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100學年度博士班招生考試試題

所別：工程科技研究所
科目：營建工程文獻評論

請從後面二個不同之附件中（附件 A:大地工程領域：A 題第 1 至 16 頁（Title: Seismic slope behavior in a large-scale shaking table model test）、附件 B: 結構工程與材料領域：B 題第 17 至 29 頁（Title: Earthquake!），考題總共爲 29 頁），選讀其中之一，並針對所選讀之附件回答所對應之問題。請清楚標明所選讀之附件編號。

A 題：請說明本篇文獻之：1.目的背景（20%）、2.研究內容（30%）、3.重要發現與結論（30%）、4.對此文章之批判與建議（20%）。

題目：**Seismic slope behavior in a large-scale shaking table model test**

Abstract

In this research large-scale shaking table model tests were conducted to study slope behavior under earthquake conditions. The model slope was installed into a model box with a length of 4.4 m, width of 1.3 m, and height of 1.2 m. A uniform medium sand was used and the specimen was reconstituted using the controlled-volume compaction and with a water content of 8% and unit weight of 16.6 kN/m^3. The size of the model slope is 0.5 m high, 1.3 m wide, and with a slope angle of 30°. The boundary and model slope were analyzed before testing using numerical analysis to verify the boundary conditions of the box and to ensure the proper lay-out of the model slope. The law of similitude after Iai [Iai, Suzumu (1989), Similitude for shaking table tests on soil-structure-fluid model in 1-g gravitational field, Soils and Foundations, v. 29, No. 1, pp. 105-118] and Meymand [Meymand, Philip J. (1998), Shaking Table Scale Model Tests of Nonlinear Soil-Pile-Superstructure Interaction in Soft Clay, Ph.D. dissertation, U.C. Berkeley] was applied for the determination of loading conditions. The rigidity of the model box was then calibrated accordingly.

A series of tests was performed with the designated loading frequency and amplitude. The responses of the slope remained linear with a loading amplitude of up to 0.4 g and a frequency of 8.9 Hz. Nonlinear responses were observed when the loading amplitude became larger than 0.5 g. The failure surface appeared to be fairly shallow and confined to the slope surface, which was consistent with the field observations of earthquake-induced landslides.

With such a model test with proper consideration of the law of similitude, the response and amplification behavior of a prototype slope can be studied in the laboratory. Such information could be used for further evaluation of the slope failure caused by an earthquake in the field, and for the study of the behavior of important slopes such as earth dams under seismic loading conditions.

1. Introduction

In 1999, the Chi-Chi earthquake with a moment magnitude of 7.6 struck central Taiwan and induced extensive slope failures, which led to severe damage to the highway system and private houses. Slope failures induced by earthquakes have been broadly investigated and discussed in the past. Typically, analyses of the dynamic slope stability are performed using three different methods: the pseudo-static method, Newmark's method, and ground response analysis. Terzaghi (1950) proposed the critical equilibrium method to calculate the factor of safety using a pseudo-static force acting on a slope under horizontal acceleration. The limit analysis method of Chen and Snitbhan (1975) and pseudo-static analysis by Seed (1979) used the same

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Fig. 1. Compaction of the specimen.

concept of pseudo-static analysis of slope stability. Newmark (1965) proposed a sliding block model to simulate slope behavior under seismic forces, in which a rigid perfect plastic sliding block was assumed, and the block moved when acceleration exceeded the critical acceleration of the slope.

However, the previous two methods do not take into account the dynamic slope behavior, and the effects become significant when soil amplification occurs. On the other hand, numerical analysis such as the finite element method and finite difference method (Clough and Chopra, 1966) can take into account the dynamic ground responses as well as the nonlinear behavior of soil, and the elastic–plastic constitutive laws can be applied. The shear beam method (Mononobe, 1936; Ambraseys and Sarama, 1967) simplifies a 2-dimensional dam into a 1-dimensional analysis, and acceleration along the height of the dam can be obtained quickly. However, the failure of the slope and its dynamic behavior are still not well understood.

Table 1
Results of the direct shear tests on the specimens obtained from different positions of the model slope

Position	Water content (%)	Unit weight (kN/m^3)	Cohesion (kPa)	Friction angle (°)
Up-slope	1.7	15.4	13.2	35.8
Surface of the slope	2.8	15.2	6.2	37
Down-slope	5.1	16.6	11.8	38.2

Two types of tests have been used for the dynamic slope model tests: the centrifuge test and the shaking table test. The centrifuge test utilizes the gravity force as the scale factor to simulate a prototype slope. Kutter (1982) used a centrifuge slope model test to study the dynamic behavior of a slope and concluded that plastic deformation contributed to an increase of damping, the slope behaved nonlinearly, and yield acceleration of the slope resulted in a strain softening effect. Wartman et al. (2001) used a small shaking table to simulate slope behavior under a 1 g gravity field. The resulting displacement data of the slope were compared with Newmark's displacements based on the peak and residual soil strength, respectively. It was

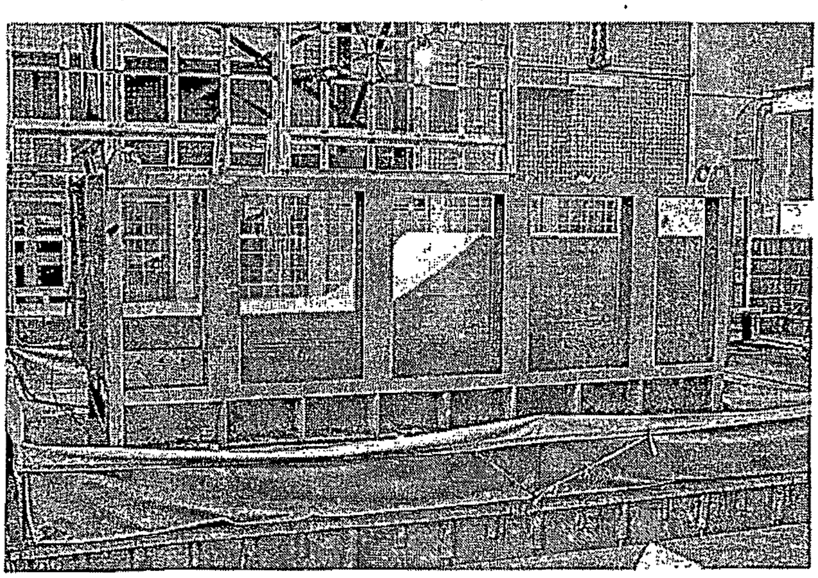


Fig. 2. The model slope before the test.

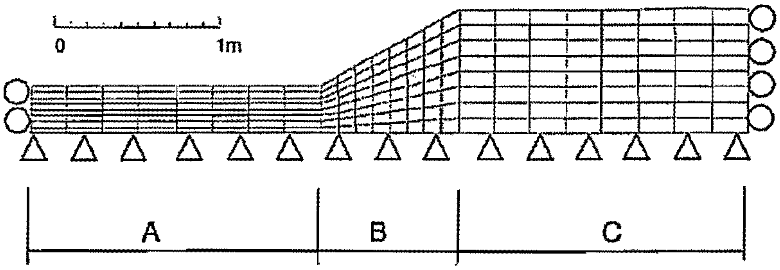


Fig. 3. The mesh and boundary conditions used for the model slope (unit: m).

suggested that the actual displacement of the slope was in the range between the results of analyses using the peak and residual soil strength.

The objective of this research is to study the dynamic slope behavior and responses using a shaking table model test to simulate an earthquake induced landslide in a prototype slope. The law of similitude after Iai (1989) and Meymand (1998) was adopted for the determination of the dynamic testing conditions of the model slope. The resulting dynamic behaviors of the slope were observed along with the recorded accelerations in the slope specimen, the failure surface and cracks at the top of the slope. A numerical analysis was performed to calibrate the dynamic properties of the model and to validate the law of similitude used in this study.

2. Specimen preparation and material properties

The soil used in this study is a uniform medium sand and is classified as poorly graded sand (SP) according to the Unified Soil Classification System. The soil was prepared by mixing with water to reach 8% water content, and then cured for 24 h. The soil specimen was compacted into the model box using the controlled-volume method and with a unit weight of 16.6 kN/m^3; the procedures of compaction are illustrated in Fig. 1. The slope surface was compacted by a modeling tool to keep the slope angle at the designated value. The final slope specimen is shown in Fig. 2 with a height of 0.5 m, a width of 1.3 m, and a slope angle of 30°; the total weight of the soil used for the test is 51 kN.

Direct shear tests were performed on the specimens obtained from the model slope; the shearing strength parameters of the material are shown in Table 1. Due to the migration and evaporation of moisture during the test, the water content of each specimen ranged from 1.7% to 5.1%.

Fig. 4. The slip surface of numerical simulation before the experiment.

The cohesion of the soil sample ranged from 6.2 to 13.2 kPa and the friction angle varied from 35.8° to 38.2°. The cohesion and friction angle of the sample reconstituted in the laboratory were 20.3 kPa and 37.6°, respectively.

The shear wave velocity of the slope specimen was also determined based on the traveling time measured by sensors aligned in the proper direction inside the slope specimen before and after the shaking table test. The shear wave velocity of the specimen before the test thus determined was approximately 133 m/sec. This shear wave velocity is within the reasonable range of a medium sand, as the experimental results reported by Andrus and Stokoe (1998) ranged from 90 to 270 m/s. Soil with a shear wave velocity larger than 200 m/s is considered as dense soil by Andrus and Stokoe (1998). A similar result for shear wave velocity was obtained after the shaking table test was performed.

3. Set-up of experiment

In order to properly set up the experiment, the boundary conditions and law of similitude need to be considered. A numerical model of the slope specimen was constructed and analysis was performed using a commercially available program FLAC to study the effects of boundary confinement on the slope specimen. FLAC used in this study is a two-dimensional explicit finite difference solution for numerical analysis. This program simulates the behavior of the material that may undergo plastic flow after reaching yield limits. The material can yield and undergo plastic flow, and the grid can deform in large-strain mode (FLAC, 2000). The mesh and boundary conditions of the model slope are shown in Fig. 3. The friction angle is 33°, cohesion is 1 kPa and unit weight is

Table 2
The law of similitude after Meymand (1998)

Mass density	1	Acceleration	1	Length	λ
Force	λ^3	Shear wave velocity	$\lambda^{1/2}$	Stress	λ
Stiffness	λ^2	Time	$\lambda^{1/2}$	Strain	1
Modulus	λ	Frequency	$\lambda^{-1/2}$	EI	λ^5

Table 3
Relationship of similarity between the prototype slope and model slope

	Prototype	Model
Unit weight (kN/m^3)	16.6	16.6
Acceleration (g)	1	1
Slope height (m)	10	0.5
Base thickness (m)	6	0.3
Frequency (Hz)	2	8.9
Strain	1	1

17 kN/m^3. Based on the resulting location of the sliding surface with respect to the boundary as illustrated in Fig. 4, it was found that the boundary would not affect the location of the sliding surface when the distance of the slope crest to the boundary (C in Fig. 3) is larger than two times the horizontal projected slope length (B in Fig. 3), and with the thickness of the base layer of 30 cm.

In order to simulate the prototype slope in the model test, the law of similitude is applied. Shunzo (1973) mentioned the law of similitude based on the static aspect, which didn't consider the dynamic properties of the soil. In the law of similitude, the loading speed of the model is faster than that of the prototype and the strength of the model material must be reduced to observe the similitude. Kagawa (1978) considered the ratio of forces acting on the prototype and model, and suggested the law of similitude for the dynamic testing of a soil structure specimen. Kagawa (1978) adopted the statistical results of dynamic tri-axial tests suggesting that the loading frequency relationship between the model and the prototype is

$$\frac{\omega_m}{\omega_p} = \lambda^{3/4} \qquad (1)$$

in which,

ω_m the loading frequency of the model
ω_p the loading frequency of the prototype

However, the law of similitude proposed by Shunzo (1973) didn't consider the dynamic properties of the soil. The law of similitude developed by Kagawa (1978) is only applicable for the shear deformation of soil structures (Iai, 1989).

In this study, the law of similitude was developed based on the factor considered as most important in the simulation. Iai (1989) derived a similitude relation with the basic equation governing the equilibrium and mass balance of the soil skeleton, pore water, pile and sheet pile structures, and external waters such as the sea. Meymand (1998) considered the law of similitude derived by Iai (1989) and used it in the simulation of the seismic pile behavior in saturated clay. The main aspect of this law of similitude was keeping the soil density the same for both the prototype and model, which would simplify the needs of scaling of parameters in the 1 g model testing. The corresponding scaling of parameters between the prototype and model used in this experiment are derived as follows, and the results are listed in Table 2.

1. Force

$$\frac{F_p}{F_m} = \frac{m_p a_p}{m_m a_m} = \frac{\rho_p V_p^3}{\rho_m V_m^3} = \frac{\lambda^3}{1} = \lambda^3 \qquad (2)$$

The subscript p denotes prototype and the subscript m denotes model, in which,

F force
m mass
a acceleration
ρ density
V volume
λ the linear scale ratio between the prototype and model

2. Stress

$$\frac{\sigma_p}{\sigma_m} = \frac{F_p/A_p}{F_m/A_m} = \frac{\lambda^3}{\lambda^2} = \lambda \qquad (3)$$

in which,

σ normal stress
A area

3. Young's modulus

$$\frac{E_p}{E_m} = \frac{\sigma_p \varepsilon_m}{\sigma_m \varepsilon_p} = \frac{\lambda}{1} = \lambda \qquad (4)$$

in which,

E Young's modulus
ε normal strain

4. Shear wave velocity

$$\frac{V_{sp}}{V_{sm}} = \frac{\sqrt{G_p/\rho_p}}{\sqrt{G_m/\rho_m}} = \sqrt{\frac{G_p}{G_m}} = \sqrt{\lambda} \qquad (5)$$

in which,

V_s shear wave velocity
G shear modulus

5. Time

$$\frac{T_p}{T_m} = \frac{L_p/V_{sp}}{L_m/V_{sm}} = \frac{L_p}{L_m} \cdot \frac{V_{sm}}{V_{sp}} = \lambda \cdot \lambda^{-1/2} = \lambda^{1/2} \qquad (6)$$

in which,

T time
L length

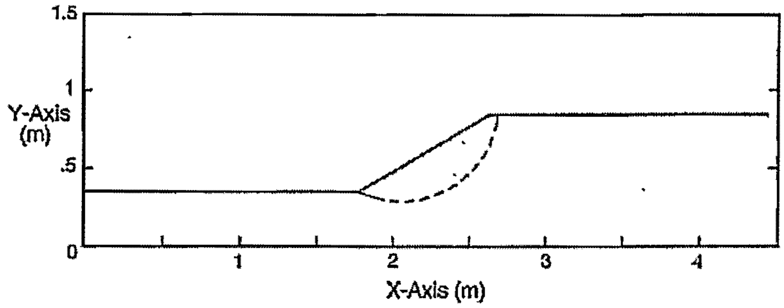


Fig. 5. Results of the pseudo-static stability analysis of the prototype slope.

However, Iai (1989) developed the law of similitude based on several assumptions: (1) the soil skeleton is regarded as a continuous medium, (2) the deformation is assumed to be small so that the equilibrium equation remains the same before and after the deformation, and (3) the strain of the soil skeleton is small. Therefore, the law of similitude is suitable for the status before the failure of the slope specimen. Note that in Table 2, the modulus and shear wave velocity of the material in the model test needed to be scaled down to the proper ratio, which was difficult to achieve for the sand specimen in this study. However, the strain ratio of the model and prototype was not affected by this problem, and using strain to characterize the slope behavior appeared to be reasonable. Therefore, the law of similitude was adopted even though it was only partially fulfilled in this study.

The scaling factor λ used in this experiment is 20. Based on the similarity requirement, the controlling factor used in the model test is frequency. The relationship between the prototype and model is shown in Table 3. The slope height of 0.5 m is used in the model in order to simulate a 10 m high prototype slope.

To determine the loading sequence for the experiment, it was essential to define the prototype slope and the loading conditions to be modeled. The potential critical condition at which the prototype slope fails was analyzed using the pseudo-static stability analysis, and the results for the prototype slope were shown in Fig. 5. The coefficient of critical horizontal acceleration thus determined was k_{hy}=0.532 when the factor of safety reached 1.0. Note that the most critical failure circle is a toe circle, and the depth to the failure surface is fairly large owing to the cohesion component of soil strength. Thus the loading sequences were designed with several steps of acceleration amplitude gradually increasing from 0.1 to 0.6 g, and a sinusoidal waveform was used as illustrated in Fig. 6.

The experiment was performed on the shaking table of the National Center for Research in Earthquake Engineering in Taiwan. The system has a payload capacity of 50 tons and the maximum loading frequency is 50 Hz. A total number of 17 sensors were used for measurements during the test;

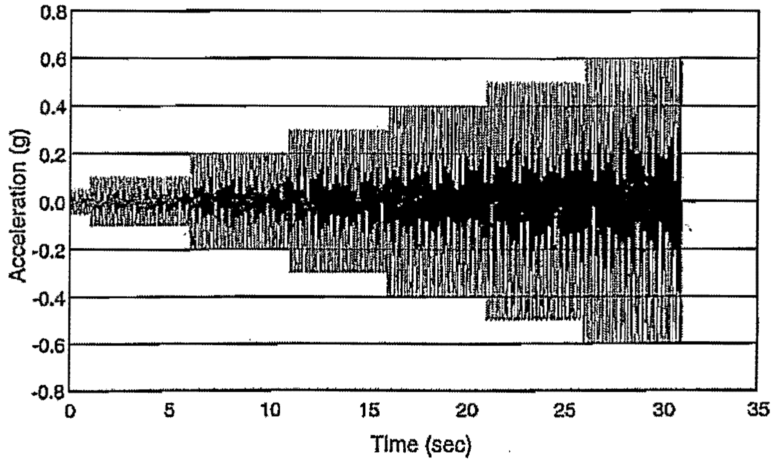


Fig. 6. Sinusoidal acceleration loading history.

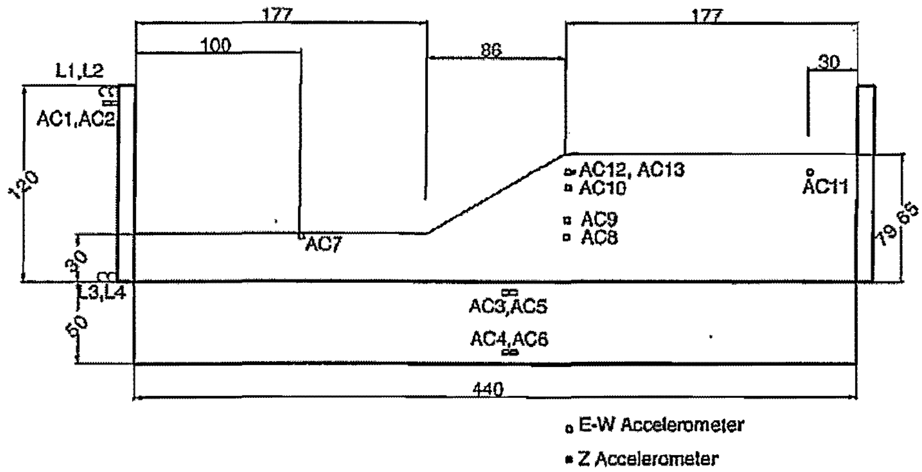


Fig. 7. The layout of instrumentation with AC indicating the accelerometer, and L indicating LVDT. (unit: cm).

among them 7 accelerometers were embedded at different elevations inside the model slope, 6 accelerometers were fixed on the outside of the box, and 4 linear variable differential transformers(LVDT) were set up at four corners of the model box to record box displacement. A layout of the instruments was shown in Fig. 7.

4. System calibration and stiffness

In order to make sure that the amplification effect produced by the system is negligible, the system was tested and its stiffness was determined. The layout of the instrumentation is slightly different from that of the model test as shown in Fig. 8, and system stiffness can be determined based on the measured responses. The loading sequence used for the system calibration is shown in Fig. 9 with an amplitude of 0.3 g and loading frequency of 8.9 Hz, which is the same frequency for calibrating the responses of the model slope.

The dynamic response of the system was modeled as a single degree of freedom system, which is composed of one spring with stiffness k and one dash pot with damping c. The weight of the system is 39,838 kN. With the given loading sequence, a typical set of system responses is shown in Fig. 10. Based on Fig. 10, no significant amplification of the system was observed during the loading, and the system appeared to behave linearly throughout the loading history. The system damping and stiffness can be computed based on the single degree of freedom system solution. The stiffness of the system thus computed is 551.9 kN/m, the system damping is 174.3 kN.sec/m, and the fundamental frequency of the system is 0.49 Hz, which is very different from

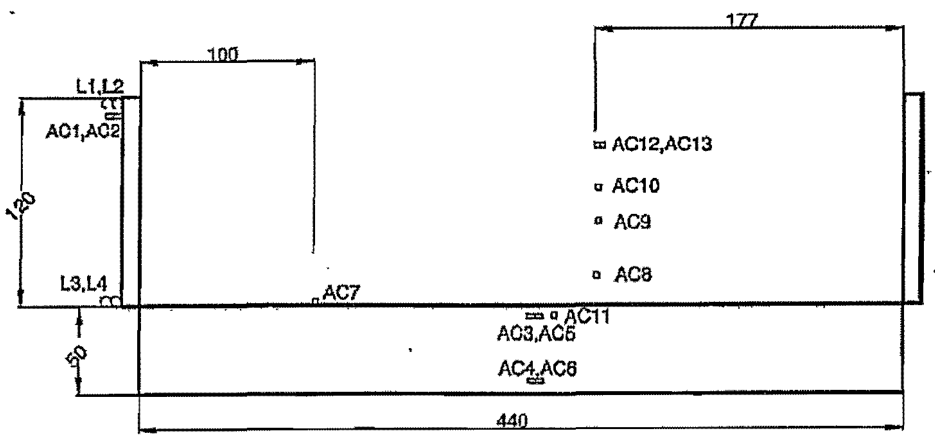


Fig. 8. The layout of instrumentation for the system calibration, (unit: cm).

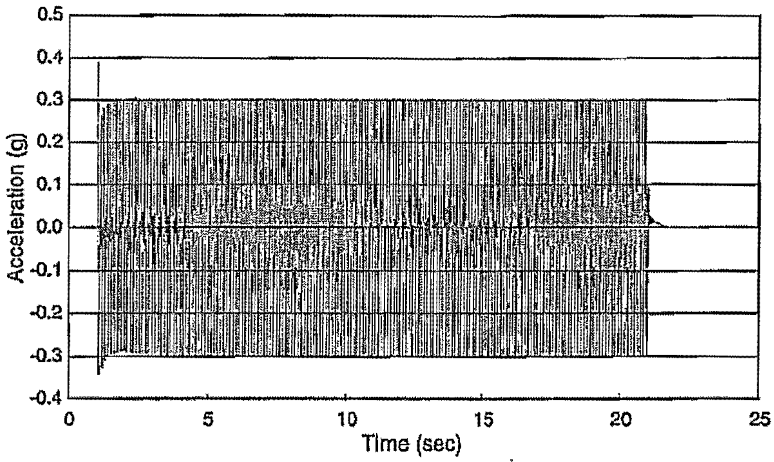


Fig. 9. The loading sequence for the system calibration.

the applied frequency of 8.9 Hz, and would not cause resonance of the system.

5. Responses of the model slope

The recorded ground response acceleration history with peak input acceleration amplitude increased from 0.1 to 0.4 g as shown in Fig. 11. The responses at AC8 which was located at the same level as the base of the slope displayed a significant amount of increasing amplification effect as the input amplitude increased from 0.3 to 0.4 g, while the responses of AC12 which was located near the top of the slope indicated a more significant amount of amplification starting with the input amplitude of 0.3 g, and a higher mode response could be observed. By comparing the responses of the accelerometers moving from the base of the slope toward the crest, the amplification effects and nonlinear responses appeared to become increasingly significant moving from the base toward the crest.

Similar behaviors with increasing effects of amplification can be observed for the input acceleration amplitude increasing from 0.4 to 0.5 g in Fig. 12 and 0.5 to 0.6 g in Fig. 13. As the input acceleration amplitude reaches 0.5 g, the response at AC8 also displays a higher mode response. This behavior of the slope could be induced by the nonlinear soil properties and degradation of the modulus as the input acceleration approached the higher stress level and caused failure of the slope. In Fig. 13, the response of AC12 near the crest became significantly amplified, and the amplitude toward the outward slope direction appeared to be larger than toward the inward slope direction. Such behavior could imply the development of a sliding surface and separation of the soil body near the crest. Thus the

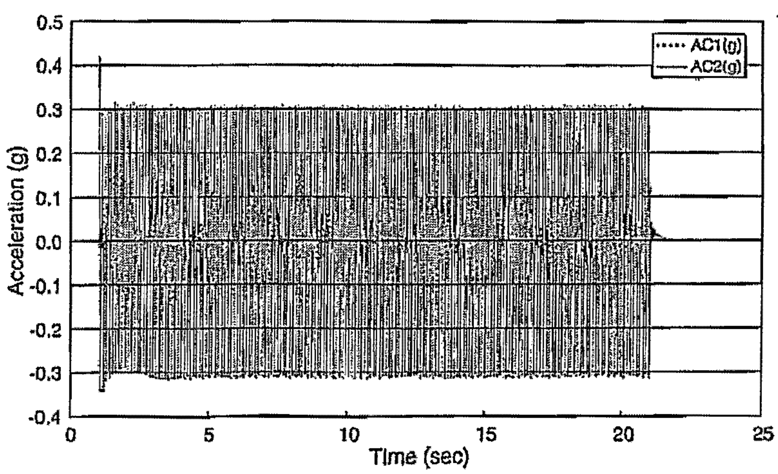
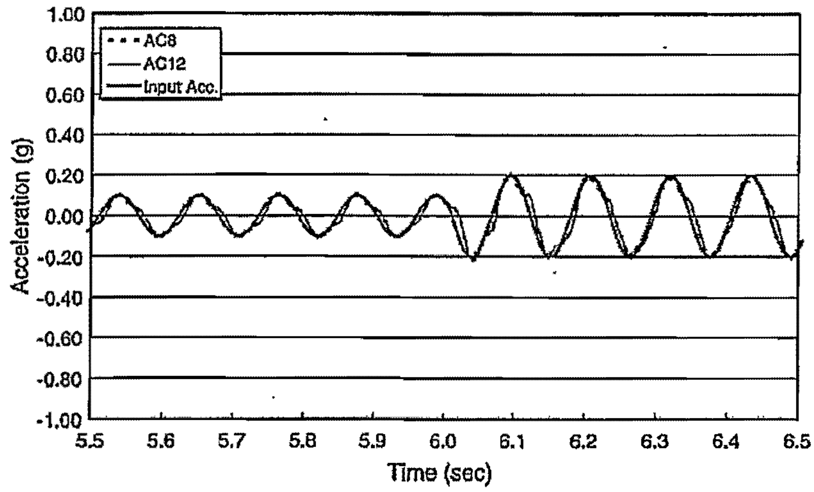
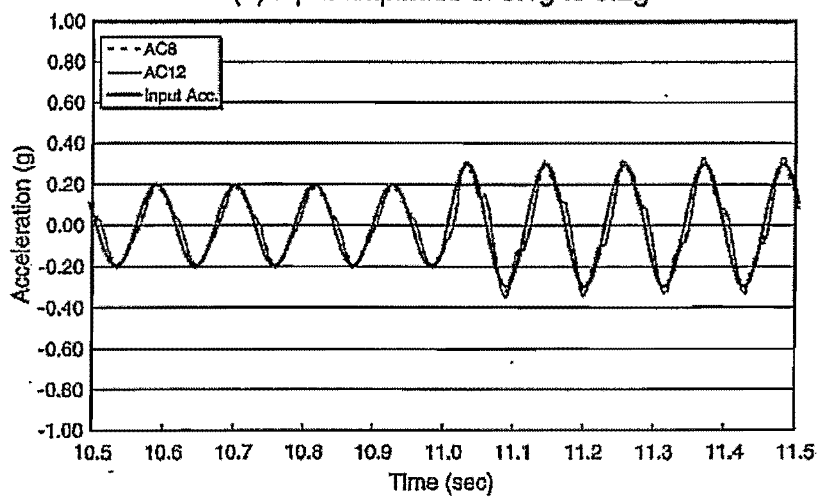


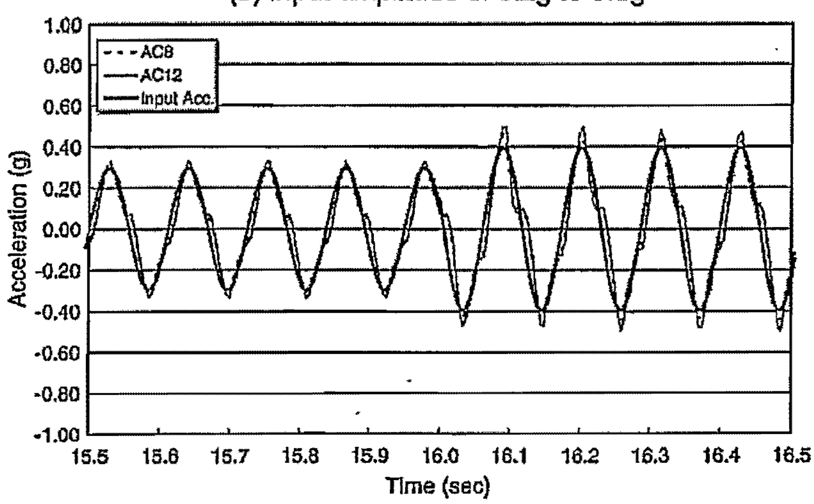
Fig. 10. Response of the system under calibration (locations of AC1 and AC2 as shown in Fig. 8).



(a) Input amplitude of 0.1g to 0.2g



(b) Input amplitude of 0.2g to 0.3g



(c) Input amplitude of 0.3g to 0.4g

Fig. 11. Response history of AC8 and AC12 with the input amplitude increasing from (a) 0.1 to 0.2 g, (b) 0.2 to 0.3 g and (c) 0.3 to 0.4 g.

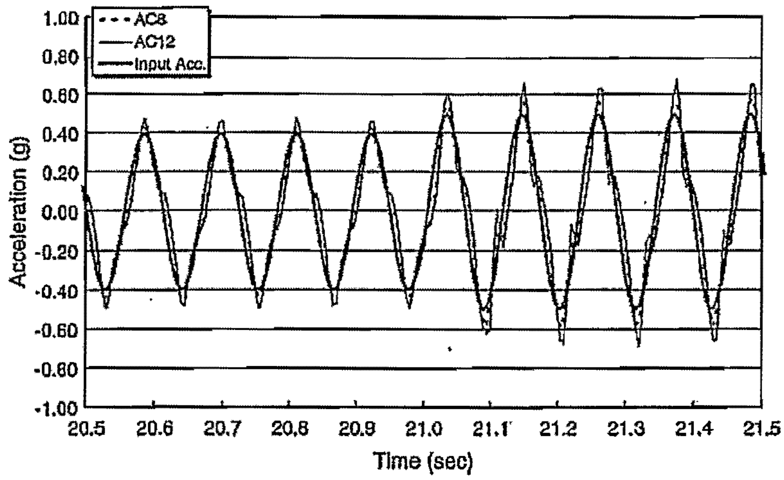


Fig. 12. Response history of AC8 and AC12 with the input amplitude increasing from 0.4 to 0.5 g.

observed amplification effect near the top of the slope could accelerate the development of a failure surface near the crest of the slope.

The response acceleration history of the slope with input acceleration amplitude of 0.6 g is shown in Fig. 14. The accelerometer responses of AC8 through AC12 have similar shapes except that as the elevation gradually increases from the base of the slope to the crest of the slope, the amplification effect, nonlinear behaviors, and displaying of a higher mode become more and more significant. However, the acceleration response at AC11 appeared to be a little smaller than those of AC10 and AC12, which could be due to the end confinement of the rigid box as illustrated in the layout of instrumentation.

Based on the response history recorded, the amplification factors between AC12 and AC7, and AC12 and AC8 are calculated, respectively, and results are shown in Fig. 15. The amplification factor between AC12 and AC7 remained about the same with increasing input acceleration amplitude up to about 0.4 g, and then increased rapidly to about 0.5 g, indicating a nonlinear soil response when the acceleration amplitude became larger than 0.4 g. As the acceleration amplitude increased from 0.5 to 0.6 g, the calculated amplification factor decreased rapidly, which implied the development of a slip surface and separation of the sliding body and the slope. A similar trend of variation was observed for the amplification factor between AC12 and AC8. The amplification factors of AC8 appeared to be lower than those of AC7, owing to the differences in the location of measurement. However, the amplification factor for AC7 became smaller than that of AC8 when the acceleration amplitude increased from 0.5 to 0.6 g.

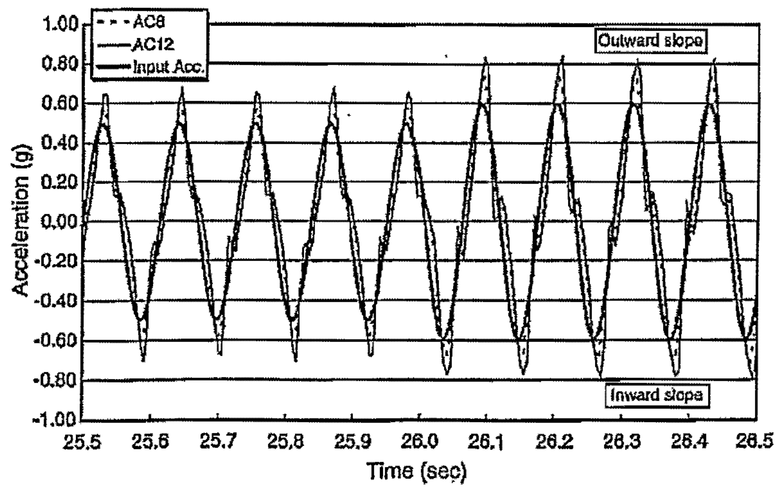


Fig. 13. Response history of AC8 and AC12 with the input amplitude increasing from 0.5 to 0.6 g.

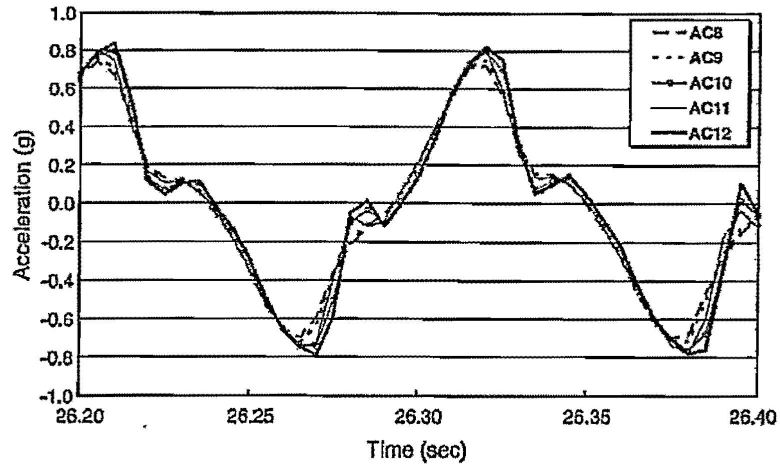


Fig. 14. Response history of the slope with the input acceleration amplitude of 0.6 g.

Observing that the location of AC7 is outside the slope, the difference in amplification could imply the development of a slip surface and separation of a sliding soil body from the slope. The responses of the slope specimen became nonlinear and amplified when the peak input acceleration became larger than 0.4 g, and the initiation of slope failure occurred with acceleration between 0.5 to 0.6 g. Such observation coincided with the measured responses of AC12 shown in Fig. 13 as discussed in the previous section. Although no fissure or crack was observed at this stage, the range of acceleration for failure initiation is consistent with the critical acceleration of 0.53 g determined by the pseudo-static analysis.

6. Development of the failure surface

Although the significant nonlinear responses and amplification factors observed with acceleration amplitude up to 0.6 g both implied a potential failure of the slope, no slip surface was observed at this stage. Thus, a second loading sequence with the amplitude quickly increased to 0.6 g as shown in Fig. 16 was applied to explore the development of a possible slip surface induced by the dynamic loading. The system was restored to its initial position before the second loading sequence was applied. The slip surface developed rapidly with this second loading sequence, and the shape and location of the slip surface were shown in Fig. 17. Observing Fig. 17, the slip surface appeared to be close to a circular shape and fairly shallow.

The profiles of the slope specimen were mapped on both sides before and after the test as shown in Fig. 18. The observed slope failure surface appeared to be quite shallow and in the upper part of the slope, and the sketch from the north side in Fig. 18 indicated that the failure surface was somewhat circular. Although the specimen was carefully excavated after the test, the shear surface inside the

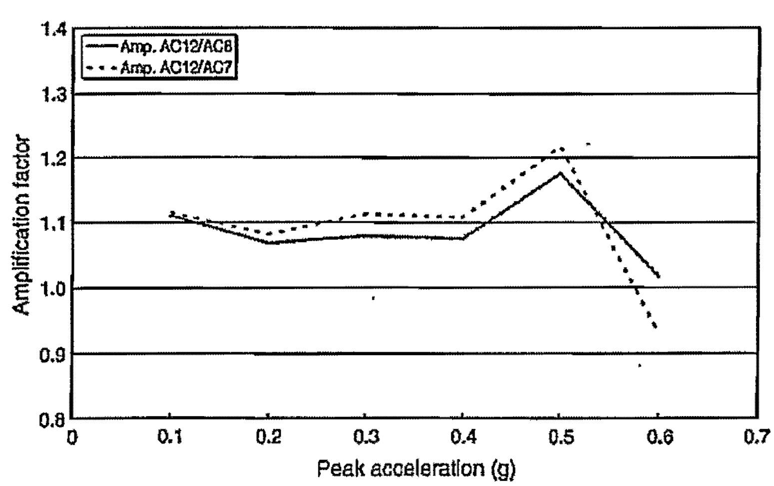


Fig. 15. The amplification factor between AC12 and AC7, AC8.

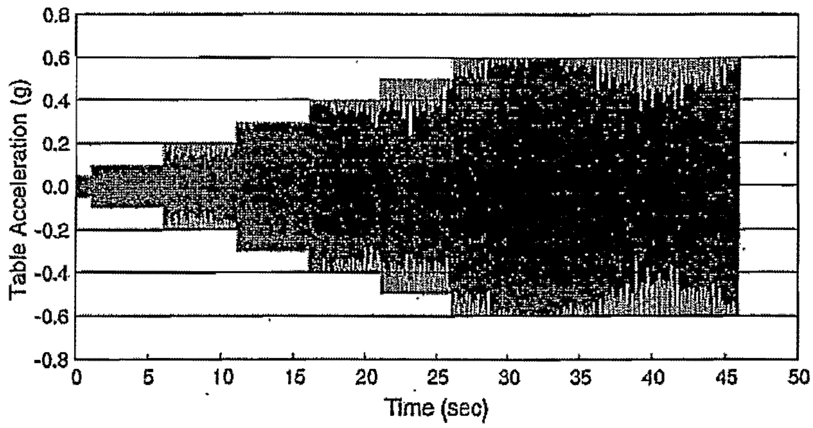


Fig. 16. The second loading sequence applied to the specimen to explore the slip surface.

specimen could not be defined well. The top view of the slope from the edge of the slope crest to the boundary of the model box was also mapped after failure as shown in Fig. 19. The crown of the failure surface close to the crest was more or less circular, and some cracks near the boundary and between the major failure surface and the boundary zone also developed. It was likely that more than one set of failure surfaces might have developed during the post-failure loading stage. Although some cracks developed near the box boundary appeared to be caused by the boundary confinement, the cracks did not interfere with the development of the major failure surface. Comparing the observed slip surface to that of the pseudo-static critical circle, the location and depth of the failure surfaces are very different. The model slope failure surface may have occurred so close to the slope crest because of the increasing amplification of motion as the elevation approaches the crest of the slope. Generally, this failure surface appeared to be quite shallow, in somewhat circular shape, and close to the crest of the slope, and these features were consistent with the slope failures observed in the field after the Chi-Chi earthquake (NCREE, 1999).

7. Analysis of the model slope

In order to analyze the test results for the model slope, a finite difference method is adopted in this research. The first step is the determination of the strength parameters and shear modulus of the material. According to the FLAC user manual, when using the Mohr–Coulomb

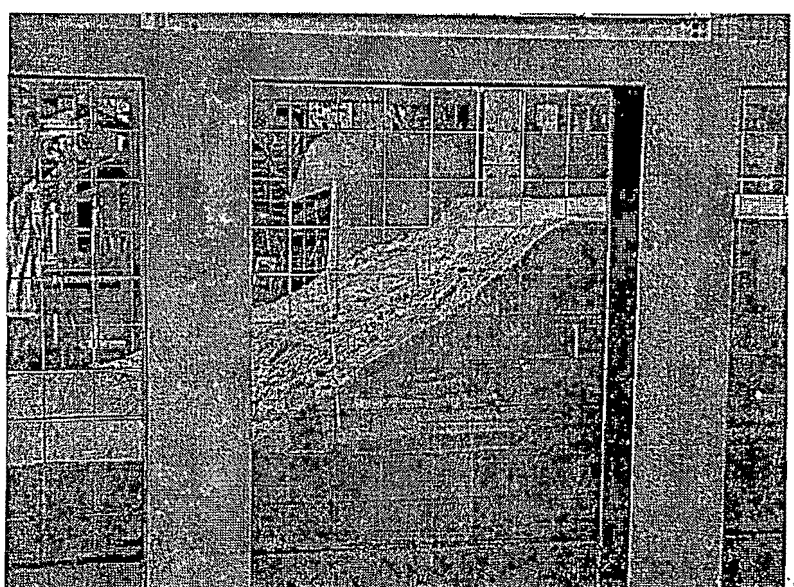


Fig. 17. Failed model slope after the application of the second loading sequence.

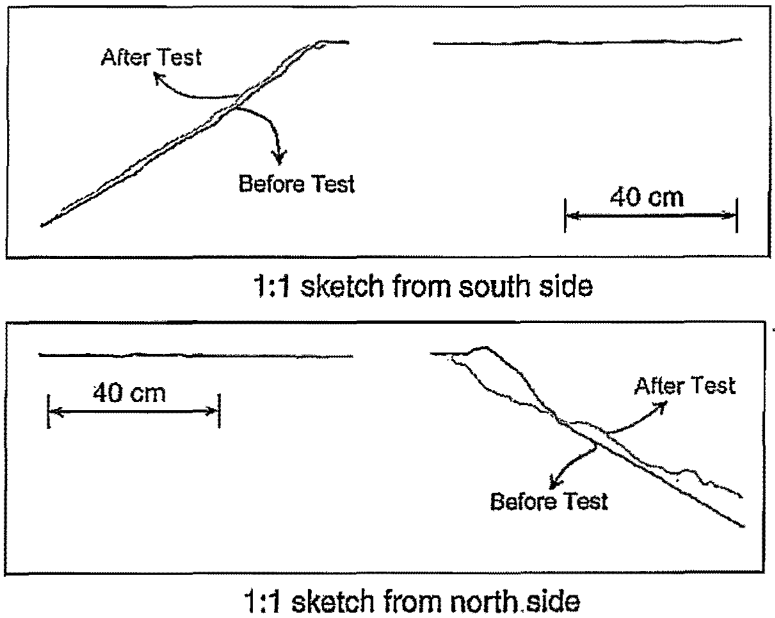


Fig. 18. Side views of the slope specimen before and after the test.

model, only a minimum percentage of damping may be required. The damping in the numerical analysis is 0.5%. Strength parameters were obtained by direct shear tests as described above. Initial shear modulus can be determined in the following ways: the empirical equation of Hardin and Drnevich (1972), the empirical equation of Assimaki et al. (2000), and the measured shear wave velocity.

Based on the testing results of the soil subjected to cyclic load, Hardin and Drnevich (1972) proposed the following equation.

$$G_0 = 3230\frac{(2.973 - e)^2}{(1 + e)}(\sigma_m')^{0.5}(\mathrm{OCR})^K \tag{7}$$

in which:

G_0 maximum shear modulus (kPa)
e void ratio
σ_m' mean effective confining pressure (kPa)
OCR over consolidation ratio
K coefficient depending on soil plasticity

Assimaki et al. (2000) used the test results from Laird and Stokoe (1993) on granular soil and suggested that the initial shear modulus can be determined as:

$$G_{max} = 107,700\ \sigma_0^{0.4426} \tag{8}$$

in which:

G_{max} maximum shear modulus (kPa)
σ confining pressure (100 kPa)

The shear modulus can be obtained using the shear wave velocity as shown in the following equation.

$$G = \rho V_s^2 \tag{9}$$

in which:

G shear modulus
ρ density of soil
V_s shear wave velocity

The shear wave velocity measurement taken before conducting the shaking table test was used in the computation of the shear modulus using Eq. (9).

No amplification of the model box was considered as the system behaved rigidly during system calibration, and the measured accelerations at both sidewalls were the same as that of the base. The cohesion and friction angle of the soil are 1 kPa and 33.5°, respectively, and the unit weight of the soil is 16.7 kN/m^3. The parameters are obtained from the direct shear test of the reconstituted specimens. Comparing the measured stiffness of model box with that of the slope, the stiffness of model box is relatively rigid to prevent amplification effects due to the box itself. The input acceleration was applied at the bottom as well as at both sides of the specimen. A sinusoidal acceleration with a frequency of 8.9 Hz and a peak acceleration of 0.4 g was applied to the specimen. Again the finite difference program FLAC was used for

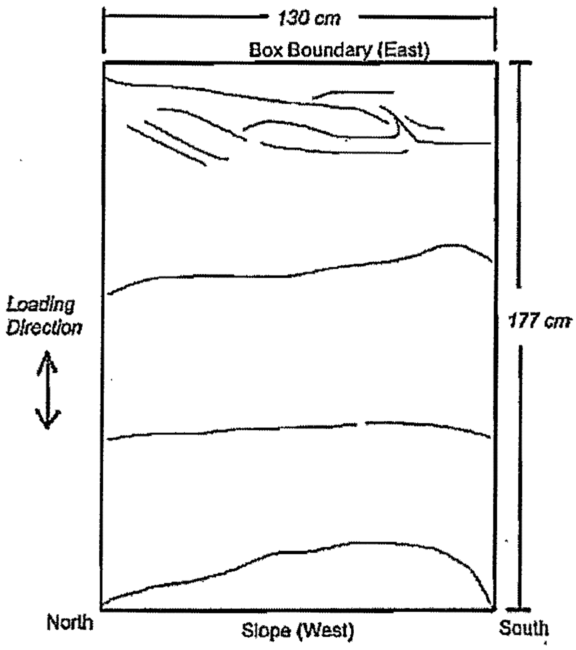


Fig. 19. Top view of the slope from the edge of the crest to the box boundary after failure.

numerical analysis, and the equivalent–linear method (Seed and Idriss, 1969) was adopted with the assumed initial shear modulus and damping ratio.

The amplification factors between the slope crest and base calculated using different moduli are listed in Table 4. The initial shear modulus of Hardin and Drnevich (1972) yielded the largest value of amplification but was still smaller than the amplification factor from the directly measured amplification, which was about 1.1. The initial modulus determined using the other two methods provided amplification factors smaller than the modulus by Hardin and Drnevich (1972). This phenomenon could be caused by the nonlinear soil behavior and degradation of the modulus. Due to the equivalent linear model used in the analysis, the adjustment of the modulus was required to reflect properly the amplification condition during the experiment.

The shear modulus of Hardin and Drnevich (1972) was used, the modulus was adjusted from 0.31 to 1.0 times the initial value listed in Table 4, and the amplification condition was analyzed. Fig. 20 shows the variations of amplification with the adjusted shear modulus. The amplification factor calculated using one-half the modulus of Hardin and Drnevich (1972) compared relatively well with the magnification factor of the model test of 1.1 under the amplitude of 0.4 g. Thus, the one-half modulus of Hardin and Drnevich (H–D) was used for the analysis of the amplification of the model slope with the amplitude ranging from 0.1 to 0.6 g. Results of the analysis for the amplification factor between the crest and the base of slope with different shear moduli are shown in Fig. 21. Note that in Fig. 21, the trends of the calculated results using the one-half H–D modulus appeared to be very consistent with the measured amplification for acceleration amplitude ranging from 0.1 to 0.4 g. As for other values of the modulus, the computed amplification factor indicated greater differences from the measured amplification. However, as the acceleration amplitude increases beyond 0.4 g, the computed amplification factors based on the one-half H–D modulus remained about the same, while the measured values increased rapidly to 0.5 g and then decreased to 0.6 g. As was discussed previously, the specimen behaved elastically with the acceleration amplitude up to about 0.4 g, and the nonlinear behavior and modulus degradation became significant with acceleration larger than 0.4 g. With amplitude larger than 0.5 g, a possible slip surface and separation of part of the soil body could have initiated, causing the measured amplification to decrease. However, the measured amplification factor at the acceleration amplitude of 0.6 g may not be reliable due to the possible development of a slip surface and exposure of AC12 near the top of slope. The one-half H–D shear modulus appears to provide satisfactory results when the peak acceleration is smaller than 0.4 g and soil behaves linearly, but as the nonlinear behavior of the slope becomes significant further degradation of the modulus is required due to the equivalent linear model used in the analysis. Thus the numerical model constructed with one-half H–D shear modulus appeared to provide satisfactory results for the slope responses before the slope motion became nonlinear.

For the numerical model used in this study, the potential slip surface was determined based on the local maximum strain in the slope. The potential slip surfaces determined from the different moduli and methods, and projected from the mapping of the experiment result are shown in Fig. 22 for comparison. The slip surface from the experiment is relatively shallow compared to all others from the numerical model and pseudo-static analysis. Moreover, the slip surface generated using the

Table 4
Amplification factor of the slope crest to the base

Model used for the modulus	Shear modulus (MPa)	Amplification factor
Hardin and Drnevich (1972)	23.9	1.05
Assimaki et al. (2000)	98.6	1.01
Shear wave velocity	293.6	1.00

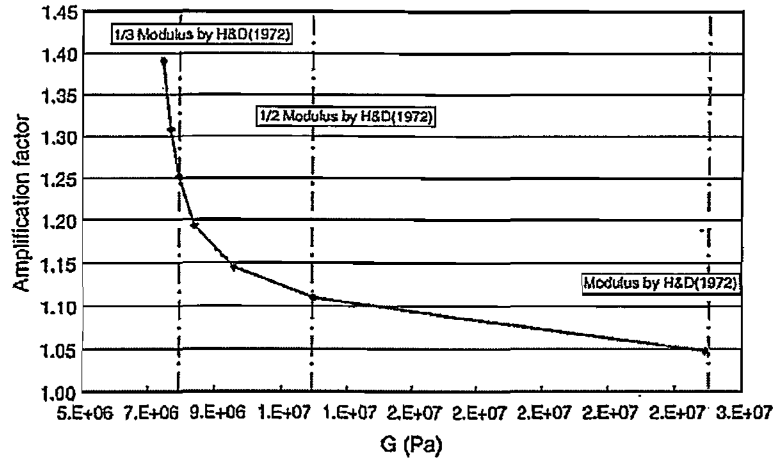


Fig. 20. Variations of the amplification factor with degradation of the shear modulus under amplitude of 0.4 g.

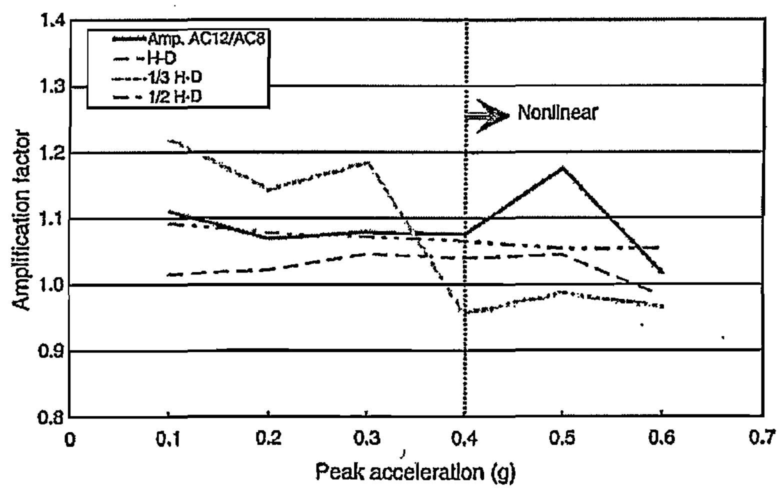


Fig. 21. Amplification factors calculated with different shear moduli.

modulus of Hardin and Drnevich (1972) appeared to be shallower than those with smaller modulus, as well as closer and more similar in shape to the one produced in the experiment. As the modulus decreased, the soil became softer, and this led to a deeper slip surface, which was not consistent with the slope failures observed in the

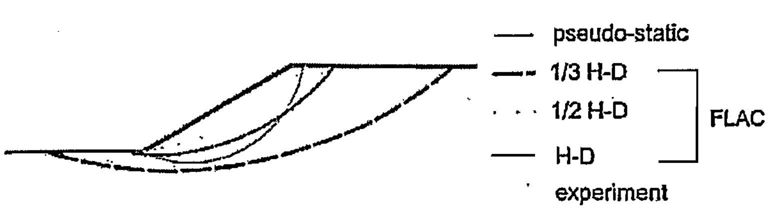


Fig. 22. Slip surfaces from different analysis methods and experiment result.

field after the Chi-Chi earthquake. Thus, for the location of the possible slip surface, the H–D modulus appeared to provide a more reasonable result than the one-half H–D modulus. Thus different conclusions resulted from either considerations of the potential slip surface or amplification factor. The differences in the modulus for determining the sliding surface and amplification effect could be caused by not being able to scale down the modulus in the model test according to the requirement of the law of similitude. Such conditions could also be caused by the degradation and nonlinear behavior of the slope, and so a more appropriate modeling of these properties is required for further analysis of the slope.

8. Discussion

For the model test developed in this research, the law of similitude of Iai (1989) was applied. Due to the limitation of our ability to scale down the modulus and shear wave velocity of the sand specimen, the law of similitude was only partially fulfilled in this study. However, the strain ratio of the model and prototype was not affected by this problem, and using strain to characterize the slope behavior appeared to be reasonable. The responses and amplification of the model test appeared to reflect the behavior of the prototype slope under seismic loading conditions reasonably well when compared to the field observations of slope failure after the Chi-Chi earthquake. Thus, the response and amplification behavior of the prototype slope can be studied in the laboratory for further evaluation of the slope behavior under earthquake conditions in the field, especially for important slopes such as earth dams under seismic loading conditions. Further modification and a more rigid law of similitude may help to improve the simulation of the prototype slope, and to provide better interpretation of the results concerning the stress field and development of the failure surface. Preliminary study of the numerical analysis also reflected the same condition, as the responses and amplification of the slope under seismic loading could be interpreted fairly well. However, the results of stress conditions in the slope and location of the failure surface were not as satisfactory. Part of this was caused by not being able to satisfy the requirement of the law of similitude for the scaling down of the modulus and wave velocity of the sand specimen. The nonlinear behavior and amplification near the crest of the slope appeared to have significant effects on the response behavior of the slope too. Further numerical modeling should be developed to take into account these factors in order to better simulate the prototype slope behaviors.

9. Conclusions

A model slope test has been performed to study the prototype slope, and acceleration responses at different positions in the model slope were measured. The strain characteristics of the prototype slope under dynamic loading can be simulated in the laboratory observing the proper law of similitude, which provide information for potential slope failure induced by earthquakes. Preliminary numerical analysis was also performed to study the behavior of the slope under seismic loading. Based on the previous discussion, some conclusions are reached as follows:

1. The model slope behaved elastically under acceleration amplitude less than 0.4 g for the sand material used in this study, and the pseudo-static analysis seemed satisfactory for determining the critical acceleration for the initiation of slope failure under seismic loading.
2. Numerical analysis using the finite difference method is helpful for the consideration of boundary effects, and accordingly the testing condition appeared unaffected by the boundary confinement.
3. The soil amplification of the slope appeared to be quite significant, and the effects increased as the nonlinear soil behavior became significant, which in turn could aggravate the development of slope failure.
4. The failure surface appeared to be shallow and likely to be circular and confined to the zone near the slope surface, and this phenomenon is consistent with the field observations.
5. Results of the numerical model analysis indicated that a shear modulus based on Hardin and Drnevich's (1972) empirical equation is more suitable for determining the slip surface and critical acceleration, while one-half of the modulus of Hardin and Drnevich (1972) is more suitable to simulate the amplification effect before the development of the slip surface. This inconsistency could be due to not being able to observe the law of similitude completely, to the nonlinear properties of soil, and to the effects of amplification.
6. The nonlinear soil behavior and degradation of modulus appeared to be important factors for slope stability under conditions of seismic loading. Development of a proper law of similitude is also essential in order to be able to better simulate the prototype slope behavior in the laboratory.

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B題：請說明本篇文獻之：1.目的背景(20%)、2.研究內容(30%)、3.重要發現與結論(30%)、4.對此文章之批判與建議（20%）。

題目：**Earthquake!**

Public Roads, September/October 2010, Vol. 74 · No. 2

by Wen-Huei (Phillip) Yen

Earthquake!

FHWA is conducting research to help mitigate the impacts of seismic events on transportation infrastructure. The results are promising.

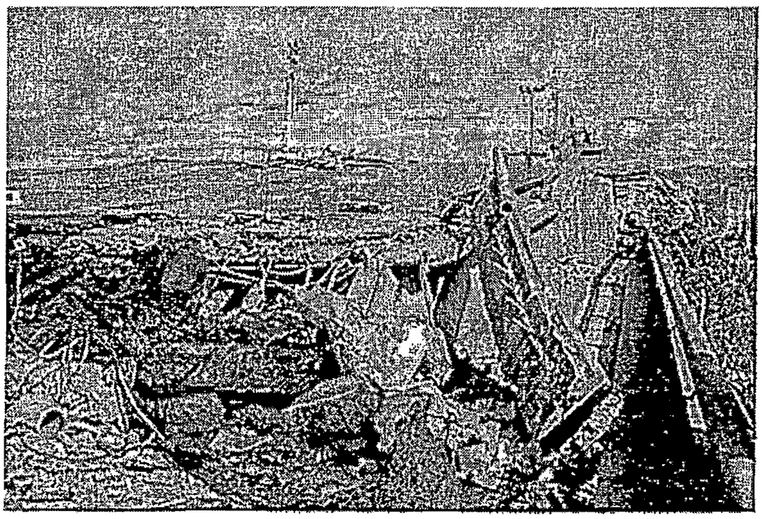


Fig. 1 Ground movement during the Loma Prieta Earthquake in California pancaked the upper deck of the Cypress Street Viaduct so that the guardrail seen on the right dropped to the lower deck.

The public relies on highways for the safe transport of goods and people across the country. Because roads serve as critical lifelines in the delivery of basic daily needs, they need to function even in the face of adverse weather and natural hazards. From 1993-1996, the United States spent an average of approximately $250 million per week responding to the impacts of natural disasters, with earthquakes, hurricanes, and floods being the major causes of monetary losses. At times, earthquakes can top the list. One of the most costly natural disasters in the United States between the late 1980s and late 1990s was California's Northridge Earthquake of 1994, which resulted in a total of $20 billion in infrastructure damages.

An earthquake is a sudden ground motion or trembling caused by an abrupt release of accumulated strains acting on the tectonic plates that comprise the Earth's crust. Earthquakes often trigger other devastating events such as landslides, fires, and lateral spreads (displacements of sloping ground, primarily due to soil liquefaction during earthquakes). In addition to destroying buildings, earthquakes can damage bridges, tunnels, pavements, and other components of highway infrastructure. If an earthquake occurs in an ocean, it can trigger a tsunami that can devastate coastal roads and bridges.

Relatively speaking, the probability of large, destructive earthquakes is much lower than hurricanes and floods. Nevertheless, an earthquake can, without warning, ravage an enormous area in less than 2 minutes through ground shaking, surface fault rupture (displacement due to the movement of tectonic plates), and ground failures (landslides, liquefaction, and lateral spreads).

The loss of life and extensive property damage inflicted by the 1989 Loma Prieta and 1994 Northridge earthquakes emphasized the need to minimize seismic risks to the U.S. highway system. Seismic research projects conducted by the Federal Highway Administration (FHWA) are developing mitigation approaches to reduce those risks, including a method for assessing seismic risks and various structural designs and retrofitting measures.

"Since 1992, FHWA has conducted a series of comprehensive seismic research studies targeting retrofitting, design, and risk analysis issues for bridges," says *Jorge E. Pagán-Ortiz*, director of FHWA's Office of Infrastructure Research & Development. "FHWA's seismic research has produced a number of nationally applicable seismic retrofitting manuals and design and risk analysis tools."

What follows is the story of that research.

Early Earthquake Mitigation Research

First, a look at the early research. FHWA initiated its earthquake investigations after the 1964 Prince William Sound Earthquake in Alaska. FHWA's follow up focused on how bridge engineers could learn from the Alaska earthquake in terms of geotechnical issues such as soil properties.


Federal Emergency Management Agency (FEMA) News Photo

Fig. 2 These two men are standing in a roadway cut in half by the force of the Loma Prieta Earthquake on October 17, 1989.

Then, following the poor performance of bridges during the San Fernando Earthquake in 1971, FHWA and the California Department of Transportation (Caltrans) began exhaustive studies of the seismic performance of bridges. FHWA and Caltrans invested $3 million in basic research to develop national guidelines for bridge seismic design. The study evaluated the criteria used at the time for seismic design, reviewed findings from seismic research for potential use in a new specification, updated guidelines for seismic design, and evaluated the impact of those guidelines on construction and costs.

In 1981, FHWA and Caltrans completed the guidelines, which the American Association of State Highway and Transportation Officials (AASHTO) adopted in 1983 as its *Guide Specification for Seismic Design of Highway Bridges*. This specification became a national standard in 1992, following the Loma Prieta Earthquake.

Table 1 Significant Earthquake Damages in the United States, 1964-1994

Location	Date	Magnitude	Damages(in Millions)	Deaths
Prince William Sound, AK	03-27-1964	8.4	$311	125
San Fernando Valley, CA	02-09-1971	6.6	$505	65
Loma Prieta, CA	10-17-1989	7.1	$6,000	63
Northridge, CA	01-17-1994	6.7	$20,000	61

Sources: Stover and Coffman, 1993; FEMA 1994.

The design philosophy underlying this specification was to prevent collapse of any span or part of a span during large earthquakes. In small to moderate seismic events, the code's intent was for bridges to resist seismic loads without significant damage to structural components. Under this code, the design earthquake had a 475-year return period, which represents not greater than a 10 percent probability of an earthquake occurring during a bridge design life of 50 years.

Fig. 3 This highway overpass at the I-5 and I-14 interchange collapsed during the San Fernando Earthquake in California on February 9, 1971.

ISTEA and the Seismic Research Program

FHWA's role in earthquake research did not end with the adoption of this 1992 standard. The agency renewed its commitment to mitigating effects on highway structures by establishing a seismic research program, as called for in the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. The studies were conducted for FHWA under a contract at the National Center for Earthquake Engineering Research, later renamed as the Multidisciplinary Center for Earthquake Engineering Research (MCEER).

Under ISTEA, Congress funded the research with more than $14.25 million between 1991 and 1997. The program covered all major highway system components (bridges, tunnels, embankments, retaining structures, and pavements).

Approximately 65 percent of the Nation's 600,000 highway bridges were constructed prior to 1971, with little or no consideration given to seismic resistance. In recognition of that situation, the FHWA seismic research program initiated two comprehensive studies. In the fall of 1992, the program began studying the seismic retrofitting of existing bridges and highway structures, and in spring 1993 began studying the seismic design of new bridges.

The first product of this research, *Seismic Retrofitting Manual for Highway Bridges* (FHWA-RD-94-052), appeared in 1995 and summarized lessons learned from more than 20 years of earthquake engineering research and implementation, and provided procedures for evaluating and upgrading the seismic resistance of existing bridges.

In 1999 the program published *Impact Assessment of Selected MCEER Highway Project Research on the Seismic Design of Highway Structures* (MCEER-99-0009), which became the major documentation used to develop recommendations for the seismic design of new bridges. In 2006 FHWA issued the final products of this research, *Seismic Retrofitting Manual of Highway Structures-Part I (Bridges)* (FHWA-HRT-06-032) and *Part II (Retaining Structures, Slopes, Tunnels, Culverts, and Roadways)* (FHWA-HRT-05-067).

These recommended seismic design specifications, proposed in 2001 under the National Cooperative Highway Research Program (NCHRP) 12-49 project, Comprehensive Specification for the Seismic Design of Highway Bridges, were performance-based. The major difference between them and the 1992 design code was that they had a two-level design criterion. The higher level was based on a 2,500-year return period, and the lower on a 100-year period. The new seismic retrofitting manuals are also performance-based and based on a two-level design criterion, but a 1,000-year return period for the high level and 100 years for the lower level.

Seismic Research Under TEA-21

While the researchers were finishing their work under ISTEA by developing the 1999 design recommendations, in 1998 FHWA launched a congressionally mandated seismic research program under the Transportation Equity Act for the 21st Century (TEA-21), funded by another $12 million, to study seismic vulnerability. In cooperation with MCEER, FHWA conducted a series of studies to develop tools for evaluating and assessing the social costs and impacts of earthquakes on the U.S. highway system. The goal was to reduce the likelihood of damage to existing and future highway structures caused by moderate to significant seismic events.

The main tasks undertaken within this program were the following:

- Development of loss estimation methods for highway systems

- Preparation of a manual for the seismic design and retrofitting of long-span bridges
- Development of protective systems and a systems design manual for bridges
- Specialized ground motion, foundation, and geotechnical studies

Under TEA-21, FHWA worked with NCHRP in 2001 to develop new seismic design specifications, NCHRP 12-49. AASHTO then reviewed and revised the new design specifications and adopted them in 2007. However, publication was delayed until 2009, when they were published as the AASHTO *Guide Specifications for LRFD* [Load and Resistance Factor Design] *Seismic Bridge Design,* 1st edition. The NCHRP 12-49 specification was developed from the 1999 recommendations. The 2007 specification is a one-level design criterion for a 1,000-year return period.

Under the TEA-21 seismic research program, FHWA developed a software package called REDARS: Risks from Earthquake DAmage to Roadway Systems to estimate the loss of highway system capacity due to earthquakes. The tool helps bridge owners estimate how earthquake damages affect post-earthquake traffic flows and enables them to consider those effects during pre-earthquake planning and prioritizations, and in post-earthquake responses, such as rescue and management of damage investigations. The seismic research program released REDARS in 2006.

Also in 2006, the program published the *Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges*(MCEER-06-SP05), which particularly addresses truss bridges that are more than 500 feet (152 meters) long. The guidelines use a performance-based seismic retrofit philosophy, focus on superstructure retrofit, and provide case studies. A *Seismic Isolation of Highway Bridges* (MCEER-06-SP07) manual also was published in 2006. It presents the principles of isolation for bridges, develops step-by-step methods of analysis, explains material and design issues for elastomeric and sliding isolators, and provides detailed examples of their application to standard highway bridges. The manual is a supplement to the *Guide Specifications for Seismic Isolation Design* published by AASHTO in 1999.

REDARS: Risk Analysis and Loss Estimations

Earthquakes are inevitable natural hazards with the potential for causing large numbers of fatalities and injuries, major property and infrastructure damage, and serious disruption of everyday life. However, a systematic risk assessment process can help keep earthquake losses to a minimum. This methodology -- called risk management -- is a process for determining which hazards should be addressed, what priority they should be given, what should be done, and what countermeasures should be used.

Earthquake damages to highway infrastructure can go well beyond human safety and the cost of repairs. Such damage also can disrupt traffic flows and therefore affect a region's emergency response and economic recovery. Impacts depend not only on the seismic performance of the highway components, but also on the highway network's configuration, including highway redundancies, traffic capacities, and the links between interstates and arterial roads.

State departments of transportation usually do not consider these factors in their risk reduction activities. One reason is lack of a technically sound and practical tool for estimating impacts. Therefore, beginning in the late 1990s, FHWA sponsored multiyear seismic research projects for developing and programming REDARS (Risks from Earthquake DAmage to Roadway Systems) software, released for public use in 2006.

REDARS is a multidisciplinary tool for seismic risk analysis of highway systems nationwide. For any given level of earthquake, REDARS uses state-of-knowledge models to estimate seismic hazards (ground motions, liquefaction, and surface fault rupture); the resulting damage (extent, type, and location) for each component in the highway system; and repairs that might be needed to each component, including costs, downtimes, and time-dependent traffic (that is, the component's ability to carry traffic as the repairs proceed over time after the earthquake).

REDARS incorporates these traffic states into a highway network link-node model to form a set of system-states that reflect the extent and spatial distribution of roadway closures at various times after the earthquake. REDARS then applies network analysis procedures to each system-state in order to estimate how these closures affect systemwide travel times and traffic flows. Finally, REDARS estimates corresponding economic losses and increases in travel times to and from key locations or along key lifeline routes. Users can apply these steps for single earthquakes with no uncertainties (deterministic analysis) or for multiple earthquakes and in estimates of seismic hazards and component damage (probabilistic analysis).

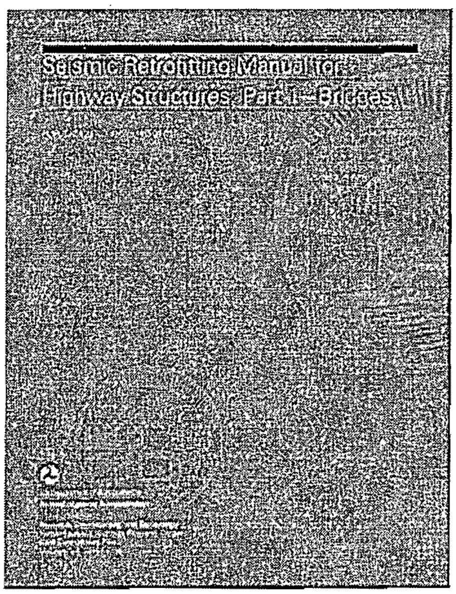
Although REDARS adequately replicated the performance of the highway system in the San Fernando Valley during the Northridge Earthquake, much work still needs to be done to enable engineers to use the methodology with confidence. Indeed, the researchers developed REDARS with the expectation that new and more sophisticated modules will be developed over time to improve its accuracy and expand its range of application.

SAFETEA-LU Seismic Research

In 2005, Congress passed the Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU). Under the legislation, FHWA oversaw $12.5 million in seismic research to work with the bridge engineering community and enhance the earthquake resistance of U.S. highway bridges. The two recipients of this congressional earmark research were MCEER and the University of Nevada, Reno.

Also, SAFETEA-LU mandated a technology exchange and transfer task, which FHWA conducted through a series of bridge engineering workshops and conferences held nationally and internationally. The meetings involved exchange of technical information and performance of cooperative studies.

One of these technical exchange programs is a panel on wind and seismic effects under the U.S./Japan Cooperative Program in Natural Resources. The outcomes of this succession of programs held over the past four decades include greater understanding in three areas: assessing seismic vulnerability of specific locations, geotechnical hazards, and infrastructure vulnerability. Building on this increased body of knowledge, FHWA currently is developing improved seismic designs for new and retrofitted bridges, plus instrumentation to monitor performance.



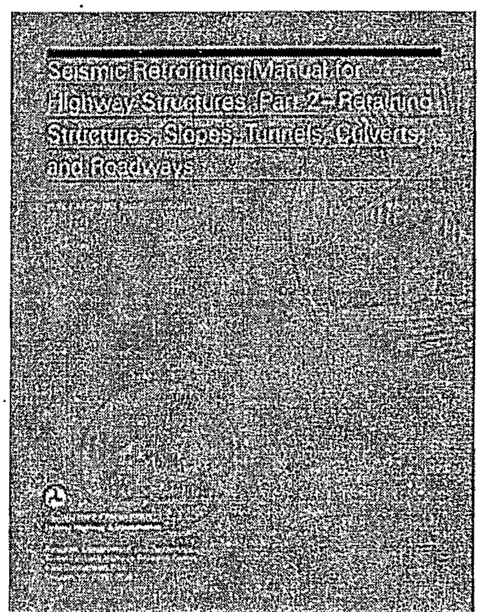


Fig. 4 FHWA currently is developing improved seismic designs for new and retrofitted bridges, plus instrumentation to monitor performance.

Assessing Seismic Vulnerability: Hazard Maps

To design a bridge to resist earthquakes, understanding the seismic vulnerability or earthquake intensity of the bridge's location is essential. This vulnerability usually is described as seismic hazard. The U.S. Geological Survey (USGS) publishes National Seismic Hazard Maps that display various probability levels of earthquake ground motions across the United States. The seismic provisions of building codes, insurance rate structures, risk assessments, and other public policy provisions commonly apply probability levels based on the hazard maps.

A 2003 update of the maps incorporates new findings on earthquake ground shaking, faults, and seismicity (that is, how prone a region is to earthquakes). USGS derived the new maps for a grid of sites across the United States by calculating seismic hazard curves that describe the frequency of exceeding a set of ground motions. Currently, the new seismic design and retrofitting criteria for bridges use a 1,000-year return period for a given level of earthquake, which represents not greater than a 7 percent probability of an earthquake occurring during a bridge design life of 75 years. USGS and AASHTO issued the updated maps and computer software for obtaining seismic hazards by entering ZIP Codes or longitude and latitude coordinates.

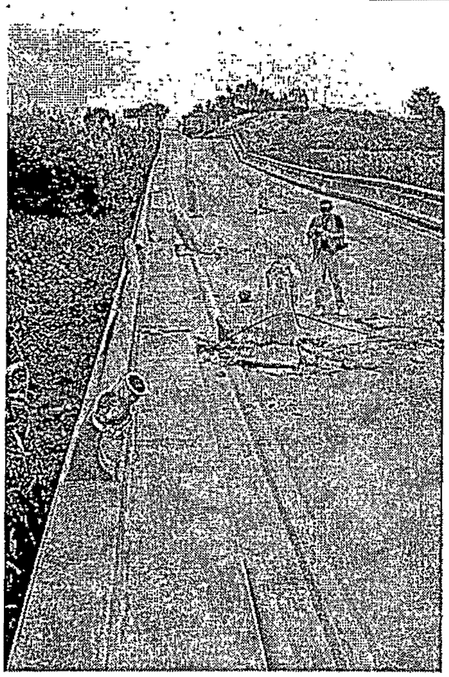

H.G. Wilshire, USGS

Fig. 5 Support columns of the Highway 1 Bridge across Struve Slough near Watsonville, CA, protrude through the roadbed, a result of lateral shaking during the Loma Prieta Earthquake.

Assessing Geotechnical Hazards

Another factor in designing and retrofitting highway bridges is the geotechnical hazards that an earthquake can trigger, such as soil liquefaction and settlement, slope failure (landslides and rockfalls), surface fault ruptures, tsunamis, and flooding. Assessing geotechnical hazards is a two-part procedure. First, engineers conduct a quick screening evaluation, generally using information available from field reconnaissance.

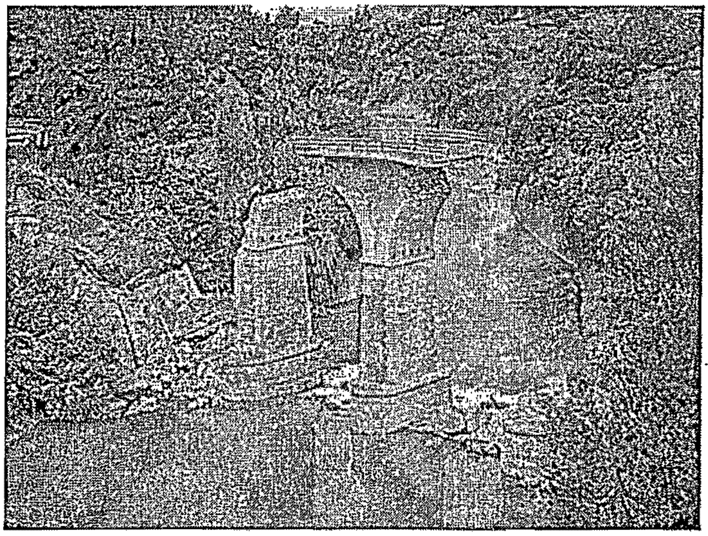

Alejandro Molina Agurre, Ministry of Public Works, Chile

Fig. 6 Shown here is the Claro River Bridge, which is located near the town of Camarico, Chile (between Santiago and Concepción), and collapsed during the February 27, 2010, earthquake.

If various criteria are satisfied, they consider the risk to be low and require no further evaluations. If a hazard cannot be screened out, they conduct more detailed and rigorous evaluations, which usually require obtaining additional data to assess the hazard and its consequences.

Assessing Infrastructure Vulnerability

To assess the seismic vulnerability of the U.S. bridge inventory, researchers often use an indices method to determine a bridge's seismic rating. The method involves assessing the structural vulnerability of the bridge, the seismic and geotechnical hazards of the site, the socioeconomic factors affecting the structure's importance, and other issues such as the structural redundancy with the bridge and nonseismic structural issues. Through this method, researchers arrive at a final, ordered determination of the retrofitting priority of individual bridges and, ultimately, for the Nation's entire infrastructure inventory.

The rating system has two parts: the quantitative part, which produces a seismic rating ("bridge rank") based on structural vulnerability and site hazard; and the qualitative part, which modifies the rank in a subjective way that accounts for importance, network redundancy, nonseismic deficiencies, remaining useful life, and similar issues to arrive at an overall priority index.

Mitigation Design of New Bridges

Based on advanced seismic research and experience with destructive earthquakes, AASHTO and FHWA have improved seismic designs for new bridges. The results include design details that directly affect bridge performance under increased loadings due to earthquakes.

"The performance of U.S. highway bridges in recent large earthquakes has shown that the current state of the art has saved many bridges from collapse by preventing unseating of the superstructure or shear failure of the columns," says FHWA's *Pagán-Ortiz.*

The fundamental design objective of current seismic specifications in small to moderate events is to resist seismic loads within an elastic range without significant damage to structural components. The objective in large earthquakes is that no span, or part of a span, should collapse. The specifications consider limited damage to be acceptable in these circumstances, provided it is confined to flexural hinging (that is, hinging that allows an angle to be adjusted while remaining in place) in pier columns. This is to allow steel rebars to yield and absorb earthquake excitation energy while not rupturing and leading to collapse. Further, damage above ground is preferable so that it is visible in sections of the bridge that are accessible for inspection and repair.

Under current specifications, the seismic performance objective is no collapse based on a one-level rather than a two-level design approach. The current one-level design criterion is based on a 1,000-year return period event with not greater than a 7 percent probability of occurring during a bridge's 75-year design life. As an operational objective, bridge designers may use a higher, two-level performance criterion, but only with authorization from the bridge owners. Current specifications, however, do not provide guidance beyond the one-level approach.

Seismic Retrofitting Of Existing Bridges

Retrofitting is the most common method of mitigating risks; in some cases, however, the cost might be so prohibitive that abandoning the bridge (total or partial closure with restricted access) or replacing it altogether with a new structure may be favored. Alternatively, doing nothing and accepting the consequences of damage is also a possible option. The decision to retrofit, abandon, replace, or do nothing requires careful evaluation of the importance of the bridge and its degree of vulnerability. Limited resources generally require that deficient bridges be prioritized, with important bridges in high-risk areas being retrofitted first.

Bridges constructed prior to 1971 in particular need to be retrofitted, based on seismicity and structural types. Toward this end, FHWA issued several publications, including *Seismic Retrofitting Guidelines for Highway Bridges* (FHWA-RD-83-007) in 1983 and *Seismic Design and Retrofit Manual for Highway Bridges* (FHWA-IP-87-6) in 1987. In 1995, FHWA updated these manuals with current knowledge and practical technology in the *Seismic Retrofitting Manual for Highway Bridges* (FHWA-RD-94-052), mentioned earlier.

Then, also as mentioned earlier, FHWA published *Seismic Retrofitting Manual of Highway Structures-Part I* and *Part II.* This two-volume manual contains the following procedures for evaluating and upgrading the seismic resistance of existing highway bridges:

- A screening process to identify and prioritize bridges that need to be evaluated for seismic retrofitting
- A methodology for quantitatively evaluating the seismic capacity of a bridge
- Retrofitting approaches and techniques for increasing the seismic resistance of existing bridges
- A methodology for determining the overall effectiveness of alternative retrofitting measures, including cost and ease of installation

The manual does not prescribe rigid requirements as to when and how bridges are to be retrofitted. The decision to retrofit depends on a number of factors, several of which are outside the engineering realm. These other factors include, but are not limited to, the availability of funding and a number of political, social, and economic issues. A bridge may be exempt from retrofitting if it is located in a seismic zone with very little ground motion or has limited remaining useful life. Temporary bridges and those closed to traffic also may be exempt if they are not crossing a major national highway (lifeline system) or defense highway.

Recognizing the earthquake vulnerabilities of highway bridges constructed prior to 1971, many State departments of transportation, including California, Illinois, Missouri, Oregon, Tennessee, and Washington, initiated and performed retrofitting funded by FHWA to increase seismic safety. Many retrofits involve hinge seat extensions, which enlarge the size of the hinges that connect sections of bridge decks, or installation of a restrainer to link superstructures (decks) together and help prevent them from separating during severe ground movement. Some single columns were retrofitted with a steel casing to increase the earthquake resistance (ductility) to prevent collapse. FHWA's new seismic retrofitting manuals provide details on this retrofitting process.

Performance Monitoring In Missouri

The seismically active New Madrid Fault region in Missouri and adjacent States requires a hazard mitigation program that addresses the possibility of strong shaking of structures and the potential for ground failures in the vicinity of bridges. Designers of the cable-stayed Bill Emerson Memorial Bridge in Cape Girardeau, MO, which is within the New Madrid Fault region, had to take into account the possibility of a strong earthquake (magnitude 7.5 or greater) occurring during the design life of the bridge.

To capture data on strong ground motions, FHWA worked with the Missouri Department of Transportation (MoDOT) and USGS to complete a seismic instrumentation plan for the bridge before the start of construction. To assess differential motions at the piers along the total bridge span of 3,956 feet (1,206 meters), the instrumentation includes 84 accelerometers attached to the pier foundations and superstructure (caissons, tower, and deck). In addition to recording events at the site, the system can broadcast the data to outside users. This real-time seismic monitoring system can support signal transmission via the Internet from combinations of one-dimensional and three-dimensional accelerometers to recorders at the site.

Synchronized systemwide timing of the accelerometers can ensure time-variant response recording at one location in the bridge relative to other locations. Real-time streaming of the data will facilitate remote maintenance and data acquisition and retrieval capabilities. The bridge owner, researchers, and engineers now are able to use the response data to assess the bridge performance; check design parameters, including comparison of dynamic characteristics with actual responses; and improve the design of similar bridges in the future.

By appropriate configuration of the streamed data, the researchers also can use the instrumentation as a health monitoring tool to serve as an early warning system for defects or unexpected behaviors, and to assess damage to the bridge. The need to monitor the response of bridges in real time or near real time usually arises when information on rapid responses is required, such as during homeland security emergencies.

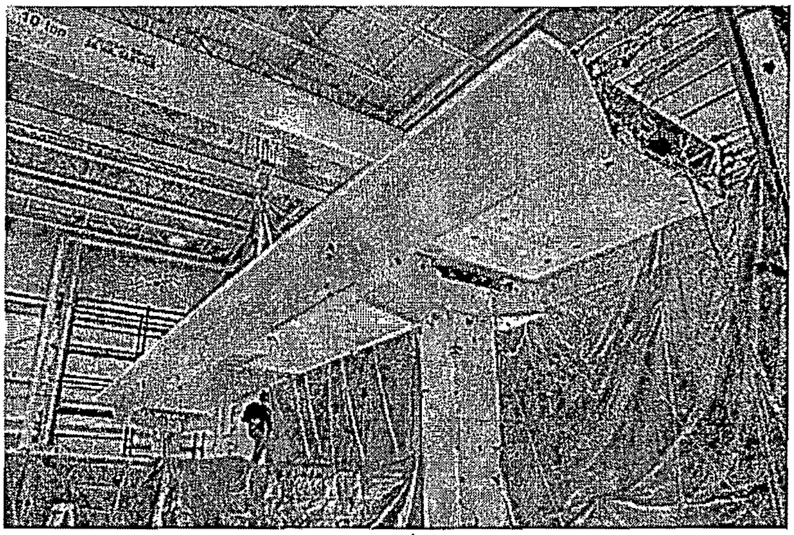


Fig. 7 Shown here is a scaled-down pier column of a segmental concrete bridge fabricated offsite and then assembled and tested at the University at Buffalo in May 2010. The purpose of the study was to test the seismic performance of a bridge built using accelerated construction techniques.

Next Steps

The recent major earthquakes in China in 2007 and Chile and Haiti in 2010 have challenged earthquake engineering disciplines around the world. The intensity of peak ground accelerations and long duration of shaking resulting from large earthquakes create greater difficulties for designing and retrofitting highway bridges. Through its seismic research program, FHWA is exchanging technical information and collaborating on research with seismically active States in the United States and with other countries, including Chile, China, Italy, Japan, Taiwan, and Turkey.

Over the past 15 years, the program has sponsored a series of conferences around the United States and bilateral workshops with other countries to promote new technology and exchange technical information. In 2009, the 25th U.S.-Japan Bridge Engineering Workshop, held in Tsukuba, Japan, marked the silver anniversary of this technology exchange and cooperation.

FHWA continues to work with MCEER, located at The State University of New York at Buffalo, and the University of Nevada, Reno. Under current legislation, two major initiatives are underway, focusing on innovative protection technologies and seismic resilience for larger earthquakes yet to come.

Developing innovative protection technologies. This initiative is to improve the seismic resistance of the U.S. highway system by developing innovative technologies, expanding their applicability, and developing cost-effective methods for implementing design and retrofitting technologies. As FHWA applies accelerated methodologies to construct new bridges and maintain existing bridges in high seismic areas, research is underway to develop more advanced design details to accommodate bridge movements due to large ground motions.

Improving seismic resilience. Life-safety (no collapse and no loss of human life) is no longer the sole requirement for success in designing a highway system capable of resisting the impacts of a major earthquake. The traveling public now expects resilience in the surface transportation infrastructure as well -- that is, rapid recovery and minimal impact on the socioeconomic fabric of modern society.

The need for resilience has led to development of the concept of performance-based seismic design. Performance measures calculated by REDARS include congestion and delay times. These measures allow system-level performance criteria to be specified for earthquakes of various sizes, such as maximum permissible traffic delay times and minimum restoration times. Thus, these measures allow resilience of a highway system to be defined and measured in quantitative terms, such as the time it takes to restore the system to pre-earthquake capacity. Accordingly, local transportation authorities can develop financial and societal incentives that will improve resilience and at the same time reduce risk to life and property.

FHWA and others have made substantial progress in this area, particularly with respect to the performance of individual components of the built environment, such as buildings and bridges. But the real potential for performance-based design comes when these concepts are applied to systems and subsystems of the infrastructure, such as transportation networks, subject to both service load conditions and extreme events.

This initiative will study the resilience of highway systems with a view to improving performance during major earthquakes. Refining the REDARS program's current loss estimation methodologies is included, along with providing a comprehensive assessment tool to measure highway resilience. Further, the project will identify factors affecting system resilience, such as damage tolerance of bridge structures and network redundancy, and will develop design aids for curved bridges and structures in near-fault regions.

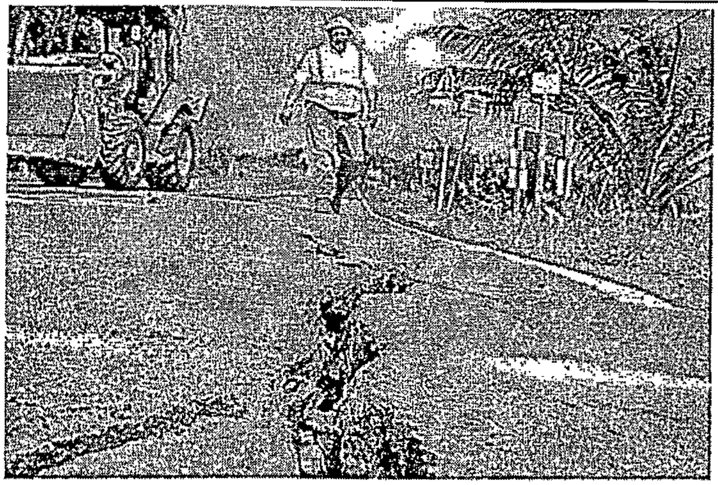


Fig. 8 A Pololu Valley crew in Kapaau, HI, works to seal a cracked road damaged during earthquakes in 2006.

Concluding Thoughts

The greatest difficulty in mitigating earthquake hazards is that seismic events occur without any notice and without any way of accurately predicting when they will occur, nor what their magnitude will be. Earthquakes are devastating, often resulting in a large number of deaths, injuries, and extensive infrastructure damage. These losses occur within minutes. Systematic approaches to evaluating earthquake risks, including indirect losses such as economic impacts, have become an important issue to the engineering community. Hazard mitigation methods to reduce earthquake losses require an enormous effort for development and implementation.

The Turner-Fairbank Highway Research Center has played an important role in developing guidance for seismic hazard reduction. The seismic research program is an important component of the multihazard research program within the Office of Infrastructure Research & Development, which includes wind, flooding and scour, and terrorism.

"FHWA is working closely with AASHTO and NCHRP and others to mitigate earthquake hazards and reduce losses," says *Pagán-Ortiz*. "These efforts to implement all practical measures to enhance the safety of the Nation's highway infrastructure and mobility of users are in a race against time with earthquakes. Fortunately, we think that the outlook is promising."

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Critical success factors for construction partnering in Taiwan

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Abstract

Partnering is perhaps one of the most innovative developments in delivering a project efficiently and reducing construction disputes. Partnering provides a sound basis for achieving a win–win situation and implementing synergistic teamwork. Ubiquitous research exists regarding the use of partnering in construction. Various potential factors contributing to partnering success have emerged and deserve future study. This study attempts to distinguish these factors based on their degrees of importance in relation to success. Through a questionnaire survey administered to project participants with first-hand partnership experience, the opinions of various parties, including government employees, owners, designers, and contractors, were sought and assessed in relation to construction partnering *critical* success factors in Taiwan. Certain requirements must be met for partnering to be successful, including a collaborative team culture, a long-term quality focus, consistent objectives, and resource-sharing. Such identification of *critical* success factors *of partnership* can be used to devise effective strategies for minimizing construction conflicts and enhancing project performance. Successful construction partnering requires a combined effort from all parties involved.

Keywords: Partnerships; Critical success factors; Construction industry

1. Introduction

During the recent decade, the annual production yield of the Taiwanese construction industry has been roughly 16.2 billion USD, representing approximately 4–6% of total GDP (gross domestic product), indicating that the construction industry contributes significantly to overall economic development in Taiwan. However, construction projects in Taiwan are generally of poor quality, and suffer problems of performance failures, cost wastage, schedule delays, and so on.

The main reasons for the unfavorable construction project outcomes mostly fall into several categories. Construction projects rely on integrated efforts of several hierarchically linked parties (including architects, engineers, surveyors, general contractors, subcontractors and suppliers) using their differentiated skills, knowledge and technology. These parties are generally independent organizations with separate objectives and goals, management styles and operating procedures.

Due to the fragmented nature of construction, communication and coordination problems are common and affect project performance and productivity [1]. The long-term and short-term benefits to different participants vary among different stages of construction project life cycle. Taking electricity engineering as an example, the costs of initial system installation and in-surface maintenance vary significantly.

Because of differences in professional background, technology, knowledge and perspective among participants, problems in communications and cooperation are commonplace, often compromising project performance and results. The traditional DBB (design-bid-build) contract goes to the lowest bidder generally, frequently creating conflict between project owners and professionals. Owing to its dismemberment attributes, the traditional DBB

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project is run with a win-lose mentality, causing conflict in communications and cooperation, and sometimes even disputes, compromising project productivity and performance.

A construction project must proceed through stages of concept, scheme design, bidding, contracting, construction, service and maintenance. The main participants differ among stages, as does the related professional know-how, technologies and experience. In practice, project management has focused on maximizing performance in terms of time, costs and quality. However, relatively little attention has been paid to the organizational structures of each participant. Recently, the Taiwanese construction industry has faced major new challenges, including increased competition, more exacting quality standards, increased competition for available resources, globalization, rapid development of new technologies and increased various risks. Additionally, construction projects in Taiwan are growing larger and more complicated. An adversarial situation, at least from the perspective of traditional contracts, thus has been created between project owners and contractors. The changes mentioned previously have caused crises for the industry. Construction firms are now searching increasingly actively for better management approaches for improving performance and maintaining a competitive advantage.

2. Construction partnering

Numerous studies have examined the definition and meaning of partnering. The fundamental principles of partnering, namely trust, commitment, communication, respect, and equality, include appropriate consideration of the interests of all parties at every level [2–4], and aim to build "trust" among the parties involved in a contract. Such trust helps avoid problems with the project that recently have tended to lead to litigation [5]. Past studies have yielded numerous definitions of partnering, among which the definition developed by the Construction Industry Institute (CII) in the United States is the most widely cited. CII defined partnering in the following manner.

A long-term commitment between two or more organizations is important for achieving specific business objectives by maximizing the resources of each participant. Consequently, it is necessary to replace traditional relationships with a shared culture without regard to organizational boundaries. Such relationship is based on trust, dedication to common goals, and an understanding of individual expectations and values. The expected benefits include improved efficiency and cost-effectiveness, increased innovation opportunities, and the continuous improvement of quality products and services.

According to Bennett and Jayes [6], partnering is a set of strategic actions that deliver marked improvements in construction performance. It is driven by a clear understanding of mutual objectives and co-operative decision-making by multiple firms all focused on using feedback to continuously improve their joint performance.

Crowley and Karim [7] defined partnering as "an organization implementing a co-operative strategy by modifying and supplementing the traditional boundaries separating companies in a competitive climate". Partnering thus involves the major project participants in an alliance that creates a cohesive atmosphere enabling project team members to openly "interact and perform". Crowley and Karim conceptualized co-operative partnering using diagrams indicating permeable boundaries and indicating a cell-like organization. Each diagram was simplified to represent the relationships between the clients, consultants and contractors. They proposed that partnering involved four dimensions: (1) adversarial (perceived by the involved parties as a win/lose situation and leading to more formal litigation); (2) guarded adversarial (relationships that strictly adhere to and are interpreted by the contracts); (3) informal partners (understand and co-operate with parties with fewer disputes); and (4) project partners (equal partners working co-operatively to pursue a common set of goals).

It is also important to measure project performance in the areas identified during the initial partnering workshop during the agreed time intervals, and to feed back the

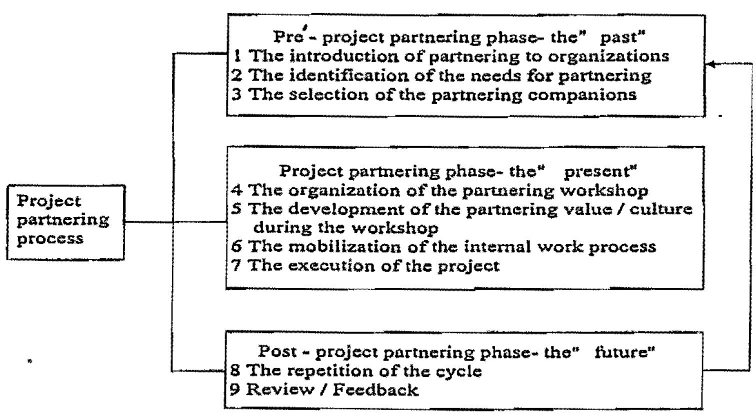

Fig. 1. Project partnering process model.

results for use in the project team for evaluation [2]. This is not necessarily easy but is essential. Fig. 1 illustrates the process from the decision to pursue partnering through to final review and feedback.

3. Research methodology

This work examines documents and theories regarding management and partnership in construction projects, and conducts questionnaires to professionals to further analyze the factors involved in successful construction partnering. These questionnaires compare the roles of different professionals in construction projects, and the effects of their various project attributes provide a reference for industry partnerships. This work also ranks critical success factors (CSF) of construction partnering. The development of strategies for achieving effective results in project management and partnership can prevent conflicts, lawsuits and inefficiency while raising project effectiveness and production, thus yielding a win–win situation for both project owners and professionals.

3.1. Questionnaire development

This work applies a Likert-type scale to the questionnaire design, running from 1 (extremely unimportant) to 5 (extremely important). To determine the questionnaire structure, a second evaluation was conducted to ensure its credibility and effectiveness. The original questionnaire design included 22 questions regarding partnership success factors (SFs). In this work, validity was used to ensure accurate measurement of the characteristics and factors. Generally the correction of the measurement results and forecasting characteristics is used to represent the degree of validity. Various studies [8–12] were referred for the questionnaires in the scale regarding important factors of partnership, partner benefits, and SFs.

3.2. Pre-test

A pre-test was performed to ensure the questionnaires were phrased appropriately. Forty-two construction professionals in Taiwan were provided with copies of the original questionnaire, respectively. The subjects were asked to comment on the readability, comprehensiveness, and accuracy of the questionnaires. Thirty-four copies were retrieved for the pre-test.

The Cronbach's α coefficient was used to determine the questionnaire reliability. A α exceeding 0.9 indicates high reliability, α between 0.9 and 0.7 indicates acceptable reliability, and α below 0.35 indicates low reliability [13,14]. The questionnaire responses that did not meet the criterion ($\alpha \geqslant 0.05$) were deleted, after which the remainder of the responses underwent reliability analysis. For the pre-test, Cronbach's α of 0.93 was achieved, and the corrected scale contained 19 structural survey questions representing 19 CSFs.

Table 1
Sampling project type, profession, and number of subjects

Profession	HLCP	LLCP	LSCP	Total
Government employee	3	22	14	39
Project owner	14	16	2	32
Design firm	4	39	20	63
Construction firm	29	48	10	87
Total	50	125	46	221

Note: HLCP stands for hi-tech large construction projects; LLCP stands for low-hi-tech large construction projects; LSCP stands for low-hi-tech small construction projects.

3.3. Questionnaire distribution

The survey sampled construction professionals and experts in Taiwan. The research subjects comprised three categories, namely hi-tech large construction projects (HLCP), low-tech large construction projects (LLCP), and low-tech small construction projects (LSCP). Hi-tech construction projects were projects that require high interface integration, for example high speed rail projects. Meanwhile, low-tech construction projects were projects without high interface integration, for example roadway construction projects.

The questionnaires were distributed via mail, e-mail, fax, telephone, and personal delivery to increase the rate of response and sample representation. Standbys were used to replace subjects who were unable to participate. Three-hundred and thirty questionnaires were distributed during December 2004 via mail, fax, e-mail, and personal delivery to construction industry subjects.

Table 1 shows that 221 copies were retrieved (67% return rate), among which 125 respondents (56.6%) were from NLCP, 50 (22.6%) were from the HLCP, and 46 (20.8%) were from NSCP. Breaking the sample down according to profession, 39 respondents were government employees (17.6%), 32 worked for the owner (Taiwan High Speed Rail Corporation; THSRC) of the largest BOT project in the world (14.5%), 63 worked for design firms (28.5%), and 87 worked for construction firms (39.4%). SPSS 10.0 was used to perform further statistical analysis.

4. Analysis, findings, and discussion

4.1. Ranking of CSFs

The SFs were ranked according to their means. If two or more SFs happened to share the same mean value, that with the lowest standard deviation was assigned the highest importance ranking. The SFs with means of 4 or more (after rounding) were recognized as CSFs based on respondent consensus. Nineteen SFs were identified as CSFs that significantly influenced the success of construction partnering. Table 2 ranks these CSFs based on mean value.

Effective communication ranks first because partnering requires timely communication of information and the

Table 2
Ranking of CSFs for construction partnering

Items	Total		Profession								Project type					
	Mean	Rank	Govern. employee		Project owner		Design firm		Construction firm		HLCP		LLCP		LSCP	
			Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank	Mean	Rank
CSF1	4.32	6	4.15		4.34	3	4.30	5	4.40	2	4.30	2	4.40	4	4.13	
CSF2	4.63	1	4.56	3	4.62	1	4.63	1	4.67	1	4.64	1	4.69	1	4.48	1
CSF3	4.25	8	4.13		4.34	3	4.27	7	4.25	7	4.22	5	4.34	8	4.02	
CSF4	4.24	9	4.15		4.28	5	4.27	7	4.25	7	4.20	6	4.26	9	4.24	5
CSF5	4.37	3	4.44	6	4.47	2	4.32	3	4.33	4	4.30	2	4.45	3	4.22	6
CSF6	4.03	17	4.18		3.94		4.05		4.07		3.84		4.14		3.89	
CSF7	4.06	14	4.18		3.91		4.05		4.07		3.84		4.09		4.22	6
CSF8	4.33	5	4.64	2	4.28	5	4.32	3	4.23	10	4.16	8	4.38	7	4.41	2
CSF9	4.16	12	4.46	4	4.09		4.14		4.06		3.82		4.27		4.22	6
CSF10	3.92	18	3.97		3.78		4.02		3.89		3.60		4.04	9	3.96	
CSF11	4.20	10	4.18		4.16	10	4.19	10	4.24	9	4.06		4.26		4.20	9
CSF12	4.05	16	4.23	8	3.72		4.05		4.08		3.78		4.16		4.02	
CSF13	3.80	19	3.87		3.59		3.86		3.79		3.50		3.88		3.89	
CSF14	4.08	13	4.15		4.06		3.89		4.20		3.90		4.18		4.00	
CSF15	4.39	2	4.67	1	4.28	5	4.35	2	4.33	4	4.24	4	4.50	2	4.26	4
CSF16	4.29	7	4.38	7	4.25	9	4.24	9	4.30	6	4.12	9	4.40	4	4.17	
CSF17	4.35	4	4.46	4	4.28	5	4.30	5	4.37	3	4.20	6	4.40	4	4.39	3
CSF18	4.06	14	4.05		4.06		4.02		4.10		3.92		4.10		4.11	
CSF19	4.18	11	4.23	8	4.06		4.16		4.22		4.08	10	4.22		4.20	9

Note: Mutual trust (CSF1), effective communication (CSF2), commitment from senior management (CSF3), clear understanding (CSF4), consistent with objectives (CSF5), dedicated team (CSF6), flexibility to change (CSF7), commitment to quality (CSF8), commitment to continuous improvement (CSF9), long-term perspective (CSF10), total cost perspective (CSF11), partnership formation at design stage (CSF12), good cultural fit (CSF13), company wide acceptance (CSF14), technical expertise (CSF15), financial security (CSF16), questioning attitudes (CSF17), availability of resources (CSF18), equal power/empowerment (CSF19).

maintenance of open, direct lines of communication among all project team members [15]. On site problems require immediate resolution once they occur. Partnering will fail if effective communication is used only for routine matters but important issues [5]. Effective communication skills can clearly help in facilitating the exchange of ideas, visions, and solutions [9]. Such exchanges require the formation of effective communication channels, which can be used to motivate partners to jointly participate in planning and goal setting, and thus exert their cooperative efforts to create compatible expectations [16].

Technical expertise ranks second. Construction projects rely on organizing the different levels of the teams involved. These teams include project owners, architects, engineers, consultants, contractors, suppliers, etc. For integrating the abilities, experience, professional knowledge and skills of these teams, and for successfully wrapping up a project, it is crucial to organize the information, skills, requirements, and experience possessed by the above participants. Advantages of partnering include risk sharing, allied problem solving, improving competitive advantages, increasing new markets, and production and benefit boosts, which together result in project success.

Consistent with objectives rank third. The partnering relationship should be formed before contracts are signed (i.e., a pre-project relationship) and should involve all the major stakeholders, including the owners, designers, engineers, general contractors, and key subcontractors. Some initial meetings should be organized for exchanging expectations and goals regarding the relationships among the parties. Moreover, an external expert can be recruited to guide and facilitate the process to reduce misunderstandings among the parties. The partnership goals may be either project-specific or relevant to organizational growth [8]. Some common goals include consistent compliance with environmental regulations, completing the project on schedule, completing the project within budget, enhancing the reputations of the partnering parties, increasing cost-effectiveness, committing to rapidly inform partners of new technologies, committing to sharing best work practices, etc. Since the parties are working as a team and share common goals, they should share resources such as knowledge, information, and technology. Resource exchange relies on the involved parties maintaining absolute trust by not disclosing confidential material to unauthorized parties and by not using such material for internal competitive purposes. Parties are reminded to restrict the leakage of confidential data. Appropriate resources should be those that can be used to accomplish the common project goals.

Questioning attitudes rank fourth. Conflicts frequently occur among parties with incompatible goals and expectations. The influence of conflict resolution can be either productive or destructive, depending largely on the manner in which partners resolve conflict [16]. Because of discrepancies in goals and expectations, conflict frequently occurs among parties. Conflict resolution techniques such as intimidation and confrontation are underproductive and fail to achieve win–win outcomes. In fact, conflicting parties seek mutually satisfactory solutions, which can be achieved by joint problem solving to seek alternative solutions. A high level of communication among parties can help in achieving a mutually acceptable solution. Win–win environments should be established rather than those which create winners at the expense of losers. It also represents open communications and the avoidance of adopting defensive attitudes during arguments. It explains that all team members can make decisions alone owing to clearly identifying responsibility and accountability. Additionally, the sharing or risks and rewards, and a willingness to exchange ideas are illustrated. Participants could make and keep real commitments. A long-term commitment to the process among the parties involved can thus be established.

Commitment to quality ranks fifth. Modernization is making the construction industry more versatile, expansive and complicated, and is causing skills and procedures to evolve into new ideas. Additionally, customers are demanding better quality and durability, increasing the importance of long-term quality in construction projects. Only through a mutual promise to present continuous improvements from both sides in a partnership can projects in progress achieve careful work by contractors, a guarantee of quality, and lasting customer satisfaction [17].

4.2. Factor analysis of CSFs of partnering

Factor analysis was used to explore and detect the underlying relationships among the CSFs. This statistical technique identifies a relatively small number of factors that can be used to represent relationships among sets of multiple interrelated variables. The appropriateness of the factor analysis for the factor extraction needs to be tested in various ways. Factor analysis can be used either in hypothesis testing or in searching for constructs within a group of variables [18]. Factor analysis is a series of methods for identifying clusters of related variables and hence an ideal technique for reducing numerous items into a more easily understood framework [19]. Factor analysis focuses on a data matrix produced from collecting numerous individual cases or respondents. This work applies factor analysis to explore the underlying constructs of the identified CSFs for construction partnering.

In this work, 19 identified CSFs were subjected to factor analysis using principal components analysis and varimax rotation. Principal components analysis is commonly used in factor analysis, and involves generating linear combinations of variables through factor analysis so that they explain as much of the variance present in the collected data as possible. Such analysis summarizes the variability of the observed data via a series of linear combinations of "factors". Each factor can thus be viewed as a "supervariable" comprising a specific combination of the actual variables examined in the survey. The advantage of principle components analysis compared to other factor analytical approaches is that the mathematical representation of the derived linear combinations makes it unnecessary to use questionable causal models [20].

The first stage of the factor analysis involves determining the strength of the relationship among the variables, namely, the *19 identified CSFs* measured by the correlation coefficients of each pair of variables. Table 3 lists the matrix of the correlation coefficients among the CSFs. The matrix is automatically generated along with the factor analysis using the software SPSS 10.0. The correlation coefficients demonstrate that the CSFs share common *factors*. The Bartlett test of sphericity is 1613.353, and the associated significance level is 0.000, indicating that the population correlation matrix is not an identity matrix. Moreover, the value of the Kaiser–Meyer–Olkin (KMO) measure of sampling accuracy is 0.900, which significantly exceeds 0.5 and thus is considered highly acceptable. The results of these tests show that the sample data is appropriate for factor analysis.

Fig. 2 shows the total variance associated with each *factor*. The plot displays a clear break between the steep slope of the large factors and the gradual trailing off of the remaining factors. This gradual trailing off is termed the "scree" because it resembles the rubble that forms at the base of a mountain [19]. Fig. 2 confirms that a four-*factor* model should be sufficient for the research model.

To avoid confusion among the extracted factors and CSFs, the extracted factor was renamed as a "*cluster*" in the interpretation of the results of the analysis. Four *clusters* with eigenvalues greater than 1 were extracted. Table 4 lists the *cluster* matrix following varimax rotation. Each of the CSFs weighs heavily on only one of the *clusters*, with the loading exceeding 0.5 (to round off). Generally, the loadings and the interpretation of the factors extracted were reasonably consistent. Table 5 lists the final statistics of the principal component analysis, and the *clusters* extracted comprise 56.576% of the variance.

4.3. Analyzing participant perspectives regarding four clusters

In accordance with participant roles and project types, one-way ANOVA was performed on the four clusters of construction partnering CSFs. Table 6 summarizes the analysis of variance procedure based on the participant perspectives for the four partnership clusters. The *p*-value indicates the statistical significance of the four clusters (collaborative team culture, long-term quality perspective, consistent objectives and resource sharing). The *p*-value is considered significant when it is below the threshold value of 0.05. As shown in Table 6, the *p*-values of the analysis of variance all exceed 0.05. Consequently, we conclude that no strong link exists among the four clusters, implying no differences exist among the perspectives of government employees, owners, design firms and construction firms regarding the four clusters of construction partnering.

Table 3
Correlation matrix of CSFs for partnering

Factor	CSF1	CSF2	CSF3	CSF4	CSF5	CSF6	CSF7	CSF8	CSF9	CSF10	CSF11	CSF12	CSF13	CSF14	CSF15	CSF16	CSF17	CSF18	CSF19
CSF1	1.000																		
CSF2	0.183	1.000																	
CSF3	0.280	0.270	1.000																
CSF4	0.288	0.330	0.347	1.000															
CSF5	0.317	0.311	0.255	0.460	1.000														
CSF6	0.339	0.229	0.198	0.426	0.410	1.000													
CSF7	0.267	0.348	0.264	0.418	0.306	0.546	1.000												
CSF8	0.182	0.210	0.177	0.288	0.321	0.343	0.323	1.000											
CSF9	0.139	0.214	0.274	0.371	0.345	0.375	0.402	0.720	1.000										
CSF10	0.300	0.263	0.210	0.420	0.331	0.423	0.412	0.437	0.499	1.000									
CSF11	0.190	0.207	0.268	0.174	0.295	0.187	0.283	0.175	0.240	0.266	1.000								
CSF12	0.122	0.232	0.166	0.331	0.264	0.371	0.304	0.353	0.434	0.402	0.195	1.000							
CSF13	0.300	0.190	0.224	0.422	0.330	0.489	0.473	0.332	0.391	0.499	0.165	0.576	1.000						
CSF14	0.191	0.185	0.202	0.354	0.372	0.456	0.423	0.225	0.345	0.386	0.254	0.474	0.564	1.000					
CSF15	0.221	0.319	0.232	0.390	0.378	0.415	0.348	0.322	0.388	0.358	0.158	0.237	0.328	0.341	1.000				
CSF16	0.193	0.284	0.352	0.284	0.360	0.254	0.334	0.309	0.351	0.319	0.424	0.239	0.310	0.366	0.432	1.000			
CSF17	0.197	0.261	0.255	0.450	0.459	0.395	0.390	0.477	0.522	0.471	0.254	0.293	0.416	0.302	0.459	0.458	1.000		
CSF18	0.318	0.279	0.304	0.362	0.300	0.390	0.512	0.310	0.360	0.414	0.398	0.267	0.386	0.457	0.314	0.367	0.445	1.000	
CSF19	0.186	0.312	0.319	0.368	0.325	0.289	0.383	0.247	0.432	0.404	0.314	0.294	0.327	0.397	0.351	0.399	0.393	0.522	1.000

Note: KMO measure of sampling adequacy = 0.900; Bartlett test of sphericity = 1613.353; degree of freedom = 171; significance = 0.000.

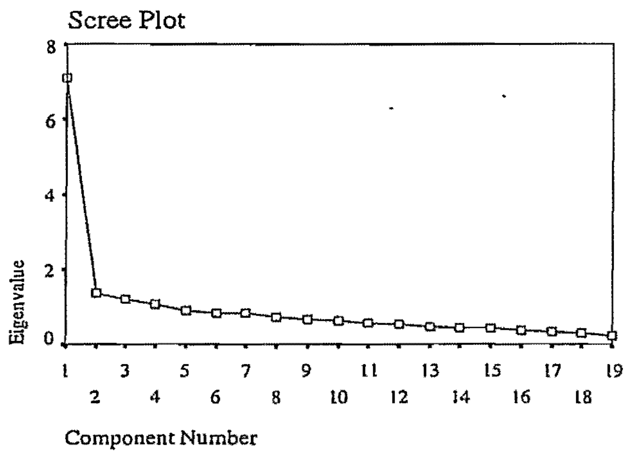

Fig. 2. Total variance associated with each factor.

Table 4
Cluster of matrix after varimax rotation

Factor	Cluster 1	Cluster 2	Cluster 3	Cluster 4
CSF13	0.776			
CSF14	0.741			
CSF12	0.702			
CSF6	0.569			
CSF7	0.499			
CSF10	0.467			
CSF8		0.831		
CSF9		0.802		
CSF17		0.615		
CSF1			0.677	
CSF4			0.607	
CSF5			0.567	
CSF2			0.487	
CSF15			0.482	
CSF11				0.767
CSF16				0.643
CSF19				0.591
CSF18				0.568
CSF3				0.475

Table 5
Final statistic of principle component analysis

Cluster	Eigenvalues	Percentage of variance	Cumulative percentage of variance
1. Collaborative team culture	7.102	37.379	37.379
2. Long-term quality focus	1.375	7.239	44.618
3. Consistent objectives	1.208	6.359	50.977
4. Resource-sharing	1.064	5.598	56.576

Table 7 reveals that three p-values (namely C1, C2, and C3, respectively) are below 0.05, while the p-value of C4 is larger than 0.05. We conclude that there are significant differences among participant viewpoints (HLCP, LLCP, and LSCP) regarding collaborative team culture (C1), long-term quality perspective (C2), and consistent objective. No significant difference is found in the viewpoints of participants (HLCP, LLCP and LSCP) regarding resource sharing (C4). Since LLCP participants expect higher benefits and longer cooperation to ensure a steady business, and since HLCPs are mostly BOT projects and

Table 6
Analysis of variance for construction partnering from the perspective of participant role

Source	D.f.	Sum of squares	Mean square	F ratio	Significance
C1 Between groups	3	48.393	16.131	1.291	.278
Within groups	217	2711.589	12.496		
Total	220	2759.982			
C2 Between groups	3	24.877	8.292	2.521	.059
Within groups	217	713.892	3.290		
Total	220	738.769			
C3 Between groups	3	.601	.200	.039	.990
Within groups	217	1104.947	5.092		
Total	220	1105.548			
C4 Between groups	3	2.650	.883	.143	.934
Within groups	217	1340.309	6.177		
Total	220	1342.959			

Note: Collaborative team culture (C1), a long-term quality perspective (C2), consistent objectives (C3), and resource sharing (C4).

Table 7
Analysis of variance for construction partnering based on different participant perspectives and project types

	D.f.	Sum of squares	Mean square	F ratio	Significance
C1 Between groups	2	145.276	72.638	6.056	.003
Within groups	218	2614.706	11.994		
Total	220	2759.982			
C2 Between groups	2	28.699	14.349	4.405	.013
Within groups	218	710.070	3.257		
Total	220	738.769			
C3 Between groups	2	36.511	18.255	3.723	.026
Within groups	218	1069.037	4.904		
Total	220	1105.548			
C4 Between groups	2	35.668	17.834	2.974	.053
Within groups	218	1307.291	5.997		
Total	220	1342.959			

Note: Collaborative team culture (C1), a long-term quality perspective (C2), consistent objectives (C3), and resource sharing (C4).

similar sized projects are unlikely to be established in Taiwan in the near future, these project participants will eventually focus on long-term quality and mutual cooperation, coordination and loyalty. Accordingly the clusters of collaborative team culture and long-term quality influence LLCP more than HLCP. In addition, LLCPs are generally larger and more complicated, participants expect better communication, mutual trust and understanding, common goals and equivalent professional knowledge and awareness among themselves, while LSCPs are smaller and simpler. Therefore, LLCPs are expected to have more consistent objectives than LSCPs.

4.4. Interpretation of clusters of underlying CSFs

Further discussion requires renaming the *clusters*. Based on an examination of the inherent relationships among the CSFs under each of the *clusters*, the four extracted *clusters* were labeled collaborative team culture, a long-term quality focus, consistent objectives, and resource sharing. The associated explanations of these *clusters* are as follows.

4.4.1. Cluster 1: Collaborative team culture

The six extracted CSFs significances for *cluster* 1 are all related to collaborative team culture, and include good cultural fit, company-wide acceptance, partnership formation at the design stage, a dedicated team, flexibility to change, and a long-term perspective. Lewis [21] advocates the involvement of key suppliers during the design phase of a project. Traditional competitive tendering invites narrow responses as suppliers must meet bidding the specifications to ensure that their offer is considered. By failing to involve suppliers in the design process, considerable potential value may be lost. Lewis argued that this stifles creativity and changes made following a competitive tendering exercise are costly because of the lost time and aborted design costs. One of the key rules related to partnership formation is that to be effective each firm must feel free to question any assumptions made by the other party. Such an arrangement helps parties to understand the reasoning behind the assumptions made and may make the expert party question its own assumptions, sometimes with surprising results.

The findings of Cheng et al. [9] are communicated to external partners and internal staff, and indicate the actions required to achieve change. Commitment and support from partnering organizations are crucial, as they are the sources of transferred knowledge and information. Additionally, internal staff at both the managerial and operational levels should appraise the findings. Management commitment should provide the necessary resources and support for implementing new programs or practices, while employee commitment will accelerate the process of change, since employees are the ones who implement the operational changes. Since a process involves changes to the status quo, particularly cultural change, internal staff familiar with the existing organizational culture may need time to adjust. The team should also encourage feedback through a two-way communication process. Such feedback can maximize understanding and minimize misinterpretation.

Construction projects were dynamic and may change constantly in accordance with the environment the projects involved. Feedback from those affected by a change should be treated carefully. Two-way communication is once again encouraged during this stage. Feedback emphasizes the programs, policies, procedures and practices being restructured to meet the partnership vision, mission, values and goals.

4.4.2. Cluster 2: Long-term quality perspective

This *cluster* contains commitment to quality, commitment to continuous improvement and questioning attitudes. Commitment refers to the willingness of individuals or organizations to exert effort [22]. Moreover, long-term commitment can be considered as the willingness of the involved parties to manage the unanticipated problems continuously [20,23]. More committed parties are expected to balance the attainment of short-term objectives with that of long-term goals, and to achieve both individual and joint missions without fearing opportunistic behavior [16].

Because of different goals and expectations, conflicting issues are generally observed among parties. Conflict resolution techniques such as coercion and confrontation are counterproductive and fail to achieve win–win situations [24,25]. Conflicts are common among parties with incompatible goals and expectations. The influence of conflict resource can be either productive or destructive, and largely depends on the manner in which the partners resolve conflict [16].

In fact conflicting parties look for a mutually satisfactory solution, which can be achieved by joint problem solving to seek alternative solutions to problematic issues. Such a high level of participation among parties may help secure a commitment to a mutually agreed solution [9].

4.4.3. Cluster 3: Consistent objectives

The five CSFs in this *cluster* indicate consistent objectives, including mutual trust, clear understanding, behaving in a manner consistent with objectives, effective communication, and technical expertise. Trust can be defined as the belief that a party can reliably fulfill its obligations in an exchange relationship [26]. Mutual trust is critical to opening the boundaries in a relationship owing to its ability to relieve stress and enhance adaptability [25], increase information exchange and joint problem solving and achieve better outcomes [16].

Compatible goals are the strategic goals of individual organizations that can converge to form the goal of the alliance and help bind the organizations together and establish firm direction, value and activities. According to Lynch [27], partnership failure mainly results from ambiguous goals and poorly coordinated activities. Clarity of focus is thus vital to partnership success. To avoid the pitfalls associated with ambiguous or different goals, participants should ensure they have synchronous goals to begin with, and then review their accomplishments in terms of their original goals at a minimum of three to six monthly intervals. Alliances are less likely to lose sight of their objectives given frequent assessments [28].

Partnering parties have their own preference. Because of cultural diversity, individual parties tend to be dominated by their own goals and objectives, which can be conflicting and consequently may cause adversarial relations [29]. Effective communication can facilitate the exchange of ideas and visions, reducing misunderstandings and stimulating mutual trust. Such communication involves the formation of effective communication channels, which can be used to motivate partners to jointly participate in planning and goal setting and thus cooperate to create compatible expectations [16].

Since a construction project usually requires various skills and technologies, different parties are normally involved (owners, architects, quantity surveyors, structural engineers, contractors, etc.). The complementary expertise of these various parties can strengthen the competitiveness and construction capability of a partnership given appropriate management.

4.4.4. Cluster 4: Resources sharing

The five extracted CSFs significant for *cluster* 4 are all related to resource sharing, including total cost perspective, financial security, equal power/empowerment, availability of resources, and senior management commitments. Owing to resource scarcity and competition for resources, it is rare for an organization to share its own resources, including technology, experience, information, knowledge, capital, power, visions, ideas, and specific skills, with other organizations. Resources can be used to improve partnering relationship competitiveness and construction capability, given effective management [2]. Nevertheless, mutual interaction should be emphasized to enhance resource sharing [30].

It is also important to clarify the maximum use of shared resources. Complementary resources from different parties can not only be used to strengthen the competitiveness and construction capability of partnering relationships [9] but can also provide major criteria for assessing partnership success. Crowley and Karim [7] used the term permeable boundaries to describe the flow of appropriate resources between organizations, and the restriction of the leakage of sensitive and confidential information. In fact, it is important to clarify the maximum use of shared resources, with the main resources being expertise (including knowledge, technology, information, specific skills, and power) and capital.

Senior management commitment and support are prerequisites for successful partnering projects [24,31]. Since senior management formulates the strategy and direction of business activities, their full support and commitment is critical in initiating and leading partnerships [8].

5. Conclusions

This work identifies and ranks the CSFs of project partnering according to importance, which is measured based on the views of experienced construction *professionals* in Taiwan. The findings of this work are generally in line with the conclusions of previous related research.

Using the factor analysis technique, the 19 identified CSFs considered in this work were divided into four *clusters*, with the most important *cluster* being collaborative team culture, followed by a long-term quality focus, consistent objectives, and resource sharing. The results indicate that project owners, designers, contractors, and other related departments who are directly or indirectly involved in this work all significantly influence the success of construction partnering. Consequently successful construction partnering requires a combined effort from all parties involved.

The requirements of the construction partnering team deserve more attention. The results of the factor analysis indicate that the CSFs are related to the collaboration culture of the project partnering team being the most significant influence the output of the construction partnership (accounting for 37.379% of variance in the factor analysis). Adequate preparation of the construction partnering team members is essential for the success of a project partnership. In assembling a construction partnering team, careful consideration should be given to professional experience level, construction partnering study experience, the personalities of the construction partnering team members, and to whether the team has sufficient skills in multiple disciplines.

This study only considers construction partnerships in Taiwan. Future studies could examine international project partnerships, specifically the factors that influence them, to improve understanding of the differences between partnerships in Taiwan and overseas.

Acknowledgment

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國立雲林科技大學
100 學年度博士班招生考試試題

所別：工程科技研究所
科目：工程數學 (1)

本試題共 8 題，共計 100 分，請依題號作答並將答案寫在答案卷上。

1. Find the solution of $y' = y\dfrac{(x-1)^2}{y+3}$, with $y(3) = 1$. (本題 15 分)

2. Find the general solution of $y'' + 4y = 2x + 2e^{-2x}$. (本題 15 分)

3. Use the Laplace transform to solve the initial value problem:
$y'' + 2y' + 2y = 2\delta(t-3)$; $y(0) = y'(0) = 0$. (本題 10 分)

4. Use the Laplace transform to solve the initial problem:
$$\left.\begin{aligned} x' + y' + x - y &= 0 \\ x' + y' + x &= 2 \end{aligned}\right\};\quad x(0) = y(0) = 0.$$ (本題 10 分)

5. Find the Fourier series for the periodic function
$$f(x) = \begin{cases} x + \pi, & \text{for } -\pi < x < \pi \\ f(x + 2\pi) \end{cases}$$ (本題 15 分)

6. Let α and K be positive numbers, and let $f(t) = \begin{cases} \mathrm{K}, & \text{for } -\alpha \le t < \alpha \\ 0, & \text{for } t < -\alpha \text{ and for } t \ge \alpha \end{cases}$.
Find the Fourier transform of $f(t)$. (本題 10 分)

7. Find (if possible) conditions on α, β, γ such that the following system of linear equations has (i) no solution, (ii) exactly one solution, and (iii) an infinite number of solutions. (本題 12 分)
$$\begin{aligned} 2x - y + z &= \alpha \\ 4x + 4y + 8z &= \beta \\ 3y + 3z &= \gamma \end{aligned}$$

8. Solve the system $\begin{bmatrix} x_1'(t) \\ x_2'(t) \end{bmatrix} = \begin{bmatrix} 6 & -3 \\ -2 & 1 \end{bmatrix}\begin{bmatrix} x_1(t) \\ x_2(t) \end{bmatrix} = A\begin{bmatrix} x_1(t) \\ x_2(t) \end{bmatrix}$ (本題 13 分)
(i) find the eigenvalues and eigenvectors of the matrix A. (8 分)
(ii) find (if possible) a nonsingular matrix P and a diagonal matrix D such that $D = P^{-1}AP$. (5 分)

國立雲林科技大學
100學年度博士班招生考試試題

所別：工程科技研究所、電子工程系
科目：工程數學 (2)

1. Solve $y' + y = (x+1)^2$ with $y(0) = 3$. (15%)

2. Solve $4y'' - 4y' - 3y = 0$ with $y(-2) = e$ and $y'(-2) = -0.5e$. (20%)

3. Solve the linear system by the Gauss elimination: (15%)

$$\begin{aligned} 4y + 3z &= 13 \\ x - 2y + z &= 3 \\ 3x + 5y &= 11 \end{aligned}$$

4. (a) If $G(f_x)$ is the Fourier transform of $g(x)$, what are the definitions of "Fourier transform" and "inverse Fourier transform"? (6%)

(b) Find the Fourier transform of the "pulse" function: (10%)

$$g(x) = \begin{cases} 0 & \text{for } x < 3 \text{ and } x \geq 7 \\ 6 & \text{for } 3 \leq x < 7 \end{cases}$$

(c) What is the convolution theorem? (4%)

5. Find first three nonzero terms of the Fourier series which represent the function: (15%)

$$f(x) = \begin{cases} -k & -2 < x < 0 \\ k & 0 < x < 2 \end{cases} \text{ and } f(x+4) = f(x).$$

6. (a) What is the divergence theorem (of Gauss)? (5%)

(b) Find the integral $\iint_S (7x\vec{i} - z\vec{k}) \bullet \vec{n}\,dA$ over the sphere S: $x^2 + y^2 + z^2 = 4$. (5%)

(c) Show that the operation "$\operatorname{div}(\operatorname{curl} \vec{V}) = 0$" is valid for any vector function $\vec{V}$. (5%)

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100學年度博士班招生考試試題

所別：工程科技研究所、環安系
科目：工程數學 (3)

1. Solve the following equations:

(a) $x^4 y' + x^3 y = y^{-3/4}$ (10%)

(b) $y' + \frac{4}{x} y = 2$; $y(1) = -4$ (10%)

(c) $y'' - 3y' + 8y = 0$ (10%)

2. Apply the series method and write the solution with first five terms at least.

$y'' - e^x y' + 2y = 1; \quad y(0) = -3, \ y'(0) = 1$ (10%)

3. Solve the following equation by Laplace transform method

$y'' + 4y = f(t)$; $y(0) = y'(0) = 0$ (10%)

$$f(t) = \begin{cases} 0 & \text{if} \quad 0 \le t < 3 \\ t & \text{if} \quad t \ge 3 \end{cases}$$

4. The heat evolved in calories per gram of a cement mixture is approximately normally distributed. The mean is thought to be 100 and the standard deviation is 2. We wish to test H_0: $\mu = 100$ versus H_1: $\mu \neq 100$ with a sample of $n = 9$ specimens.

(a) If the acceptance region is defined as $98.5 \leqq \bar{x} \leqq 101.5$, find the type I error probability. (10%)

(b) Find type II error probability for the case where the true mean heat evolved is 103. (15%)

5. The fraction of defective integrated circuits produced in a photolithography process is being studied. A random sample of 300 circuits is tested, revealing 13 defectives. Find a 95% two-sided confidence interval on the fraction of defective circuits produced by this particular tool. (10%)

6. A rivet is to be inserted into a hole. A random sample of $n = 15$ parts is selected, and the hole diameter is measured. The sample standard deviation of the hole diameter measurements is $s = 0.008$ millimeters. If the standard deviation of hole diameter exceeds 0.01 millimeters, there is an unacceptably high probability that the rivet will not fit. Is there strong evidence to indicate that the standard deviation of hole diameter exceeds 0.01 millimeters? Use $\alpha = 0.01$. (15%)

國立雲林科技大學
100學年度博士班招生考試試題

所別：工程科技研究所、環安系
科目：工程數學 (3)

$$\Phi(z) = P(Z \leq z) = \int_{-\infty}^{z} \frac{1}{\sqrt{2\pi}} e^{-\frac{1}{2}u^2}\, du$$

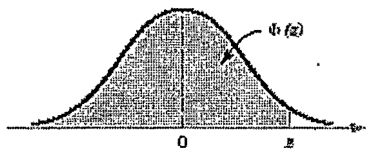

Table II Cumulative Standard Normal Distribution (*continued*)

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.500000	0.503989	0.507978	0.511967	0.515953	0.519939	0.532922	0.527903	0.531881	0.535856
0.1	0.539828	0.543795	0.547758	0.551717	0.555760	0.559618	0.563559	0.567495	0.571424	0.575345
0.2	0.579260	0.583166	0.587064	0.590954	0.594835	0.598706	0.602568	0.606420	0.610261	0.614092
0.3	0.617911	0.621719	0.625516	0.629300	0.633072	0.636831	0.640576	0.644309	0.648027	0.651732
0.4	0.655422	0.659097	0.662757	0.666402	0.670031	0.673645	0.677242	0.680822	0.684386	0.687933
0.5	0.691462	0.694974	0.698468	0.701944	0.705401	0.708840	0.712260	0.715661	0.719043	0.722405
0.6	0.725747	0.729069	0.732371	0.735653	0.738914	0.742154	0.745373	0.748571	0.751748	0.754903
0.7	0.758036	0.761148	0.764238	0.767305	0.770350	0.773373	0.776373	0.779350	0.782305	0.785236
0.8	0.788145	0.791030	0.793892	0.796731	0.799546	0.802338	0.805106	0.807850	0.810570	0.813267
0.9	0.815940	0.818589	0.821214	0.823815	0.826391	0.828944	0.831472	0.833977	0.836457	0.838913
1.0	0.841345	0.843752	0.846136	0.848495	0.850830	0.853141	0.855428	0.857690	0.859929	0.862143
1.1	0.864334	0.866500	0.868643	0.870762	0.872857	0.874928	0.876976	0.878999	0.881000	0.882977
1.2	0.884930	0.886860	0.888767	0.890651	0.892512	0.894350	0.896165	0.897958	0.899727	0.901475
1.3	0.903199	0.904902	0.906582	0.908241	0.909877	0.911492	0.913085	0.914657	0.916207	0.917736
1.4	0.919243	0.920730	0.922196	0.923641	0.925066	0.926471	0.927855	0.929219	0.930563	0.931888
1.5	0.933193	0.934478	0.935744	0.936992	0.938220	0.939429	0.940620	0.941792	0.942947	0.944083
1.6	0.945201	0.946301	0.947384	0.948449	0.949497	0.950529	0.951543	0.952540	0.953521	0.954486
1.7	0.955435	0.956367	0.957284	0.958185	0.959071	0.959941	0.960796	0.961636	0.962462	0.963273
1.8	0.964070	0.964852	0.965621	0.966375	0.967116	0.967843	0.968557	0.969258	0.969946	0.970621
1.9	0.971283	0.971933	0.972571	0.973197	0.973810	0.974412	0.975002	0.975581	0.976148	0.976705
2.0	0.977250	0.977784	0.978308	0.978822	0.979325	0.979818	0.980301	0.980774	0.981237	0.981691
2.1	0.982136	0.982571	0.982997	0.983414	0.983823	0.984222	0.984614	0.984997	0.985371	0.985738
2.2	0.986097	0.986447	0.986791	0.987126	0.987455	0.987776	0.988089	0.988396	0.988696	0.988989
2.3	0.989276	0.989556	0.989830	0.990097	0.990358	0.990613	0.990863	0.991106	0.991344	0.991576
2.4	0.991802	0.992024	0.992240	0.992451	0.992656	0.992857	0.993053	0.993244	0.993431	0.993613
2.5	0.993790	0.993963	0.994132	0.994297	0.994457	0.994614	0.994766	0.994915	0.995060	0.995201
2.6	0.995339	0.995473	0.995604	0.995731	0.995855	0.995975	0.996093	0.996207	0.996319	0.996427
2.7	0.996533	0.996636	0.996736	0.996833	0.996928	0.997020	0.997110	0.997197	0.997282	0.997365
2.8	0.997445	0.997523	0.997599	0.997673	0.997744	0.997814	0.997882	0.997948	0.998012	0.998074
2.9	0.998134	0.998193	0.998250	0.998305	0.998359	0.998411	0.998462	0.998511	0.998559	0.998605
3.0	0.998650	0.998694	0.998736	0.998777	0.998817	0.998856	0.998893	0.998930	0.998965	0.998999
3.1	0.999032	0.999065	0.999096	0.999126	0.999155	0.999184	0.999211	0.999238	0.999264	0.999289
3.2	0.999313	0.999336	0.999359	0.999381	0.999402	0.999423	0.999443	0.999462	0.999481	0.999499
3.3	0.999517	0.999533	0.999550	0.999566	0.999581	0.999596	0.999610	0.999624	0.999638	0.999650
3.4	0.999663	0.999675	0.999687	0.999698	0.999709	0.999720	0.999730	0.999740	0.999749	0.999758
3.5	0.999767	0.999776	0.999784	0.999792	0.999800	0.999807	0.999815	0.999821	0.999828	0.999835
3.6	0.999841	0.999847	0.999853	0.999858	0.999864	0.999869	0.999874	0.999879	0.999883	0.999888
3.7	0.999892	0.999896	0.999900	0.999904	0.999908	0.999912	0.999915	0.999918	0.999922	0.999925
3.8	0.999928	0.999931	0.999933	0.999936	0.999938	0.999941	0.999943	0.999946	0.999948	0.999950
3.9	0.999952	0.999954	0.999956	0.999958	0.999959	0.999961	0.999963	0.999964	0.999966	0.999967

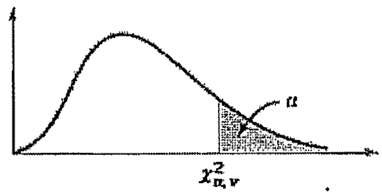

Table III Percentage Points $\chi^2_{\alpha,\nu}$ of the Chi-Squared Distribution

ν \ α	.995	.990	.975	.950	.900	.500	.100	.050	.025	.010	.005
1	.00+	.00+	.00+	.00+	.02	.45	2.71	3.84	5.02	6.63	7.88
2	.01	.02	.05	.10	.21	1.39	4.61	5.99	7.38	9.21	10.60
3	.07	.11	.22	.35	.58	2.37	6.25	7.81	9.35	11.34	12.84
4	.21	.30	.48	.71	1.06	3.36	7.78	9.49	11.14	13.28	14.86
5	.41	.55	.83	1.15	1.61	4.35	9.24	11.07	12.83	15.09	16.75
6	.68	.87	1.24	1.64	2.20	5.35	10.65	12.59	14.45	16.81	18.55
7	.99	1.24	1.69	2.17	2.83	6.35	12.02	14.07	16.01	18.48	20.28
8	1.34	1.65	2.18	2.73	3.49	7.34	13.36	15.51	17.53	20.09	21.96
9	1.73	2.09	2.70	3.33	4.17	8.34	14.68	16.92	19.02	21.67	23.59
10	2.16	2.56	3.25	3.94	4.87	9.34	15.99	18.31	20.48	23.21	25.19
11	2.60	3.05	3.82	4.57	5.58	10.34	17.28	19.68	21.92	24.72	26.76
12	3.07	3.57	4.40	5.23	6.30	11.34	18.55	21.03	23.34	26.22	28.30
13	3.57	4.11	5.01	5.89	7.04	12.34	19.81	22.36	24.74	27.69	29.82
14	4.07	4.66	5.63	6.57	7.79	13.34	21.06	23.68	26.12	29.14	31.32
15	4.60	5.23	6.27	7.26	8.55	14.34	22.31	25.00	27.49	30.58	32.80
16	5.14	5.81	6.91	7.96	9.31	15.34	23.54	26.30	28.85	32.00	34.27
17	5.70	6.41	7.56	8.67	10.09	16.34	24.77	27.59	30.19	33.41	35.72
18	6.26	7.01	8.23	9.39	10.87	17.34	25.99	28.87	31.53	34.81	37.16
19	6.84	7.63	8.91	10.12	11.65	18.34	27.20	30.14	32.85	36.19	38.58
20	7.43	8.26	9.59	10.85	12.44	19.34	28.41	31.41	34.17	37.57	40.00
21	8.03	8.90	10.28	11.59	13.24	20.34	29.62	32.67	35.48	38.93	41.40
22	8.64	9.54	10.98	12.34	14.04	21.34	30.81	33.92	36.78	40.29	42.80
23	9.26	10.20	11.69	13.09	14.85	22.34	32.01	35.17	38.08	41.64	44.18
24	9.89	10.86	12.40	13.85	15.66	23.34	33.20	36.42	39.36	42.98	45.56
25	10.52	11.52	13.12	14.61	16.47	24.34	34.28	37.65	40.65	44.31	46.93
26	11.16	12.20	13.84	15.38	17.29	25.34	35.56	38.89	41.92	45.64	48.29
27	11.81	12.88	14.57	16.15	18.11	26.34	36.74	40.11	43.19	46.96	49.65
28	12.46	13.57	15.31	16.93	18.94	27.34	37.92	41.34	44.46	48.28	50.99
29	13.12	14.26	16.05	17.71	19.77	28.34	39.09	42.56	45.72	49.59	52.34
30	13.79	14.95	16.79	18.49	20.60	29.34	40.26	43.77	46.98	50.89	53.67
40	20.71	22.16	24.43	26.51	29.05	39.34	51.81	55.76	59.34	63.69	66.77
50	27.99	29.71	32.36	34.76	37.69	49.33	63.17	67.50	71.42	76.15	79.49
60	35.53	37.48	40.48	43.19	46.46	59.33	74.40	79.08	83.30	88.38	91.95
70	43.28	45.44	48.76	51.74	55.33	69.33	85.53	90.53	95.02	100.42	104.22
80	51.17	53.54	57.15	60.39	64.28	79.33	96.58	101.88	106.63	112.33	116.32
90	59.20	61.75	65.65	69.13	73.29	89.33	107.57	113.14	118.14	124.12	128.30
100	67.33	70.06	74.22	77.93	82.36	99.33	118.50	124.34	129.56	135.81	140.17

ν = degrees of freedom.

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$$\Phi(z) = P(Z \le z) = \int_{-\infty}^{z} \frac{1}{\sqrt{2\pi}} e^{-\frac{1}{2}u^2}\, du$$

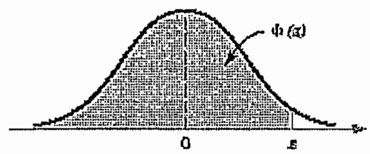

Table II Cumulative Standard Normal Distribution (*continued*)

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.500000	0.503989	0.507978	0.511967	0.515953	0.519939	0.532922	0.527903	0.531881	0.535856
0.1	0.539828	0.543795	0.547758	0.551717	0.555760	0.559618	0.563559	0.567495	0.571424	0.575345
0.2	0.579260	0.583166	0.587064	0.590954	0.594835	0.598706	0.602568	0.606420	0.610261	0.614092
0.3	0.617911	0.621719	0.625516	0.629300	0.633072	0.636831	0.640576	0.644309	0.648027	0.651732
0.4	0.655422	0.659097	0.662757	0.666402	0.670031	0.673645	0.677242	0.680822	0.684386	0.687933
0.5	0.691462	0.694974	0.698468	0.701944	0.705401	0.708840	0.712260	0.715661	0.719043	0.722405
0.6	0.725747	0.729069	0.732371	0.735653	0.738914	0.742154	0.745373	0.748571	0.751748	0.754903
0.7	0.758036	0.761148	0.764238	0.767305	0.770350	0.773373	0.776373	0.779350	0.782305	0.785236
0.8	0.788145	0.791030	0.793892	0.796731	0.799546	0.802338	0.805106	0.807850	0.810570	0.813267
0.9	0.815940	0.818589	0.821214	0.823815	0.826391	0.828944	0.831472	0.833977	0.836457	0.838913
1.0	0.841345	0.843752	0.846136	0.848495	0.850830	0.853141	0.855428	0.857690	0.859929	0.862143
1.1	0.864334	0.866500	0.868643	0.870762	0.872857	0.874928	0.876976	0.878999	0.881000	0.882977
1.2	0.884930	0.886860	0.888767	0.890651	0.892512	0.894350	0.896165	0.897958	0.899727	0.901475
1.3	0.903199	0.904902	0.906582	0.908241	0.909877	0.911492	0.913085	0.914657	0.916207	0.917736
1.4	0.919243	0.920730	0.922196	0.923641	0.925066	0.926471	0.927855	0.929219	0.930563	0.931888
1.5	0.933193	0.934478	0.935744	0.936992	0.938220	0.939429	0.940620	0.941792	0.942947	0.944083
1.6	0.945201	0.946301	0.947384	0.948449	0.949497	0.950529	0.951543	0.952540	0.953521	0.954486
1.7	0.955435	0.956367	0.957284	0.958185	0.959071	0.959941	0.960796	0.961636	0.962462	0.963273
1.8	0.964070	0.964852	0.965621	0.966375	0.967116	0.967843	0.968557	0.969258	0.969946	0.970621
1.9	0.971283	0.971933	0.972571	0.973197	0.973810	0.974412	0.975002	0.975581	0.976148	0.976705
2.0	0.977250	0.977784	0.978308	0.978822	0.979325	0.979818	0.980301	0.980774	0.981237	0.981691
2.1	0.982136	0.982571	0.982997	0.983414	0.983823	0.984222	0.984614	0.984997	0.985371	0.985738
2.2	0.986097	0.986447	0.986791	0.987126	0.987455	0.987776	0.988089	0.988396	0.988696	0.988989
2.3	0.989276	0.989556	0.989830	0.990097	0.990358	0.990613	0.990863	0.991106	0.991344	0.991576
2.4	0.991802	0.992024	0.992240	0.992451	0.992656	0.992857	0.993053	0.993244	0.993431	0.993613
2.5	0.993790	0.993963	0.994132	0.994297	0.994457	0.994614	0.994766	0.994915	0.995060	0.995201
2.6	0.995339	0.995473	0.995604	0.995731	0.995855	0.995975	0.996093	0.996207	0.996319	0.996427
2.7	0.996533	0.996636	0.996736	0.996833	0.996928	0.997020	0.997110	0.997197	0.997282	0.997365
2.8	0.997445	0.997523	0.997599	0.997673	0.997744	0.997814	0.997882	0.997948	0.998012	0.998074
2.9	0.998134	0.998193	0.998250	0.998305	0.998359	0.998411	0.998462	0.998511	0.998559	0.998605
3.0	0.998650	0.998694	0.998736	0.998777	0.998817	0.998856	0.998893	0.998930	0.998965	0.998999
3.1	0.999032	0.999065	0.999096	0.999126	0.999155	0.999184	0.999211	0.999238	0.999264	0.999289
3.2	0.999313	0.999336	0.999359	0.999381	0.999402	0.999423	0.999443	0.999462	0.999481	0.999499
3.3	0.999517	0.999533	0.999550	0.999566	0.999581	0.999596	0.999610	0.999624	0.999638	0.999650
3.4	0.999663	0.999675	0.999687	0.999698	0.999709	0.999720	0.999730	0.999740	0.999749	0.999758
3.5	0.999767	0.999776	0.999784	0.999792	0.999800	0.999807	0.999815	0.999821	0.999828	0.999835
3.6	0.999841	0.999847	0.999853	0.999858	0.999864	0.999869	0.999874	0.999879	0.999883	0.999888
3.7	0.999892	0.999896	0.999900	0.999904	0.999908	0.999912	0.999915	0.999918	0.999922	0.999925
3.8	0.999928	0.999931	0.999933	0.999936	0.999938	0.999941	0.999943	0.999946	0.999948	0.999950
3.9	0.999952	0.999954	0.999956	0.999958	0.999959	0.999961	0.999963	0.999964	0.999966	0.999967

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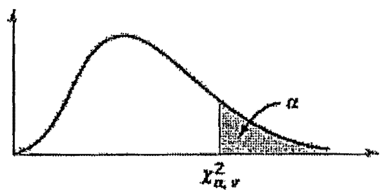

Table III Percentage Points $\chi^2_{\alpha,\nu}$ of the Chi-Squared Distribution

ν \ α	.995	.990	.975	.950	.900	.500	.100	.050	.025	.010	.005
1	.00+	.00+	.00+	.00+	.02	.45	2.71	3.84	5.02	6.63	7.88
2	.01	.02	.05	.10	.21	1.39	4.61	5.99	7.38	9.21	10.60
3	.07	.11	.22	.35	.58	2.37	6.25	7.81	9.35	11.34	12.84
4	.21	.30	.48	.71	1.06	3.36	7.78	9.49	11.14	13.28	14.86
5	.41	.55	.83	1.15	1.61	4.35	9.24	11.07	12.83	15.09	16.75
6	.68	.87	1.24	1.64	2.20	5.35	10.65	12.59	14.45	16.81	18.55
7	.99	1.24	1.69	2.17	2.83	6.35	12.02	14.07	16.01	18.48	20.28
8	1.34	1.65	2.18	2.73	3.49	7.34	13.36	15.51	17.53	20.09	21.96
9	1.73	2.09	2.70	3.33	4.17	8.34	14.68	16.92	19.02	21.67	23.59
10	2.16	2.56	3.25	3.94	4.87	9.34	15.99	18.31	20.48	23.21	25.19
11	2.60	3.05	3.82	4.57	5.58	10.34	17.28	19.68	21.92	24.72	26.76
12	3.07	3.57	4.40	5.23	6.30	11.34	18.55	21.03	23.34	26.22	28.30
13	3.57	4.11	5.01	5.89	7.04	12.34	19.81	22.36	24.74	27.69	29.82
14	4.07	4.66	5.63	6.57	7.79	13.34	21.06	23.68	26.12	29.14	31.32
15	4.60	5.23	6.27	7.26	8.55	14.34	22.31	25.00	27.49	30.58	32.80
16	5.14	5.81	6.91	7.96	9.31	15.34	23.54	26.30	28.85	32.00	34.27
17	5.70	6.41	7.56	8.67	10.09	16.34	24.77	27.59	30.19	33.41	35.72
18	6.26	7.01	8.23	9.39	10.87	17.34	25.99	28.87	31.53	34.81	37.16
19	6.84	7.63	8.91	10.12	11.65	18.34	27.20	30.14	32.85	36.19	38.58
20	7.43	8.26	9.59	10.85	12.44	19.34	28.41	31.41	34.17	37.57	40.00
21	8.03	8.90	10.28	11.59	13.24	20.34	29.62	32.67	35.48	38.93	41.40
22	8.64	9.54	10.98	12.34	14.04	21.34	30.81	33.92	36.78	40.29	42.80
23	9.26	10.20	11.69	13.09	14.85	22.34	32.01	35.17	38.08	41.64	44.18
24	9.89	10.86	12.40	13.85	15.66	23.34	33.20	36.42	39.36	42.98	45.56
25	10.52	11.52	13.12	14.61	16.47	24.34	34.28	37.65	40.65	44.31	46.93
26	11.16	12.20	13.84	15.38	17.29	25.34	35.56	38.89	41.92	45.64	48.29
27	11.81	12.88	14.57	16.15	18.11	26.34	36.74	40.11	43.19	46.96	49.65
28	12.46	13.57	15.31	16.93	18.94	27.34	37.92	41.34	44.46	48.28	50.99
29	13.12	14.26	16.05	17.71	19.77	28.34	39.09	42.56	45.72	49.59	52.34
30	13.79	14.95	16.79	18.49	20.60	29.34	40.26	43.77	46.98	50.89	53.67
40	20.71	22.16	24.43	26.51	29.05	39.34	51.81	55.76	59.34	63.69	66.77
50	27.99	29.71	32.36	34.76	37.69	49.33	63.17	67.50	71.42	76.15	79.49
60	35.53	37.48	40.48	43.19	46.46	59.33	74.40	79.08	83.30	88.38	91.95
70	43.28	45.44	48.76	51.74	55.33	69.33	85.53	90.53	95.02	100.42	104.22
80	51.17	53.54	57.15	60.39	64.28	79.33	96.58	101.88	106.63	112.33	116.32
90	59.20	61.75	65.65	69.13	73.29	89.33	107.57	113.14	118.14	124.12	128.30
100	67.33	70.06	74.22	77.93	82.36	99.33	118.50	124.34	129.56	135.81	140.17

ν = degrees of freedom.