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Full Length Research Paper

The seismic performance comparison of integral bridge model by using finite element program and shaking table test

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In this paper, the experimental approach was conducted to validate the result of numerical approach. The integral concrete box-girder bridge was modelled by using finite element method (FEM) program and comparing by using dynamic shaking table. There are four steps that were involved in validating the result: (1) scaling integral concrete box-girder bridge by implementing Buckingham PI theorem; (2) setting the shaking table, shaker controller, strain gauge, load varied displacement transducer (LVDT) and accelerometer; (3) analyzing finite element modelling of scaled integral concrete box-girder bridge; (4) comparing the acceleration of structure response from accelerometer and finite element modelling. The results show that the percentage of difference of experimental test and finite element analysis is only 11% for structure acceleration and 18% for structure displacement. It was found that the experimental test result had proved that the finite element analysis as a numerical approximate method can demonstrate significant actual bridge response under earthquake loading.

Key words: Dynamic shaking table, finite element program, acceleration, displacement, strain.

INTRODUCTION

In Malaysia, integral concrete box-girder bridge is currently becoming a popular choice of system as it reduces the cost of bridge maintenances. Figure 1 showed the problem that can occured in such system when considering the seismic effect where rigid connection of column and deck will produce potential hinge failure (secondary stress) (Patty et al., 2001).

The movement of integral concrete box-girder bridge due to earthquake loading may possibly be larger than the movement due to thermal loading. There are not quite a number of researchers whom study the seismic of integral concrete box-girder bridge (Chen, 1996; Khan, 2004).

Numerous experimental studies on the behaviour of bridge columns under reversed cyclic loading have been reported (Park et al., 2003; Suhatril et al., 2011). However, the majority of these works are based on experiments in which members are subjected to pseudostatic, reversed cyclic motions rather than to realistic earthquake ground motions. Characterization and modelling of the deformation and damage behaviour of structural members under seismic loading should duly account for the dynamic effects arising from realistic earthquake scenarios.

The performance of integral concrete box-girder bridge under earthquake loading can be implemented by conducting experimental test and finite element modelling. However, the accuracy of finite element modelling as approximate method to predict the actual behaviour of bridge under earthquake loading is very important to be investigated (Karbakhsh et al., 2011).

There are two main purposes of structural modelling experiment. First, the structural modelling may be constructed and the experimental information provided by testing may then be used to confirm theoretical analysis. This analysis may be utilized to analyze and hence, provide calculations for the design of a prototype structure. Second, the structural modelling may be constructed due

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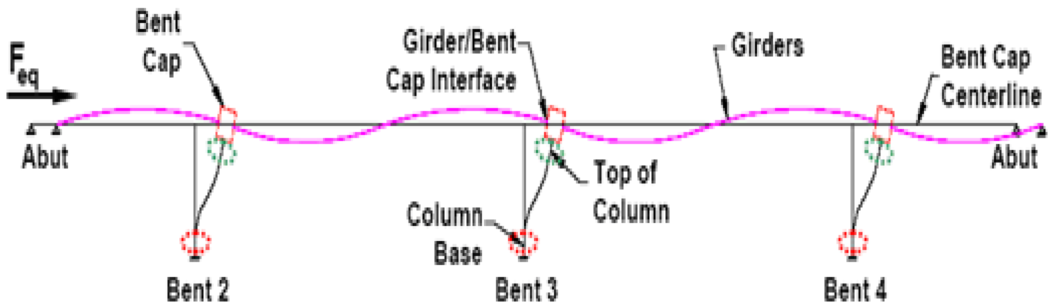


Figure 1. Potential hinge locations.

Figure 2. Table top grid of R-51 hydraulic shaking table.

Table 1. Shaking table specification.

Parameter	Specification
Table size	1.2 m × 3.2 m
Maximum weight	1800 kg
Maximum lateral force	10,000 pound (44 kN)
Maximum displacement	+/- 80 mm
Maximum velocity	30 ips (0.8 m/s)
Maximum acceleration	1.8 g w/maximum load of 1800 kg
Directions of motion	Horizontal
Driving force	Hydraulic oil pressure servo
Wave shape	Earthquake
Frequency limit	1-50 Hz

to the current limit of structure behaviour. The main advantage of using finite element modelling is its ability to model a bridge in a very complicated structure, whilst experimental test cannot perform the same ability due to the limitation of shaking table (Hashamdar et al., 2011; Yashinsky and Ostrom, 2000).

To properly evaluate the accuracy of finite element method (FEM) in presenting the integral bridge behaviour earthquake loading, the experimental test by using dynamic shaking table was conducted.

EXPERIMENTAL

Shaking table specification and capability

The ANCO R-51 hydraulic shake table in Universiti Teknologi Malaysia, Skudai was used for this investigation (Figure 2). The characteristics of the shake table used in the experiment are given in Table 1 (Anco Engineers, 2006).

Preparation of bridge model

The RapidKL concrete box-girder bridge deck was the adopted model used to design the box-girder bridge model in this study. The RapidKL box-girder bridge was selected due to its simple cross section deck and uniform depth of deck along the span. The actual bridge deck section is impossible to model due to the limitation of shaking table size in the laboratory. The bridge scaling can be seen in Table 2. The cross section of box-girder bridge deck model has been simplified as Figure 3.

The bridge model is made up of 1500 mm +1500 mm span (total 3000 mm). The bridge model is a reinforced concrete box-girder deck which neglected the pre-stressed tendon of actual bridge deck. It consists of two span decks supported by three piers (Figure 3).

The steps of constructing the bridge model are shown in Figure 4 and are explained as follows:

Step 1: Plywood with a thickness of 1 cm was used to construct the formwork for deck and pier bridge model.

Step 2: Steel with a diameter of 0.5 cm was used to present the reinforcement of concrete box-girder bridge deck.

Step 3: Longitudinal and transverse rods were used to make sure the bottom of piers were fixed at the shaking table.

Step 4: Polystyrene was used to provide hollow at the middle of the box-girder deck and the piers used the reinforcement with a diameter of 10 mm.

Step 5: Concrete was poured to formwork.

Preparation of shaking table

In this experimental study, three parameters were investigated, that is, acceleration, displacement and strain. The parameters, as shown in Figure 5 were measured by using an accelerometer, strain gauge and load varied displacement transducer (LVDT).

Table 2. Bridge model scale analysis.

No.	Deck	Formula	Bridge prototype	Bridge model	Bridge model modification
1	Span (*m*)	$L_m = \sqrt{\frac{1}{\Delta}} \times L_P$	24	1.5	1.5
2	Area (m^2)	$A_m = \frac{1}{\Delta^2} \times A_P$	3.3320	0.51e-4	0.220e-1
3	Selfweight of Beam (*kN/m*)	$W_m = \frac{1}{\Delta^2} \times W_P$	81.71685	0.49e-5	0.53955
4	Moment of Inertia (m^4)	$I_m = \frac{1}{\Delta^4} \times I_P$	2.31714	0.5e-9	1e-4
5	Section Modulus (m^3)	$Z_m = \frac{1}{\Delta^2} \times Z_P$	2.627127	1.57e-7	1.376e-3

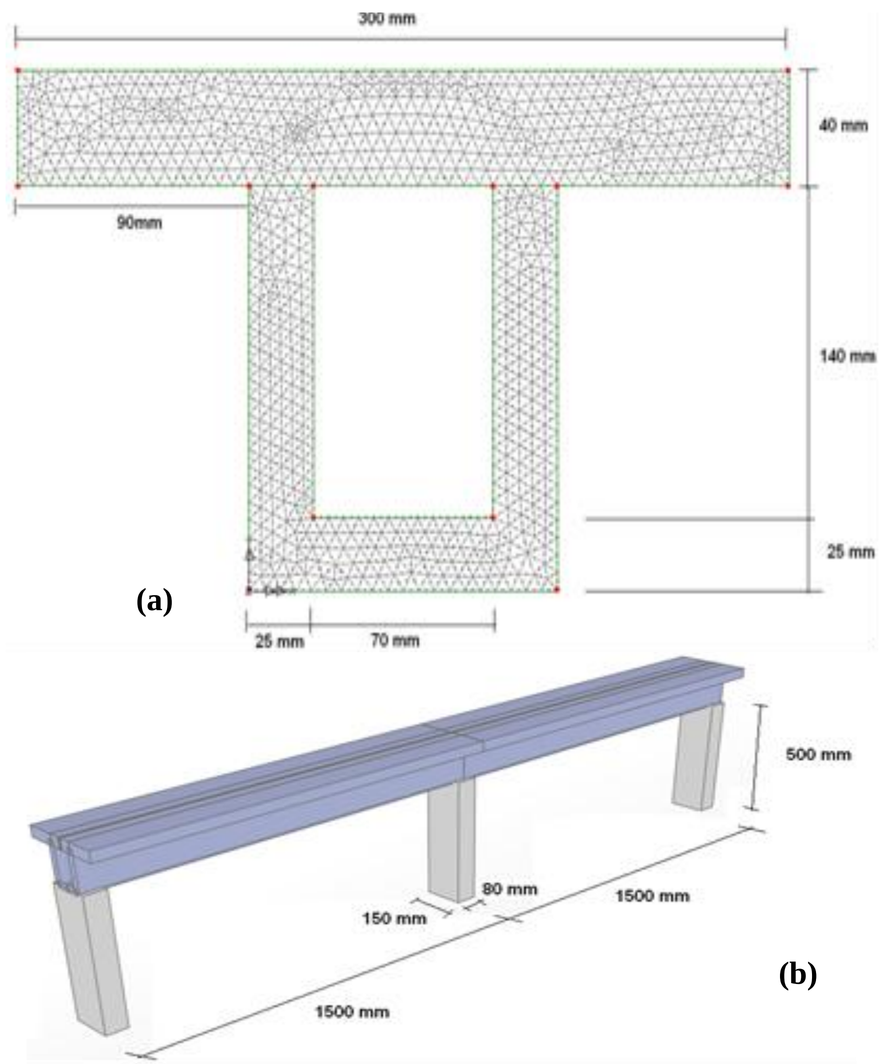


Figure 3. Cross section and dimension of box-girder bridge model (Rapid KL bridge).

Figure 4. The process steps of constructing Rapid KL bridge model.

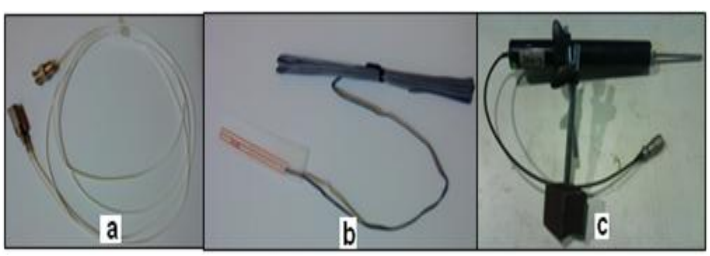


Figure 5. (a) Accelerometer; (b), strain gauge; (c), LVDT.

Figure 6 shows the locations of structure response observation which is installed at the deck and pier model.

Dynamic experimental test was carried out by using the shaking table. The shaking table was driven in the transverse direction of two span of bridge model. The input excitation of the table was El-Centro time history with the maximum acceleration of 0.3 g.

To evaluate seismic behaviour of the structure model, the El-Centro earthquake is selected. The earthquake loading were scaled to 0.06 g, 0.07, 0.1, 0.12, 0.15, 0.18, 0.20 and 0.25 g, and the original peak ground acceleration (PGA) of El-Centro earthquake is 0.3 g. The scaling of earthquake loading at the same frequency content is used to monitor the bridge model responses.

Finite element bridge modelling

The bridge was modelled by using SAP2000 program (SAP2000, 2009) which used frame elements to present deck and piers. The finite element bridge model is shown in Figure 7.

RESULTS AND DISCUSSION

Acceleration responses of bridge model as mentioned earlier, two accelerometers were used to monitor the acceleration of bridge responses. The locations of the installed instruments are channels 7 and 8.

Figure 8 shows acceleration response of bridge model at channels 7 and 8 by experimental test and finite element analysis under earthquake loading with PGA 0.3 g. The bridge response shows that the experimental test results do not have much difference with finite element analysis. The pattern of acceleration history is more or less the same for both methods.

The results comparison of bridge model acceleration response by experimental test and finite element analysis is shown in Table 3. The results show that the percentage

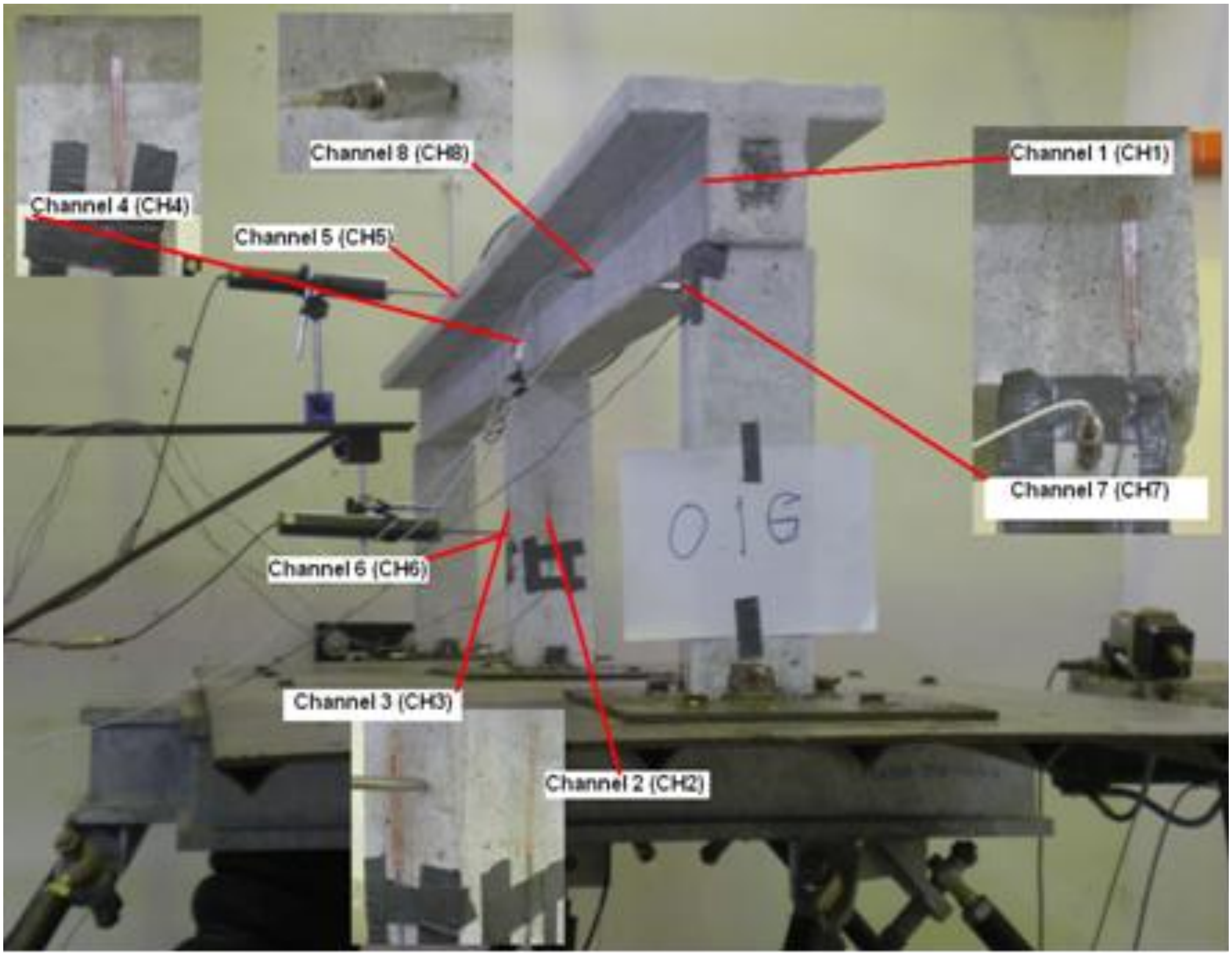


Figure 6. Response data recorder position (accelerometer, LVDT and strain gauge).

of difference of experimental test and finite element analysis is only 11%.

Displacement responses of bridge model

There are two LVDTs used to monitor the displacement of bridge responses. The locations of the installed instrument are channels 5 and 6.

Figure 9 shows the acceleration response of bridge model at channels 5 and 6 by experimental test and finite element analysis under earthquake loading with PGA 0.3 g. The bridge response shows that the experimental test results do not have much difference with the result using finite element analysis. However, the pattern of displacement history is different for both methods.

The comparison of bridge model displacement responses obtained by experimental test and those by finite element analysis is shown in Table 4. The results show that the percentage of difference of experimental test and finite element analysis is only 18%.

Strain responses of bridge model

Figure 10 shows the comparison of strain responses of bridge model by using experimental test and finite element analysis for 0.18 g PGA.

CONCLUSIONS

Based on the experimental test and finite element analysis result, it shows that the result of experimental test is almost the same with the finite element analysis. It also shows that the finite element as a numerical approximate method can actually it is not 100% the same. Fourth, the homogenous material properties such as strength and modulus of

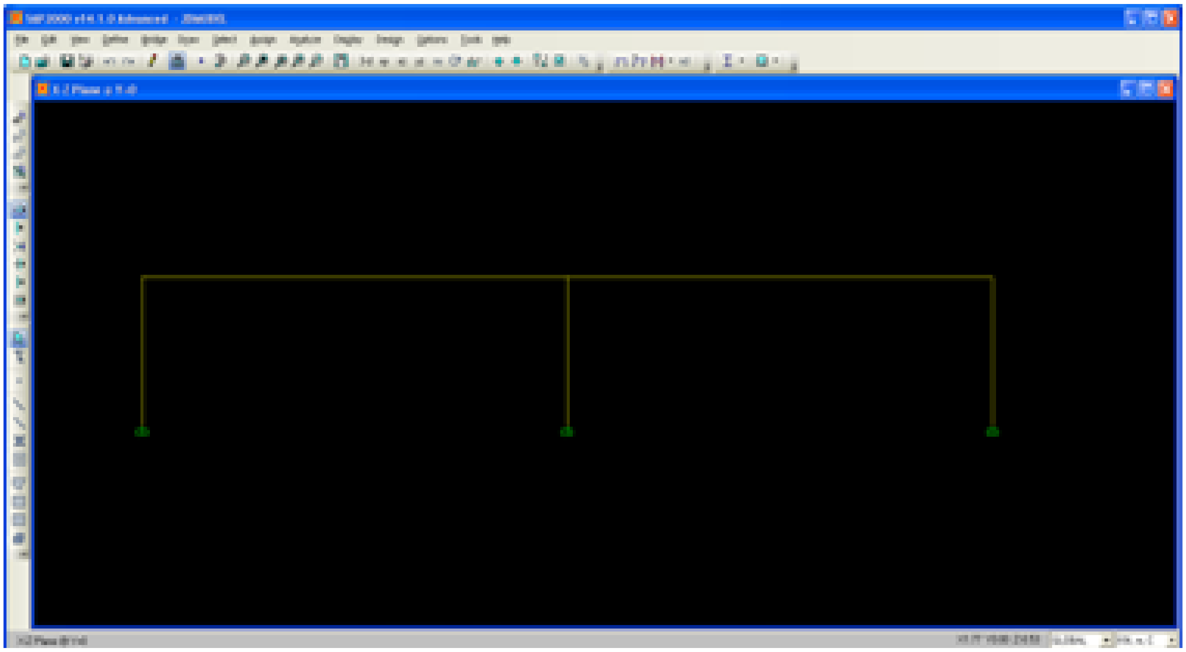


Figure 7. Finite element modelling of bridge model.

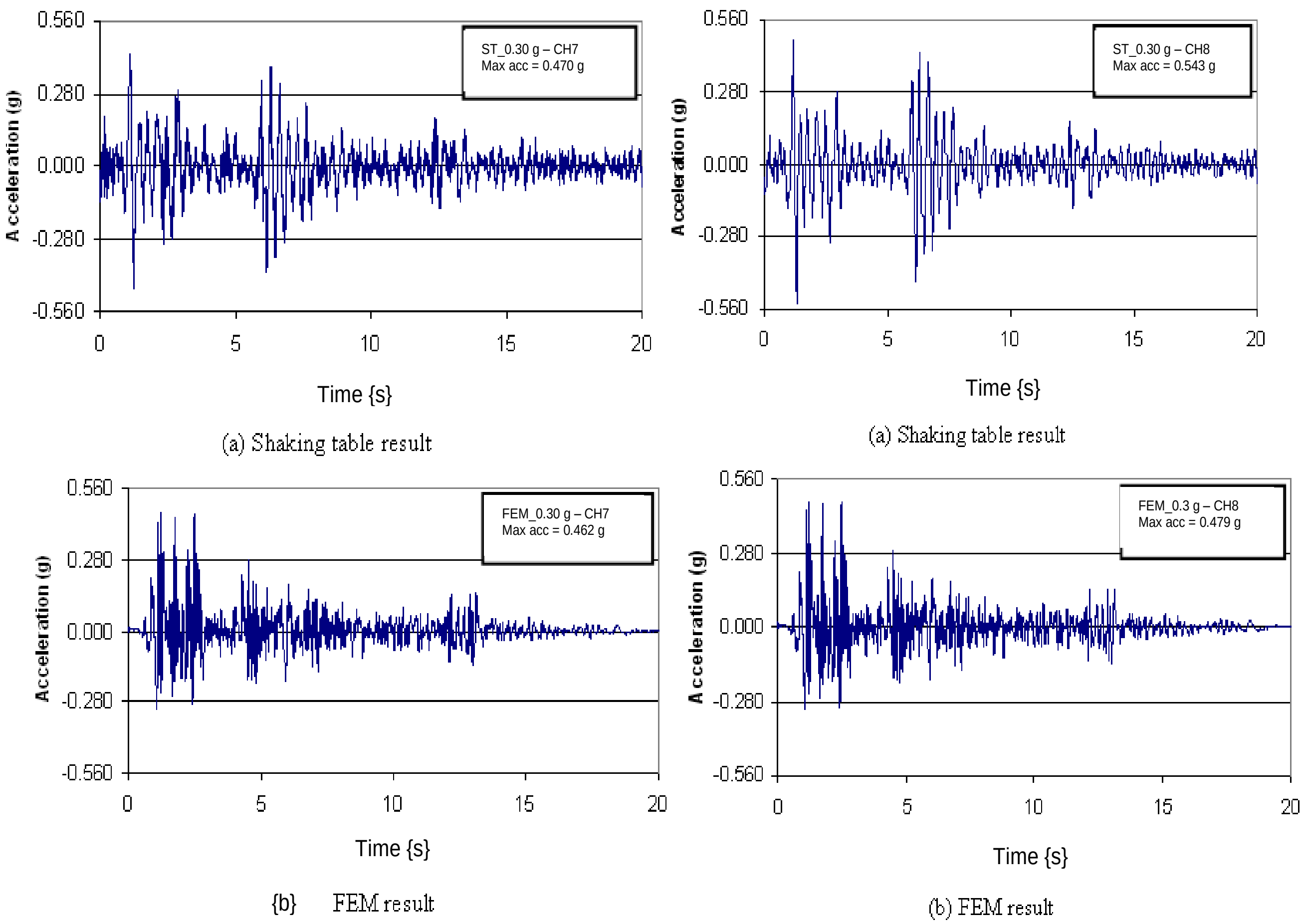
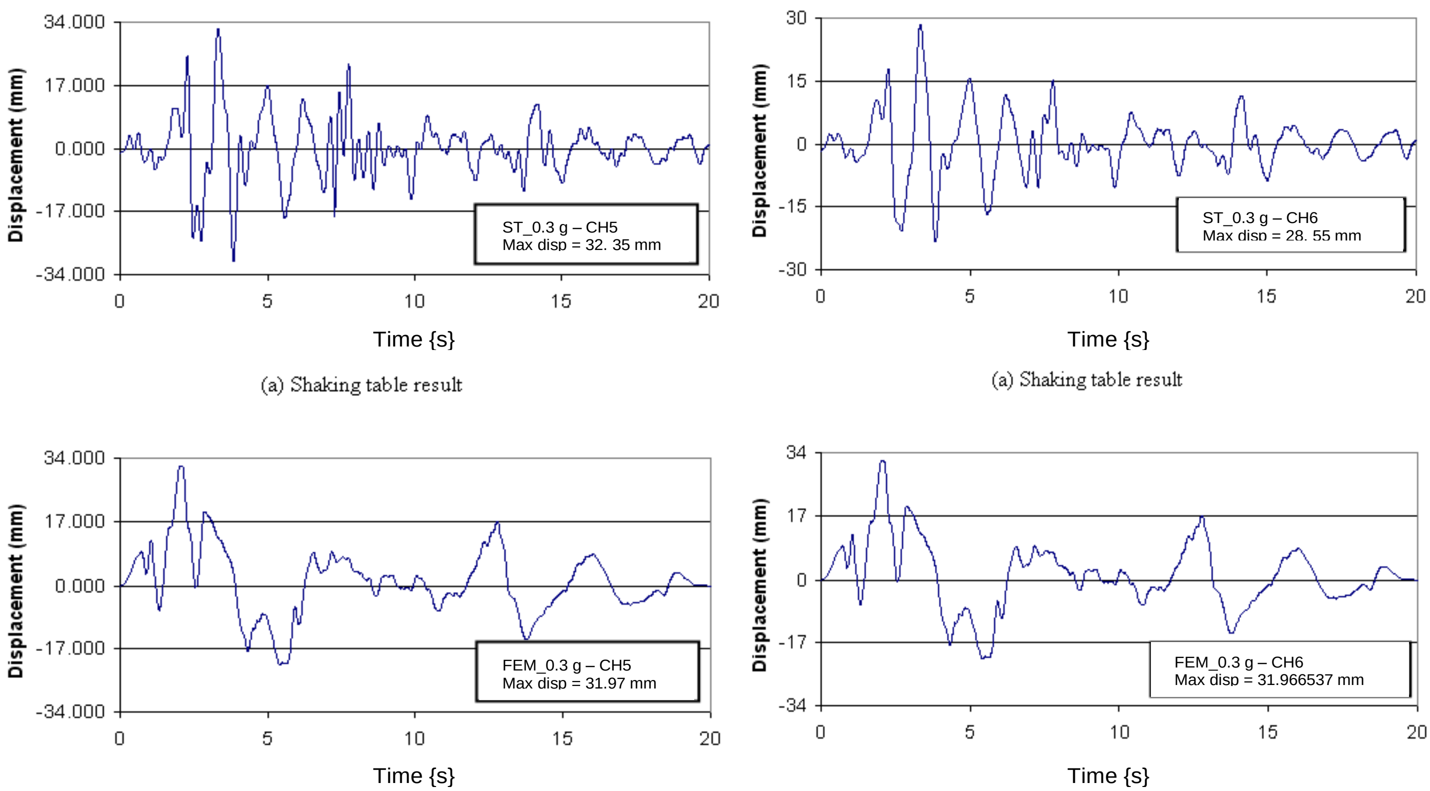


Figure 8. Acceleration response of bridge model for channels 7 and 8. Note: acc, acceleration; max, maximum.

Table 3. The comparison of bridge model acceleration response by experimental test and finite element analysis (Rapid KL bridge model).

PGA EQ (g)	Location accelerometer	Test	PGA response	% diff
0.06	CH8	ST	0.102	11.76
		FEM	0.090	
	CH7	ST	0.109	20.18
		FEM	0.087	
0.07	CH8	ST	0.130	16.15
		FEM	0.109	
	CH7	ST	0.137	23.36
		FEM	0.105	
0.10	CH8	ST	0.185	15.68
		FEM	0.156	
	CH7	ST	0.160	6.25
		FEM	0.150	
0.12	CH8	ST	0.194	3.61
		FEM	0.187	
	CH7	ST	0.208	12.98
		FEM	0.181	
0.15	CH8	ST	0.249	6.02
		FEM	0.234	
	CH7	ST	0.252	10.32
		FEM	0.226	
0.18	CH8	ST	0.279	0.71
		FEM	0.281	
	CH7	ST	0.308	12.01
		FEM	0.271	
0.20	CH8	ST	0.343	9.04
		FEM	0.312	
	CH7	ST	0.269	11.90
		FEM	0.301	
0.25	CH8	ST	0.419	23.87
		FEM	0.319	
	CH7	ST	0.376	0.53
		FEM	0.376	
0.30	CH8	ST	0.543	11.79
		FEM	0.479	
	CH7	ST	0.470	1.70
		FEM	0.462	
			Average	10.99

elasticity are assumed to be the same for each material. Fifth, in experimental test, the pier cannot be fixed perfectly at the bottom but in finite element analysis, the pier is fixed perfectly.



(a) Shaking table result (a) Shaking table result

{b} FEM result (b) FEM result

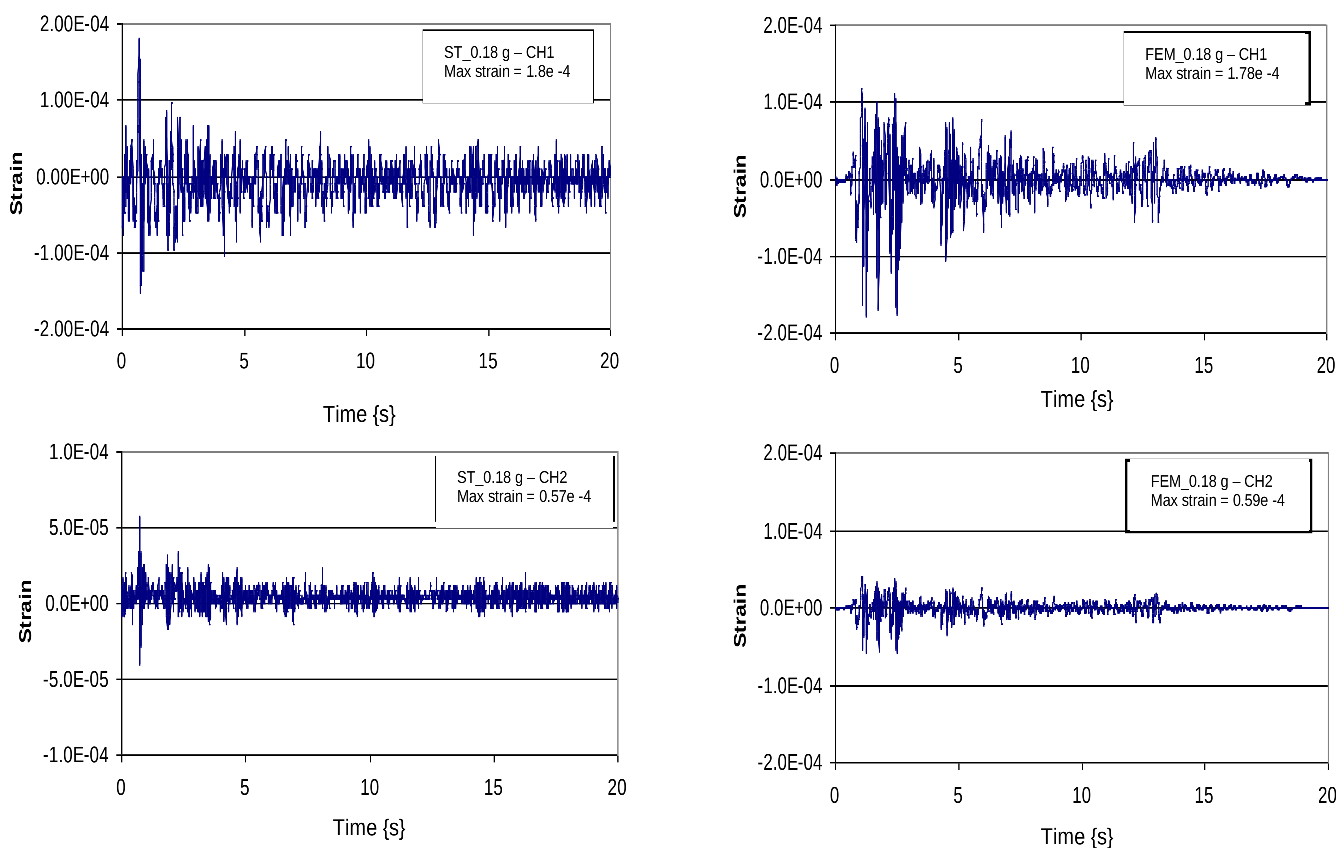
Figure 9. Displacement response of bridge model for channels 5 and 6. Note: disp, displacement; max, maximum.

Table 4. The comparison of bridge model displacement response by experimental test and finite element analysis.

PGA EQ (g)	Location LVDT	Test	Maximum displacement (mm)	% diff
0.06	CH6	ST	2.55	59.20
		FEM	6.25	
	CH5	ST	2.55	59.20
		FEM	6.25	
0.07	CH6	ST	5.30	27.30
		FEM	7.29	
	CH5	ST	5.60	23. 18
		FEM	7.29	
0.10	CH6	ST	8.05	22.67
		FEM	10.41	
	CH5	ST	9.35	10.18
		FEM	10.41	
0.12	CH6	ST	11.05	11.60
		FEM	12.50	
	CH5	ST	12.85	2.72
		FEM	12.50	

Table 4. Contd.

0.15	CH6	ST	12.70	18.69
		FEM	15.62	
	CH5	ST	16.90	7.57
		FEM	15.62	
0.18	CH6	ST	15.30	18.36
		FEM	18.74	
	CH5	ST	17.05	9.02
		FEM	18.74	
0.20	CH6	ST	16.75	19.66
		FEM	20.85	
	CH5	ST	19.70	5.69
		FEM	20.82	
0.25	CH6	ST	22.75	12.60
		FEM	26.03	
	CH5	ST	25.35	2.61
		FEM	26.03	
0.30	CH6	ST	28.55	10.70
		FEM	31.97	
	CH5	ST	32.35	1.17
		FEM	31.97	
			Average	17.90



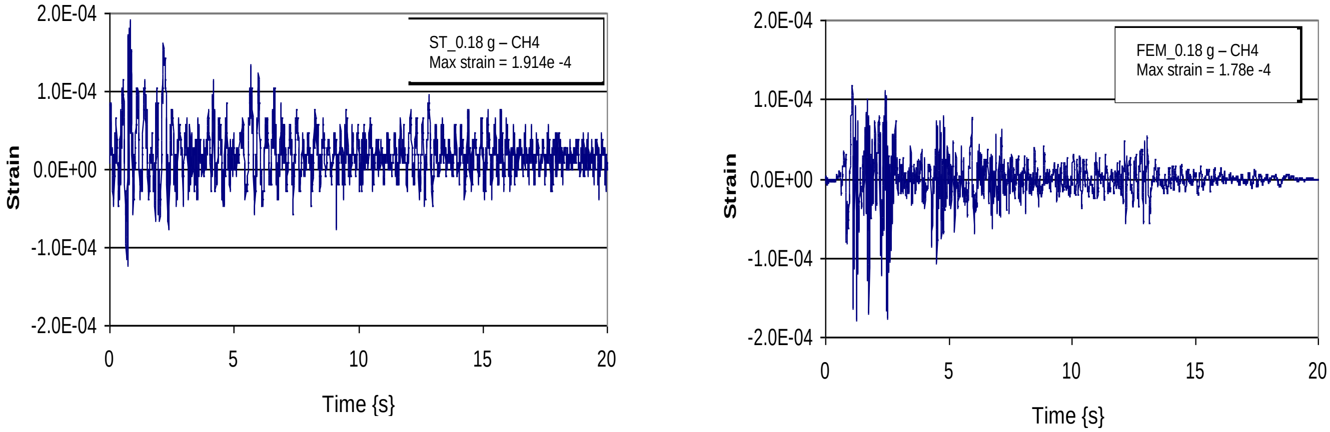


Figure 10. Strain response of bridge model by experimental test as PGA = 0.18 g.

The quality of this type of modelling which is not currently regulated depends on numerous factors. Thus, these models should take few things into account:

1) The material effects, which may cause local nonlinearities;
2) The structural effects (mass distribution and behaviour of the bond);
3) The environment effects (support-structure interaction).

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