DAMS AND EARTHQUAKES IN NEW ZEALAND

A. V. Hatrick*

ABSTRACT

Recently there has been increased interest in small hydro-electric schemes, many of which will include dams. Although there is no official New Zealand code for the design of dams the practice of the Ministry of Works and Development, for reasons which are explained, influences the design of most dams in this country. The paper describes the guide-lines currently used by the Ministry of Works and Development for the design of dams against earthquakes. Some background information is given on seismicity, history of design approaches, and experience of dams in earthquakes, all as they relate to New Zealand. Reference is made to some general aspects of dam design which help to set the anti-seismic guidelines in perspective.

INTRODUCTION

Recent developments in energy policy have created increased interest in small hydro-electric power schemes, many of which will include dams. There is in New Zealand no official design code for dams, but for various reasons their design has been, and will continue to be, influenced by the practice of the Ministry of Works and Development (MWD). The following description of the department's current practice, so far as it relates to the design of dams against earthquakes, will, it is hoped, be of interest to New Zealand engineers engaged in such work.

The paper is an expanded version of one called "Seismic Aspects of New Zealand Dams" which has been published in the ANCOLD Bulletin (1), and was presented at a technical session of the 16th ANCOLD Annual General Meeting and Study Tour of New Zealand in December 1976. The present paper has been updated, and some revisions have been made as a result of discussion of the earlier one. It has also been expanded to include some additional discussion of dam design in general, which should help to set the anti-seismic provisions in perspective. However, it does not pretend to deal with all aspects of MWD practice in relation to the design of dams.

SEISMICITY OF NEW ZEALAND

Although many hundreds of earthquakes are recorded each year in New Zealand most are too small to be of interest in connection with engineered structures.

Damage to dams may occur as a result of shallow earthquakes of Richter magnitude 6 or greater[2]. Figure 1 shows the occurrence of such earthquakes in New Zealand over the period of reliable record. Typically one can expect one magnitude 6 earthquake per year somewhere in the country, a magnitude 7 shock in 10 years and a magnitude 8 shock in 100 years. In the historical period only one earthquake (South West Wairarapa 1955) has reached magnitude 8[3].

The absence of large earthquakes in the north-west and south-east corners of the country in the period of record is in accordance with the geology[4] and can fairly be interpreted as indicating a lower risk in these regions than in the main tectonic zone. Such an interpretation is supported by a recently-published paper by Dr. W. D. Smith of the Geophysics Division, DSIR[5]. However smaller earthquakes do occur in these lower-risk regions, and the possibility of an isolated large one cannot be ignored.

The question of risk in connection with the selection of design parameters is discussed later.

EXPERIENCE OF NEW ZEALAND DAMS IN EARTHQUAKES

No significant damage is known to have occurred to any dam in New Zealand due to earthquakes, although it cannot be said that many have been put to the test. A number have experienced shakes of moderate intensity, but few have been exposed to a high intensity shock such as, for example, would have caused significant damage to conventional buildings nearby.

However six dams in the lower North Island were only about 50 kilometres away from the centre of one or other of the 1934 or 1942 earthquakes, (magnitudes 7.6 and 7.3 respectively), and they suffered no damage. Of these, four are concrete gravity dams around 30 m high and one earth-fill, 26 metres high.

In the 1968 Inangahua earthquake it is reported that at the Arnold Dam, some 100 km from the epicentre, the 15 metre high earth fill section was affected to the extent that a gap, some 30 mm wide, opened up between the concrete core-wall and the adjacent fill[6].

EARTHQUAKES ASSOCIATED WITH RESERVOIR FILLING

The association of increased seismic

* Senior Design Engineer, Power Division, Ministry of Works and Development, Wellington.

activity with the filling of large reservoirs has only come into prominence in recent years.

The largest reservoir so far created in New Zealand is Lake Benmore where, in 1964, the water level was raised 96 m, the added volume of water being 2×10^9 m^3. Although no obvious seismic effects were noted at the time of filling subsequent examination of the records has shown a significant increase in activity. The frequency of earthquake occurrence within 80 km of the dam has increased by a factor between 3 and 6, with activity concentrated near the dam and falling off with distance out to about 60 km. The largest earthquakes recorded in the post-impounding period were two of magnitude 5(7).

With the completion of the new Lake Pukaki dam the water level will be raised 38 m with an added storage volume of 5.5×10^9 m^3. The Geophysics Division of the Department of Scientific and Industrial Research has installed a network of seismographs to detect any changes in the level of seismic activity associated with the filling.

HISTORY OF DESIGN APPROACHES

The basis of design of many of the early dams built in New Zealand is now uncertain, but some research has been done on two owned by the State.

Arapuni Dam, completed in 1928, is a 64 m high gravity arch non-overflow dam. It appears that in its design no account of earthquakes was taken. However it was designed for an extreme static load of water up to crest-level so that with water at normal level it has a considerable margin for seismic loads. This, together with its arch shape, means that it has a capacity for seismic resistance comparable with the more recent dams.

Waitaki Dam, a 48 m high gravity dam with overflow and non-overflow sections, was completed in 1934 and it too was not designed against earthquakes. It has subsequently been strengthened by post-tensioned tie-downs and although this was primarily to provide for high water levels in floods it also provides a margin for earthquakes in combination with normal water levels.

A further twenty-three dams higher than 15 m were completed prior to 1938 and although information is not certain it seems unlikely that earthquake loads were specifically provided for in their design. Provision for earthquake loads on dams was not then common anywhere. For instance there is no mention of earthquakes in "Engineering for Masonry Dams" (1929) by W. P. Creagar, or "Earth Dam Projects" (1932) by J. D. Justin, both authoritative American texts in their day. Reference (8) records that in Japan the earliest research on the subject was in 1934.

There was a lull in dam construction during the war years and after 1938 the next large dam completed was Karapiro in 1947. It was designed for earthquake loading and it is likely that the same has been done for all large dams designed since then.

From the 1940's onwards the practice in the Ministry of Works and Development was to design for additional inertia forces acting on the dam body, and additional thrust from the reservoir acting on the upstream face, and to allow somewhat lower safety factors than for long-term loads. The inertia forces were calculated by applying a horizontal acceleration (usually 0.1 g) uniformly over the height of the dam. In the case of some (but not all) concrete dams a simultaneous vertical acceleration of 0.1 g was applied in the most unfavourable direction. The thrust from the reservoir was calculated by Westergaard's or Zangar's methods using the same horizontal acceleration as assumed for the dam. Current practice, which has evolved from these earlier approaches, is described in the next section.

ANTI-SEISMIC DESIGN OF DAMS - THE CURRENT SITUATION

There is in New Zealand no national code for the design and construction of dams. However it seems likely that as a result of the application of various pieces of legislation some degree of uniformity and minimum standard of practice is achieved.

The Ministry of Works and Development is the agency with most experience in dam engineering, and has been responsible for the design and construction of most of the larger dams. Although it has not produced any detailed code it has developed its own policies and guide-lines which are applied to dams designed by or for it. These have been, and still are based very largely on the policies of the major United States agencies such as the Bureau of Reclamation and the Corps of Engineers, on Japanese codes and, more recently, on publications of the International Commission on Large Dams (ICOLD). Those relating to aseismic design are outlined later.

In New Zealand any local government agency intending to carry out works involving the expenditure of loan money is required to obtain the consent of the Local Authorities Loans Board. Where a dam is concerned the Board refers the proposal to the Ministry of Works and Development for assessment of its technical soundness, and would not sanction one which did not meet the general standards that the department applies to its own work.

Another, and more recent, control is imposed by the Water and Soil Conservation Act 1967 which requires every person proposing to dam a river or stream to obtain a water right from the regional water board. Since the regional water boards act under delegated authority from the National Water and Soil Conservation Authority, and since the Authority's technical and administrative services are supplied by the Ministry of Works and Development, one can expect that the conditions of a water right for a dam would include ones ensuring that it met the general safety standards of MWD.

It can thus be seen that in one way or another the policies and guidelines adopted by the Ministry of Works and Development for the design of dams will become minimum standards for all dams of any significance in the country.

The guidelines relating to anti-seismic

design are set out in the next section, and are followed by a commentary.

MWD GUIDELINES FOR ANTI-SEISMIC DESIGN OF DAMS

(1) These provisions apply only to dams up to 45 m high. Higher dams shall be treated individually, considering such factors as dynamic response, interaction between dam and foundations, and dynamic strength properties of the materials.

(2) Earthquakes shall be assumed to occur in conjunction with the most critical commonly-occurring water levels, but need not be assumed to occur in conjunction with extreme floods or other infrequent events.

(3) Normally the external loads considered to be acting in combination are:

(a) Hydrostatic, earth and uplift pressures normally present.
(b) Hydrodynamic pressure.
(c) Inertia forces on the dam body.

(4) Hydrodynamic pressures are assumed to act normal to the upsteam face and may be calculated by any suitable method, eg Westergaard's or Zangar's. For pressures on vertical or near-vertical faces the following formula may be used:

$$p = \frac{5}{6} \, wk \sqrt{Hy}$$

where p = hydrodynamic pressure at depth y

w = unit weight of water

k = seismic coefficient

H = depth of reservoir at the dam

y = depth from surface to level considered

(5) The dynamic effect on silt (if present) may be allowed for by computing the hydrodynamic pressure for the full depth of the reservoir (including the depth of the silt) but otherwise ignoring the silt.

(6) Structures attached to the dam (eg spillway piers) shall be treated as structures founded on the ground and designed in accordance with NZS 4203[18] using the seismic coefficient for Zone A. The possibility of adjacent items colliding while vibrating shall be considered and methods (eg tying together or giving ample clearance) adopted to avoid it.

(7) Gravity Dams - Inertia Forces

The horizontal seismic coefficient is applied uniformly over the height of the dam. The coefficient shall be within the range 0.10 to 0.15. Conditions tending to justify low values are:

(a) Height less than 30m.
(b) Conditions producing abutment restraint and monolithic action such as small ratio of length to height, joints well keyed or grouted, and strong abutments.
(c) Sound rock foundations with a high elastic modulus.

Conditions tending to warrant a high value are:

(d) Height approaching 45 m or more.
(e) Large ratio of length to height, and absence of rigid abutments.
(f) Soft or closely-jointed foundations with a low elastic modulus.
(g) Concentration of mass near the crest.

(8) Gravity Dams - Stability Requirements

For normal static loads the resultant forces at any horizontal plane shall lie within the kern or "middle third" (ie no tension at the upsteam face). Under the combination of normal static loads and earthquake loads the resultant force shall lie within the structure and tensile stress at the upsteam face shall not exceed 0.02 f'c, where f'c is the compressive strength of the concrete.

The following sliding criteria shall be satisfied:

(a) Foundations of average New Zealand rock, eg greywacke or volcanics:

Under normal static loads $\frac{H}{V} \not> 0.70$

Under normal loads plus earthquake loads $\frac{H}{V} \not> 0.85$

(b) Non-rock foundations:

Under normal loads plus earthquake loads $\frac{H}{V} \not> 0.5 \tan \theta$

where H = net horizontal force

V = net vertical force

θ = angle of friction of foundation as found by test.

It is assumed that uplift does not change under seismic conditions.

(9) Arch Dams

When low dams are designed by approximate methods the following provisions may be applied:

(a) The horizontal design coefficient for the ground is selected in the range 0.10 to 0.15.
(b) The horizontal design coefficient for the dam shall be twice that for the ground. It is used to determine the inertia forces on the dam and the hydrodynamic force from the water.
(c) For the combination of normal static load and earthquake the allowable compressive stress may be increased 30% above that for normal static loads.
(d) For dams with pronounced overhang the effect of vertical accelerations should be considered.

(10) Earth and Rock-fill Dams

The horizontal seismic coefficient is applied uniformly over the height of the dam. The seismic coefficient shall be in the range 0.12 to 0.20. Conditions justifying the lower values are:

(a) Dams constructed mainly of rock-fill or compacted coarse gravel.
(b) Small ratios of length to height, and rigid abutments.
(c) Hard rock foundations.

Conditions tending to warrant the higher value are:

(d) Dams constructed of fine-grain material.

(e) Large ratios of length to height.
(f) Soft flexible foundations.

Under the normal static loads plus earthquake loads the slopes of the dam, when tested by slip-circle analysis, shall have a factor of safety not less than 1.5 for circles which cut the upsteam water surface and 1.25 for those which do not. In the analysis static values of strength criteria and pore-pressures shall be used.

Consideration shall be given to the ability of the core to remain essentially functional despite major dislocation. Downsteam of the core there shall be a filter of hard clean granular non-cohesive material and a drain of similar material. Steepening of the slopes near the crest shall be avoided. The crest width shall not be less than (0.1 H + 3) metres, where H = height of dam.

The freeboard above the maximum water-level expected to coincide with the earth-quake shall be (0.1 H + 2) metres, but need not be greater than 3 metres, except that consideration shall be given to the shock-waves from land-slides or rock-falls where such a hazard exists.

COMMENTARY ON THE GUIDELINES

This section contains a commentary on the guidelines, together with some additional observations on dam design in general, which will help to explain some of the anti-seismic provisions. The numbering of the paragraphs corresponds to the numbering of the clauses in the preceding section.

(1) The use of the guide-lines is limited to dams up to 45 m high. The definition of "height" would normally follow that used by the International Commission on Large Dams (ICOLD), viz. "from the lowest point of the general foundation to the crest". The need to provide such a limit should be evident; both safety and economy require that the design of a large dam be treated as an individual problem, with a solution worked out after considering all the local circum-stances, rather than the result of following standard codes and guidelines. On the other hand it should not be thought that a dam under 45 m in height is small. The International Commission on Large Dams defines a "large" dam as one more than 15 m high, and the US Bureau of Reclamation (USBR) takes the same view[9].

(2) The appropriate water level which is to be assumed to coincide with the design earthquake has to be decided by the designer, taking into account all the relevant circum-stances. Where a hydro-electric dam is normally held full, to provide head, it would be appropriate to assume the maximum normal water level, rather than any higher level occurring when discharging floods. On the other hand, a storage dam may only be full occasionally. But if it then stays full for any considerable length of time (say more than a week every few years) it might be appropriate to assume it to be either full or part-full when the design earthquake occurs.

Although high water levels will usually produce the critical conditions, design should be checked against other possible conditions, eg earthquake forces acting in an upstream direction with the reservoir empty.

(3) The designer should always review the possible combinations of loads, and decide which should be assumed as possible design conditions.

It is customary to assume that internal water pressures such as uplift in concrete dams, or pore pressures in earth fill dams, are not changed in earthquake conditions. But such an assumption is clearly not valid for materials which could undergo liquefaction. They could occur in foundations, or in the materials of some older earth fill dams. They should be avoided when selecting materials for new dams.

(4) The formula given is an approximate one, but adequate for dams up to 45 m high. (It is homogeneous and can be used with any consistent system of units.) The seismic coefficient to be used is that referred to in clauses 7, 9 and 10 of the guidelines. The hydrodynamic pressure on the sloping face of an earth fill dam is small, and MWD practice is usually to ignore it.

(5) This simplified treatment of the dynamic effect of silt is widely used.

(6) The main purpose of this clause is to provide protection for structures on the crest such as gates, hoists, cranes or lifts. The general provisions of NZ 4203 regarding structural clearances, non-ductile items etc. should be observed.

(7) Vertical accelerations are not usually considered for gravity dams of moderate height. Generally speaking such dams will have an almost-vertical upsteam face and a downsteam face whose slope is dictated by the seismic accelerations assumed. For any combination of assumed horizontal and vertical accelera-tions there is a horizontal acceleration which by itself will lead to the same end-product with less calculation. For example a combination of horizontal and vertical accelerations of 0.1 g each will lead to a similar dam cross-section as the assumption of a horizontal acceleration of 0.15 g with no vertical acceleration.

Since the selection and use of these pseudo-accelerations is merely a design convention there does not seem to be any point in complicating the convention unnecessarily.

(8), (9) and (10)

The selection of the seismic coefficient is in the hands of the designer, and reflects his feelings towards the risks, type of structure and foundation conditions. For example two recently built dams in the south-west corner of the South Island (Manapouri Lake Control and Te Anau Lake Control) were designed using a coefficient of 0.2. This was partly because the area is highly seismic (see Fig. 1), partly because the foundations were sands and silts, and partly because the main areas of concern were slope stability and dynamic earth pressures on retaining walls, which are not well understood.

(9) This guideline is derived from the code of the Japanese National Committee on

Large Dams and is only an outline. Arch dams should not be tackled by inexperienced designers, and the more experienced ones will consult the literature as well as refer to codes.

(10) MWD philosophy for the design of earth and rock fill dams places great stress on the careful selection of materials, and arrangement of the various zones, to ensure that even if damage leads to leakage through the dam the risk of progressive erosion is minimised. Observation of the behaviour of overseas dams in earthquakes suggests that the most probable form of serious damage is slumping or settlement of the crest, with the consequent risk of overtopping. Hence the emphasis on ample freeboard, and avoidance of steepening or narrowing the crest. In recent years researchers have focussed attention on design methods which use displacements rather than shear strength as the safety criterion. Reference (10) contains a recent account and provides an extensive list of further references. MWD is developing a computer program which will permit deformation analyses of this type to be undertaken. When it is available it will be assessed for use as an alternative to the pseudo-static methods in current use. The approach provides useful additional insight into the problem but in its present form it does not, in the writer's opinion, take sufficient account of the very complex displacements and accelerations which fill-type dams undergo in earthquakes. These have been demonstrated by Japanese researchers through dynamic model testing, and observations of dams in real earthquakes[11].

SEISMIC RISK

Most engineers in New Zealand accept that in the design of structures such as buildings or bridges it is appropriate to take into account differences in seismicity between different parts of the country. These differences are evident in the historic record, and can be accounted for in the framework of tectonic activity and history[4] [16]. However, our historical record is short, and it is known that in countries with longer records there have been periods of activity or quiescence lasting for centuries[4]. Considerable caution is needed in applying conclusions drawn from our historic record to the design of structures such as dams, which may have unpredictably long lives. Even our least seismic areas have experienced earthquakes of magnitude 5 in recent times [17], and presumably more severe ones are possible. No-one knows what the life of a dam will be. Once built they are usually difficult to demolish, and experience throughout the world is that societies do not demolish old dams unless the need is pressing.

All these considerations point to the need for caution before designing for lower levels of seismic resistance in dams in the apparently less seismic parts of the country. It is probably permissible for appurtenances not vital to the safety of the dam, and may perhaps be permissible in situations where failure of the dam could not lead to loss of life or severe damage to property. But how often is it possible to make such forecasts in the time-scale that must be used?

SOME GENERAL DESIGN MATTERS

Some indications of MWD practice in designing for other conditions than earthquakes have been given in guidelines (8) and (10).

The remarks in the commentary on guideline (10) about careful selection of materials, and arrangement of various zones, in earth and rock fill dams apply in all contexts. Particular attention must be paid to the contacts between the zones, and between the fill and the foundations or abutments. Compatibility of materials, and ample provision for transition zones and filters are important considertions.

In connection with guideline (8) it will be noted that no mention is made of the shear friction factor, a sliding criterion used by some overseas authorities. It requires knowledge of the cohesion and internal friction values for the foundation rock, and MWD experience with New Zealand rocks (which are typically very closely jointed) has been that their behaviour seldom conforms to such classical concepts. Normal practice is to treat rock as a non-cohesive frictional material, and determine the coefficient of friction by large-scale field tests which simulate the prototype loading conditions. However, some New Zealand rocks would be suitable for the application of the shear friction factor, and guidance can be found in the literature.

Many other matters must be taken into account to ensure the safety and continued function of the structure. These include preparation and treatment of the foundations; provision for drainage and seepage control, and subsequent inspection; and sound construction practices. All these are as important to seismic resistance as they are to more usual conditions. Most important of all is the provision of adequate spillway capacity; vastly more dams have failed from this cause than from earthquakes. Some useful texts dealing with general design matters references (9), (12), (13) and (14); references (11) and (15) provide some special background for seismic aspects.

CONCLUSION

It will be evident from what has been written that the design of dams against earthquakes is still very much an empirical art. The guidelines which have been listed are in accordance with current world practice, as described in References (2) and (8), and their application should produce designs which are adequately safe by such standards.

It should be borne in mind that the guidelines are intended only for dams of moderate height, normal proportions, and on foundations which have been fully investigated.

It should also be remembered that anti-seismic design is only one part of the larger task of designing a dam which will perform safely and well under all conditions.

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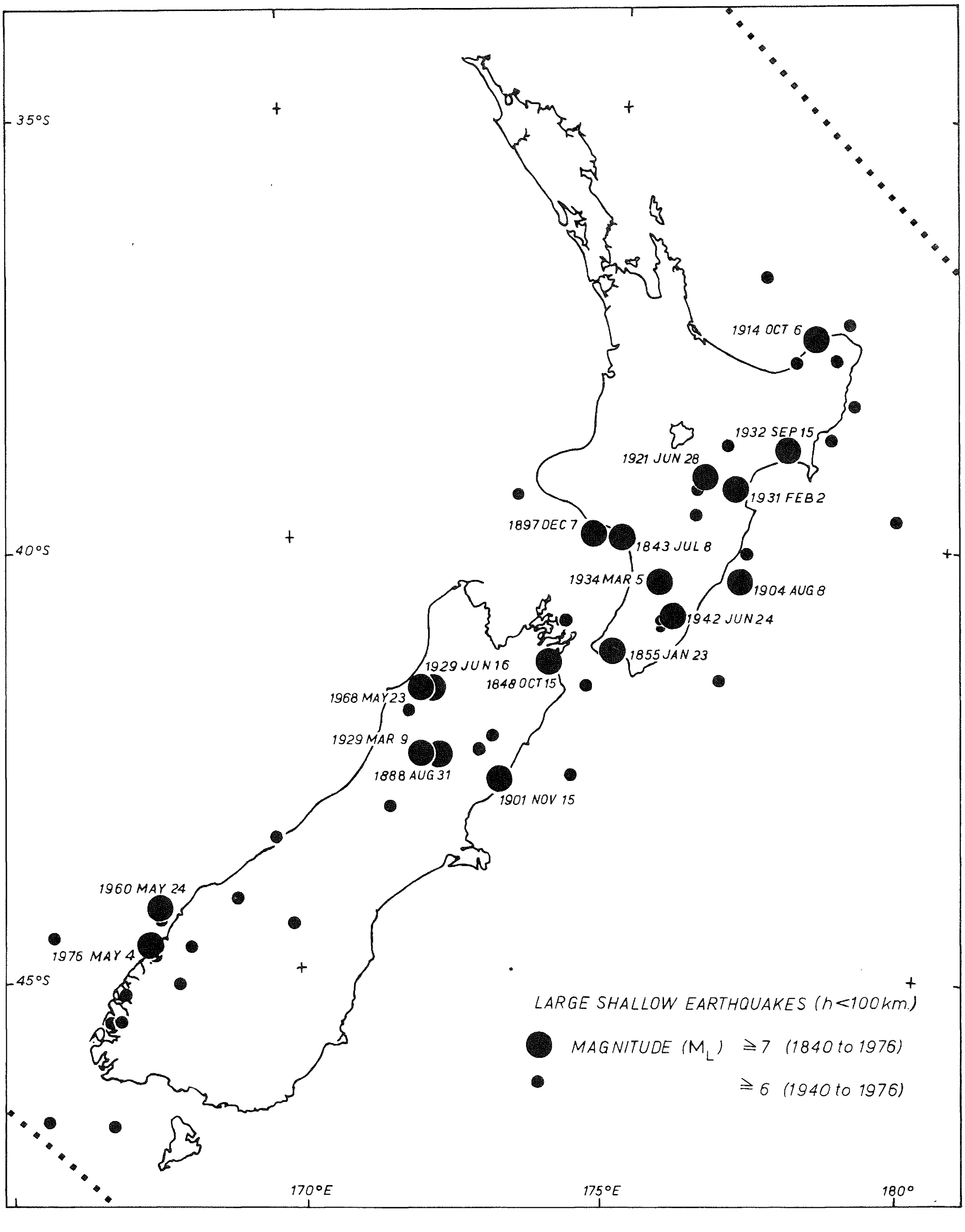

FIGURE 1: LARGE SHALLOW EARTHQUAKES IN NEW ZEALAND
(Supplied by the Seismological Observatory, Department of Scientific and Industrial Research, Wellington).