THE INFLUENCE OF BRIDGE GEOMETRY ON THE SEISMIC BEHAVIOUR OF BRIDGES ON ISOLATING BEARINGS

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SUMMARY

The response of bridge decks supported on elastomeric or a combination of elastomeric and lead-rubber bearings is computed and compared with the response of a bridge deck monolithically connected to piers designed to have a ductile behaviour. A study of the sensitivity of bridge response to changes in geometry was made using the El Centro 1940 N-S earthquake component. The geometrical changes covered a range of uniform pier heights, a range of non-uniform pier heights and a range of number of spans.

INTRODUCTION

For many decades, the majority of the research into the design of earthquake resistant structures has been concerned with building structures and relatively little attention was paid to bridge structures. This was presumably due to the belief that social and economic consequences of earthquake damage to buildings were likely to be more serious than those resulting from damage to bridges. However, a survey of bridges damaged beyond repair in the San Fernando earthquake of 1971 exposed serious defects in supposedly earthquake-resistant bridge designs, though admittedly, most codes of practice contained little real information on the seismic design of bridges. Since that time, a considerable amount of theoretical and experimental research has been undertaken, both within New Zealand and overseas, to determine the seismic behaviour and performance of bridge structures.

In New Zealand over the last ten years or so, extensive research has been carried out into the dynamic behaviour of bridges and the ductile design of reinforced concrete bridge piers. Some of the former research has considered the use of mechanical energy dissipating devices with much of the initial work being carried out by Blakeley [1]. His studies used a simple structure/spring model from which design charts were produced.

The aim of this research was to carry out sensitivity studies of the effects of more general bridge geometries, foundation stiffnesses, different earthquake excitation, and travelling seismic waves on the seismic response of bridge structures. This paper describes the effect of bridge geometry on seismic behaviour while the influence of other parameters are discussed elsewhere [2,3,4].

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SEISMIC DESIGN PHILOSOPHY

In general, a structure resists earthquake attack by a combination of strength, deformability and energy-absorbing capacity, the relative effectiveness of these factors depending on the characteristics of the earthquake. The conventional approach to earthquake resistant design is to allow the structure to deform plastically in certain structural components. Plastic deformation has two effects on the response of a bridge. The first is that the natural period of the bridge increases as a result of the reduced stiffness and secondly the amount of damping increases because of the hysteretic properties of the plastic hinges. The consequence of both these effects is to generally reduce the response of the bridge.

The concept of equal displacements is commonly used to relate ductility to design force level although this approach has shortcomings, especially for long period structures. Recent research has shown that it is possible to design piers to maintain their integrity over a number of cycles for ductilities in excess of 6. However, where it is not possible to use fully ductile members, this deficiency can be remedied by isolating the superstructure and by concentrating the inelastic response in components which can be relatively easily installed and replaced if this is necessary. Even where ductile members are used it may be appropriate to design for lower ductility and to protect them by base isolation using for example, lead-rubber bearings.

Deck isolation is a most promising method of seismic protection. The superstructure is supported on devices which can provide flexibility and energy absorbing capacity. This usually means mounting the superstructure on flexible bearings, which can translate in any horizontal direction, and limiting the cyclic build-up of inertia forces by providing extra damping by means of energy dissipating devices. The decrease in effective stiffness of the overall structure resulting from the use of these devices means that the natural period of the structure increases. This may move it away from the period at which the ground motion has its greatest effect and a lower level of seismic force will result. The restriction of plastic deformation to the isolating dampers means that the substructure can be designed to either remain elastic or designed for a limited ductility demand. Thus the uncertainties associated with the provision of plastic hinges within the structure can be avoided, leading to a simpler and more reliable structural design.

BEARINGS AND ENERGY ABSORBING DEVICES

The most economical and effective flexural mount at present is the laminated rubber bearing which is stiff in the vertical direction and flexible in the horizontal direction. If the substructure is designed to remain elastic or of limited ductility, large horizontal deflections are induced in laminated rubber bearings under seismic excitation and some extra damping may be required to control the deflection.

Since 1971, lead-rubber bearings have been developed as hysteretic dampers (or mechanical energy dissipators) for base and deck isolation systems [5]. These consist of steel reinforced elastomeric bearings with a cylindrical lead insert through the centre. The tightly fitted lead plug deforms uniformly in pure shear when subjected to a sufficiently large seismic excitation.

More detailed information and discussion can be found in reference [6]

DYNAMIC RESPONSE ANALYSES

INTRODUCTION

The bridge modelled in this study consisted of four 20 m spans supported on stiff abutments and 1.676 m diameter piers. Each 10 m high pier supported a 1.5 m deep pier cap and was founded on rigid ground. The deck comprised of four pre-tensioned I-beams and a cast-in-situ reinforced concrete slab, 180 mm thick. The deck slab was continuous over the entire bridge and expansion joints were provided at the abutments.The bridge was modelled in plan with a view to studying the transverse response to seismic excitation and the computer model is shown in Figure 1.

The time-history analyses were carried out for 10 seconds of the El Centro N-S 1940 earthquake accelerogram because all the greatest responses takes place within this time frame.

PARAMETER STUDIES

1. BRIDGE TYPES

Dynamic analyses were carried out on two alternative structural systems. The first was a "conventionally ductile" system which responds to large earthquakes by forming plastic, ductile hinges in the piers. The second, "deck-isolation" system, responds to large earthquakes by plastic deformation of bearings which are easily replaceable. The purpose of this study was to determine whether there was any improvement in seismic performance of bridge structures where the deck rests on elastomeric bearings or a combination of elastomeric bearings and energy dissipators over those connected monolithically to the piers. Typical hysteresis loops for lead-rubber bearings and elastomeric bearings are shown in Figure 2.

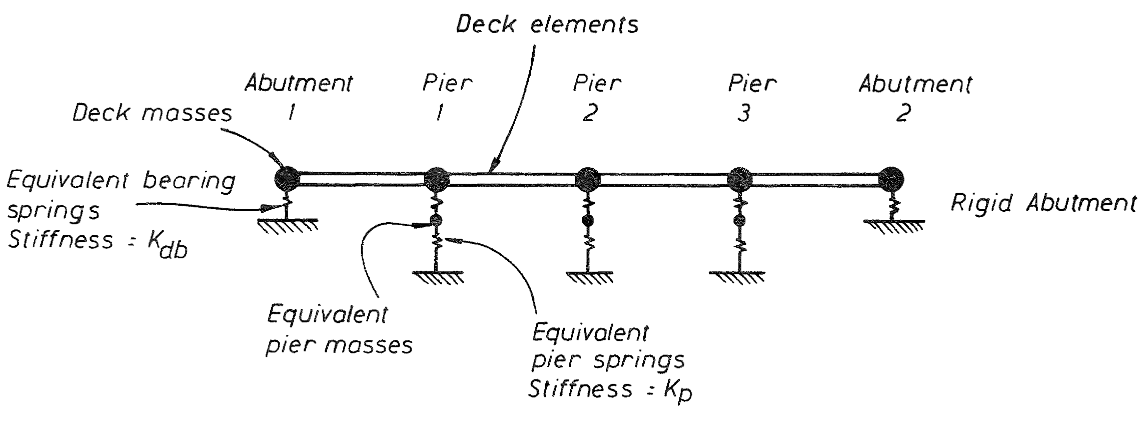

Figure 1. Bridge modelled in plan.

TABLE 1 Results of analyses for alternative bridge types ($I_c = 0.3\ I_g$)

	Conventional system Elastomeric bearings at abutment only		Base isolated system Elastomeric bearings			
			only		+Lead-Rubber	
	Elastic pier	Inelastic pier	Elastic pier	Inelastic pier	Elastic pier	Inelastic pier
Bridge number	1	2	3	4	5	6
Natural period (sec)	1.33	1.33	1.49	1.49	0.81	0.81
Maximum displacements (mm):						
Deck at abutment	102	102	117	117	57	57
Deck at pier	105	107	123	123	68	68
Bearing at pier			40	40	12	11
Pier	105	107	83	84	58	58
Maximum forces (kN):						
Bearing at abutment	374	375	429	429	440	440
Bearing at pier			301	296	251	251
Pier shear	570	435	449	410	314	314
Pier moment (kN.m)	5700	4350	4490	4100	3140	3140
Maximum pier ductility demand		1.34		1.11		<1.0

TABLE 2 Results of analyses for alternative bridge types ($I_c = 0.45\ I_g$)

	Conventional system Elastomeric Bearings at abutment only		Base isolated system Elastomeric bearings			
			only		+Lead-Rubber	
	Elastic pier	Inelastic pier	Elastic pier	Inelastic pier	Elastic pier	Inelastic pier
Bridge number	1	2	3	4	5	6
Natural period (sec)	1.15	1.15	1.38	1.38	0.76	0.76
Maximum displacements (mm):						
Deck at abutment	100	93	107	107	57	57
Deck at pier	102	98	112	112	66	66
Bearing at pier			49	48	17	17
Pier	102	99	64	67	51	51
Maximum forces (kN):						
Bearing at abutment	369	344	395	395	443	443
Bearing at pier			359	359	306	306
Pier shear	825	410	517	435	401	401
Pier moment (kN.m)	8250	4100	5170	4350	4010	4010
Maximum pier ductility demand		1.96		1.25		<1.0

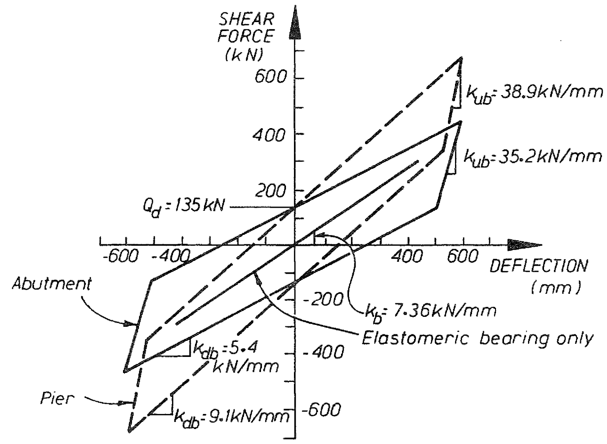

Figure 2. Typical hysteresis loops for elastomeric and lead-rubber bearings.

In the "conventionally-ductile" system, the piers were monolithically connected to the deck and the end spans were mounted on four elastomeric bearings at each abutment with total stiffness K_b = 3.68 kN/mm. Two "base-isolated" models were used: in the first, the deck was supported on elastomeric bearings and in the second, on a combination of elastomeric and lead-rubber bearings. This enabled a study to be made of the separate influence of the two bearing types on structural response. The piers in each model were either allowed to remain elastic or yield in a fully ductile manner. The deck in the first model rested on four elastomeric bearings, (K_b = 3.68 kN/mm) at each abutment, and eight elastomeric bearings (K_b = 7.36 kN/mm) at each pier. In the second model the deck rested on four lead-rubber bearings at each abutment, (K_b = 5.40 kN/mm), and a combination of four lead-rubber bearings and four elastomeric bearings at each pier, (K_{db} = 9.10 kN/mm) for which the total Q_d value was 6.6% of superstructure weight. The piers were allowed to have fully ductile yielding at the flexural strength of 4350, 4100 and 5000 kNm, for the three bridges respectively, at the "design level earthquake" [8]. The different values reflect the different natural periods of the three bridges and the consequent differences in the longitudinal reinforcement in the piers. The elastic pier stiffnesses were calculated using a cracked moment of inertia for the pier of $0.3I_g$ and $0.45I_g$ respectively for Tables 1 and 2.

It is apparent from Tables 1 and 2 that there is very little difference in displacement response between the decks supported on elastic piers and those supported on inelastic piers. The largest difference is in pier shear force and associated base moment. For the monolithic bridge, the inelastic pier values are 24% and 50% less than the elastic values for $I_c = 0.3I_g$ and $0.45I_g$ respectively. The ductility demands on the inelastic piers are small, only 1.96 maximum. Acceptable bearing shear strains of approximately 40% are reached by the elastomeric bearings and these are substantially reduced to approximately 17% by the inclusion of the lead plug in the lead-rubber bearings. Deck deflections and pier shear forces are also reduced when lead-rubber bearings are used.

The comparative analyses of the conventional ductile system [4] shows that the inclusion of energy dissipators into bridge structures supported on elastomeric bearings reduces the maximum deck displacement considerably. Although the forces on the substructure are reduced, the benefit gained is not large compared to the reduction in deck displacement. In general, in comparison with a conventional ductile design, pier ductility demand in this case is eliminated by the inclusion of energy dissipators and, as a result, structural damage is minimised. The use of elastomeric bearings also reduces pier ductility demand and hence damage, to a manageable level but deck displacements, as to be expected, are greater than those for the conventional ductile design.

2. PIER HEIGHTS

Previous studies [1] have reported that mechanical energy dissipators are ineffective when mounted on flexible substructures. In order to substantiate this statement, a range of uniform and non-uniform pier heights were considered to determine whether variations in pier stiffness produced any noticeable trend.

2.1 A Range of Uniform Pier Heights

The results of a number of analyses for very stiff, moderately stiff and flexible piers are summarised in Table 3. The standard bridge #5 with 10 m piers used in the first analyses, was used as the base for this part of the study. Pier heights of 5,6,7,8,9,10,12,14,16,18 and 20 m were selected; the results are plotted in Figure 3 but only the results for the bridges with 5,10 and 20 m piers are listed in Table 3. The flexural stiffnesses of the piers covered the range between 1.68 and 19.3 kN/mm. The design parameters of the lead-rubber bearing remain unchanged from the earlier analyses as shown in Figure 2.

Figure 4 shows the variation of deck displacement, bearing displacement and pier shear force for 5, 10 and 20 m high piers with respect to time. Maximum responses occur at either 2, 3 or 6 seconds; 2 seconds for the 5 m pier, 3 seconds for the 10 m pier and 6 seconds for the 20 m pier.

An examination of Table 3 and Figure 3 indicates that the magnitude of the maximum deck displacement increases with decreasing pier stiffness. The deck displacement of bridges supported on short stiff piers is made up from approximately the same magnitude of bearing and pier component whereas for long flexible piers the bearing component is very small compared to the pier component. In the latter case, the flexural deformation of the deck becomes more significant with the result that the

TABLE 3 Results of analyses for range of uniform pier heights

Bridge number	7	5	8
Number of spans	4	4	4
Piers 1, 2 and 3:			
Pier dimensions - Height (m)	5	10	20
- Diameter (m)	1.5	1.676	2.0
Pier flexural stiffness (kN/mm)	19.3	5.41	1.68
Natural period (seconds)	0.63	0.81	0.98
Maximum displacements (mm):			
Deck at abutment	51.2	56.5	75.9
Deck at pier	54.7	66.7	87.0
Bearing at pier	37.3	12.2	4.4
Pier	25.7	58.1	87.5
Maximum bearing force (kN):			
Deck at abutment	411.4	440.0	545.0
Deck at pier	484.3	250.8	180.5
Maximum pier shear (kN)	495.3	314.2	147.1
Maximum pier moment (kN-m)	2477	3142	2942

TABLE 4 Results of analyses for non-uniform pier heights

Bridge number	5	9	10
Number of spans	4	4	4
Piers height (m) Pier 1 and 3	10	5	10
Pier 2	10	10	20
Natural period (seconds)	0.81	0.69	0.87
Maximum displacements (mm):			
Deck at abutment	56.5	50.7	58.9
Deck at Pier 1, 3	63.6	54.7	64.2
Deck at Pier 2	66.7	56.6	66.5
Bearing at abutment	53.9	47.7	57.0
Bearing at Pier 1, 3	7.1	38.7	9.9
Bearing at Pier 2	12.2	6.8	5.8
Maximum forces (kN):			
Bearing at abutment	440.0	409.0	452.8
Bearing at Pier 1, 3	200.1	489.2	214.7
Bearing at Pier 2	250.8	196.9	186.3
Pier shear 1, 3	307.3	475.8	303.1
Pier shear 2	314.2	277.4	113.2
Pier moment 1,3	3073	2379	3031
Pier moment 2	3142	2774	2264

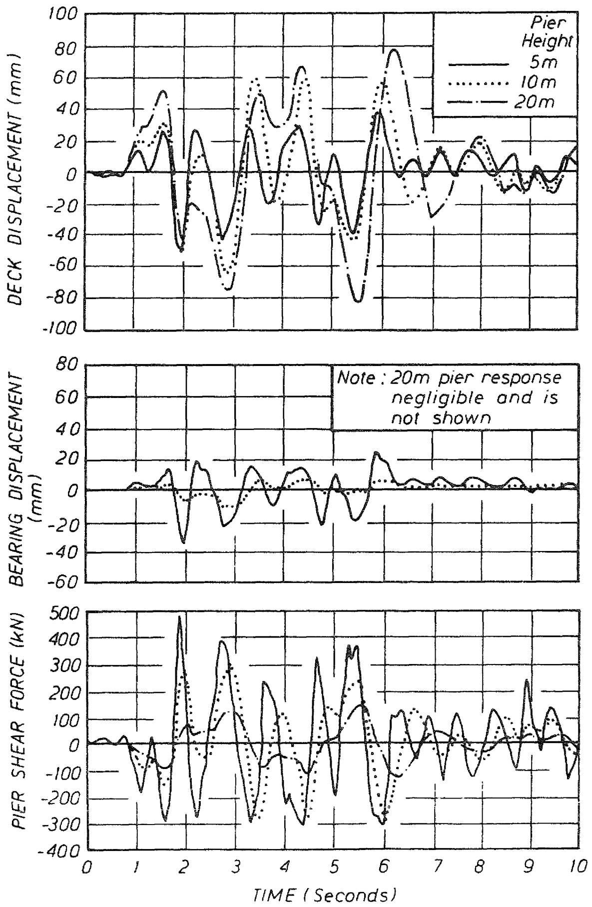

Figure 3. Effect of combined pier and bearing stiffness on bridge response at pier 2 -- with lead-rubber bearings.

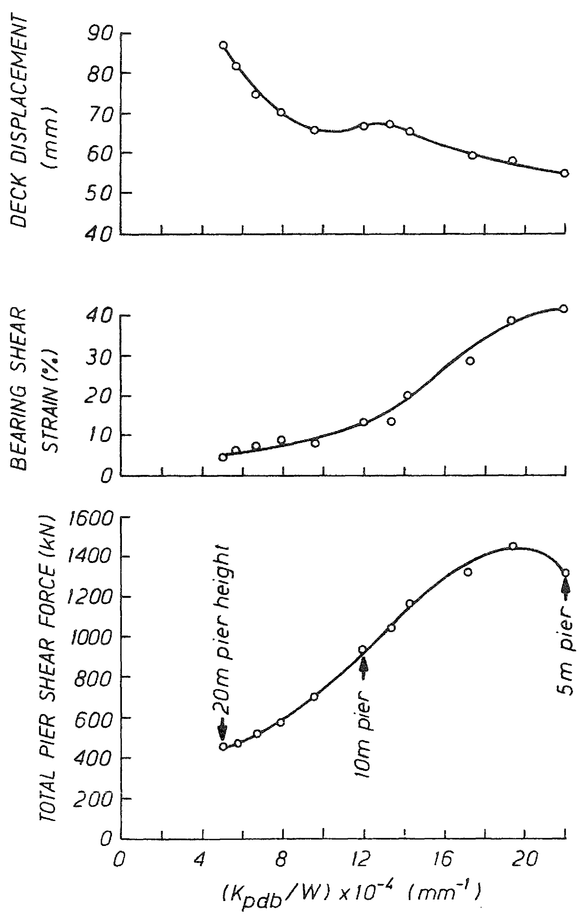

Figure 4. Effect of pier height on bridge response at pier 2 -- with lead-rubber bearings.

deck displacement at the central pier is approximately 20% greater than at the abutments for the bridges with 10 and 20 m pier heights. Figure 3 shows that the bearing shear strain at the pier remains constant at approximately 10% for piers longer than 10 m and that even at 5 m the shear strain is still within the 50% maximum limit specified by MWD [7]. Total pier shear force increases approximately uniformly with increasing ratio of total pier stiffness to superstructure weight, k_{pdb}/W.

2.2 A Range of Non-Uniform Pier Heights

The results for two sets of analyses with non-uniform pier heights are summarised in Table 4 where the results for the case of uniform 10 m pier heights, Bridge #5, have also been included for comparison. The diameter and flexural stiffness taken for the 5 m, 10 m and 20 m pier heights were the same as those given in Table 3. The ratio of flexural stiffness of piers 1 and 3 to that of pier 2 was 3.6 for Bridge #9 and 3.2 for Bridge #10.

An examination of Table 4 shows that the maximum deck displacement decreases with stiffer substructures. The deck displacements at the piers are similar even when their flexural stiffnesses differ greatly. The reason is that the bearing and pier displacement components are such that the total value effectively remains unchanged and approximately equal to the spectral displacement for that natural period. Bridge #9 is a stiffer bridge than Bridge #5, and the increased flexural stiffness of piers 1 and 3 results in an increase in the bearing strain there and a reduction at the abutments and central pier. Consequently, the lateral forces increase at the stiffer piers and decrease at the central pier. The small bearing shear strain at pier 2 of Bridge #10 indicates that lead-rubber bearings are probably unnecessary at flexible piers, also confirmed by the small bearing displacement of 6.8 mm at pier 2 of Bridge #9. The difference in pier shear force between piers 1 and 2 of Bridges #9 and 10 is approximately the same, 200 kN. In summary, the effects of non-uniform pier stiffness on bearing shear strain and pier shear force are significant.

TABLE 5 Results of analyses for range of spans

Bridge number	5		11		12		13*	
Number of spans	4		6		8		8*	
Pier height (m)	10		10		10		10	
Natural period (seconds)	0.81		1.09		1.29		1.49	
Maximum deck (1) and bearing (2) displacements (mm):	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
Abutment	57	54	58	58	58	57	53	40
Pier 1	64	7	70	13	68	16	59	8
Pier 2	67	12	79	16	84	22	81	20
Pier 3	-	-	82	19	96	26	103	29
Pier 4	-	-	-	-	101	27	127	41
Maximum bearing (3) and pier shear (4) forces (kN):	(3)	(4)	(3)	(4)	(3)	(4)	(3)	(4)
Abutment	440	-	450	-	447	-	423	-
Pier 1	200	307	251	332	287	334	207	285
Pier 2	251	314	287	362	344	371	325	347
Pier 3	-	-	311	379	385	429	403	476
Pier 4	-	-	-	-	396	450	511	637

*with central seismic gap

3. A RANGE OF SPANS

The number of different spans considered in the study were 4, 6 and 8. In addition, an 8 span bridge was analysed with a seismic gap incorporated at the centre of the structure. Since the design parameters of the lead-rubber bearings for the 6 and 8 span bridges were the same as for the 4 span bridge, their total Q_d value reduced from 6.6% to 6.2% and 6.0% of superstructure weight respectively. The results of the analyses are summarised in Table 5.

As the number of spans increases the deck structure becomes relatively more flexible, indicated by the increase in the elastic natural periods. This increased flexibility of the deck causes it to deform in a flexural mode which is particularly noticeable in the 8 span bridge where the maximum deck displacement at the central pier is considerably greater than the deck displacement at the abutments. For the 6 span bridge, the deck displacement at the abutment is about the same as for the 4 span bridge but the flexural deformation of the deck is evident. In all cases, the variation in bearing shear strains at the abutment is small compared to those at the central pier. However, the abutment bearing strains now reach 60%, exceeding the limiting design value of 50%. The pier shear forces show a similar trend to the deck deflections. These results indicate that as the number of spans increases, the deck responds more as a flexural member under the lateral loading, forcing the piers and their bearings to carry more of the load than would be the case for a shorter, more rigid deck which transfers its load to the relatively stiffer abutments.

The incorporation of a seismic gap at the centre of the deck results in a 27% increase in deck displacement at the gap. The two individual parts of the deck extend over four spans and are now less flexible with the result that the deck displacements at piers 1 and 2 are reduced. Pier shear force and bearing displacements follow the same trends as the deck displacements.

CONCLUSIONS

1. BRIDGE TYPES

The following conclusions can be drawn from the analytical study of these particular bridges with isolated decks and subjected to El-Centro 1940 N-S earthquake component.

1.1 The maximum pier ductility demand is only 1.96 for the deck monolith- -ically connected to the piers. Consequently, there is very little difference between the bridge responses when the piers are assumed to behave elastically or inelastically.

1.2 Maximum ductility demand is reduced to 1.11 by the use of elastomeric bearings and eliminated entirely by using lead-rubber bearings.

1.3 Maximum bearing strains are at all times within the 50% acceptible limit.

1.4 Maximum deck displacements are considerably reduced when lead-rubber bearings are used. Smaller reductions in pier shear force were also achieved.

2. PIER HEIGHTS

The following conclusions about the response of bridges with lead-rubber bearings to variations in pier height may be drawn.

2.1 Maximum bearing shear strains remain constant at approximately 10% for bridges with pier lengths between 10 and 20 m or for k_{pdb}/W values between $5x10^{-4}$ and $12x10^{-4}$. Bearings become more effective on the shorter, stiffer piers -- the strain is only 5% for the 20 m pier and increases to 40% for the 5 m pier.

2.2 As the maximum bearing strains increase, deck displacements decrease.

2.3 Maximum total pier shear force increases approximately linearly with increasing values of k_{pdb}/W, from 400 kN for the 20 m piers to 1400 kN for the 6 m pier.

2.4 The effect of non-uniform pier heights on bearing strain and pier shear force can be very significant, and should be taken into account in design.

3. RANGE OF SPANS

The following conclusions summarise the effect of the number of spans on the response of bridges with lead-rubber bearings.

3.1 As the bridge deck length increases the deck behaves more as a flexural member, and less as a rigid member.

3.2 The maximum deck displacements and maximum pier shear forces are greatest at the central pier -- all piers are of the uniform height -- and these are increased further if a seismic gap is incorporated in the deck at the central pier.

3.3 Maximum bearing strains at the piers are all below the 50% limit, even when the seismic gap is introduced into the bridge. However, maximum bearing strains at the abutments are approximately 60% for the 4,6 and 8 span bridges but are reduced to 40% for the bridge with the central seismic gap.

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NOTATION

I_c cracked second moment of area of pier, at first yield of the longitudinal reinforcement,

I_g gross uncracked second moment of area of a pier,

k_b shear stiffness of elastomeric bearing,

k_{db} combined stiffness of lead-rubber bearing after yielding plus elastomeric bearing,

k_{pdb} total elastic shear stiffness of pier (including effects of foundations and elastomeric bearings),

k_{ub} unloading stiffness of lead-rubber bearings plus elastomeric bearings,

Q_d characteristic dissipator strength,

W total weight of the superstructure.