Bridges & Earthquakes

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Introduction

Lifelines in general are excluded from the provisions of AS1170.4 and the question must then be asked,

'Do we have no guidance for designing lifelines?'

Insofar as bridges are concerned some guidance has been provided in Australian bridge design codes since at least 1965. The 1965 Edition of the NAASRA Highway Bridge Design Specification provided a formula to calculate earthquake lateral force EQ as

$$EQ = C \times D$$

where D = dead load of structure
C = coefficient depending on founding conditions

It was noted that the value of 'C' may be increased by the engineer at his discretion in the light of local seismic records. It was also noted that all details should be designed to prevent displacement due to earthquake with special attention being given to bearings. In this context 'displacement', is equivalent to 'falling off'.

Unfortunately the section on earthquake forces was introduced with the words 'In regions where earthquakes of significant intensity may occur, provision shall be made to accommodate lateral forces from these earthquakes...'. It was commonly thought that earthquakes of significant intensity did not occur in Australia so the section was not applied in design.

The 1970 edition of the Bridge Design Specification had essentially the same wording as the 1965 edition.

The 1976 edition modified the lateral force expression to

$$EQ = KCD \text{ (min } 0.02 \text{ D)}$$

The coefficient 'C' was now a function of the structure stiffness and 'K' represented the ability of the structure to absorb energy. An increase of 50% in force was applied to structures founded in weak deep soil.

Reference was made to seismic zoning studies and the work of the National Committee on Earthquake Engineering to determine whether earthquake design forces were to be applied. Aspects requiring particular attention were stated more explicitly than in the earlier editions.

The 1992 edition of the Austroads Bridge Code makes explicit reference to AS2121 for zoning and uses the same expression as AS2121 to calculate horizontal earthquake force, with the zone factor modified for a 2000 year average recurrence interval.

It is seen then that guidance has existed for earthquake design of bridges in Australia for many years but that even the most recent Bridge Design Code (1992) is already out of date. The balance of this paper discusses aspects of bridge design that need special consideration based on performance in previous earthquakes and outlines an approach to calculating earthquake forces in tune with AS1170.4.

Bridge damage

In a simplistic sense it may be stated that most earthquake damage to bridges is related to inadequate detailing. While design for earthquake forces cannot be ignored it is the author's contention that careful, thoughtful detailing is more productive than careless, thoughtless design for a force.

A relatively common form of damage is where the superstructure falls off the supporting structure. This was evident in Niigata in 1964 with the Showa bridge, Madang in 1970 where several bridges collapsed on their roller bearings, San Fernando in 1971 and Northridge in 1994.

An original concern with bridges was to cater for movements due to temperature changes and details provided for this did not always cater for relative movements induced during earthquakes.

The Austroads Code covers this in Section 2.13.5.1 with words such as `restraining devices shall be provided with the specific aim of preventing dislodgment of the superstructure from the support structure'.

Failure of columns in compression where the longitudinal bars are not adequately confined (as is common in Australian construction) is another common failure cause evident at San Fernando, Northridge and other earthquakes. The Austroads Code addresses this aspect in Section 2.13.5.3 as follows, `special attention should be given to the detailing of concrete members bearing in mind the manner in which earthquake-induced energy will be dissipated and the desirability of avoiding brittle failures'.

Subsidence of approach embankments is another form of distress during an earthquake. It has occurred in many earthquakes including Madang in 1970 and San Fernando in 1971. A secondary effect of this can be to impose large forces on abutments, wing walls, etc., leading to structural failures. The Austroads Code covers this in section 2.13.5.2 with the words `consideration shall be given to the effects of excessive settlement of approach embankments and allowances made for increased earth pressures on earth retaining structures', Densification of the underlying material and better compaction of the embankment helps ameliorate this effect.

Liquefaction of granular soils can occur when they are subjected to earthquakes of sufficient duration and intensity. This leads to loss of support for piers and approach embankments. Densification may be appropriate for the approaches and piles need to found in material that will maintain support during such an event.
The Austroads Code addresses this aspect also in Section 2.13.5.2, as follows:

> `This possibility of soil liquefaction should be investigated where saturated sandy soils within 10 m of ground surface have a SPT value of 10 or less'.

It has been argued that raking piles should be avoided because of the large forces generated on headstocks and the limited ductility of the pile configuration. The argument is based, at least in part, on performance in Alaska 1964 and Madang 1970. A counter argument is that the resulting cracking and distress was caused by the failure to detail for the resulting forces rather than any inherent shortcomings of a raking pile system since similar damage was caused in some cases where the piles were vertical.

Current approach to design

For the immediate future a reasonable approach to designing bridges for earthquakes in Australia would be as follows:

- •Obtain acceleration coefficient from the information in AS1170.4 with 'a' factored to reflect a 2000 recurrence interval for bridges as against a 500 year interval in AS1170.4. An appropriate factor for this might be 2.

- •Obtain values of R_f for the bridge type from ATC6 'Seismic Design Guidelines for bridges'.

- •Obtain the earthquake base shear force from Clause 6.2.2 of AS1170.4 noting that the importance factor and the period may be obtained from the 1992 Austroads Code.

- •Design and detail the bridge for this force and other provisions of the Austroads Code.

Additional guidance may be obtained from New Zealand and Papua New Guinea documents listed in the references and from the Australian Earthquake Engineering Manual.

Continual reading about failures in earthquakes will add to one's experience but care is needed in interpretation.

Conclusions

The following conclusions may be drawn:

- •Even though AS1170.4 excludes bridges from its provisions, there is ample guidance available for design purposes in the literature.

- •Austroads Bridge Design Code, in conjunction with AS1170.4 and ATC6, addresses modes of failure and calculation of forces.

- •Detailing is critical - think detailing and not simply forces.

- •All design actions should be catered for. Do not become obsessed with one to the exclusion of others.

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The Feasibility of Base Isolation for the Seismic Protection of Lifeline Structures in Australia

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Abstract With the increasing use of base isolation for the seismic protection of buildings in high risk areas, some engineers are beginning to consider its use in Australia. However, unlike Japan, New Zealand and parts of the United States, Australia is an area of low to moderate seismicity with few recorded earthquakes greater than Richter magnitude 7. This paper first discusses the fundamental principles of base isolation and its background. The feasibility of base isolation with regard to lifeline facilities in Australia is then examined. The paper concludes with a discussion of several potential applications.

Introduction

The rarity of large earthquakes in Australia, and the sparsity of the population over the continent have resulted in the community perception that earthquakes pose little risk to the general population. Notably, this view is also held by many in the engineering profession. However, Australia regularly experiences earthquakes greater than Richter magnitude M=5 (about 1 in 6 months according to AS 1170.4 Supplement 1, (SA, 1993-b)) and M=6 (about 1 every 5 years). When this size of earthquake occurs near a centre of population, much damage and even deaths can result as evidenced by the city of Newcastle in December 1989. It is of interest to note that since 1843 the New Zealand city of Auckland has experienced only one earthquake of Modified Mercalli (MM) intensity 6-7 (in 1891); none of MM 5-6 and only 5 or 6 of MM 4-5 magnitude (Dowrick, 1991). For comparison, Meckering in Western Australia has experienced two earthquakes of Richter magnitude greater than M_L=6 (M_L=6.9 in 1968 and M_L=6.2 in 1979), a magnitude M_L=5.5 (MM 7-8) earthquake shook Adelaide in 1954; and the 1989 Newcastle earthquake had a magnitude of M_L=5.6 (MM 8-9) (Gaull et al, 1990; McCue, 1991). Clearly, there is a discrepancy between the perceived seismicity of Auckland and that of Australia whereas the historical evidence suggests that there may be little difference. In fact, recent work by Australian seismologists has led to an updated version of the Australian seismic risk map which has been incorporated in the new Australian earthquake loading code, AS 1170.4 (SA, 1993-a). Based on this work and the work of Dowrick in New Zealand, it appears that the seismicity of parts of Australia and Auckland are not all that much different. Noting that one base-isolated building has already been built in Auckland (Boardman et al, 1983), there is no reason to believe that base isolation is any less feasible a seismic resistant design strategy for Australia and as such should at least be considered as an option at the preliminary design stage for lifeline facilities and other critical structures including hazardous facilities.

Fundamental Principles Of Base Isolation

Base isolation is rapidly becoming accepted as a technique for protecting structures against earthquake attack. For example, since 1985 at least 67 buildings in Japan and at least 6 bridges and 17 buildings in the US have been built or retrofit using some form of base isolation. The story is the same in New Zealand where, since 1974, at least 7 buildings and 29 bridges have been built or retrofit using base isolation for protection from earthquake induced forces (Aiken, 1994). To date, the best "test" of a base isolated structure occurred during the 1994 Northridge earthquake in southern California (EERI, 1994). This earthquake had a local Richter magnitude of M_L=6.4 so was not an exceptionally large earthquake, even by Australian standards. Reports on the effect of this earthquake on lifeline facilities are not encouraging. Of particular interest here is the performance of one base-isolated hospital relative to six fixed-base hospitals in the epicentral region. Nonstructural damage forced the temporary closure, evacuation, and/or patient transfer in at least 5 fixed-base hospitals while significant structural and nonstructural damage forced the closure of a sixth fixed-base hospital. In contrast, the University of Southern California's 7-storey base-isolated teaching hospital suffered no damage, "even pharmacy shelf contents did not fall" (EERI, 1994). In view of the estimates that nonstructural damage will be more costly to repair than structural damage, the popularity of base isolation as a means of protecting structures and their contents from earthquake attack continues to grow. Nevertheless, while the initial novelty of base isolation as a design strategy has virtually disappeared in Japan, New Zealand, and California, it is still largely unheard of in Australia.

Seismic isolation is a design strategy which, rather than increasing structural strength, attempts to uncouple a structure from the ground to protect it from the damaging effects of earthquake ground motion. To achieve this result, while at the same time satisfying all of the in-service functional requirements, additional flexibility is introduced at the base of the structure. Additional damping is also required to control the deflections which occur across the "isolation interface". This is illustrated schematically in Figure 1 below. Note how the deformations in the isolated structure are concentrated in the isolation devices whereas the deformations are distributed throughout the conventional fixed-base structure. In essence, by positioning a layer of horizontally flexible isolators at the base of a structure, the ground is then free to move horizontally beneath the structure without inducing large inertia forces in the structure above the isolation interface.

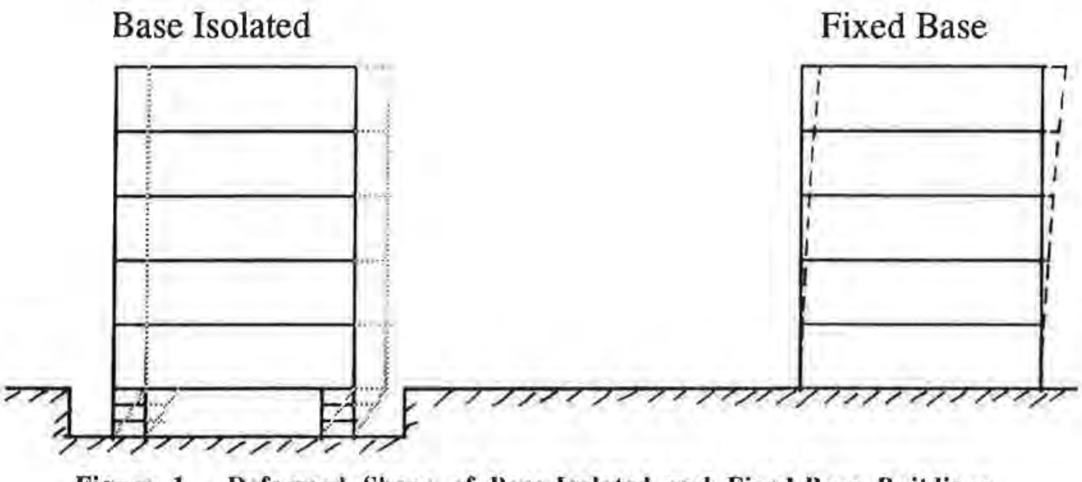


Figure 1 - Deformed Shape of Base-Isolated and Fixed-Base Buildings.

How Does Base Isolation Work?

The main reason for the success of base isolation is best illustrated by considering an idealised acceleration response spectrum (Figure 2). By increasing the fundamental period of a structure with horizontally flexible base isolation devices, the earthquake induced inertia force (which is directly related to the acceleration response) can be significantly reduced. For example, USC's base-isolated teaching hospital (EERI, 1994) was subjected to a peak ground acceleration of 0.37g during the Northridge earthquake and responded with a maximum acceleration at the roof level of only 0.21g. This corresponds to an amplification ratio of 0.57 (=.21/.37) which is in stark contrast to the usual amplification ratios seen in fixed-base buildings of between 2 and 4.

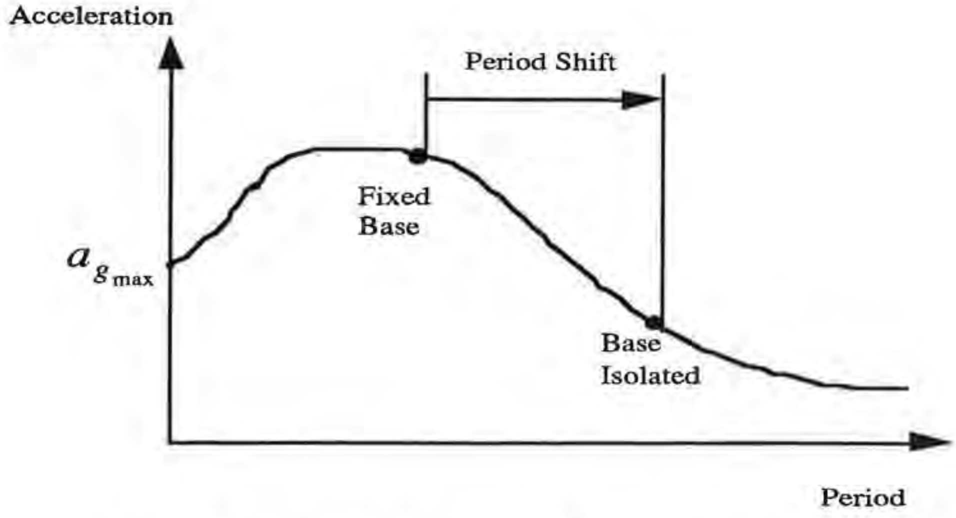


Figure 2 - Idealised Acceleration Response Spectrum.

The benefit of reduced accelerations comes at a cost, however, of increased displacements. This is illustrated in Figure 3 where it can be seen that an increase in structural period results in increased displacements. This is controlled in practice with the use of additional damping; an essential component of any practical base isolation system. A further consideration is the behaviour of the base-isolated structure under serviceability loads. To address this problem it is common to include a mechanism which restricts the movement of the base of the isolated structure under serviceability wind loads, and serviceability earthquake loads in highly seismic areas.

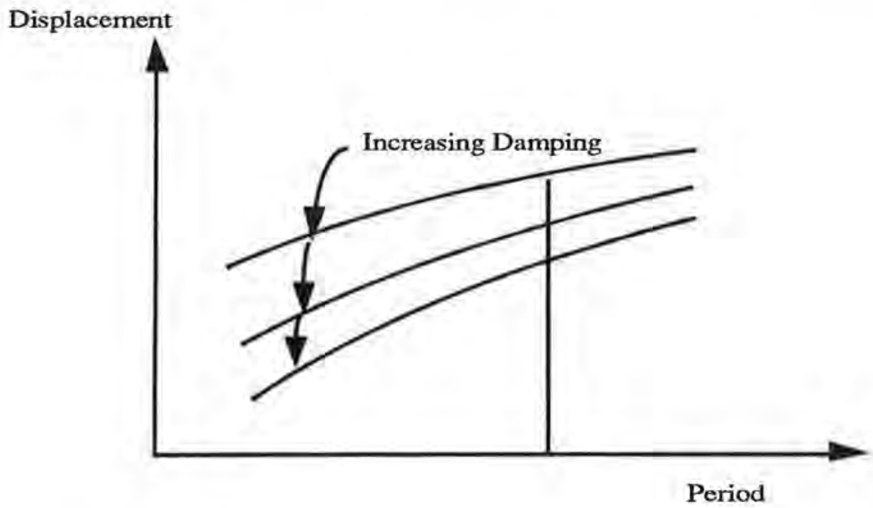
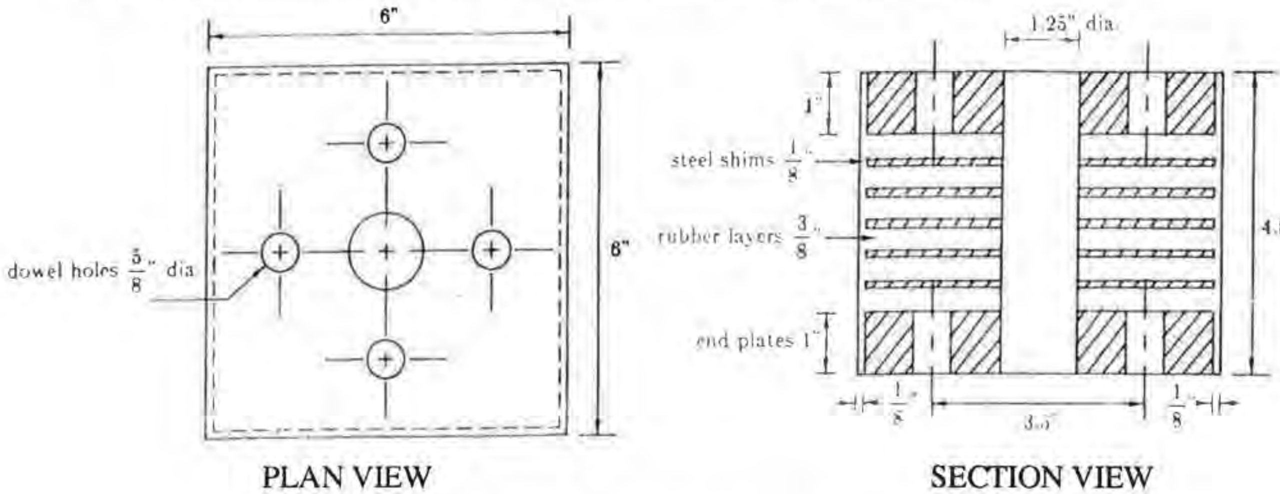
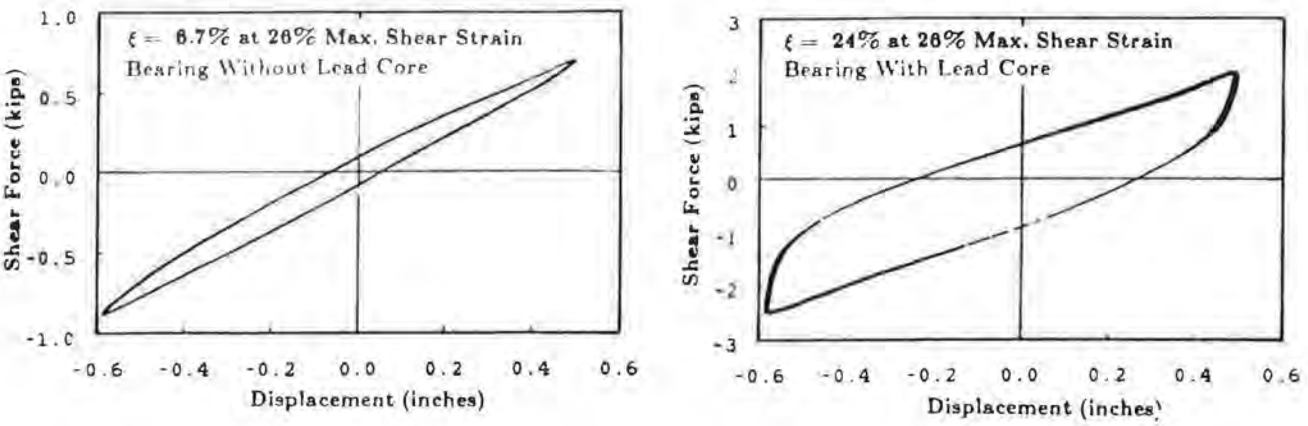


Figure 3 - Idealised Displacement Response Spectrum.

There are many different types of base isolation devices, too many to discuss in detail here. Interested readers can find more detailed information in a book by Skinner et al (1993) or Kelly (1993). However, one system which has been used widely is called a lead-rubber base isolation bearing. This particular device contains all the components of a practical base isolation system so for illustrative purposes it is highlighted here. It is horizontally flexible, it provides increased damping due to the lead core, and it is sufficiently stiff at serviceability force levels to restrict movement of the structure. That is, at force levels below the yield force level of the lead core the horizontal stiffness of the bearing is quite high, thereby keeping the horizontal movements to a minimum. A representative lead-rubber bearing is shown below in Figure 4 together with test results showing a plot of its load versus displacement characteristics with and without the lead core (Griffith, 1998).



(a) Plan and Cross-Section Views of a typical Lead-Plug Bearing



(b) Plot of Shear Force versus Shear Displacement for Bearing with and without Lead-Plug

Figure 4 - Typical geometric and load-displacement characteristics for a lead plug bearing (after Griffith, 1988)

Earthquake Resistant Design Philosophy

It should be emphasized that the basic objective underlying the Australian Earthquake Loading Code, AS1170.4 (SA, 1993-a) is the prevention of loss of life. Consequently, earthquake resistant design in Australia for conventional buildings consists mainly of an ultimate strength limit state check to ensure that buildings do not collapse during a large earthquake. It is assumed that buildings designed for serviceability and ultimate strength limit state load combinations of dead, live, and wind loads will have the inherent strength to resist serviceability level earthquake forces. The key point here is that it is NOT assumed that NO DAMAGE will occur in structures designed in accordance with the earthquake loading code. In fact, it is assumed that significant yielding and corresponding damage probably will occur but that the structure will not collapse during a *design magnitude earthquake*. This basis for design may be acceptable for most structures but it is clearly not acceptable for lifeline facilities which are essential to the emergency response and recovery in the aftermath of a large earthquake.

The Australian earthquake loading code attempts to address this problem by increasing the earthquake design force by 25% through the use of an importance factor, $I = 1.25$. Essentially, this increase is tantamount to increasing the return period of the design earthquake. However, with regard to a lifeline facility, a 25% increase in design force may not be adequate to ensure the facility is operational immediately after a large earthquake. An alternative and possibly more reliable approach may be to use base isolation as a means of limiting the amount of inertia force which an earthquake can induce within a structure and design the structure and its essential components to resist this lesser amount of force elastically, ie, without damage. This technique has the twin advantages of simultaneously reducing the forces on the structure AND its contents!

Feasibility Of Base Isolation

The feasibility of base isolation of lifeline facilities in regions of low to moderate seismicity depends upon a number of factors. These include the construction technique, structural system, the magnitude of the design earthquake force, but probably most important is the level of performance demanded of the structure during an earthquake. The benefits of improved performance must then be evaluated from the point of view of total life cycle costs for the structure taking into account the additional cost of isolation. However, assigning a dollar amount to the improved performance is difficult. In regions where earthquake insurance costs are significant it would be possible to relate improved performance to reduced insurance premiums but to date the insurance industry has not recognised the benefits of base isolation in this way. Consequently, it has been suggested that the base isolation system could be thought of as being the insurance policy in these areas and not taking out conventional earthquake insurance for the structure and its contents. These arguments are not, however, required to study the feasibility of base isolation of lifeline facilities. Indeed, from the point of view of functionality after a large earthquake this question can be considered by looking at the strength demands for a lifeline structure to withstand the design earthquake without significant damage.

If it is assumed that elastic response is commensurate with no structural damage, then it is possible to use the earthquake design force equations in AS1170.4 to obtain some indication of the benefits of base isolation.

The earthquake design force, V, is given by

$$V = \text{the lesser of } (CSI/R_f)W \text{ and } (2.5aI/R_f)W$$
$$\text{where } C = 1.25a/T^{2/3}$$

In this expression, C is referred to as the seismic design coefficient, S is the soil amplification factor, I is the structural importance factor, R_f is the structural response factor, a is the site earthquake acceleration design coefficient, T is the fundamental structural period in seconds and W is the total gravity load of the structure. The structural period can be estimated using the equation

$$T = \frac{h}{46}$$

where h is the height of the structure in metres. Thus, if the expression for the earthquake design force V is plotted against the fundamental structural period T (see Figure 5) it is evident that the design force curve has a shape which is very similar to that of the acceleration response spectrum discussed earlier (Figure 2). Hence, this curve can be used to estimate the decrease in the seismic strength demand of the structure which would result through an increase in structural period.

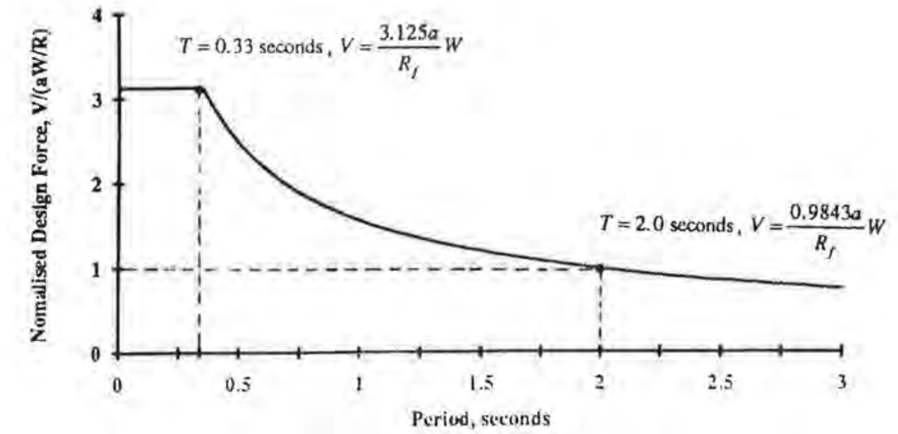


Figure 5 - Normalised Earthquake Design Force Spectrum (S = 1.0, I = 1.25)

For example, consider the case of a lifeline structure (I = 1.25) which is a 5-storey fixed-base frame plus concrete shear wall building with storey heights of $3m$ at a site corresponding to a soil amplification factor S = 1. The fundamental period is given by

$$T_{fixed-base} = \frac{h}{46} = \frac{3m \cdot 5storeys}{46} = 0.33 \text{ seconds}.$$

For T = 0.33 seconds, $CS > 2.5a$ so that

$$V_{fixed-base} = \frac{2.5aI}{R_f}W = \frac{3.125a}{R_f}W \; .$$

The earthquake design force for the same facility constructed using base isolation to give a fundamental period of T =2 seconds is given by

$$V_{base-isolated} = \frac{CSI}{R_f}W = \frac{1.25aSI}{T^{2/3}R_f}W = \frac{1.5625a}{2^{2/3} \cdot R_f}W = \frac{0.9843a}{R_f}W \; .$$

Hence, base isolation gives a force reduction of approximately 3 ($\approx 3.125/0.9843$) for the same R_f due solely to the period shift from T = 0.33 seconds to T = 2 seconds.

Interestingly, the ratio of 3 between the fixed-base and base-isolated design forces is similar in magnitude to the assumed ductility demand associated with the structural response factor, $R_f = 6.0$, given in AS1170.4 for frame plus concrete shear wall structures. In other words, the strength required for the base-isolated structure to resist the design earthquake without damage is only slightly larger than that required for the fixed-base structure to resist the design earthquake allowing significant structural damage (ductility demand, , of between 3 and 4). In fact, the lateral strength required for the fixed-base structure to resist the design earthquake without damage is between 3 and 4 times the strength required for the base-isolated structure to resist the design magnitude earthquake without damage!

The design forces associated with these levels of response were calculated for each of the capital cities in Australia, plus Newcastle because of the 1989 earthquake there, and are presented in Table 1 below. As can be seen, the lateral earthquake force for conventional fixed-base design varies between 5.7% and 2.6% of the weight of the building. Most designers would not have difficulty designing a building to resist these levels of force, however, it should be noted that these force levels correspond to substantial damage in the building during a "design" earthquake (ductility demand ratio of at least $\mu = 3$). In contrast, the design force for the same building (fixed-base) designed to remain elastic ($\mu = 1$) varies between 10% and 23% of the gravity load of the building. Finally, the last column of Table 1 shows the values for the design force for the structure in the base-isolated condition with elastic response ($\mu = 1$). These forces are very similar to the inelastic response design forces in column 2.

CITY	FIXED-BASE DESIGN FORCE ($R_f = 6.0$)	FIXED-BASE DESIGN FORCE ($R_f = 1.5$)	BASE-ISOLATED DESIGN FORCE ($R_f = 1.5$)
Newcastle	0.057W	0.23W	0.072W
Adelaide	0.052W	0.21W	0.066W
Perth	0.047W	0.19W	0.059W
Canberra, Darwin, Melbourne, and Sydney	0.042W	0.17W	0.052W
Brisbane	0.031W	0.12W	0.039W
Hobart	0.026W	0.10W	0.033W

Table 1 - Earthquake Design Forces for 5-Storey Lifeline Building

A further, and possibly the most important, point in favour of base isolation for lifeline and essential facilities is related to the uncertainty surrounding the assessment of seismic risk in regions of low seismicity. Probably more than anything else, the question of earthquake data reliability characterizes the problem of earthquake design in low to moderate earthquake risk areas. While the probability of an earthquake occurring is lower in intraplate zones like Australia, the likelihood of earthquake damage is not necessarily lower since the level of force for which the buildings are designed is also accordingly lower. For example, research at The University of Adelaide (Griffith, 1994) indicates that reinforced concrete structures should perform well in a "design magnitude" earthquake but that they tend to have catastrophic collapse mechanisms which are activated when overloaded.

Melchers (1991) has argued that the current attenuation models and methods of seismic risk determination are inappropriate for low seismicity regions with short recorded earthquake history. Dowrick (1991) states that current practice tends to over estimate the seismicity of Auckland, New Zealand, however, because of the uncertainty surrounding the estimation of seismicity for low to moderate risk areas, Booth et al (1990) suggests use of longer return period earthquakes (2000 years) for design in low risk regions. This is in contrast to the more commonly used 500 year earthquake for design in high seismicity regions. Thus, while Dowrick is in basic agreement with Melchers that current practices tend to over estimate the seismicity in regions such as Australia, Booth suggests that it might be appropriate to design for more rare events in these regions to provide an acceptably small chance of building collapse. Clearly, if this approach were adopted in Australia, base isolation could become a feasible design option for a wider range of building projects.

Potential Applications

Lifeline and essential facilities are facilities with functions which are vital to the community. Their functionality is critical for both the emergency response in the immediate aftermath of a large earthquake and the socioeconomic recovery of the community over the longer term. Many lifeline facilities reported damage following the 1994 Northridge earthquake near Los Angeles even though the earthquake was not large. For example, electric power was lost over a large part of the Los Angeles area and because interconnection of power grids, people as far away as Idaho were also affected for up to three hours (EERI, 1994). Transportation systems, water supply, wastewater systems, gas, liquid fuel, and telecommunications facilities also all suffered damage to varying degrees. Health services were also severely affect by the earthquake with structural and nonstructural damage forcing the evacuation and closure of a number of hospitals. Fortunately, the Los Angeles area has an extensive health care system so that health care services were able to cope with the emergency. The remainder of this section discusses some potential applications for the use of base isolation in Australia as a means for the protection of lifeline facilities and structures, drawing largely on the experience of the Newcastle earthquake.

Telecommunications Facilities: The communication system did not function well after the Loma Prieta earthquake. This had a negative impact on the efficiency of emergency service providers and highlights the need for reliability in this lifeline system as it was widely felt that adequate measures were in place before the earthquake to cope with a much larger seismic event. Closer to home, a Telecom building housing switching equipment was damaged in the 1989 Newcastle earthquake. The Hamilton telephone exchange building in Newcastle did not suffer any significant structural damage, however, extensive damage occurred to infill masonry walls (both internal and external), a lift motor room, plant room, chimneys and parapets. While no equipment was damaged, service was reduced (by 87% at one stage) for nearly seven hours when equipment racks vibrated loose from their positions, severing several power cables (IEAust, 1990-b). Clearly, any building required to serve a post-disaster function should have a high degree of reliability associated with it and base isolation has been shown to be a feasible method for providing such reliability from a structural and contents point of view.

Electric Power Supply: Several porcelain insulators supporting 132kV switch gear were damaged during the 1989 Newcastle earthquake at the Killingworth substation. This interrupted power for up to four hours as the insulators were replaced at a cost of over $1.5 million (IEAust, 1990-a). Subsequently, it has been proposed to isolate some of these transformers using a three dimensional spring system (Safi, 1989) as the cost of implementing such a system is small compared to the consequences of further shut-downs. For example, the power outage which occurred as a result of the shaking at the Killingworth substation came within half an hour of causing permanent damage to the pot line in an aluminium smelter. Interestingly, the local electricity supplier had signed an agreement at the time of construction of the smelter plant making the electricity supplier responsible in the case of just such an event. The replacement cost would have been well over one billion dollars but much to their relief, power was restored in time.

Hospitals, Fire and Police Stations: A large number of these buildings in Australia are built using unreinforced masonry construction (URM) due to the durability, low maintenance costs and economy of the building material. However, it is widely known that URM construction does not perform as well in earthquakes as other more ductile forms of construction. With the increasing awareness of earthquake hazards in Australia, the earthquake safety of these essential facilities should be investigated. In some cases, it may even be practical to seismically upgrade these structures using procedures similar to those outlined by Whittaker (1994) and which have already been employed on a number of projects around the world.

Transportation Networks, including Bridges: The transportation system was not badly affected in Newcastle during the 1989 earthquake. However, some of the larger cities in Australia have components in their transportation networks which would lend themselves quite easily to base isolation. Many bridges, in fact, are already supported on elastomeric bearings in Australia to allow for thermal expansion and support settlement. With minor modifications, these devices can be made to provide seismic protection in addition to these other design considerations. Indeed, a number of bridges have already been seismically isolated in the eastern United States (Aiken, 1994), a region with similar seismicity to much of Australia.

Summary

Obviously, base isolation is not suited to all types of lifeline structures. It is useful mainly for low- to medium-rise structures which are restricted in size. For example, it would not be feasible to isolate an entire road network whereas it could be feasible to isolate a bridge in that network. Hence, base isolation is rapidly being adopted as a practical method for providing a high level of protection against earthquake attack for bridges and buildings in countries commonly associated with having a high probability of earthquake occurrence. It is becoming increasingly apparent that base isolation may also be applicable in low to moderate earthquake risk regions of the world such as Australia where less ductile forms of construction are common and when a high level of performance is demanded from a structure during earthquakes, i.e. lifeline structures.

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Attenuation of Strong Shaking in Southeastern Australia

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Introduction

New valuable recordings of strong ground motion have been obtained following earthquakes at Ellalong near Cessnock NSW and Eugowra NSW. These and earlier data recorded in both eastern and western Australia can be normalised to a common magnitude to compute attenuation in Australia, to compare the attenuation in Eastern and Western Australia and provide better estimates of the ground motion in Newcastle NSW during the magnitude 5.6 earthquake there in 1989.

This data set also provides the basis for a review of earthquake hazards estimates in Australia and the foundation for innovative research into the mechanics of intraplate earthquakes.

Here we confine our study to a preliminary analysis of the Ellalong earthquake dataset.

The recording network

A quantum leap has taken place in the strong motion recording capability in Australia over the last 5 years. This happened in response to the Newcastle earthquake and was made possible by the development in Australia of modern digital recorders over the last 2 decades.

Analogue recorders were installed in Adelaide in 1972 by the University of Adelaide and in Dalton NSW and the SouthWest Seismic Zone WA by BMR in 1974 (Figure 1). A handful of useful records was obtained from these instruments. In the mid 1980's these accelerographs were supplemented with early digital recorders, Australian Yerillas in Eastern Australia and American A700s in Western Australia.

As a result of the Newcastle earthquake and the lack of ground motion data obtained near the epicentre, Commonwealth and State representatives met in Canberra in February 1990 and put together a plan for monitoring the major urban areas. Federal Cabinet approval was subsequently won but the Commonwealth funding was not met by the States and the total amount allocated only allowed for the purchase of accelerographs and a few seismographs. There was no ongoing allowance for maintenance, for installation or running costs, or for the analysis of the extra data.

Fortunately the State Governments of South Australia, Queensland, New South Wales, Victoria and Tasmania have agreed to pay the annual running costs of the instruments which have now been installed in those States.

Digital accelerographs were also installed by the ACTEW on their dams in the Canberra region, by Telecom in Black Mountain Tower, and by the Parliament House Construction Authority. The (Sydney) Water Board has installed a large network of recorders around their dams south of Sydney.

The major engineering requirement was fulfilled so that in the event of a major urban area being shaken, it is at least possible that a measure of the ground motion will be obtained.

The Data

Following implementation of part of the urban monitoring proposals, dividends were almost immediately forthcoming with small local earthquakes being recorded in Brisbane, Adelaide, Newcastle and Canberra. When most of the instruments had been installed and were operational in southeastern Australia, a damaging magnitude ML 5.3 earthquake occurred near Cessnock NSW. This earthquake is discussed elsewhere (Jones & others this volume) but preliminary analysis of the strong motion data is outlineded below.

An accelerograms recorded at North Lambton, NSW at about 43 km from the epicentre of the Ellalong earthquake epicentre is shown in Figure 1 below. The vertical component is the bottom trace and the time scale is in seconds. The record is notable for the short duration of strong shaking of about 5 s, the low amplitude of ground motion 0.015 g, and the frequency range. The dominant ground motion has a frequency of 2 Hz which is lower than observed in earlier Australian accelerograms recorded close to the focus but normal for accelerograms recorded overseas.

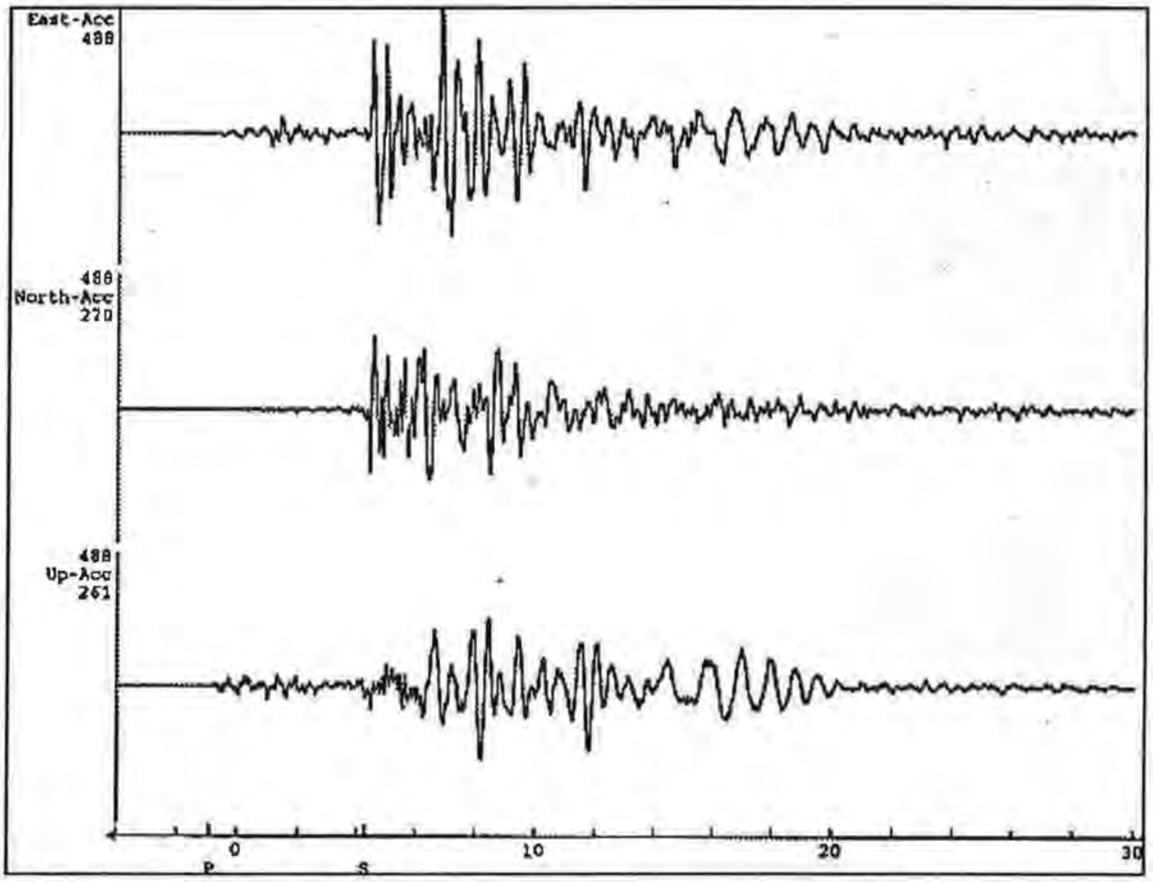

Figure 1 Accelerogram recorded at North Lambton NSW, 43km from the epicentre of the Ellalong earthquake

As the instruments were being retrieved from Cessnock, a swarm of small earthquakes commenced near Eugowra NSW, so the instruments were reinstalled there. (Granite was mined near Eugowra for the new Parliament House in Canberra. When the overburden was removed, pop-ups occurred in the granite and the cut facia slabs were found to later deform with removal of the in-situ stress.) This earthquake swarm, with a largest earthquake of magnitude ML 4.0, has provided hundreds of interesting and useful accelerograms (Gibson & others, this volume). The Ellalong earthquake dataset has allowed some early estimates of attenuation in the distance range 40 - 350 km.

Attenuation

A plot of peak horizontal ground accelerations versus distance *R (km)* recorded from the ML 5.3 Ellalong earthquake is shown below in Figure 2. Only those accelerographs sited on rock were used, some of the instruments at greater distances were in basements of buildings and these have been deleted. The axes have log-log scaling so the amplitudes show considerable scatter. A least squares curve has been fitted to the data and appears to be a reasonable fit (R = 0.94) though underestimating accelerations *a (g)* at the closer stations compared with those observed:

$$a = 5.1 * R^{-1.8}$$

The relatively low ground motions are rather surprising, the imputed peak ground acceleration is already below 0.1g only 10 km from the epicentre but there were no recordings closer than 43 km.

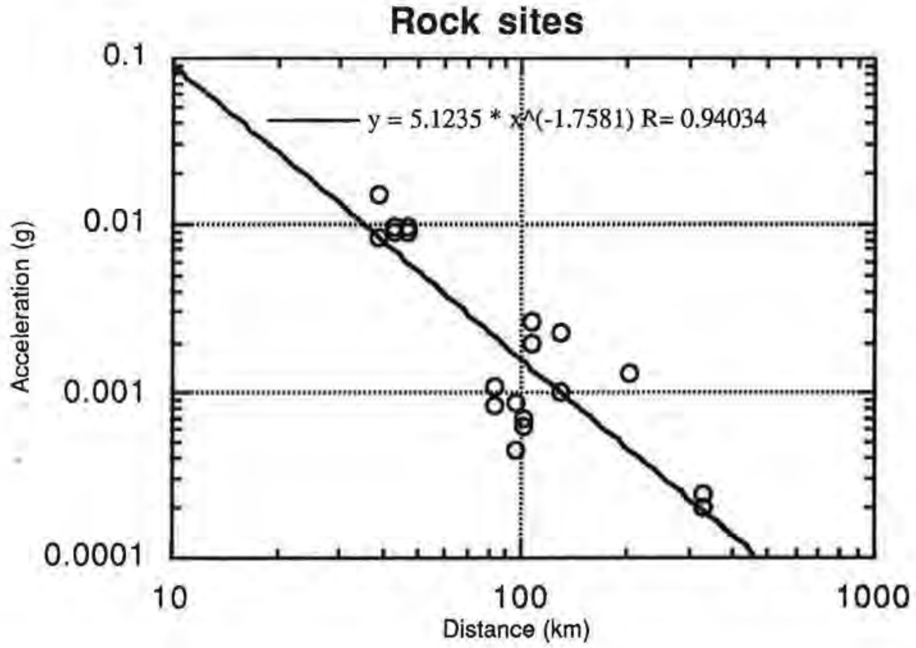

Figure 2 Peak ground acceleration (g) recorded on digital accelerographs on rock in South-eastern Australia during the Ellalong earthquake.

The computed ground motion at Hamilton during the Newcastle earthquake

An important question which the Cessnock data allows us to examine is: 'what was the ground motion in Hamilton and the Newcastle CBD during the 1989 Newcastle earthquake?'. There were no instruments, neither seismographs nor accelerographs, capable of recording the shaking in Hamilton or the Newcastle CBD at the time.

McCue (1991) made an estimate of the peak ground motion and concluded that *the strong motion lasted only 1 or 2 seconds and had a peak acceleration in the range 0.3 to 0.8 g at a frequency near 10 Hz.*

The Cessnock earthquake dataset enables a check of this educated guess. The epicentral distance of the closest recorder at North Lambton from the focus near Ellalong was 43 km compared with 15 km during the 1989 Newcastle earthquake. The ground motion at 15 km can be estimated by extrapolation of the line of best fit constrained to pass through the centroid of the clustered closer points. This gives 0.05 g with a standard deviation range of 0.02 - 0.14 g.

The Cessnock earthquake magnitude was 5.3 compared with 5.6 for the Newcastle earthquake. To convert the accelerations (a) from one magnitude (M_1) to another (M_2) at the same distance, Esteva's (1974) relation was used. This gives the ratio of the accelerations as $\exp\{0.8 (M_1-M_2)\}$ which is 1.27 in this case.

The resultant peak ground motion on rock at 15 km distance from a magnitude 5.6 earthquake is 0.063 g with a standard deviation range of 0.025 - 0.18 g

At the surface of a soil layer such as that at Hamilton, the estimated magnification factor is 2 to 4, from the intensity difference between Hamilton and the epicentral region (McCue & others, 1990; Somerville & others, 1993). Using the mean value of 3, the estimated mean peak ground acceleration at ground level under Hamilton is 0.19 g with a standard deviation ranging from 0.08 - 0.54 g.

The nearest recorder was at 43 km focal distance and we are extrapolating linearly to 15 km to compute the ground motion at Hamilton. If we scale up the Eugowra data (ML4.0, 0.43 g at 1.1 km) to magnitude ML 5.3 then we get a ground acceleration of 1.2 g at 1 km.

Comparison of Western and Eastern Australian data

The peak ground acceleration from a few WA accelerograms of earthquakes of magnitude 3.5 or more were normalised using the Esteva scaling relation above to convert to peak ground accelerations for a magnitude ML 5.5 earthquake. A low magnitude cutoff of 3.5 was adopted to minimise the uncertainty in the normalising factor used. These data were then plotted with the NSW data from the Cessnock earthquake.

Though not shown here, the few data points for the two regions are virtually inseparable out to 100 km though there is some suggestion that beyond this distance the WA amplitudes may be systematically higher than those in Eastern Australia. The largest amplitude waves on seismograms of local earthquakes recorded in WA are those of the surface waves, much larger normally than either the P or S body waves, and relatively larger than those observed in Eastern Australia. More data is needed to confirm this apparent similarity in the attenuation rates in eastern and western Australia.

For an earthquake of magnitude 5.5, the expected mean peak acceleration is 0.1 g or more to distances of about 10 km from the focus. The scatter in the data is large however and the 0.43 g horizontal acceleration recorded at Tennant Creek in 1988 at 8 or 9 km from a magnitude ML 4.9 earthquake (McCue & Paull, 1991) is within the scatter.

Discussion

An acceleration attenuation relation has been developed for southeastern Australia, (strictly for a magnitude ML 5.3 earthquake):

$$a(g) = 5.12 \exp(M-5.3)\, R^{-1.8}$$
$$M \leq 5.6$$

This relationship has been used to estimate the ground motion on 28 December 1989 on alluvium in the Hamilton region of Newcastle where the magnification factor was estimated to be about 3. On this model the peak ground acceleration was estimated to be in the range 0.08 - 0.54 g with a mean at about 0.19 g. The scatter is large about this mean value, as it is in other countries such as the USA and New Zealand.

Comparison of the data from southwest WA and southeastern Australia shows agreement out to a distance of about 100 km which is surprising on face value. The ground motions are also lower than observed in the US or New Zealand, though most Australian data are from close small eathquakes and overseas data are generally from large more distant earthquakes.

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The 1994 Northridge California Earthquake and Bridge Performance

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Introduction

The Northridge earthquake of January 17, 1994, strongly affected the northern parts of Los Angeles and the San Fernando Valley and surrounding areas in southern California. It was the most costly single natural disaster in the history of the United States. This magnitude 6.7 earthquake occurred at 4:31 am local time on a Monday, and resulted in about 65 deaths and over 5000 injuries. Preliminary damage estimates are in the range of $US15-30 billion.

The earthquake occurred in a highly-populated urban area. Most affected structures were built in this century. The earthquake caused serious damage and failures in commercial and residential buildings, destruction of the contents of many structures, damage to critical transportation systems, and widespread disruption of utilities and other lifelines. Of great public concern was the collapse or partial collapse of seven bridges of the freeway system (Figure 1). In part because of the time of the occurrence, only one life was lost from these lifeline collapses.

The 1994 Northridge earthquake was in an urban area containing structures of many types. It provided a first test for many modern seismic design practices. Many of these appear to have been very successful, but some now appear to be questionable. The damage to steel bridges and recently completed steel-braced and welded moment frame buildings was unexpected. Recently-constructed bridges and post-1987 retrofitted reinforced concrete bridges, on the other hand, appeared to perform reasonably well.

Report to the California Department of Transportation

Caltrans appointed a Seismic Advisory Board in September, 1990, following recommendations contained in the report, *Competing Against Time: Governor's Board of Inquiry Report on the Loma Prieta Earthquake* (G. Housner, 1990. Office of Planning and Research, Sacramento, Ca.). This report was in response to the widespread damage and destruction of lifelines in the San Francisco Bay area in the Loma Prieta earthquake. The charge to the Seismic Advisory Board was to provide continued, focused evaluations of Caltrans seismic policy and technical procedures. Since that time, the eight-member Board has regularly reviewed Caltrans seismic design, retrofit and hazard mitigation activities. The Board also had discussions with senior staff engineers and made numerous recommendations.

The 1994 Northridge earthquake provided an opportunity for the Seismic Advisory Board to evaluate after strong ground shaking the performance of Caltrans bridges, retrofit programs, peer review programs, and technical procedures. In response to the Northridge earthquake, a special report of the Seismic Advisory Board was prepared that:

- evaluated the past four years of changes and developments in seismic design criteria and the highway bridge retrofit program.

- summarised Board findings on the performance of highway bridges in the Northridge earthquake.
- recommended improvements to Caltrans bridge seismic design and retrofit programs and procedures.

This summary paper is taken from this report to the Director of the California Department of Transportation, entitled "The Continuing Challenge" (October 1994). For the detailed findings and recommendations of the Board, readers are referred to this report (see Appendix A).

Damage to Highway Bridges in the Northridge Earthquake

Caltrans has approximately 12 000 state highway bridges in California and is responsible for a total of 2523 state and interstate highway bridges in Los Angeles County. Additionally, about 1500 bridges are maintained by Los Angeles County and 800 by the City of Los Angeles; most of these latter bridges are small, single-span bridges and most were remote from the area of strong ground motion. Only a few of the city and county bridges were significantly damaged.

The Northridge mainshock (Mw = 6.7; Mw is the moment magnitude) caused the collapse of seven Caltrans highway bridge structures and the consequent disruption of a large portion of the northwest Los Angeles freeway system. Figure 1 shows the locations of these bridges in relation to the earthquake source. Of the seven bridges that collapsed in the earthquake, five had been scheduled as requiring retrofit. Two bridges, the Mission & Gothic Undercrossing and Bull Creek Canyon Channel on State Route 118, had been identified as not requiring retrofit. The collapsed structures can be classified by vintage into three groups: three bridges designed and built before the 1971 San Fernando earthquake (Mw = 6.6) centred about 10 km NE of the 1994 Northbridge source; two bridges designed before 1971, but construction completed after 1971; and, two bridges designed and built a few years after the San Fernando earthquake, but not to current standards.

Performance of Retrofitted Bridges

All structures in the region of strong shaking that were retrofitted since 1989 performed adequately, thus demonstrating the validity of the Caltrans retrofit procedures; there were 24 retrofitted bridges in the region of very strong shaking and a total of 60 in the region having peak accelerations of 0.25g or greater. The retrofitted structures resisted the earthquake motions much better than the unretrofitted structures. The Board's conclusion was that if the seven collapsed bridges had been retrofitted to the adopted standards, they would have survived the earthquake with little damage.

Caltrans has seismic design performance criteria that set standards for two categories of bridge structures - important and common. Important structures are those that do not have convenient alternative routes, whose economic consequences of failure are large, or that provide secondary life safety or are designated as important by local emergency officials. Technical evaluations are made for each type for two levels of earthquake ground motions - the functional and safety levels.

For the safety level evaluation, the Board interpreted the performance statement as explicitly containing the goal that collapse be avoided in earthquakes for all state bridges, whether new or retrofitted. For the functionality level evaluation, Caltrans has adopted performance criteria that will allow post-earthquake damage inspection and repair with minimal traffic interruptions.

Since the 1971 San Fernando earthquake, Caltrans has been engaged in a multi-phase bridge retrofit program. To date, most expansion joints have been provided with restrainers or seat extensions and most critical single-column-bent bridges have been retrofitted. Prompted by

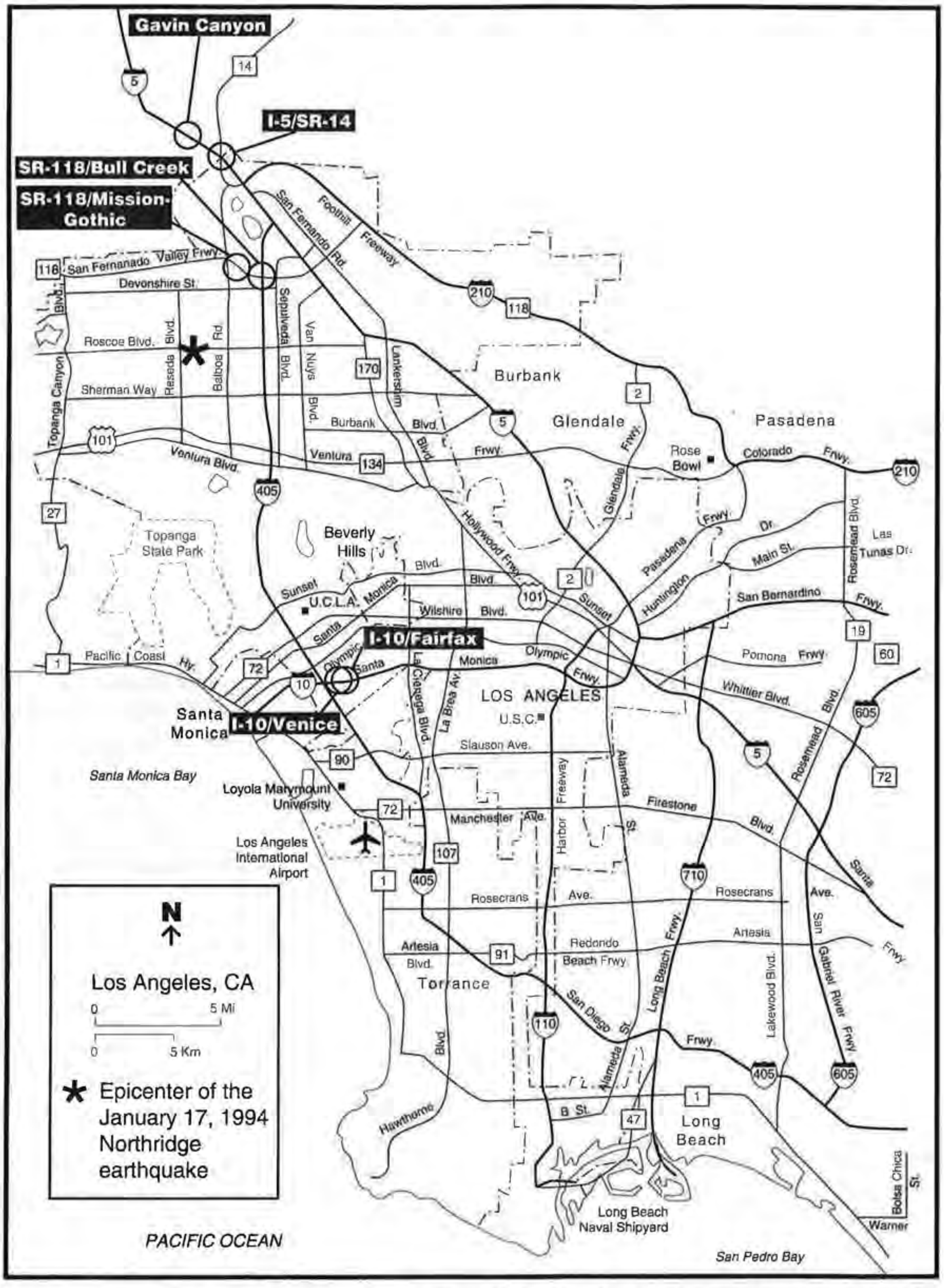

Figure 1 The location of bridges that collapsed in the Northridge earthquake, 2 bridges, the north and south connector overcrossing, collapsed at the 1-5/SR-14 interchange.

the 1987 Whittier Narrows earthquake and amplified by the 1989 Loma Prieta earthquake, both in California, Caltrans accelerated its bridge retrofit program and initiated significant changes in bridge design criteria.

Earthquake Characteristics

The January 17, 1994 main shock of the Northridge earthquake was generated beneath the San Fernando Valley (see Figure 1) near Northridge at a focal depth of about 18 km. It occurred on a blind-thrust fault so called because the principal rupture did not break the surface. The area from Santa Monica north in Figure 1 suffered Modified Mercalli Intensities (MMI) of VII and above. It is of interest that within the intensity VII zone, pockets of intensity VIII are mapped south of the Santa Monica Mountains.

Since the 1987 Whittier Narrows earthquake, the occurrence of blind-thrust fault earthquakes through the Los Angeles and San Fernando Valleys has been widely accepted, with the likelihood of earthquakes of magnitudes about 6.5 generated by slip on them. In this geological sense, the type of faulting which produced the Northridge earthquake was not unexpected. The exact position of the causative fault, however, was not predicted. The intensity of shaking appeared to be systematically somewhat higher than expected, based on average attenuation curves for past California earthquakes. Nevertheless, the majority of ground motions fell within the 84% expected levels (mean plus one standard deviation) and would thus be accommodated by the present probabilistic methods of seismic motion assessment. Apart from a few anomalous sites, contrary to some public impressions, the measured peak vertical accelerations (as compared to the observed horizontal values) were also in the expected range of values.

Numerous strong motion instruments had been placed by the California Strong Motion Instrumentation Program (CSMIP) and the U.S. Geological Survey. One-hundred and thirty-two instruments within a 100-mile radius of the fault rupture area recorded the free-field, strong ground motions. A number of recorded horizontal component peak accelerations, within 30 km of the rupture surface, were in the range 0.5g to 1.0g with a singular value above 1.2g. In summary, this data set shows that:

- Duration of strong motion was about 9 seconds.
- With few exceptions, peak ground motions recorded were within the statistical ranges expected for such an earthquake.
- Ratios of vertical to horizontal peak ground accelerations were typical of past earthquakes, averaging about 2/3.

Strong motion records were obtained from six bridges at distances ranging from 14 to 115 miles. The most significant of these was the record from the I-10/I-405 Interchange, a curved concrete box girder structure, 1037 feet long having nine single-column bents and two open-seated abutments. The bridge was retrofitted in 1991 with steel jackets on some columns. Installation of instruments was completed, funded by Caltrans, just before the earthquake. A peak acceleration of 1.83g was recorded at the box girder near the west abutment. This bridge is located about 4 miles west of the section of the I-10 Freeway that collapsed (see Figure 1).

Summary of Damage to Highway Bridges in the Northridge Earthquake

Damage to Caltrans bridge lifelines was predictable given the high ground motions recorded during the Northridge earthquake. The older bridges were designed for only a small fraction of the earthquake forces they were subjected to in this earthquake, and their damage or collapse was inevitable. The many bridges in the regions of strong shaking that were constructed or retrofitted to current Caltrans criteria had, at most, minor damage; and all remained in service and none posed an increased safety threat during the earthquake.

In more detail, information published by Caltrans has identified earthquake damage to State Highway bridges in the Los Angeles County as follows:

1. Initial Assessment Dated January 21, 1994: Significant bridge damage occurred within an area of about 270 square miles. A total of 506 Caltrans bridges are located within this area. The report damage was:

Collapsed or partly collapsed	7
Major damage	4
Moderate damage	2
Minor damage	18

No damage was reported to post-1987 retrofitted structures or new construction. Damage occurred primarily in older structures designed prior to the 1971 San Fernando earthquake that were not retrofitted or had only partial or inadequate retrofits. An exception to the above is the partial collapse of two 1976-vintage bridges on State Highway 118 in the epicentral area.

2. Detailed Assessment Dated February 9, 1994: This later, and more detailed, assessment of State Highway bridge damage by Caltrans lists the following:

Collapsed or partly collapsed	7
Major damage	39
Other damage requiring repair	194
Hinges requiring repair or replacement	46

Table 1 lists the seven major bridges that collapsed during the Northridge earthquake, along with the data of their design and construction and the probable cause of failure. All seven were constructed to design standards that were much less stringent than those Caltrans currently uses. Many other bridges in the strongly shaken region sustained damaged, but did not collapse and remained in service, either full or limited. The damage ranged from minor cracking and spalling of concrete to more severe damage that necessitated closing the bridge to traffic while repairs were made.

Impact on Traffic Flow in Los Angeles County

Immediately following the Northridge earthquake, Caltrans moved quickly to mobilise construction equipment and personnel to remove debris and restore or reroute traffic where damage had occurred to the highway system.

The failure of the bridges listed in Table 1 caused substantial rerouting of traffic. Traffic records for each of 10 days preceding February 4, show that the average delay on each route decreased as alternate routes were opened and drivers became accustomed to changed highway conditions. As of February 4, 1994, the delays ranged from 2 to 25 minutes, many times less than the initial delay times, which had been as much as 2 hours. These travel time reductions indicate that, while it may require considerable time before the collapsed bridges are replaced, Caltrans has established effective detours, and, except for State Routes 10 and 118, traffic flow was essentially restored to normal volumes within a few weeks.

Table Summary of highway bridge collapses in the Northridge earthquake

Bridge	*Route*	*Bridge number*	*Design*	*Construction completion*	*Restrainer retrofit*	*Probable cause of collapse*
Gavin Canyon Undercrossing	1-5	53-1797 P/L	1964	1965	1974	Skew geometry and unseating of expansion joints
N.Connector Overcrossing	SR-14/1-5	53-1964 F	1968	1974	1974	Short column brittle shear failure
S. Connector Overcrossing	SR-14/1-5	53-1960 F	1968	1974	1974	Short column shear failure
Mission & Gothic Undercrossing	SR-118	53-2205	1973	1976	____	Flexure/shear failure in architectural flared columns at bottom of flare
Bull Creek Canyon Channel Undercrossing	SR-118	53-2206	1973	1976	____	Flexure/shear failure in shortened columns by channel wall & low transverse reinforcement ratio
Fairfax & Washington Undercrossing	1-10	53-1580	1962	1964	1974	Flexure/shear failure short and stiff columns
La Cienega & Venice Undercrossing	1-10	53-1609	1962	1964	1978	Brittle shear failure of stiff columns

Appendix A

Members of the Seismic Advisory Board to the California Department of Transportation.

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The Ellalong, New South Wales, Earthquake of 6 August 1994

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Abstract The Ellalong earthquake occurred at 1103 UTC on 6 August 1994 about 12 km southwest of Cessnock NSW. Its magnitude was 5.4 ML (5.3 MB; USGS) and its computed depth was 1.4 ± 2.3 km. The earthquake was the largest in eastern Australia since the ML 5.6 1989 Newcastle earthquake and its epicentre is only 30 km west of the epicentre of the 1989 earthquake. The two events had similar reverse faulting mechanisms with horizontal northeast-southwest pressure axes. These two earthquakes and three others with magnitudes in the range ML 5 - 5.5 in 1842, 1868 and 1925, have occurred in the Hunter region in the past 154 years. Twenty four micro-seismic events with maximum magnitude of less than ML 1 occurring in the two weeks following the mainshock were located using a field network. Their hypocentres, at depths of up to 1.8 km, weakly define an unmapped 2.6 km long fault striking NNW-SSE beneath the 500 m deep Ellalong longwall coal mine. It is unclear whether these events were aftershocks or part of a sequence of continuing seismicity which has accompanied mining at Ellalong for several years. The earthquake was felt in eastern NSW over a radius of more than 220 km. It caused no serious injuries and few structural collapses although the total damage (about \$34 M from insurance claims) makes it the third most costly earthquake in Australian history. Damage in the meizoseismal area was limited by the presence of firm shallow soils, generally good standards of construction and mostly single storey buildings. Maximum intensities of MM VII were observed near the epicentre and the radius of the MM VI isoseismal is about 8 km. A maximum peak ground velocity of more than 160 mms^{-1} was recorded by a mine vibration monitor within a few kilometres of the hypocentre and a maximum vertical acceleration of 0.34 g was calculated from the same record.

Introduction

The Ellalong NSW earthquake occurred at 9:03 pm AEST on Saturday 6 August 1994 near the villages of Ellalong and Paxton, about 12 km southwest of Cessnock. The earthquake had a local (Richter) magnitude of ML 5.4, the largest earthquake in eastern Australian since the ML 5.6 1989 Newcastle earthquake. Its epicentre was only 30 km to the west of the epicentre of the Newcastle earthquake. Residents in the lower Hunter Valley had experienced strong shaking (MM V-VI) during the Newcastle earthquake.

Insured damage from the earthquake amounted to about \$A34 million (Insurance Council of Australia, 1995) making it the third most destructive earthquake in Australian history after the 1989 Newcastle earthquake and the 1954 Adelaide earthquake.

In this paper we concentrate on only two aspects of the earthquake. The first is the macroseismic data. Meizoseismal data collected for this earthquake are among the most comprehensive ever collected for an Australian earthquake and indicate what the effects of future Australian earthquakes of similar magnitude will be in urban areas founded on rock or shallow soils. The second aspect is the locations of the seismic events recorded by a

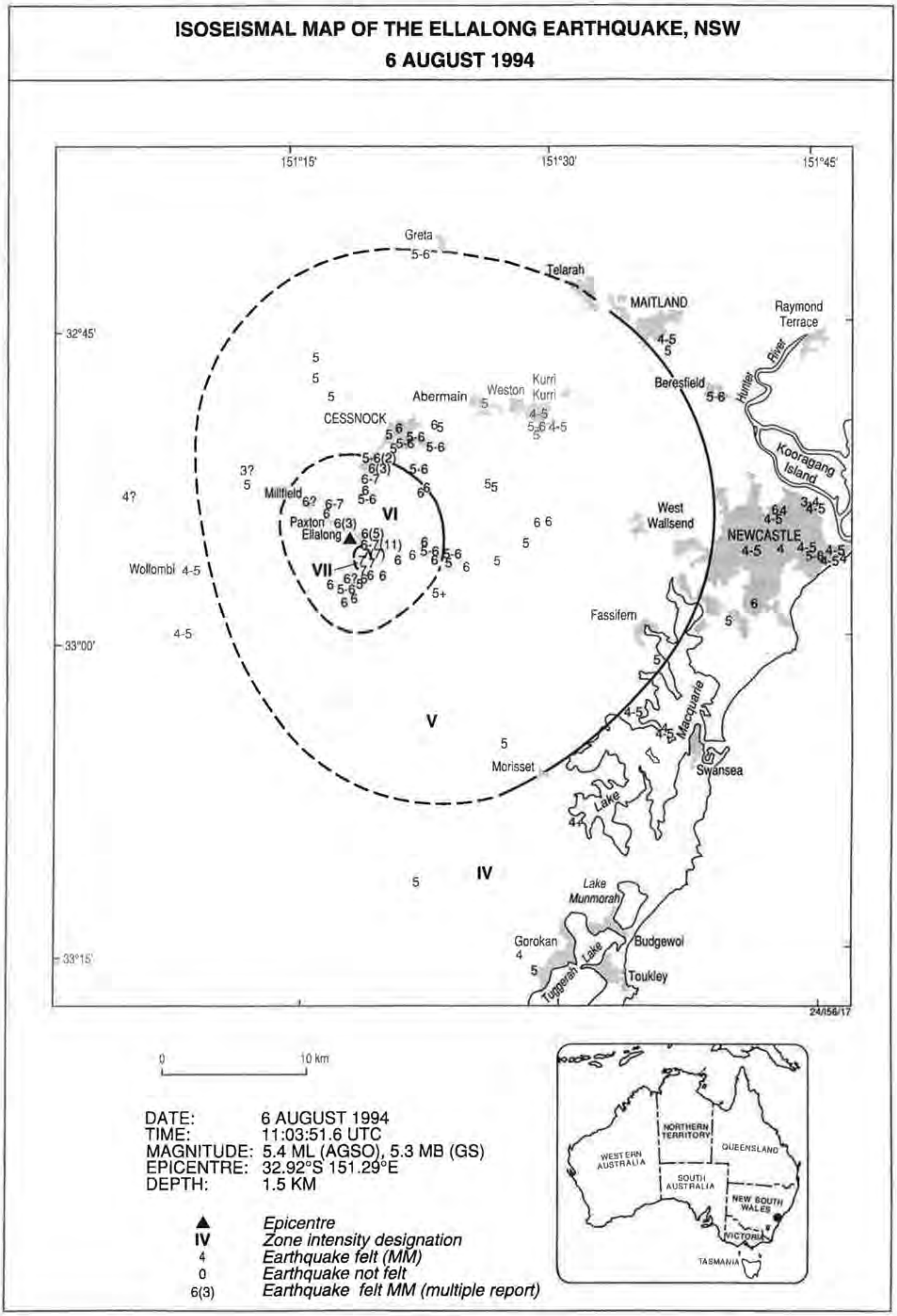

Figure 1 Isoseismal map for the epicentral area of the Ellalong earthquake, 6 August 1994.

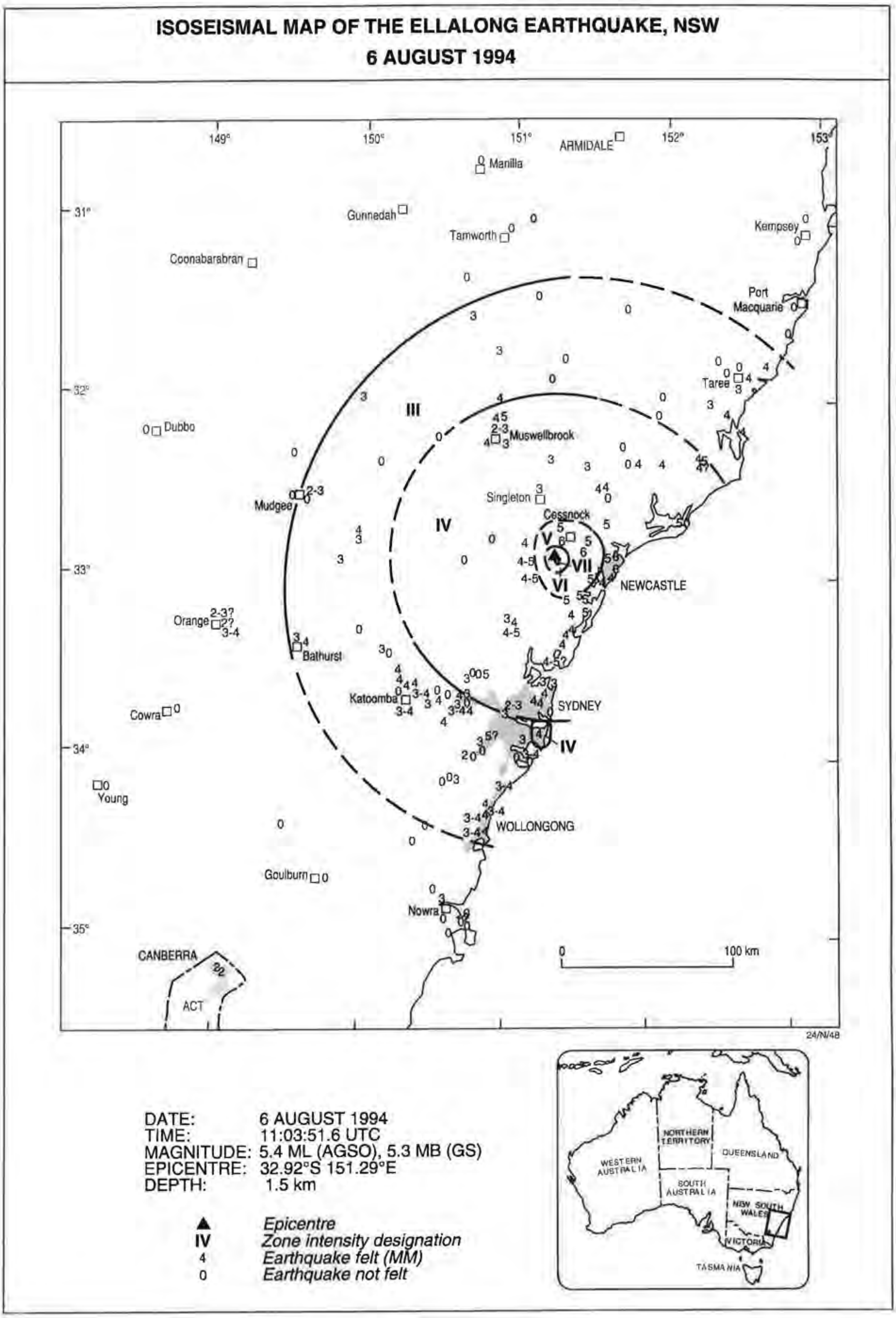

Figure 2 Isoseismal map of the Ellalong earthquake, 6 August 1994.

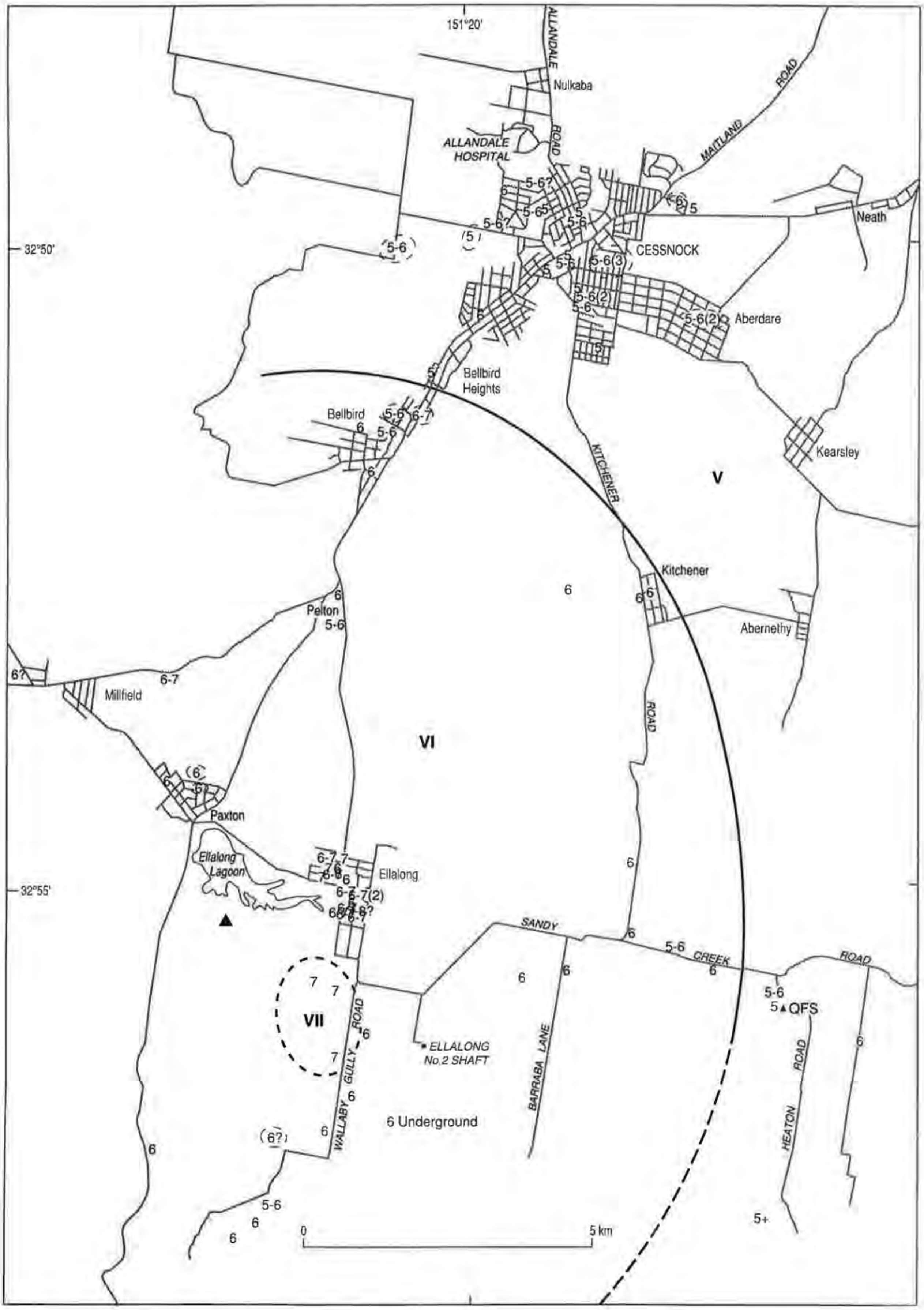

Figure 3 Isoseismal map for the Lower Hunter-Newcastle-Lake Macquarie area.

network of field seismographs installed after the earthquake. We comment on the relation of these small events to the mainshock and to the area of highest intensity.

Several important issues regarding this earthquake are not addressed in detail here. These issues include the relation of the earthquake to regional and local seismotectonics and to local mining, and the comprehensive strong ground motion recorded and its impact on estimates of Australian earthquake hazard.

This is the first significant Australian earthquake for which on-scale, triaxial, digital ground motion was recorded over sufficient distances to formulate useful attenuation relations - in this case the distances ranged from 39 to 319 km. These data have been analysed by McCue et al. (these Proceedings) and are mentioned only in passing in this work.

Earthquake lights were reported to have been observed during the earthquake. In a questionnaire distributed by authors WP and CB, Ms Pam Davis of Paxton made an unsolicited mention of earthquake lights observed by herself and a friend. The following is an abbreviated version of an account given by Ms Davis to Trevor Jones on 20 February 1995.

Ms Davis was attending a barbecue at Paxton (Figure 1) and was looking into bush to the southeast before the earthquake struck. Ms Davis saw a frightening red glow which appeared when the ground shaking began. She described the light as a 'big red ball' low to the ground, in fact probably on the ground, and between 100 m and 1 km in diameter. It was less bright than the moon, it glowed red, had no defined edges and appeared to be approaching her.

Ms Davis looked at the ball for a couple of seconds, by which time the ground shaking was severe. People were thrown off chairs and goods were falling off tables. She was forced to steady herself by putting her head on the table and when she looked up again, the lights had disappeared.

There have been many reports of strange lights accompanying earthquakes overseas but this is one of only a few such observations in Australia.

Felt Effects and Damage

The earthquake was felt over a radius of more than 220 km in eastern NSW; from Nabiac, Coopernook and Taree in the north to Nowra in the south and Orange in the west (Figure 2). The isoseismal maps (Figs 1, 2, 3) were prepared from about 360 reports. About 140 reports were gathered in the field by authors TJ, WP, CB and VW. About 200 additional reports were gathered from questionnaires distributed by AGSO within a few days of the earthquake, and from information obtained by telephone. Seventy six reports stated that the earthquake was not felt. Intensity evaluations for several data points of intensity MM IV were taken from an unpublished map prepared by J. Rynn.

The most distant felt reports were from Canberra, approximately 320 km from the epicentre, both were from AGSO staff who recognised the cause of the weak vibration. Reports in the media that the earthquake was felt at Albury at a distance of more than 400 km could not be substantiated by the authors.

Figures 1 to 3 show the isoseismal map and in more detail, intensities in the epicentral area and the Lower Hunter region. The isoseismals for MM V and MM VI are asymmetric, although they are reasonably well-constrained only in the east. We discuss possible reasons for this asymmetry below. Their average radii are low at about 19 km and 8 km respectively, as may be expected for a shallow earthquake. We took care to determine representative values for intensity in Newcastle, Cessnock and Ellalong where numerous reports were available. These average intensities were about MM IV-V, MM V-VI, and

MM VI-VII respectively. We also tried to eliminate the artificial increase in isoseismal radii that can occur through a bias in reporting only the higher intensities. For instance, at Beresfield (Figure 3) a sole intensity report, of MM V-VI, alludes to minor non-structural damage to a house which, to the authors' knowledge, was the only one damaged. Intensities in Newcastle ranged from not felt to MM VI, although the latter reports were rare. There is preliminary evidence of elevated intensities in Hamilton on alluvium compared to intensities at rock sites such as Merewether, confirming ground response effects devastatingly clear in the 1989 earthquake. Similarly, there is preliminary macroseismic evidence of amplification of ground motion by sediments in the eastern suburbs of Sydney and in Wollongong (Figure 2).

The earthquake was felt strongly at Ellalong, Paxton and nearby Wallaby Gully and the loss of electric power locally, causing immediate darkness, contributed to emotions described as 'general panic' by many local residents (see Maree Callaghan's report in this volume). In Ellalong village the intensities ranged from MM VI to MM VII-VIII and averaged MM VI-VII.

A maximum peak horizontal ground velocity of more than 160 mms^{-1} was recorded by a vibration monitor near Number 2 Shaft at Ellalong Colliery (Figure 1), within a few kilometres of the hypocentre, before the horizontal channels became overloaded. A maximum acceleration of 0.34 g was calculated for the vertical channel, which did not saturate. Strong shaking (pgv $\geq \sim 25$ mms^{-1}) continued for about one and three quarter seconds. These values are not corrected for instrumental response. The closest accelerograph in the Newcastle network, at a distance of 39 km, recorded a peak ground acceleration of 0.015 g.

Felt reports obtained from near the epicentre suggested strong ground shaking of short duration, high frequency, high accelerations and low displacements. Some reports contained information on the direction from which the seismic waves originated and this information was consistent, with some exceptions, although the reliability of such information is questionable.

Gerrard O'Leary of Ellalong gave a particularly lucid account. He reported a 'low (<10 Hz) rumble followed by very loud (thunder crack overhead) sharp "crack" then continued rumble for 8-10 seconds ... almost all damage done to objects on east/west oriented walls - nothing fell from any N/S oriented walls ... dominant vertical motion followed initial rumble; sharp, hammer blow like feeling from below. This blow was accompanied by very loud (>120 dB) "crack". Then commenced N-S dominant rocking/resonance of structure. At the time I was convinced the building would not survive motion we observed:- power was off immediately but we could still see well owing to radiant wood heater being well alight.'

Others reported strong north-south and vertical shaking at Ellalong. Andrew Struyf reported that most fallen objects in his house, including a large cupboard, had toppled in that direction. The east-west walls were not damaged. A. Meyn described that the ground motion 'was coming straight up' and her next door neighbour pointed obliquely down and south when asked by author TJ to indicate the source of the seismic waves. However, a report from Paxton indicated that the seismic waves 'came from SW'. The shaking at Ellalong indicated by these reports is consistent with P-waves and vertically-polarised S-waves radiating from a source below and to the south of Ellalong.

The rich high-frequency content of the seismic waves near the epicentre is indicated by the extremely loud sounds reported by local residents. For example, Colin Smith of Wallaby Gully reported 'unable to describe the sound ... shattering experience.' 'General panic' was reported by several respondents to the AGSO questionnaire, symptomatic of an intensity of MM VIII-IX although the lesser choice the questionnaire offers respondents is 'frightened all' (which indicates a minimum intensity of MM VI), a term with which almost all of the remainder agreed.

Robert Davies, one of 17 miners underground at Ellalong Colliery (Figure 1), reported 'a loud noise ... my whole surroundings shook for about 5 seconds - coal fell from the headings sides and dust immediately filled all the headings and cut throughs ... I suspected a gas explosion in nearby longwall excavations.'

Structural damage was remarkably minor, indicating that the shaking was short-lived and that considerable energy in the seismic waves was contained in frequencies above the natural frequency of the structures. Almost all buildings in Ellalong, Paxton and Wallaby Gully are single storey residences, with examples of most types of construction materials. Their ages range from new to possibly 80 years old and the condition of buildings is generally good except for some of the older houses. Of the few larger buildings, all of unreinforced masonry, the Ellalong and Bellbird hotels suffered considerable damage to the upper storey whilst the modern brick structure at the top of the Ellalong No. 2 Shaft (Figure 1) was undamaged.

In Ellalong and Wallaby Gully the brick skins of several houses moved laterally on their damp courses, up to about 25 mm, and this phenomenon was also observed in Newcastle in 1989. The chimneys at Ellalong Public School were demolished by NSW Public Works but there was no apparent damage to several tall brick chimneys with weak or missing mortar.

One of the authors (TJ) observed hairline cracking and spalling in the brick veneer of a modern steel-frame house which otherwise had no visible external damage. Several houses and the Ellalong Public School had moved laterally on their piers.

The authors observed mostly minor or no external damage to buildings. One notable exception was a mud brick house in Ellalong in which most walls had suffered shear damage. However, many houses had minor but extensive internal damage which is consistent with the average domestic insurance claim of $3500.

The dramatic, if short-lived effect on the population of Ellalong and Wallaby Gully implies an intensity of MM VII-VIII but a lower intensity, perhaps MM VI-VII, is ascribed by the damage to buildings.

Favourable geological conditions contributed to the limited damage. Soils in the meizoseismal area are shallow and firm. Commonly, south of Ellalong soils are alluvial and colluvial deposits with depth to bedrock in the range 1.0 to 1.2 m. North of Ellalong soils are derived from in situ weathered rock with depths to bedrock commonly 2^{+} m (Kovac and Lawrie, 1991). Quaternary alluvium is also found along stream beds (Figure 4) The highest intensities were observed on alluvium at Wallaby Gully (Figure 1), but the alluvium in this area may not have significantly amplified the intensity of shaking. An excavation for a dam inside the MM VII isoseismal showed about 3-5 m depth of weathered rock and firm soil over bedrock and a water table at least 5 m below the surface. The higher intensities in this area resulted from the radiation pattern of seismic shear waves and the relative proximity of fault rupture.

Location of Mainshock and Regional Seismicity

The earthquake was recorded by seismographs across Australia and beyond and an accurate epicentre was determined in less than one hour using data recorded mainly in the Newcastle and Sydney areas (see Table 1 for hypocentral parameters). The closest seismograph was the North Lambton station, at an epicentral distance of 39 km. Unfortunately, a closer seismograph at Quorrobolong Fire Station (QFS; Figure 4) about 9 km east of the epicentre was not operational at the time of the earthquake.

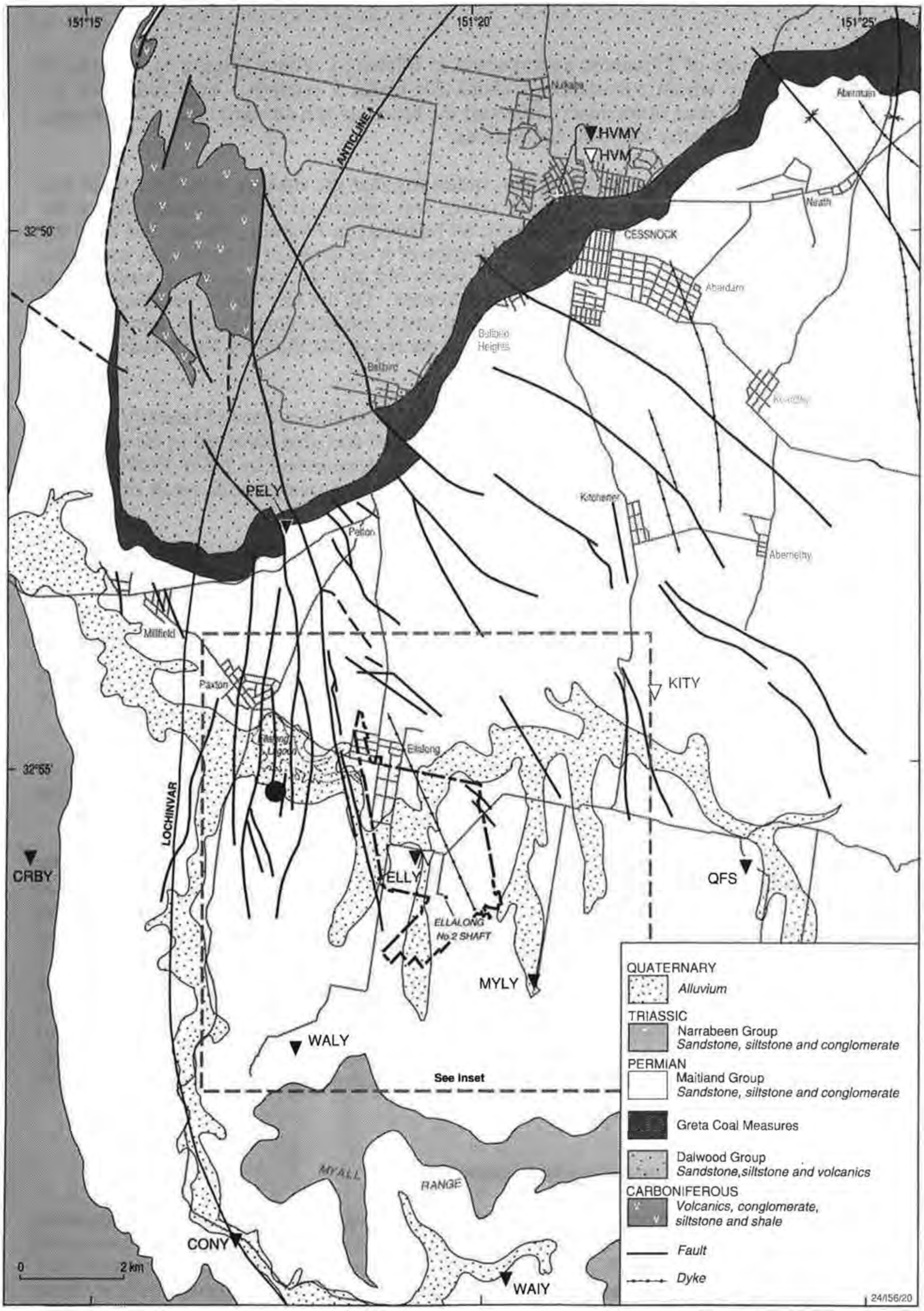

Figure 4 Simplified geological map of the Cessnock and Ellalong area (after Hawley et al., 1994). Extra fault locations are from Stevenson (written comm., 1994). The mainshock epicentre is shown by the filled circle. Locations of the field seismographs (closed triangles) and accelerographs (open triangles) are shown. Permanent seismograph QFS is also shown. The inset within the dashed lines is shown in Figure 7.

The preliminary computed hypocentral location was improved by a new preliminary velocity model (Table 2) which was compiled by one of the authors (GG). The uppermost layer represents Permian and Triassic Sydney Basin sediments, and estimates of their thickness in the Ellalong area, approximately 2 km, were provided by A. Brakel and D. Stevenson (written comms, 1994) and R. Rigby, (verbal comm., 1994). R. Rigby also provided estimates of near-surface P-wave velocities. The Sydney Basin sediments overlie Carboniferous sediments which in turn overlie crystalline rocks of the Lachlan Fold Belt down to the Moho at 41 km depth.

The computed uncertainty in the epicentre, within a 95% confidence ellipse, is about 1 km (Table 1). However, the position of the epicentre is dependent on the velocity model and the distribution of seismographs used in its location. The most important stations used to locate the event - those within 200 km of it - have azimuths ranging from 26° to 230° measured from the epicentre. If the velocities in the model are too high, then the epicentre will be pushed to the northwest, and if they are too low, the epicentre will be displaced to the southeast. We consider that a more realistic estimate of the uncertainty in the epicentre is 2 or 3 km.

The computed depth of the earthquake (1.4 ± 2.3 km) is strongly model-dependent but we consider our model fairly robust, indicating that rupture probably began either in the Permian sediments of the Sydney Basin or in Carboniferous sediments beneath them (A. Brakel, written comm., 1994;, D. Stevenson, written comm., 1994). The depth of this earthquake is clearly shallower than the 12 km deep focus of the 1989 earthquake (McCue et al., 1990) which occurred in crystalline rocks of the Lachlan Fold Belt.

Five historic earthquakes of magnitude ML 5 or greater are known to have occurred in the Hunter region (McCue in Rynn et al., 1987; McCue et al., 1990; Hunter, 1991; McCue (Compiler), 1995). These occurred in the Maitland-Newcastle area in 1842 (ML (I) 5.3), near East Maitland in 1868 (ML(I) 5.3), at Boolaroo in 1925 (ML 5.0) near Boolaroo in 1989 (ML 5.6) and at Ellalong in 1994 (ML 5.4). Each of these earthquakes caused damage and each was sufficiently well recorded in contemporary print media for isoseismal maps to be drawn and for magnitudes to be calculated using the empirical formula of McCue (1980). The 1989 Newcastle earthquake was the largest of these events.

Two seismic events of magnitudes ML 2.3 and ML 2.5 occurred near Ellalong in the week preceding the mainshock. These events were part of a sequence of hundreds of seismic events at Ellalong possibly dating back to the start of longwall mining at Ellalong Colliery. Seismic events are associated with all longwall mines. They are generated by the fall under gravity of the 'goaf', or roof left suspended after the longwall has extracted the coal from the mined seam. The seismic events at Ellalong colliery have been particularly energetic, perhaps because there is a competent sandstone layer above the coal seam. The largest event since December 1989 when the first stations in the Newcastle seismographic network were commissioned occurred on 5 April 1993 at 1648 UTC and had a magnitude of ML 3.3. The nature and relationship of these events to the 1994 earthquake is beyond the scope of this paper.

Geological Setting and Focal Mechanism

The earthquake occurred in the Hunter Thrust and Folds Zone which, trending southeast from about 4 km west of Muswellbrook to the coast, marks the northeastern margin of the Sydney Basin (Scheibner, 1993; Bembrick et al., 1973; Herbert and Helby, 1980). In this zone, low-angle thrusts were formed by the south and southwest movement of New England Fold Belt rocks over folded Sydney Basin sediments in the late Permian and Triassic (Roberts and Engel, 1987).

Figure 4 shows the simplified geology in the meizoseismal area, after Hawley et al., 1994. The dominant local structural feature is the Lochinvar Anticline, a Permian thrust-related dome whose arcuate axis trends approximately north-south near Paxton and northeast-southwest north of Cessnock.

Middle to Late Permian marine conglomerates, sandstones and siltstones of the Maitland Group occur in the epicentral area . Immediately south these are unconformably overlain by more resistant Early Triassic conglomerates, sandstones and siltstones of the Narrabeen Group which are evident in the Myall and Watagan Ranges. Quaternary gravels, sands silts and clays are found in limited areas along stream beds (Hawley et al., 1994).

West of the axis of the anticline the strata dip almost vertically and east of the axis the sediments have a gentle southeasterly dip. The Greta Coal Measures outcrop near Cessnock and are buried to a depth of about 500 m at Longwall 12 at Ellalong Colliery, the active longwall at the time of the earthquake.

There is detailed knowledge of near-surface faults in the epicentral area through geophysical and geological mapping both at the surface and underground (Figure 4). These faults are of Permian age, are extensional, and are associated with the formation of the Lochinvar Anticline (Russell Rigby, verbal comm., 1994). Near the epicentre most faults have north-south strikes and east of it, within 2 or 3 km, the strikes are northwest to north-northwest. Immediately west of Ellalong Colliery, two normal faults trending north-northwest have been mapped. These faults dip to the east at an unknown angle. A dyke trending northwest-southeast has been mapped in the colliery workings but not at the surface. Its dip is presumably near-vertical (Russell Rigby, verbal comm., 1994).

Focal mechanism A fault plane solution was prepared from P-wave polarities recorded at seismographs in eastern and central Australia (Figure 5). Focal parameters are listed in Table 3. The southwest-dipping nodal plane is constrained by an emergent, presumably nodal arrival at STKA (Broken Hill) and arrivals of opposing polarities at stations to the southwest of the epicentre. Similarly, the nodal plane dipping northwest is constrained by clear compressional arrivals at ARMA (Armidale), KIM (Merewether) and NPS (Newcastle CBD), and dilatational arrivals at BRS (Brisbane) and seismographs of the nearby Wivenhoe network (not plotted). However, neither nodal plane is well constrained.

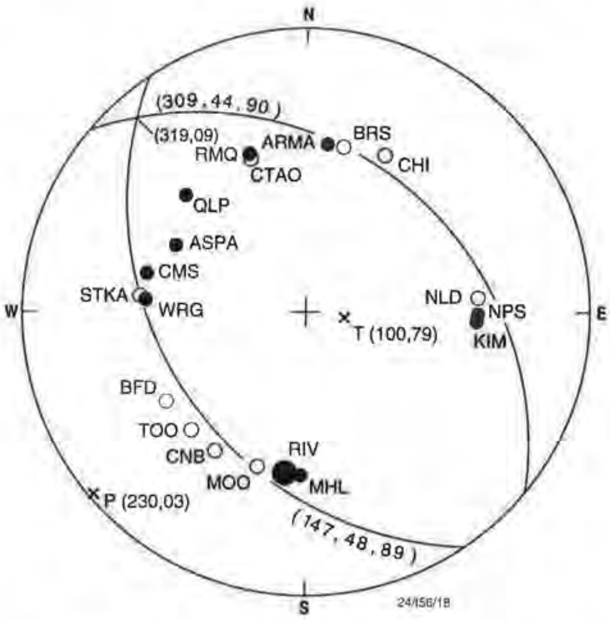

Figure 5 Fault plane solution for the mainshock. The lower hemisphere is plotted on an equal area projection. P-wave compressions are plotted as filled dots (small, short period; large, long period) and dilatations are shown as hollow dots. P and T represent the pressure and tension axes respectively.

The mechanism indicates almost pure reverse faulting on either of the two nodal planes which strike approximately northwest-southeast. The pressure axis is horizontal and trends southeast-northwest. The mechanism for this earthquake is similar to that of the 1989 Newcastle earthquake (McCue et al, 1990). There is good agreement also between the orientation of the principal stress direction for these two earthquakes and the maximum principal stress determined from crustal stress measurements made northeast of Ellalong (Enever and Wooltorton, 1981).

Field Network and Aftershocks

After the earthquake, seismologists from RMIT and AGSO travelled to the epicentral area where they installed a network of 12 portable seismographs and accelerographs to augment the permanent seismograph QFS in monitoring aftershock activity. The first station was installed 17 hours after the mainshock, and within 24 hours of the mainshock a sufficient number of seismographs were in place to locate any seismic events of magnitude ML 1.0 or more.

The field recorders were 'Kelunji' digital recorders designed and manufactured by RMIT recording in triggered mode at 200 samples per second, sensors were triaxial. The corrections of the recorders' internal clocks were checked using GPS time as a reference. This check was performed daily until 12 August and during this period seismograph timing was probably accurate within about 0.005 to 0.01 s. From 12 August to 19 August the timing was probably accurate to within about 0.03 s and from 26 August to 12 September the network timing was probably accurate to around 0.08 s. The locations of the field network were determined by GPS receiver to an accuracy of about 100 m. A history of the portable network is given in Figure 6.

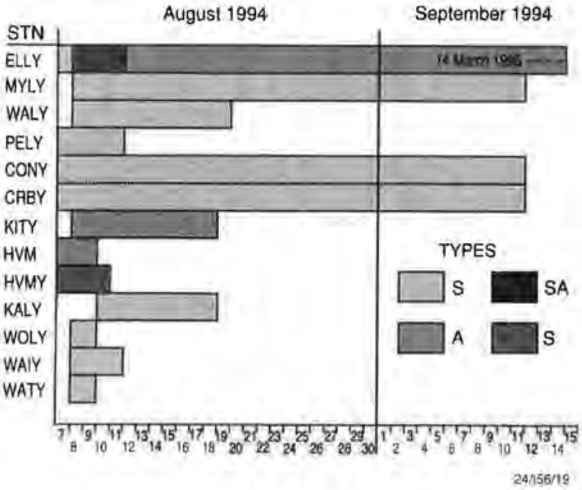

Figure 6 Histogram of the operation of the portable seismograph and accelerograph network. Station ELLY continued until 14 March 1995. Instruments were Kelunji triggered digital recorders made by RMIT. Types were S, triaxial seismograph; A, triaxial accelerograph; SA, both S and A. Station HVMY was a vertical component seismograph with a Sprengnether MEQ 800 analogue recorder.

Ellalong Colliery staff had walked all underground roads the morning after the earthquake and found them clear but a roof fall on the main conveyor belt in the drive on the northwestern side of the mine (Figure 7) occurred either during or shortly after an event at around 0207 UTC on 7 August (just after noon on Sunday, AEST). This event was felt at the mine. This fall prevented full production of coal until 15 August at about 11 pm AEST. Newcastle seismographs at North Lambton and Merewether, only about 40-45 km away, did not record this event so its magnitude must have been less than about ML 2.5.

The first event located using the five portable recorders at that time installed occurred at 0024 UTC on 8 August, about 37 hours after the mainshock. A total of 68 events was recorded by three or more seismographs in the network in 35 days of recording from 8 August to 12 September. The epicentres of the first 24 of these events to occur are shown in Figure 7. The hypocentres of the events were classified according to the computed

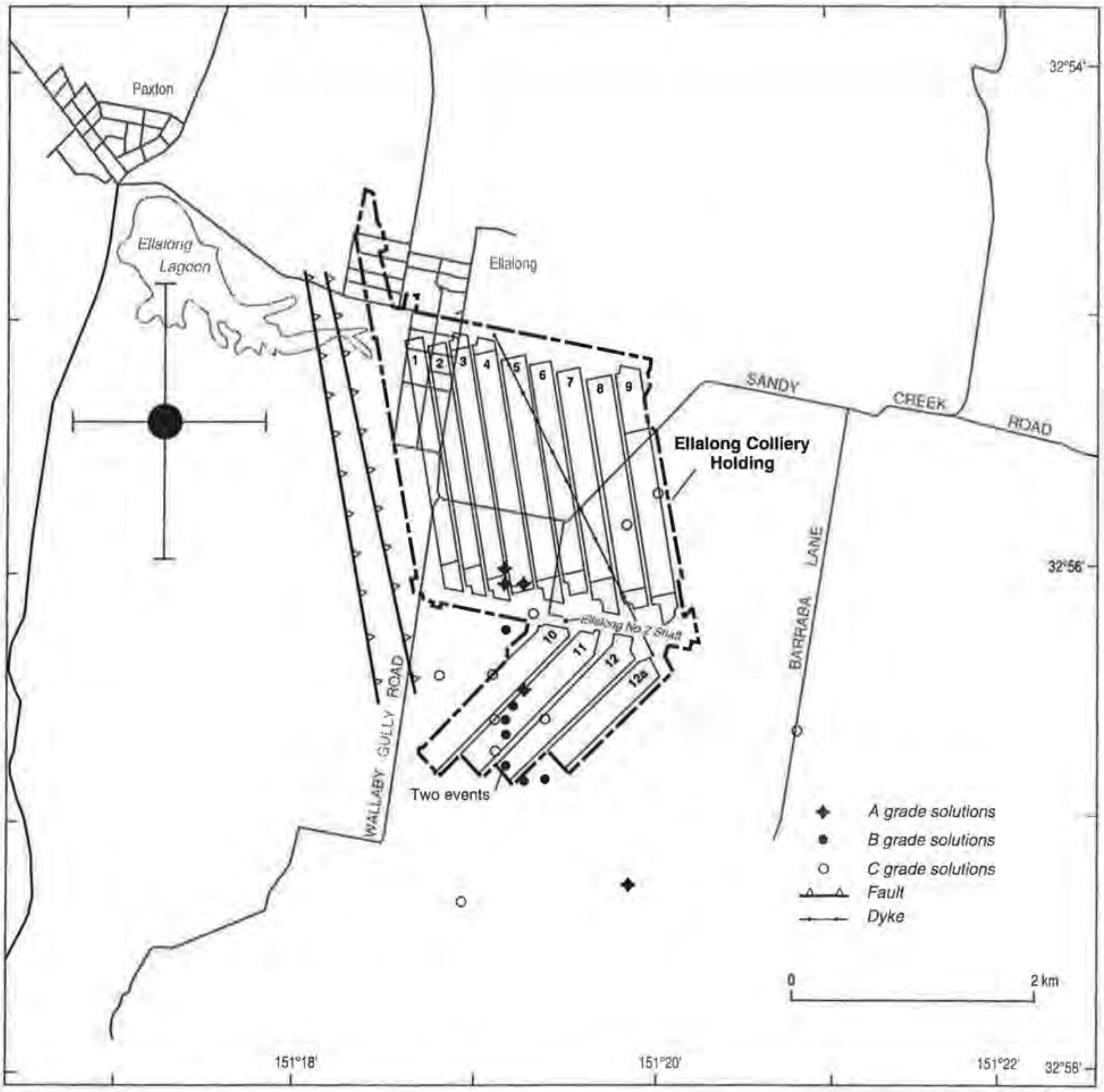

Figure 7 Inset map of the epicentral area. The symbol for the mainshock epicentre is a large filled circle with 95% error bars shown. Small circles and small circles with cross are epicentres of events located by the field network (see text for explanation). The outline of the workings of the Ellalong Colliery is shown and longwalls are numbered.

horizontal and vertical locational errors, which correspond approximately to the axes of the 95% confidence ellipsoid. Quality A solutions have computed locational errors less than 1 km in all three orthogonal directions and the best solutions have maximum computed errors of less than 250 m. Quality B solutions have errors of less than 1 km along two axes and an error of between 1 km and 2 km for the third axis. Quality C solutions have an error of less than 1 km in one direction or an error of 2 km or greater along one axis and a lesser error in the direction of the other two axes. The remaining solutions were assigned Quality D.

Magnitudes have not been calculated for the events because of their remarkably small size. The largest event recorded had a magnitude of ML 0.9. The absence of any sizeable aftershocks is remarkable but may be a feature of Hunter earthquakes. The only aftershock of the 1989 Newcastle earthquake in the week following the earthquake had a magnitude of ML 2.1 and occurred about 34 hours after the mainshock (McCue et al., 1990).

Uncertainties of several kilometres in the C and D Grade solutions for outlier events in Figure 7 leads us to downgrade the importance of these events in an analysis of the seismicity. What remains is a tight cluster of events aligned north or north-northwest. The maximum dimension of this pattern is about 2.6 km in a north-northwest direction, from the northernmost A or B Grade event to a single well-located event to the south-southeast.

Epicentres of the events are located about 4 to 5 km southeast of the computed epicentre of the mainshock, outside of the formal computed errors (Table 1; Figure 7). The events are about 1 to 3 km east of the small zone of highest intensities.

Twelve of the thirteen Grade A and B epicentres plot inside the boundary of Ellalong Colliery Holding, or are within about 200 m of it. The epicentres of the northernmost of these events are superposed on the central part of the mine, from which coal was extracted in the late 1980s. Epicentres of a further eight events with quality A or B are superposed on the southern, most recently-mined longwalls. Longwall 12 was active at the time of the mainshock. A single event with quality A is located about 1 km southeast of this group.

The events lie at indicated depths from 1.26 km to 1.79 km. This places them underneath the 0.5 km depth of the mine workings. However, we have not investigated the sensitivity of these depths to changes in the velocity model or to the geometry of the field array. The depths of the seismic events are in good agreement with the depth of 1.4 km determined for the mainshock.

The northernmost events are the deepest and there is a weak trend for shallowing towards the southwest. Not enough events have been located so far to produce cross sections showing any well defined faulting or clustering.

The first five events located occurred while the mine was inoperational due to the roof fall on the conveyor belt on Sunday 8 August (0200 UTC on 7 August). The last of these was on 8 August at 1621 UTC. The epicentres of these events are identified by a filled circle with cross in Figure 7. The next event recorded by three or more stations did not occur until nearly six days later, on 14 August at 0753 UTC. The conveyor had been cleared and full production began on Longwall 12 on 15 August at about 1300 UTC. From that time until the final stations in the network were dismantled, locatable events were detected almost every day.

Discussion

We first examine whether the events located using data from the field network may be blasts. Staff at the Ellalong colliery kept a log of shots fired, mainly at the face of Longwall 12A. The time of firing was logged for 20 shots fired between 9 August and 28 August. Only one of the shots was recorded by three or more seismographs, so we exclude the possibility that the events located by the field network were blasts, with one exception.

The exception is the event at 1632 UTC on 8 August, which was identified by colliery staff as a blast and which was located at the mine level using the field network (Figure 4).

The sensitivity of the seismographic network was demonstrated by its ability to detect events with magnitudes less than ML 1. All of the events located by the field array occurred within close proximity of Ellalong Colliery. Any other significant seismic events near the computed epicentre of the mainshock or elsewhere within the confines of the field array would also have been detected by the network, provided that the events were not deeper than about 15 km. The high intensities near the epicentre, the small radii of the MM VI and MM V isoseismals and the computed depth of the mainshock indicate that this earthquake was very shallow and so we think that any aftershocks should also have been shallow.

Therefore, we examine three scenarios:

The computed earthquake epicentre is reliable and the aftershocks did not occur on the same fault as the mainshock We think this scenario is unlikely and we can provide no explanation of how it could occur.

The computed earthquake epicentre is unreliable and the recorded aftershocks indicate the true location of the mainshock - that is, underneath Ellalong Colliery A plausible case for this scenario can be presented subject to our discussion above concerning the accuracy of the computed location.

The isoseismals are lobate to the east-northeast and it has been observed for other shallow Australian earthquakes that the hanging wall block is subject to higher shaking than the footwall block (eg Jones et al., 1991). Therefore the fault plane for the earthquake may dip approximately northeast.

Two faults to the west of Ellalong Colliery and a dyke that cuts through the mine workings have similar strikes to the nodal planes in the earthquake mechanism. They dip to the northeast although their dips and throws are unknown (R. Rigby, verbal. comm., 1994). We think it unlikely that the earthquake ruptured on a southern extension of one of these faults because they strike at about 349°, somewhat more northerly than the nodal planes of the earthquake mechanism. However, the earthquake rupture may have occurred on a subparallel, unmapped fault nearby.

The dimensions of this rupture may be suggested by the locations of the 'aftershocks'. The rupture length (maximum distance between epicentres) is about 2.6 km for a north-northwest strike and the fault width is estimated to be about 0.5 km (Figure 7). A resulting rupture area of about 1.3 km^2 is appropriate for an earthquake of this magnitude.

The zone of maximum intensity was adjacent to, and west and updip of this hypothetical fault rupture. The event may have ruptured upwards toward the surface and, in any case for the scenario proposed, a lobe of the S-wave radiation pattern would have been oriented updip, producing strong ground motions.

Such a scenario places the earthquake's epicentre closer to Ellalong than to Paxton and may explain why intensities were higher in Ellalong than at Paxton. The computed epicentre is about equidistant from both villages.

The computed earthquake epicentre is reliable and there were no recorded aftershocks This argument also is plausible. We have mentioned the almost complete lack of aftershocks from the 1989 Newcastle earthquake and the relatively low computed uncertainties in the earthquake's epicentre. If the fault plane dipped northeast then stronger shaking would be expected in the hanging wall block, that is, at Ellalong and Wallaby Gully.

In this case the seismicity recorded by the field network would be considered a sample of mine-related seismic activity which was present before the mainshock occurred and which presumably continues in association with coal extraction. The apparent increase in microseismic activity when mining recommenced on 15 August supports this argument.

Conclusions

The Ellalong earthquake was shallow and occurred either in Sydney Basin sediments or in Carboniferous sediments beneath the basin. It had a reverse faulting mechanism. The earthquake was the fifth moderate-magnitude (ML 5-5.6) earthquake in the Hunter region in the past 154 years.

Although there were two local seismic events in the week preceding the earthquake these cannot be labelled foreshocks with certainty. Like the 1989 Newcastle earthquake which occurred about 30 km away, the earthquake had few aftershocks and none of magnitude greater than about ML 2.5.

It is not clear whether the micro-seismic events occurring in the weeks after the earthquake were aftershocks. The computed epicentre of the earthquake is offset by about 3-5 km from the epicentres of later events and the probable rupture length of the earthquake is less than this. These seismic events were close to the Ellalong Colliery and, from limited evidence, their occurrence was temporally linked to mining activity. We therefore associate these events with longwall mining activity.

We have not presented enough information to determine any relationship between the earthquake and coal mining, past and present, in the area. However, because of the significant impact of the earthquake, because of the history of small magnitude seismic events at Ellalong, and because the possibility of a future damaging seismic event in the area cannot be excluded, the earthquake sequence should be the subject of further investigation, and the mine should be monitored with a permanent seismograph network.

Acknowledgements

We thank the staff of Ellalong Colliery, particularly Richard Porteous, the manager at the time of the earthquake, Russell Rigby and Brian McCowan, for providing information on local blasts, stress fields and geology. We thank Maree Callaghan, Mayor of Cessnock, and John McNaughton, Mayor of Newcastle, for offering the resources of their Councils. We thank Jack Rynn and Harold Stuart for unpublished macroseismic data and Albert Brakel who provided an unpublished geological cross section. We thank Dick Glen, Erwin Scheibner Darryl Stevenson for volunteering information on local geology and tectonics. Angela Bullock, Lesley Hodgson and Li Yue helped to prepare the isoseismal maps. Jill Clarke drew the figures.

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Table 1 Hypocentral parameters of the Ellalong earthquake

Date y m d	Time UTC h m s	Latitude °S	Longitude °E	Depth km	Magnitude
1994 08 06	11 03 51.6±0.5	32.924±1.02 km	151.288±0.94 km	1.4±2.3	5.4 ML

Table 2 Velocity model SYD3A used to locate Ellalong earthquakes

Depth to bottom of layer (km)	Vp (km s^{-1})	Vs (km s^{-1})	ρ (gm cm^{-3})	Qp	Qs
2.0	4.0	2.38	2.4	100	50
7	5.5	3.18	2.65	200	100
18	6.36	3.74	2.65	200	100
41	6.8	3.93	2.87	300	150
	8.04	4.62	3.35	500	250

Table 3 Focal mechanism parameters

	Strike (°)	Dip (°)	Slip (°)	Azimuth (°)	Plunge (°)
NP1	309	44	90		
NP2	147	48	89		
P-axis				230	03
T-axis				100	79
N-axis				319	09