a higher level along the coast to the south-east, and the land rises progressively to the south and west.

Land usage can be divided roughly into five types (Rural, Semi-Urban, Urban, Parklands, and Heavy Industrial) and these are shown in Figure 3.2. The study of maps of Newcastle produced in 1860 and 1925 has allowed some understanding of how the city has developed. The limit of urban areas shown on these maps is indicated in Figure 3.2.

### 3.3 Geology

The general geology in the Sydney and Newcastle area has been relatively stable since late Permian times. Figure 3.3 shows a plan and section of the basic structure of the underlying rocks. Newcastle is seen to lie on the remains of the Hunter and Mooki thrust system which originally extended about 1000km northwards from Newcastle. The movements across the thrust system led to marine sediments in the Sydney area and mainly terrestrial sediments in Newcastle. These sediments are referred to as the Newcastle Coal measures and consist of layers of Coal (12%), Tuff and Claystone (19%), Shale (17%), Sandstone (23%) and Conglomerate (29%) (Diessel 1980).

The Newcastle geological map (Dept. of Mines, N.S.W. 1966) shows recent alluvium overlying these coal measures. Generally the map shows that areas below 10m above sea level are alluvial and areas above 10 to 30m are underlain directly by the coal measures.

### 3.4 Coal Extraction

Directly beneath Newcastle the main economic seam to be worked is the borehole seam which is about 2m thick. Figure 3.4 shows the extent of coal extraction from the seam (Moelle 1990) and contours to the base of the seam (Diessel 1980). The method of coal extraction under Newcastle is pillar and stool with long wall methods being employed south and west of the town. It is known that there are older workings in higher seams but the location and extent of these is largely uncertain.

### 3.5 Ground Conditions

Coffey and Partners International Pty Ltd provided the EEFIT team with logs of 38 boreholes in the Newcastle area and the locations of these are shown in Figure 3.5. The boreholes are between 5 and 33m deep with about two thirds penetrating the underlying coal measures. The soils encountered in the boreholes vary considerably from loose silty sands to dense sands and from very soft clays to hard clays. Soil that is residual rather than alluvial, that is derived from weathered coal measures, are generally stiff clay to sandy clay. While generally there is up to 8m of residual soils where the coal measures outcrop, in the alluvial areas the alluvial soils sometimes lie directly over hard rock.

Because of the great variability of soil types and consistency it has only been possible to plot the thickness of soil overlying the coal measures. The Coffey and Partners plot (reproduced from the report by the Institution of Engineers Australia 1990) is shown in Figure 3.5 and clearly shows a much greater soil thickness (up to about 40m) under the port area. The industrial areas to the north of the Hunter River are partially sited on land reclaimed with hydraulic fill during dredging of the river bed carried out to enable larger ships to use the port facilities (see Figure 3.5).

## 4.0 SEISMOLOGICAL ASPECTS

### 4.1 Historical Seismicity

The Australian Bureau of Mineral Resources has compiled records of Australian earthquakes and has produced maps for Modified Mercalli Intensity and peak ground accelerations having a 10 per cent chance of being exceeded in 50 years (Gaull et al. 1990). Figure 4.1 shows their map for intensity and it can be seen that Newcastle has a value of Intensity V. They also indicate that a peak acceleration of about 4 percent of gravity (g) has the same probability of occurrence. Given the limited historical data in Australia they state that there is uncertainty in the calculations and that they can be used for indicative purposes only. This level of motion is small and the Australian Code of Practice shows Newcastle to be in the zero area (see Appendix A).

The City of Newcastle has experienced two previous earthquakes, in 1868 and again in 1925. Isoseismal maps for these earthquakes are produced in Figures 4.2 and 4.3 and it can be seen that they are very similar, both having a peak intensity of VI and a magnitude of 5.3 $M_L$ where magnitude has been calculated from microseismic intensity data. It is understood that in the 1925 earthquake slight damage was reported consisting principally of some superficial cracking to masonry structures, but no reliable data from this event has been obtained.

### 4.2 Regional Intensities

The Australian Seismological Centre has produced an isoseismal map for the 1989 earthquake and this is shown in Figure 4.4. Comparison with Figures 4.2 and 4.3 show it to be centred in similar locations to those previously but the recent earthquake differs in that the radii of the isoseismals correspond to one intensity higher, that is the intensity V isoseismal for the 1989 earthquake approximately matches the intensity IV isoseismals for the 1868 and 1925 earthquakes.

### 4.3 Epicentral Location and Instrumental Data

From instrumental data the Australian Seismological Centre estimates that the location of the earthquake epicentre was at 151.61°E and 32.95°S. Due to the sparse instrumental coverage they can only give an accuracy of about ±15km. This location is at the town Boolaroo, about 14km west of Newcastle. The accuracy however, shows the earthquake could have occurred almost anywhere under Newcastle. The depth of the earthquake has been assessed from surface reflections observed on seismograms in Scotland. Appendix B outlines this procedure which indicated a focal depth of about 10km. The magnitude of the earthquake has not been precisely computed, but it has been estimated using various methods and a Richter magnitude ($M_L$) of 5.6 has been assigned by McCue et al. (1990).

Two aftershocks have been reported, one a magnitude $M_L$ of 2, thirty minutes after the main shock (Ambraseys et al. 1990) and another with $M_L$ of 2.1 at 20:08 local time on the 29th December (McCue et al. 1990). The Australian Seismological Centre deployed 10 recorders in the area within 24 hours of the main shock and consequently located the second aftershock with reasonable accuracy. The location is 151.62°E and 32.95°S with an accuracy of ±3.18km east west and ±2.42km north south. The focal depth of the aftershock was 13.6 ±0.8km. This location agrees well with that for the main shock but there can be no guarantee that they occurred at the same location.

### 4.4 Strong Motion Parameters

No instruments were installed in Newcastle at the time of the earthquake, but instrumental readings for the earthquake have been reported by Ambraseys et al. (1990) who state that in Sydney (120km to the south) a peak acceleration of 0.13%g was recorded and at a location 50km west of Newcastle a peak velocity of 7mm/sec was recorded by a vibration monitor.

Observers in Newcastle describe the event as being like an explosion and agree that its duration was very short, lasting no more than 3 or 4 seconds. Others reported difficulty in standing and some observed waves travelling down the road or pavement (McCue et al. 1990).

Assuming that the focal depth was about 10km, then the hypocentral distance of the earthquake from the City of Newcastle would be about 18km for the observed epicentre, reducing to 12km if the epicentre was close to Newcastle. Using the attenuation laws published by Gaull et al. (1990) for Southeastern Australia gives the following ground motion parameters:-

| | | |
|---|---|---|
| Hypocentral distance (km) | 12 | 18 |
| Peak Intensity (MM) | 8.1 | 7.4 |
| Peak velocity (m/sec) | 0.22 | 0.14 |
| Peak acceleration (g) | 0.21 | 0.13 |

Calculations forecasting ground motion parameters have also been carried out using the attenuation laws devised by Toro and McGuire (1987) and Atkinson and Boore (1990) for the Eastern United States. The response spectra appropriate to a rock outcrop calculated using their relationships are shown in Figure 4.5. The peak acceleration values, represented by the spectral values at a structural period of 0.025 seconds, are seen to agree well with those calculated using Gaull et al. listed above. The peak velocity values from the Gaull relationship are however about three times higher than indicated by the response spectra in Figure 4.5.

The observed damage (detailed in Section 5) indicated that there was a distinct directionality of motion. Buildings in Hamilton (and Beaumont Street in particular) showed that the motion was stronger in the east-west direction with the main thrust being eastwards. An easterly direction was also indicated by the damage in the central business district. The main cathedral at the eastern end of Newcastle suffered about 20mm slippage on a metal damp course about halfway up the structure. This slippage was in an easterly direction. Buildings in Silsoe Street, Mayfield, however showed a principal direction of motion of north-east south-west.

## 5.0 TYPES OF DAMAGE

### 5.1 General Overview

This section describes the types of damage observed in Newcastle and its environs to both non-engineered and engineered buildings, and to other facilities. A high proportion of the damaged buildings was constructed of unreinforced brick masonry with very little resistance to lateral loading, and consequently there were many examples of partial or total collapse, particularly of gable ends, parapets, facades and chimneys.

The central business district and other older commercial areas experienced a very high density of damage to this type of building, particularly for older shops and warehouses (see Section 6) which often had weakened street facades resulting from the requirement for ground floor access. It was also evident that single storey buildings performed much better than multi-storey buildings, with chimneys and roof parapets in the taller buildings being particularly vulnerable.

Similar damage was caused to older two storey houses, and those with double leaf cavity construction suffered heavily from the lack of adequate ties, with the outer leaf collapsing as a result of the lack of restraint. Examples of this were also found in schools and colleges, some of relatively modern construction. Unreinforced masonry used as infill walls in reinforced concrete frame buildings also suffered extensive minor damage due to shear cracking, and there was also some evidence of damage to cladding.

Few modern engineered steel or reinforced concrete frame buildings suffered more than relatively minor non-structural damage, such as cracking of in-fill panels or facades. The collapse of the Working Mens' Club in King Street (where 9 of the 12 fatalities occurred) was an exceptional case, with failure caused probably as a result of the poor detailing to reinforced concrete floor slabs at their column and wall connections. Another notable example of poor resistance to dynamic earthquake loading was The Junction Motel, a three storey reinforced concrete frame building with pronounced vertical and horizontal stiffness irregularities. This building was demolished following shear failure of the outer columns along one side of the structure. These are discussed further in Section 5.3.

There was some moderate damage to most church spires and towers, and to other monumental buildings, mostly resulting in the cracking and loosening of masonry, which made the structure and surrounding area unsafe.

In the outlying residential areas the predominant form of housing construction is timber weatherboard of relatively flexible, lightweight construction. These buildings, which are mostly single-storey, were largely undamaged by the earthquake due to their greater flexibility and ability to absorb the energy transmitted to them by the ground motions.

There was no reported damage to industrial facilities or equipment, although some minor spalling of concrete storage silos was noted, and similarly services and transportation suffered only minor disruption as a result of the earthquake.

In the following sections examples of the damage caused to each of the above categories of structure or facility are presented. In many cases the causes of failure are obvious, but in others there was a more complex pattern involving specific design or detailing deficiencies which led to some isolated cases of partial structural failure or collapse.

Most examples are illustrated by plates and the locations of these are shown in Figure 5.1.

## 5.2 Unreinforced Masonry Buildings

### 5.2.1 Residential

In the older suburbs of the city (with many buildings of pre-1920 construction), the predominant form of housing is cavity brick, with the outer leaf of unreinforced brickwork being poorly tied back to the inner load bearing wall. These structures have generally been badly maintained, and often exhibited prior cracking (perhaps due to foundation movement), erosion of mortar and corrosion of brick ties. Typical damage is shown in Plate 5.1, where the outer wall has mainly collapsed, revealing the poor condition of the mortar and ties (Plate 5.2).

Buildings with solid double brick walls were also vulnerable to collapse (Plate 5.3), particularly above first storey level in the case of multi-storey buildings (Plate 5.4). Where gable ends in buildings with pitched roofs had not been adequately restrained by proper tying to roof trusses, outward collapse of brickwork was commonplace (Plate 5.5). Some older unreinforced masonry single storey dwellings in residential areas such as Mayfield suffered severe external and internal damage. In addition to the collapse of the outer skin (Plate 5.6), there was also in the same building severe diagonal shear cracking, especially at corners and surrounding window frames (Plate 5.7). This cracking also affected the internal load-bearing walling (Plate 5.8). Ornamental features such as exterior brick columns and arches were also heavily damaged (Plate 5.9).

The danger from collapsing masonry walls and gable ends was accentuated by the close proximity of buildings (Plates 5.10 and 5.11), with neighbouring structures suffering secondary damage to walls and roofs. Facades and parapets were vulnerable features, such as in the two-storey building situated close to the central business district shown in Plates 5.12 and 5.13. Outward leaning of the masonry walling is also evident, with severe danger of subsequent collapse making the building unsafe for re-habitation.

Modern masonry buildings were generally less heavily damaged, although poor design features resulted in some cases of failure. An example of this is shown in Plates 5.14 and 5.15. This three-storey apartment building in The Junction area of Newcastle has a pronounced lack of stiffness at the ground floor level on one side (to the right in Plate 5.14) due to the garage entrances (Plate 5.15). In contrast, access at the rear of the building to the first and second floor apartments is via stairwells sited close to the rear facade, and consequently there is a high longitudinal stiffness in the direction of the photograph shown in Plate 5.14 (left hand side). As a result, the earthquake loading was transferred almost completely to the walls and stairwells at the rear entrances to the building (Plates 5.16, 5.17), resulting in wide cracking of the unreinforced brickwork, especially near corners, doors and window openings. This building was evacuated following the earthquake, pending a decision on whether or not repair of the damage could be effected.

### 5.2.2 Commercial

In commercial districts such as Hamilton, Tighes Hill and The Junction, there was extensive collapse and failure of unreinforced brickwork buildings, many of which had been weakened by shop and warehouse frontages (Plate 5.18). Beaumont Street in Hamilton was particularly hard hit by the earthquake, as shown by the aerial photograph in Plate 5.19 and the rescue operations from a collapsed facade of a two storey building immediately after the event, shown in Plate 5.20. A 1.4km length of the worst affected part of Beaumont Street was photographed in detail ten days after the earthquake, and the results have been analysed along with others in Section 6 to determine the detailed distribution of damage amongst buildings of different types, height, age and usage. A sequence of 12 photographs taken in Beaumont Street (eastern side) are shown in Plates 5.21-5.32, where the mixture of modern and older commercial buildings, and the types of damage experienced, are clearly evident.

The damage to commercial structures was concentrated mainly in the older unreinforced masonry buildings, with failures to parapets and gable ends being widely evident. Most failures and collapses (which resulted in three deaths in this street) occurred at the upper storey or roof level, and there were several cases of dangerously leaning facades, such as shown in Plate 5.33.

A significant contributory factor to failures of building facades was the presence of awnings in the front of shops, hotels, theatres and so forth. (Plates 5.34, 5.35). Originally, such buildings had balconies or awnings supported by timber posts at the edge of the footpath. During the 1950's this form of construction was considered to be a hazard to traffic and consequently to pedestrians should traffic accidentally remove one or more of the posts. As a result of this, the posted verandas and awnings were removed and replaced by suspended awnings. These are held up by so-called 'tie-backs' to the masonry walls above the ground floor level (Plate 5.35). In most cases the tie-backs are anchored in the masonry wall using a plate on the inside face of the wall. In other cases the tie-backs are anchored at cross-walls, and in very few cases anchored back further into the cross-wall through a connecting set of plates or bars. This form of construction has proved quite adequate for conventional wind loads, but the earthquakes caused exceptionally high additional dynamic loads which resulted in many cases in the collapse of the facade, or rendered it dangerous as a result of cracking close to the connecting points (Plate 5.35).

Had the earthquake been of longer duration, it is very likely that many more awnings and facades would have collapsed. Plate 5.36 shows a commercial building in the central business district with an unstable facade showing evidence of cracking and parapet collapse. Modern unreinforced masonry commercial buildings also suffered large numbers of failures, particularly to tall parapets (Plate 5.37) and to gable ends (Plate 5.38).

### 5.2.3 Public Buildings

Schools and colleges in the inner suburbs of Newcastle suffered extensive damage as a result of the earthquake, and it was very fortunate that the event occurred during a holiday period. A 100 year old school in The Junction suffered extensive collapse of its roof, parapets and gable ends (Plates 5.39, 5.40), and will probably have to be completely rebuilt. A school building of similar age on the edge of the central business district also suffered extensive cracking of its facade (Plate 5.41). In Merewether, the modern girl's school showed several examples of partial collapse of unreinforced brick walls which had not been tied back to the steel trusses forming the structural building frame and roof frame (Plates 5.42, 5.43). The dynamic response of the steel portal frames may have caused or contributed to the damage in this case.

In Tighes Hill, an estimated A$10m damage was caused to the Technical College, consisting of widespread collapse of parapets and collapse or cracking of outer skin masonry cladding (Plates 5.44 and 5.45), and diagonal shear failure in internal masonry walls infilling the reinforced concrete frame structure (Plate 5.46). The earthquake also caused a fire to start in a third floor laboratory (Plate 5.47), but this was soon brought under control by the fire fighting authorities.

The Royal Newcastle Hospital reportedly (Tiedemann 1990) experienced extensive internal damage and plant failure in the North Wing (Plate 5.48), which was evacuated and closed following the earthquake. A 30m chimney at the same hospital escaped without any apparent damage (Plate 5.49).

## 5.3 Engineered Buildings

Damage to engineered buildings was confined mainly to non-structural elements such as the failure of infill masonry panels which act as stiffening elements within a reinforced concrete frame (Plate 5.46), and whose strength prior to failure acts as an additional protection against damage to the structural frame itself. Some taller buildings in the central business district such as the seven storey steel framed office building shown in Plate 5.50 and the Hamilton Telephone Exchange shown in Plate 5.51 suffered partial collapse of unreinforced masonry parapets, and there was also loss of cladding and cracking of masonry in-fill in an office building in King Street (Plates 5.52 and 5.53) and at the Newcastle Permanent Building Society office in Beaumont Street, Hamilton. Cracking of unreinforced masonry cladding was also observed at a multistorey reinforced concrete car park building in King Street (Plates 5.54 and 5.55), whilst a similar building in Hunter Street (Plate 5.56) suffered total collapse of the lower five storeys of unreinforced brick masonry walling which had been inadequately attached to the reinforced concrete frame at the corner of the building.

Other reinforced concrete frame and frame-shear wall buildings in the central business district suffered no damage to either structural or nonstructural components (see Plates 5.57 and 5.58, for example). The Newcastle City Council offices in King Street (Plate 5.59, circular building) was another example of a modern engineered building which suffered virtually no damage from the earthquake, and was able to function immediately after the earthquake as the centre for monitoring and directing the damage assessment surveys (see Section 7). The police station at the eastern end of Hunter Street (Plate 5.60) was designed to resist earthquakes up to magnitude 5, since it is considered to be an essential facility for post-earthquake recovery. The engineering design of this reinforced concrete frame building involved a 10m deep excavation surrounded by reinforced concrete slabs and a grid of 1m diameter reinforced concrete bored piles. No damage to this building was reported following the earthquake.

Two buildings which did suffer extensive structural damage or collapse due to the earthquake were the Newcastle Working Mens' Club in King Street (Plates 5.61, 5.62), and the Junction Motel on Darby and Tooke Street (Plates 5.63,5.64). The reasons for the dramatic collapse of the Working Mens' Club (where nine died) are not clear, and at present no conclusions can be drawn from this structure, which had been demolished before the arrival of the EEFIT investigation team. Two contributory factors in the collapse, however, appear to have been the lack of adequate shear reinforcement in the concrete floor slabs at their connections with the columns of the reinforced concrete frame (Plate 5.62), and the failure of an exterior wall which triggered the sudden roof collapse (Plates 5.61 and 5.62).

In the case of The Junction Motel (Plate 5.63), a classic combination of a soft first storey and a highly asymmetric stiffness layout led to shear failure and crushing at the top of a row of exterior reinforced concrete columns (to the right of the photograph in Plate 5.63, see detail in Plate 5.64), together with cracking and more severe fracturing of some large central columns at the bar overlap. This 3 storey building was demolished a few days after the earthquake.

## 5.4 Churches and Monumental Buildings

Newcastle's Christ Church Cathedral showed minor cracking to exterior walls, and some finials mounted on top of the parapet rotated by up to 45° during the earthquake (Plate 5.65). This building had previously been damaged and repaired in the 1925 earthquake (see Section 4.1), and a crack reputedly dating from this event was evident in the north wall (Plate 5.66). Many other churches suffered partial collapse or cracking of masonry spires (Plate 5.67) and towers (Plate 5.68) and were thereby rendered unsafe for use. A war memorial sited on a hill above the central business district lost its capstone as a result of the earthquake (Plate 5.69).

## 5.5 Timber Buildings

Of the various types of housing construction prevalent in Newcastle, the best performance was observed in timber framed weatherboard dwellings (Plate 5.70). These are of relatively flexible, lightweight construction and are therefore able to accommodate earthquake movements more readily than the various forms of construction comprising unreinforced masonry walls (see Section 5.2). They also have lower natural frequencies than masonry buildings, and this would have been advantageous in such an earthquake which as a result of its magnitude and relative proximity to the City was probably of short duration consisting of primarily high frequency motion. Some cracking of internal plasterboard was noted and cornices were loosened. The most common area of concern appears to have been the separation of masonry fireplaces from the timber structure, but in most cases this did not represent a serious problem. There were also many instances of relative movement between the timber structure and its foundations (Plate 5.71), apparently caused by lateral movement of the complete building.

A common form of construction for single storey houses built post-1960 consists of a brick veneer on a timber frame. These were mostly located in the modern, outlying suburbs of Newcastle and suffered only minor forms of damage, except in a few isolated cases. Damage was occasionally observed due to the flexible internal frame pounding the brickwork cladding. Plates 5.11 and 5.38 show an extreme form of the damage with brickwork gable ends that probably have been pushed by the timber roof trusses behind them.

## 5.6 Industrial Facilities

There are considerable areas of Newcastle close to the Hunter River and port concerned with industrial processing and handling of raw materials (Plates 5.72 and 5.73). Initial reports of an explosion at a blast furnace at the BHP steel works were unfounded, the furnace having been vented prior to shutting down as a precautionary measure. Blong et al (1990) report some damage to the steel making shop roof. No other reports of damage or disruption were made, although some minor spalling of concrete was evident in a few storage silos (Plates 5.74 and 5.75).

## 5.7 Services

There was generally little or no significant damage to services in the Newcastle area in the aftermath of the earthquake. The electricity supply comes via the 330kV/132kV sub-station at Killingworth and the six principal sub-stations under the Council's authority. At Killingworth, about 20km west of Newcastle, the ground vibration tripped pressure sensitive relay switches and several porcelain insulators supporting 132kV switch gear were damaged (see Plate 5.76). There was damage to some 10-15 sub-stations in the City, out of a total of about 400. This damage consisted mainly of collapsing masonry walls and roofs. In these cases the transformers tripped, which was recorded on the automated information system installed in all sub-stations. The average interruption to supply was about three hours following the earthquake, and there were no known instances of fires caused by damage to electrical circuits. At the time of the EEFIT visit ten days after the earthquake, all circuits were back in action but isolation of certain districts or buildings was in progress to ensure that unsafe buildings could be inspected and damage assessments carried out.

The gas supply was uninterrupted by the earthquake, with no lines ruptured, and similarly the water and sewage systems reported very few additional leaks or cracks.

The Hunter District Water Board reported that there was an increase in water main repairs to 12 on 26th December and 11 on 29th December. After 30th December the repairs reverted to the average of 4.3 repairs per day. Defective service repairs to individual services increased from an average of 13 per day to 27 on the 28th, 56 on the 29th, 16 on the 30th and 20 on the 31st of December. In the Stockton area there were many 'no water' complaints due to an automatic valve closure.

Some telephone lines were disrupted for a few hours after the earthquake, but otherwise no problems were reported.

## 5.8 Transportation

No significant damage was reported to any roads, bridges, drainage systems or subways. The western embankment of the Stockton Bridge (Plate 5.77) settled by about 50mm. The embankment is about 6m high. The bridge was closed for several hours and a detailed investigation carried out. No structural damage was detected.

The only other disruption to transportation was due to collapsed masonry, and within a few hours of the earthquake the Police closed off the central business district and other local areas, as described in Section 7.2. The Police also set up barricades around unsafe buildings (see Plate 5.41, for example) to prevent unauthorised access. Many damaged awnings were propped to eliminate the danger of collapse (Plates 5.33, 5.35).

## 5.9 Geotechnical Aspects

Newcastle has several areas with steep slopes and there have been continual slope stability problems over the years. The Institution of Engineers Australia report (1990) discusses one such slope which was monitored during the earthquake. An increase in pore pressure up to 2m head was recorded, but no activation of the slip occurred. The Institution of Engineers Australia report also states that there is strong evidence of an older slip being re-activated by the earthquake, causing some building damage. Following the heavy rains in February some major slips have occurred, but the earthquake is not considered to be a major contributory factor to these events.

As expected for an earthquake of this magnitude and duration, there were no observed instances of liquefaction even though some of the soils consist of loose sands near the surface. Densification has possibly occurred, however, as the Newcastle City Council has recorded settlement of up to 60mm in The Junction area and 15 to 20mm of settlement in Hunter Street in Newcastle West (Institution of Engineers Australia 1990).

Unlike many larger earthquakes there was very little evidence of ground distortion during the earthquake. Very few buried services were damaged and paving stones and kerbing were generally not affected. There was some evidence, however, of ground compression indicated by bulging brick paving as shown in Plate 5.78. It was also reported by several people that similar damage had been observed in basement floor tiling systems, but the EEFIT team did not observe this.

Indirect evidence of continuing ground movement was obtained from many reports of slow continuing damage occurring for several weeks after the earthquake (Perry 1990). This was reported to be occurring in the car park wall shown in Plate 5.55 (Page 1990). While this ongoing damage could have been due to ground movement, it may also be possible that the structural cracking led to stress redistribution on the foundation inducing consolidation of the underlying soils.

## 6.0 DISTRIBUTION AND EXTENT OF BUILDING DAMAGE

### 6.1 Introduction

The extent to which the various types of damage occurred and their distribution are of vital importance to the understanding of the significance and impact of this event. This section presents the results of the two damage surveys carried out by the EEFIT team.

### 6.2 Detailed Photographic Surveys

#### 6.2.1 Methodology

Two detailed street surveys were carried out in Beaumont Street and Lawson Street, Hamilton and in Hunter Street, which is the main thoroughfare in the Newcastle central business district. A total of 625 buildings were logged photographically, in sufficient detail for later analysis. Each survey involved photographing externally every building (either individually or in small groups), on both sides of the street. The survey in Hamilton covered approximately a 1.5 km length of Beaumont Street from its start at Maitland Road to the junction with Dumaresq Street at the corner of the Newcastle Racecourse (Figure 5.1), and then continued in the reverse direction along a 1 km length of Lawson Street from the junction with Dumaresq Street to its termination at Donald Street. The building stock in the surveyed part of Beaumont Street consists primarily of older commercial unreinforced brick masonry buildings, mostly one or two storeys in height (see Plates 5.18 - 5.32), with brick masonry and timber residential buildings in the southern part in the approaches to the Racecourse. Lawson Street is primarily residential (dominated by low rise timber framed dwellings), with some commercial/office developments and the steel framed Telephone Exchange building clad with unreinforced brick masonry (see Plate 5.51).

Three kilometres of Hunter Street was surveyed from its eastern end at the junction with Telford Street, to the junction with Tudor Street at the boundary of Newcastle West and Hamilton (Figure 5.1). This is primarily a commercial and business district dominated by multi-storey shops and offices, mostly of reinforced concrete frame or unreinforced masonry construction. As described in Section 2.5, the information obtained from these photographic surveys has been collated in the UK at the offices of Cambridge Architectural Research. The following parameters were recorded for each building:

**Location by Australian Grid Reference**

**Primary Classification of Construction Type**

Categorization of Primary Construction type could usually be carried out with relatively high confidence from the general appearance of the building. Load bearing brick masonry buildings, for example, are usually fair-faced, and timber framed buildings often weather-boarded. Where building type was ambiguous or uncertain, for example due to heavily rendered exteriors or where external masonry may have been used as a cladding rather than for structural purposes, other indicators such as size and regularity of fenestration, beams, and timber framing elements were used to determine the construction type.

**Secondary Classification of Construction Type**

Categorization of secondary construction type was important mainly for brick masonry buildings. Part of the investigation of the earthquake damage was to establish the seismic vulnerability of cavity construction masonry, which is a common building method in the United Kingdom but rarely found in earthquake zones. The Newcastle earthquake provided an opportunity to assess the comparative performance of solid masonry construction, with two leaves of masonry through-bonded, against that of cavity construction. Cavity construction was identifiable from the stretcher bond of the exterior wall, air vents in the lower courses and moisture weep holes or flashing around openings. Cavity masonry is also identifiable with a certain period of construction, not being a common construction technique until the 1930's. Solid masonry was identified from Flemish bonding, thickness of masonry walls, use of decorative or moulded brickwork, pilasters and other such indicators. The categorization of secondary construction type is less reliable, because the detail needed to determine this parameter with any confidence cannot always be obtained from photographs. Construction type can be determined more accurately at high levels of damage, because walls are split apart revealing their exact construction type, generally related to the construction period and architectural detail of the building.

**Number of Storeys**

The number of storeys could be determined from the photographs.

**Age Classification**

The age was determined from the stylistic treatment of the architectural form and facade. In a number of cases the actual date of construction is given on the exterior of the building and this was used to confirm dating of building styles. The classifications of age used were fairly broad, relating to quite clear periods of construction in Newcastle, i.e. pre-1920, 1920-1940, 1940-1960, and post-1960. These periods relate to the phases of development of Newcastle, with much of the main street being older than the buildings around it, and intermixed with a recent phase of commercial development within the City centre.

**Usage Category**

A few broad categories of usage were also employed, depending on the exterior appearance of the building. Generally the distinction being sought was between residential and commercial building stock. Where a particular type of use was obvious, for example a bank or school, this was noted. The usage category has a significant link with architectural style. Shops and delicatessens have large front facade parapets, often in masonry, to signify their purpose. Larger stores tend to have free openings or long spans on the ground floor. Commercial buildings in the town centre are typically retail units on the ground floor with office premises on the floor above. Residential buildings tend to be detached or occasionally terraced, in their own grounds.

**Damage Level**

The MSK categorisation of damage was taken, as elaborated in past Martin Centre (Coburn et al 1990) and EEFIT damage surveys:

| | Degree of Damage | Definition for load bearing masonry | Definition for reinforced concrete buildings |
|---|---|---|---|
| D0 | Undamaged | No visible damage | No visible damage |
| D1 | Slight Damage | Hairline cracks | Infill panels damaged |
| D2 | Moderate Damage | Cracks 5-20mm | Cracks <10mm in structure |
| D3 | Heavy Damage | Cracks <20mm or wall material dislodged | Heavy damage to structural members, loss of concrete |
| D4 | Partial Destruction | Complete collapse of individual wall or individual roof support | Complete collapse of individual structural member or major deflection to frame |

| | | | |
|---|---|---|---|
| D5 | Collapse | More than one wall collapsed or more than half of roof | Failure of structural members to allow fall of roof or slab |

The damage level is based on evaluation of the exterior of a structure and was generally easily identifiable for most buildings. The lower levels of damage, such as D1, were not so readily identifiable as these caused thin cracks not easily seen from the photographs.

**Damage Type**

The damage observed on the photographs was recorded in the database. Generally this meant noting the type and extent of cracking and for the more severely damaged buildings recording which parts of the structure had collapsed.

### 6.2.2 The Building Database

The database which has been compiled from the photographic surveys carried out in Newcastle consists of 625 buildings (Figure 6.1), classified by the primary type of construction (see Section 2.5) into brick masonry (372 buildings or 60 percent of the total), reinforced concrete frame (137 buildings, 22 percent), timber frame (104, 16 percent); the remaining 12 buildings (2 percent) were either steel frame or composite construction. Of the 372 brick masonry buildings in the survey, the great majority (99 percent) were of either solid double (31 percent) or cavity construction (68 percent). The remaining 1percent were of solid single leaf construction.

Each building was designated either commercial or residential, according to its primary usage (Figure 6.1). Of the 625 buildings surveyed, 428 or 68.5 percent were commercial and 147 or 23.5 percent were residential. The remaining 50 buildings (8 percent) consisted of car parks, a police station, sports buildings, churches, meeting halls and so forth.

Further classification of the surveyed buildings was carried out by age and number of storeys. Classification by age is shown in Figure 6.2. 48 buildings were dated pre-1920 (8 percent), mostly consisting of brick masonry construction. The greatest proportion of buildings (260 or 42percent) were between 1920-1940, whilst 176 buildings (28 percent) were from the period 1940-1960. Recently constructed buildings (post-1960) numbered 126 (20 percent). Of the remaining 15 buildings (2 percent), 12 were of non-classified construction types and 3 were brick masonry buildings of unknown age. Over half of the timber framed buildings dated from the period 1920-1940, whilst 80 percent of the reinforced concrete frame buildings dated from post-1940.

Classification by number of storeys is shown in Figure 6.3. The majority of the buildings surveyed were low-rise with less than 5 storeys, with only 34 buildings (6 percent) being in the mid-height range, namely 5-10 storeys. No buildings of more than 10 storeys were surveyed. These figures are representative of the City as a whole, where low-rise construction dominates. The proportion of buildings with 1, 2, 3 and 4 storeys in the survey was 31 percent, 44 percent, 15 percent and 4 percent, respectively. A high percentage of the brick masonry buildings (55 percent) were of 2 storeys height, with only 19 percent being 3 storeys or higher. The reinforced concrete frame buildings were almost entirely multi-storey (93 percent), as expected, with 30 buildings (22 percent) being over 4 storeys in height. In contrast, 85 percent of the timber framed buildings were single storey.

### 6.2.3 Seismic Vulnerability Estimation

The building database has been used to estimate the vulnerability to earthquake damage of the three main types of construction prevalent in Newcastle, namely unreinforced brick masonry, reinforced concrete frame and timber frame. The majority of the categories into which the buildings have been classified contain at least 20 buildings and hence the results of the survey are statistically viable and are considered to give an accurate assessment of the vulnerability and seismic risk associated with the building categories analysed. From examination of statistical samples of building damage, the performance of a sample of less than 20 buildings of any classification are considered too influenced by the performance of individual structures to be considered representative of the classification (Coburn et al 1990). In this context, categories such as solid single leaf brick masonry buildings (4 in total), residential reinforced concrete frame buildings (1) and pre-1920 reinforced concrete frame buildings (2) have not been included in the damage survey results.

The results of the damage survey have been presented for each category in terms of the percentage of buildings with damage level greater than or equal to D1, D2, D3 and D4. Figure 6.4 shows the damage level distributions associated with the three main building types. Greatest damage was observed, as expected, in the brick masonry buildings where more than one third (37percent) suffered at least light damage, and 21percent experienced moderate to heavy damage. The solid double leaf and cavity construction showed similar damage proportions for ≥D1, but considerably fewer cases of D2 or D3 were observed for the solid double brick masonry buildings.

For the reinforced concrete frame buildings, the percentages suffering ≥D1 (20 percent) and ≥D2 (12 percent) are about half those for brick masonry buildings (Figure 6.4), whilst for timber framed buildings the majority of the damage occurred at level D2 (14 of 104 buildings, or 13 percent). Very few buildings in Newcastle suffered damage of Level D4 (Partial destruction, involving the collapse of a complete wall). Overall, only 5 brick masonry buildings, no reinforced concrete frame buildings and one timber framed building were assigned this damage level, representing 2percent of the surveyed buildings. Apart from the Working Mens' Club and The Junction Motel, no buildings experienced damage classifiable as D5, involving the collapse of roofs or floors.

Breaking down the data to analyse the effects of other characteristics of the building stock is limited by the size of the data sample. Figure 6.4 shows the distribution of damage levels by commercial or residential usage, within each primary building type. For brick masonry construction, there were considerably higher proportions of commercial buildings with heavy damage or partial destruction than for residential buildings. The proportions suffering light or moderate damage, however, were very similar. As expected, the reinforced concrete frame buildings were almost entirely commercial and hence the damage distribution was representative of this construction type as a whole. Timber frame buildings, however, were primarily residential and these suffered considerably more light to moderate damage than commercial buildings.

Figure 6.5 shows the vulnerability of brick masonry and reinforced concrete frame buildings according to their age of construction. There is a clear trend towards greater light or moderate damage in the older brick masonry buildings, with over half of the pre-1920 buildings suffering some damage. Conversely, 79percent of the post-1960 buildings escaped without damage. The pattern is less consistent for damage ≥D2 and ≥D3, where in the latter case the buildings constructed between 1940-1960 appeared to be most vulnerable. No buildings of post-1940 construction suffered partial destruction (D4), but one pre-1920 building and four 1920-1940 buildings fell into this category. For reinforced concrete frame buildings, the proportion of buildings with some damage is about 20percent for all ages, but surprisingly no buildings constructed in the period 1920-1940 suffered more than slight damage, whereas for more recent buildings there was significant moderate or heavy damage, the effect being noticeably worse in the buildings of recent (post-1960) construction, possibly due to reduced conservatism in design codes.

Finally, Figure 6.6 shows the effect of building height on the distribution and extent of damage. For brick masonry buildings, those with more than one storey showed about twice as much damage overall than the single storey buildings with consistency for damage ≥D1 for buildings with 2, 3 and >3 storeys. For buildings with moderate or heavy damage (≥D2), buildings with >3 storeys were about 4 times more vulnerable than single-storey buildings and about twice as vulnerable as buildings with 2 or 3 storeys. None of the single storey reinforced concrete frame buildings suffered any noticeable damage, but about 20percent of the multi-storey buildings showed at least slight damage. This value is about half that for brick masonry buildings. Single storey timber framed buildings were found to be only half as vulnerable to damage ≥ D2 as compared with two-storey timber framed buildings. One two-storey building was damaged to level D4, but no single-storey buildings were observed in this category, although in the latter case 13percent were damaged at level D2.

### 6.2.4 Discussion of Results

The detailed photographic survey of building damage in two of the worst affected areas of Newcastle has revealed significant and consistent trends in the variation of vulnerability and risk of earthquake damage for three important types of construction. The results have been further analysed for the effects of building usage, age and height. A summary of the results has been presented in Figure 6.7, which shows the locations of the 625 buildings on which the survey was conducted, together with the level of damage D0-D4 for the three main types of construction. The difficulties of using relatively small statistical samples of building damage for multi-variate analysis have been well documented (Coburn 1986). It is clear that in Newcastle the most vulnerable building stock (with the highest damage levels) were those with a number of key characteristics: the older buildings (pre-1920) of two or three storeys, constructed of unreinforced brick masonry with some commercial use. It is difficult from a sample of 625 buildings to assess which of the key characteristics are the determining factors in the building stock vulnerability. Hence it is impossible to quantify the effect of any individual variable, for example age, independently of the other variables, such as the construction type. Indeed the individual variables tend to co-exist in certain classes of building: two-storey solid brick masonry for commercial buildings typifies pre-1920 construction, and reinforced concrete structures mostly post-date the 1940 age classification, so it is impossible to examine the effect of a characteristic like age on vulnerability independently of the construction type variable.

Similarly the analysis of damage by storey height (Figure 6.6) is not independent of the analysis of damage by age (Figure 6.5), since the same data set is being looked at in a different way. The age parameter in masonry buildings appears more influential in determining damage than storey height. The variation in storey height across the sample of masonry buildings is limited (Figure 6.3), with almost half the sample being of two storeys. Therefore, samples may be too uniform for the effect of that variable to be fully apparent. There is a slight indication that taller reinforced concrete buildings have suffered higher damage levels (Figure 6.6) but with sample sizes of around 30 buildings in each storey/height classification for reinforced concrete buildings, the difference in damage levels is only two or three buildings. These buildings could well be damaged due to other factors (design oddities, construction flaws and so forth) and it is difficult to be confident that the height of the building has actually influenced its damageability in this event.

As mentioned above, it is clear that the types of buildings that are characterised as load bearing solid wall masonry, of two storey construction, built around the turn of the century that form the bulk of the older commercial buildings in the town centre are considerably more vulnerable that the other building types found in Newcastle. From the observations of the damage this could be a feature arising from a number of factors including:-

a) The decay of masonry over the century. Subsidence and weathering have tended to induce cracking that increases the vulnerability of masonry.

b) The construction technique and materials used at the time. The solid wall built as a double brick thickness, often with little bonding between the two brick skins and with lime mortars, appears prone to cracking and spalling under vibration.

c) The architectural styling and building form of the commercial buildings of early 20th Century Newcastle may have accentuated the damage. These buildings consist of street-fronted, terraced, large-span ground floors, with brick ornamentation that may suffer damage, and a large number of parapet walls on street frontages.

Any or all of these reasons may have contributed to the vulnerability of this particular building type. This building type constitutes almost a third of the total building stock in the most damaged areas of the town. Another third is masonry commercial buildings built later. Even modern masonry buildings have higher damage levels than pre-war concrete frame structures or timber-framed buildings of any type. Within masonry buildings as a class there may be a slightly higher damage levels discernable in cavity wall construction than in solid wall construction, particularly at the higher damage levels (D2 to D4). This might be expected from the point of view of structural stability.

The distributions of damage observed in Newcastle appear to conform with distributions of damage to similar building types observed in other earthquakes elsewhere (Coburn et al 1990), as shown in Figure 6.8. With about 10 percent of brick masonry buildings damaged to D3 or worse, as in this earthquake it is unusual to find any collapse (no damage D5), and typical distributions consist of D4 of a few percent and D2 or worse of between 20 to 30 percent. One major difference between the distributions of brick masonry damage recorded in Newcastle compared with distributions surveyed elsewhere is that in the latter case the average percentage of at least D1 corresponding to damage distributions of D3 or worse of 10 percent is 70 percent. In the Newcastle photographic survey, the proportion of at least D1 is less than 40 percent. One explanation of this is that the threshold of damage for D1, the hairline cracks normally noted on building survey forms, may be less discernable from photographs and hence the photographic interpretation of damage may have missed a number of minor cracks.

The relative levels of damage between reinforced concrete buildings and brick masonry structures also appears consistent with other relative damage levels (although the relative vulnerability from the comparative performance of unreinforced brick masonry structures and reinforced concrete frame structures without seismic design has considerable scatter, using data collected worldwide). The difference in vulnerability may be slightly less than worldwide averages; in Newcastle 12 percent of reinforced concrete frame structures have suffered damage level D2 or worse, compared with 21 percent of brick masonry buildings. In locations elsewhere in the world where 21 percent of brick masonry buildings have suffered damage D2 or worse the average of damage at level D2 or worse to reinforced concrete frame structures without seismic design is about 8 percent, but the scatter is large (Coburn 1986). If this observation is valid then either brick masonry structures are stronger in Newcastle than elsewhere or the reinforced concrete structures are slightly more prone to low damage levels than elsewhere.

The general distributions of damage, particularly to brick masonry buildings suggest that the damage level is at the low end of MSK Intensity VII ("Many (20 to 50 percent) brick buildings suffering moderate damage D2"), see Figure 6.8. Fitting the Newcastle distributions to the average Gaussian vulnerability functions derived from the Martin Centre database, suggest a best fit at around an intensity level of 6.7 $\psi$ units (Martin Centre Intensity Scale). Damage distributions of this order surveyed around strong motion instruments, such as at Bisaccia in the Italian earthquake of 1980 (16 percent D2 or worse for brick masonry structures) correlate with a peak ground acceleration around 0.1g (Coburn et al 1982). It should be noted that this comparison with Bisaccia is for the effects of an earthquake at distance from a larger magnitude event rather than, as in Newcastle, close to the epicentre of a smaller magnitude event, so other characteristics like duration, frequency content and vertical components may be significantly different.

## 6.3 General Damage Survey

### 6.3.1 Methodology

As described in Section 2.3 a widespread damage survey was carried out, generally by vehicle, where buildings with visible damage were marked on the 1:4000 scale aerial photo montage. In the Hamilton area and central business district all streets were surveyed. In the adjacent areas generally alternate streets were surveyed due to time limitations. Two levels of damage were recorded as follows:

| | | |
|---|---|---|
| Moderate Damage | Yellow | Clear visible damage that is repairable and is unlikely to cause severe injury. Collapsed chimneys were included in this category. |
| Heavy Damage | Red | Partial collapse of the structure sufficient to cause severe injury (if people had been within or adjacent to the structure). |

Generally the Yellow category is similar to damage level D2 described previously and the Red category similar to damage level D3 or greater. The primary intention of this survey was to establish the density and extent of the earthquake damage. It must be noted, however, that the ground survey is not as thorough as the detailed photographic survey and the risk of not identifying damage is much greater. The observed damage must therefore be considered as a lower bound and it is likely that considerably more damage occurred.

### 6.3.2 Distribution of Damage

Figure 6.9 shows the locations of buildings with moderate or heavy damage observed in the general survey. While it can be seen that the areas of heaviest damage are in the vicinity of the detailed photographic survey there is also considerable damage in the areas of The Junction, Tighes Hill and Broadmeadow. For comparative purposes the percentage of residential buildings damaged has been calculated and is shown in Figure 6.10. These levels of damage are much lower than that observed in detailed photographic survey and are believed to reflect a combination of:

i) poorer data in that more damage is missed;
ii) residential districts had a much smaller proportion of vulnerable brick masonry buildings compared with the commercial areas (Figure 6.1), and greater than 50 percent of residential buildings are of timber frame construction which showed significantly better earthquake resistance (Figure 6.4);
iii) less damage occurred away from principal streets.

In areas away from principal streets the building type is predominantly single storey with timber frame dominating. However, in Silsoe Street, Mayfield, there was a concentration of 1930's residential single-storey masonry construction which suffered significant damage.

## 6.4 Other Damage Survey Data

The Newcastle City Council inspected most buildings in the days after the earthquake (see Section 7.2). They employed a four-colour coding scheme as follows:

Red - severe damage, immediate public danger
Amber - severe damage, possible danger
Blue - damaged, but habitable
Green - minor damage

In the follow-up visit made by Colin Taylor in June 1990, EEFIT obtained copies of the colour-coded map for the area shown in Figure 6.10. Their Red classification is similar to that adopted by EEFIT and their Blue and Amber correspond approximately to the Yellow classification used by EEFIT. It must be noted that there will be considerable variability in the assessments between different inspectors and, as for the EEFIT survey, not all damage is reported. In addition damage to schools, colleges, churches and so forth was not reported on the map. Percentage damage levels for the residential areas using the Newcastle City Council data are shown in Figure 6.10 In areas where comparisons between the EEFIT and Council data are shown (for example in the more heavily damaged areas of Hamilton and Newcastle City centre) there is good agreement based on the Red and Yellow/Amber categories.

The Newcastle City Council map showing the locations of all residential buildings damaged in the Red and Amber categories is reproduced in Figure 6.11. The pattern of damage is similar to the EEFIT survey, except in Merewether which was not surveyed by EEFIT, and in Mayfield where the density of damage recorded by EEFIT was much less than that indicated in Figure 6.11.

The Newcastle City Council have also published a series of plans for the heaviest affected areas of Hamilton and the Newcastle central business district. These were carried out for principal streets only and are reproduced in Appendix C. No analysis of these has been carried out by EEFIT.

## 6.5 Factors Influencing Spatial Distribution of Damage

Both Figures 6.9 and 6.11 show that the heaviest concentration of damage was in Hamilton and the Newcastle central business district. The detailed damage survey has demonstrated that the weak masonry structures that suffered the worse degrees of damage, and to some extent the reinforced concrete structures that also exhibited the more severe categories of damage, are predominantly commercial building types. The residential buildings that populate much of the region affected by the earthquake are primarily the less vulnerable timber framed construction and have consequently suffered much lower damage levels. The concentrations of damage along Hunter Street in Newcastle and Beaumount Street and Lawson Street in Hamilton chiefly indicate the older commercial streets of the original settlements where the concentrations of the most vulnerable building stock are found.

How the locations of worst damage relate to the position of the epicentre is not clear. The teleseismic instrumental location for the epicentre is some 10 to 14km from the worst-damaged areas. This location determination is to some degree uncertain and could have been closer to the most damaged areas. However, the central business district of Newcastle is the greatest concentration of masonry (particularly older masonry) buildings for at least 30km around, and it is likely that any earthquake occurring in this region would have caused higher damage levels in the older central business district of Newcastle than in the residential areas around it. In the detailed building survey, the damage distributions of Beaumont Street and Lawson Street in Hamilton are almost identical to the damage distributions of similar building types in Hunter Street in the central business district. The intensity appears relatively uniform across the four kilometres covered by the detailed survey and it is impossible to determine from the damage plots (Figures 6.7 and 6.9) any particular concentrations of damage that might mark the focus of a localised earthquake epicentre.

There may however be additional influences on the spatial distribution of damage, and these are further considered in the following sections.

### 6.5.1 Soil Response Effect

The Institution of Engineers Australia (1990) report concludes that soil effects were largely responsible for the observed damage distribution.

Figure 6.12 shows the damage distribution from the general survey superimposed on the soil thickness diagram. Generally, there is seemingly no correlation between damage distribution and soil thickness. The damage recorded in the detailed photographic survey in Hunter Street in the City centre (Figure 6.7) seems to confirm this. A fairly uniform distribution of damage was observed, even though the soil thickness varies from near zero at the eastern end to about 30m at the western end. The extent of damage in Hunter Street is comparable to that in Beaumont Street, Hamilton where the soil thickness is between 10 and 20m. In these areas, with a high concentration of the more vulnerable multi-storey older brick masonry buildings, structural factors probably outweighed the influence of variable ground motion intensities due to site soil effects. To attempt to remove this effect and work with a more homogeneous dataset, the distribution of percentage damage levels to residential buildings only was superimposed on the soil thickness diagram, as shown in Figure 6.13. Here it is apparent that in most affected areas the soil thickness is between

0 and 10m except in the Georgetown area, where there is a high level of damage on thicker soils.

In order to study further the effects of soil response and to provide analytical data for evaluating possible site amplification effects, a series of analyses has been carried out using the one-dimensional (1-D) program SIREN developed at Ove Arup and Partners (Heidebrecht et al 1990). In this program, the soil deposits overlying rock are modelled as 1-D layered systems with propagation of shear waves only in the vertical direction. The model is non-linear, and SIREN solves the problem in the time domain using the finite difference method. The aim is to determine the surface ground motions resulting from a specified bedrock earthquake motion, and hence to study the amplification or attenuation of the peak parameters and the variation of the frequency content of response spectra computed for the bedrock and surface motions, as a result of resonance effects in the soil layer(s).

Five of the set of borehole data supplied by Coffey and Partners have been used in the analyses of site response effects. The boreholes are illustrated in Figure 6.14 and their locations are shown in Figure 6.12. For the site response analyses an earthquake time history with a similar response spectrum to those shown in Figure 4.5 was selected and used to represent a rock surface motion appropriate to the Newcastle earthquake. The selected rock motion is taken from the Honshu, Japan earthquake of 5th April 1966, and was measured 4km from the magnitude 5.5 earthquake. The record was scaled by a factor of 0.45 in order that its response spectrum (thick solid line in Figure 6.15) approximates the probable range of response spectra indicated by the shaded region in Figure 6.15 (the range of response spectra reported earlier in Figure 4.5). The scaled record has a peak acceleration of 0.12g and peak velocity of 0.05m/s. Dynamic soil properties were derived from SPT results and soil descriptions as given in Heidebrecht et al (1990) and Henderson et al (1990).

The resulting calculated ground surface acceleration response spectra for 5percent structural damping are shown in Figure 6.15 over the range of fundamental building period (0.1-1.0 seconds) relevant to buildings in Newcastle.

It is clear that the shallow soil deposits (boreholes B and E) amplify the short period motion (in the range 0.1-0.3 seconds) by factors of 2½-3. Low-rise structures of up to 3 or 4 storeys would therefore be expected to have shown a greater level of damage in the areas underlain by these soils. As the soil thickness increases (boreholes A and D) the amplification is seen to reduce in magnitude but extend over a wider period range. For the deepest deposit (borehole C) a slight attenuation compared with bedrock motion is indicated. These results agree reasonably well with the trends observed in Figure 6.13 but there is too large a variability in the data to draw any firm conclusions.

The calculations show that unlike other earthquakes such as Mexico City in 1985 (EEFIT 1986) and San Francisco in 1989 (EEFIT 1991), deep soft soil deposits do not necessarily amplify the bedrock ground motion to any significant extent. This is confirmed by the results of detailed site response analysis by Heidebrecht et al (1990) who conclude that spectral amplification is most significant when the seismic excitation has substantial energy in the region of the site period (which for deep soft sites could be in the order of 1-2.5 seconds). The very high site amplification effects recorded in Mexico City (factors of about 6) and San Francisco resulted from far field earthquakes (epicentral distances of about 400km and 100km, respectively) where the attenuated bedrock motion had a frequency content shifted towards the longer periods. For the lower magnitude, near field earthquake in Newcastle the energy would almost certainly have been concentrated at the shorter periods (Figure 4.5) and would therefore tend to excite site resonance effects in the shallower, stiffer soil deposits as indicated in Figure 6.15.

Hence it is concluded that the suggested correlation of greatest damage with the areas of deep alluvial soil deposits in Newcastle (Institution of Engineers Australia, 1990 and Brunsdon, 1990) is not substantiated by the observed damage distribution or by the analytical results presented here. The actual situation is clearly more complex with the possibility of some quite localised ground motion amplification effects in the shallower and/or stiffer soil deposits. The widespread variation of soil types and thickness in the Newcastle area (Figure 3.5) together with the analytical results presented herein show that the attempt to relate damage distribution simply to soil depth or type is unrealistic and unlikely to lead to useful or accurate results. The overall result of this study is that whilst the analytical results are useful for indicating localised variations of ground motion, amplitude and frequency content, the complexity of the problem makes it very difficult to develop generalised conclusions regarding the significance of site effects in the response distribution. However, it is clear that the deeper, softer sites are unlikely to have demonstrated any significant amplification effects in this earthquake.

### 6.5.2 Effects of Mining

Sections 3.3 and 3.4 refer to the geology of the Sydney/Newcastle area, and the coal extraction in the borehole seam beneath the City of Newcastle.

It is noticeable that in several earthquakes damage has been associated with mining activity. In the Liege earthquake this was associated with settlement induced by mining (EEFIT, 1984). In Newcastle there has been no evidence of damage to mines currently in use. If, however, the pattern of damage is plotted together with the area of coal extraction as in Figure 6.16 there appears to be a clear correlation between damage and the edge of mining activity. This may be due to motion being concentrated at the perimeter of the coal extraction. An alternative explanation is sociological in that the mine perimeters tend to be aligned along major roads. This is also where the most vulnerable buildings are situated and consequently there may be purely historical reasons why the damage is concentrated as it is.

### 6.5.3 Discussion

The reasons why the most severe earthquake damage occurred apparently 10 to 14km from the epicentre is not entirely clear, although it must be recognised that there is still significant uncertainty in the epicentral location. It is certain that the most vulnerable buildings are located principally in these areas and other possible explanations include some geological feature which may have directed the strong ground motion towards the heavily damaged areas such as Hamilton and the Newcastle central business district. As in most earthquakes, the most likely explanation involves a combination of these effects, since rarely is it obvious why certain damage distributions are observed. The exception to this is when soil amplification effects are dominant, as in Mexico in 1985 and San Francisco in 1989. In both cases there was a clear, unambiguous correlation between building damage and soil type, but as indicated in this study, this was not the case in the Newcastle earthquake.

## 7.0 SOCIAL RESPONSE TO THE EARTHQUAKE

### 7.1 Preparation and Initial Response

Prior to the earthquake, Council leaders and surveyors had been sent on Disaster Preparation courses run by the State Government of New South Wales. These courses dealt with the threat to life and property of floods, hurricanes and earthquakes. Whilst the risk of a damaging earthquake occurring close to an urban centre like Newcastle was considered small in comparison with other natural dangers, such courses nevertheless contributed to the effective response of the authorities to this event, which revealed a high level of organisation amongst the Police, rescue and recovery services. This was despite the fact that the event occurred during a holiday period, with many key people away from the City when the earthquake struck. For example, on the day of the earthquake Newcastle City Council was manned by a 'skeleton' staff, with all senior officers on leave. This was accepted as normal practice at that time of the year.

Newcastle City Council does not have a disaster plan, although such plans exist for the Police and other emergency services. The State's Special Emergency Services (S.E.S.) was called in to assist with general policy in handling and coordinating the immediate response of the various authorities, but did not at any stage declare an Emergency Situation. The Council and Police coordinated within a few hours a policy of closing off to the general public the badly damaged areas such as Beaumont Street in Hamilton and the City's central business district (see Section 7.2). Access was allowed only to emergency personnel in the first few days, and thereafter to residents and those with businesses within the cordoned off areas. Entry to buildings was controlled by the Council Surveyors and Building Inspectors who drew up a procedure of building damage assessment in order to designate those buildings which were unsafe for entry (see Section 6.4). The danger of aftershocks accentuated the importance and urgency of this work, which was coordinated by the City Council Surveying Department. The advice of earthquake experts from the Australian Seismological Centre in Canberra indicated that at least one aftershock of magnitude 4-4.5 could be expected within 48 hours of the main shock. This influenced decision making on matters such as building inspection and demolition.

The Authority's initial response to the earthquake was therefore understandably cautious, and their priority was to act conservatively with respect to the maintenance of public safety in view of the lack of previous experience of such events. The Australian Army sent four teams of engineers on the day following the earthquake (29th December 1989), and the personnel included structural engineers trained in methods of building safety assessment. Buildings Surveyors and Senior Engineers were sent by other cities within 2 or 3 days of the earthquake to assist with the initial damage assessment exercise. For example, Brisbane sent two engineers to join the Newcastle City Council operations for a period of 3 weeks, and Sydney sent one engineer for an indefinite period.

A Control Headquarters was established on the 6th floor of the Council Administration Centre, King Street, Newcastle utilising the existing management structure of the Council's Health and Building Services Division (Figure 7.1). The headquarters incorporated liaison with the Police and other emergency services. Harold Stuart, the Director of the Health and Building Services Division, was appointed on the day following the earthquake as the Council's Coordinator. The basis of his role was to instigate measures to ensure public safety and provide an orderly and efficient restoration of access to the affected areas of the city. He was also to act as the liaison point with the Police.

On 30th December a professional media organisation was drafted on to the Council staff to issue regular releases and organise press conferences, in order to free Council staff to tackle the task of building inspection unhindered.

## 7.2 Building Inspection Procedures

Within two days of the earthquake, a largely voluntary force of local structural engineers, architects and building inspectors began safety assessments of buildings and houses. This work was coordinated by the Newcastle City Council, who responded immediately to the earthquake to set up a Property Information Database to monitor the damage assessment procedures. Similar safety inspections were carried out by the Public Works Department for hospitals, schools and other public buildings, and by Federal Agencies for Federal Properties.

The Property Information Database took four days to establish, during which time Newcastle City Council received 6-7,000 enquiries to request assistance in damage assessment and safety evaluation on private properties. On the basis of the database, it has been estimated that roughly 10,000 buildings (about 10 percent of the total building stock) were damaged as a result of the earthquake, although much of this damage was relatively minor such as cracking of plaster and ceilings, and so forth. It has been reported (Financial Times 1990) that about 500 buildings (mostly residential) were partly or completely demolished in the weeks following the earthquake.

In addition to the Army rescue teams and structural engineers mentioned in Section 7.1, Council Building Surveyors were used to increase the Inspection Teams to a total of eight by 30th December, two days after the earthquake. These teams carried out a brief initial inspection of all buildings with significant damage, and advised the Council on whether or not a full structural investigation was required. The inspections were initiated in the central business district of Newcastle, within the outermost closed off perimeter indicated in Figure 7.2. In total there were twenty building surveyors to provide this initial advice, together with twelve structural engineers assigned to look at special problems arising in the more difficult cases.

A procedure involving coloured tagging to indicate the status of buildings with moderate to severe damage was begun four days after the earthquake. Two categories were employed, as follows:

Yellow: Likely structural damage. Building subject to assessment by independent engineers.

Red: Building severely damaged and dangerous to enter. Liable for part or total demolition.

Examples of the notices posted on building entrances, designating one of the above two categories, are shown in Figures 7.3 and 7.4. In both cases, the authority to carry out demolition or reconstruction work was vested with the Town Clerk. Entry to such buildings was restricted to owners and/or lawful occupiers, together with authorised structural engineers and inspectors.

The system of closing off the road and footpath access to certain parts of the city centre and inner suburbs, as described in Section 7.1, was carried out primarily as a safety precaution in view of the high proportion of damaged buildings in certain areas. These presented in many cases severe danger resulting from partial or total collapse, particularly in view of the warning of likely aftershocks. By January 2nd, 5 days after the earthquake, it was apparent that some movement was still occurring, causing new or additional damage to buildings. This was of concern, particularly with regard to the objective of making the central business district safe for general access by January 8th. The restricted access also enabled rapid decisions to be made on those buildings requiring demolition of part or all of the structure, which proved in some cases to be a controversial matter with protests from building conservationists and the Newcastle Heritage. Examples of this were the George Hotel in the central business district, a five storey unreinforced brick masonry building which had to be demolished nine days after the earthquake (Plate 7.1), the three storey Newcastle RSL club on the corner of King Street and Perkins Street in the central business district

(Plate 7.2), and a cinema in Perry Street. Pressure from Heritage groups built up from the impression that wholesale random demolition was taking place, but in fact only eight major buildings were lost, including the Newcastle Working Mens' Club and The Junction Motel, as discussed in Section 5.3 of this report (see Plates 5.61-5.64).
The decisions on barricading individual or groups of buildings, and for carrying out repair and demolition work, were assisted by two further forms or notices issued by the Newcastle City Council. Figure 7.5 shows the Rapid Evaluation Safety Assessment Form used in the first few days after the earthquake to establish the Property Information Database described above and to determine the appropriate damage level notice (Figures 7.3 and 7.4). The form was completed by an authorised building inspector, and identified the building usage, number of storeys, damage condition (under 7 categories, see Figure 7.5), and Council works action required such as propping unsafe awnings and barricading footpaths or road access. All repairs or demolition also had to receive Council authorization using a form such as that shown in Figure 7.6.

Once the danger of damaging aftershocks had subsided and cases of critical or dangerous damage dealt with by demolition or emergency repair as appropriate, the Council was able to review the extent of the closed perimeter around the Newcastle central business district, in order to re-establish general access to parts of the city so that businesses and offices could resume their activities as quickly as possible. The central business district was re-opened in a staged programme as illustrated in Figure 7.2, with reduction of the size of the restricted area as districts were cleared for re-occupation. The central business district was fully re-opened (except for certain street closures) on 8th January, 11 days after the earthquake, when all safety checks and essential demolition had been carried out. Further closures in the Hamilton area (Figure 7.2), particularly the heavily damaged Beaumont Street (see Section 5), were in force until after the EEFIT team left Newcastle on 11th January.

## 7.3 Aid and Recovery Programme

An earthquake information centre was established on 15th January 1990 and operated until 23rd March 1990. The centre provided information from all the appropriate authorities and other services such as architects, solicitors, structural engineers, builders and the insurance industry. In co-operation with the Department of Family and Community Services, the centre answered thousands of enquiries to those people requiring assistance and advice.

A major consideration in addition to the procedures for assessing public safety as described in Section 7.2 above was the need to re-open business and commercial districts as quickly as possible, even if demolition of dangerous buildings was required in order to facilitate the process of returning the City to normal operation. It was recognised by the authorities soon after the earthquake that the biggest loss to the City would be the danger of small businesses being forced to close or be severely interrupted, especially in view of the poor economic climate which pervaded the city even prior to the earthquake. In areas such as Beaumont Street, Hamilton, closure even for 3 or 4 weeks could have caused several businesses to fold, since many were experiencing severe economic difficulties even before the earthquake happened. This risk was minimised by the rapid and effective response to the disaster made by the authorities, who regarded the re-establishment of normal business and commercial activity as a high priority. At the time of writing this report 30,000 insurance claims had been filed and estimates of the total cost of reconstruction range as high as $A1.2bn (Financial Times 1990).

The human cost of the earthquake has also taken its toll. Up to 3000 people were displaced from their homes, mostly on a temporary basis, but 6 months after the earthquake it has been estimated (Financial Times 1990) that 500 houses are still uninhabitable and 500 more will have to be demolished. As a result, there are 1000 families sharing accommodation, with all its associated social problems. Trauma and community breakdowns were dealt with in the aftermath of the earthquake by special counselling and advice services set up by the authorities, and financial aid was sought from a national appeal launched by John McNaughton, the Lord Mayor of Newcastle. Federal and state government assistance has poured into the reconstruction programme, but the cost of the disruption to an already ailing industry and commerce is impossible to cover in this manner. All sectors of local industry have reported (Financial Times 1990) that they are now attempting to make up for the losses suffered in the aftermath of the earthquake, but according to the Lord Mayor it will be 5 years before reconstruction is completed, and perhaps a decade before the city fully recovers.

In conclusion, the authorities response to this earthquake was commendably swift and effective, and was quick to identify the major issues involved and to draw up procedures for dealing with them. A number of important lessons have been learnt, which if recognised further afield (not least in the UK) could give vital information which would be useful in dealing with future events of a similar nature. If this is the case, then the unfortunate experiences of Newcastle could prove to be a valuable learning ground for the establishment of effective earthquake preparedness and recovery programmes.

# 8.0 DISCUSSION

## 8.1 Lessons on Building Construction

### 8.1.1 Overview

The most important lesson learned from the extensive damage caused by the Newcastle earthquake is the importance of proper detailing required to ensure adequate earthquake protection. This is applicable to buildings of all types of construction, but particularly to unreinforced brick masonry buildings. About 37 percent of the brick masonry buildings in the worst affected areas were damaged in the earthquake (Figure 6.4), with 21 percent suffering moderate damage such as partial collapse of parapets, awnings and/or chimneys, and severe cracking of walls or cladding. Much of this damage could have been prevented or reduced by the more widespread use of wall ties for cavity construction and stronger methods of supports for awnings. Improved detailing of such features and other unsupported brickwork would have greatly reduced the risk arising from falling masonry in this earthquake. Three people were killed by falling masonry, and if this earthquake had occurred in a normal working week and not in a holiday period, the loss of life could have been many times worse.

Collapse of parapet or gable ends was the most common form of failure in both domestic and commercial unreinforced masonry buildings, and in almost all cases this failure was the result of inadequate tying to the structure, either because there was a lack of ties or because they were corroded. Many masonry walls, particularly long, laterally unsupported walls showed evidence of out-of-plane deformation where the nominal lateral support provided by roof trusses or ceiling beams supported by these walls was clearly inadequate. Much better performance was observed where there was physical anchoring of the ends of the roof trusses or beams to the top of the masonry walls.

Some structures had been weakened with respect to earthquake resistance by the effect of alteration such as the anchoring of steel beams into masonry piers forming part of a wall system. In these cases, considerable damage was observed at the steel beam/masonry intersection. The effect of masonry infill panels in modern reinforced concrete or steel framed structures was generally beneficial, since even though cracking had occurred in many instances (see Plate 5.46), the infill had protected the structural frame from significant damage by the addition of considerable strength and energy dissipation to the structure.

Lack of proper maintenance and the consequent deterioration of older brick masonry buildings played a significant role in the risk of earthquake damage. In the central business district many of the buildings are exposed to a sea atmosphere since the city is situated close to the Pacific Ocean and to the Hunter River mouth. Evidence of loss of mortar in brickwork joints and of stone or masonry deterioration was prevalent, together with corrosion of brick ties in cavity wall and veneer construction. Problems with reinforcement corrosion even in modern reinforced concrete buildings have also been reported, even prior to the earthquake.

Masonry chimneys in all forms of domestic construction showed widespread failure above the flashing at roof level. Numerous chimney tops had to be dismantled for safety reasons, and many others need repair.

### 8.1.2 Revision of Building Design and Construction Regulations

The State and National Building Codes refer for structural detailing to specific Australian Standards, such as the concrete code AS 1480, the steel code AS 1250 and the loading code AS 1170. These state-of-the-art documents have each been compiled by a committee of experts using all available information. Even though Newcastle was zero-rated for earthquake loading in the SAA Earthquake Code AS 2121 (1979), as detailed in Appendix A, very little major structural damage occurred to modern engineered buildings (Section 5.3). This is an important fact when assessing the adequacy of modern building codes.

The revision of the SAA Earthquake Code which was in progress even prior to the Newcastle earthquake will ensure that greater attention is given to detailing to enhance structural continuity in new structures, particularly in joint design. It is also important that the code should give greater attention to the provision of lateral support, particularly for unreinforced masonry construction. This may involve tightening up the requirements for wall ties, and the ductility requirements for both reinforced concrete and steel structures.

For existing buildings and domestic construction (which at present is not covered by the design code except for multi-storey isolated dwellings, see Appendix A.1), the problem is more complicated. Retrofitting of existing buildings is expensive and in many cases the age and construction (particularly for masonry buildings) makes such procedures uneconomical. At a conference on the Newcastle earthquake held at Newcastle University on February 15-16, 1990, and sponsored by the Institution of Engineers Australia and the Royal Australian Institute of Architects, proposals were made for tying parapets to roof trusses in order to prevent their collapse under earthquake loading. Two such proposals are illustrated in Figure 8.1. It is likely that these or similar proposals will be included in the detailing requirements of the revised earthquake design code.

The conference held at Newcastle University brought together over 400 structural and civil engineers, architects, seismologists and local government officials, and gave an indication of the interest in earthquakes and earthquake engineering which has been generated throughout Australia by the Newcastle earthquake. Topics discussed (Institution of Engineers Australia 1990) included the behaviour of buildings and other structures during earthquakes, post-event assessment of earthquake damage, requirements for new buildings including risks and insurance, maintenance and repairs of existing structures, and the special requirements of heritage buildings. The conference reached general agreement that existing buildings should have no retrospective upgrading requirements for their continued use, with three possible exceptions - buildings which have suspended awnings or parapets or other projections likely to be damaged by earthquake forces, buildings and structures having a possible post-disaster function (e.g. hospitals and fire stations), and buildings such as schools where significant loss of life might occur in an earthquake. There was also good general support for the suggestion that Australian-wide minimum design requirements for earthquake resistance of buildings should be established.

The outcome of these active and on-going discussions should be a significantly revised and improved SAA Earthquake Code, with an emphasis on improved detailing for features known to be vulnerable to lateral forces. Provided the provisions are formulated on a rational and realistic basis, the additional cost of such vital design features should be minor compared with the benefits arising from increased safety and earthquake resistance, for the prevention of damage and (in moderate or severe earthquakes) possible loss of life.

### 8.1.3 Repair and Reconstruction in Newcastle

A lesson learned as a result of the Newcastle earthquake is the importance of careful control applied to the design of repaired or re-instated features on buildings damaged by the earthquake. The Newcastle City Council proposed in January 1990 (at the time of the EEFIT visit) to maintain records of the rebuilding programme, and insisted that as part of this process all repairs and new construction should be subjected to stringent checking through the standard building application procedures. For this purpose, the Council assigned 20-25 Planning Officers to cope with the extra workload. It was stated (Stuart 1990) in this respect that some repairs to buildings carried out following the 1925 earthquake (see Section 4.1) had been poorly carried out and had consequently failed again in the 1989 shock.

A report published on 19th March 1990 by the Newcastle Town Clerk's Co-ordination Committee on Matters Relating to the Health and Building Services Divisions (Newcastle City Council 1990) outlines the Council policy on interim requirements for the design of earthquake resistant buildings. It recognises that whilst regional and national standards and legislation are to be reviewed in the light of the Newcastle earthquake (see Section 8.1.2), it will take one to two years for such amendments to be effected. The report, therefore, reflects the urgent need to set interim requirements to provide guidelines for construction of new buildings in the Newcastle area, and to address the upgrading and maintenance of existing buildings. This urgency was highlighted by the amount of repair and reconstruction work being undertaken in the aftermath of the earthquake, without formal guidelines in regard to earthquake-resistant construction.

The report noted that existing codes/standards have been set for brickwork construction, mortar and wall ties and strict adherence to these should provide a substantial level of earthquake resistance, particularly in relation to dwelling construction.

For new buildings, the report proposed that the interim requirements should be based on AS 2121 - SAA Earthquake Code for buildings in Zone A with the exception of post disaster buildings whereby it is proposed that construction will be in accordance with Zone 1 requirements (see Appendix A).

The implications of these proposals are negligible for buildings in Zone A in regard to ductile construction (having ability to withstand inelastic deformations). However, non-ductile buildings (in Zone A) and post-disaster buildings (in Zone 1) will be of a substantially higher standard than currently required.

With regard to domestic housing no specific requirements were proposed for single storey dwellings. However, the report emphasised that strict compliance and supervision of current building regulations is necessary. It was also considered that domestic housing in excess of one storey requires specific design consideration particularly in relation to high masonry walling, and a design provision has been included in the recommendations.

For existing buildings, the extensive damage to brick parapets and awnings of commercial buildings has highlighted the need for upgrading of hazardous buildings to ensure public safety in any future earthquake. It was proposed that all buildings considered to be potentially hazardous should be examined and upgraded where necessary. Upgrading is not a simple task for either Council or the building owners and a reasonable period of time needs to be allowed. Five years has been suggested, but this should be flexible based on the circumstances of each situation.

In summary, the Newcastle Town Clerk's Report recommends the following:

1. In accordance with the Local Government Act, 1919 the Council should adopt the following interim requirements for the design of new buildings and the upgrading of existing buildings within the City of Newcastle.

   Certification of such design shall be required by a qualified practising structural engineer at the building application stage.

   a) New buildings, additions and alterations to existing buildings excluding detached single dwellings and multiple dwellings side by side and not on top of another, shall comply with Australian Standard 2121-1979 SAA Earthquake Code, Zone A, with the exception of essential facilities (post-disaster buildings) which shall comply with the requirements for buildings in Zone 1.

   b) Single and multiple dwellings of masonry construction and in excess of one storey construction shall be designed strictly in accordance with AS 1640-SAA Brickwork Code.

   c) Where considered necessary for public safety, existing buildings incorporating repairs and restoration shall be strengthened to resist earthquakes to a minimum standard as determined by Council in the particular case.

2. A further report is to be provided by the Director of Health and Building Services in regard to the supervision of domestic housing pertaining to the strict adherence to current regulations and standards.

3. An earthquake hazard mitigation programme is to be implemented in regard to masonry parapets and awnings adjacent to or over public areas. Such structures where necessary are to be upgraded and structurally certified within five years.

4. Regular reports are to be provided to Council in regard to legislation review and the earthquake hazard mitigation programme.

The Report also issued detailed guidance on design and structural considerations for effective earthquake resistance, up-grading of existing buildings, general requirements for wall ties, mortars and damp-proof courses, and regulations for building approvals. These have been reproduced in Appendix D, and represent a thorough treatment of the subject of repair and reconstruction in Newcastle to prevent such extensive damage occurring in any future earthquakes.

## 8.2 Implications for Areas of Low Seismicity such as the UK

The Newcastle earthquake has important implications to many areas of low seismicity such as the UK. In global terms, it was of moderate magnitude (5.6), and given the present understanding of earthquake source mechanisms it is possible that earthquakes of this size could occur, albeit with low probability, in virtually any location. Generally, however, such shocks cause little damage to engineered structures because of their remoteness from populated areas. The exception with Newcastle is that the earthquake occurred near a city, which having no special provision against earthquakes was vulnerable to seismic ground motion.

The earthquake represents a graphic demonstration of what would happen in any other similar large town or city which has been constructed with no special measures incorporated for earthquake loadings. The lesson for these sites is clear. Structures or components with easily identifiable vulnerable features will fail. Engineers need therefore to evaluate the risk associated with existing building stock and those under construction using present-day codes (with no earthquake provisions) as compared with the additional cost of incorporating minimum detailing requirements. For example, decisions need to be made on whether all new parapets be reinforced, and existing parapets be demolished or strengthened (Figure 8.1), whether structures designed with high eccentricity of strength or mass (such as The Junction Motel which was demolished soon after the earthquake as a result of column failure, see Plates 5.63, 5.64) or structures containing "soft" storeys should be designed with more stringent controls to ensure they have adequate capacity to withstand moderate earthquake loadings. From an engineering viewpoint, the answers to these issues are clear. The only remaining question is whether the cost to society can be afforded, given the level of risk of earthquake damage compared to other types of natural or man-made hazard.

## 9.0 CONCLUSIONS

The field investigation and detailed follow-up studies presented in this report have led to the following conclusions:-

1. The type of building stock most vulnerable to moderate or severe earthquake damage in areas of relatively low seismic risk (where building regulations do not make any specific allowance for lateral earthquake loading) is unreinforced brick masonry. This construction type, particularly in commercial usage, is about twice as vulnerable to damage compared with reinforced concrete frame buildings, and nearly three times as vulnerable as buildings of timber frame construction.

2. The detailed and general surveys of building damage in Newcastle carried out by EEFIT have given data which correlates very closely with that obtained from similar surveys following other earthquakes of similar magnitude in Europe and elsewhere. This consistency of damage data for various common forms of construction leads to the conclusion that accurate estimates of seismic vulnerability of existing building stock are possible in areas of low risk such as Australia or the UK.

3. A rigorous analytical approach to the study of site soil amplification effects due to this earthquake has indicated, contrary to previous reports, that such effects were evident primarily for the shallow, stiff soils near the border of the alluvial basin, and that there was a reasonable correlation between such areas and the locations of the most severe building damage. For a near-field earthquake of this type, deep soft soils tend to attenuate the bedrock ground motion, except in the long period range which is relevant only to buildings taller than those currently existing in the Newcastle area.

4. The comparison of damage distribution with the perimeters of coal mining activity showed a degree of correlation which could be recognised as a significant feature of the earthquake damage pattern. This may be connected with historical and sociological urban development, since the mine perimeters tend to be aligned along major roads, which is also where the most vulnerable buildings are situated, such as older, commercial brick masonry building stock.

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**FIGURES and PLATES**

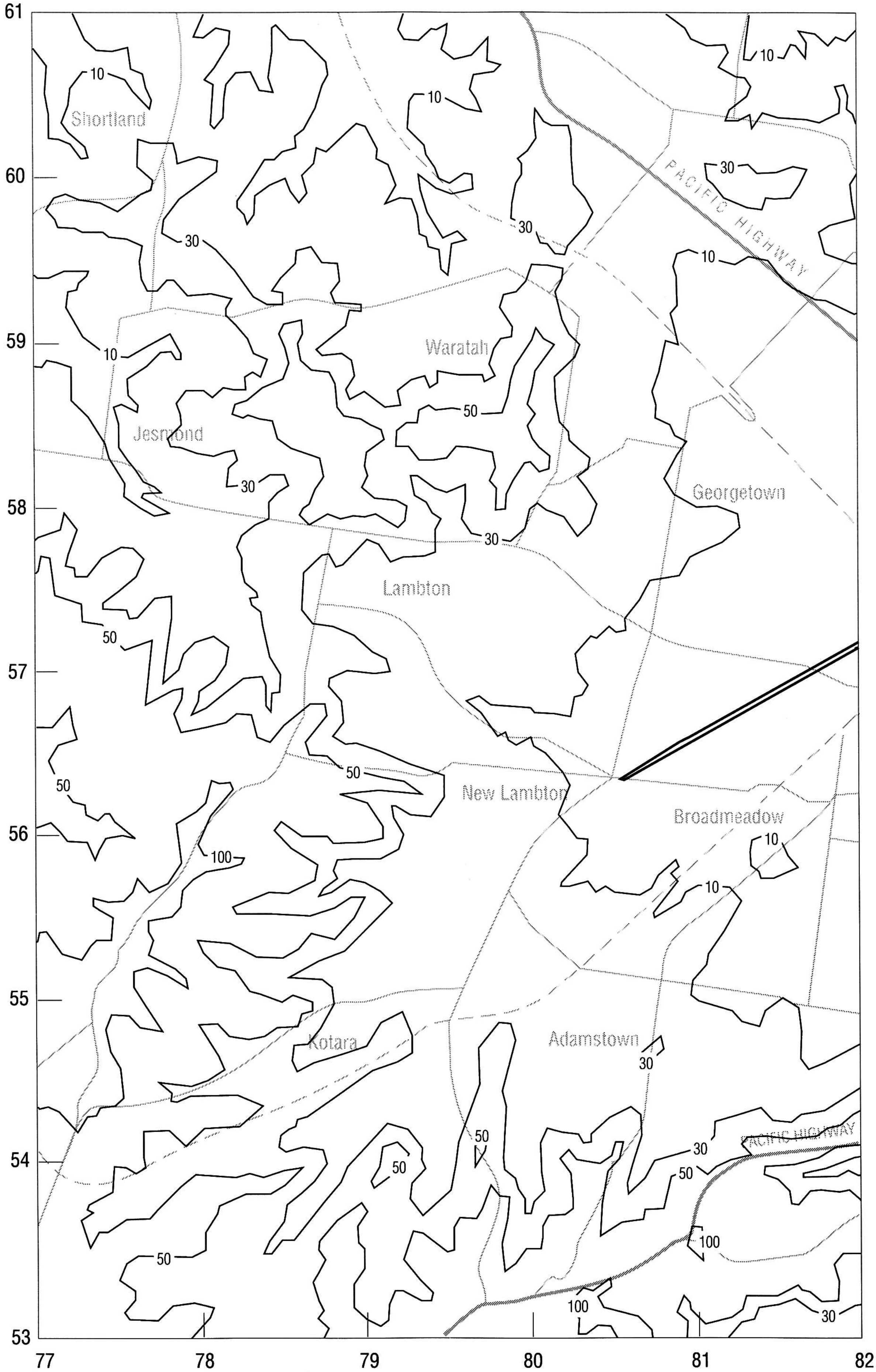
The authors wish to acknowledge the following sources for photographs reproduced in this report

**Source**

| | |
|---|---|
| Central Mapping Authority of N.S.W. "Orthophotomap" Sheet "Adamstown U6350-3" 1:4000 Series 1988 | Figure 2.1 |
| The Newcastle Herald "Quake Special" Friday December 29 1989 edition | Plates 5.19, 5.20, 5.61, 5.62 and 7.2 |
| Institution of Engineers Australia, 1990 "Newcastle Earthquake Study" | Plates 5.63 and 5.76 |
| Working Magazine 1990 "Earthquake Report - Massive Task for PWD Staff" Vol. 4 | Plate 5.64 |


Part of aerial photographic montage of Newcastle, NSW (scale 1:4 000)

Figure 2.1

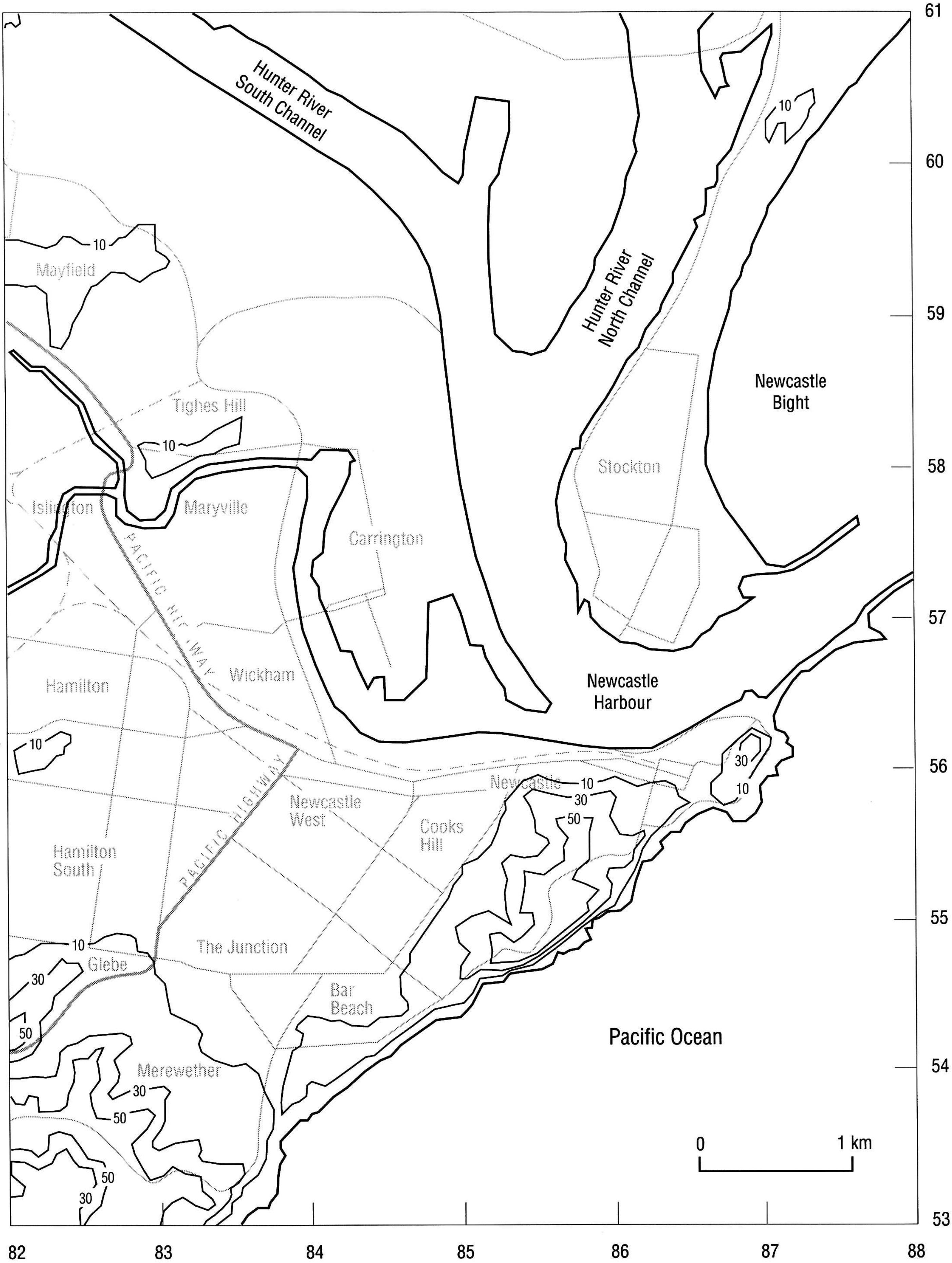



30

Ground level contours
(m above mean sea level)

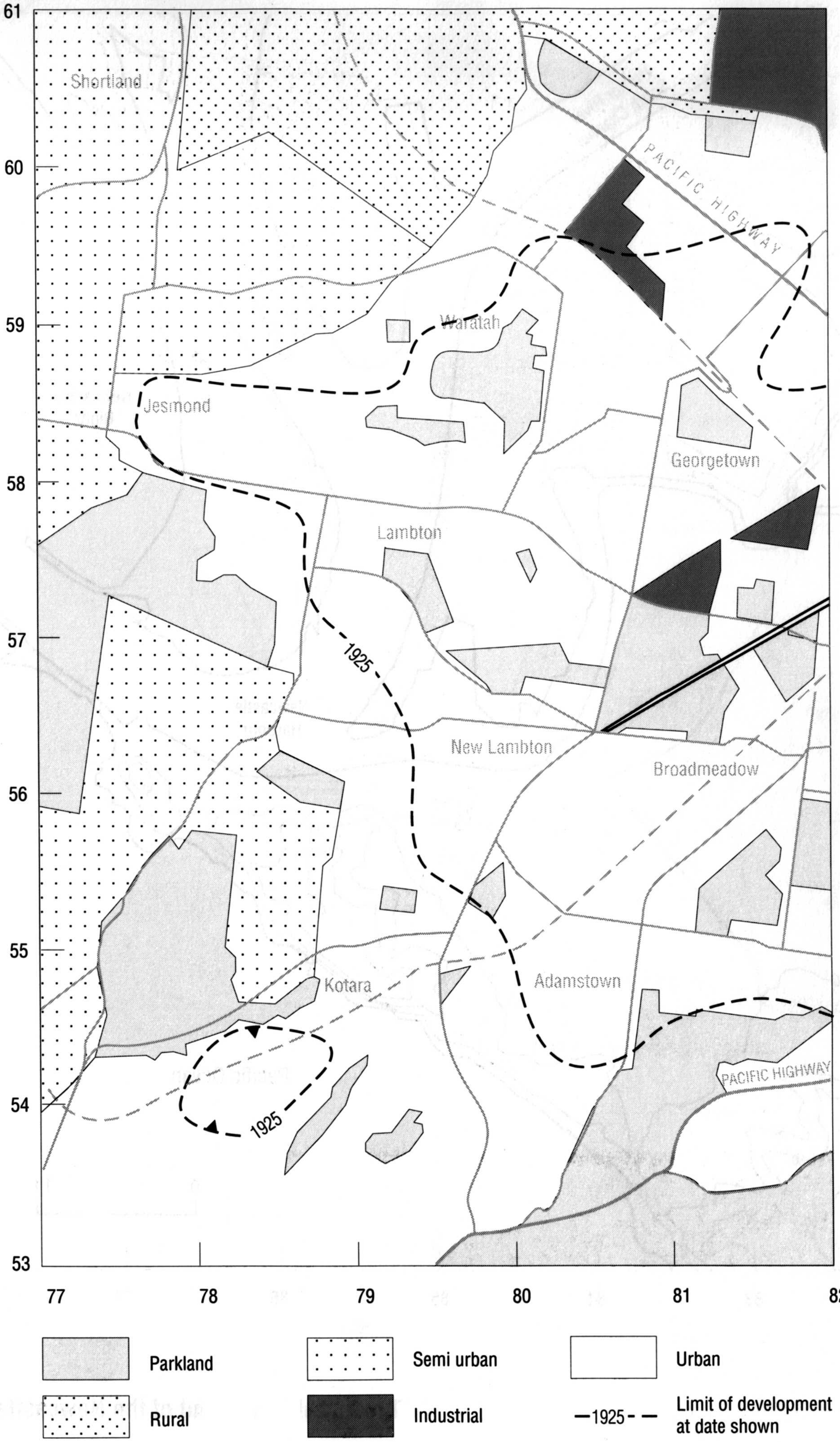
Roads

Railways




Topographical layout of the Newcastle area

Figure 3.1



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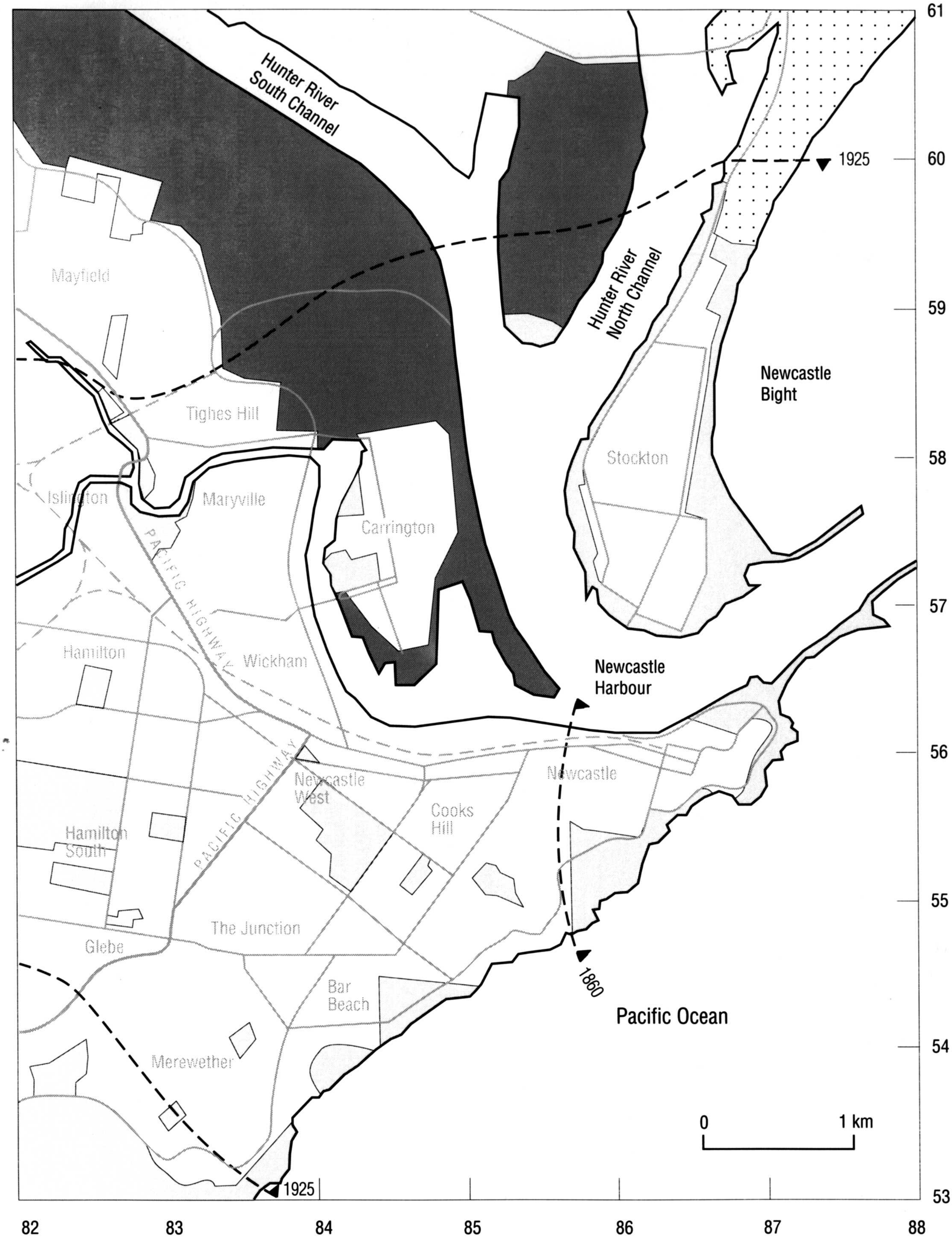
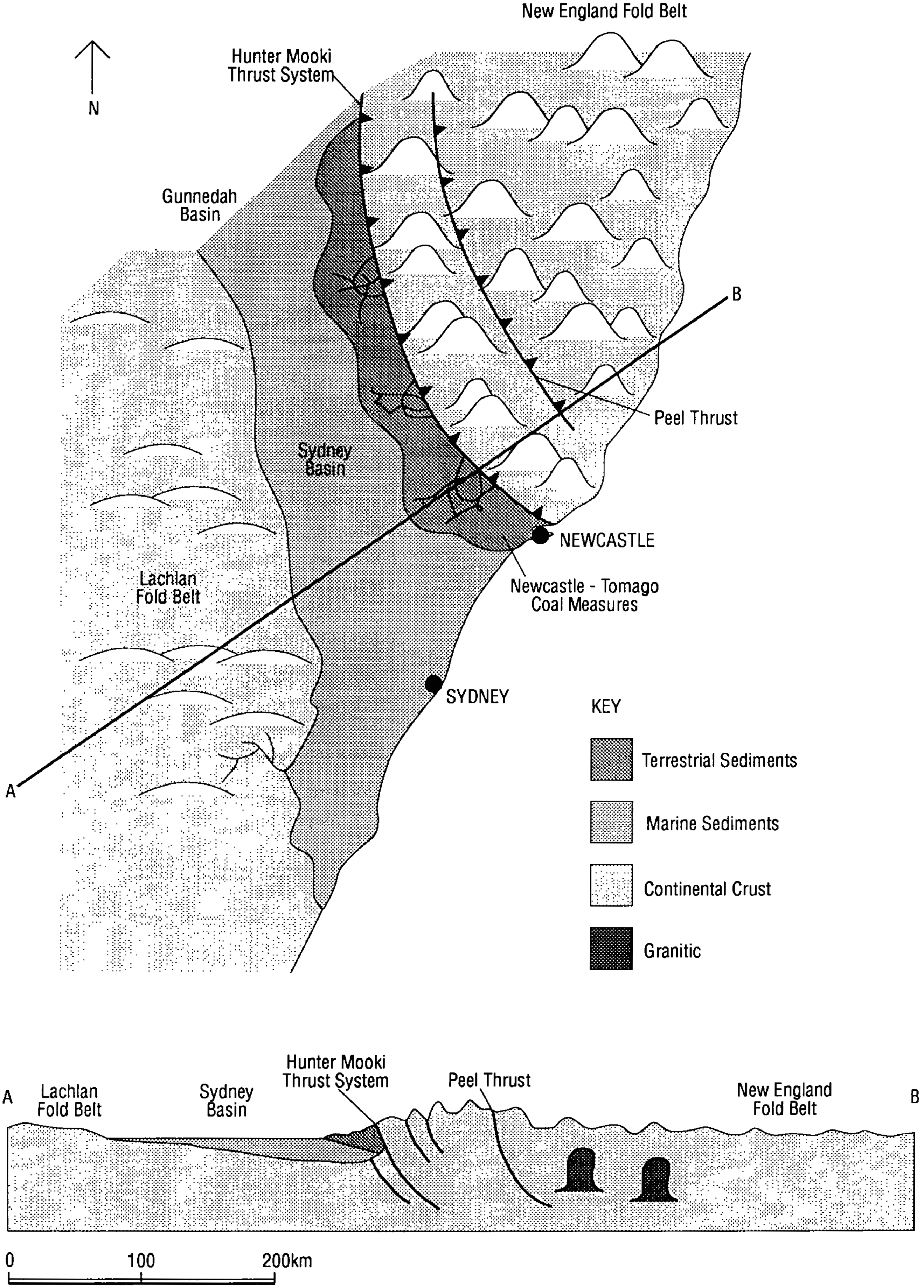
Shortland
PACIFIC HIGHWAY
Waratah
Jesmond
Georgetown
Lambton
1925
New Lambton
Broadmeadow
Adamstown
Kotara
PACIFIC HIGHWAY
1925
Parkland
Semi urban
Urban
Rural
Industrial
1925
Limit of development at date shown




**Land use in Newcastle and surrounding areas**

**Figure 3.2**

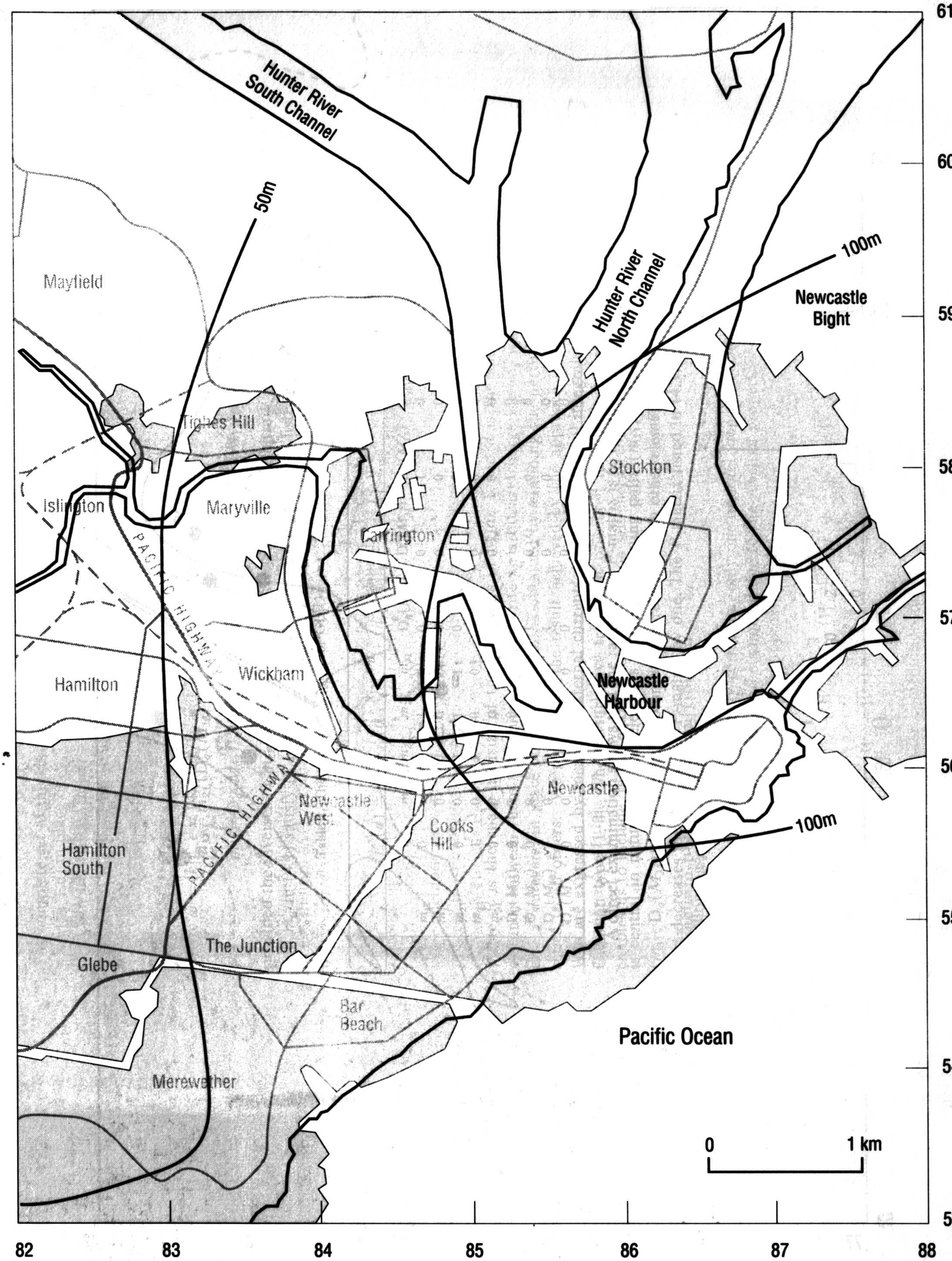



**Geological plan and section of the Sydney Basin and adjacent regions** (after Herbert, C. 1980)

Figure **3.3**

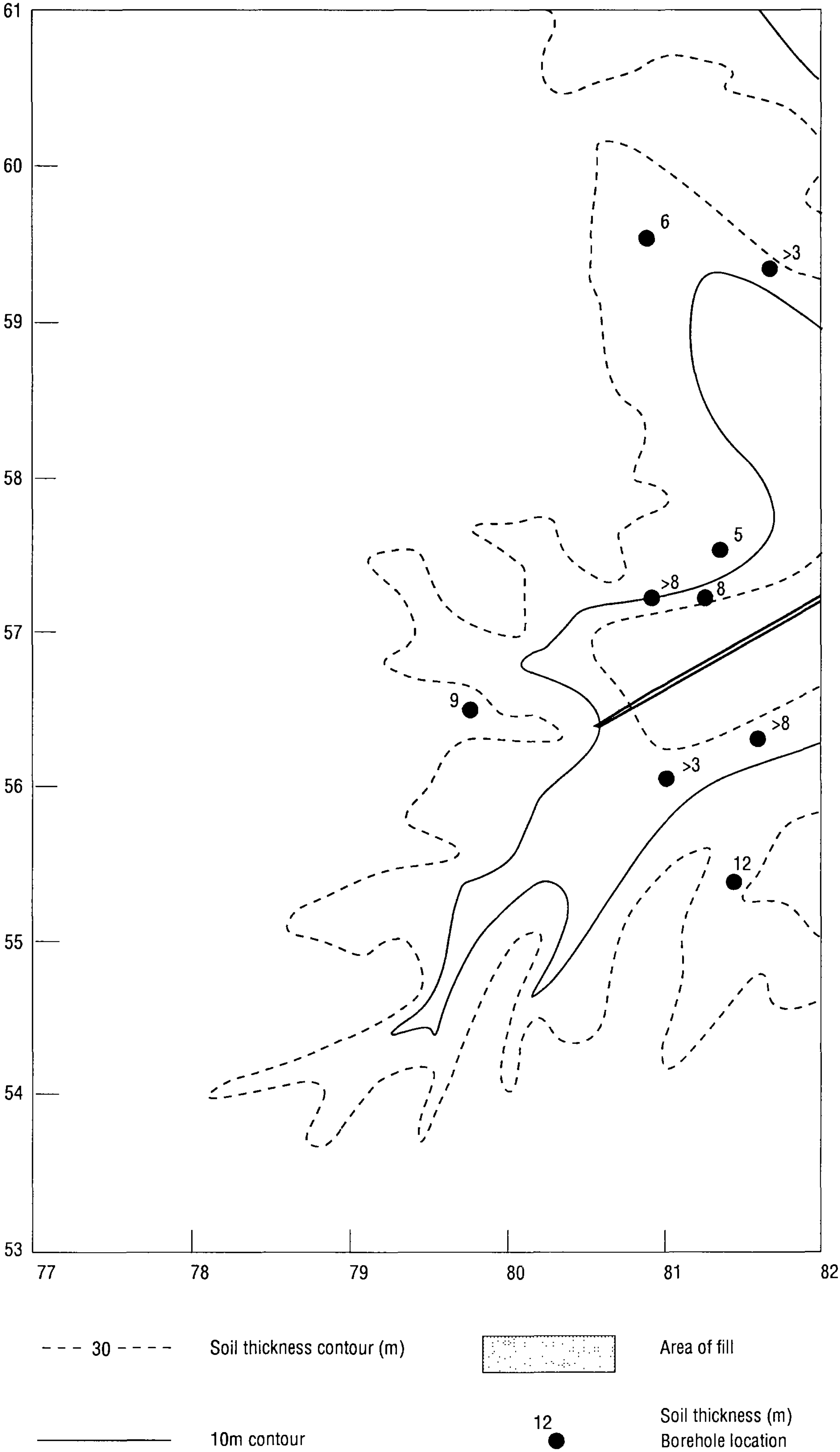

Area of coal extraction from the borehole seam - Moelle 1990

50m Contour of base of seam (metres below mean sea level) - Diessel 1980




**Boundaries of coal extraction and contours to base of borehole seam**

**Figure 3.4**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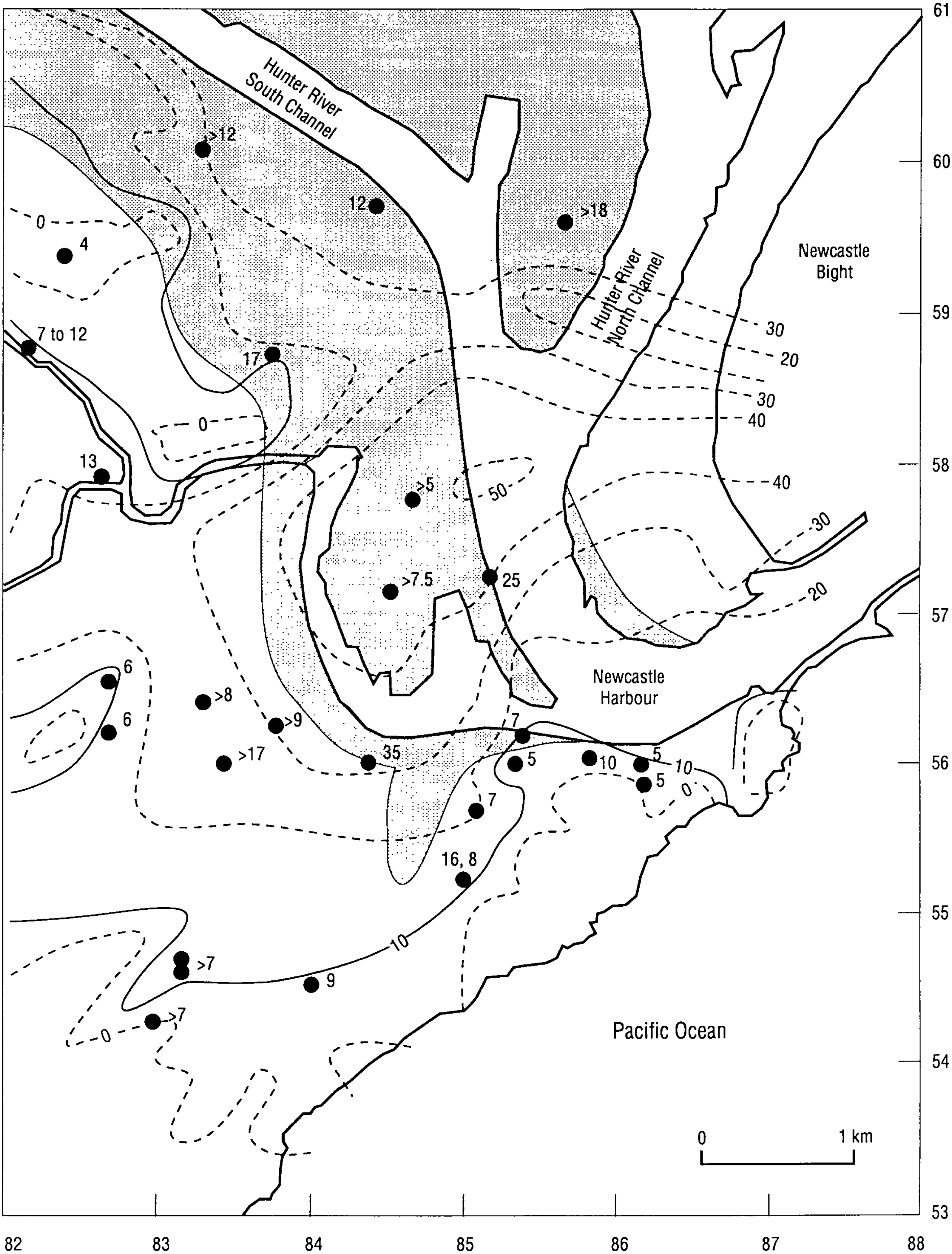
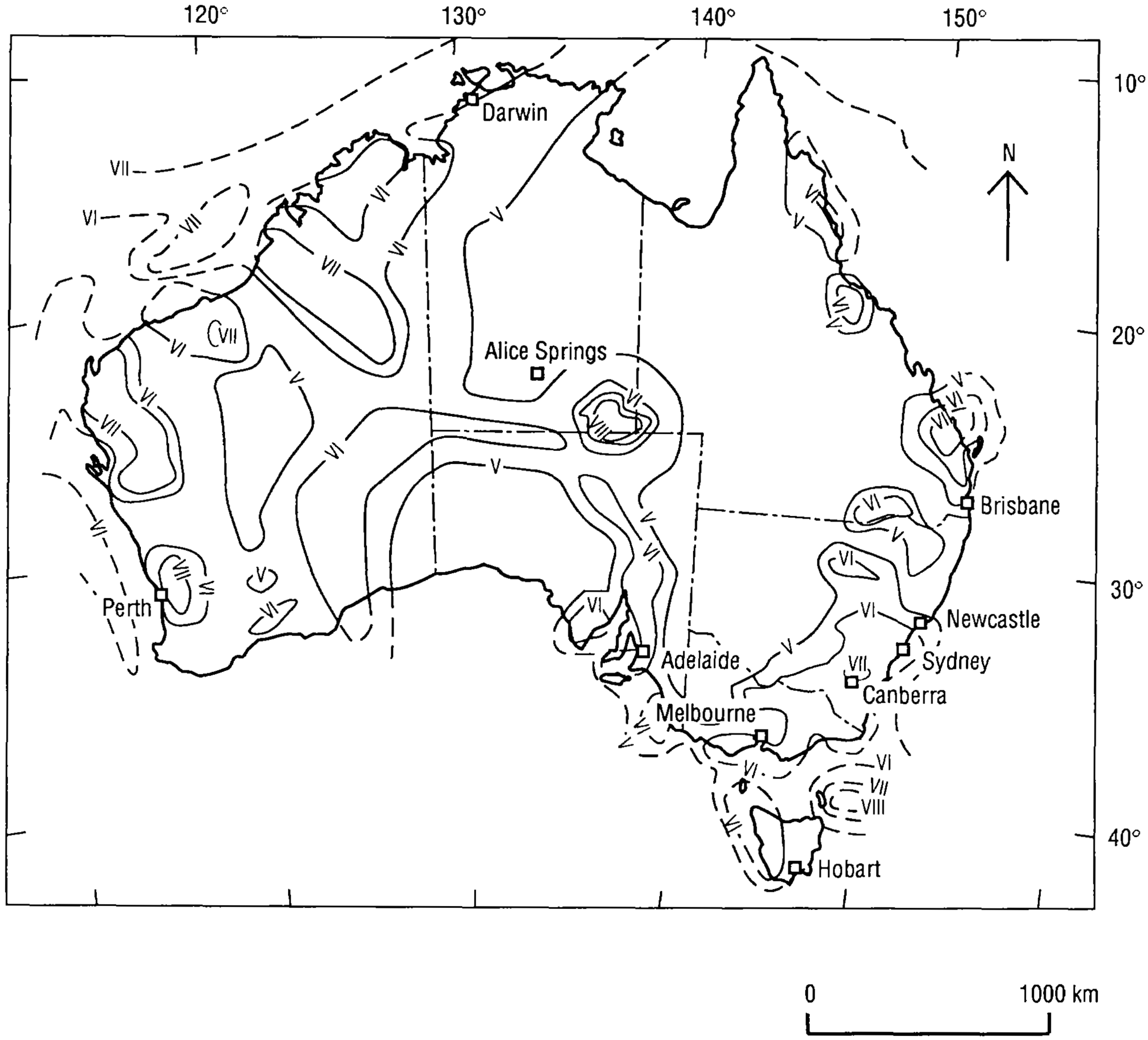
30
Soil thickness contour (m)
Area of fill
10m contour
12
Soil thickness (m)
Borehole location



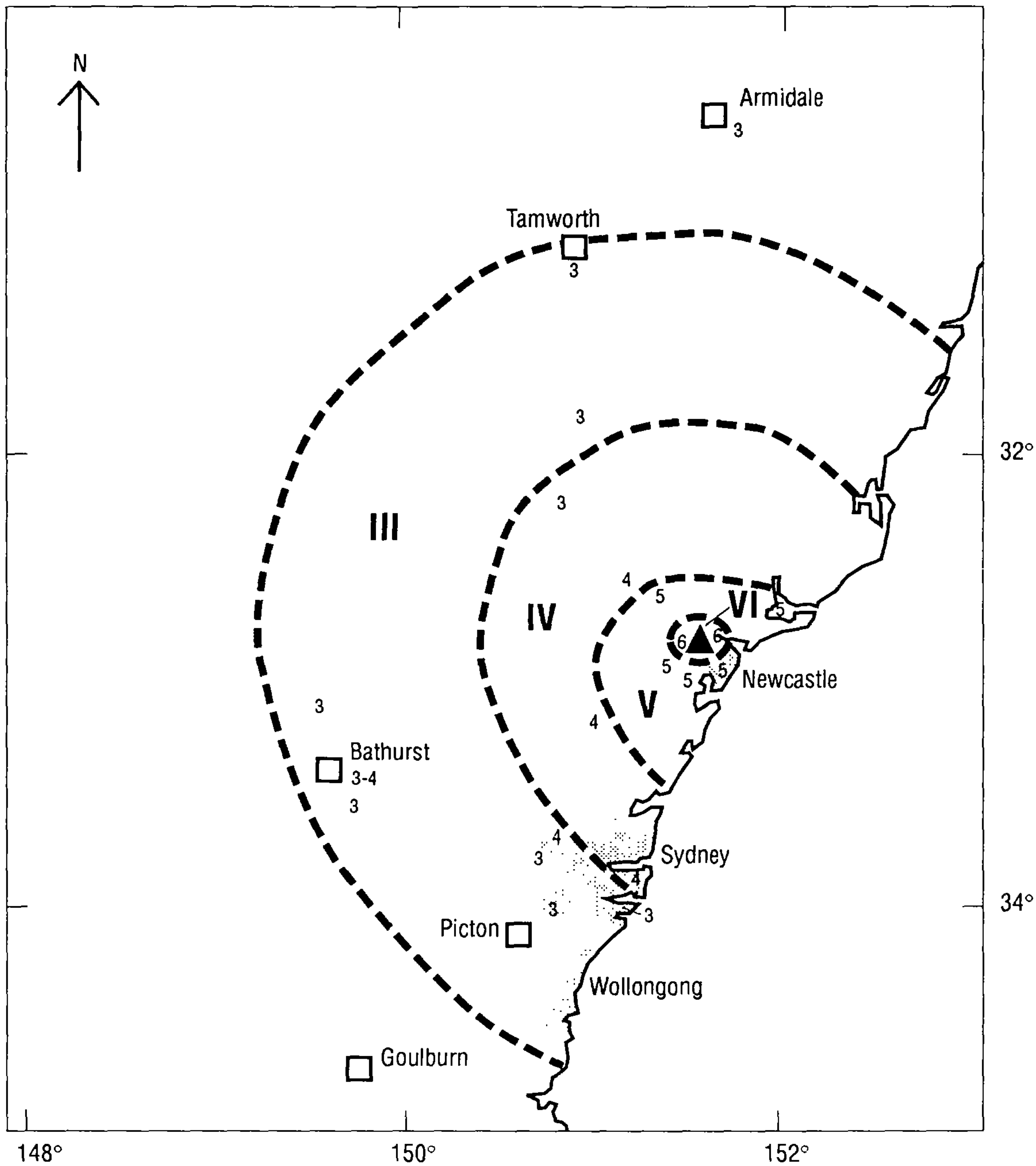

**Borehole locations and depth of soil deposits**

**Figure 3.5**




**Intensity (modified Mercalli scale) with a 10% chance of being exceeded in a 50 year period** (after Gaull et al 1990)

**Figure 4.1**




0 100 km

DATE: 18th June 1868
TIME: 14:00 UT
MAGNITUDE: 5.3 ML
EPICENTRE: 32.8°S 151.6°E

▲ Epicentre

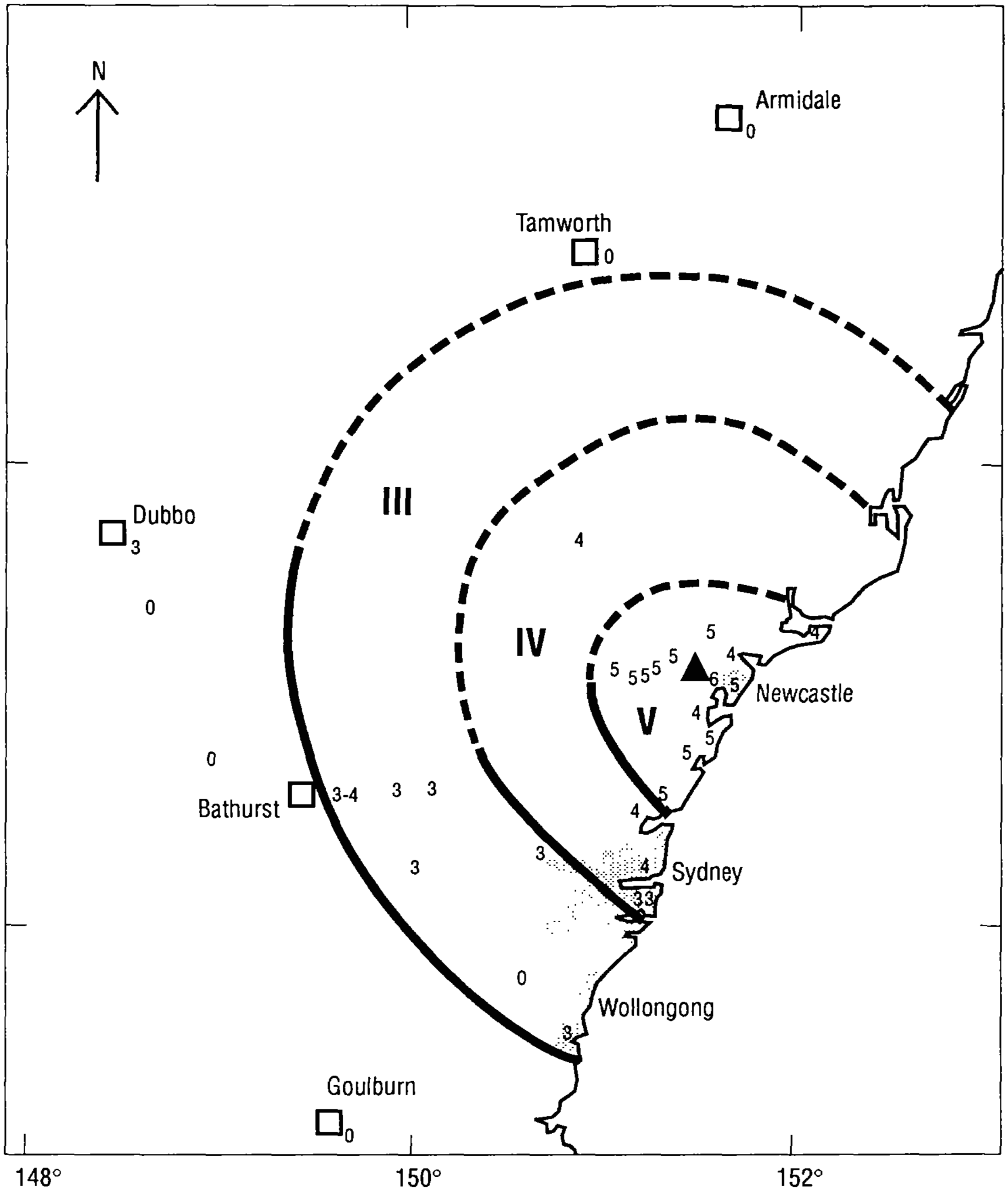
**IV** Zone intensity designation

4 Earthquake felt (MM)

0 Earthquake not felt

**Isoseismal map for 1868 Newcastle earthquake (after McCue 1990)**

**Figure 4.2**




DATE: 18th December 1925
TIME: 10:47:10 UT
MAGNITUDE: 5.0 ML (BMR), 5.3 ML (I)
EPICENTRE: 33.0°S 151.6°E
DEPTH: Crustal

▲ Epicentre

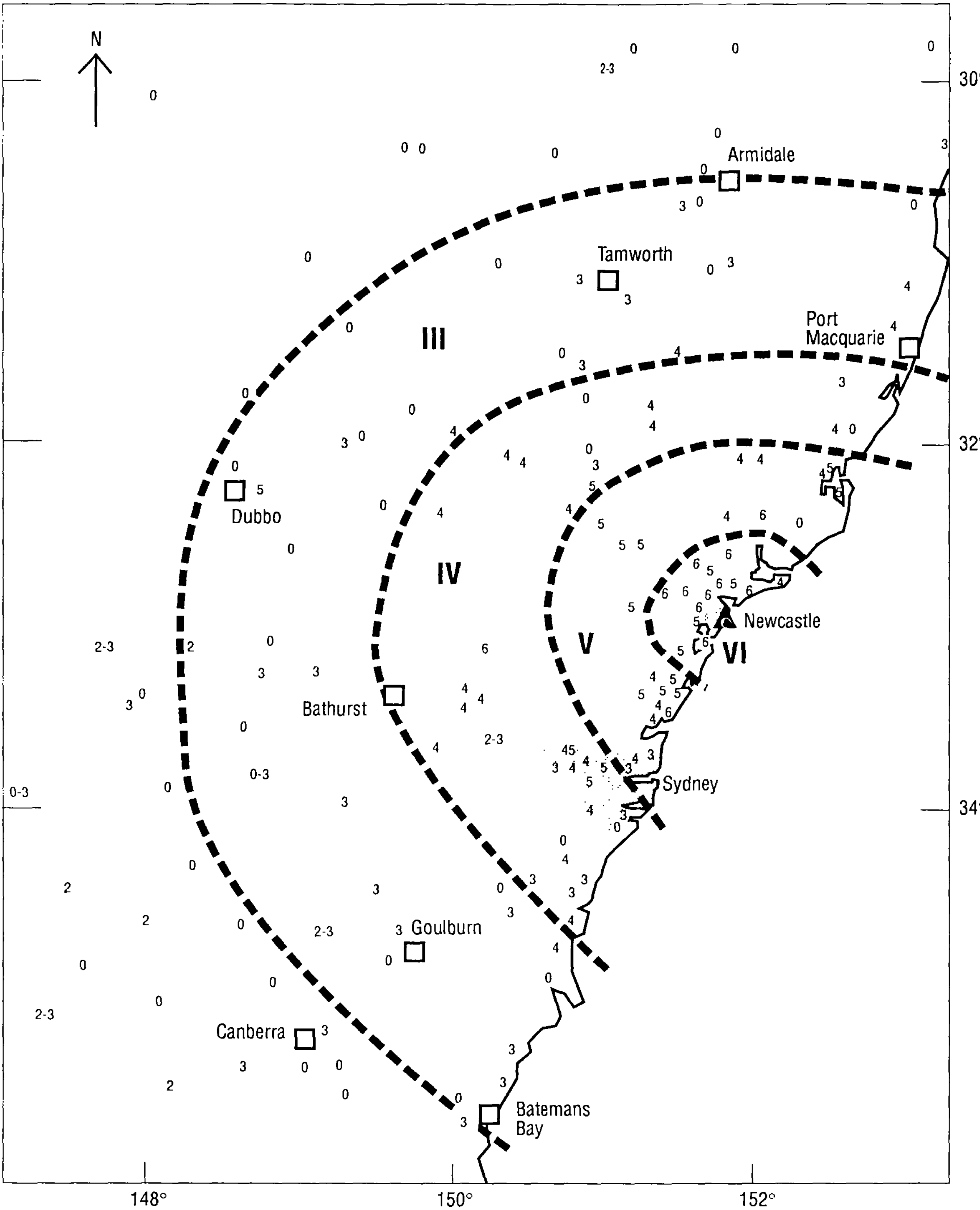
**IV** Zone intensity designation

4 Earthquake felt (MM)

0 Earthquake not felt

**Isoseismal map for 1925 Newcastle earthquake** **(after Rynn et al 1989)**

**Figure 4.3**

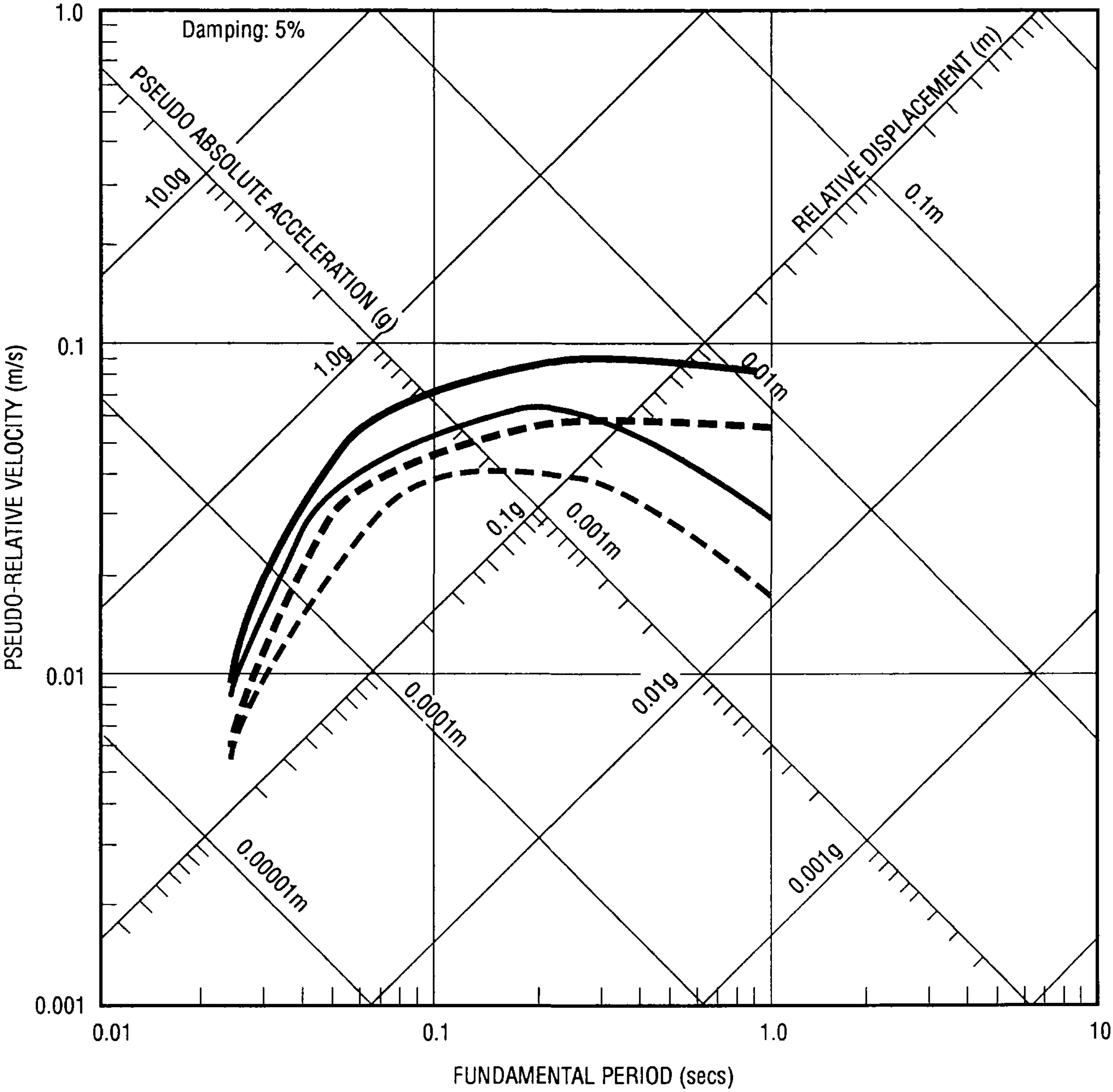



| | |
|---|---|
| DATE: | 28th December 1989 |
| TIME: | 23:26:58 UT |
| MAGNITUDE: | 5.6 ML |
| EPICENTRE: | 32.9°S 151.6°E |
| DEPTH: | 11 (±1) km |
| ▲ | Epicentre |
| **IV** | Zone intensity designation |
| 4 | Earthquake felt (MM) |
| 0 | Earthquake not felt |

0 100 km

**Isoseismal map for 1989 Newcastle earthquake (after McCue et al 1990)**

**Figure 4.4**



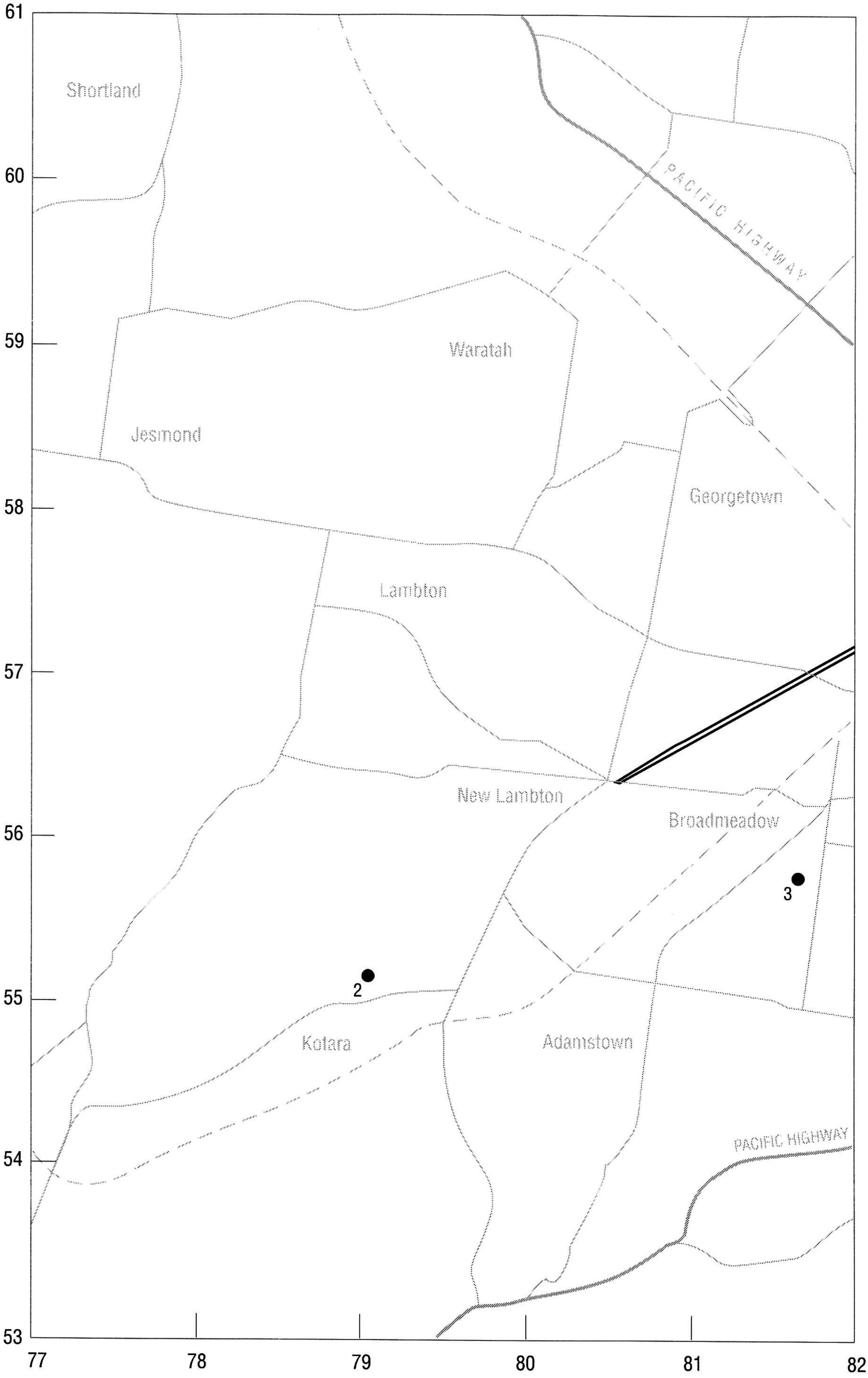

R = 12 km, R = 18 km — Atkinson & Boore (1990)

R = 12 km, R = 18 km — Toro & McGuire (1987)

(R = hypocentral distance)

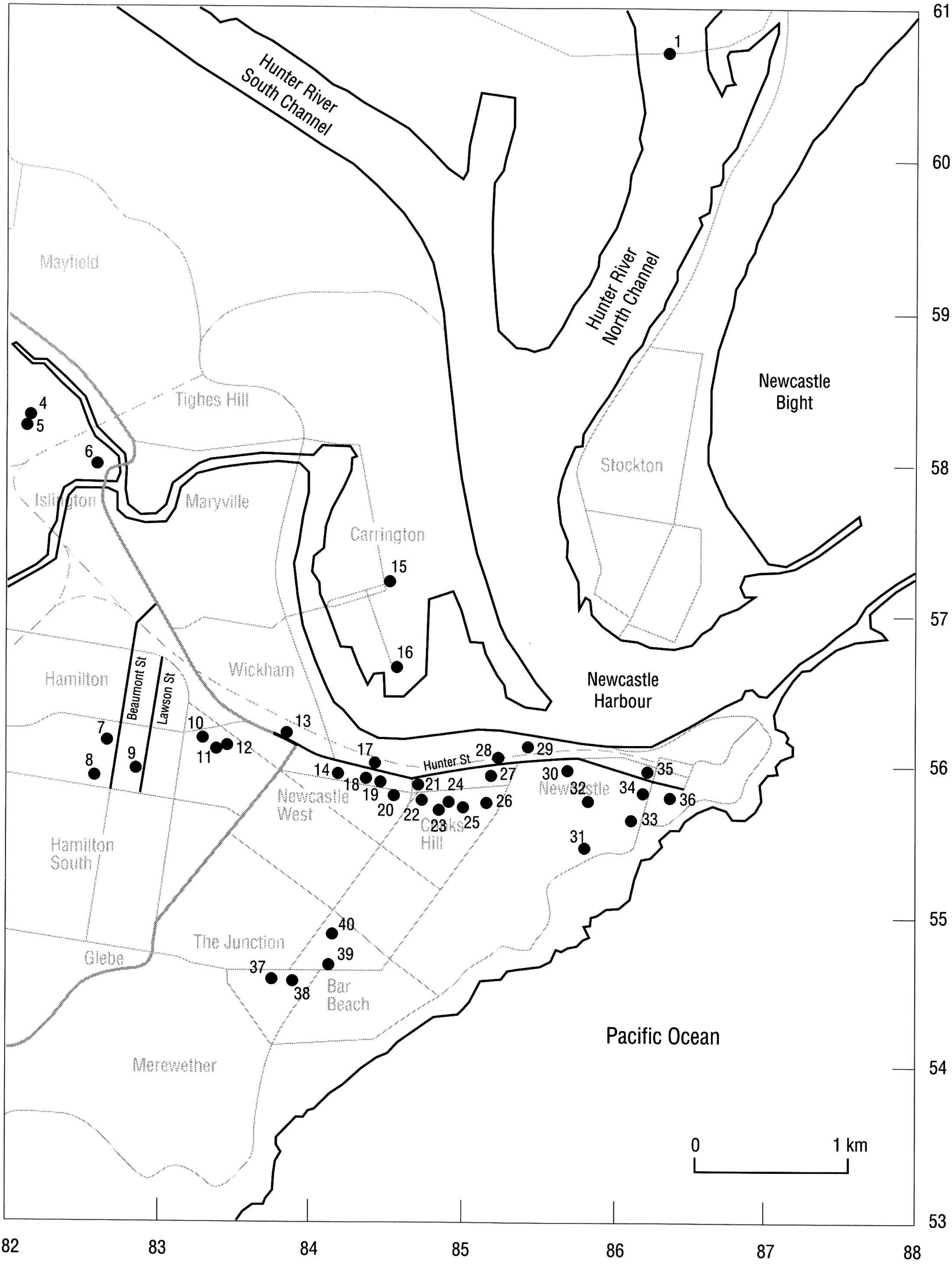
**Response spectra for rock surface motion**

**Figure 4.5**



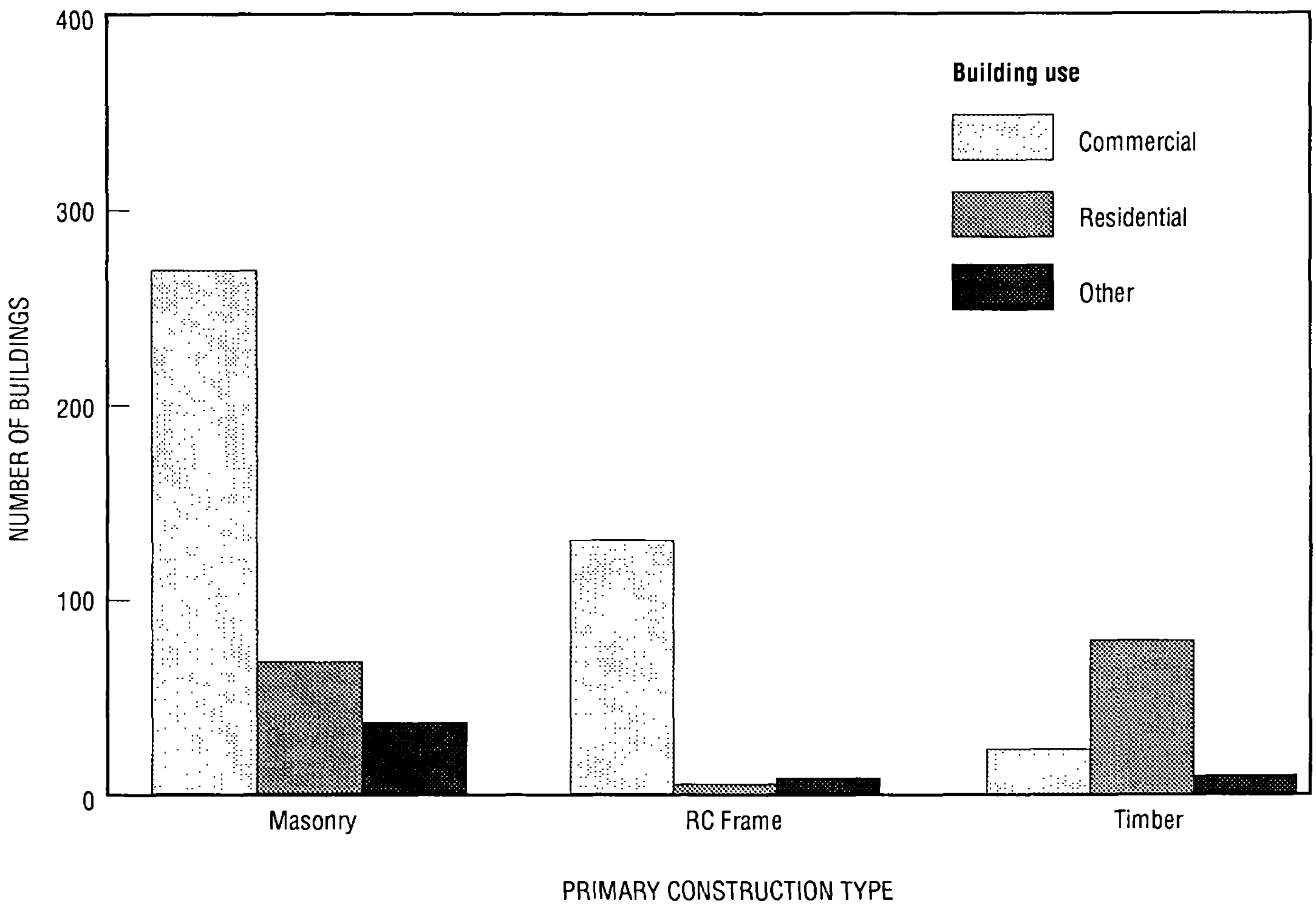

Streets covered in detailed photographic survey

Location numbers for plates



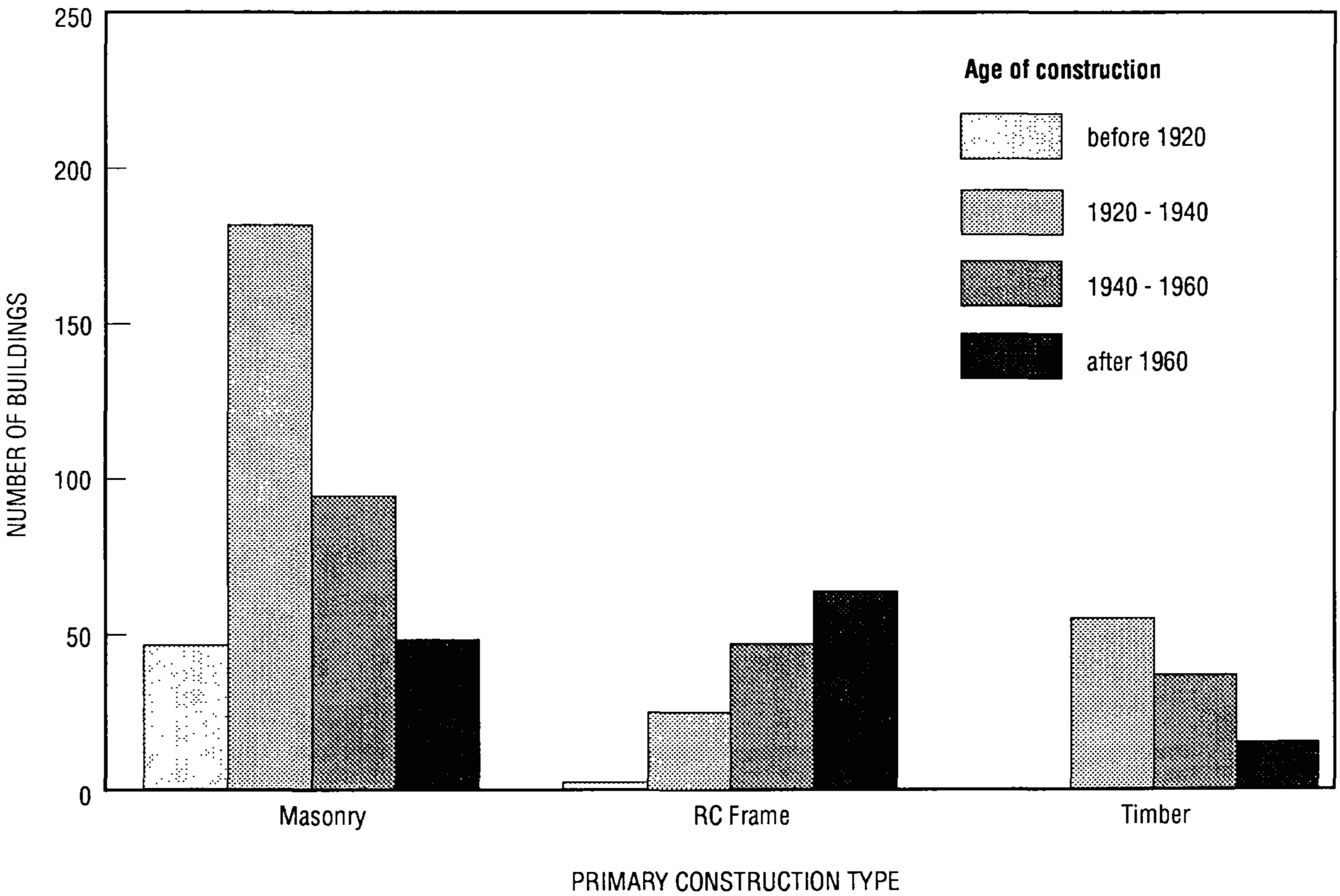

**Locations of photographs in plates and the detailed photographic surveys in Newcastle and Hamilton**

**Figure 5.1**



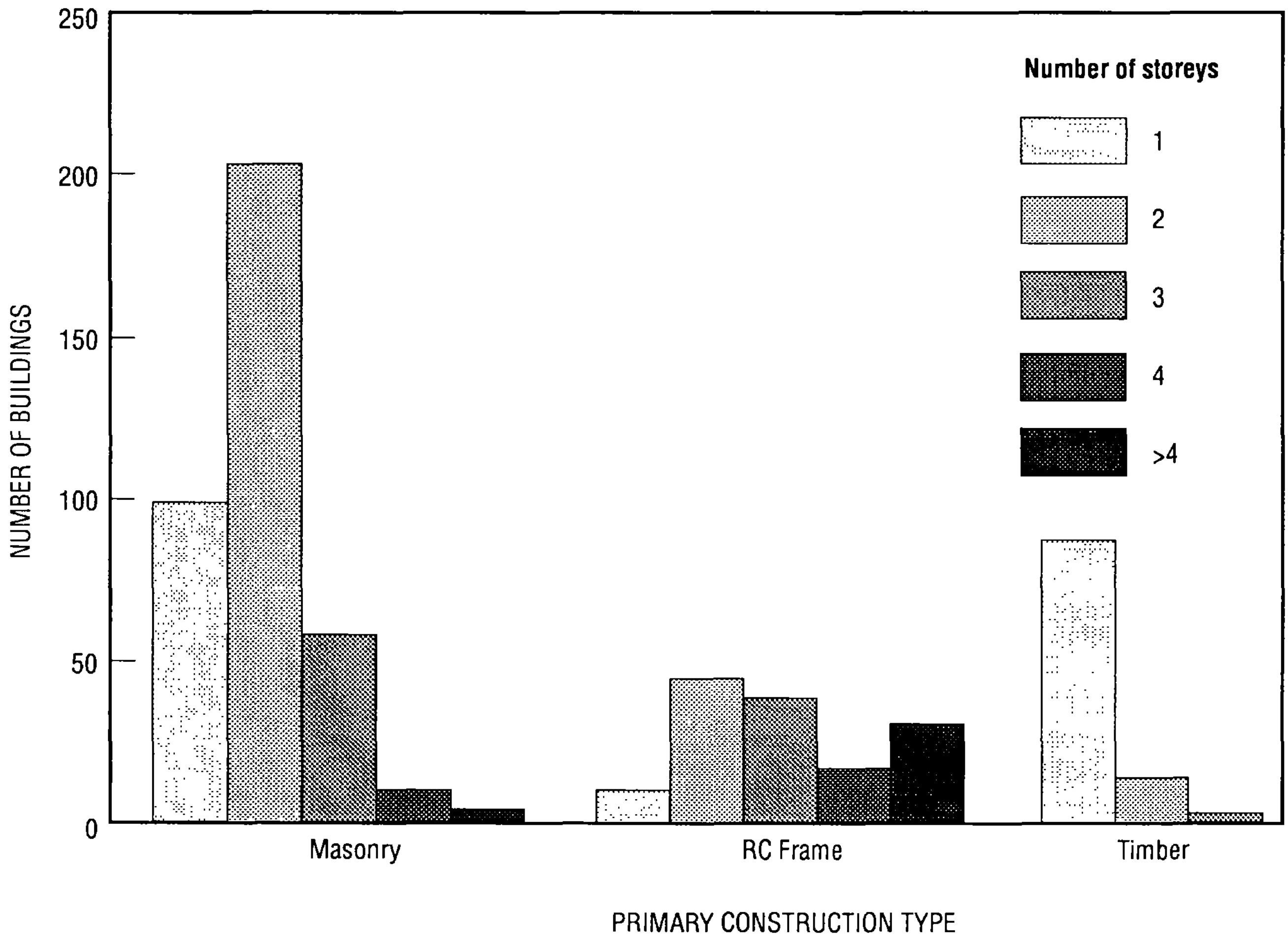

**Number of buildings by primary construction type and use**

**Figure 6.1**



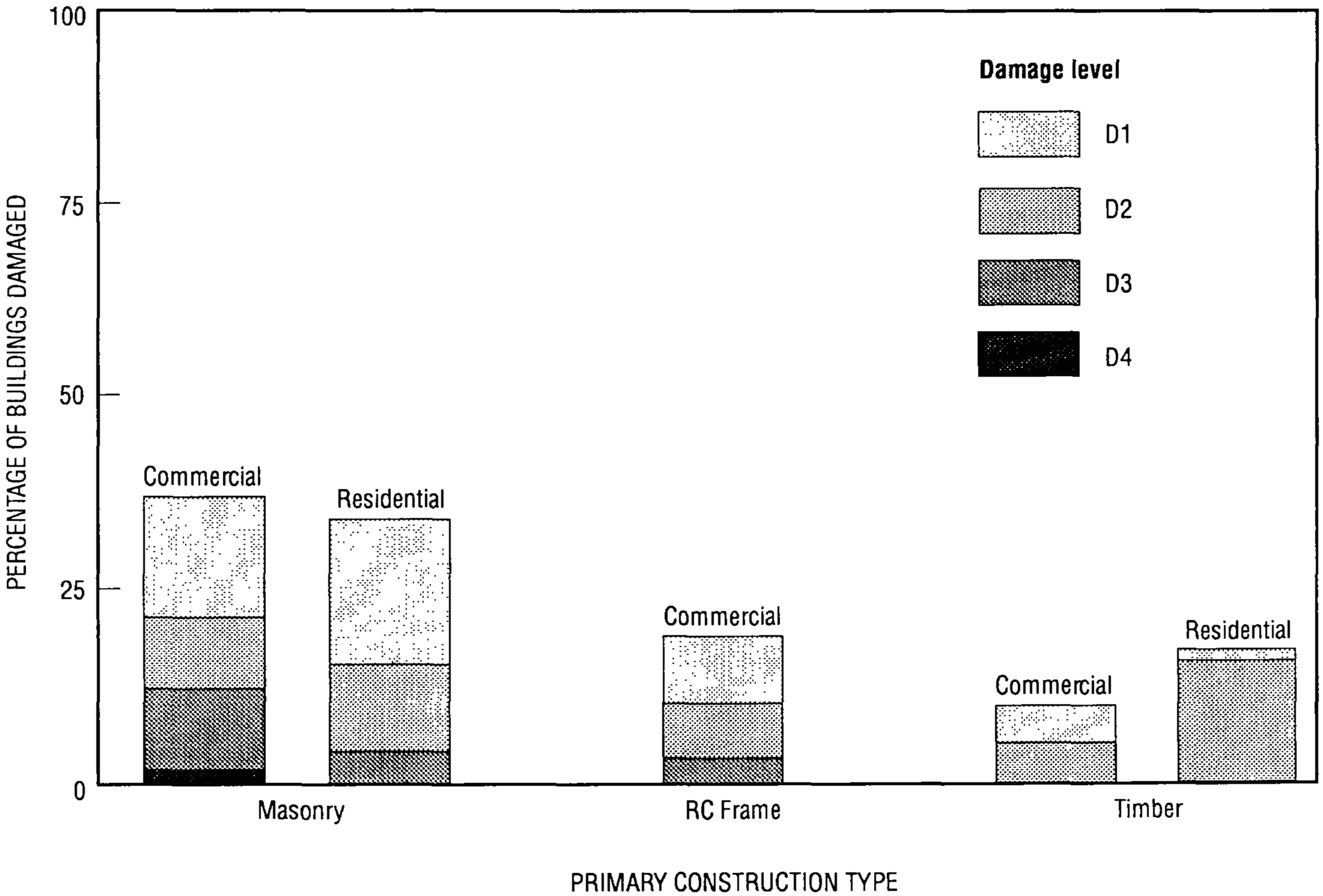

**Number of buildings by age of construction**

**Figure 6.2**



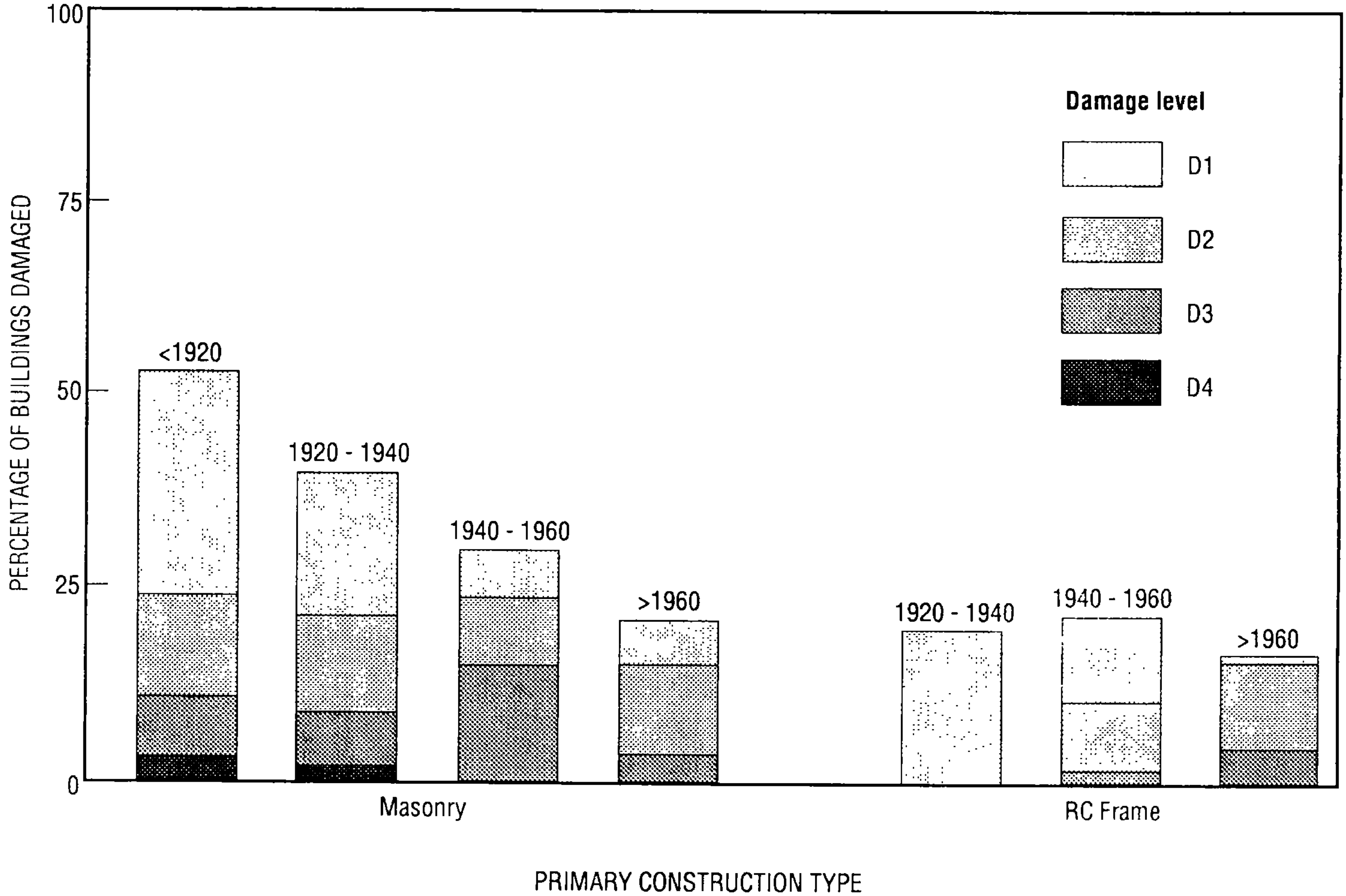

**Number of buildings by number of storeys**

**Figure 6.3**



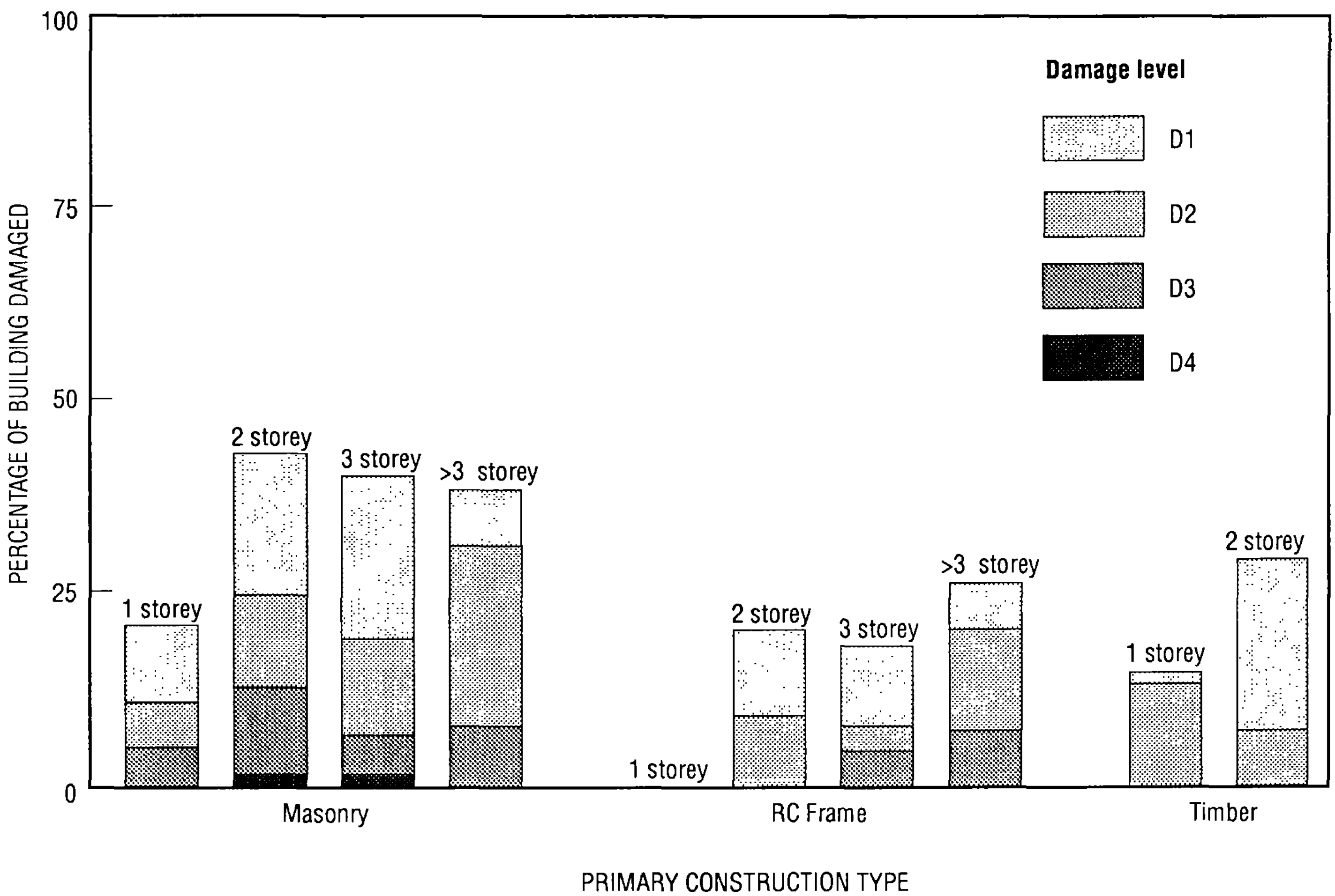

**Variation of damage levels with primary construction type and use**

**Figure 6.4**



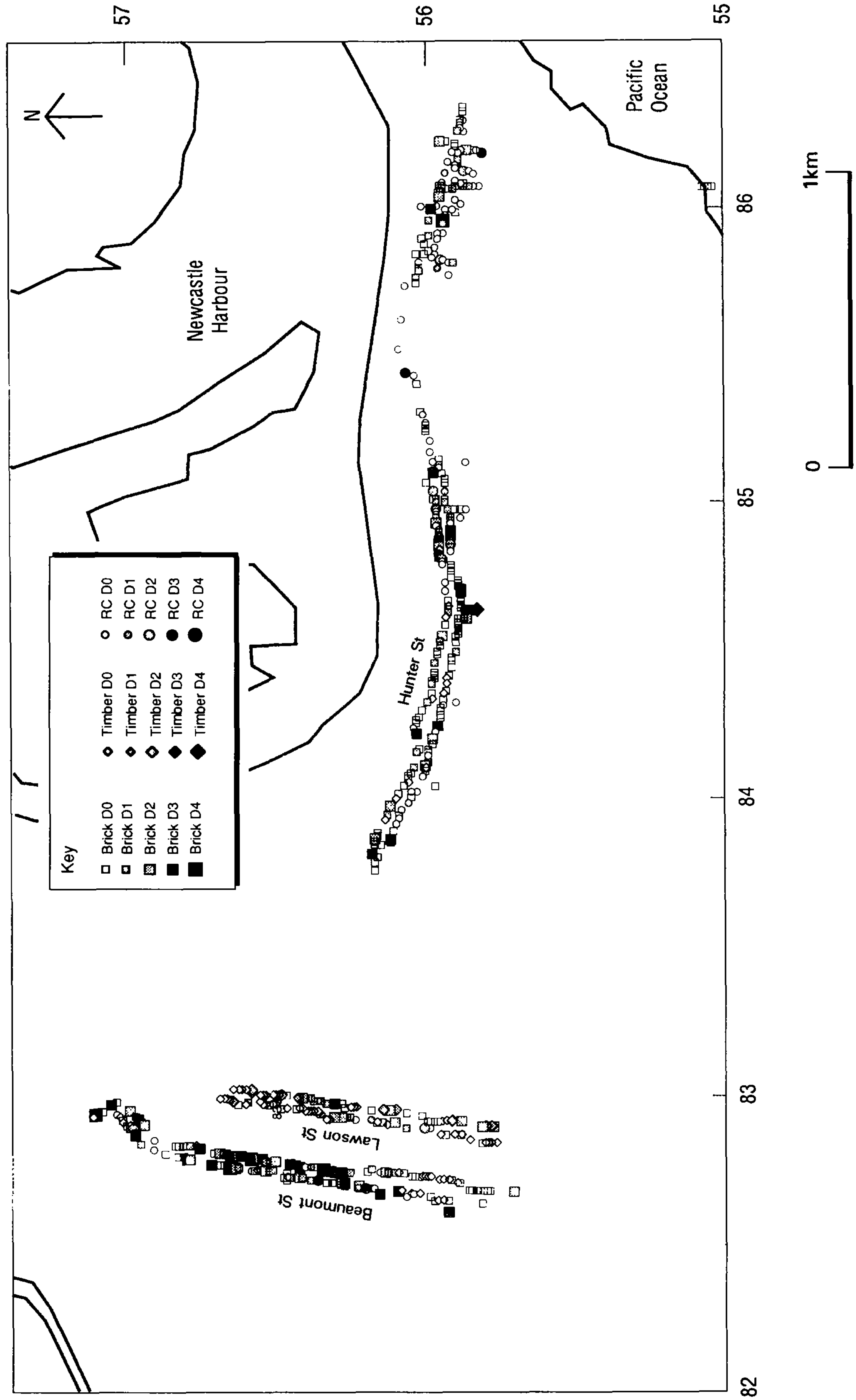

**Variation of damage levels with age of construction**

**Figure 6.5**



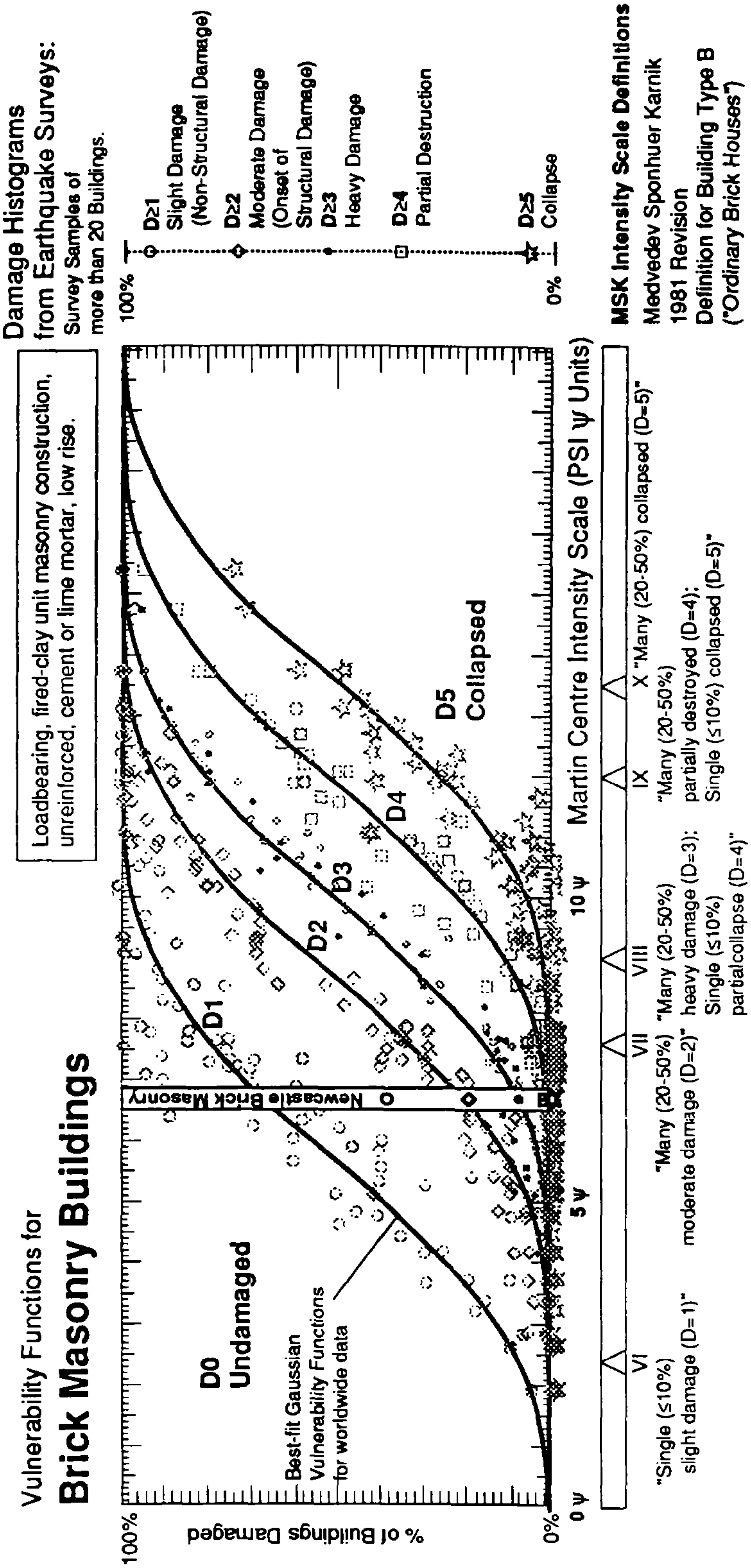

**Variation of damage levels with number of storeys**

**Figure 6.6**




**Summary of results from detailed damage surveys in Newcastle and Hamilton**

**Figure 6.7**

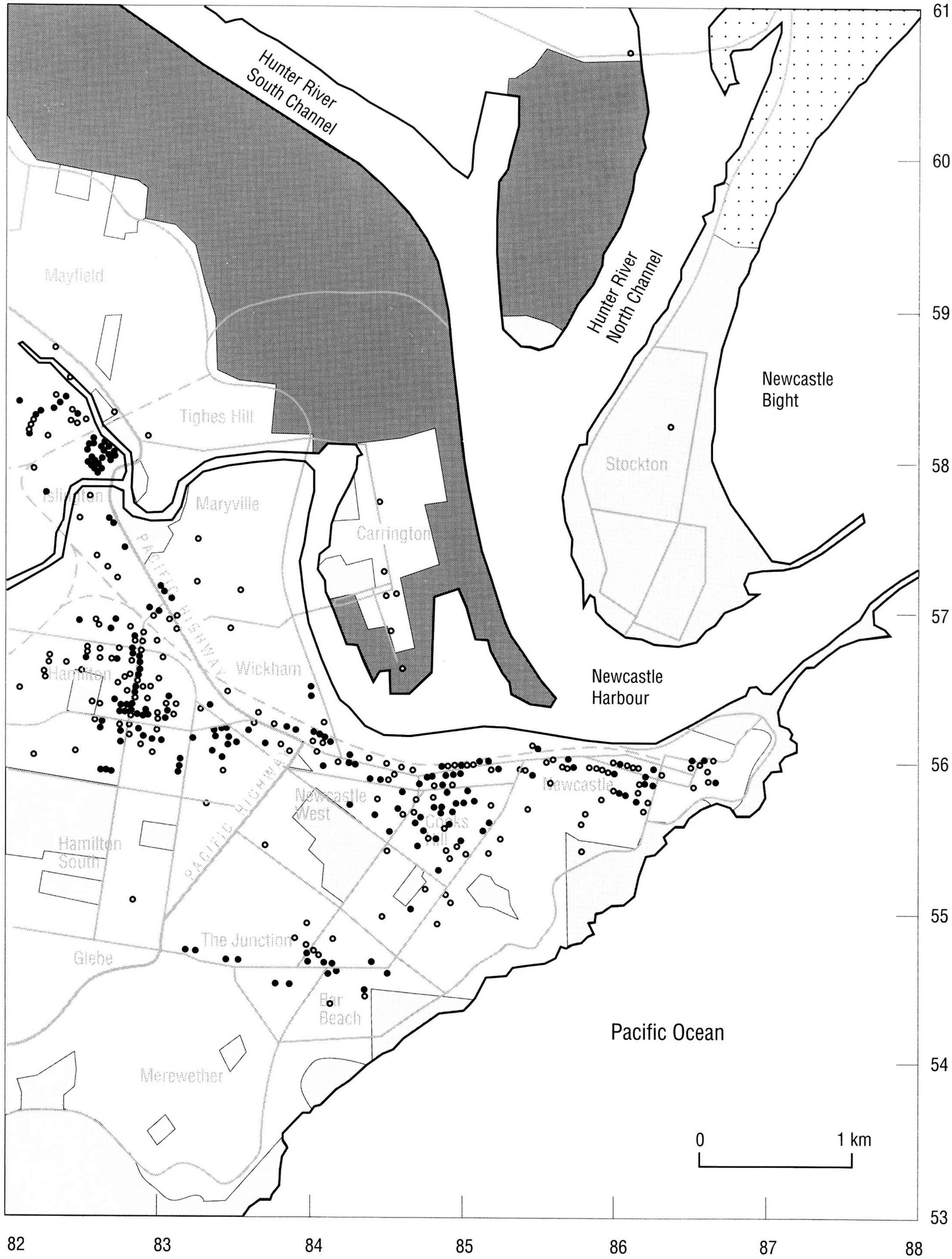



**Distribution of damage to Newcastle brick masonary buildings compared to similar building stock in other earthquakes worldwide** (from Coburn et al. 1990)

Figure **6.8**

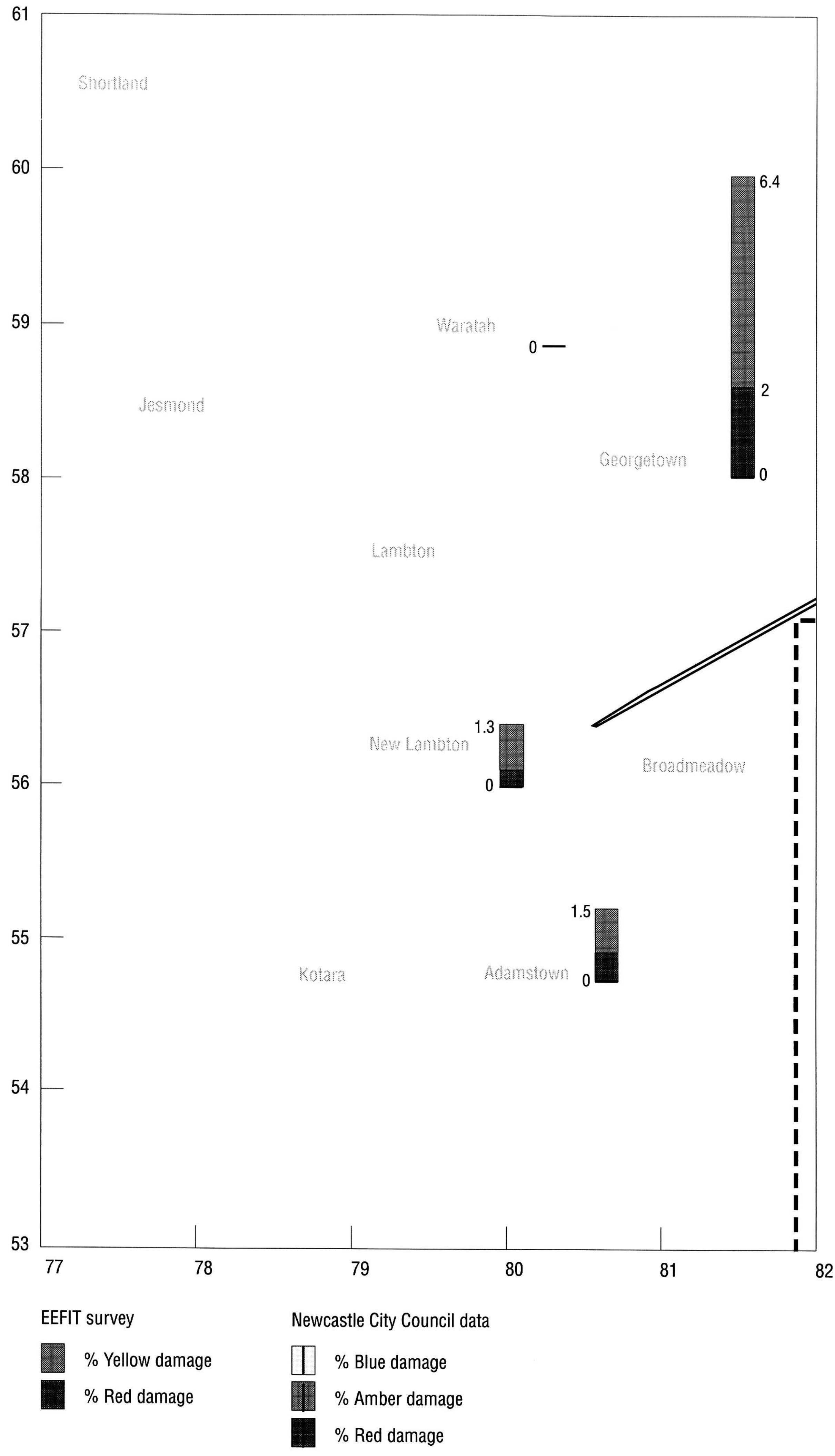

• Severe (red) damage level

○ Moderate (yellow) damage level




**Locations of structures with moderate or heavy damage from EEFIT general survey**

**Figure 6.9**



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Lambton
New Lambton
Broadmeadow
Kotara
Adamstown
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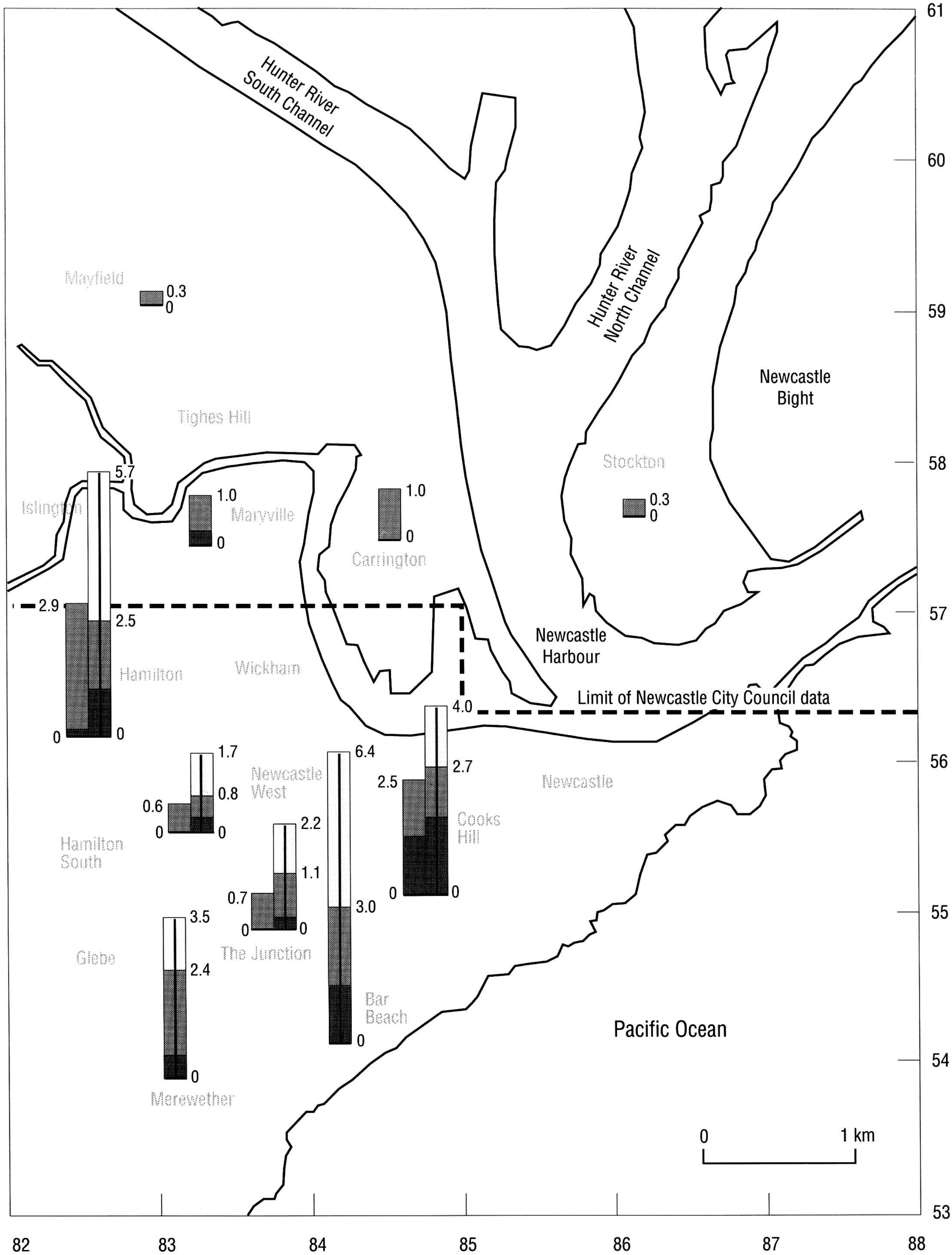
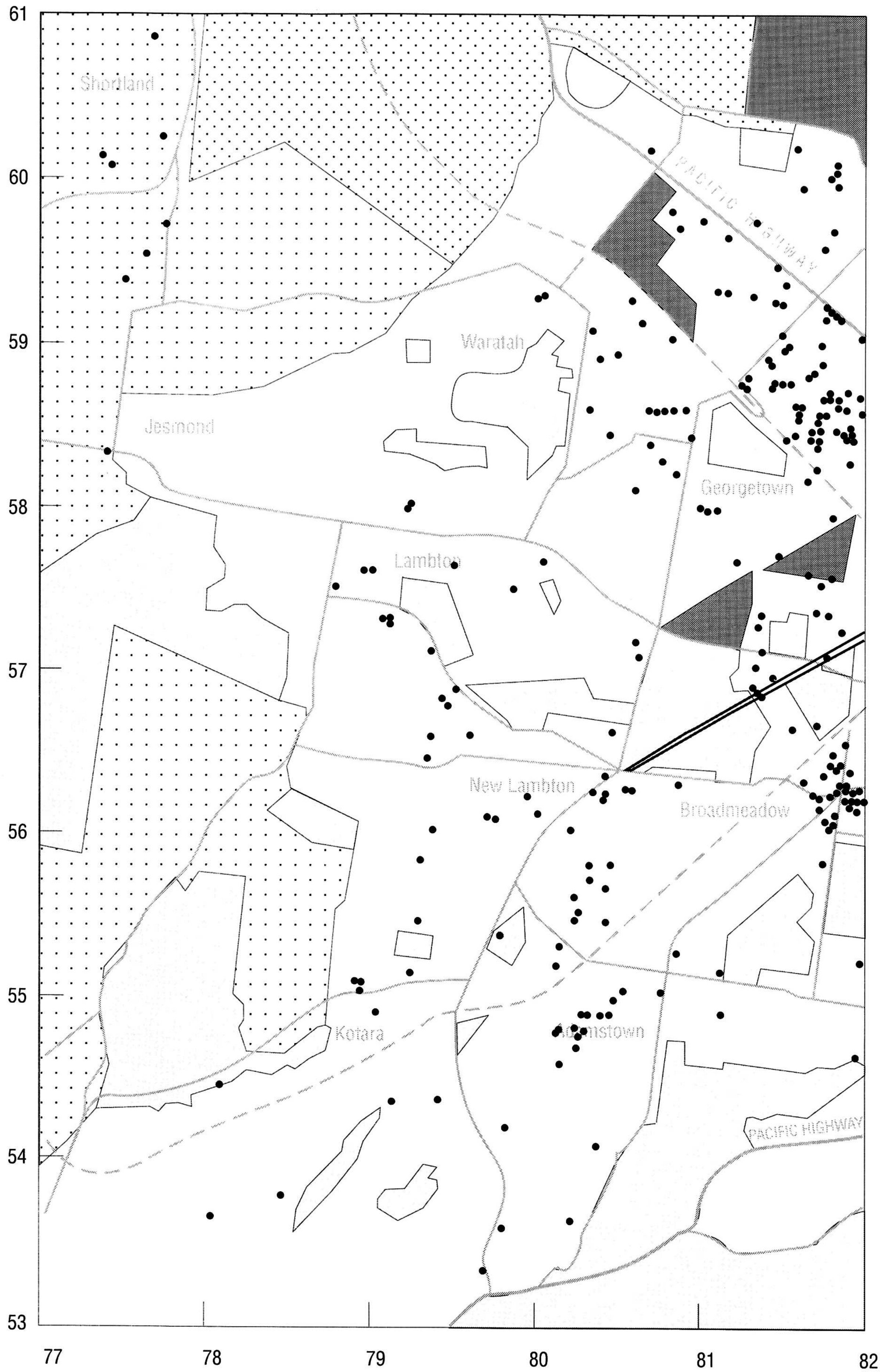
1.5
EEFIT survey
% Yellow damage
% Red damage
Newcastle City Council data
% Blue damage
% Amber damage
% Red damage

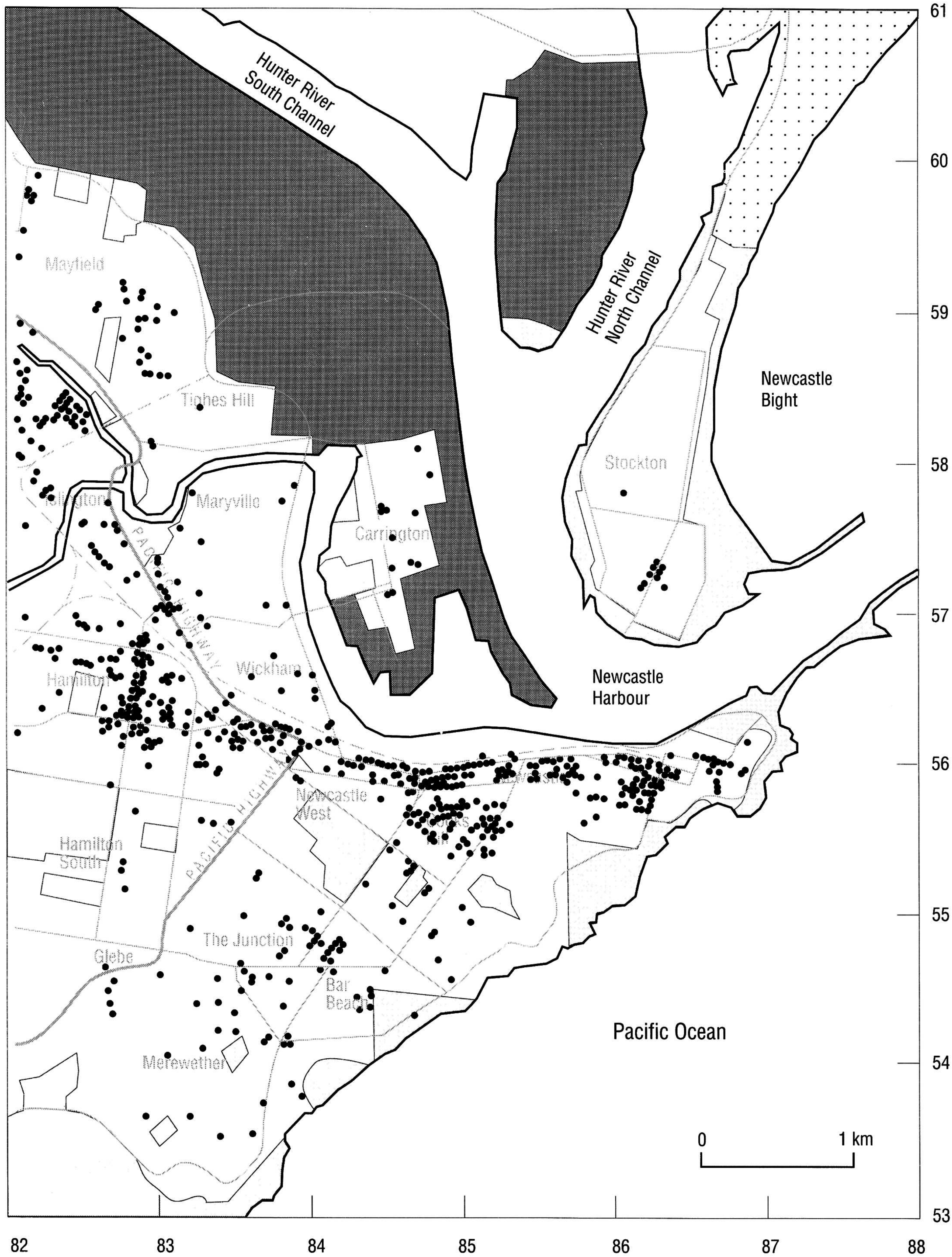



**Percentages of damage to residential buildings**

**Figure 6.10**

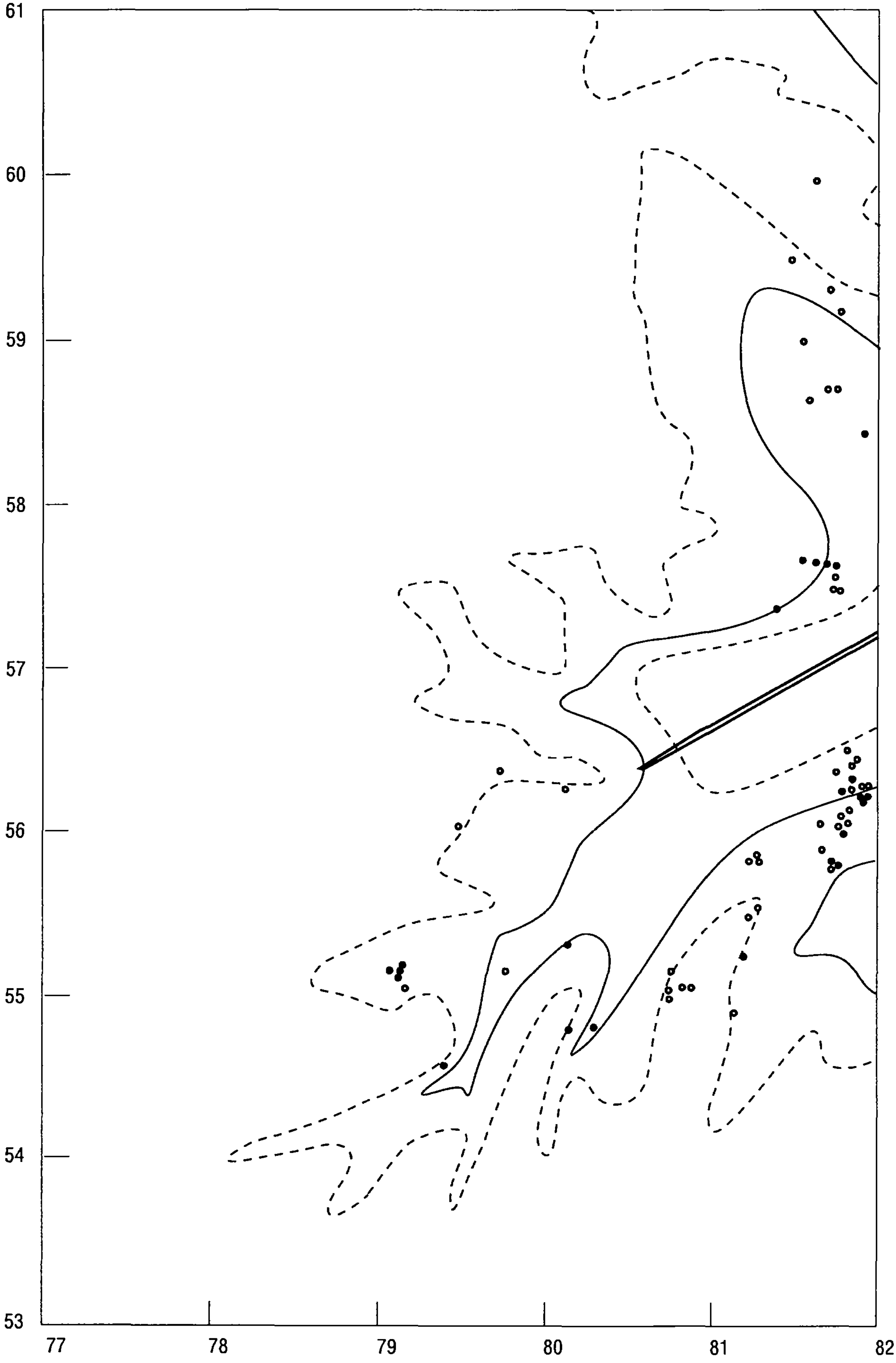



• Location of structures with red or amber damage level




**Locations of structures with moderate or heavy damage from Newcastle City Council data**

Figure **6.11**



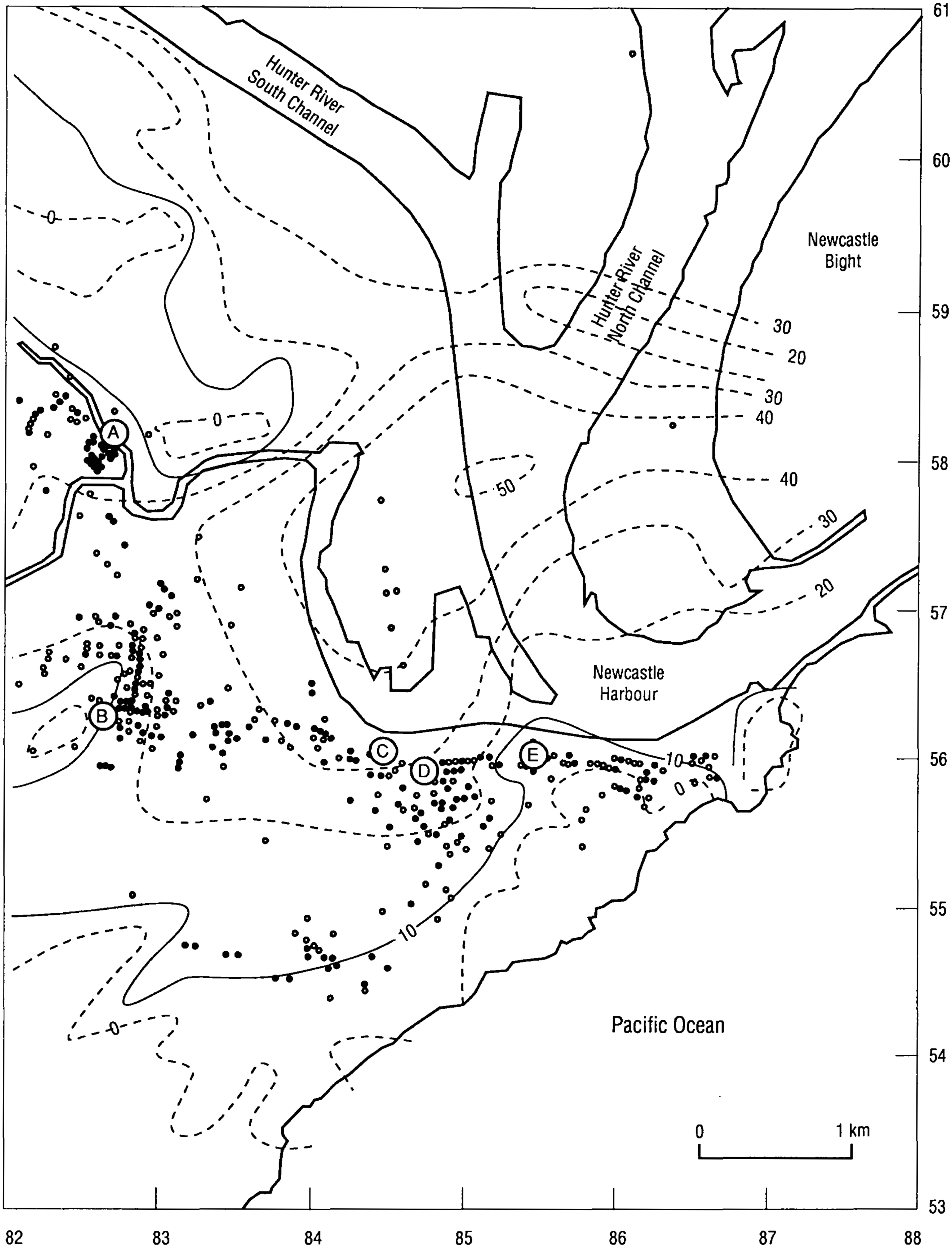

Ⓒ Borehole location

- - - 50 - - - Soil thickness contour (m)

—— 10 —— 10m soil thickness

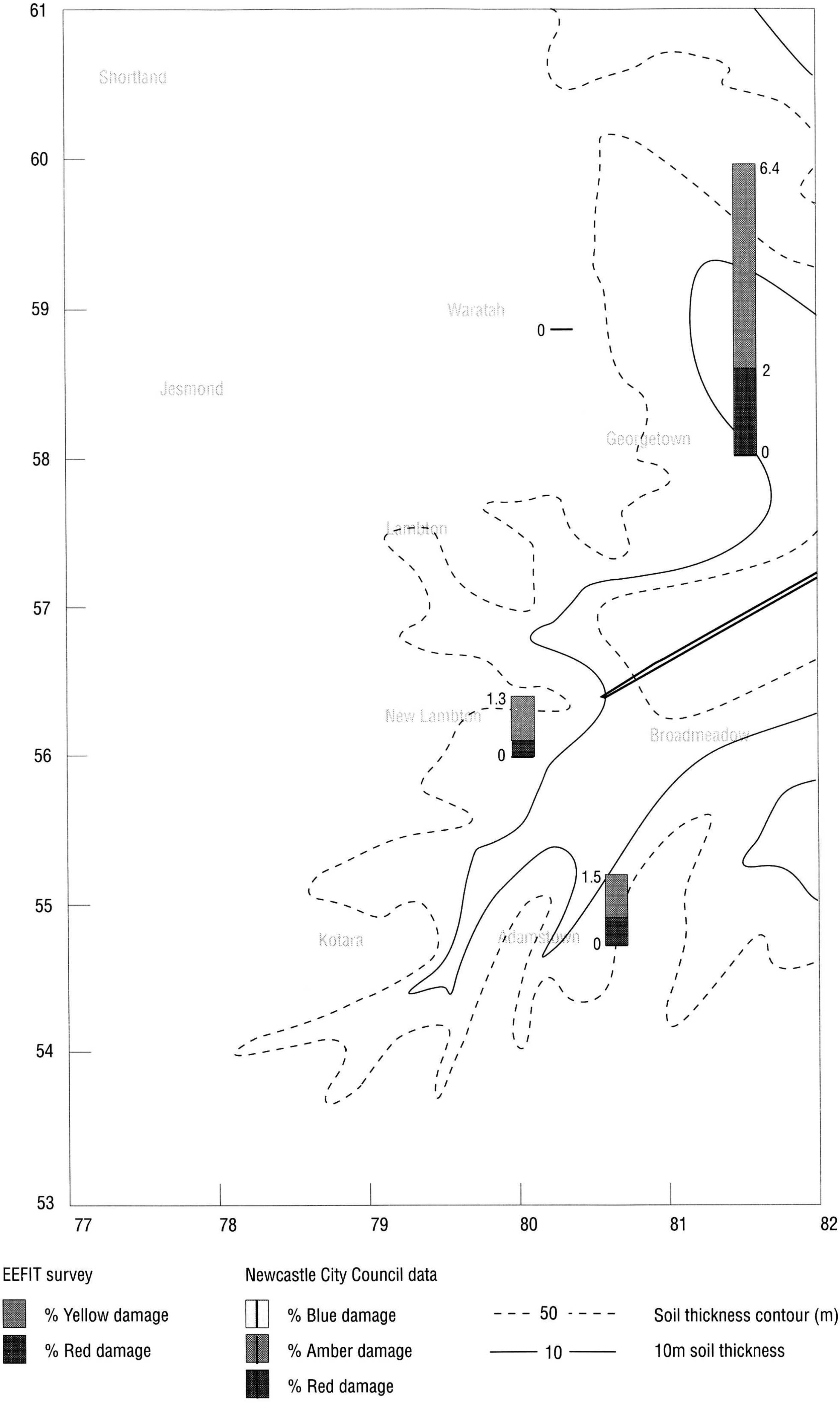
● Red damage level

○ Yellow damage level




**Distribution of damage in relation to soil thickness**

**Figure 6.12**



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Lambton
New Lambton
Broadmeadow
Kotara
Adamstown
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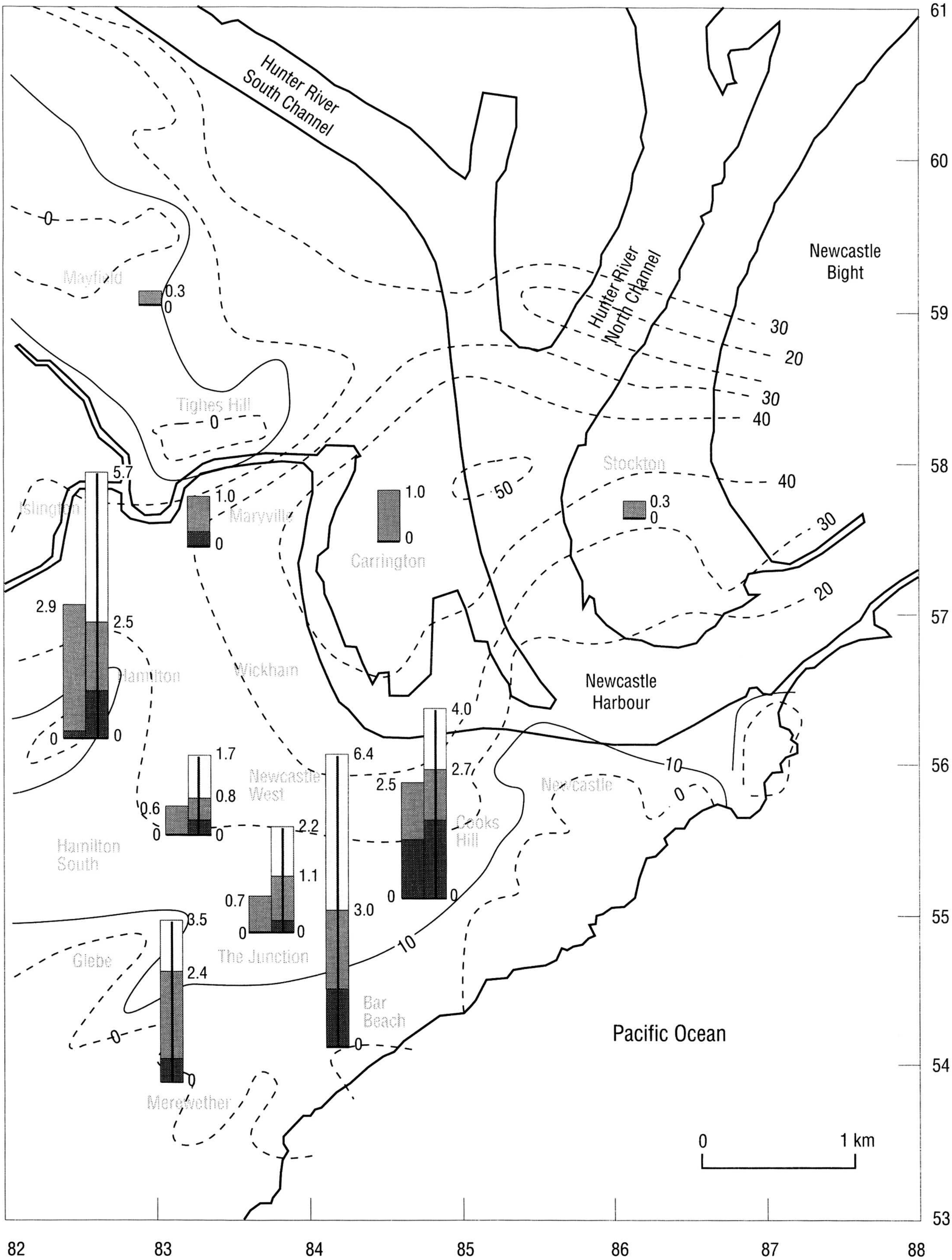
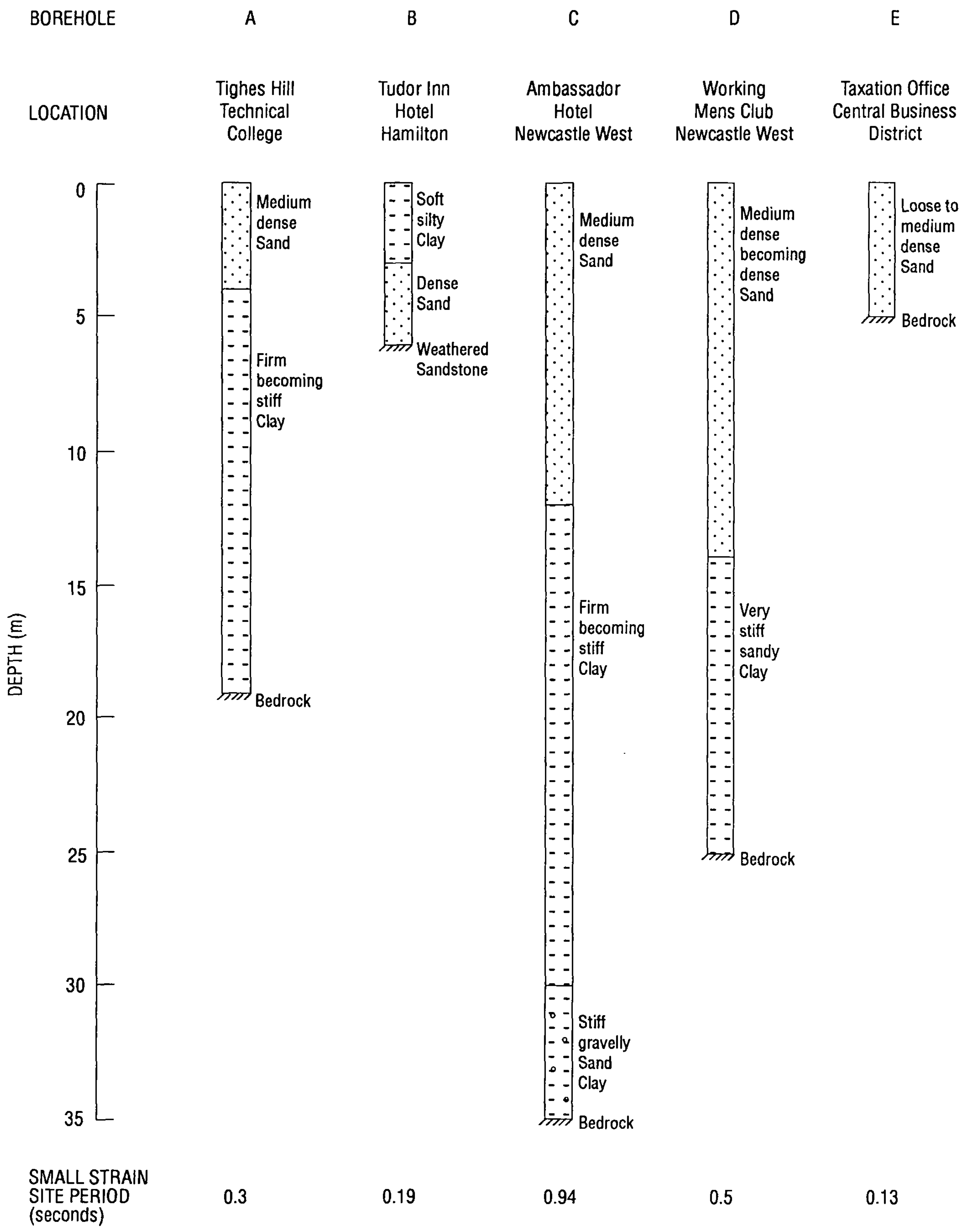
2
0
1.3
0
1.5
0
EEFIT survey
% Yellow damage
% Red damage
Newcastle City Council data
% Blue damage
% Amber damage
% Red damage
50
Soil thickness contour (m)
10
10m soil thickness




**Percentages of damage to residential buildings in relation to soil thickness**

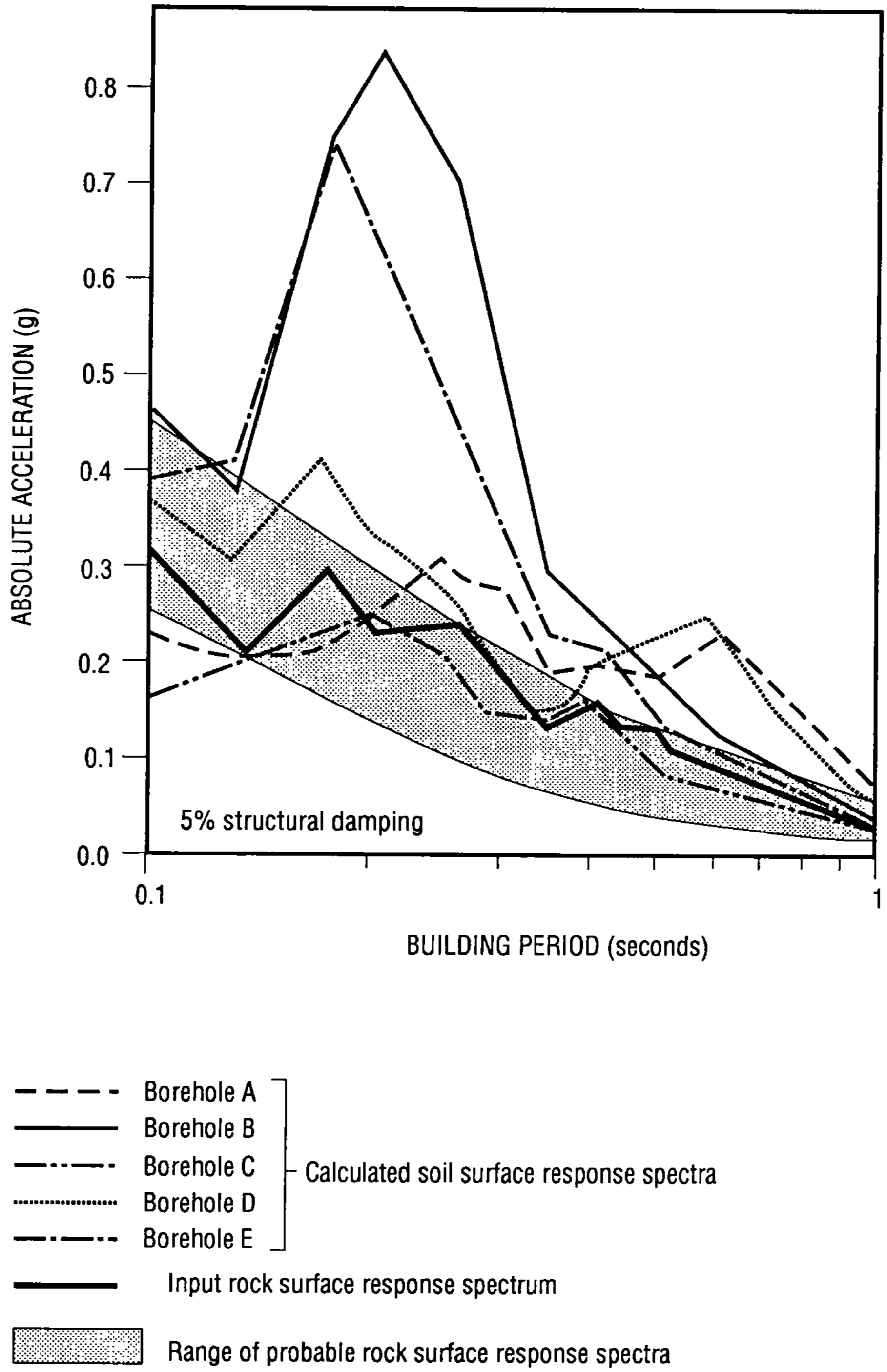
Figure **6.13**




See Figure 6.12 for borehole locations

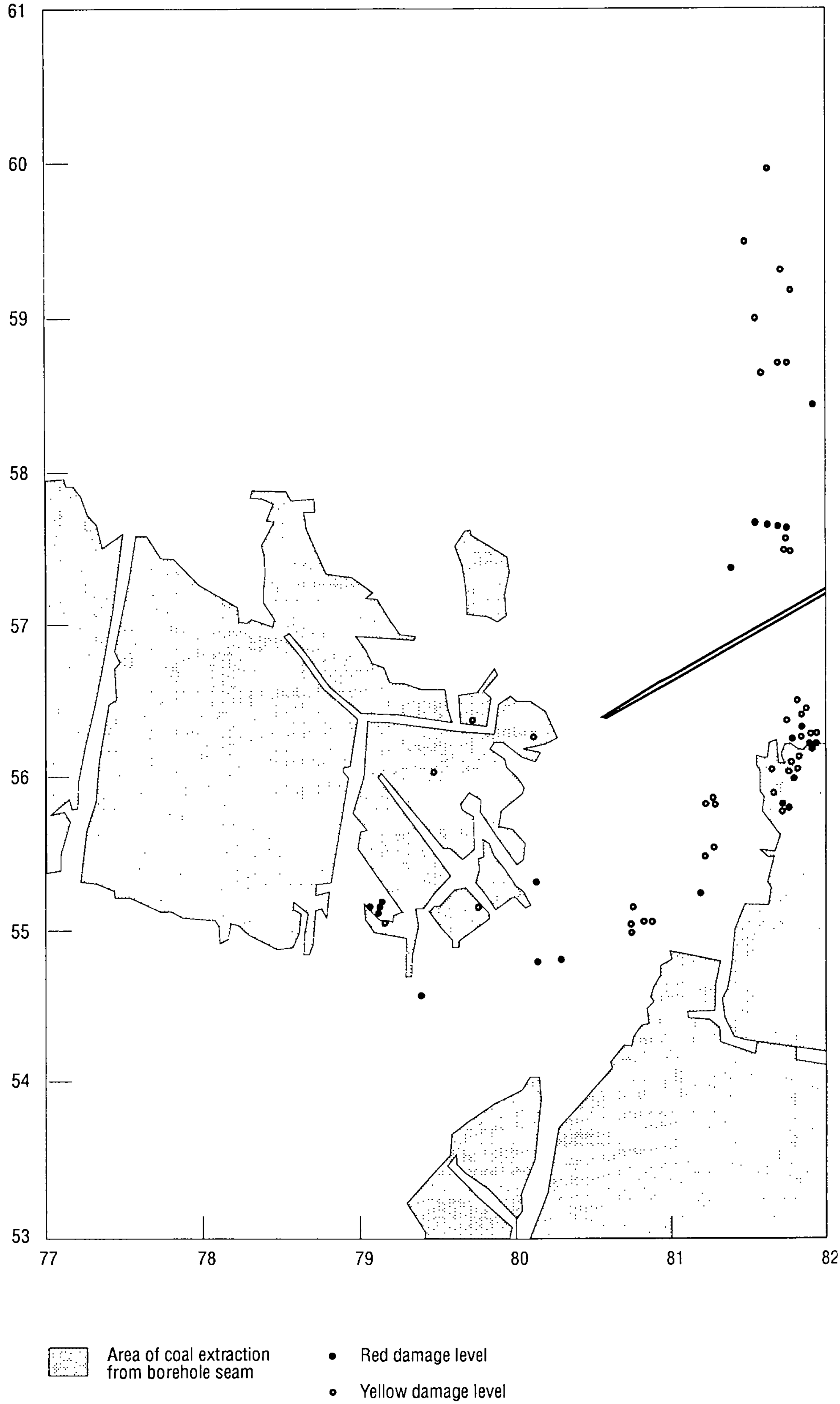
**Borehole data for study of site soil response effects**
**(Courtesy of Coffey and Partners International Pty Ltd)**

**Figure 6.14**




**Response spectra obtained from study of site response effects**

**Figure 6.15**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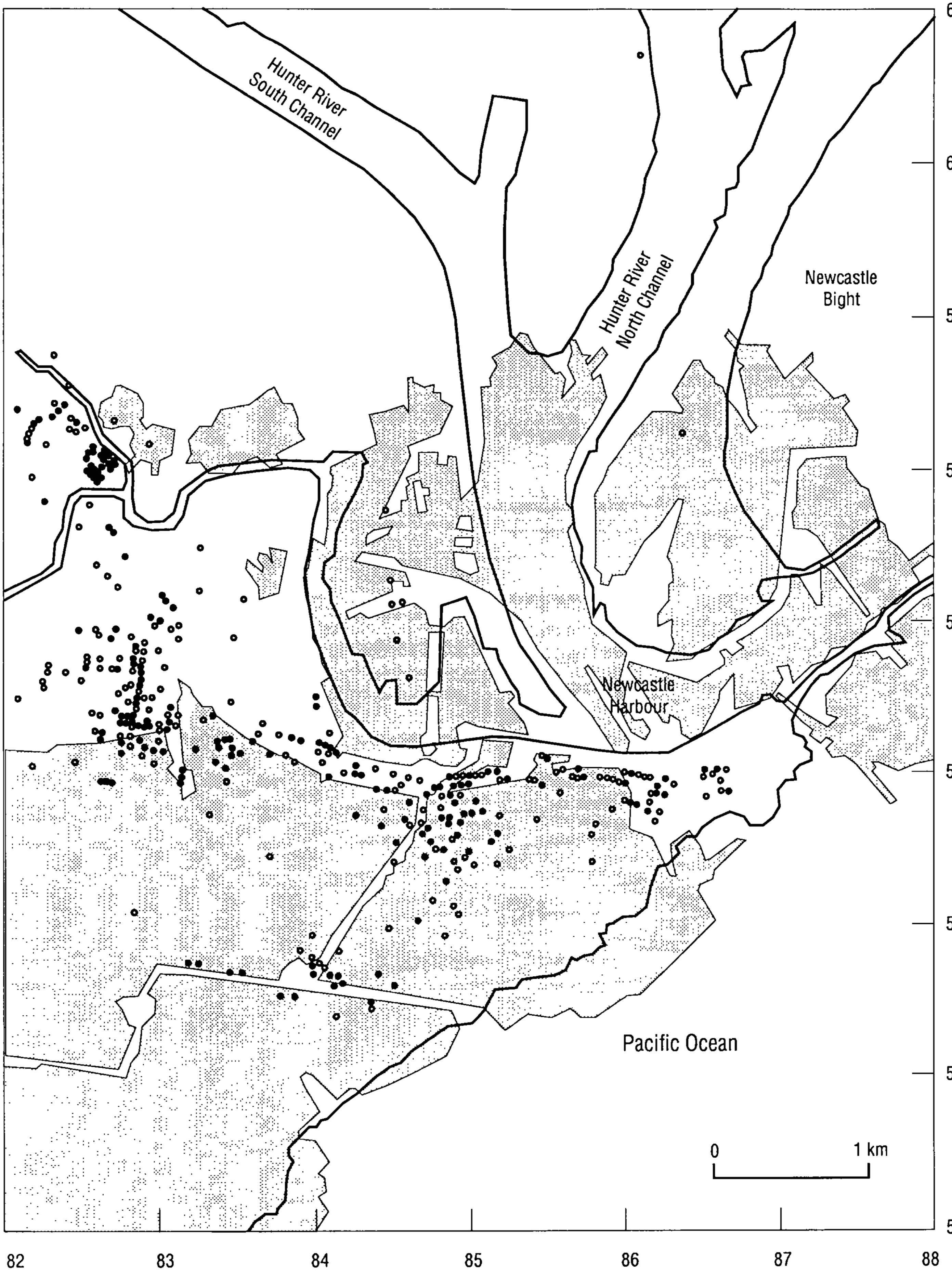
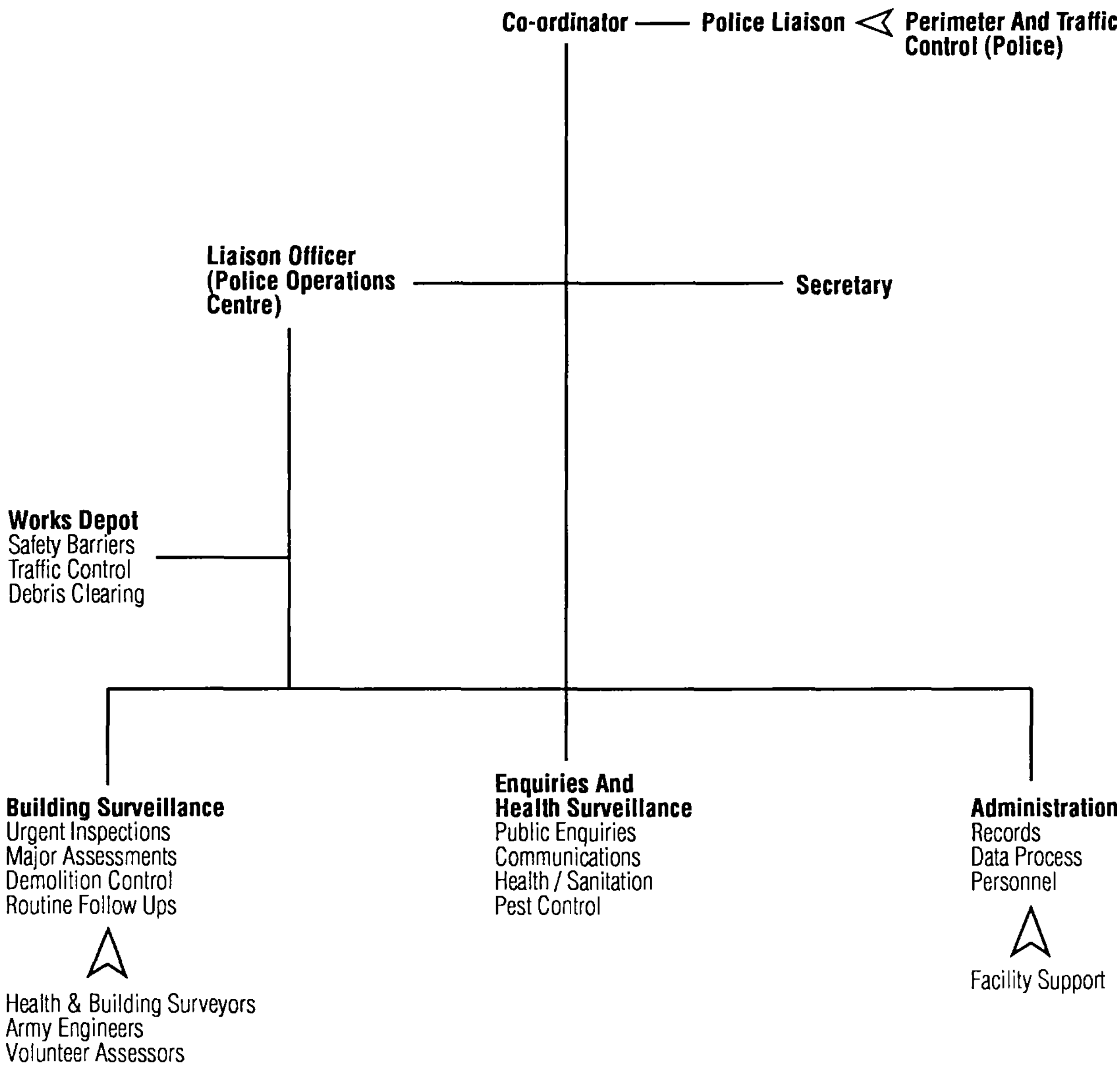
Area of coal extraction from borehole seam
Red damage level
Yellow damage level



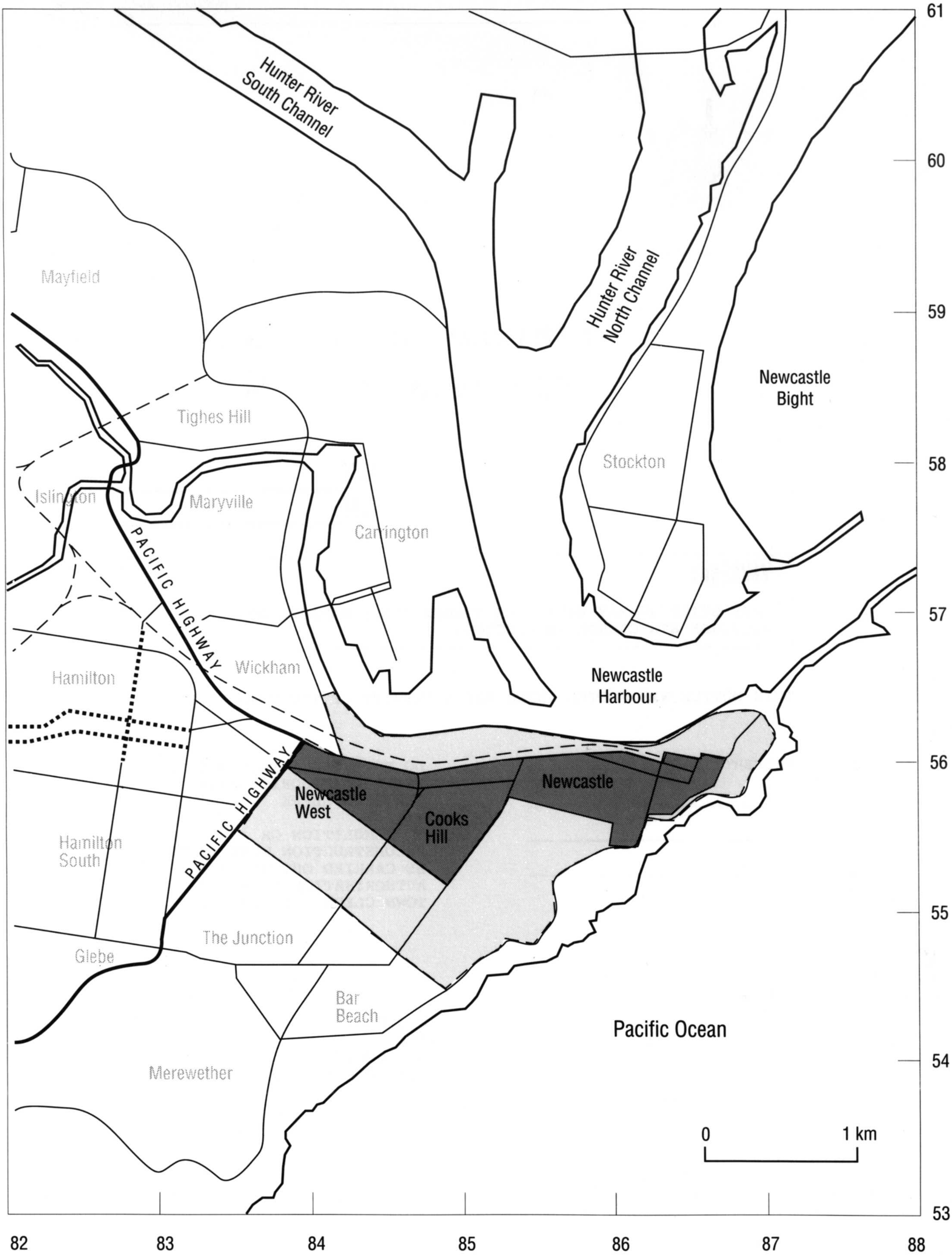

**Distribution of damage in relation to areas of coal extraction**

**Figure 6.16**




**Management structure of the Newcastle City Council Division of Health and Building Services**

**Figure 7.1**




Road closure

Area restricted until 8th January

Area restricted until 4th January

**Perimeter of restricted area imposed in the aftermath of the earthquake**

**Figure 7.2**

Newcastle City Council NEW SOUTH WALES AUSTRALIA

P.O. BOX 489, NEWCASTLE 2300
PHONE: (049) 29-9111
FACSIMILE: (049) 29-6157
DX: 7872 NEWCASTLE

ALL COMMUNICATIONS TO BE ADDRESSED
TO THE TOWN CLERK

# LIMITED ENTRY

## ENTER AT YOUR OWN RISK

DATE__________________

TIME__________________

**WARNING:**

THIS BUILDING MAY HAVE BEEN STRUCTURALLY DAMAGED AND REQUIRES STRUCTURAL INSPECTION.

THIS NOTICE WAS POSTED UNDER THE AUTHORITY OF THE TOWN CLERK.

PROPERTY NAME AND ADDRESS:

__________________________

__________________________

__________________________

DO NOT REMOVE THIS NOTICE WITHOUT AUTHORITY FROM THE TOWN CLERK

NO DEMOLITION OR RECONSTRUCTION WORK IS TO BE CARRIED OUT WITHOUT AUTHORISATION FROM THE TOWN CLERK. PH 299306

SIGNED................
for
**W B LEWIS**
**TOWN CLERK**

**Newcastle City Council Damage Notice (category yellow)**

**Figure 7.3**

Newcastle City Council NEW SOUTH WALES, AUSTRALIA

P.O. BOX 489, NEWCASTLE 2300
PHONE: (049) 29-9111
FACSIMILE: (049) 29-6157
DX: 7872 NEWCASTLE

ALL COMMUNICATIONS TO BE ADDRESSED TO THE TOWN CLERK

# UNSAFE

## ENTER AT YOUR OWN RISK

DATE ____________________

TIME ____________________

**WARNING:**

THIS BUILDING IS DAMAGED AND PRESENTS A DANGER.

ANY ENTRY SHOULD BE RESTRICTED TO OWNER, AND/OR LAWFUL OCCUPIER

THIS NOTICE WAS POSTED UNDER THE AUTHORITY OF THE TOWN CLERK

PROPERTY NAME AND ADDRESS:

________________________

________________________

________________________

DO NOT REMOVE THIS NOTICE WITHOUT AUTHORITY FROM THE TOWN CLERK

NO DEMOLITION OR RECONSTRUCTION WORK IS TO BE CARRIED OUT WITHOUT AUTHORISATION FROM THE TOWN CLERK. PH 299306

SIGNED................
for
**W B LEWIS**
**TOWN CLERK**

**Newcastle City Council Damage Notice (category red)**

Figure **7.4**

NEWCASTLE CITY COUNCIL
RAPID EVALUATION SAFETY ASSESSMENT FORM

| PROPERTY DESCRIPTION: | OVERALL RATING: |
|---|---|
| Name:____________________ | Limited Entry (yellow) ☐ |
| Address:__________________ | Unsafe (red) ☐ |
| __________________________ | INSPECTOR: ______________ |
| __________________________ | |
| No of stories:____________ | Inspection Date: __________ |
| Basement: Yes /No /Unknown | |

PRIMARY OCCUPANCY:
Dwelling: ☐ Other Residential: ☐ Commercial/Office: ☐ Shop: ☐
Industrial: ☐ Public Assembly ☐ School: ☐ Government: ☐ Other: ☐
Heritage Item: ☐

Note: This assessment is to classify public footpaths and roadways as safe for passage and to classify buildings which may be entered by the owner and/or occupier. It is made from an external inspection of building faces to the street.

Every building is to be posted with this notice at the time of this assessment.

Instructions: Review building for the conditions listed below. Post the building UNSAFE if the answers to 1,2,3 or 4 is yes, or in your discretion having regard to observed conditions. Otherwise, post LIMITED ENTRY, pending the owner's report.

| CONDITION: | *Yes* | *No* |
|---|---|---|
| 1. Collapse, partial collapse, or building leaning off foundation:....... | ☐ | ☐ |
| 2. Building or storey noticeably leaning:....................................... | ☐ | ☐ |
| 3. Severe racking of walls, obvious severe damage and distress:........ | ☐ | ☐ |
| 4. Apparent danger from neighbouring building:.............................. | ☐ | |
| 5. Chimney, parapet or other falling hazard:.................................. | ☐ | ☐ |
| 6. Awning Unsafe:........................................................................ | ☐ | ☐ |
| 7. Other hazard present:............................................................. | ☐ | ☐ |

COUNCIL WORKS ACTION REQUIRED:
1. Prop unsafe awnings [specify] :______________________________
_____________________________________________________________
_____________________________________________________________
2. Barricades required as follows:______________________________
[draw plan using overleaf if necessary]__________________________
_____________________________________________________________
3. Other:______________________________________________________
_____________________________________________________________

OTHER COMMENTS:_____________________________________________
_____________________________________________________________
_____________________________________________________________
_____________________________________________________________

| OFFICE USE ONLY: | *Signed* | *Date* |
|---|---|---|
| 1. Heritage assessment:______________ | ________ | ________ |
| ___________________________________ | ________ | ________ |
| 2. Health and Building endorsement__________ | ________ | ________ |
| 3. Copy of this form sent to Council Works Engineer:____ | ________ | ________ |
| 4. Owner advised:______________ | ________ | ________ |
| 5. Works completed by Council Works Engineer and form returned to Health and Building:__________ | ________ | ________ |
| 6. Computer records updated:______________ | ________ | ________ |

**Newcastle City Council Rapid Evaluation Safety Assessment Form**

**Figure 7.5**

Newcastle City Council NEW SOUTH WALES, AUSTRALIA.

ALL COMMUNICATIONS TO BE ADDRESSED
TO THE TOWN CLERK

P.O. BOX 489, NEWCASTLE 2300
PHONE: (049) 29-9111
FACSIMILE: (049) 29-6157
DX: 7872 NEWCASTLE

AUTHORISATION

The owner of the premises situated at ______________________________

______________________________________________________________

is authorised to demolish/repair the following unsafe and

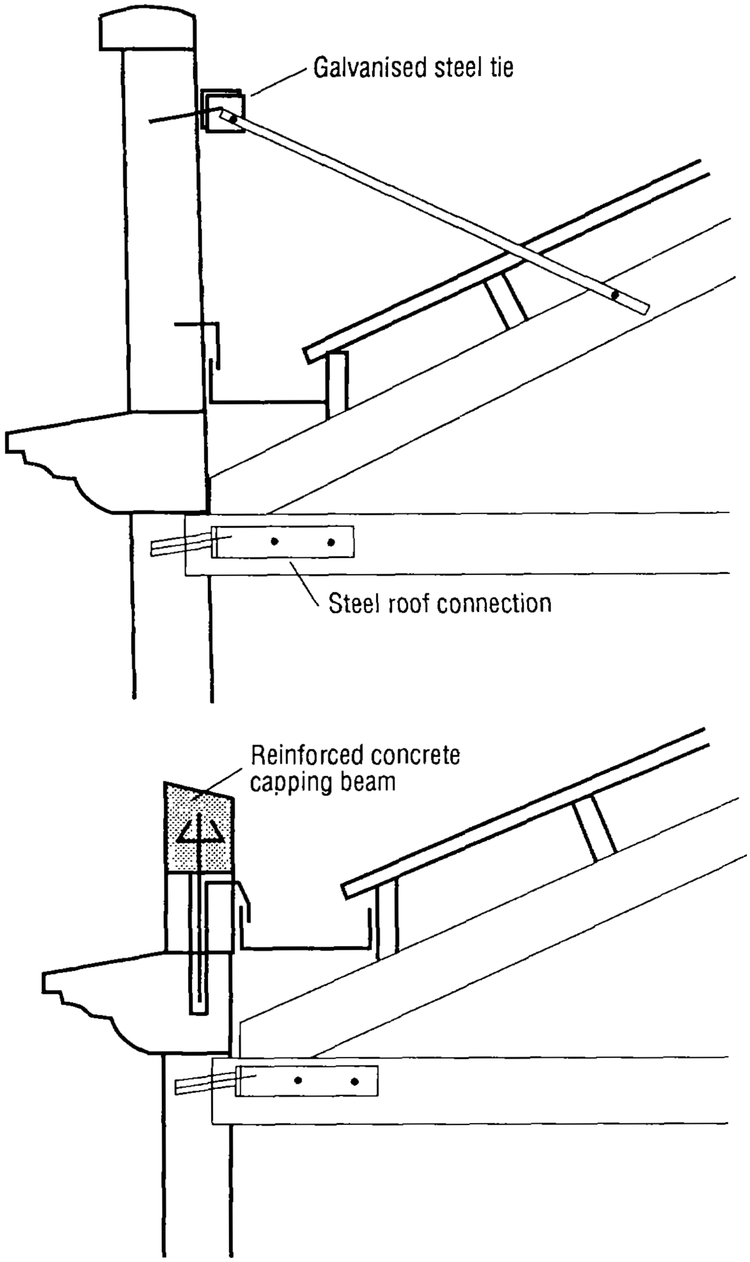
dangerous hazards:-

for
W B LEWIS
TOWN CLERK ________________________ DATE: ___________________

NOTE: While Council is giving consent only to undertake the works recommended by the Structural Engineer and requested by the owner to make the premises safe, it is the owner's responsibility to obtain consents from insurers, financiers and other parties having pecuniary interests in the premises to undertake this work and any further works in respect of these premises.

**Newcastle City Council Authorisation Form for building repairs or demolition**

Figure **7.6**




**Recommended arrangements for tying or attaching parapets**

**Figure 8.1**

# PLATES

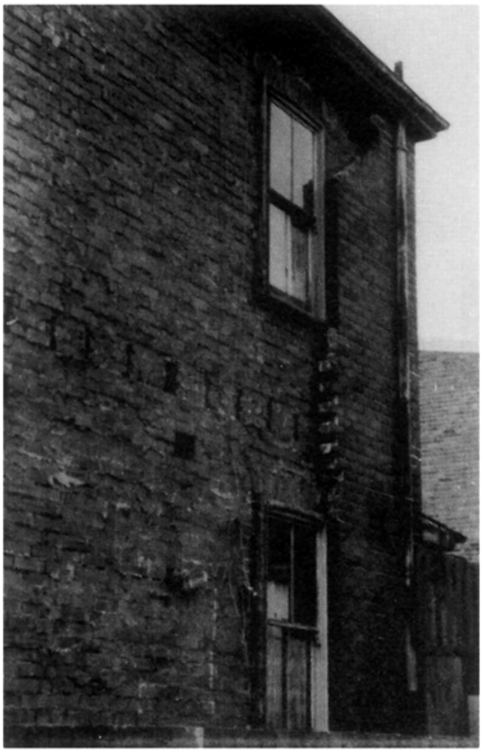


PLATE 5.1 *Collapse of outer skin of cavity unreinforced brick masonry wall (location 18)*

PLATE 5.2 *Corroded ties in cavity brick masonry wall construction (location 18)*


PLATE 5.3 *Collapse above first floor level of solid double unreinforced brick masonry wall (location 25)*

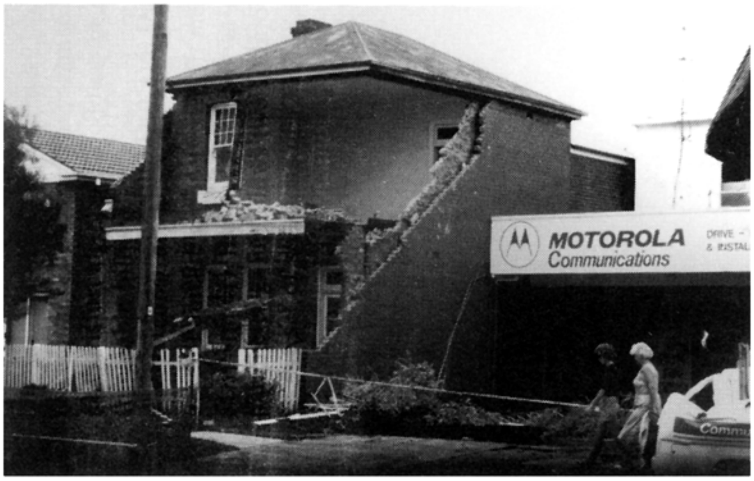



PLATE 5.4 *Front and side wall collapse in modern unreinforced brick masonry two storey dwelling (location 12)*

PLATE 5.5 *Gable end wall collapse in residential two storey brick masonry building (location 11)*

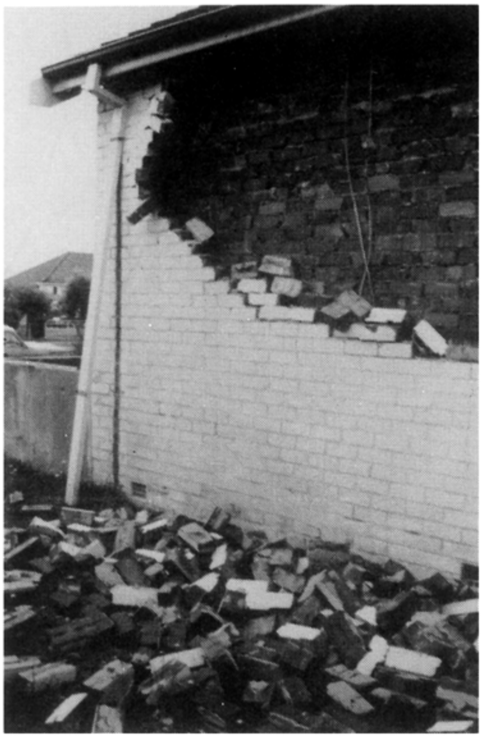


PLATE 5.6 *Outer skin cavity wall failure in older residential brick masonry building (location 4)*

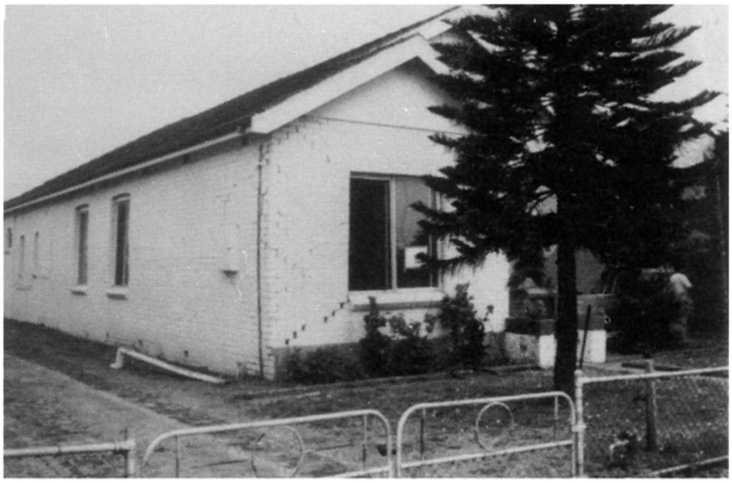


PLATE 5.7 *Diagonal shear cracking near corners of brick masonry dwelling (location 4)*

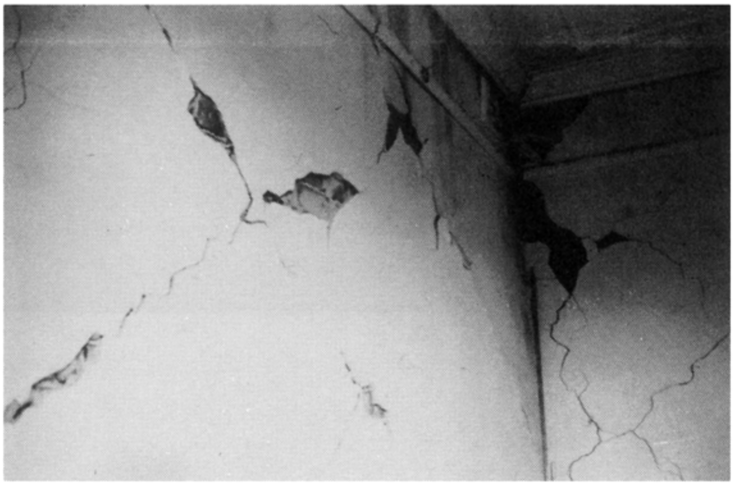
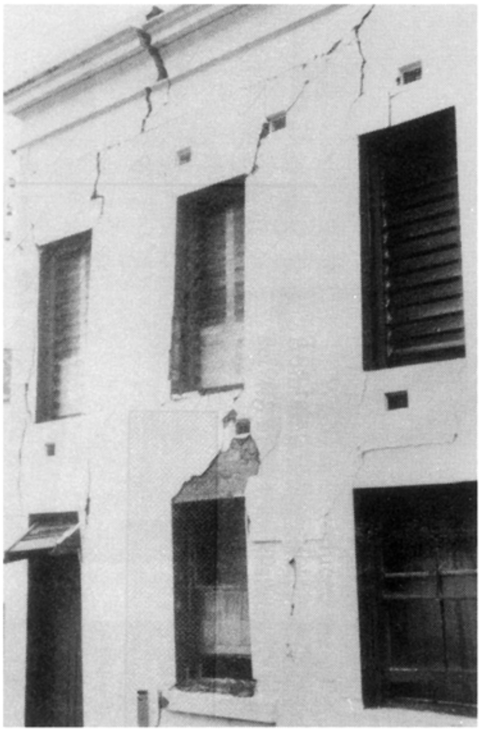


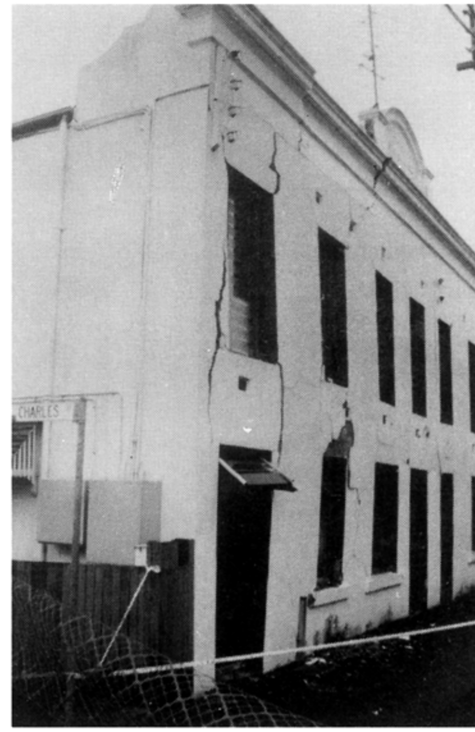
PLATE 5.8 *Cracking of internal load-bearing walling in single-storey brick*

PLATE 5.9 *Damage to exterior ornamental brick columns and arches (location 8)*

PLATE 5.10 *Secondary damage to buildings on right occurred due to side wall collapse in the left-hand building (location 23)*

PLATE 5.11 *Secondary damage to roofing due to chimney and side wall collapse in an adjacent building (location 37)*






PLATES 5.12 and 5.13 *Cracking and leaning of parapets and facades in an older two storey brick masonry building (location 24)*

PLATE 5.14 *Modern three storey unreinforced brick masonry apartment building with severe damage to left side-wall (see PLATES 5.16, 5.17) (location 40)*

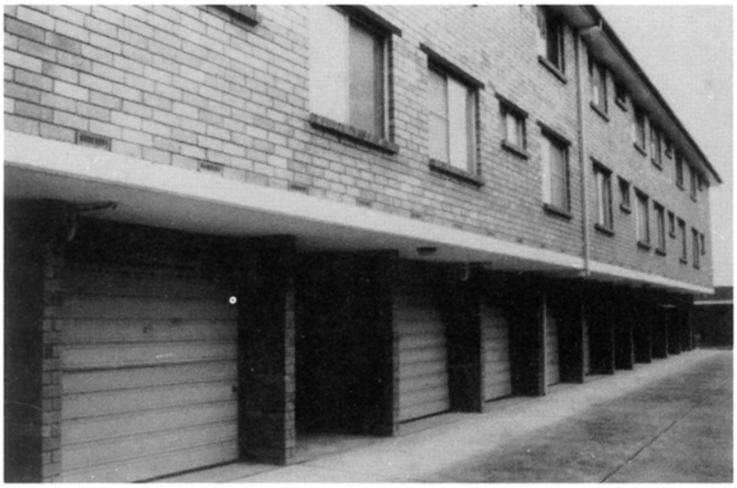
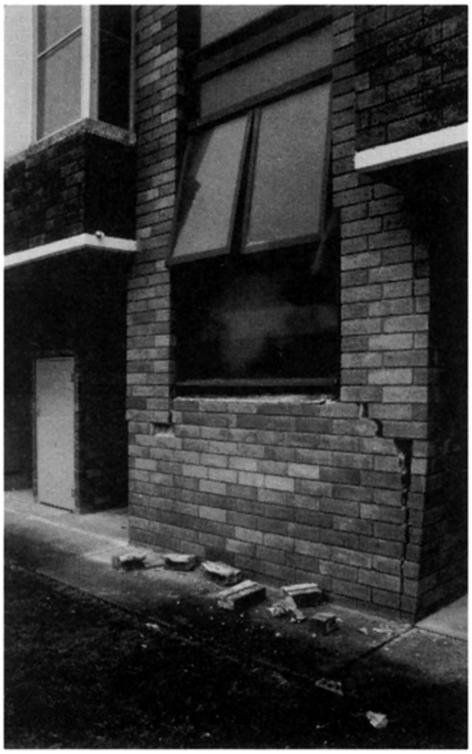


PLATE 5.15 *Ground floor garage entrances to three storey apartment building shown in PLATE 5.14. Damage to opposite side wall is shown below*



PLATES 5.16 and 5.17 *Severe cracking in unreinforced brickwork near corners, doors and window openings at first storey level of the building shown in PLATES 5.14 and 5.15*

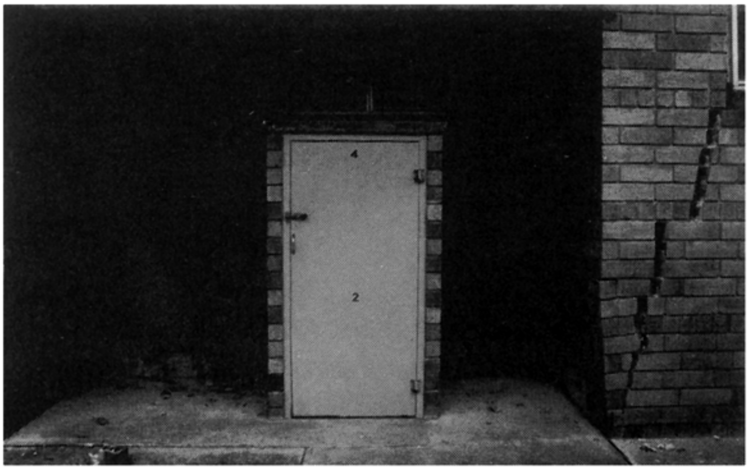


PLATE 5.18 *Heavily damaged section of Beaumont Street, Hamilton, taken 12 days after the earthquake*

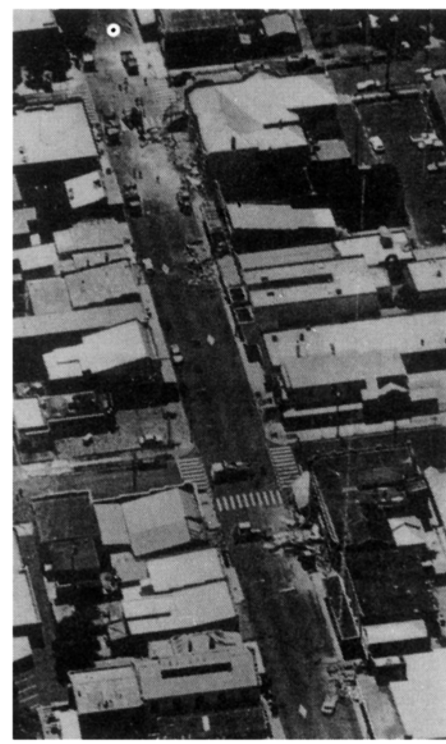
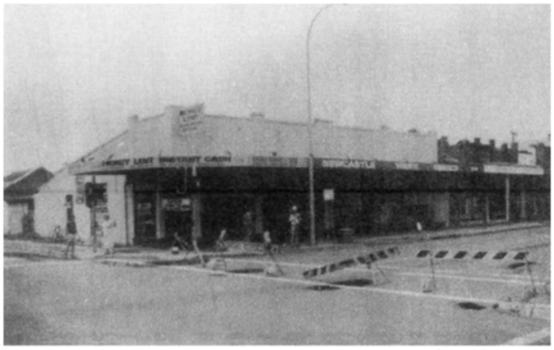
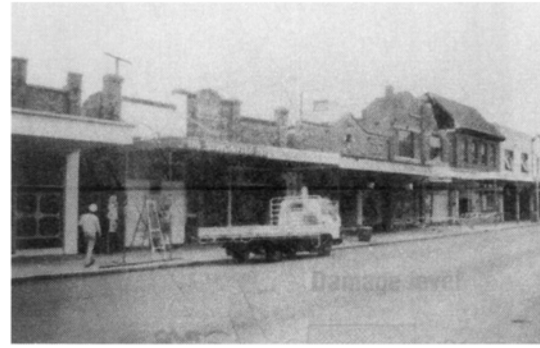


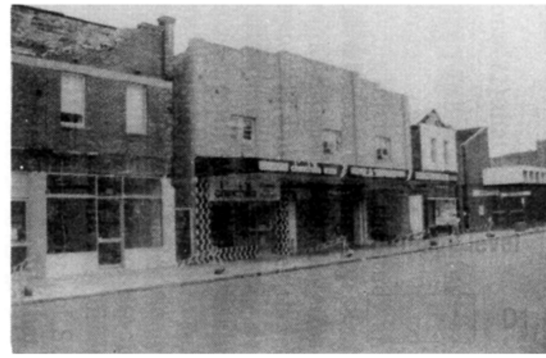
PLATE 5.19 *Aerial photograph of Beaumont Street, Hamilton, taken immediately after the earthquake*

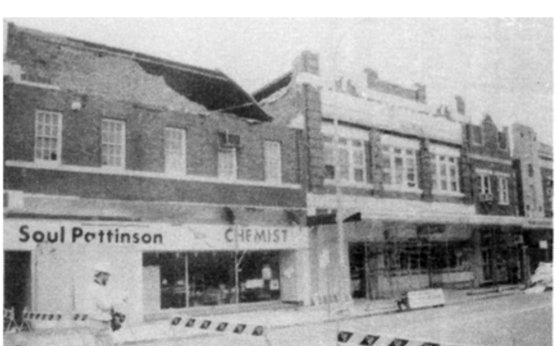
PLATE 5.20 *Rescue operations and emergency strengthening of a partially collapsed two storey brick masonry building in Beaumont Street, Hamilton*





PLATES 5.21 - 5.26 *Sequence of photographs taken of the eastern side of Beaumont Street, Hamilton, 12 days after the earthquake*






PLATES 5.27 - 5.32 *Sequence of photographs taken of the eastern side of Beaumont Street, Hamilton (continued from previous page)*

PLATE 5.33 *Collapsed second storey brick masonry facade in two storey commercial building. The remaining facade is leaning dangerously away from the building (location 21)*


PLATE 5.34 *Long awning tied back to second storey facade, a typical feature of older commercial buildings in Newcastle (location 15)*

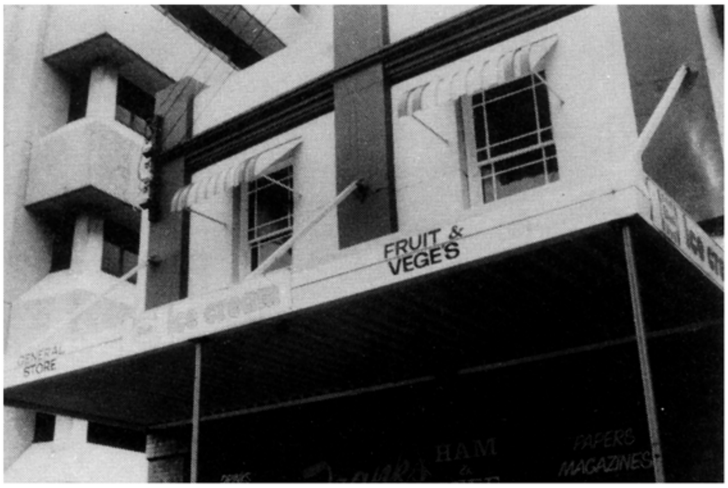
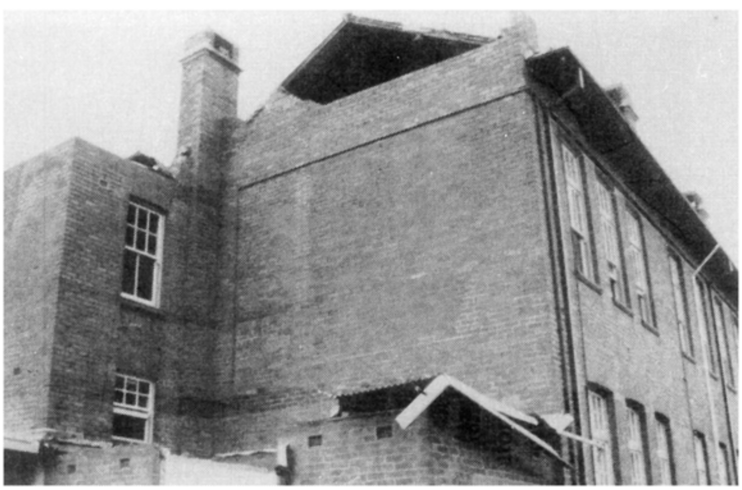



PLATE 5.35 *Cracking of facade near tie-back positions for suspended awning*


PLATE 5.36 *Commercial building in the Newcastle central business district with an unstable facade following the earthquake (location 13)*

PLATE 5.37 *Failure of tall parapet in a modern commercial unreinforced brick masonry building (location 10)*


PLATE 5.38 *Gable end failures in a group of modern two storey commercial brick masonry buildings (location 2)*

PLATES 5.39 and 5.40 *Extensive damage to roof, parapets and gable ends in a 100 year old school in The Junction, Newcastle (location 38)*

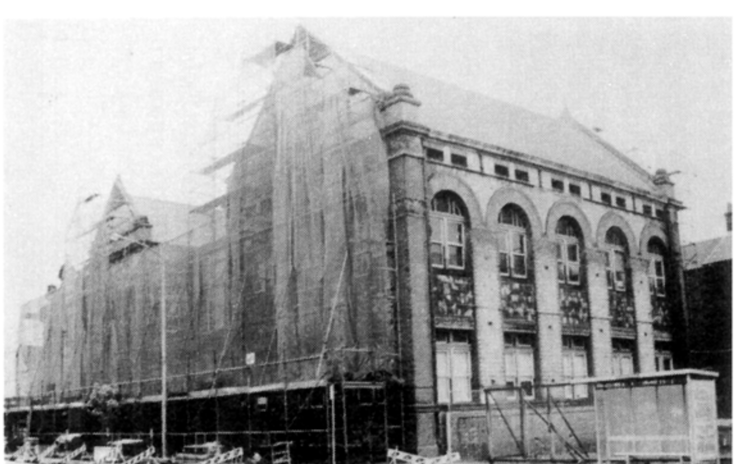
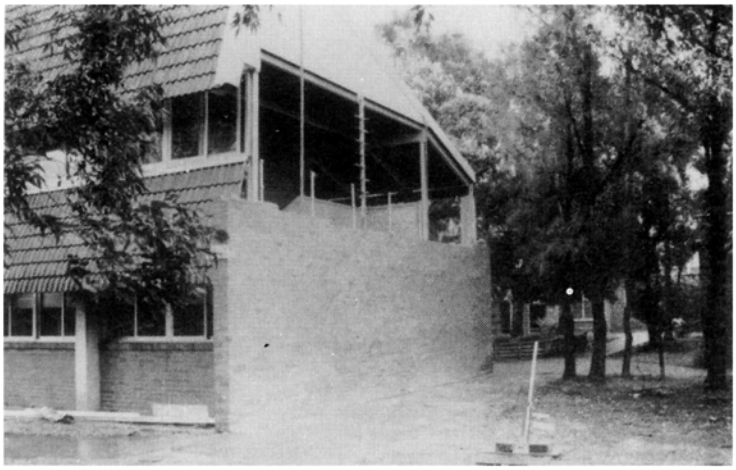


PLATE 5.41 *Extensive cracking was observed in the facade of this brick masonry college building in the Newcastle central business district (location 17)*



PLATES 5.42 and 5.43 *Partial collapse of unreinforced brick masonry walls due to inadequate tying back to building or roof frames (location 3)*

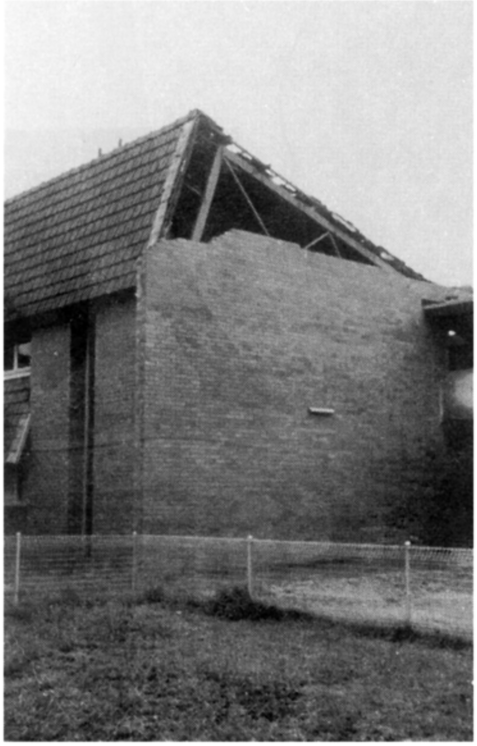


PLATE 5.44 *Brick masonry parapet collapse at the Tighes Hill Technical College, Newcastle (location 6)*

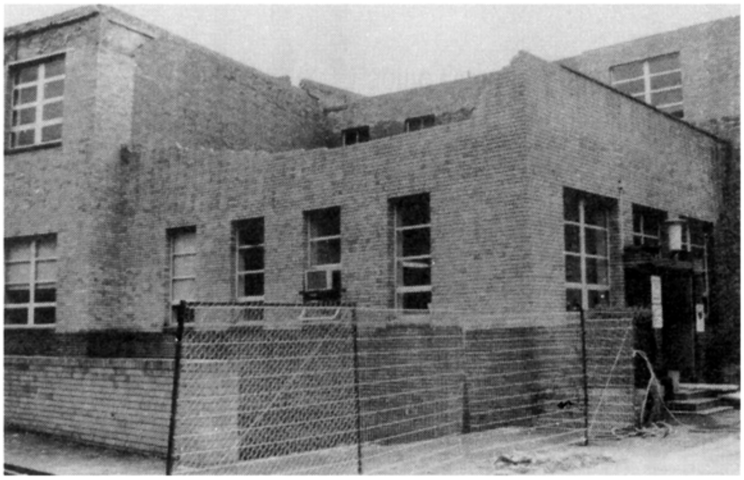


PLATE 5.45 *Damage to brick masonry cladding at the Tighes Hill Technical College, Newcastle (location 6)*

PLATE 5.46 *Diagonal shear failure in internal masonry walls in-filling reinforced concrete frame structure, Tighes Hill Technical College*

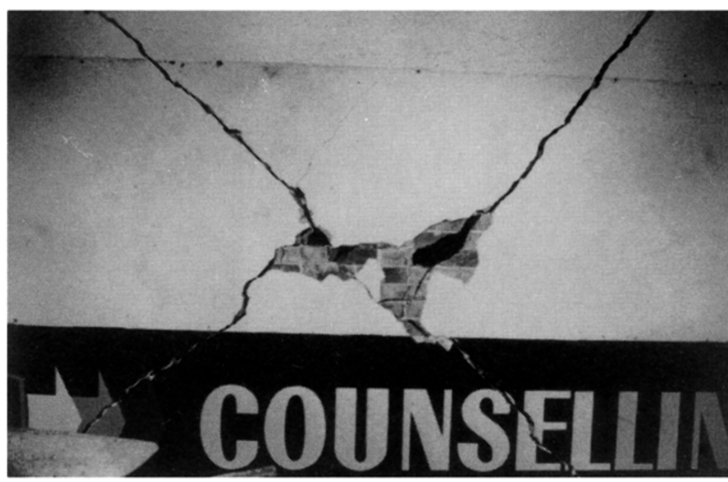



PLATE 5.47 *The earthquake caused a fire to start in a third floor laboratory at the Tighes HIll Technical College, but was soon brought under control*

PLATE 5.48 *Extensive internal damage and plant failure was reported in the North Wing of the Royal Newcastle Hospital, which was evacuated after the earthquake (location 36)*

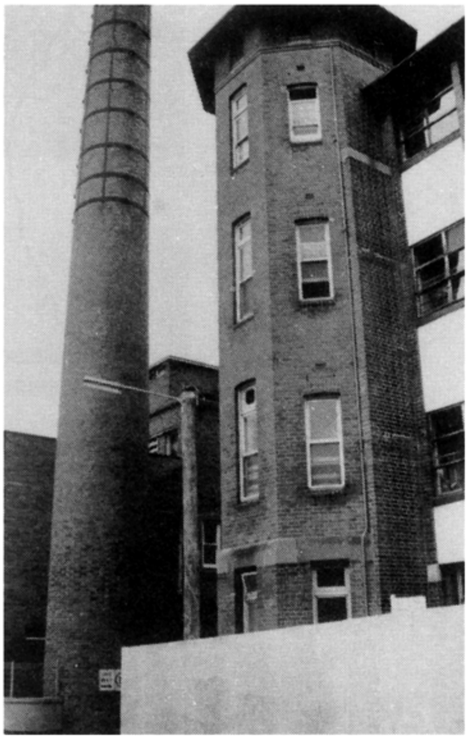


PLATE 5.49 *The 30m high brick chimney at the Royal Newcastle Hospital showed no signs of earthquake damage*

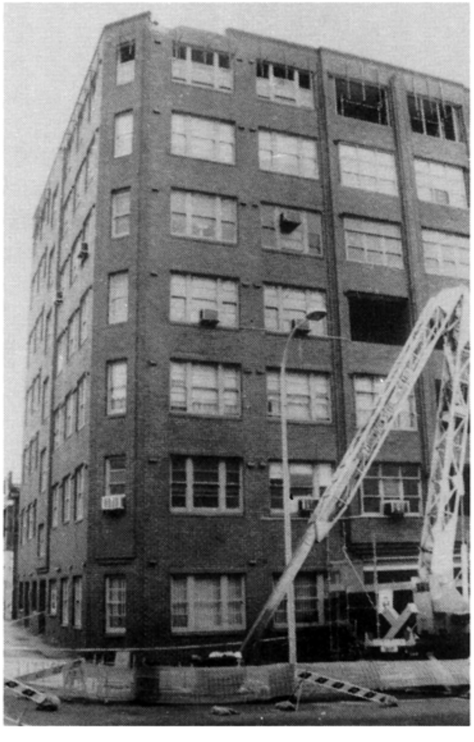
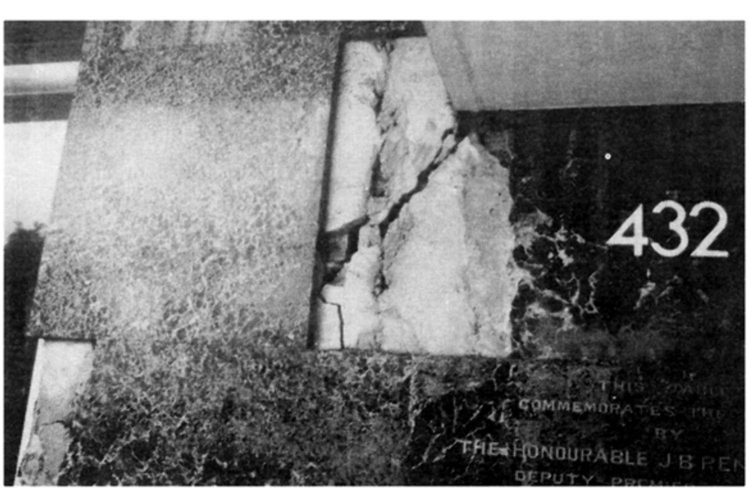
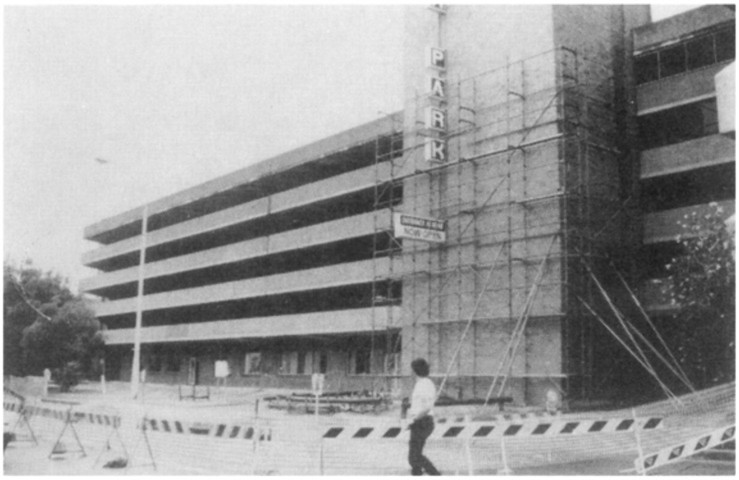


PLATE 5.50 *Parapet failure in a brick clad, seven storey steel framed office building in the Newcastle central business district (location 34)*

PLATE 5.51 *Partial collapse of parapet in the steel-framed Hamilton Telephone Exchange building (location 9)*




PLATES 5.52 and 5.53 *Loss of cladding and cracking of masonry in-fill in a modern office building in King Street, Newcastle (location 19)*




PLATES 5.54 and 5.55 *Cracking of unreinforced masonry cladding at a reinforced concrete multi-storey car park building in King Street, Newcastle (location 22)*

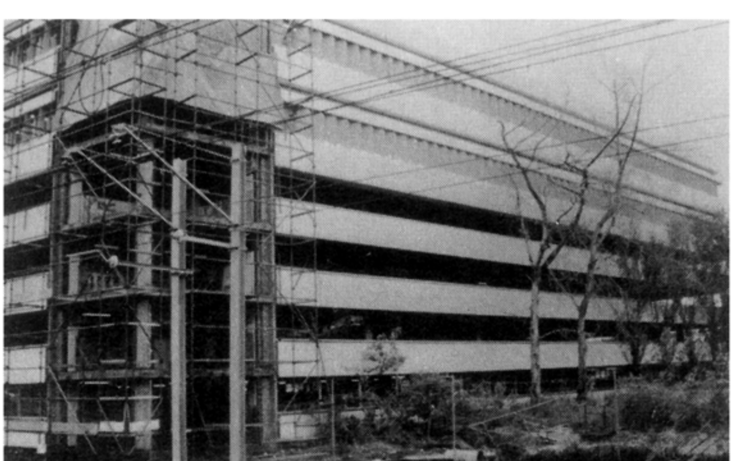



PLATE 5.56 *Total collapse of the lower five storeys of unreinforced brick masonry walling attached to reinforced concrete frame car park and office building (location 29)*


PLATES 5.57 and 5.58 *There was no recorded earthquake damage to these two modern multi-storey reinforced concrete frame and frame-shear wall buildings in the Newcastle central business district (locations 28 and 14)*

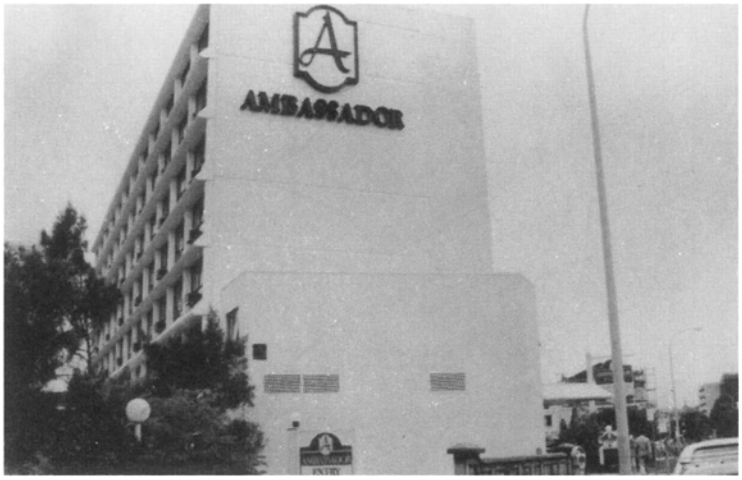
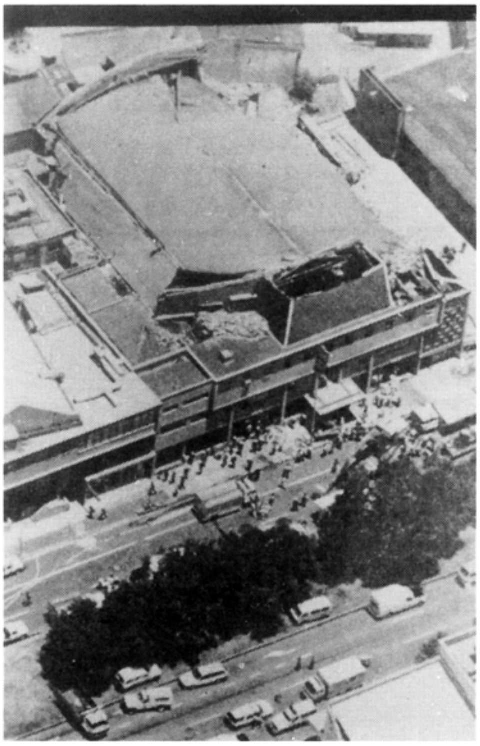
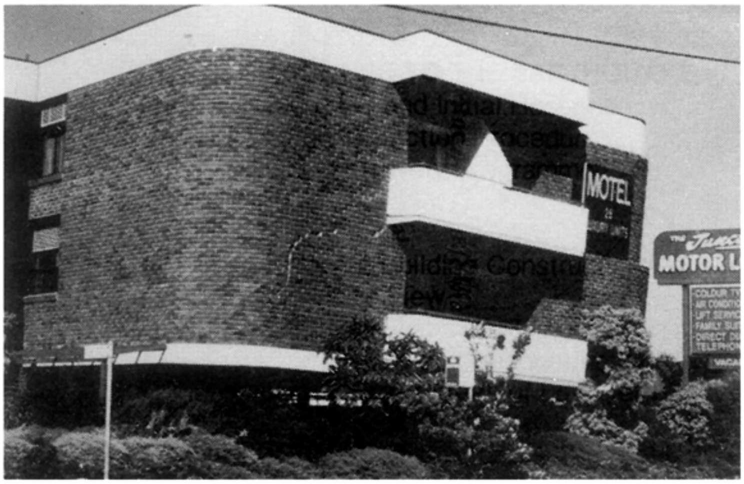



PLATE 5.59 *The Newcastle City Council offices (circular building) in the centre of the city suffered no damage, and operated as the focus for monitoring and directing the building damage assessment surveys (location 27)*

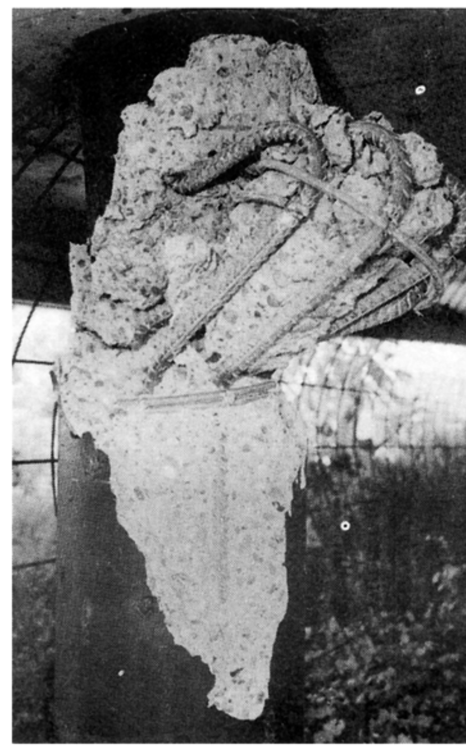
PLATE 5.60 *The police station at the eastern end of the central business district. This building was designed to resist a moderate earthquake and was undamaged (location 33)*



PLATES 5.61 and 5.62 *Extensive structural damage and collapse at the Newcastle Working Mens' Club, where nine persons died in the earthquake (location 20)*




PLATES 5.63 and 5.64 *The combination of a soft first storey and a highly asymmetric stiffness layout led to shear failure and crushing at the top of a row of exterior reinforced concrete columns at the Junction Motel. This building was demolished immediately after the earthquake (location 39)*



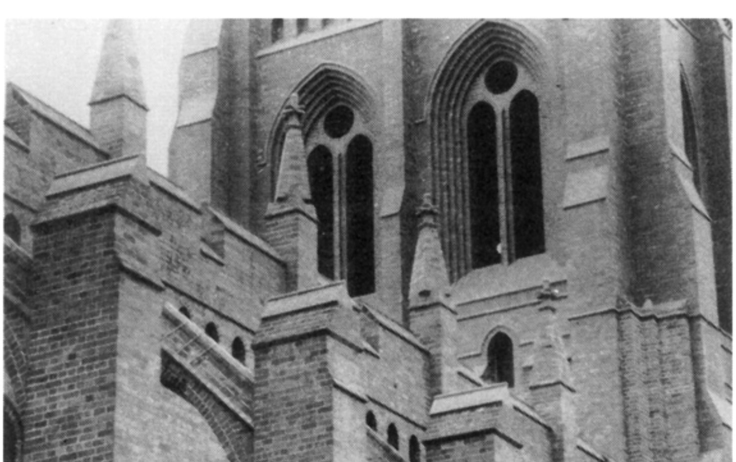
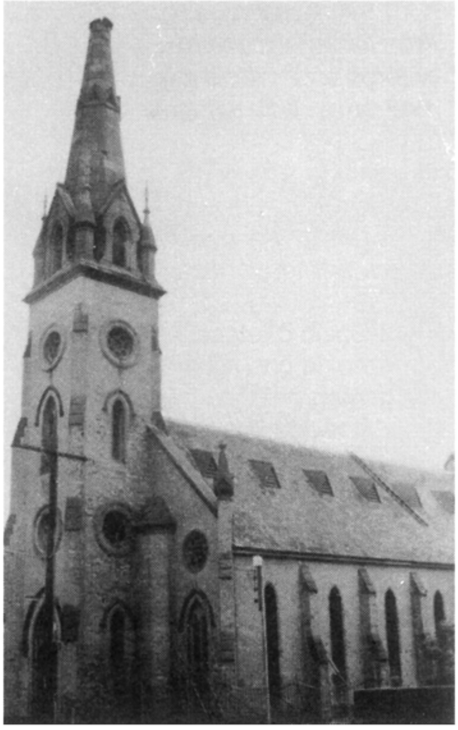
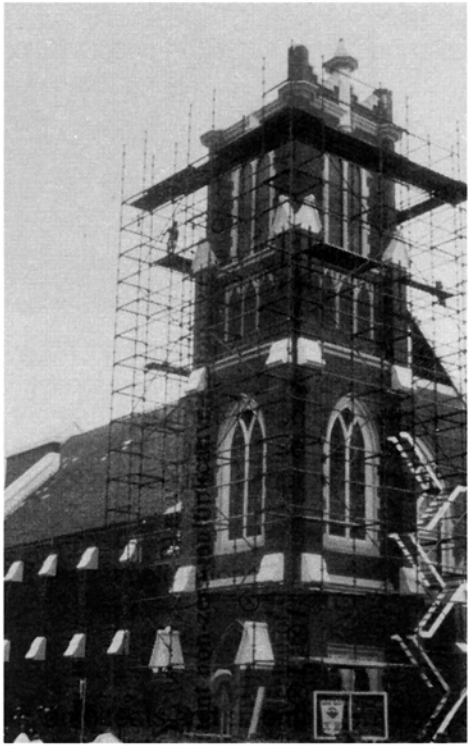


PLATE 5.65 *Rotation of finials mounted on top of the brick masonry parapet at Christ Church Cathedral, Newcastle (location 32)*

PLATE 5.66 *Vertical crack in the outer wall of the Newcastle Christ Church Cathedral, reportedly dating from the earthquake in 1925 (location 32)*





PLATES 5.67 and 5.68 *Partial collapse and severe cracking of masonry spire and tower, a common form of damage to churches in Newcastle (locations 26 and 7)*

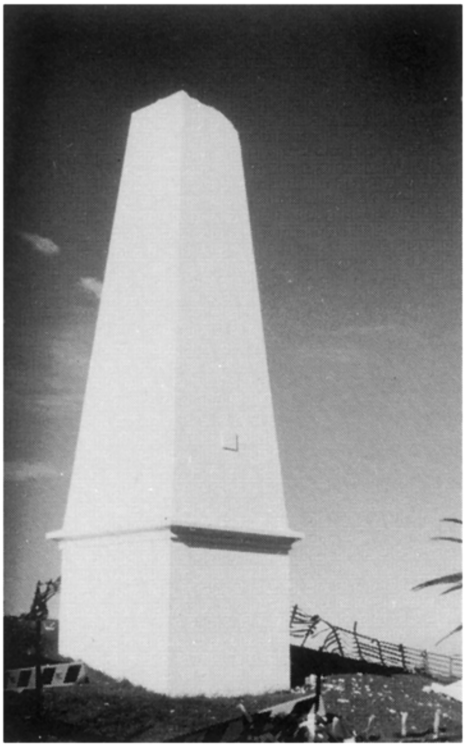


PLATE 5.69 *The war memorial lost its capstone as a result of the earthquake (location 31)*

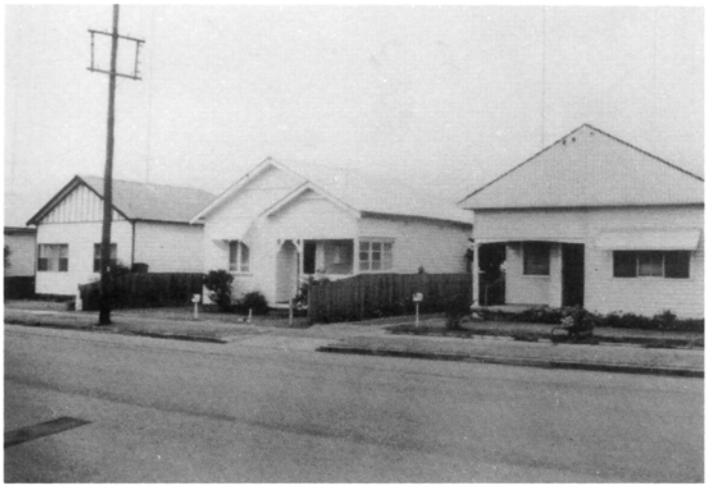
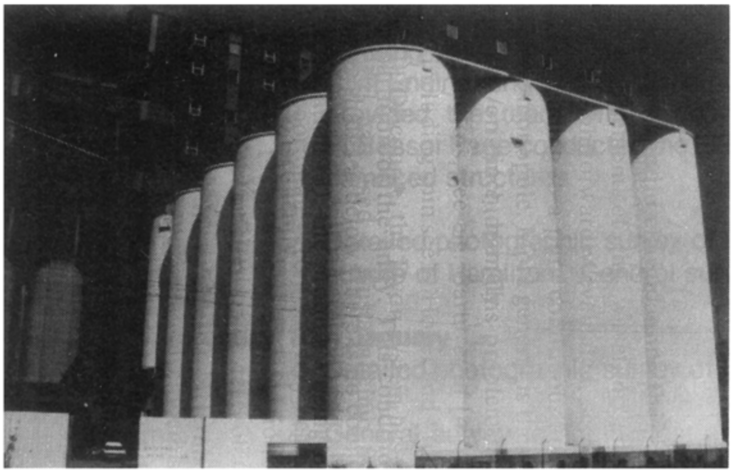
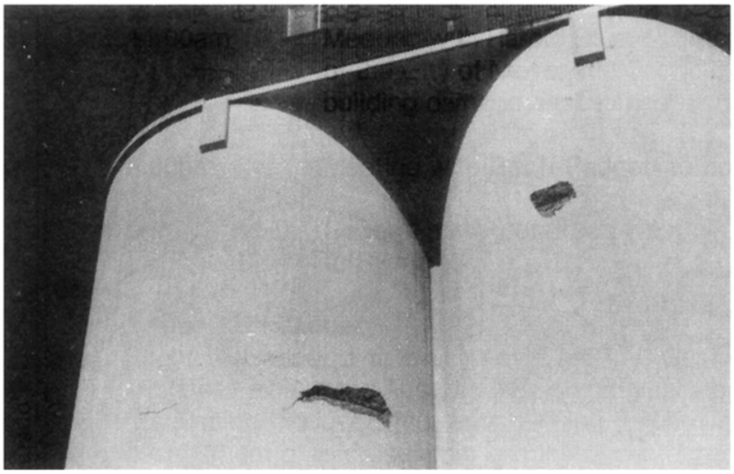


PLATE 5.70 *Typical single storey timber framed weatherboarded dwellings, which suffered only minor damage in the earthquake*

PLATE 5.71 *Sliding movement between timber structure and its support foundations (location 5)*

PLATES 5.72 and 5.73 *General views of the industrial sector of the City of Newcastle, concentrated close to the Hunter River and Harbour (taken from location 32 looking north west and location 15 looking south)*





PLATES 5.74 and 5.75 *Minor spalling of concrete in storage silos was the only evident form of damage to industrial facilities in Newcastle (location 16)*

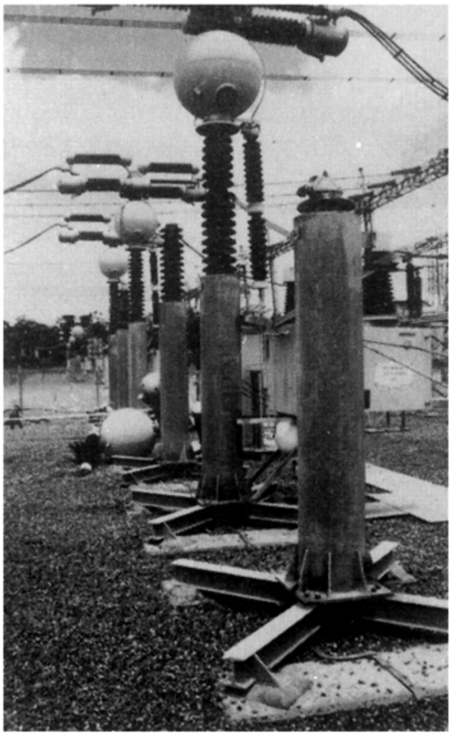


PLATE 5.76 *Failure of porcelain insulators supporting 132kV circuit breakers at Killingworth, 20km west of Newcastle*

PLATE 5.77 *General view of the Stockton Bridge, north of Newcastle which showed slight settlement of the western abutment due to the earthquake (location 1)*

PLATE 5.78 *Evidence of localised ground compression causing bulging of brick paving (location 14)*

PLATE 7.1 *The George Hotel in the Newcastle central business district, a five-storey unreinforced brick masonry building which had to be demolished nine days after the earthquake (location 34)*

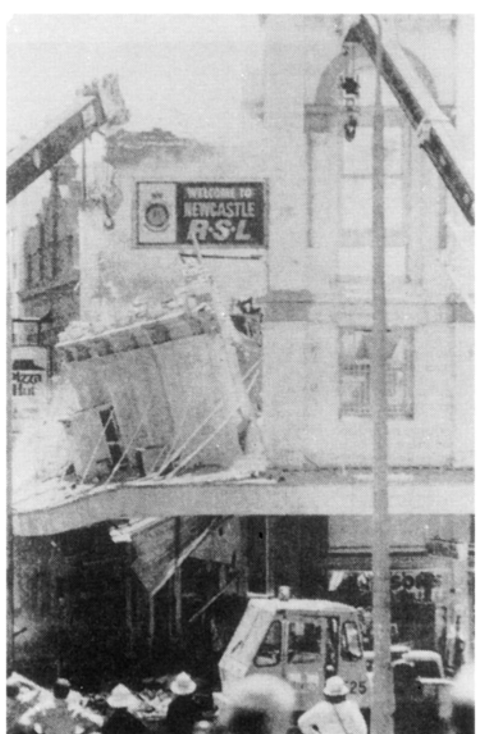



PLATE 7.2 *Demolition of the three-storey Newcastle RSL club in the central business district (location 30)*

## APPENDIX A
## Selected details from the Australian Earthquake Code (SAA 1979)

## A.1 Background and Exclusions

The Australian Earthquake Code (SAA 1979) is based largely on the United States of America provisions of ATC-3 (1978) and UBC (1976). It is an empirically based design code, which sets minimum standards with regard to public safety, to safeguard against major structural failure and loss of life. The main aim is to prevent structural collapse, rather than the prevention of structural, or more particularly non-structural damage.

The code specifically excludes the following types of structure:

a) Special structures (nuclear power stations, hazardous chemical facilities, etc.)

b) Structures with unusual characteristics, where a dynamic analysis is required

c) Bridges and Dams

d) Small domestic structures, especially in single-storey, isolated units.

The reason for exclusion (d) above is that the traditional Australian dwelling construction with a well braced timber frame and metal roof has a high inherent earthquake resistance. The code is however recommended for use in the design of multi-storey single dwellings or multiple side-by-side dwellings.

## A.2 Seismic Zone Map

The seismic zone map shown in Figure A.1 was compiled from the Bureau of Mineral Resources earthquake data file. This data file contains the locations and magnitudes of all known Australian earthquakes occurring between 1897 and 1976, and is reasonably complete from 1969 onwards for magnitudes $M_L \geq 4$. Prior to 1969, the data is complete only for $M_L \geq 5$. According to the seismic zone map, Newcastle is rated in zone zero and therefore all construction is excluded from the provisions of the code, except for long-period structures which might be affected by strong far-field earthquakes. In Zone A, ductile construction requires no lateral force analysis, but for non-ductile buildings such as unreinforced masonry or brittle precast concrete panel construction the code requires design to ensure elastic behaviour, with a behaviour factor K (see Section A.3 below) equal to 3.2. There is no height limit imposed on structures in Zone A. In Zones 1 and 2, both ductile and non-ductile construction must be designed for earthquake forces. For example, for ductile structures the behaviour factor K in applying the lateral force provisions (Section A.3) is taken to be 0.67 for ductile moment-resisting space frames, 0.80 for ductile moment-resisting space frames with bracing or shear walls and 1.33 for boxed shear wall systems. For non-ductile structures, K=3.2 as in Zone A and there is also a height limit of 50m imposed on such structures.

## A.3 Minimum Earthquake Forces

The minimum lateral force H for design of earthquake-resistant structures is given as:

$$H = ZIKCSW \qquad \text{(A.1)}$$

where

W is the building's dead weight plus a proportion of vertical live loading,

S is the site factor taken as 1.5 unless calculated by properly substantiated analysis based on site investigation data.

C is the seismic response factor
= $T^{-1/2}/15$ but $\leq 0.12$ and $CS \leq 0.14$

and T is the fundamental building period found by analysis or by empirical methods

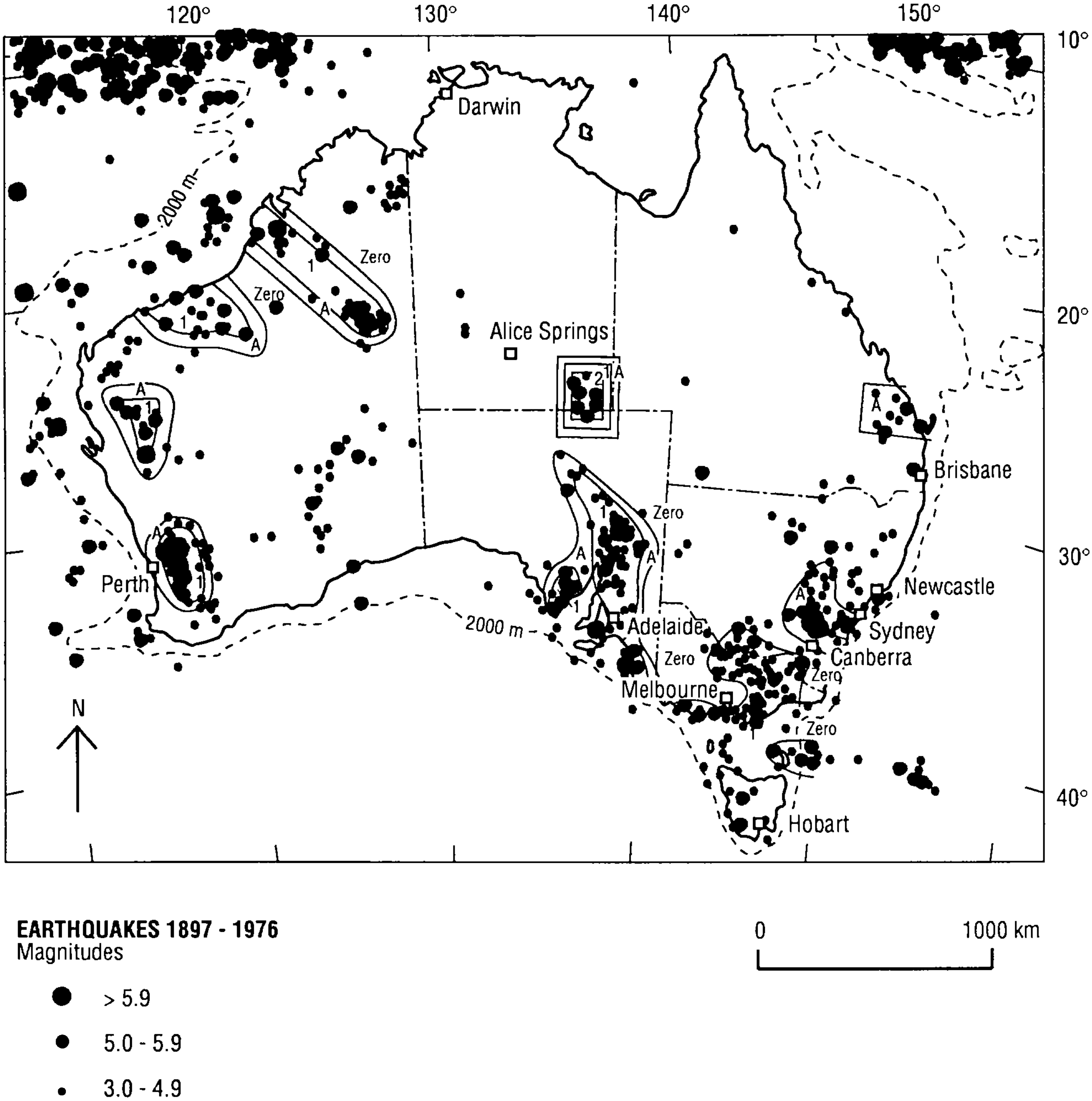
K is the ductility factor described in Section A.2 above.

I is the importance factor
= 1.0 or 1.2 for normal buildings and essential facilities, respectively.

Z is the zone factor
= 0.0 for Zone Zero
= 0.09 for Zone A (non-ductile construction)
= 0.18 for Zone 1 for all types of construction
= 0.36 for Zone 2 for all types of construction

A minimum force level H of 0.02 W is required if Z is greater than 0.0.

The above requirements are is similar to that of UBC (1976). The lateral force H is to be applied non-concurrently in the building's principal directions, and non-simultaneously with the wind loading. In Zone 2 the maximum lateral force coefficient H/W is taken to be 0.034 for normal structures with a ductile moment-resisting space frame and 0.041 for equivalent essential facilities, which is roughly equivalent to UBC Zone 2 (Moderate Seismicity), with a 500 year return period peak ground acceleration of about 0.2g.




**Seismic zoning map in Australian Earthquake Code (SAA 1979)**

**Figure A.1**

## APPENDIX B
## Recordings of the Newcastle Earthquake in Britain - A Determination of Depth

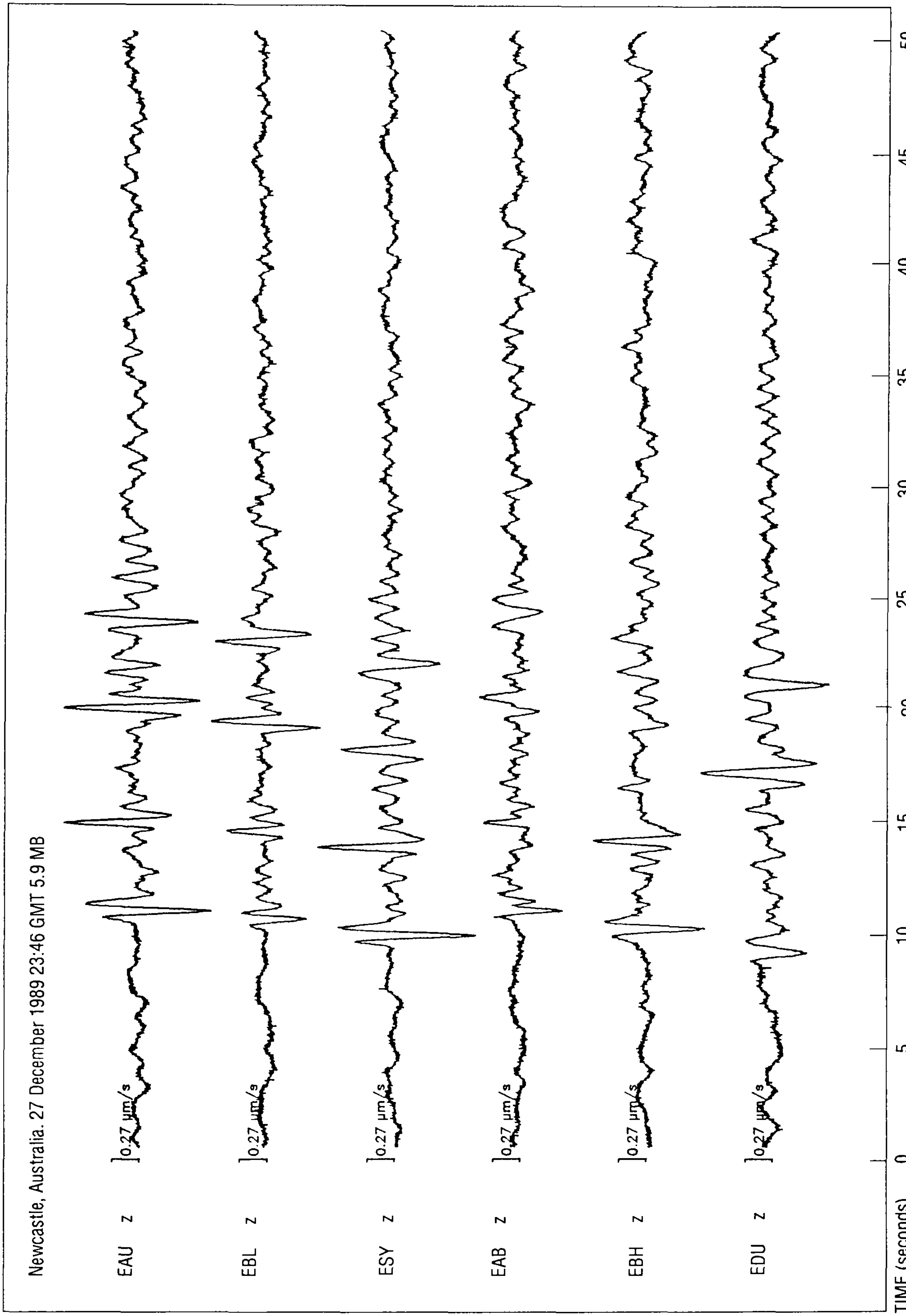
by
R.D. Adams of the International Seismological Centre

The British Geological Survey operates a network of short-period seismographs, the Lownet Array, around Edinburgh. These six stations gave very clear recordings of seismic phases that have traversed the Earth's interior from the earthquake that occurred near Newcastle, Australia, on 28 December 1989. Fig. B.1 shows the traces at the six stations, each identified by a three-letter code. The closest station, with the earliest arrival, is EDU (Dundee) at a distance of 150.68° from the adopted epicentre, and the furthest is EAB (Aberfoyle) at 151.47°.

Seismic waves at this distance, called PKP phases, traverse the Earth's core by three distinct paths which, in order of arrival and decreasing depth of penetration into the core are termed the DF, BC and AB branches. The theoretical travel times of these waves are shown in Fig. B.2, with their first derivatives, which give the inverse velocity or "slowness" of the phase. The power of a close network of stations such as Lownet is that it is possible to identify branches of phases by slowness, as well as by absolute travel time.

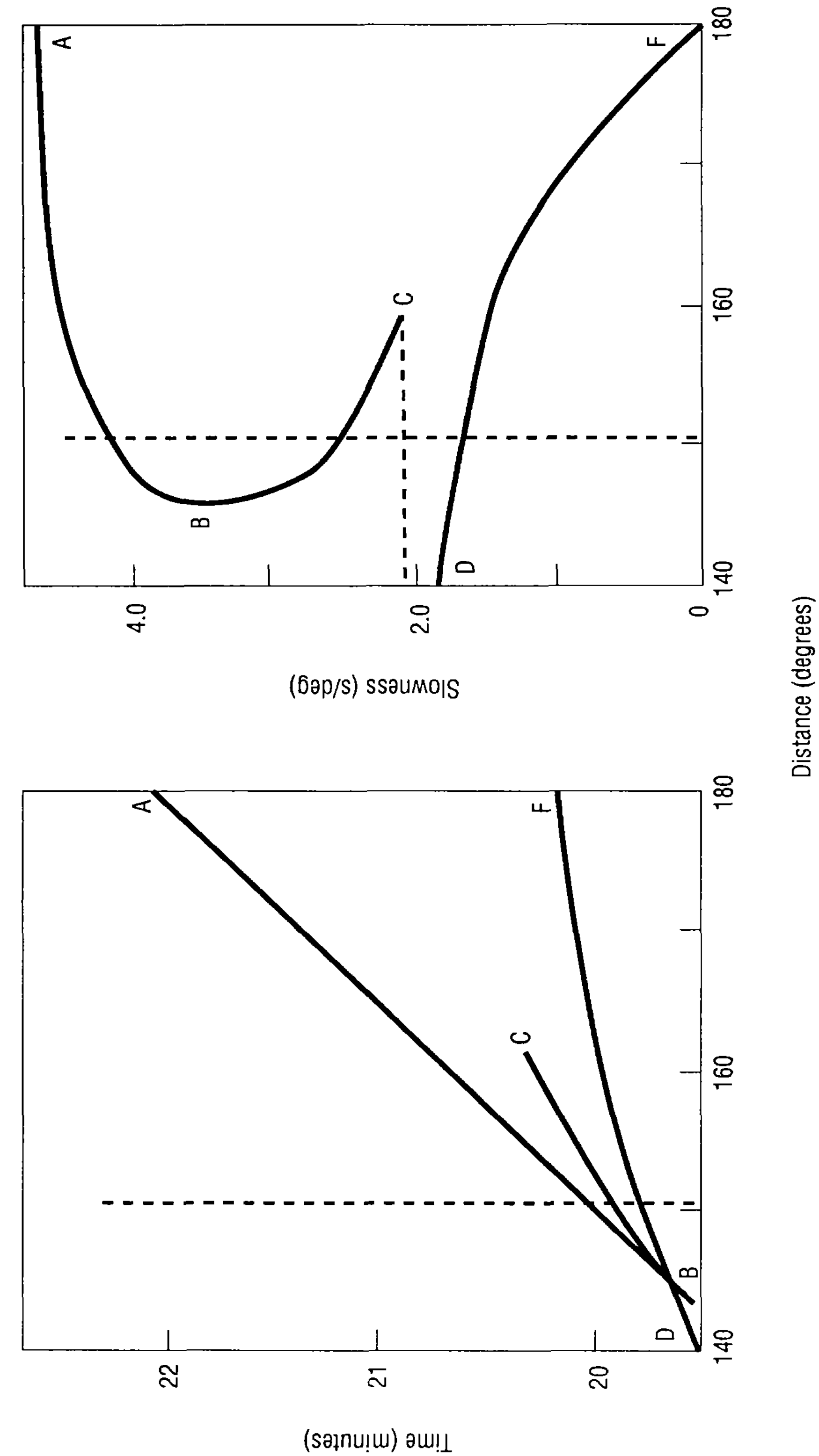
The records show four distinct pulses at each station. The first recorded phase has a time difference of 2.1 seconds between the closest and furthest stations, which over a distance of 0.78° gives a slowness of 2.5 seconds/degree. This phase is thus clearly identified as the BC branch of PKP, independently of its arrival time which also agrees within 2s with that for BC. Similarly, the third recorded phases has a slowness of 4.3s/degree showing it to be the AB branch. It is interesting to seismologists to note the complete absence in this instance of the theoretical first arrival of the DF branch, which penetrates the inner core of the Earth, and would be expected about 6s before the BC branch.

Of particular interest both to seismologists and engineers are the second and fourth pulses, which at each station follow the preceding phases by 4 seconds, and are of opposite polarity. This indicates that they are surface reflections from near the epicentre, and establishes the time taken for seismic waves to travel from the focus to the surface as 2 seconds. This gives a good constraint on the depth of focus, which for the usual value of velocity in the upper crust must be close to 10km.




**Recordings of the Newcastle earthquake made at stations of the BGS Lownet array**

Figure **B.1**




**Theoretical travel times of PKP phases and derived value of slowness (dT/dΔ)**

**Figure B.2**

## APPENDIX C
## Damage Survey Maps Produced by Newcastle City Council

The following maps were produced by the Department of Engineering, Newcastle City Council from its own and other records. The following coding is used.

| | | | |
|---|---|---|---|
| Construction Material | Brick | Wood | |
| Building Use | Commercial | Industrial | Residential |
| Damage Intensity | Modified Mercalli Scale Value defined on the following page. | | |

## THE MODIFIED MERCALLI SCALE

MM 1 Not felt by humans, except in especially favourable circumstances, but birds and animals may be disturbed.
Reported mainly from the upper floors of buildings more than ten storeys high.
Dizziness or nausea may be experienced.
Branches of trees, chandeliers, doors and other suspended systems of long natural period may be seen to move slowly.
Water in ponds, lakes, reservoirs, etc. may be set into seiche oscillation.

MM 2 Felt by a few persons at rest indoors, especially by those on upper floors or otherwise favourably placed.
The long period effects listed under MM 1 may be more noticeable.

MM 3 Felt indoors, but not identified as an earthquake by everyone.
Vibration may be likened to the passing light traffic.
It may be possible to estimate the duration, but not the direction.
Hanging objects may swing slightly.
Standing motorcars may rock slightly.

MM 4 Generally noticed indoors, but not outside.
Very light sleepers may be wakened.
Vibration may be likened to the passing of heavy traffic, or to the jolt of a heavy object falling or striking the building.
Walls and frame of buildings are heard to creak.
Doors and windows rattle.
Glassware and crockery rattles.
Liquids in open vessels may be slightly disturbed.
Standing motorcars may rock, and the shock can be felt by their occupants.

MM 5 Generally felt outside, and by almost everyone indoors.
Most sleepers awakened.
A few people frightened.
Direction of motion can be estimated.
Small unstable objects are displaced or upset.
Some glassware and crockery may be broken.
Some windows cracked.
A few earthenware toilet fixtures cracked.
Hanging pictures move.
Doors and shutters swing.
Pendulum clocks stop, start, or change rate.

MM 6 Felt by all.
People and animals alarmed.
Many run outside.
Difficulty experienced in walking steadily.
Slight damage to Masonry D.
Some plaster cracks or falls
Isolated cases of chimney damage
Windows, glassware, and crockery broken
Objects fall from shelves, and pictures from walls
Heavy furniture moved. Unstable furniture overturned.
Small church and school bells ring.
Trees and bushes shake, or are heard to rustle.
Loose material may be dislodged from existing slips, talus slopes, or shingle slides.

MM 7 General alarm.
Difficulty experienced in standing.
Noticed by drivers of motorcars.
Trees and bushes strongly shaken
Large bells ring.
Masonry D cracked and damaged.
A few instances of damage to Masonry C.
Loose brickwork and tiles dislodged.
Unbraced parapets and architectural ornaments may fall.
Stone brick veneers damaged.
Decayed wooden piles broken.
Frame houses not secured to the foundation may move.
Cracks appear on steep slopes and in wet ground.
Landslips in roadside cuttings and unsupported excavations.
Some tree branches may be broken off.
Changes in the flow or temperature of springs and wells may occur.
Small earthquake fountains.

MM 9 General panic.
Masonry D destroyed.
Masonry C heavily damaged, sometimes collapsing completely.
Masonry B seriously damaged.
Frame structures racked and distorted.
Damage to foundations general.
Frame Houses not secured to the foundations shifted off.
Brick veneers fall and expose frames.
Cracking of the ground conspicuous.
Minor damage to paths and roadways.
Sand and mud ejected in alluviated areas, with the formation of earthquake foundations and sand craters.
Underground pipes broken.
Serious damage to reservoirs.

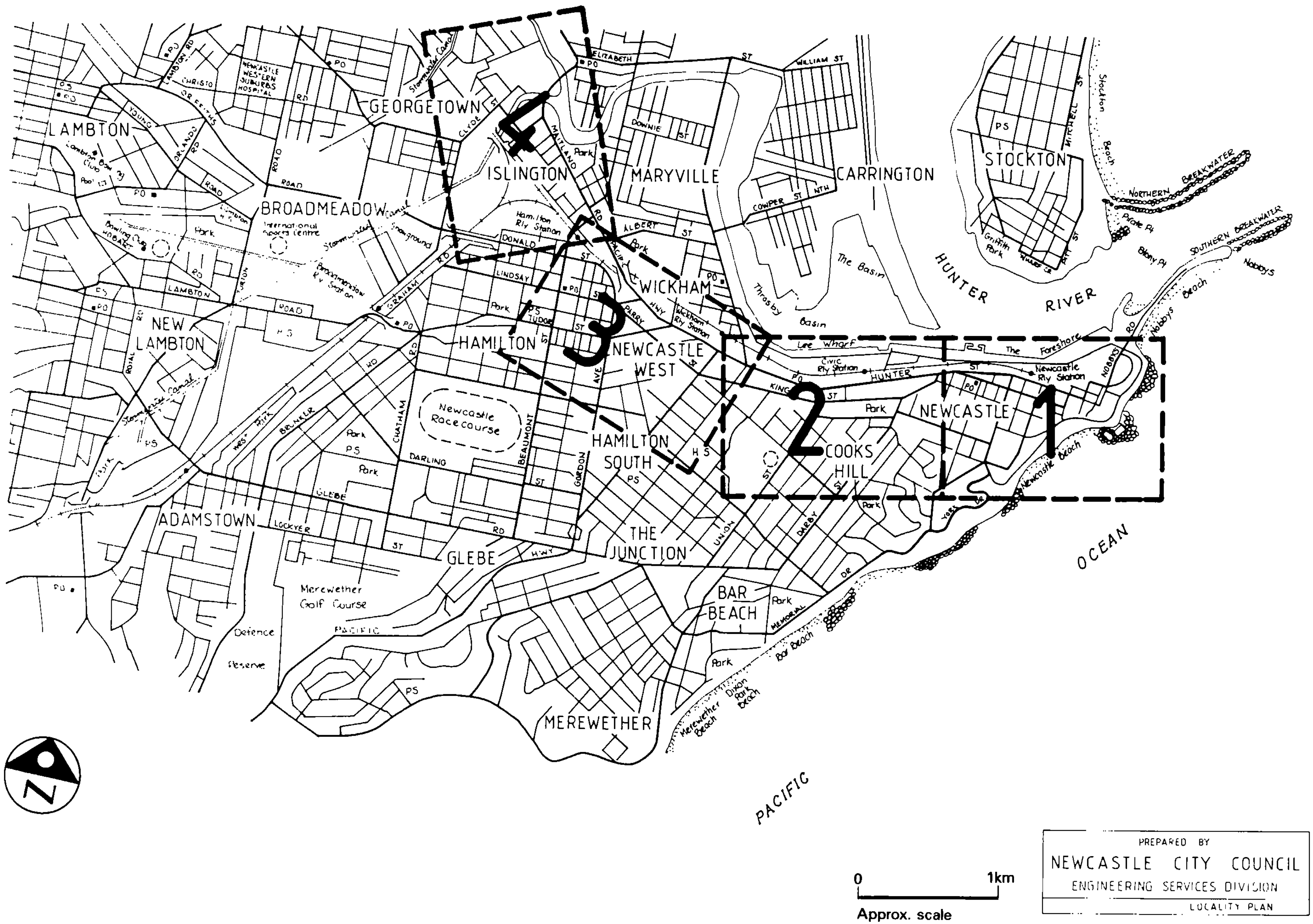
MM 10 Most masonry structures destroyed, together with their foundations.
Some well built wooden buildings and bridges seriously damaged.
Dams, dykes and embankments seriously damaged.
Railway lines slightly bent.
Cement and asphalt roads and pavements badly cracked or thrown into waves.
Large landslides on river banks and steep coasts.
Sand and musd on beaches and flat land moved horizontally.
Large and spectacular sand and mud fountains.
Water from rivers, lakes, and canals thrown up on the banks.

MM 11 Wooden frame structures destroyed.
Great damage to railway lines.
Great damage to underground pipes.

MM 12 Damage virtually total. Practically all works of construction destroyed or greatly damaged.
Large rock masses displaced.
Lines of slight and level distorted.
Visible wave-motion of the ground surface reported.
Objects thrown upwards into the air.

**Categories of non-wooden construction**

| | |
|---|---|
| Masonry A | Structures designed to resist lateral forces of about 0.1g. Typical buildings of this kind are well reinforced by means of steel or ferro-concrete bands, or are wholly of ferro-concrete construction. All mortar is of good quality and the design and workmanship is good. Few buildings erected prior to 1935 can be regarded as in category A. |
| Masonry B | Reinforced buildings of good workmanship and with sound mortar, but not designed in detail to resist lateral forces. |
| Masonry C | Buildings of ordinary workmanship, with mortar of average quality. No extreme weakness, such as inadequate bonding of the corners, but neither designed nor reinforced to resist lateral forces. |
| Masonry D | Building with low standards of workmanship, poor mortar or constructed of weak materials like mud brick and rammed earth. Weak horizontally. |
| Windows | Window breakage depends greatly upon the nature of the frame and its orientation with respect to the earthquake source. Windows cracked at MM 5 are usually either large display windows, or windows tightly fitted to metal frames. |
| Chimneys | The "weak chimneys" listed under MM 7 are unreinforced domestic chimneys of brick, concrete block, or poured concrete. |
| Water Tanks | The "domestic water tanks" listed under MM 7 are of the cylindrical corrugated-iron type. If these are only partly full, movement of the water may burst soldered and riveted seams. |



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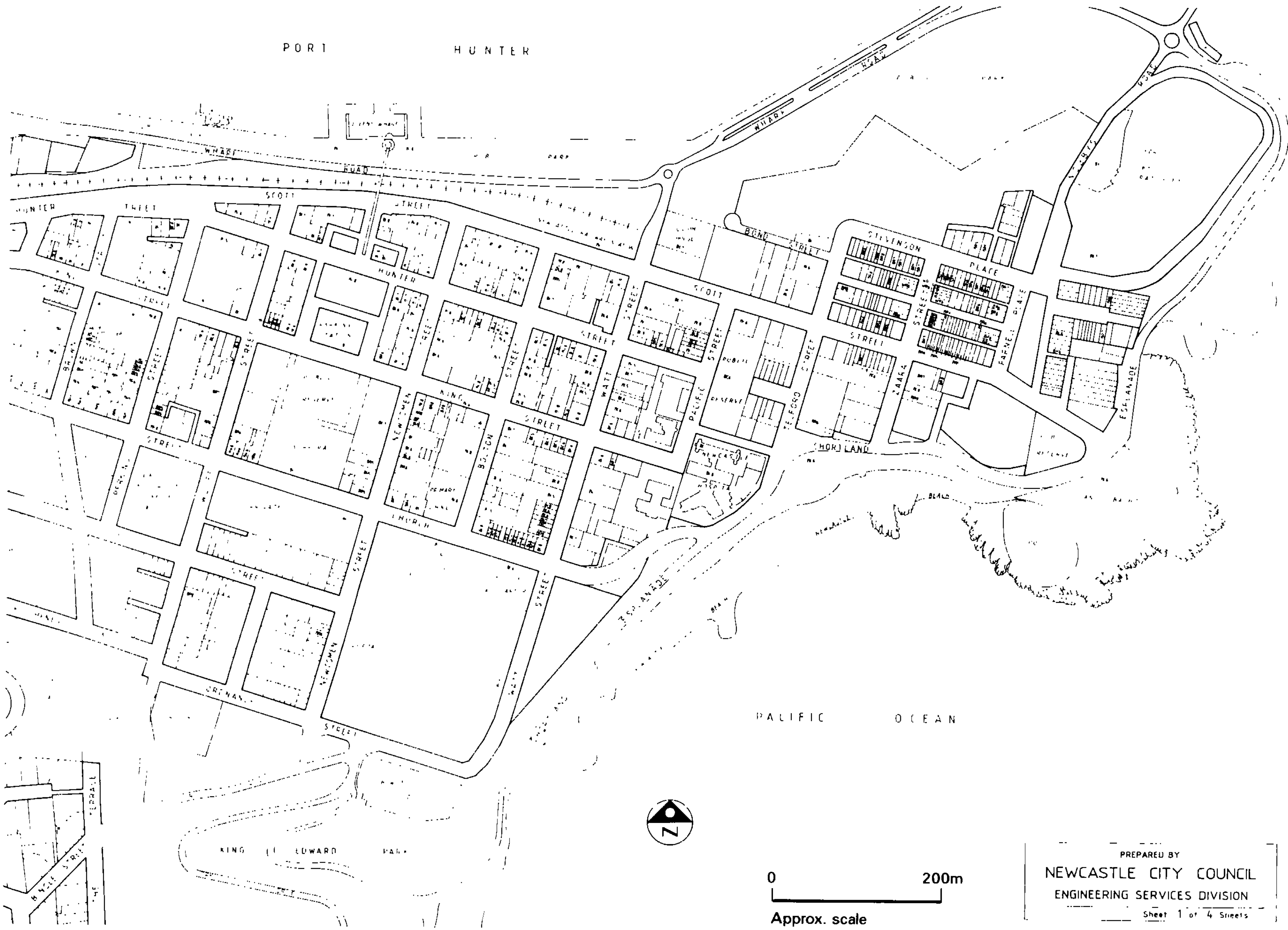
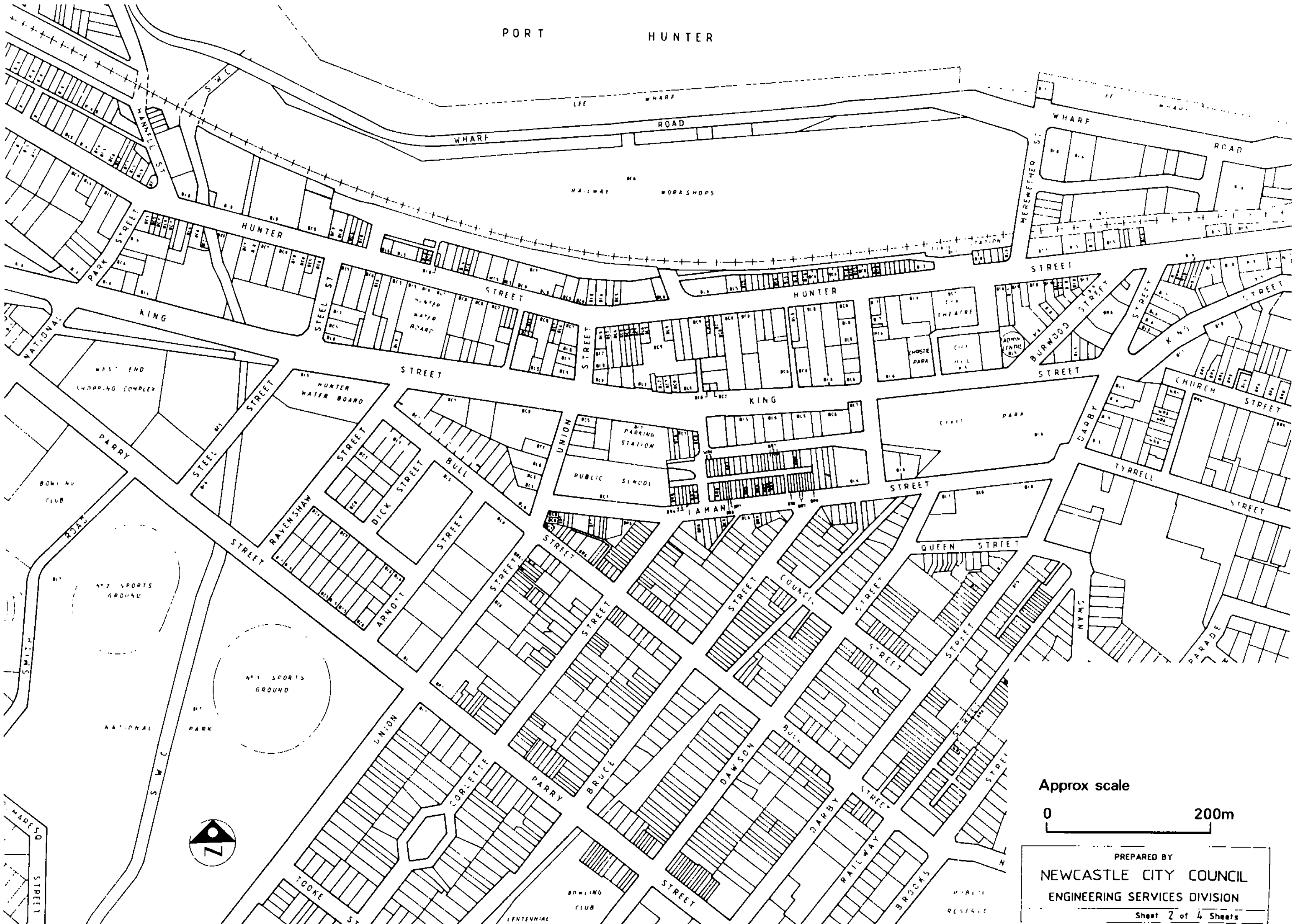
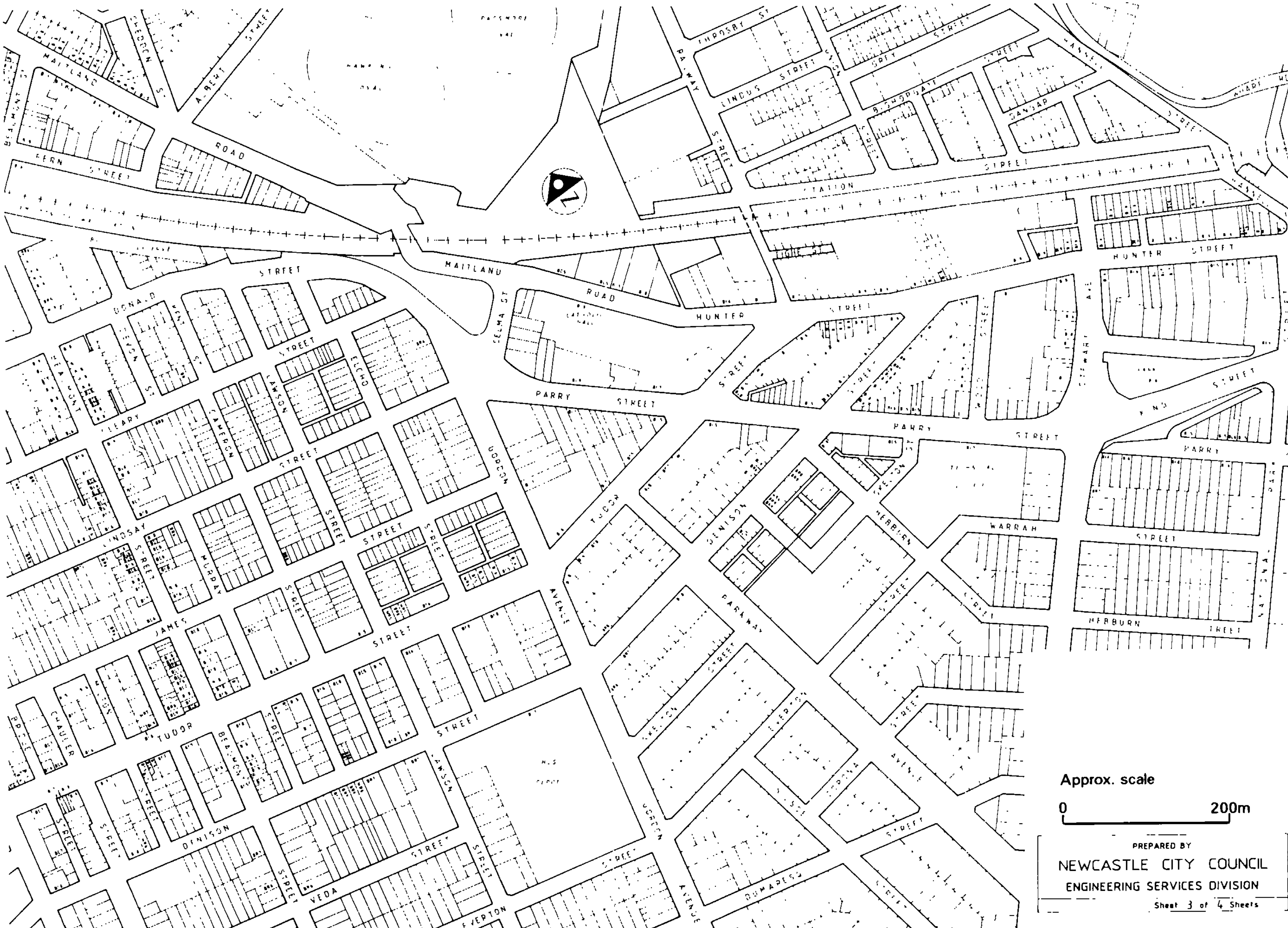
ENGINEERING SERVICES DIVISION
LOCALITY PLAN


PORT HUNTER
WHARF ROAD
SCOTT STREET
HUNTER STREET
KING STREET
CHURCH STREET
BOND STREET
STEVENSON PLACE
SHORTLAND
ESPLANADE
PACIFIC STREET
WATT STREET
BOLTON STREET
NEWCOMEN STREET
TELFORD STREET
PUBLIC RESERVE
PACIFIC OCEAN
KING EDWARD PARK
0
200m
Approx. scale
PREPARED BY
NEWCASTLE CITY COUNCIL
ENGINEERING SERVICES DIVISION
Sheet 1 of 4 Sheets


PORT HUNTER
WHARF ROAD
WHARF ROAD
RAILWAY WORKSHOPS
HUNTER STREET
KING STREET
PARK STREET
NATIONAL
STEEL ST
WEST END SHOPPING COMPLEX
HUNTER WATER BOARD
PARRY STREET
STEEL STREET
UNION STREET
PARKING STATION
PUBLIC SCHOOL
LAMAN
BULL
RAVENSHAW STREET
DICK STREET
ARNOTT
CIVIC PARK
CIVIC THEATRE
MEREWETHER ST
BURWOOD STREET
CHURCH STREET
TYRRELL STREET
DARBY
QUEEN STREET
COUNCIL STREET
SWAN
PARADE
BRUCE STREET
DAWSON
CORLETTE
TOOKE ST
BOWLING CLUB
CENTENNIAL PARK
No 1 SPORTS GROUND
No 2 SPORTS GROUND
NATIONAL PARK
BOWLING CLUB
SMITH
RAILWAY
BROCKS
S W C
Approx scale
0
200m
PREPARED BY
NEWCASTLE CITY COUNCIL
ENGINEERING SERVICES DIVISION
Sheet 2 of 4 Sheets


MAITLAND ROAD
FERN STREET
HUNTER STREET
PARRY STREET
STATION STREET
RAILWAY STREET
LINDUS STREET
THROSBY ST
CHEERO ST
SELMA ST
DONALD STREET
CAMERON STREET
LAWSON STREET
ECHO STREET
LINDSAY STREET
MURRAY STREET
JAMES STREET
TUDOR STREET
DENISON STREET
VEDA STREET
EVERTON STREET
GORDON AVENUE
PARKWAY AVENUE
WARRAH STREET
HERBURN STREET
KING STREET
DUMARESQ STREET
Approx. scale
0
200m
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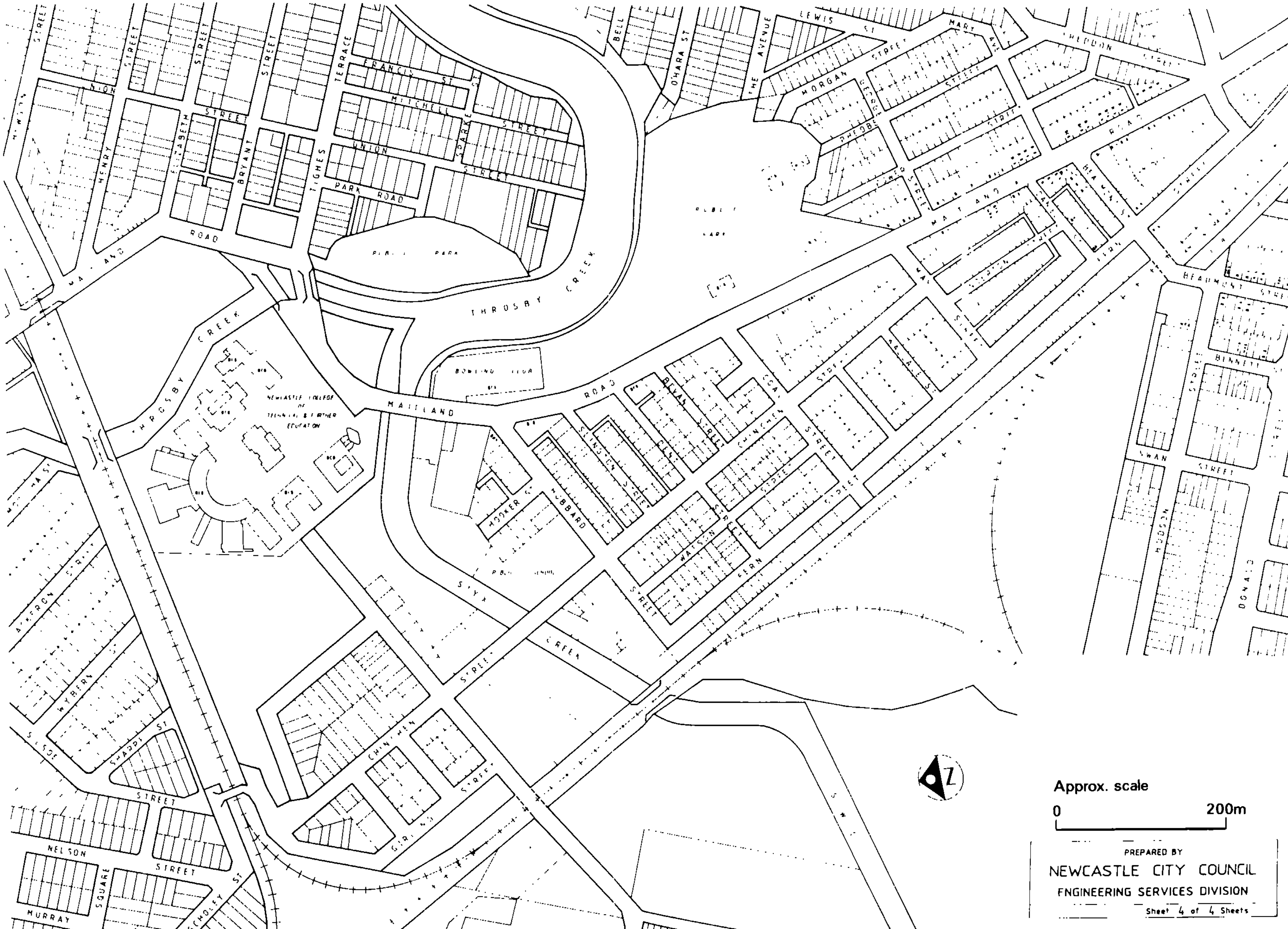
Sheet 3 of 4 Sheets



MAITLAND ROAD
THROSBY CREEK
NEWCASTLE COLLEGE OF TECHNICAL & FURTHER EDUCATION
PUBLIC PARK
BOWLING CLUB
STYX CREEK
FRANCIS ST
MITCHELL STREET
UNION STREET
PARK ROAD
TIGHES TERRACE
BRYANT STREET
ELIZABETH STREET
HENRY STREET
CLARKE ST
LEWIS ST
MARY ST
BELL ST
O'HARA ST
THE AVENUE
MORGAN STREET
GEORGE STREET
BEAUMONT STREET
BENNETT ST
SWAN STREET
HUDSON STREET
DONALD
HUBBARD ST
HOOKER ST
BRYANT STREET
CHINCHEN STREET
WATSON STREET
FERN STREET
CHINCHEN STREET
GIRLING STREET
WYBERN ST
SHARPE ST
NELSON STREET
MURRAY
SQUARE
SCHOLEY ST
Approx. scale
0
200m
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ENGINEERING SERVICES DIVISION
Sheet 4 of 4 Sheets

**APPENDIX D**
**Guidelines for Repair and Reconstruction of Buildings in Newcastle**



This appendix consists of extracts from the report by Newcastle City Council (1990)

## D.1 Design Considerations

Some of the factors that should be considered by designers in building earthquake resistance into structures are as follows:

1. **Configuration of Building.** Symmetry in plan and elevation is desirable. Compact plan shapes are more desirable than extended wings. If planning requirements dictate undesirable shapes, a detailed knowledge of earthquake engineering becomes important.

2. **Configuration of Structure.** Symmetry of lateral load resisting elements is desirable. Dynamic torsional rotations will be greater than those suggested by a simple static assessment of eccentricities.

3. **Materials.** Adequately detailed reinforced concrete will have a ductility or capacity to yield but still carry load that unreinforced masonry does not have. The use of reinforced concrete, or structural steel or reinforced masonry for the main structural elements is therefore desirable. This is not to preclude the use of unreinforced masonry, but its use requires a more careful assessment by an engineer experienced in earthquake design.

4. **Roofs and Floors.** Concrete floors in load-bearing masonry structures will be integrated with the walls as a matter of course. In all other walls, it is important to ensure adequate vertical support for the floors and roofs, and to ensure the whole building acts integrally and walls do not fall away.

5. **Projecting Parts.** Overhanging parts such as projecting cornices, and parapets and chimneys are the first to fall during an earthquake. Not only is there damage to the building when such parts fall, but they may injure people. They should be avoided as far as possible or care taken to reinforce them and anchor them to the main structure.

## D.2 Structural Considerations

1. **Unreinforced Masonry Structures.** For unreinforced masonry structures the following is recommended:

   a) A lateral load resisting system must be clearly identified, for forces along each of the two principal axes. Non-load bearing masonry elements must be clearly identified separately from load bearing elements; if they are connected to the structure such that shear loads will be induced in them they become load bearing.

   b) Every load bearing shear wall shall be capable of resisting the horizontal forces induced in it by its own mass and those transmitted to it, and based on a distribution of forces calculated from relative stiffnesses of all elements.

   c) Shear walls must exist along both principal axes.

   d) Floor and roof elements must be connected to walls to ensure that the structure has a three-dimensional integrity.

   e) Floor and roof systems shall have horizontal bending and shear capacity sufficient to transmit inertial (earthquake induced) forces to shear walls.

   f) Non-structural elements must be effectively stabilised. In particular, free-standing walls (e.g. partitions and parapets not connected to the structure at their tops) shall be designed as vertical cantilevers for inertia face-loads unless it can be reliably demonstrated that they have sufficient edge support and horizontal bending capacity.

2. **Reinforced Masonry and Concrete, Timber and Steel Frame Structures.** For frame structures the following is recommended:

a) The lateral load resisting system and non-load bearing elements should be clearly identified as for unreinforced masonry structures.

b) For framed structures in reinforced masonry and concrete, members are to be designed and detailed as ductile or non-ductile, with appropriate earthquake design force coefficients.

c) Steel framed structures are classified as either non-ductile or ductile pure or braced frames. The appropriate earthquake design force coefficient must be chosen and the frame detailed accordingly. As steel structures are generally the most flexible, drift limits rather than strength often govern the design of members, but the strength of joints is important to ensure that ductility is available in accordance with the load assumptions.

d) Buildings with timber frames are generally designed as braced frames or as shear wall structures. Usually timber will be designed as non-ductile except where ductile joints, such as nail plates with less strength than timber members, are specially designed for non-linear cyclic behaviour.

e) Non-structural elements (e.g. infill panels, partitions, precast cladding panels) shall be separated from the structure with clearances greater than the horizontal deflection calculated for the structure as if it were non-ductile.

f) The stability of the non-structural elements shall be as for unreinforced masonry buildings. The design of connections shall provide adequate provision for movements of the main structure.

3. **Shear Wall Structures.** Shear walls are the main members transferring all the earthquake induced loads to the foundations and because of their stiffness generally offer more protection to non-structural elements than pure frame buildings. Adequately reinforced concrete and masonry walls, timber plywood clad stud walls and carefully designed structural steel walls can possess ductility. The recommendations in paragraph 1 above apply equally to shear walls of all materials. Ductile shear walls should in addition be designed so that they will always fail in flexure in preference to either overturning or shear failure.

## D.3 Upgrading of Existing Buildings

1. **Earthquake Resistance.** Existing buildings which are subject to repair or restoration will require strengthening in accordance with the codes, as specified in the interim requirements.

   It is recognised that the economic and practical feasibility to comply fully will in certain circumstances be difficult. However, any lessening of earthquake resistant requirements will only be considered where there is no risk to life or major structural failure.

2. **Fire Safety Requirements.** Fire safety upgrading will be considered at the building application stage and though Council may not insist on immediate upgrading during the earthquake recovery period it will advise the applicant/owner of works considered necessary in accordance with Ordinance 70 and Council's Fire Safety Upgrading Programme. This will allow owners to plan and budget for such fire safety measures.

3. **Fire Resistance Ratings.** The use of adhesives e.g. epoxy based adhesives and non fire-protected structural steel must be examined in earthquake restoration.

Council must consider these materials and be satisfied that in use they:-

a) will not unduly reduce the existing level of fire protection afforded to persons accommodated in or resorting to the building;

b) will not unduly reduce the existing level of resistance to fire of the building structure; and

c) will not unduly reduce the existing safeguards against spread of fire to adjoining buildings.

Subsequently the use of these materials may be prohibited in certain parts and locations of buildings.

## D.4 General Requirements

1. **Wall Ties.** The minimum spacing of ties shall be strictly as required by Table 3.2 of Australian Standard 1640-1974. The material of cavity ties and masonry veneer ties shall be corrosion resistant and selected having regard for the prevailing environmental conditions and the design life of the building.

   It is noted that the failure of wall ties was a major contributing factor to building failure during the earthquake: the more wall ties the stronger the wall.

2. **Mortars.** The composition of mortar required by Australian Standards consist of portland cement, lime and fine aggregate. In this region the practice of using fire clay and ungraded dune sand is contrary to the Australian Standard. Current engineering research is proving the poor quality of mortar. The availability and use of the correct mortar ingredients is a most important issue and needs to be addressed by the building industry.

3. **Damp Proof Courses - Flashing.** The loss of brickwork bond by insertion of flashings and damp proof courses in bed joints must be considered and only used when absolutely necessary, and such locations and use must be considered and detailed at the design stage.

   Well designed flashings may be built partly into a bed joint and still provide the required waterproofing to the building. Consideration should be given to the use of mortar type damp proof courses (water proof mortar) to retain full brickwork bond.

## D.5 Building Approvals

A building permit is required for all new buildings, and most repairs or re-building works over the value of A$1,000.

Approval is not required for cosmetic works such as minor crack filling, plastering, brick pointing and painting where there are no structural repairs required.

Prior to the issue of a building permit inspections to determine the required performance standards of the repairs/rebuilding will be made. It may not be possible to make all works strictly comply with the provisions of Ordinance 70, e.g. existing footings may be sufficient when a wall is being partly rebuilt. Each building must be considered in the individual circumstances of the case.

Council will carry out normal progress inspections and will specifically nominate these on the permit e.g. footings, frame, final.

1. **Plans and Specifications.** Detailed plans and specification may not be necessary for minor works, however, Council will require as a minimum a schedule of works and may require a report from a Professional Engineer or from a Licensed Builder.

   Note that the object in requiring a building application is to ensure that mandatory building regulations are observed for the protection of life, property and the welfare of the people of Newcastle both in the earthquake recovery period and for the design life of the building.

2. **Engineering Reports - Certification.** When Council cannot determine structural design compliance, under deemed to comply standards, a report by a practising structural engineer will be required by Council. Such reports will certify design structural integrity on completion of works etc and include the basis on which the design and conclusions are made and the extent to which the Engineer has relied on relevant specifications, rules, codes of practise of publications in respect of the construction including seismic load considerations.

   Designs should include the necessary detailing of connections and anchorages of composite building materials and members e.g. shear walls and roof-floor diaphragms connections including all necessary bracing and tie downs. Such detailing will ensure that building trades and supervisors can properly plan and complete the works.

   Site geology must be considered in the recognition that alluvial soft soils (dependent on depth and type) can have a liquefaction effect and accentuate seismic forces.

   Buildings should be symmetrical of plan whenever possible and particularly at ground and lower storey levels of multi-level buildings. They must have sufficient shear walls to resist lateral loads, this may mean the reduction of large shopfront windows and large open floor plans. The alternative is an engineering design that takes cognisance of the "soft storey considerations" in earthquake-resistant design.

3. **Unauthorised building work.** Building work carried out without approval and/or required Council inspections cannot be accepted as complying with appropriate building regulations. Council may require part or complete rebuilding and would have to defer and/or refuse the issue of approval Certificates when compliance with building regulations are in doubt. The issue of Statutory Notices and legal action may be a consequence of unauthorised works.