Design of highway structures for seismic forces in Turkey

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Introduction

The Ankara-Gerede Motorway forms part of the new toll road between Istanbul, Turkey's largest city, and the Capital, Ankara. The northern end of the road crosses the North Anatolian Fault near the town of Gerede about 130 km from Ankara. This fault line separates the Eurasian and Arabian plates and has been subject to strong seismic activity, see Fig. 1 (Lomnitz and Rosenblueth, 1976) and Fig. 2 and 3 (Allen, 1976). The structural and highway design for this road was carried out between 1987 and 1990. The road passes through extremely rugged terrain, requiring cuts up to 70 metres deep, and structures up to 90 metres high. In addition to numerous two lane bridges carrying local roads over and under the motorway several major structures were required to carry the motorway over deep glacial valleys and river flood plains. The largest structure in the area of highest seismic activity is the Markusa Dere Bridge, which is 560 metres long and 45 metres high. Seismic forces controlled all aspects of the design, requiring enormous spread footings and very heavily reinforced columns.

In this case study I intend to outline the design procedures used on this project, to examine the details used to ensure earthquake resistant structures, and to discuss the extent to which earthquake resistant design and details should be employed in Australia.

The performance of highway bridges is of vital importance in a severe earthquake, both because of the likelihood of loss of life in the event of a bridge collapse, and because of the importance of the road system to rescue and medical teams immediately after the earthquake. The prime aim of this study is therefore to examine the design procedures and details required to ensure that a structure will remain trafficable, although not necessarily undamaged, after a severe earthquake.

The AASHTO Code for seismic design of highway bridges

The motorway structures were designed to the AASHTO design rules, with some minor project specific amendments, and seismic design followed the AASHTO Code for Seismic Design of Highway Bridges (SDHB), which was then in draft form. This document had been developed following the disastrous San Fernando Earthquake of 1971, and was strongly influenced by the Applied Technology Council Guidelines (1981).

The SDHB has three design procedures of increasing complexity, depending on the importance of the structure and the severity of the design earthquake. The procedure described below is for Category C (see Fig. 4), which applied to all structures in the highest seismic zone near Gerede, and to large or irregular structures in the lower risk area.

Seismic acceleration

The ground acceleration used in design was derived from a report prepared by the University of Ankara. Structures were designed so that no damage would occur under a `50 year' earthquake, and damage under a `350 year' earthquake would be limited to removal of knock off pieces at abutments, and spalling at column hinges. Deck restraining members, and inaccessible members such as piles were designed with a

reserve of strength, so that plastic behaviour would be limited to suitably detailed and accessible members.

Preliminary Design

Preliminary design was carried out with spreadsheets developed on the project using a single mode spectral analysis (see Fig. 4). In the longitudinal direction the results of the single mode analysis were within 5% of those given by the more rigorous analysis, except for one highly skew structure with very wide blade piers. Results in the transverse direction were more variable, but were within 10% for regular structures. Output from the spreadsheet used for longitudinal analysis is shown in Fig. 5. The spreadsheets were invaluable for preliminary design, allowing the effect of varying column dimensions or restraint conditions to be assessed instantly, compared with 30 to 60 minutes for a full multi-mode analysis.

Structures crossing river flood plains were invariably subject to poor foundation conditions, with up to 30 metres depth of alluvium. Bored piles of 1.65 metres diameter were used throughout the project, with horizontal forces being resisted by vertical piles. The use of vertical rather than raked piled foundations increased the flexibility of structures with short stiff columns by a significant proportion. Assessment of the stiffness of the pile groups thus became a significant factor in the design. This was carried out with the computer program COM624, which analyses pile deflections and loads under lateral loads, using soil P-Y curves developed by Matlock (1970) and Reese (1974, 1975). Spreadsheets were developed as pre and post-processors to simplify input and to analyse and graph the results of the program. Typical output is shown in Figs. 6 and 7. Because of the sensitivity of the seismic analysis to the foundation stiffness used the stiffness analysis was carried out under a range of assumptions to find lower and upper bound stiffness values.

Multi-mode Spectral Analysis

The final seismic analysis was carried out on a Prime mini computer using the program Seisab supplied by Roy Imbsen and Associates, who also carried out a review of seismic design procedures and details. The program is specifically intended for the design of highway bridges, and is written to allow standard bridge types to be modelled with a minimum of input data. The program produces seismic elastic deflections and forces found by complete quadratic combination of a specified number of modes of vibration. Forces are output for earthquake forces in the longitudinal and transverse directions, and also combined as required by AASHTO.

Column Design

Column elastic forces are divided by a response modification factor, which varies between two and five, depending on the ductile capacity of the column, and the degree of redundancy at the support. In order to take advantage of the lower forces the bottom of the column (and the top for a built in column) must be detailed to act as a plastic hinge. Lap splices are not permitted and closely spaced transverse confinement reinforcement is required. This must be detailed to remain effective after the cover to the reinforcement has spalled. In addition the moment capacity of the column under axial is greatly reduced. A capacity reduction factor is applied to the moment capacity, varying from 0.9 at zero axial load to 0.5 at an axial stress of 0.2 f'c. An interaction diagram comparing the moment capacity of a column under AASHTO and SDHB rules is shown in Fig. 8.

In addition to the requirements of the SDHB the moment-curvature relationship was found for each column, and the rotational capacity of the column hinges was checked. In all cases it was found that the columns had a large reserve rotational capacity after absorbing the calculated elastic seismic forces.

An important consideration in detailing column reinforcement is ensuring that hinging will occur at the base, where confinement reinforcement is provided, and not at a weak point higher up the column. Design and detailing the column reinforcement for the high viaducts was a lengthy iterative process, involving the designers, the construction contractor, the client, and the checking engineer. It required the balance of conflicting requirements for strength, ductility, ease of construction, and efficient use of materials. Once again spreadsheets generated on the project were invaluable for the rapid assessment of design alternatives, and for column design with the automated preparation of reinforcement schedules.

Foundations and Seismic Restraints

Foundations and seismic restraints are designed for the maximum moment and shear force that can be developed by the column plastic hinge, multiplied by an overstrength factor of 1.3. Due to the application of the overstress factor foundations and piles will not be stressed to their ultimate capacity, even under an earthquake greater than the design event. Nonetheless closely spaced confinement reinforcement is required to the head of piles, because of the danger of brittle behaviour under high axial loads. In spite of the reduced design loads very large footings are required for high bridges subject to seismic loading. At the Markusa Dere Bridge footings of up to 1500 cubic metres were required, supporting 8 metre by 3 metre columns with up to six layers of 36 mm diameter reinforcement.

Seismic Details

Having designed the columns, foundations and deck restraints to resist seismic loads the remaining concerns are to ensure that:

sliding supports are wide enough to cater for the maximum deck movement

bearings and expansion joints have sufficient movement range for moderate earthquakes

restraint forces can be transmitted through the deck without distress

impact damage at abutments will be restricted to small accessible areas that can be easily repaired

Details used on the Ankara-Gerede Motorway

Inadequate deck restraint is one of the most common causes of failure of bridges under earthquake loading; properly designed restraints are thus of vital importance. Preliminary designs used steel dowels to restrain the deck; however it was found that as seismic forces increased it became very difficult to provide the required capacity with this type of connection. The detail ultimately used throughout the job was concrete restraints cast against the pre-cast beams, with a 10 mm thick joint filler to absorb thermal movements. The restraints were designed and detailed as corbels, and required quite heavy reinforcement at the northern end of the project.

The longitudinal and transverse restraints were poured against the precast beams, followed by a link slab providing continuity for longitudinal forces.

Thermal and shrinkage movements were accommodated by the piers in bending. In the case of the longer bridges the short piers furthest from the centre of the bridge, which would have been subject to very high forces, were provided with sliding bearings, and only restrained in the transverse direction.

At the abutments expansion joints were sized to accommodate a `50 year` earthquake, with a larger gap between the beams and curtain wall accommodating a `350 year' earthquake. The expansion joints were supported on a lightly reinforced knock off piece, to avoid damage to the main abutment wall under impact.

Summary and Conclusions

The details required for seismic resistance are summarised in Fig. 9. Many of the basic requirements can be provided at very little cost, and may produce benefits other than safety under seismic loads, such as increased resistance to accidental impact. Designing for full resistance to high seismic forces on the other hand will add considerably to the cost of bridges. Substructure costs for a large bridge may be two to three times higher than for a conventional structure. A decision on the appropriate level of seismic design forces for a `lifeline' system such as a highway is a complex issue involving both an assessment of the risks to the users of the individual elements of the system, and the effect on the system as a whole of a failure of one part. The difficulty is increased in Australia by a shortage of reliable seismic data. However the difficulty of the decision is matched by the high potential cost of the failure of the system. In Japan a holistic approach is adopted to planning for and mitigation of the effects of earthquakes, incorporating all levels of government and the private sector (Schiff et al, 1984). A similar approach is essential in Australia to obtain the greatest benefit from limited resources available for earthquake research, planning and design.

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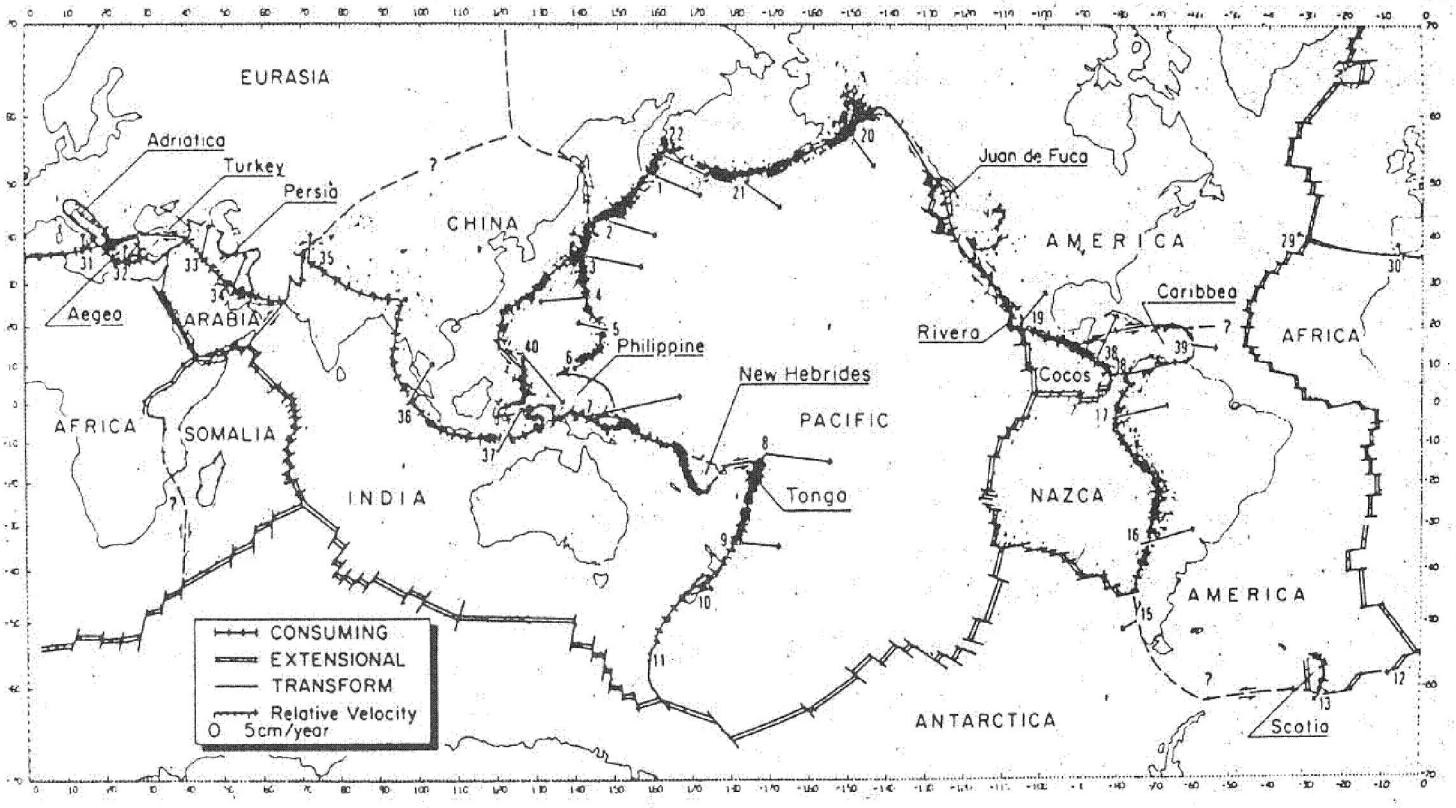

Present plate kinematic pattern on earth's surface. Vectors of differential motion are given at selected points. Six plates (shown hachured) have been added to the original six large-plate model of Le Pichon. Seismicity from 1961 to 1967 is also shown. (After Le Pichon et al., 1973.)

Fig. 1. From Lomnitz and Singh, 1976

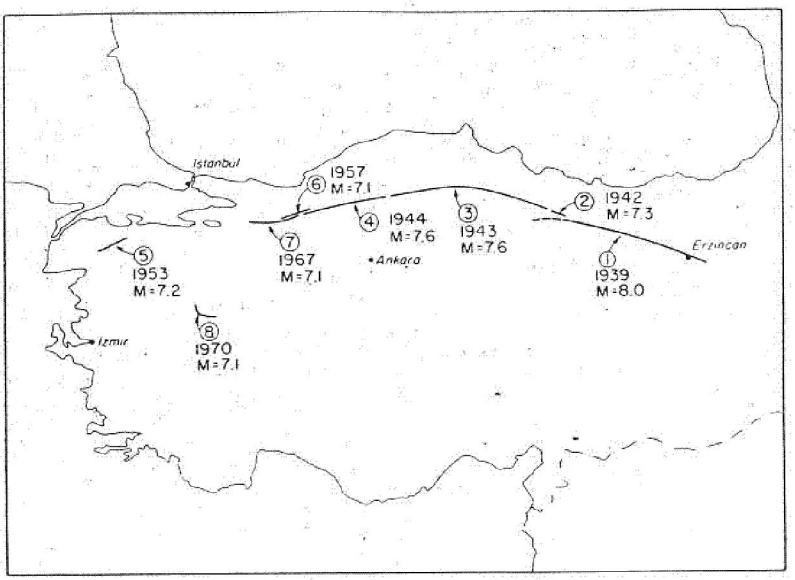

Figure 2. (from Allen,1976) Faulting associated with earthquakes of magnitude 7.0 and greater in Turkey since 1939.

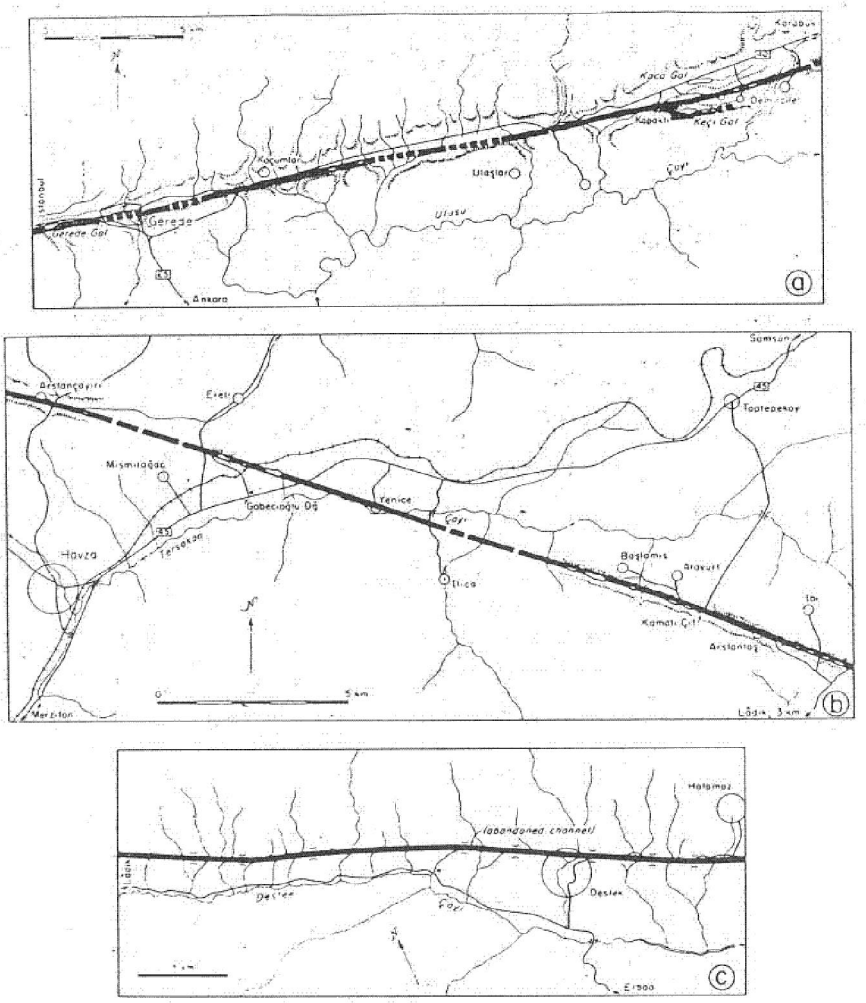

Figure 3. (from Allen,1976) Sketch of three widely spaced areas along the North Anatolian Fault Turkey.

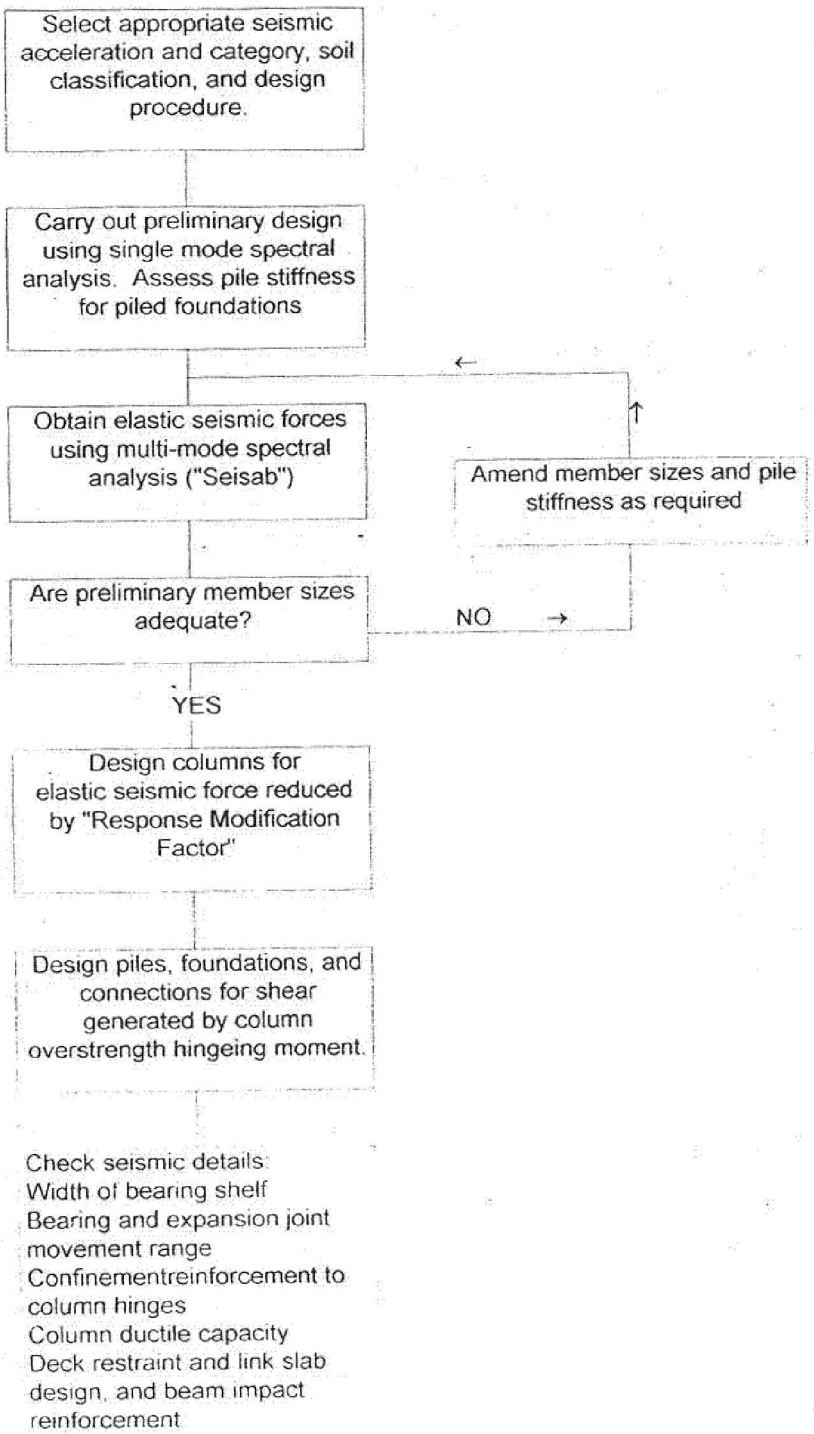

Fig. 4. Flow chart for AASHTO Category C Design Procedure

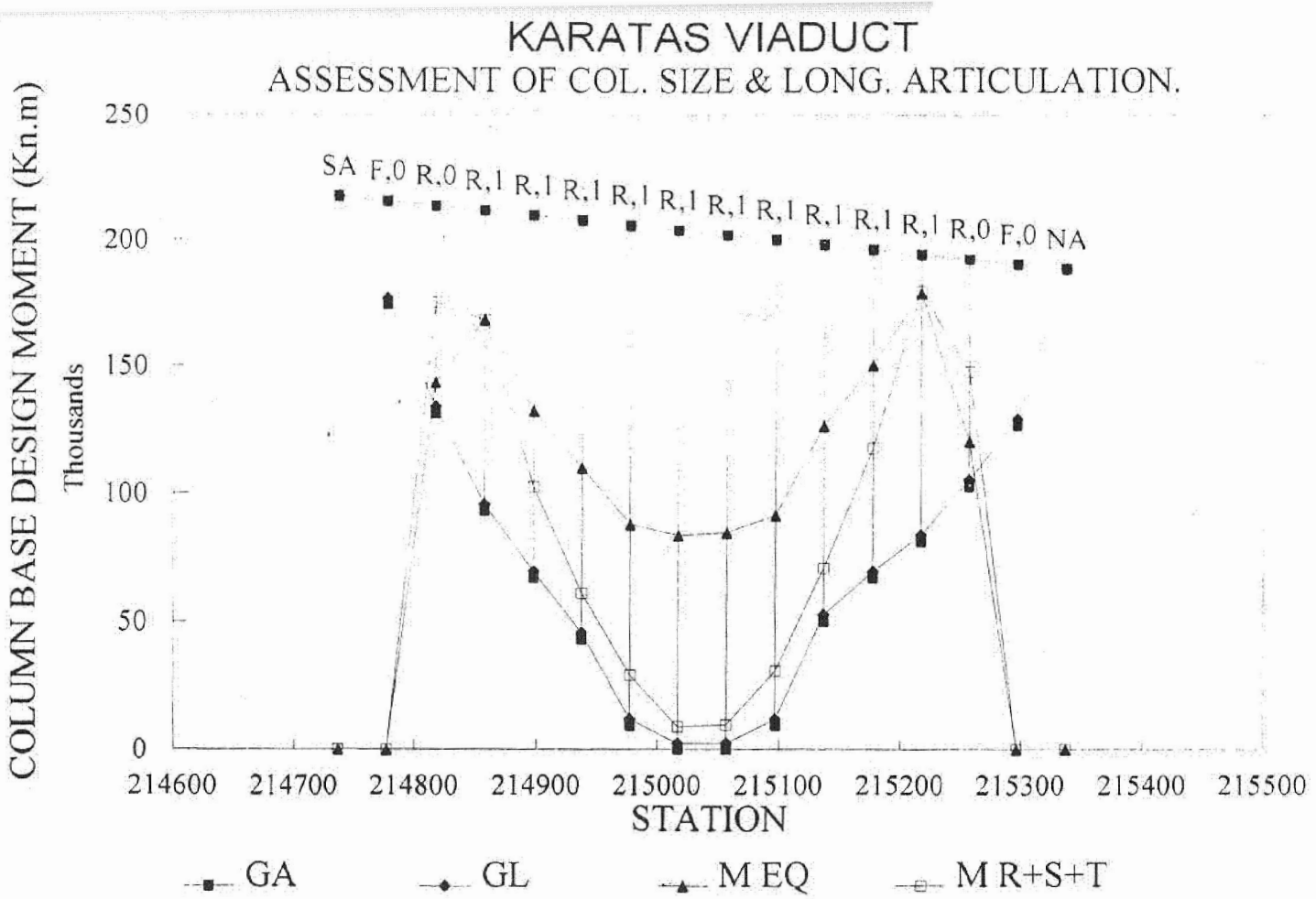

Fig. 5. Output from Single Mode Analysis Spreadsheet

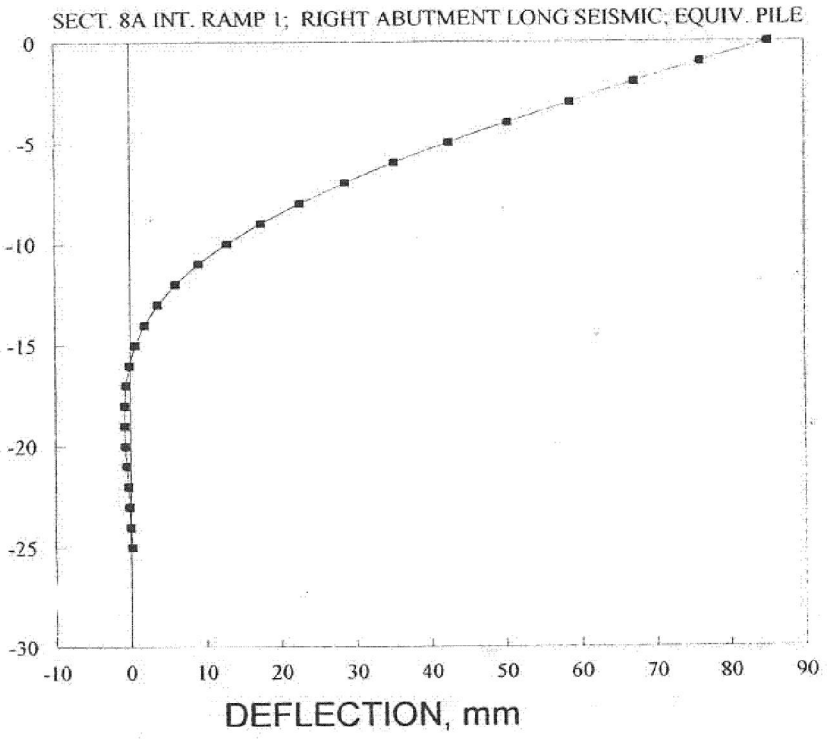

Fig. 6. Pile Deflection Output from COM624

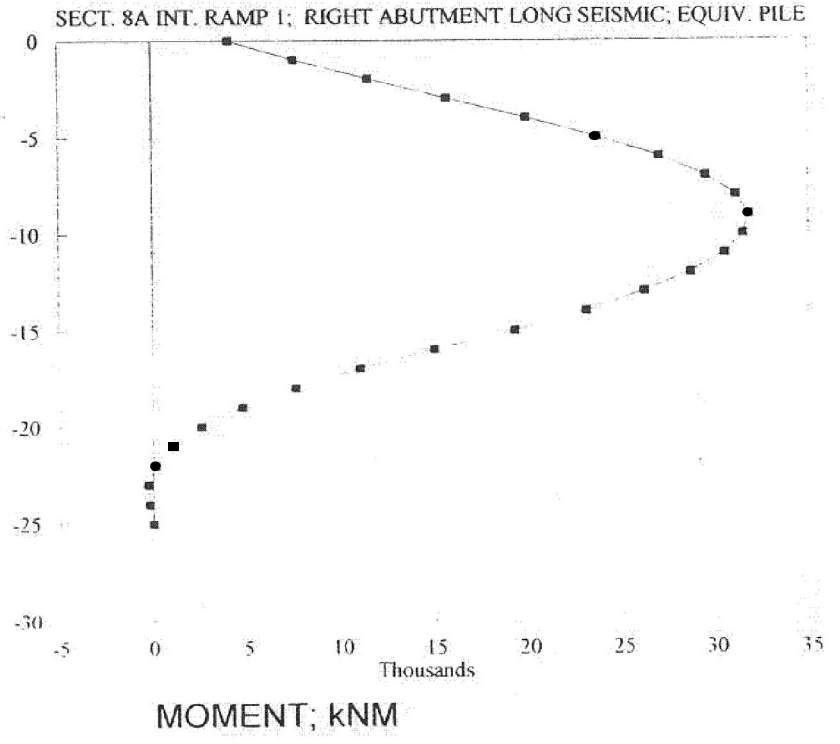

Fig. 7. Pile Moment Output from COM624

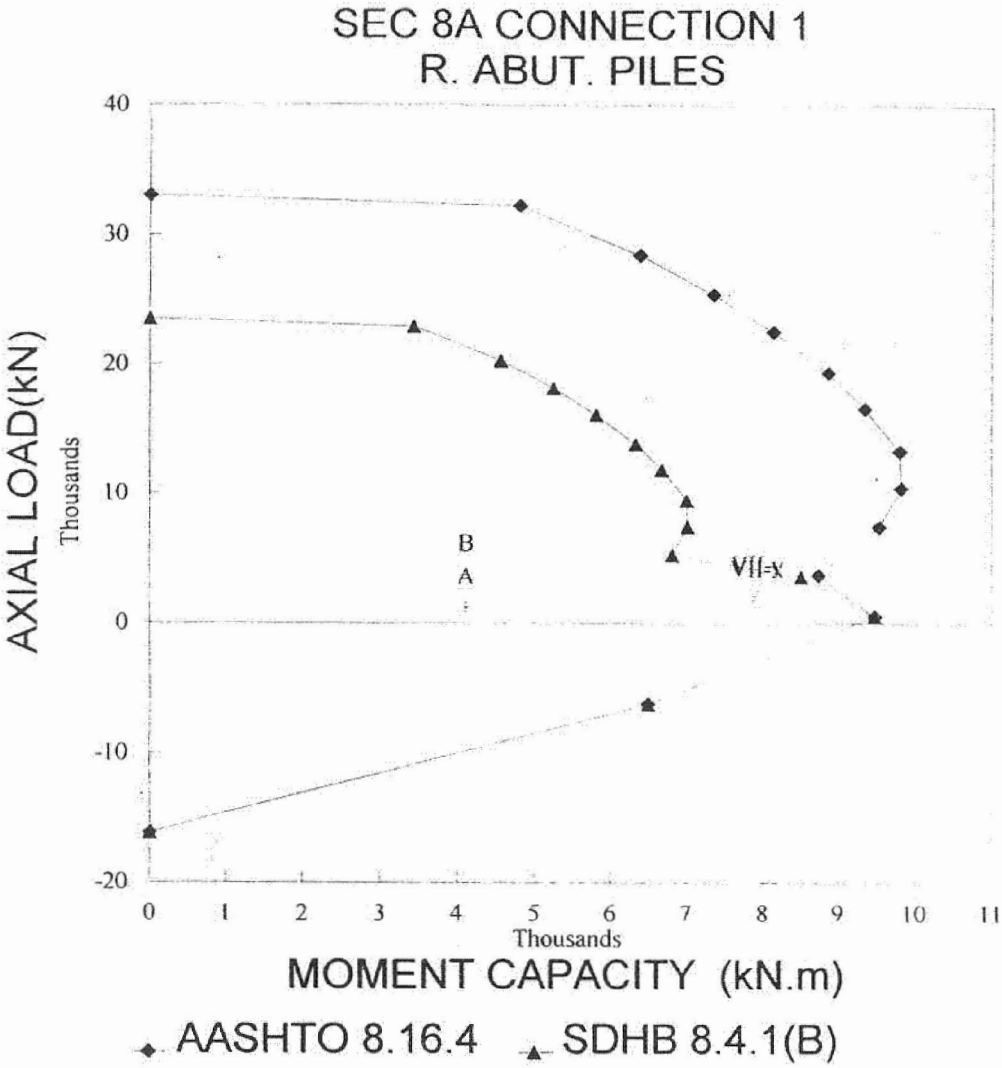

Fig. 8. Comparison of Column Capacity to AASHTO and SDHB

Fig.9. Seismic Design Details

Low Cost:

Restrain deck against transverse horizontal forces

Provide wide bearing shelf

Detail reinforcement for ductility

Detail to ensure earthquake damage is confined to accessible areas

Medium Cost

Provide additional confinement reinforcement to piers

Make deck continuous over piers for longitudinal forces

Use vertical rather than raking piles

Provide additional sliding resistance to piers, abutments and retaining walls

High Cost

Design foundations and piers for maximum seismic forces