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TECHNICAL PAPERS

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SEISMIC RETROFITTING OF STATE HIGHWAY BRIDGES IN N.Z.

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1. INTRODUCTION

This paper summarises the scope for seismic retrofitting of state highway bridges, outlines current policy, and concludes with a description of the only major programme of seismic retrofitting undertaken to date in New Zealand. Some suggestions for future research are also included. This paper was originally presented at the Applied Technology Council Joint U.S./N.Z. Workshop on the "Seismic Resistance of Highway Bridges", held in San Diego on May 8-10, 1985, and attended by a team of N.Z. bridge design engineers and researchers.

2. SCOPE FOR SEISMIC RETROFITTING N.Z. STATE HIGHWAY BRIDGES

New Zealand has experienced many major earthquakes since first European settlement in the early 1800's. However, it was not until the Napier Earthquake in February 1931, which caused 265 fatalities, that any significant advancements were made towards improving the earthquake resistance of bridges.

Of the 2600 bridges on state highways, 15% of the total were built prior to 1932 and predate the earliest attempts to design and detail bridges for earthquakes. Although many of these bridges would sustain extensive damage even in a modest earthquake, most are either reaching the end of their economic life or are functionally obsolete and are therefore likely to be replaced within the next 25 years. Seismic retrofitting in most of these bridges would not seem to be economically viable.

Designing for seismic resistance became accepted practice in the 1930's following the Napier Earthquake. This period coincided with a rapid development of the roading system in N.Z. which saw many of the early timber bridges replaced with modern structures in reinforced concrete. Most of these replacement bridges were based on a range of standard designs developed by the Public Works Department (PWD), later to become the Ministry of Works, (MOW). The principal features of these bridges are simply supported tee-beam spans, integral abutments, reinforced concrete hinge bearings at intermediate slab piers, and vertical precast concrete piles. These features provided strong positive connections between components, with the parts most vulnerable to earthquake damage being the relatively weak vertical piles and the slab piers which lack confining steel.

Designing for seismic resistance became a mandatory requirement in 1944 when the P.W.D. issued a preliminary code of practice for highway bridging. This was followed in 1956 by the publication of the M.O.W. Bridge Design Manual. Both Documents, largely based on the AASHO Specifications, included a requirement that bridges be able to resist an earthquake design load, equivalent to 10% of the superstructure weight applied as a horizontal static force. This load was to be resisted without exceeding 133% of normal allowable working stresses. This design approach was simple to apply and ensured bridges could withstand moderate earthquakes with minimal damage, as observed following the Inangahua earthquake (magnitude 7) of May 1968. However, 71% of all present state highway bridges built between 1932 and 1972 using this design approach usually have limited ductility. They are therefore likely to sustain extensive damage, particularly to piers and foundation piles, during a major earthquake. Strengthening of foundation piles would in most cases be very costly because of difficult access. Nevertheless, there is obviously considerable scope of carry out cost effective seismic retrofitting on a selective number of the most important bridges built during this period.

The Highway Bridge Design Brief, first published by the MOW in 1971, with subsequent revisions issued in 1972, 1973 and 1978, adopted a design approach which required bridges to be designed and detailed to ensure predictable post elastic behaviour, with energy dissipated either in piers capable of ductile yielding or in specially developed mechanical energy dissipating devices. The 14% of state highway bridges designed since 1971 should therefore remain servicable after a major earthquake, with damage limited to areas which are readily accessible and relatively easily repaired. It would be desirable for all strategically important bridges in the state highway system to be upgraded to this standard.

3. CURRENT NRB POLICY

The National Roads Board (N.R.B.) provides funds for all state highway bridging, and also

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contributes a substantial subsidy towards the cost of all local authority bridging.

It is currently NRB policy to give priority to the upgrading or replacement of bridges to provide full legal (Class I) load carrying capacity, with the proposals being assessed on an economic basis. Consequently the seismic retrofitting of bridges has tended not to be given a high priority by the NRB and there is no national programme to bring either state highway or local authority bridges up to current seismic code standards. Proposals for retrofitting are considered by the NRB on a case by case basis. The only significant seismic retrofitting undertaken by the NRB to date involves eight bridges on State Highway 35 in the East Cape region of the North Island. The work is being carried out as part of a comprehensive structural upgrading involving concrete deck replacement and girder strengthening. A description of this seismic retrofitting work follows.

4. BACKGROUND TO RECENT SEISMIC RETROFITTING OF EIGHT BRIDGES ON SH35

All eight bridges, which were built between 1956 and 1962, consist of a conventional arrangement of steel plate girders composite with a reinforced concrete deck slab and supported on reinforced concrete piers and abutments. The bridges were designed to the seismic design requirements of the 1956 Bridge Design Manual. Design live loading was H20-S16. A summary of span arrangements is given in table 1.

TABLE 1

SUMMARY OF SPAN ARRANGEMENTS

Bridge	Span Arrangement
Hikuwai 1 & 2	100.5,101.0,100.5 (ft)
Hikuwai 3	60.5, 91.0, 90.5
Hikuwai 4	90.5, 91.0, 90.5
Makatote	70.5,71.0,71.0,70.5
Makarika 2	50.5,51.0,51.5
Kopuaroa 1	45.5,45.5
Kopuaroa 4	45.5,46.0,46.0,45.5

Figure 1 below shows the general arrangement of Hikuwai 2, which has tall slender open frame type piers typical of all four Hikuwai Bridges. Seismic loads in the longitudinal direction are designed to be resisted at both abutments by an arrangement of eleven 16 inch octagonal reinforced concrete piles rakes at 1:3. Transverse seismic design loads are designed to be resisted at all supports.

Figure 2 below shows the general arrangement of Makarika 2 which has low slab type piers typical of the remaining four bridges. The distribution of seismic design loads is the same as that in Hikuwai 2 bridge.

Extensive cracking was observed in all bridge decks soon after construction. The cause of the premature failure of these concrete decks has been attributed to a number of factors including poor mix design and the use of smooth polished course aggregate.

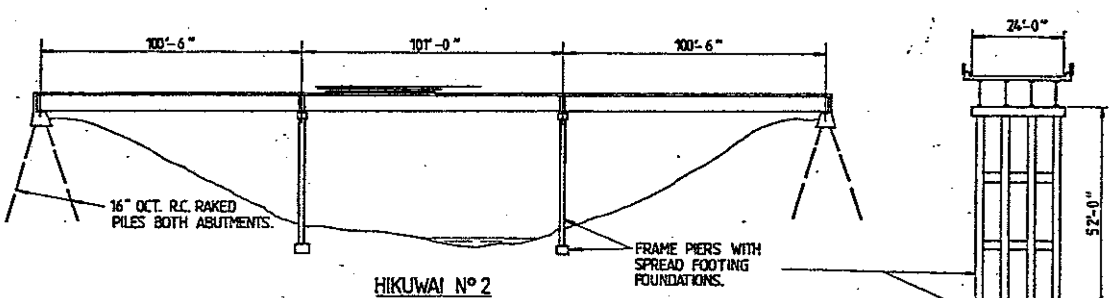
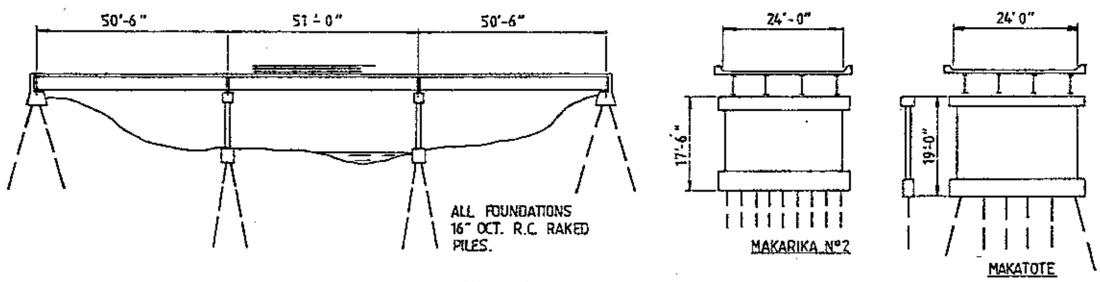


FIGURE 1 - GENERAL ARRANGEMENT TYPICAL OF HIKUWAI BRIDGES WITH FRAME PIERS



MAKARIKA N° 2

FIGURE 2 - GENERAL ARRANGEMENT TYPICAL OF SLAB PIER BRIDGES

The decks continued to deteriorate with use, with a substantial reduction in stiffness and progressive spalling of cover concrete from the underside of the decks being observed over the years. After a vehicle punched a hole through the deck on Hikuwai 1 in 1980 it was decided too replace the decks on all eight bridges. The girders required strengthening to carry the thicker replacement deck and current design live loads.

5. METHOD OF SEISMIC RETROFITTING

Analysis of the original raked piled abutments confirmed that they had considerable capacity to resist a high level of lateral loading without distress. On the other hand the piers, particularly the open frame type, were found to be lightly reinforced with little or no confining reinforcement in areas of potential plastic hinges. Studies showed that these bridges could be retrofitted for a small incremental cost, by isolating the superstructure with flexible elastomeric bearings and incorporating energy dissipators at the abutments, providing the work was carried out at the same time as redecking and girder strengthening.

The typical structural arrangement adopted for retrofitting on all eight bridges is shown in Figure 3 below. The typical original and retrofitted details at the abutments and piers are shown figures 4 and 5 respectively. Note deck slab elements are linked together over the piers to enable the deck to act as a horizontal diaphragm spanning between abutments.

Seismic retrofitting design loads for the four Hikuwai bridges were determined using the design charts developed by Blakeley in reference 1. Lead/rubber energy dissipators were found to be suitable and were designed in accordance with reference 2.

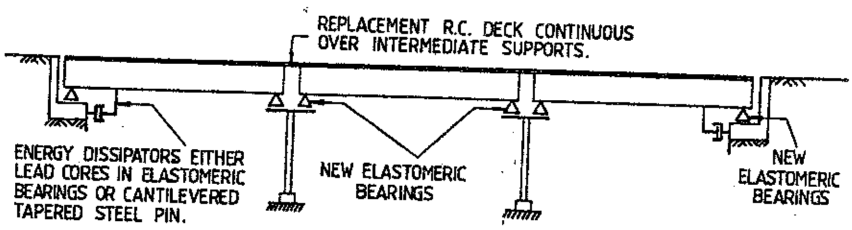


FIGURE 3 - TYPICAL STRUCTURAL ARRANGEMENT OF RETROFITTED BRIDGES

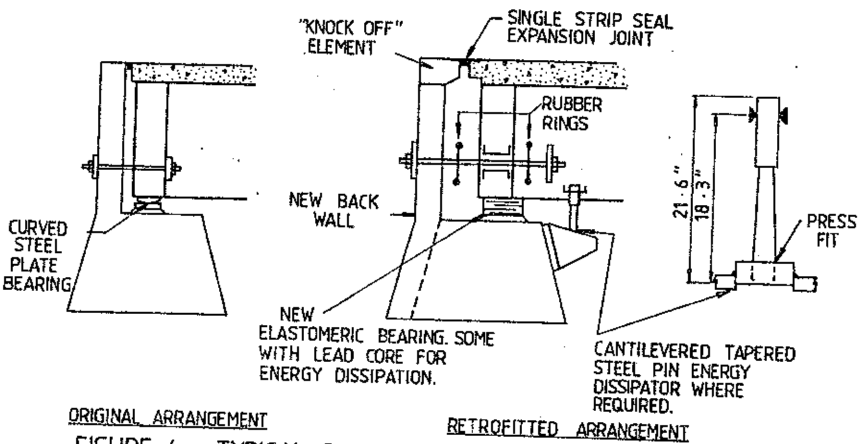


FIGURE 4 - TYPICAL DETAILS AT ABUTMENTS

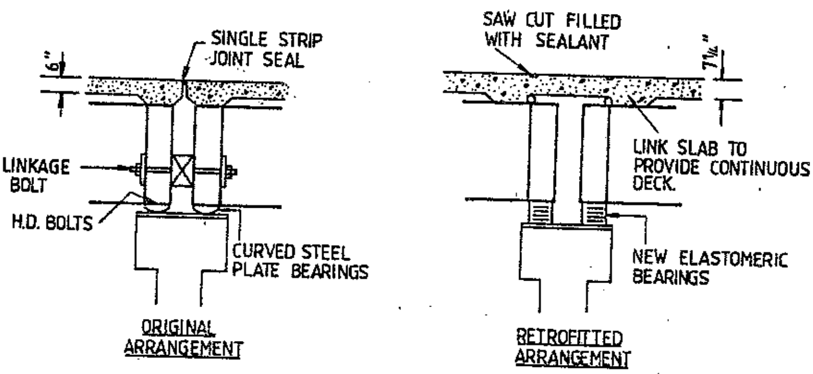


FIGURE 5 - TYPICAL DETAILS AT PIERS

However, for the remaining bridges with slab piers which have a substructure mass of the same order as the superstructure a specific dynamic analysis was carried out as recommended by Blakeley. The FEPLANE programme, which was developed at Berkeley, was used for the computer modelling and to establish design loads for retrofitting. Lead/rubber bearing energy dissipators were not considered suitable for Makarika, or for Kopuaroa 1 and 4, as their dead load is not sufficient to confine the lead core. Cantilever tapered steel pin energy dissipators, designed using reference 3, were therefore used instead.

The retrofitted structures were analysed for both the corrected El Centro 1940 Accelerogram (magnitude 6.3), and also the Artificial B1 shaking (magnitude 7.0). Maximum horizontal shear deflection of the elastomeric bearings for all bridges were about 2.5 to 3 inches for El Centro shaking and 4 to 4.5 inches for B1 shaking. All substructures have sufficient "dependable strength" to resist El Centro and sufficient "probable strength" to resist B1 shaking. The retrofitted structures can therefore be expected to sustain only minimal damage in a severe (MM8-9) earthquake, whereas before retrofitting they were likely to have sustained extensive damage in a modest (MM7-7.5) earthquake.

6. SUGGESTIONS FOR FURTHER RESEARCH

(a) Study of the technical feasibility and economic evaluation of seismic retrofitting typical standard designed R.C. tee beam bridges built throughout the country in the 1930's and 40's.

(b) Further development of compact energy absorbing devices and techniques for installation into existing structures.

(c) Further development of lead/rubber bearing energy dissipators to increase efficiency at low levels of axial load.

(d) Development of simplified methods for initial evaluation of the earthquake resistance of existing bridges and the economic benefits of retrofitting.

The permission of the Commissioner of works to present this paper is acknowledged with thanks.

7. REFERENCES

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