6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch, New Zealand

Earthquake Design of Bridges With Integral Abutments

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ABSTRACT

Because of the desirability of eliminating maintenance costs related to deck joints in bridges many new bridges are designed with superstructures that are continuous at the piers and have either fully integral or semi-integral abutments. A large number of bridges with integral abutments have been subjected to strong ground shaking in the 1989 Loma Prieta, 1994 Northridge and other significant earthquakes in California. There was less damage to bridges with integral abutments than for bridges with structural separations at the abutments. A number of older State Highway bridges with integral abutments were subjected to strong shaking in the 2010-2011 Canterbury Earthquake sequence and the 2013 Lake Grassmere earthquake. The New Zealand bridges performed well with only minor observed damage. The paper reviews the earthquake performance of bridges with integral abutments and the available design methods of determining the stiffness, passive pressure resistance and damping for different types of integral abutments.

Introduction

In a fully integral bridge the superstructure and substructure are constructed monolithically and there are no movement joints in the superstructure between spans, and between spans and abutments. Figure 1 shows the main elements of one type of integral bridge abutment system, which consist of, girders, integral cast abutments and approach slabs. The bridge movement is accommodated at the ends of the approach slabs. In California it is common to have an end diaphragm wall integral with box girder superstructures.

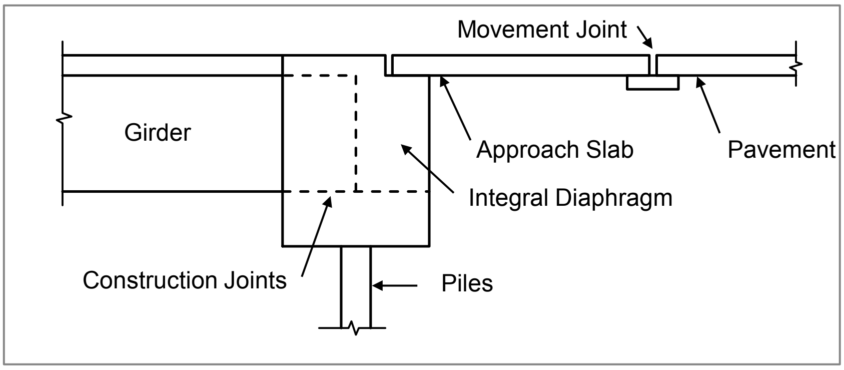

Figure 1. Integral bridge system used for girder type superstructures.

Both in New Zealand and internationally, there is an increasing interest in the design and construction of integral and semi-integral bridges which have some marked advantages over other bridge construction forms, such as reduced maintenance. However, there are a number of issues related to the design of integral bridges which require either further investigation or clearer guidance. These issues are mainly related to soil-structure action effects at the

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abutments that arise from concrete creep and shrinkage, temperature movements and passive resistance under earthquake loads.

A large number of integral abutment type bridges have been subjected to strong ground shaking in earthquakes that have occurred over the past 45 years in New Zealand (NZ) and California. The performance of the bridges in California is of particular interest since approximately half the concrete bridges on the state highway system, constructed after about 1960, have integral abutments. In contrast to California, the number of bridges constructed with integral abutments in New Zealand is only a small percentage (< 5%) of the total and most of these were constructed prior to 1960.

Performance of Integral Abutment Bridges in New Zealand

Table 1 summarises the performance of the NZ integral abutment bridges subjected to strong ground shaking in one or more of the 2010 Darfield, 2011 Christchurch, 2013 Cook Strait and 2013 Lake Grassmere earthquakes. All the bridges listed were constructed between 1930 and 1955 and are of monolithic reinforced concrete T beam construction except for the Needles Creek Bridge which has a reinforced concrete slab superstructure.

Four of the eight bridges received minor structural damage which typically consisted of cracking in the tops of the concrete piles and fine cracking in the abutment walls. Flexural failures occurred in the abutment walls of the Halswell River Bridge and were attributed to the effects of lateral spreading from liquefaction. Apart from this bridge there was little evidence of significant longitudinal movement (less than 30 mm) with only minor settlement at the abutment wall/backfill interfaces and no damage to abutting flexible pavements.

Table 1. Bridges with integral abutments subjected to strong earthquake shaking.

Bridge Name	**Age**	**Len. m**	**Abut. Wall Ht m**	**PGA Est. g**	**Main EQ Event**	**EQ Related Damage**
Wairau River	1939	293	1.3	0.13	Cook Strait Grassmere	Fine cracking in abutment walls
Spring Creek	1939	43	1.8	0.15	Cook Strait Grassmere	Fine cracking in abutment piles and walls
Flaxbourne River	1955	64	1.1 1.6	0.4	Grassmere	Spalling on beam faces at bearings on piers
Needles Creek	1953	43	1.2	0.35	Grassmere	No visible damage
Tirohanga Stream	1939	21	2.3	0.17	Grassmere	No visible damage
Halswell River	1937	6.7	2.1	0.3	Darfield Christchurch	Flexural failures in abutment walls
Selwyn River	1931	92	2.1	0.45	Darfield	No visible damage
Hawkins River	1939	82	1.4	0.45	Darfield	Cracking and spalling in the tops of pier piles

Performance of Integral Abutment Bridges in California

San Fernando Earthquake (1971, Magnitude = 6.9)

Approximately 70 freeway bridges were located within a 17 km radius of the center of energy release of the San Fernando earthquake, and of these, approximately 40 received significant damage including five bridges that collapsed. These bridges were subjected to very intense ground motions with PGA's in the range of 0.25 to 0.5 g (Jennings, 1971).

Abutment damage was widespread and most severe on skew bridges which have a tendency to rotate in a horizontal plane. Shear type failures in the components that either restrained or provided a connection to the superstructure were common (Wood and Jennings, 1971). Insufficient information was reported to enable a comparison to be made between the performance of bridges with integral and separated abutments.

Loma Prieta Earthquake (1989, Magnitude = 7.1)

The majority of the bridge damage was in the San Francisco Bay area about 100 km away from the epicenter. Soft soil at bridge sites contributed to the extensive damage observed this far away from the epicenter. Sections of three major bridge structures collapsed (Oakland Bay Bridge, Cypress Street Viaduct and Struve Slough). Bridge data gathered by Caltrans identified a total of 76 damaged bridges.

Basoz and Kiremidjian (1998) carried out a correlation study to identify the structural characteristics that contributed most to the damage observed on concrete bridges subjected to strong ground accelerations (> 0.1 g). They categorised the bridges using three abutment, three superstructure and three pier types. Approximately 310 continuous reinforced concrete bridges with monolithic abutments at both ends were subjected to strong shaking. Their study showed that the bridges with monolithic abutments were less likely to be damaged than bridges with other abutment types.

Northridge Earthquake (1994, Magnitude = 6.7)

Caltrans (1994) estimated that 1600 state and county bridges were subjected to PGA's of 0.25 g or greater in the Northridge Earthquake. Seven highway bridges suffered partial collapses and a further 170 bridges suffered damage ranging from minor cracking to the slumping of abutment fills.

Basoz and Kiremidjian (1998) carried out a correlation study similar to their Loma Prieta earthquake study to identify the structural characteristics that most contributed to the observed damage to concrete bridges. Approximately 560 continuous reinforced concrete bridges with monolithic abutments at both ends were subjected to strong shaking (PGA > 0.1 g). Again the results indicated that the bridges with monolithic abutments were less likely to be damaged than bridges with non-monolithic abutments but the difference in performance was less marked than was the case for the Loma Prieta event.

The Caltrans (1994) Northridge Post Earthquake Investigation Report stated that; "End-diaphragm abutments protected structures, or parts of structures, in which they were present to a greater degree than seat type abutments…."

Design Considerations

The performance review of integral abutments identified the following design considerations:

- Integral abutments are much stiffer than adjacent piers and therefore attract a large part of both the longitudinal and transverse inertia loads from the superstructure.
- Earthquake forces on the abutments of long and wide bridges can be large and special detailing of the backwall and their foundations is required to resist these forces.
- The NZ bridges (listed in Table 1) did not have approach slabs but approach or friction slabs should be used and well anchored to avoid separation from the abutment.
- To reduce longitudinal displacements of straight bridges and horizontal rotations of skewed bridges, backfilling with densely compacted cohesionless soil is important.
- Damping from abutment soil-structure interaction can increase the overall damping to a much higher value than the 5% often assumed in bridge design.

Seismic Design of Integral Abutments

Structural Form

The shape and height of the abutment walls will generally be determined by considerations other than seismic performance. The small initial stiffness of the backwall passive load response is almost independent of the wall height but the ultimate passive resistance is proportional to the square of the wall height so generally there are advantages in having the backwall as high as practicable.

Analysis Method

The displacement based design method (DBD, Priestley et al, 2007) should be used for the analysis of bridges with integral abutments. This method provides a more satisfactory way of allowing for the relative stiffness of the piers and abutments and the effects of soil-structure interaction (damping and stiffness) from the sub-structure components than force based design procedures.

Abutment Stiffness

The stiffness of the soil against the abutment walls can be determined using the hyperbolic force-displacement (HFD) relationship presented by Khalili-Tehrani et al (2010). This relationship has been calibrated against earlier Log-Spiral Hyperbolic force-displacement models (Shamsabadi et al, 2005, 2007) which in turn were calibrated against several small-scale and full-scale tests of abutment walls and pile caps. The form of the HFD equation is:

$$F(y) = \frac{a_r y}{H + b_r y} H^n \qquad (1)$$

where F and y are the lateral force per unit width of the backwall and deflection respectively. The parameters a_r and b_r depend only on the backfill internal friction, cohesion, unit weight and soil strain at 50% of ultimate stress. H is the backwall height. Exponent n is dependent on the soil cohesion and internal friction and has a range of 1.0 < 2.0 (2.0 for zero cohesion).

Equation 1 is strictly only applicable for a wall that is uniformly translated against the backfill. In many applications the wall will rotate as well as translate against the backfill and for these cases it is best to represent the force against the backfill by a series of Winkler springs over the height of the wall. The force-displacement relationship for each spring should be based on assuming a linear increase of stiffness with depth and with the total stiffness of the springs adding to the stiffness represented by the HFD equation.

The passive resistance and stiffness of abutment walls is reduced by skew angles. Passive force-deflection curves for skewed abutments based on laboratory testing of model walls have been presented by Jessee and Rollins (2013) and compared with numerical studies undertaken on skewed abutments by Shamsabadi et al (2006).

Abutment Damping

The damping associated with abutment dynamic cyclic loading can be estimated from the test results of Rollins et al (2010) who undertook full-scale tests to quantify the effects of cyclic and dynamic loading on the force-displacement relations for typical pile caps and abutment walls. The values that they measured during slow cyclic loading appear to be appropriate for most bridge abutment applications. Median values for densely compacted sand, fine gravel and coarse gravel were all about 18%.

Backfill Soil

Densely compacted coarse gravel should be used as backfill material when this type of material is available. The length of the compacted zone of backfill should extend for at least the height of the wall behind the backface of the abutment and vertically below the bottom of the wall about 25% of the height of the wall.

Soil Gapping Effects

Static cyclic load tests reported by Rollins and Cole (2006) on a full-scale pile cap indicate that gaps of 50% to 70% of the peak displacement may develop in a coarse gravel backfill. However, these tests do not simulate the inertia loads in the backfill that arise in strong shaking. Cyclic inertia loads in the backfill force the backfill material back against the backwall to potentially develop active pressures against the backwall. The impact of gapping on the dynamic response can be reduced by using settlement or friction slabs which are also a very effective method of increasing damping.

Predicted Performance of Integral Abutments

Friction Slab Performance

Yeo (1987) carried out what appears to be the only published experimental research on the earthquake performance of friction slab abutments. A friction slab was combined with a vertical abutment wall in the final stage of a more comprehensive study of the performance of model abutment walls subjected to cyclic loading. The model walls were 1.0 m high x 2.4 m wide and the friction slab was 2.5 m long buried at a depth of 0.68 m. Cyclic loading was applied to the wall with a hydraulic actuator and a loading system that restrained the motion of the wall to pure horizontal translation. A moist medium dense sand backfill was used.

Figure 2 shows the force-displacement results for both the wall with a friction slab and a similar model without a friction slab. The axes in Figure 2 are dimensionless with the force $P_d = P/0.5\gamma H^2 B$, where P is the total force on the wall, γ the unit weight of the soil and H and B the height and width of the wall respectively.

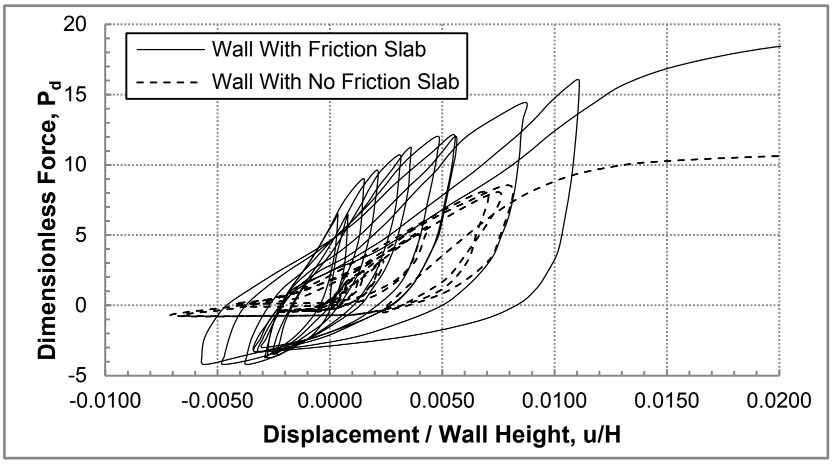

Figure 2. Experimental force-displacement curves for translation of model abutments.

Comparison of the cyclic force-displacement curves in Figure 2 shows the benefits of a friction slab for abutment walls under cyclic loading. It increases the stiffness and failure loads and for a given load level reduces the permanent displacement or “gapping” effect. Damping represented by the areas within the force-displacement loops was estimated to be approximately 22% and 16% for the walls with and without the friction slab respectively.

Bridge Abutment Example

To quantify the longitudinal resistance available from a typical bridge integral abutment results were calculated for a 3 m high abutment wall fitted with and without a 6 m long friction slab located at a depth of 1.5 m below the top of the wall. The backfill material was assumed to be cohesionless gravel with an internal friction angle of 35^o and unit weight of 20 kN/m^3. A force-displacement relationship for the abutment without the friction slab was calculated using Equation 1 and the influence of the slab on the response was investigated using LimitState:Geo (2014). This software applies limit analysis plasticity theory to provide an estimate of plastic collapse loads for structures interacting with soil foundations.

The failure slip lines for the abutment with the friction slab are shown in Figure 3 for both the case when the force applied by the bridge is directed towards the backfill (push) and when it is directed away from the backfill (pull). Failure loads from LimitState:GEO were; 950 and 640 kN/m (of width) for the push direction with and without a friction slab respectively, and 105 kN/m for the pull direction. Equation 1 gave a load of 620 kN/m at large displacements (> 200 mm) – in good agreement with the LimitState:GEO result for no friction slab. The LimitState:GEO result for the pull case is approximately equal to $0.85W\tan\phi$; where ϕ is the soil internal friction angle and W is the weight of soil above the friction slab.

The force-displacement relationship from Equation 1 for the 3 m high wall described above is plotted in Figure 4 together with curves for the wall with the 6 m long friction added. The total force curve (Push + Pull) is for the case when abutments at both ends of the bridge provide combined resistance to longitudinal loads. The curves for the walls with friction slabs added are approximate as LimitState:GEO does not provide displacements.

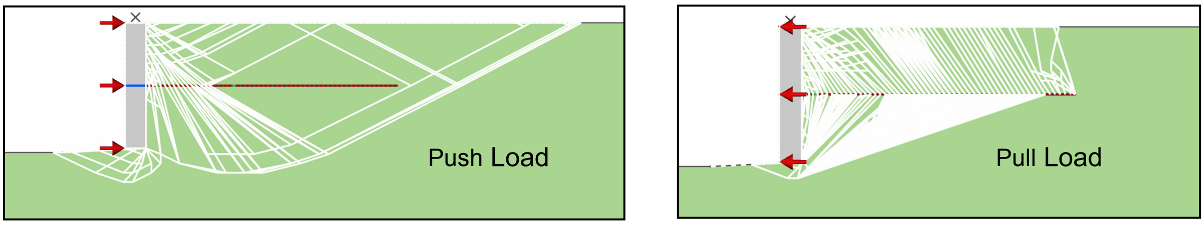

Figure 3. Slip-lines from LimitState:GEO for abutment fitted with friction slab.

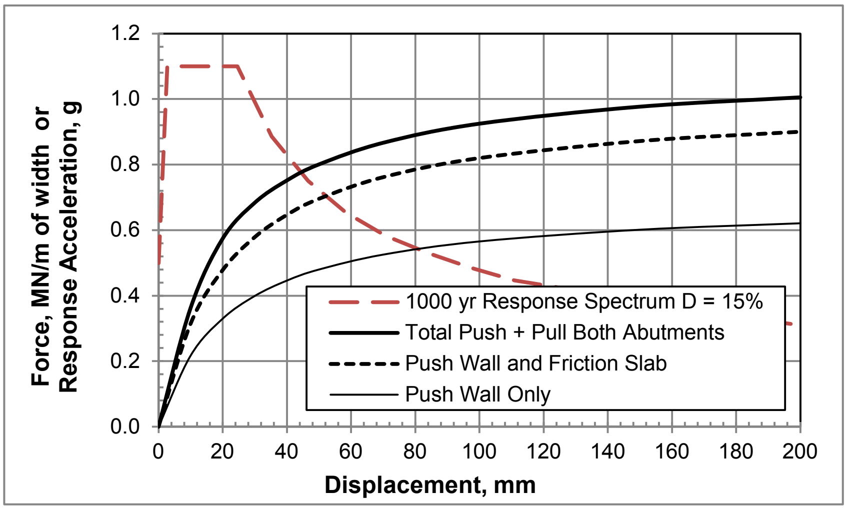

Figure 4. Force versus displacement response and earthquake demand curves.

The superstructure of a typical NZ highway bridge constructed from prestressed concrete has a weight of approximately 10 kN/m^2. The force ordinate in Figure 4 can therefore be interpreted as the response acceleration acting on a 100 m long bridge (weight = 1 MN/m of width) subjected to a longitudinal static acceleration. To assess the performance of this hypothetical bridge a demand curve based on the NZS 1170.5 response spectrum for a 1000 year return period and Category C site subsoil is superimposed on Figure 4. The spectrum is scaled to 15% equivalent viscous damping to account for the high damping expected from soil-structure interaction at the abutments.

Intersection of the response and demand curves indicates that the wall passive resistance and the frictional resistance from slabs at both ends of the bridge (ignoring any resistance from the piers) limits the longitudinal displacement to less than 50 mm under a design level event. Integral abutments enhanced with friction slabs can therefore be very effective in reducing the longitudinal displacement response to a level unlikely to cause significant damage to any of the substructure components.

Conclusions

- Bridges with integral abutments have performed well in strong ground shaking in both NZ and California. Passive resistance and damping at the abutments limits the longitudinal response leading to less damage than for bridges with seat type abutments.

- Friction slabs should be used with integral abutments to limit gapping and add increased damping. For flexible pavements commonly used in NZ they can provide the combined function of both a friction and settlement slab.
- Recent research results provide a basis for predicting the resistance of integral abutments to bridge longitudinal earthquake loads.

Acknowledgement

This paper is based on research undertaken by the writer for Opus International Consultants Ltd (Opus) and is part of a research project on integral bridges funded by the New Zealand Transport Agency. Darren Goodall of Opus provided reference documents.

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