SERIES
SEISMIC ENGINEERING RESEARCH INFRASTRUCTURES FOR EUROPEAN SYNERGIES

Work package [WP10 – TA6, IFSTTAR Centrifuge]

DRESBUS
Centrifuge Modeling of Dynamic Behavior of Box Shaped Underground Structures in Sand

- Final Report -

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Revision: Final

July, 2013
ABSTRACT

Seismic safety of underground facilities such as pipelines, culverts, subways and tunnels becomes an essential requirement for continuing economic and social development. Many engineers earlier thought that the underground structures had been inherently safe against earthquakes, but then, especially after the failure of some underground facilities during 1995 Kobe, Japan, 1999 Kocaeli, Turkey and 1999 Chi Chi, Taiwan earthquakes the safety evaluation of the underground structures became a major concern of the engineers.

This research aimed to investigate the dynamic response of box shaped underground structures buried in dry sand. For this purpose, a series of centrifuge tests were carried out under harmonic sinusoidal motions by considering the nonlinear behavior of both structure and surrounding soil. The dynamic earth pressure is one of the most important parameters in the seismic design of culverts. However, there was no an established methodology clarifying the mechanism and evaluations of the dynamic earth pressures. Hence, response acceleration in the ground, dynamic strains of the buried models and dynamic soil pressures acting on the buried model were examined considering the dynamic soil structure interaction. Effects of input motion parameters and relative stiffness between the soil and underground structure were also taken into account. Influence of relative stiffness on dynamic behavior of underground structures was investigated by using three culvert models with different rigidities. Results of the experiments were evaluated in order to make an assessment on load transfer and deformation mechanism of underground structures in dry sand under different motions. Furthermore, the findings of this study were compared with the predictions of closed-form solutions recommended by Penzien (2000).

Keywords: Box-Shaped Culvert, Dynamic soil pressure, Centrifuge test, Dynamic soil structure interaction, Experimental Analysis,
ACKNOWLEDGMENTS

The research leading to these results has received funding from the European Union Seventh Framework Programme [FP7/2007-2013] under grant agreement n° 227887 [SERIES].
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</tr>
</tbody>
</table>
CONTENTS

CONTENTS................................................................................................................................... i
List of Figures .................................................................................................................................... iii
List of Tables ...................................................................................................................................... vi
1 The Dresbus Project ....................................................................................................................... 1
  1.1 Introduction ............................................................................................................................. 1
2 Centrifuge Model Tests ................................................................................................................ 5
  2.1 Centrifuge Test System ......................................................................................................... 6
    2.1.1 Earthquake Simulator .................................................................................................... 6
    2.1.2 Soil Container ............................................................................................................... 7
    2.1.3 Data acquisition system .............................................................................................. 8
    2.1.4 Accelerometers, Transducers & Strain Gauges ......................................................... 9
  2.2 Reduction Scaling & Scaling Effects ..................................................................................... 11
  2.3 Physical Properties of Soil .................................................................................................... 14
    2.3.1 Dynamic soil properties of Fontainebleau Sand ........................................................... 15
  2.4 Preparation of Model Ground ............................................................................................... 17
  2.5 Design of Culvert Models ..................................................................................................... 18
  2.6 Instrumentation ...................................................................................................................... 20
    2.6.1 Instrumentation in soil model and on ESB box ........................................................... 20
    2.6.2 Instrumentation on the Culvert Model ...................................................................... 21
    2.6.2.1 Accelerometers on the culvert model ................................................................. 21
    2.6.2.2 Extensometers in the culvert model ................................................................. 22
  3 Centrifuge Tests ........................................................................................................................ 21
    3.1 Input Motion Quality & Shaker Performance ................................................................. 21
    3.2 In-Flight CPT .................................................................................................................... 25
    3.3 Testing Program & Test Procedure .................................................................................. 27
4 Free Field Test Results ............................................................................................................................................29
4.1 Maximum Accelerations along Soil Profile .................................................................................................................29
4.2 Evaluation of displacements and shear strains ............................................................................................................32
5 Test Results for Culvert Models....................................................................................................................................34
5.1 Amplification of Maximum Acceleration along Soil Profile .........................................................................................34
5.2 Surface Settlements ..........................................................................................................................................................35
5.3 Shear Strains .................................................................................................................................................................37
5.4 Sidewall Deformations .......................................................................................................................................................44
5.5 Racking Deformations .......................................................................................................................................................71
5.6 Lateral Earth Pressures acting on Sidewalls ....................................................................................................................73
6 Summary & Conclusions....................................................................................................................................................75
References............................................................................................................................................................................77
List of Figures

Fig. 2.1: General view of centrifuge test system in IFSTTAR .................................................. 6
Fig. 2.2: General view of the earthquake simulator .................................................................. 7
Fig. 2.3: General view of the equivalent shear beam (ESB) container ...................................... 8
Fig. 2.4: Rough shear rods for sustaining complementary shear stresses .............................. 8
Fig. 2.5 Fast data acquisition system ....................................................................................... 9
Fig. 2.6: Piezoelectric type accelerometer ................................................................................ 9
Fig. 2.7: Laser displacement sensor ............................................................................................ 9
Fig. 2.8: Horizontal extensometer .............................................................................................. 10
Fig. 2.9: Calibration bench of horizontal extensometer ............................................................... 10
Fig. 2.10: Diagonal Extensometer ............................................................................................ 11
Fig. 2.11: Calibration bench of diagonal extensometer ............................................................. 11
Fig. 2.12: Vertical stress distribution in a centrifuge model and its corresponding prototype (Taylor, 1995) ................................................................. 14
Fig. 2.13: Shear modulus degradation curves proposed by Li et al. (2013) and Darendeli (2001). .................................................................................................................. 16
Fig. 2.14: Damping curves proposed by Li et al. (2013) and Darendeli (2001) ......................... 17
Fig. 2.15: Schematic representation of sand pluviation ............................................................. 17
Fig. 2.16: Cross sections of the culvert models ........................................................................ 18
Fig. 2.17: Layout of the measurement sensors .......................................................................... 19
Fig. 2.18: Schematic drawing of designed extreme sections of the culvert model .................... 20
Fig. 2.19: Layout of accelerometers and laser displacement sensors ........................................ 21
Fig. 2.20: Layout of the accelerometers placed on the culvert model ..................................... 22
Fig. 2.21: Layout of extensometers placed inside the culvert model ........................................ 23
Fig. 3.1: Schematic illustration of shaking table and position of accelerometers ...................... 22
Fig. 3.2: Triaxial acceleration records on the shaking table ...................................................... 23
Fig. 3.3: Closer view of triaxial acceleration records on shaking table ..................................... 24
Fig. 3.4: Fourier transformation of triaxial accelerations records on shaking table ................. 24
Fig. 3.5: CPT device in IFSTTAR ............................................................................................ 25
Fig. 3.6: Variation of tip resistance with respect to depth (Model Scale) ................................. 26
Fig. 3.7: Variation of tip resistance with respect to depth (Prototype Scale) .......................... 26
Fig. 4.1: Layout of the measurement sensors .......................................................................... 30
Fig. 4.2: Variation of maximum acceleration amplification along the soil profile .................. 30
Fig. 4.3: Fourier spectra of the acceleration records along the soil profile in prototype scale a)Soil base b)10.5m below surface c)40cm below surface ....................................................................... 31
Fig. 4.4: Shear strain time history at mid-depth of culvert model (Test 1) ................................. 32
Fig. 5.22: Sidewall horizontal deformation time histories for Test 13  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.21: Sidewall horizontal deformation time histories for Test 12  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.20: Sidewall horizontal deformation time histories for Test 11  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.19: Sidewall horizontal deformation time histories for Test 10  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.18: Sidewall horizontal deformation time histories for Test 9  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.17: Sidewall horizontal deformation time histories for Test 8  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.16: Sidewall horizontal deformation time histories for Test 7  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.15: Sidewall horizontal deformation time histories for Test 6  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.14: Sidewall horizontal deformation time histories for Test 5  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.13: Sidewall horizontal deformation time histories for Test 4  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.12: Sidewall horizontal deformation time histories for Test 3  
a) Deformation time history for right sidewall  
b) Closer view of deformation time history for right sidewall  
c) Deformation time history for left sidewall  
d) Closer view of deformation time history for left sidewall

Fig. 5.11: Shear strain calculated by 1st order and 2nd order approximation vs. depth  
for test 1  
For test 2

Fig. 5.10: Shear strain calculated by 1st order and 2nd order approximation vs. depth  
for test 1  
For test 2

Fig. 5.9: Shear strain calculated by 1st order and 2nd order approximation vs. depth  
for test 1  
For test 2

Fig. 5.8: Shear strain calculated by 1st order and 2nd order approximation vs. depth  
for test 1  
For test 2

Fig. 5.7: Shear strain calculated by 1st order and 2nd order approximation vs. depth  
for test 1  
For test 2

Fig. 5.6: Shear strain calculated by 1st order and 2nd order approximation vs. depth  
for test 1  
For test 2

Fig. 5.5: Layout of accelerometer arrays

Fig. 5.4: Measurement locations of surface settlements

Fig. 5.3: Maximum acceleration amplification vs. depth for Culvert Model #3

Fig. 5.2: Maximum acceleration amplification vs. depth for Culvert Model #2

Fig. 5.1: Maximum acceleration amplification vs. depth for Culvert Model #1

SERIES 227887  DRESBUS Project
Fig. 5.23: Sidewall horizontal deformation time histories for Test 14 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.24: Horizontal deformation recorded by reciprocal extensometers during Test 3
Fig. 5.25: Horizontal deformation recorded by reciprocal extensometers during Test 4
Fig. 5.26: Horizontal deformation recorded by reciprocal extensometers during Test 5
Fig. 5.27: Horizontal deformation recorded by reciprocal extensometers during Test 6
Fig. 5.28: Horizontal deformation recorded by reciprocal extensometers during Test 7
Fig. 5.29: Horizontal deformation recorded by reciprocal extensometers during Test 8
Fig. 5.30: Horizontal deformation recorded by reciprocal extensometers during Test 9
Fig. 5.31: Horizontal deformation recorded by reciprocal extensometers during Test 10
Fig. 5.32: Horizontal deformation recorded by reciprocal extensometers during Test 11
Fig. 5.33: Horizontal deformation recorded by reciprocal extensometers during Test 12
Fig. 5.34: Horizontal deformation recorded by reciprocal extensometers during Test 13
Fig. 5.35: Horizontal deformation recorded by reciprocal extensometers during Test 14
Fig. 5.36: Maximum horizontal deformations recorded on sidewalls
Fig. 5.37: Maximum horizontal deformations recorded on sidewalls
Fig. 5.38: Maximum horizontal deformations recorded on sidewalls
Fig. 5.39: Dynamic earth pressures acting on the underground structure a) Theoretical distribution of shear and normal stress b) Simplified distribution proposed by Wang (1993)
Fig. 5.40: Variation of dynamic lateral coefficient, $k_{sd}$, with respect to IFR
List of Tables

Table 2.1: Characteristics of the earthquake simulator .......................................................... 7
Table 2.2: Scaling laws between prototype and model ............................................................ 12
Table 2.3: Physical properties of soil .................................................................................... 15
Table 2.4: Cross-section dimensions of the culvert models ................................................... 19
Table 3.1: Characteristics of the sinusoidal motions applied in centrifuge tests .................. 23
Table 3.2: Testing program for the dynamic centrifuge tests ............................................... 27
Table 5.1: Surface settlements recorded by laser displacement sensors .............................. 36
Table 5.2: Racking ratios obtained from centrifuge tests and Penzien (2000) solution ........ 72
Table 5.3: Comparison of racking ratios obtained from extensometers .............................. 72
1 The Dresbus Project

1.1 Introduction

During the earthquakes of last century, failures of embedded structures and pipelines have been observed within different ground deformations, settlements, lateral spread and flotation due to soil liquefaction. In most of the studies entreating the seismic performance of the embedded structures, it was revealed that the embedded structures were subjected to damage during the seismic effects and afterwards. ASCE (1974) reported the descriptions of the damages of the buried structures at the city of Los Angeles after the 1971 San Fernando Earthquake. Duke and Leeds (1959), Stevens (1977), Dowding and Rozen (1978), Owen and Scholl (1981), Sharma and Judd (1991), Power et al. (1998) generated databank of damaged underground structures. Power et al. (1998) improved the existing data bank and examined 217 embedded structures with damages (Hashash et al. 2001).

Seismic performance of embedded structures has generally been studied by means of the models constituted on 1g shaking tables. Tamari and Towhata (2003) investigated soil-structure interaction in liquefiable soils by means of the buried models on 1g shaking table. In the study, the alteration in shear modulus of the soil, the relationship between the seismic soil pressure on the model and the deformations of the model were investigated depending upon the amplitudes of the motions applied. Clough and Penzien (1993) reported that the culvert type embedded structures were subjected to shear stresses due to vertically propagating waves during earthquake. Generally, the structures having high rigidity as compared with surrounding soil have been investigated. The soil-structure interaction of the buried structures with flexible sides, which are to be subjected to shear stresses, is scarcely mentioned.
Kobe’s rapid transit railway system experienced significant damage during Hyogo-ken Nanbu (Kobe) earthquake having moment magnitude of 6.9. Iida et al. (1996) reported that the major collapse was observed in the Daikai Subway Station constructed by cut and cover method. Iida et al. (1996) stated that the dynamic lateral forces may cause the relative displacement between base and ceiling of the station. Huo (2005) applied numerical analyses to investigate the dynamic behaviour of Daikai Subway Station under Kobe Earthquake. Numerical analyses showed that vertical accelerations had almost no influence on dynamic response of subway as compared to horizontal acceleration. Moreover, the author emphasized that internal friction and relative stiffness between the ground and structure plays a significant role in seismic design of underground structures. Che and Iwatate (2002) conducted shaking table tests for modeling dynamic response of Daikai Subway Test under Kobe earthquake. It was pointed that the damage is most likely to be caused by dynamic lateral forces.

Rectangular underground structures are usually designed to be able to withstand shear distortions or racking deformations. The most commonly used of all design approaches is the free-field racking deformation analysis. In this approach, deformation of soil in the absence of structure is directly imposed to the structure. In other words, it is assumed that the structure deforms in accordance with the soil. Due to ignorance of relative stiffness between soil and structure, racking deformations of the underground structure may be underestimated or overestimated when the free-field deformation analysis is applied (Hashash et al. 2001). Wang (1993), Penzien (2000) and Huo et al. (2006) proposed simplified frame analysis methodologies for considering soil-structure interaction effect. In these methodologies, first, racking deformations of the structures are estimated on the basis of relative stiffness and free-field deformations. Then, sectional forces occurred due to racking deformations are calculated by performing a static analysis. All those approaches are deformation based methods. There is not any established or accepted forced-based method for the seismic design of underground structure. Current seismic design methods of underground structures have shortcomings and some limitations. They rely on empirical analyses and ignore dynamic soil structure interaction. One commonly used method is based upon Mononobe-Okabe theory for estimating dynamic earth pressures acting on buried structures. However, Mononobe-Okabe method gives unrealistic results and it is not recommended for underground structures with rectangular cross-section (Hashash et al. 2001).
Gravitational body force is an important aspect of geotechnical engineering problems. Physical modeling within geotechnical field is governed by complex stress dependent behavior, especially in soil-structure interaction. Thus, the gravitational force must be well represented during model tests. Such a problem can be overcome by using full scale model tests. Although full scale model tests have the advantage of the actual field conditions, this implementation may be impracticable and feasible. Consequently, many phenomena of interest to the geotechnical engineer cannot be reproduced in laboratory prototype models. The centrifuge technique, hence, becomes an alternative solution in which small scale models are tested within an increased gravity field. The increased stress level enables modeling of soil behavior incorporating soil-structure interaction in small samples.

Centrifuge modeling has been used as a research tool to provide solutions in a realistic way for demanding problems in geotechnical engineering. Hence, the effects of the seismic events have increasingly become an important issue for centrifuge modeling. However, there were only limited cases of study on the seismic behavior of pipe and culvert type buried structures. O’Rourke et al. (2003) performed centrifuge tests to investigate the effect of surface faulting on buried pipes. The fault movement was represented by a split box having two halves, one was fixed and the other was capable of moving horizontally to simulate an offset. The centrifuge test results were compared with the results of Finite Element (FE) models. The pipe strain values measured in the centrifuge were reported to be quite similar with those predicted by FE analyses for small offsets, whereas the similarity was reported to decrease at large offsets where the pipe was in inelastic range. High density polyethylene (HDPE) pipelines crossing fault zones were investigated through centrifuge modeling by Ha et al. (2008). The strains of the pipeline observed at the centrifuge tests were compared with those predicted by the analytical model of Kennedy et al. (1977). Although the experimental data generally showed similar trends with those of Kennedy model, the measured peak strain amplitudes were much less than the analytically predicted values.

Very few experimental and field data are currently available regarding the dynamic response of underground structures. Thus, seismic behavior of underground structures has not been sufficiently clarified yet. More research is needed to investigate the seismic behavior of underground structures.
In this study a series of centrifuge tests were conducted to investigate the dynamic response of box-shaped underground structures buried in sand. The main objectives of the study can be summarized as follows:

- To investigate the deformations of the box-type underground structures subjected to dynamic loading buried in dry sand.
- To examine the effects of flexibility ratio on racking deformation by considering the dynamic soil-structure interaction.
- To evaluate the dynamic soil lateral pressures acting on the box-type culverts.
2 Centrifuge Model Tests

Centrifuge model tests were conducted in the LCPC-IFSTTAR Centrifuge Laboratory located in Nantes-France. The reduction scale of this series experiment is 1/40 so that the experiments are performed in a centrifuge field of 40g. Three types of culvert models having different rigidities were used in the tests. All culverts were box-type underground structures made of aluminium. Thickness of the top and bottom slab was kept constant as 6mm. Height of the culverts was 5 cm and the thicknesses of the sidewalls were 1.5mm, 3mm and 5mm, respectively. Fontainebleau sand NE24 was used as model ground with average particle size diameter of D_{50}=200\mu m. The model ground was prepared by applying pluviation technique to provide uniform and repeatable specimen density. Relative density of the soil was 70% during the tests. In order to check the uniformity of sand specimen Cone Penetration Tests were conducted after pluviation.

Centrifuge tests were performed in absence (free-field condition) and presence of culverts with different stiffness. There were totally 13 tests involving one free-field and 12 box-type model experiments. Simple harmonic motions having peak accelerations of 0.25g and 0.40g with frequencies of 2Hz and 3.5Hz were applied as base excitations. The uniaxial input motions were given only in horizontal directions. Equivalent shear beam box was used as soil container in the centrifuge tests. Thus, shear deformation of soil mode can be simulated under dynamic loading.

Accelerometer and strain gauges were used for instrumentation of buried box-type culvert models. Data obtained from the instrumentation system was used to evaluate the displacements and shear strains along the soil profile and culvert model by considering the effects of input motion characteristics.
2.1 Centrifuge Test System

Fig. 2.1 shows the general view of geotechnical centrifuge facility located in IFSTTAR / NANTES / FRANCE. The test system consists of a motor, a rotating beam of arm around vertical axis, balancing counterweights, a swinging bucket carrying soil container, servo hydraulic shaker and data acquisition system. This centrifuge is a beam type centrifuge with a rotating arm 5.5m in radius. It can rotate with a maximum of 100g centrifugal acceleration by carrying 2 tones model. There is a swinging bucket attached to the end of beam with an area of 1.54m². Soil container, data acquisition and shaking system were placed into this bucket.

![General view of centrifuge test system in IFSTTAR](image)

Fig. 2.1: General view of centrifuge test system in IFSTTAR

2.1.1 Earthquake Simulator

Base excitations were applied via an earthquake simulator having an electro-hydraulic system (Fig. 2.2). The shaker is able to perform harmonic and real earthquake motions in horizontal direction perpendicular to the centrifuge acceleration and gravity acceleration. The characteristics of the earthquake simulator are given in Table 2.1.
Fig. 2.2: General view of the earthquake simulator

Table 2.1: Characteristics of the earthquake simulator

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
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<tbody>
<tr>
<td>Length of shaking table</td>
<td>1m</td>
</tr>
<tr>
<td>Width of shaking table</td>
<td>0.5m</td>
</tr>
<tr>
<td>Payload mass</td>
<td>400kg</td>
</tr>
<tr>
<td>Centrifugal acceleration</td>
<td>20g to 80g</td>
</tr>
<tr>
<td>Maximum displacement</td>
<td>5mm</td>
</tr>
<tr>
<td>Maximum velocity</td>
<td>1m/s</td>
</tr>
<tr>
<td>Maximum acceleration</td>
<td>0.5g</td>
</tr>
<tr>
<td>Frequency range for earthquake motions</td>
<td>20-300Hz</td>
</tr>
<tr>
<td>Frequency range for harmonic motions</td>
<td>20-200Hz</td>
</tr>
</tbody>
</table>

2.1.2 Soil Container

Equivalent shear beam (ESB) box was used as a model container in dynamic centrifuge tests (Fig. 2.3). Side walls of ESB box was allowed to free in shaking direction and restrained in transverse direction. Internal dimensions of ESB container is 0.80m in length, 0.35m in width and 0.41m in height. ESB box consists of 14 aluminium alloy horizontal layers, separated by rubber sheets having 4mm thickness. Thus, soil can conform to soil deformations and boundary effects are eliminated. Dynamic centrifuge tests conducted on this container show that the resonant frequency of ESB box varied between 32 and 37 Hz (Escoffier, 2008).
In order to sustain the complementary shear stresses developed on the vertical face of the container, rough shear rods were used throughout the dynamic centrifuge tests (Fig. 2.4).

![Fig. 2.3: General view of the equivalent shear beam (ESB) container](image)

![Fig. 2.4: Rough shear rods for sustaining complementary shear stresses](image)

2.1.3 Data acquisition system

An LMS fast data acquisition system (Fig. 2.5) was used in the centrifuge tests. The system had 72 channels for voltage or ICP accelerometers and 48 channels for strain gauge, voltage and ICP accelerometers. A 24 bit resolution data acquisition card was located in the system supporting sampling rates from 100Hz to 25kHz. The system was placed into the swinging basket and data was transmitted to control room through wireless network.
2.1.4 Accelerometers, Transducers & Strain Gauges

Bruel & Kjaer 4317-10 mv/g type piezoelectric accelerometers (Fig. 2.6) were used in the dynamic tests. They were very compact and lightweight, therefore particularly suitable for the centrifuge tests. A natural internal bandpass filter of 1Hz to 20kHz existed in the accelerometers. There were totally 26 accelerometers buried in soil and mounted on culvert model and equivalent shear beam container.

Laser displacement (Fig. 2.7) sensors were used for measuring the soil settlement before and after shaking. Due to low frequency rate of measurement, it was not possible to obtain reliable information during dynamic loading. The displacement sensors were mounted on a suspended beam over the ESB container.
In order to measure the deformations of the box-type culvert, horizontal extensometers and diagonal extensometers developed by IFSTTAR and GAROS were used in the centrifuge experiments. There were five pairs of horizontal extensometers piled together on a fork shaped system as illustrated in Fig. 2.8. Each extensometer included two strain gauges giving the bending deformation of the extensometer.

![Horizontal extensometer](image1.png)

**Fig. 2.8: Horizontal extensometer**

A calibration bench (Fig. 2.9) was developed by IFSTTAR to calibrate the horizontal extensometers. The piled fork system was placed into calibration device and each extensometer was calibrated individually in a simple way by giving deformation and recording voltage values.

![Calibration bench of horizontal extensometer](image2.png)

**Fig. 2.9: Calibration bench of horizontal extensometer**
Diagonal extensometers (Fig. 2.10) were developed by IFSTTAR for measuring diagonal deformation along the tunnel. To investigate the boundary effects due to end walls and validity of plain strain conditions, extensometers were placed in the middle and near the end of tunnel model. For calibration purposes, a special calibration device (Fig. 2.11) was designed by IFSTTAR. In the calibration process, at first, the extensometer was deformed until the length of bow chord was equal to the definite length of culvert diagonal. Then, changes in output voltages were recorded by changing the bow chord length of the extensometer.

Fig. 2.10: Diagonal Extensometer

Fig. 2.11: Calibration bench of diagonal extensometer

2.2 Reduction Scaling & Scaling Effects

A series of centrifuge tests were conducted by IFSTTAR to investigate the performance ESB box at different centrifugal acceleration “g” levels. Results showed that 40g was the most reasonable centrifugal acceleration to eliminate the boundary effects of soil model. For this reason, dynamic
centrifuge tests were performed under 40g of centrifugal acceleration. Scaling laws between prototype and model were derived for a 40g centrifugal field and presented in Table 2.2.

Table 2.2: Scaling laws between prototype and model

<table>
<thead>
<tr>
<th>Parameter</th>
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<tr>
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</tbody>
</table>

Acceleration level is not constant within the soil model. Due to change in radius, acceleration along the depth of soil model increases proportionally with the radius. In this study, the mid-depth (7.5cm) of the box-type culvert model was taken as a reference point, and the corresponding radius is selected as $r_{Ng}=4.794m$. If a scale factor is represented by “n” then, $a(z_{Ng})$ is defined as follows (Taylor, 1995):

$$a(z_{Ng}) = \omega^2 a_{Ng} = ng$$

(2.1)

Likewise, the centrifuge acceleration $a(z)$ at any depth can be calculated as follows:

$$a(z) = \omega^2 r_z$$

(2.2)

where $\omega$ is the angular frequency and $r_z$ is the radius at depth $z$:

$$r_z = r_{Ng} + (z - z_{Ng})$$

(2.3)

The vertical stress $\sigma_z$ at depth $z$ can be calculated by:
\[ \sigma_z = \int_0^z \rho u(z)d(z) \] (2.4)

Then, using Equation 2.2, 2.3, 2.4, vertical stress can be given by:

\[ \sigma_z = \int_0^z \rho \omega^2 \left[ r_{ng} + (z - z_{ng}) \right] d(z) \] (2.5)

The solution for vertical stress is obtained as:

\[ \sigma_z = \rho g \left[ z + \frac{z^2}{2r_{ng}} - \frac{z_{ng} z}{r_{ng}} \right] \] (2.6)

Equation 4-7 gives the stress at depth \( z \) in the model under a gravitational field of \( n_g \). At the prototype scale the stress can be defined as:

\[ \sigma_z = \rho g z_p \] (2.7)

where \( z_p \) is the depth in the prototype. If the stresses are to be the same in model and prototype, then \( z_p \) is computed as follows:

\[ z_p = N \left[ z + \frac{z^2}{2r_{Ng}} - \frac{z_{Ng} z}{r_{ng}} \right] \] (2.8)

where \( n=40, z_{Ng}=0.075m \) and \( r_{ng}=4.794m \)

Vertical stress distribution along the depth of a soil profile in a centrifuge model and its corresponding prototype is depicted in Fig. 2.12. As can be seen from Fig. 2.12, there is a difference between the stress fields between centrifuge model and prototype. Maximum error (\( err_{max} \)) due to stress difference is given by:

\[ err_{max} = \frac{h_m}{6R_e} \] (2.9)

where \( h_m \) is the height of model and \( R_e \) is the effective centrifuge radius obtained from:

\[ R_e = R_t + h_m / 3 \] (2.10)

where \( R_t \) is the measured distance from rotation axis to the top of model.
In the present study, $err_{\text{max}}$ due to stress difference remains below $\%1.5$, hence it could be negligible.

![Vertical stress distribution in a centrifuge model and its corresponding prototype (Taylor, 1995)](image)

In a centrifuge test, linear dimensions are reduced by a scale factor. In most cases, this reduction factor is not applied for particle size of the soil so as to represent stress-strain characteristics of prototype. However, it should be noted that particle size effect can appear in soil structure interaction problems. Iglesia et al. (2011) suggested that the grain size effects can be minimized when the ratio of model dimensions to the average particle size of soil is at least 20. This ratio was specified as 235 for the present study, thus grain size effect was eliminated (Ulgen, 2011).

### 2.3 Physical Properties of Soil

All centrifuge tests were performed in dry Fontainebleau sand. It is a fine quartz sand composed of round shaped grains. Median grain size ($D_{50}$) of the Fontainebleau Sand is 0.20mm. Physical properties of the sand are summarized in Table 2.3.
Table 2.3: Physical properties of soil

<table>
<thead>
<tr>
<th>Soil</th>
<th>Fontainebleau Sand NE34</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e_{\text{min}}$</td>
<td>0.55</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>0.86</td>
</tr>
<tr>
<td>$\gamma_{\text{min}}$ (kN/m$^3$)</td>
<td>13.93 kN/m$^3$</td>
</tr>
<tr>
<td>$\gamma_{\text{max}}$ (kN/m$^3$)</td>
<td>16.78 kN/m$^3$</td>
</tr>
<tr>
<td>Mean diameter ($D_{50}$)</td>
<td>0.20 mm</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.64</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>38°</td>
</tr>
</tbody>
</table>

2.3.1 Dynamic soil properties of Fontainebleau Sand

A rough estimate for the maximum shear modulus of Fontainebleau sand was made by using the following empirical relationship (Eq. 2.11) proposed by Hardin and Drnevich (1970):

$$G_{\text{max}} = 3230\left(\frac{2.973-e}{1+e}\right)^2 \cdot OCR \cdot (\sigma_m')^{1/2}$$

(2.11)

where $G_{\text{max}}$ is the maximum shear modulus in kPa, $e$ is the void ratio, OCR is the ratio of maximum past effective stress to current effective stress, $k$ is an overconsolidation ratio exponent and $\sigma_m'$ is the effective confining pressure in kPa. For a void ratio of 0.64 and an effective confining pressure of 28 kPa maximum shear modulus at mid-depth of culvert model was approximately calculated as 56500 kPa.

Li et al. (2013) experimentally studied on the dynamic properties of Fontainebleau sand with emphasis on shear modulus degradation and damping ratio. They carried out dynamic centrifuge tests at a centrifugal acceleration of 40g using dry sand. Based on the test results, specific values of empirical equations proposed by Ishibashi and Zhang (1993) were determined for describing shear modulus reduction and damping ratio with respect to shear strain. Simplified form of the empirical equation (Eq. 2.12, Eq. 2.13, Eq. 2.14) for shear modulus degradation curve was given as:
\[
\frac{G}{G_{\text{max}}} = K(\gamma)\sigma_c^{m(\gamma)-m_0}
\]  
(2.12)

\[
K(\gamma) = 0.5 \left[ 1 + \tanh \left\{ \ln \left( \frac{0.000102}{\gamma} \right)^{0.613} \right\} \right]
\]  
(2.13)

\[
m(\gamma) - m_0 = 0.34 \left[ 1 - \tanh \left\{ \ln \left( \frac{0.000556}{\gamma} \right)^{0.4} \right\} \right]
\]  
(2.14)

where \(\gamma\) is the shear strain.

Damping ratio for Fontainebleau sand was expressed as a function of \(G/G_{\text{max}}\) (Eq. 2.15):

\[
D = 25.3 \left\{ 0.513 \left( \frac{G}{G_{\text{max}}} \right)^2 - 1.351 \left( \frac{G}{G_{\text{max}}} \right) + 1 \right\}
\]  
(2.15)

Variation of shear modulus degradation and damping ratio with respect to shear strain proposed by Li et al. 2013 and Darendeli (2001) are illustrated in Fig. 2.13 and Fig. 2.14, respectively. The curves are plotted for a confining pressure of 28kPa, representing stress state of mid-depth of the culvert model. As seen from Fig. 2.13 and Fig. 2.14, shear modulus and damping curves are consistent with each other. In this study, the curves proposed by Li et al. (2013) were used when evaluating the dynamic response of the ground model.

![Figure 2.13: Shear modulus degradation curves proposed by Li et al. (2013) and Darendeli (2001).](image-url)
2.4 Preparation of Model Ground

The model ground was prepared by pluviation technique to achieve an average relative density of 70%. In this technique, dry sand was rained into the model container from a height of 60cm by means of an automatic hopper (Fig. 2.15, Fig. 2.16). Density control boxes were buried into model ground to check uniformity and to measure relative density of soil. Furthermore, in-flight cone penetration tests (CPT) were performed for evaluating the homogeneity of the sand model.
2.5 Design of Culvert Models

The underground culvert models were made of aluminium. They were manufactured by IFSTTAR by the use of electro-erosion technology. Hence, it was possible to avoid stress concentrations, discontinuity and particularly prestressing. There were totally three box-types of culvert models used in the centrifuge tests. Internal dimensions of these culverts were kept constant for simplicity in instrumentation, whereas outside dimensions varied with thickness of sidewalls to obtain different rigidities. Thickness of the roof and invert slabs were relatively thick and stiff as compared to that of sidewalls for eliminating structural effects due to bending. Cross sections and dimensions of the culvert models are given in Fig. 2.17 and Table 2.4, respectively.
Table 2.4: Cross-section dimensions of the culvert models

<table>
<thead>
<tr>
<th>Culvert Model</th>
<th>Internal Dimensions (mm)</th>
<th>External Dimensions (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
</tr>
<tr>
<td>1 Thinnest Walls</td>
<td>38</td>
<td>44</td>
</tr>
<tr>
<td>2 Intermediate Thickness</td>
<td>38</td>
<td>44</td>
</tr>
<tr>
<td>3 Thickest Walls</td>
<td>38</td>
<td>44</td>
</tr>
</tbody>
</table>

In order to satisfy the plain strain conditions, dimension ratio between the length and width of culvert model was approximately specified as 7. Besides, extremities of culvert section and interface between the tunnel and ESB box were designed to provide free movement at culvert ends. Fig. 2.18 shows the schematic drawing of the designed extreme sections. As seen from Fig. 2.18, longitudinal sides of ESB box was covered with Teflon sheet to reduce the surface friction. Furthermore, neoprene foams were placed at the end of culvert to allow deformation of culvert extremities.
2.6 Instrumentation

2.6.1 Instrumentation in soil model and on ESB box

There were totally 16 accelerometers buried in the soil model, 6 accelerometers were placed on ESB box and 3 laser displacement sensors over the container as shown in Fig. 2.19. As seen in Fig. 2.19 the accelerometers were distributed into five arrays. The main purposes of this instrument layout can be summarized as follows:

- To analyse boundary conditions and free-field behavior.
- To investigate the shear strain and acceleration response of soil near culvert model.
- To investigate acceleration response in soil
- To analyze the effect of the box on the experiments.
- To measure the soil settlement at the surface.
2.6.2 Instrumentation on the Culvert Model

2.6.2.1 Accelerometers on the culvert model

There were totally six accelerometers placed outside of the culvert model. Two of them were located in the central section, at the bottom and top of the model. They measured the horizontal accelerations in shaking direction. Two more horizontal accelerometers were mounted at each end of culvert to check whether plain strain behaviour was maintained or not. Besides, two vertical accelerometers were glued on the culvert to investigate the vertical movement of the buried model. Fig. 2.20 shows the layout of accelerometers placed on the culvert model.
2.6.2.2 Extensometers in the culvert model

Position of the extensometers placed inside the culvert was illustrated in Fig. 2.21. As shown in Fig. 2.21, five pairs of horizontal extensometers were mounted at the central section of the culvert model to measure the transverse deformation of sidewalls during dynamic centrifuge tests. Furthermore, four diagonal extensometers were placed inside the culvert to measure the change in diagonal length. Two of them were located near the central and the other two were located near the ends of culvert model. Thus, it was possible to check whether plain strain behaviour was valid for the culvert model under dynamic loading.
Fig. 2.21: Layout of extensometers placed inside the culvert model
3 Centrifuge Tests

3.1 Input Motion Quality & Shaker Performance

In order to evaluate the input motion quality and the shaker performance, controlled experiments were performed by using accelerometers. In the evaluation process, following conditions were taken into consideration:

- To check whether the defined motion by shaker is consistent with the obtained motion in terms of amplitude, frequency content and duration.
- To analyse spurious accelerations occurred in X and Z directions due to yawing or rocking of the shaking table. Here, Z is the direction of centrifugal acceleration and X is the direction of horizontal acceleration perpendicular to shaking direction Y.

A procedure was developed by IFSTTAR for analysing the above conditions. In this procedure, first, a control accelerometer set was placed on the ESB box and shaking table as shown in Fig. 3.1. Then, several excitations were applied by shaker using control loops. There was a software program sending the command by an electric signal to servo-valves in shaking system. Servo-valves transform the electric signal into output hydraulic signal and accordingly jacks pushes the shaking table. Two accelerometers in Y direction were glued on the shaking table to construct a control loop. Several shakes were performed for calibrating the transfer function of the servo-hydraulic system to obtain the target signal at the bottom of box. Furthermore, as seen in Fig. 3.1, a triaxial accelerometer was placed on the table so as to verify the consistence of transmitted motion and to analyse the spurious accelerations in X and Z directions.
There were four types of harmonic motions having different frequencies and acceleration amplitudes applied in the dynamic centrifuge tests. Characteristics of the motions were given in Table 3.1. Input quality and shaker performance was checked under these sinusoidal motions. As an example, the acceleration records obtained from triaxial accelerometer placed at the end of table (Fig. 3.1) under the motion “sin80hz-8gh” are given in Fig. 3.2 and Fig. 3.3. Besides, fourier spectra of the acceleration records is shown in Fig. 3.4. As can be seen from these figures, spurious acceleration components arise in X and Z directions. The quality criteria for the spurious accelerations and input acceleration should be normally:

- Input acceleration in Y direction should be close to the expected reference signal. The difference on the RMS acceleration should be smaller than 5
• Spurious RMS accelerations in X and Z directions should be less than 10% of the RMS accelerations in Y direction.

**Table 3.1: Characteristics of the sinusoidal motions applied in centrifuge tests**

<table>
<thead>
<tr>
<th>Model Scale</th>
<th>Applied Motion</th>
<th>Acceleration (g)</th>
<th>Frequency (hz)</th>
<th>Duration (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sin80hz-10gh</td>
<td>10</td>
<td>80</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>Sin80hz-16gh</td>
<td>16</td>
<td>80</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>Sin140hz-10gh</td>
<td>10</td>
<td>140</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Sin140hz-16gh</td>
<td>16</td>
<td>140</td>
<td>0.50</td>
</tr>
</tbody>
</table>

**Fig. 3.2: Triaxial acceleration records on the shaking table**
Fig. 3.3: Closer view of triaxial acceleration records on shaking table

Fig. 3.4: Fourier transformation of triaxial accelerations records on shaking table
3.2 In-Flight CPT

A series of in-flight cone penetration tests (CPT) were carried out in order to verify the uniformity and repeatability of the soil model prepared by pluviation. Following figure (Fig. 3.5) shows the CPT equipment set on the ESB box in the centrifuge tests before the shake. The CPT device has a 12mm diameter rod with a 500daN sensor at the end for measuring the cone tip resistance. CPTs were performed before and after shake and side friction was not measured during the tests. The step by step testing procedure for CPT can be explained as follows:

- The CPT equipment was set on ESB box before beginning centrifuge tests.
- The centrifuge was then switched on and it accelerates smoothly up to a centrifuge acceleration of 40g.
- When the centrifuge spins with a constant acceleration of 40g, CPT was conducted and test data were taken via remote connection.
- The centrifuge is switched off and CPT equipment is demounted.
- Laser displacement sensors were placed over the box to measure the soil settlement during the shake.
- CPT was repeated to observe the changes in tip resistance after the shake.

Fig. 3.5: CPT device in IFSTTAR
Fig. 3.6: Variation of tip resistance with respect to depth (Model Scale)

Fig. 3.7: Variation of tip resistance with respect to depth (Prototype Scale)
3.3 Testing Program & Test Procedure

In this study, two free field tests and twelve dynamic centrifuge tests were conducted on underground structure models buried in dry sand. Each model was tested under four different harmonic motions. The testing program can be summarized as indicated in Table 3.2.

<table>
<thead>
<tr>
<th>Tests #</th>
<th>Culvert Model</th>
<th>Desired Input Acceleration (Prototype) (g)</th>
<th>Measured Input Acceleration (Prototype) (g)</th>
<th>Frequency (Prototype) (hz)</th>
<th>Duration (Prototype) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No culvert: Free-field</td>
<td>0.25</td>
<td>0.25</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>2</td>
<td>No culvert: Free-field</td>
<td>0.4</td>
<td>0.46</td>
<td>3.5</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>0.25</td>
<td>0.28</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>0.25</td>
<td>0.27</td>
<td>3.5</td>
<td>20</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>0.4</td>
<td>0.47</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>0.4</td>
<td>0.42</td>
<td>3.5</td>
<td>20</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>0.25</td>
<td>0.27</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>8</td>
<td>2</td>
<td>0.25</td>
<td>0.28</td>
<td>3.5</td>
<td>20</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>0.4</td>
<td>0.41</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>10</td>
<td>2</td>
<td>0.4</td>
<td>0.4</td>
<td>3.5</td>
<td>20</td>
</tr>
<tr>
<td>11</td>
<td>3</td>
<td>0.25</td>
<td>0.27</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>12</td>
<td>3</td>
<td>0.25</td>
<td>0.28</td>
<td>3.5</td>
<td>20</td>
</tr>
<tr>
<td>13</td>
<td>3</td>
<td>0.4</td>
<td>0.49</td>
<td>2</td>
<td>35.2</td>
</tr>
<tr>
<td>14</td>
<td>3</td>
<td>0.4</td>
<td>0.39</td>
<td>3.5</td>
<td>20</td>
</tr>
</tbody>
</table>

All tests were performed under 40g centrifugal acceleration. Details of testing procedure are described in the following steps.

- ESB box was filled with dry Fontainebleau sand by using pluviation technique and the relative density of the ground model was obtained as 70%.
• During pluviation, accelerometers were placed at various locations in the soil as shown in Fig. 2.19. When the soil height is 216mm, the underground structure model equipped with extensometers and accelerometers was placed in the middle of ESB box. After that, accelerometers were buried into the soil and ESB box is filled with dry sand.

• CPTs were performed so as to check the uniformity and homogeneity of the soil.

• After CPT, the centrifuge was started and spun at 40g centrifugal acceleration. During the flight, a harmonic motion was applied from the base of ESB box in horizontal direction perpendicular to the plane of rotation of centrifuge.

• Experimental data obtained from accelerometers, extensometers and laser displacement sensors were taken remotely and recorded.

• After recording data, the model was prepared again and centrifuge test was repeated for various harmonic motions having different accelerations and frequencies.
4  Free Field Test Results

Dynamic free field tests were conducted at a centrifugal acceleration of 40g. Peak accelerations of the motion were 0.25g and 0.4g and frequencies of the motion were 2Hz and 3.5Hz, respectively. Layout of the measurement sensors is given in Fig. 4.1. There were four accelerometer arrays buried in the soil and mounted on the ESB box, namely: a) V1 (6,19,20,21), b) V2 (5,15,16,17,18), c) V3 (2,3,4,11,12,13,14)  d) V4 (7,8,9,10). Since, the region around the culvert model was of interest, most of the accelerometers were placed at different depths where culvert model was to be buried. The test results were used to examine the acceleration response of model ground and strain levels in the soil.

4.1  Maximum Accelerations along Soil Profile

Fig. 4.2 shows the variation of maximum acceleration amplification factor along the soil profile. Amplification factor is defined as the ratio of maximum recorded acceleration within the soil to the maximum input acceleration recorded on the shaking table. As seen from Fig. 4.2, acceleration amplification gradually increases from the bottom of soil profile to 4m, but then, it increases sharply near the surface. Fourier spectra of acceleration time histories recorded at different depths in the model ground were illustrated in Fig. 4.3. As seen from the figure, at low frequencies, Fourier amplitude increases from the bottom toward the surface. On the other hand, soil tends to damp high frequency contents of the motion (Ulgen, 2011).
Fig. 4.1: Layout of the measurement sensors

Fig. 4.2: Variation of maximum acceleration amplification along the soil profile
Fig. 4.3: Fourier spectra of the acceleration records along the soil profile in prototype scale
a) Soil base  b) 10.5m below surface  c) 40cm below surface
4.2 Evaluation of displacements and shear strains

Displacement responses were calculated through integrating twice the acceleration time histories. Before the integration process, uncorrected acceleration signals were filtered using the bandpass filtering, to avoid the unwanted errors and misinterpreting data. There were totally twenty-three accelerometers used in free-field tests in Fig. 4.1. Average shear strain between the two successive accelerometers can be calculated by using first order approximation (Eq. 3.1):

\[ \gamma = \frac{d_2 - d_1}{z_2 - z_1} \]  

(3.1)

where \( \gamma \) is the shear strain, \( d_1, d_2 \) are the displacements at points 1, 2 and \( z_1, z_2 \) are the depths of points 1, 2, respectively. If there is a three-accelerometer vertical array in a soil column, a better second order approximation of shear strain at depth \( z_i \) can be calculated by (Eq. 3.2) (Zeghal & Elgamal, 1994):

\[
\gamma(z_i) = \frac{\left[ (d_{i+1} - d_i) \left( \frac{z_i - z_{i-1}}{z_{i+1} - z_i} \right) + (d_i - d_{i-1}) \left( \frac{z_{i+1} - z_i}{z_i - z_{i-1}} \right) \right]}{(z_{i+1} - z_{i-1})}
\]  

(3.2)

Variation of shear strain calculated from first and second order approximations were plotted against soil depth as shown in Fig. 4.5. An example shear strain time history calculated in Test 1 at mid-depth of culvert model is given in Fig. 4.4.

![Fig. 4.4: Shear strain time history at mid-depth of culvert model (Test 1)](image-url)
Fig. 4.5: Shear strain calculated by 1\textsuperscript{st} order and 2\textsuperscript{nd} order approximation vs. depth a) For test 1 b) For test 2
5 Test Results for Culvert Models

5.1 Amplification of Maximum Acceleration along Soil Profile

Variation of maximum acceleration with respect to soil depth for tests conducted on culvert models #1, #2 and #3 are illustrated in Fig. 5.1 to Fig. 5.3. The maximum acceleration measured at the base of model ground was almost the same with that of input motion for all the tests. There may be a small amount of sliding on the soil-ESB container interface. Although acceleration amplification gradually increases from base of ESB box up to level of 4m below the surface where the base of the culvert models are located over, a remarkable increase in amplification takes place within the stratum of 4m in thickness just below the surface. As seen in the figures, the amplification effect of the soil decreases with increasing characteristics (acceleration and frequency) of the input excitation for all culvert models.

Fig. 5.1: Maximum acceleration amplification vs. depth for Culvert Model #1
Fig. 5.2: Maximum acceleration amplification vs. depth for Culvert Model #2

Fig. 5.3: Maximum acceleration amplification vs. depth for Culvert Model #3

5.2 Surface Settlements

The resultant surface settlements after shaking were measured by means of laser displacement sensors at three different locations as given in Fig. 5.4. The settlements measured by those sensors are given in Table 5.1. Approximate settlements observed at different locations points
out a homogeneous settlement along the soil profile. Hence, the surface settlement at the end of the test can be indicated in terms of average. Average settlement is varying between 3.5mm – 6mm depending on the characteristics of the input motion.

Fig. 5.4: Measurement locations of surface settlements

Table 5.1: Surface settlements recorded by laser displacement sensors

<table>
<thead>
<tr>
<th>Tests #</th>
<th>Culvert Model #</th>
<th>Surface Settlement (mm) (Left Side)</th>
<th>Surface Settlement (mm) (Center)</th>
<th>Surface Settlement (mm) (Right Side)</th>
<th>Average Settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>1</td>
<td>4.2</td>
<td>4.1</td>
<td>4.7</td>
<td>4.3</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>4.3</td>
<td>4.6</td>
<td>4.5</td>
<td>4.5</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>7.4</td>
<td>8.0</td>
<td>7.0</td>
<td>7.5</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>6.0</td>
<td>5.7</td>
<td>6.0</td>
<td>5.9</td>
</tr>
<tr>
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<td>4.7</td>
<td>5.1</td>
<td>4.9</td>
</tr>
<tr>
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</tr>
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<td>6.0</td>
<td>5.6</td>
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</tr>
<tr>
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<td>3.4</td>
<td>4.1</td>
<td>3.9</td>
<td>3.8</td>
</tr>
<tr>
<td>12</td>
<td>3</td>
<td>4.7</td>
<td>4.0</td>
<td>4.3</td>
<td>4.3</td>
</tr>
<tr>
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<td>3</td>
<td>3.8</td>
<td>3.9</td>
<td>3.7</td>
<td>3.8</td>
</tr>
<tr>
<td>14</td>
<td>3</td>
<td>5.5</td>
<td>4.3</td>
<td>4.4</td>
<td>4.7</td>
</tr>
</tbody>
</table>
5.3 Shear Strains

Shear strains were calculated from the acceleration time histories as explained in Section 4.2. Accelerometers were arranged in columns to calculate the shear strain along those arrays. The labeling of accelerometer arrays (Fig. 5.5) is described as:

- One array was placed on ESB box and labeled as V1.
- One array was placed at a distance of 180mm from the side of ESB and labeled as V2.
- One array was placed at a distance of 15mm from the culvert model and labeled as V3.
- One array was placed at a distance of 50mm from the culvert model and labeled as V4.
- One array was placed at the middle of ESB box, labeled as V5.

Shear strains in soil were calculated by using the 1st and 2nd order approximations and plotted against soil depth as shown in Fig. 5.6-5.11. Shear strains of soil were generally in the range of 0.1% to 1.2%. Li at al. (2013) stated that the resonant frequency of the model ground was close to the 3.5Hz at low strain levels. In the present study, strain level has a tendency to increase when the input motion has a frequency of 2 Hz. At large strains, shear modulus decreases and accordingly resonant frequency may approach to 2Hz. Generally, ESB box cannot successfully conform to soil movement for shear strains larger than 0.2%. Since, the model was placed at the center of container, the boundary effects were minimized.

![Fig. 5.5: Layout of accelerometer arrays](image-url)
Fig. 5.6: Shear strain calculated by 1\textsuperscript{st} order and 2\textsuperscript{nd} order approximation vs. depth
a) Test 3  b) Test 4
Fig. 5.7: Shear strain calculated by 1<sup>st</sup> order and 2<sup>nd</sup> order approximation vs. depth
(a) Test 5  b) Test 6
Fig. 5.8: Shear strain calculated by 1st order and 2nd order approximation vs. depth
  a) Test 7  b) Test 8
Fig. 5.9: Shear strain calculated by 1\textsuperscript{st} order and 2\textsuperscript{nd} order approximation vs. depth  
(a)Test 9  (b)Test 10
Fig. 5.10: Shear strain calculated by 1\textsuperscript{st} order and 2\textsuperscript{nd} order approximation vs. depth

a) Test 11
b) Test 12
Fig. 5.11: Shear strain calculated by 1st order and 2nd order approximation vs. depth
a) Test 13  b) Test 14
5.4 Sidewall Deformations

The deformations along the culvert models were measured primarily using horizontal and diagonal extensometers. The diagonal extensometers were placed in longitudinal direction of the tunnel as shown in Fig. 2.21, and they were used to examine the validity of plain strain conditions. It was observed that the diagonal deformations measured at different locations in longitudinal direction were quite consistent and thus proved the validity of plain strain conditions.

As it is seen in Fig. 2.21 there were 5 pairs of extensometers each of which measures the horizontal deformations at relevant level of both sidewalls. They were labeled from HE1 to HE10 as given in Fig. 5.12. The structural deformations observed during the tests are introduced below. Although cyclic deformations at the left and right sidewalls were not exactly equal to each other, records were reasonably consistent. Such slight differences might occur due to the super sensitivity of extensometers used in centrifuge testing. Horizontal deformation at top slab level was observed to be the highest where it was almost zero at the bottom. Nevertheless, the increase in horizontal deformation from bottom to top mostly starts to decrease after mid-height of the sidewalls. Since all the extensometers were calibrated under the same mechanical system, the reciprocal extensometers are expected to read opposite signals of almost the same absolute values of deformations. Horizontal deformation-time histories at the sidewalls of culvert model #1, #2 and #3 are shown in Fig. 5.12-5.23. In some tests, the data was missing due to noise in the data acquisition system or measurement errors. In order to comprehend the deformations recorded by different extensometers the deformation data are zoomed in as illustrated in Fig. 5.12-5.23. The deformations recorded by reciprocal extensometers during the tests are also shown in Fig. 5.24 – 5.35. As expected the reciprocal extensometers record opposite signals of almost the same absolute values of deformations. It is seen that there exists difference between readings of HE5 and HE10 which are the reciprocal extensometers located at the closest to the base. It is believed that reading differences of about 0.4 mm at most at the location closest to the base is noticeable due to the total amount of maximum deformation does not exceed 0.65 mm. Maximum racking deformations occurred at the sidewalls of the culvert models are presented in Fig. 5.36 – 5.38. As seen from the figures, there is almost a clear racking displacement profile as given in theoretical predictions.
Fig. 5.12: Sidewall horizontal deformation time histories for Test 3
(a) Deformation time history for left sidewall
(b) Closer view of deformation time history for left sidewall
(c) Deformation time history for right sidewall
(d) Closer view of deformation time history for right sidewall
Fig. 5.13: Sidewall horizontal deformation time histories for Test 4

(a) Deformation time history for left sidewall
(b) Closer view of deformation time history for left sidewall
(c) Deformation time history for right sidewall
(d) Closer view of deformation time history for right sidewall
Fig. 5.14: Sidewall horizontal deformation time histories for Test 5 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.15: Sidewall horizontal deformation time histories for Test 6
(a) Deformation time history for left sidewall
(b) Closer view of deformation time history for left sidewall
(c) Deformation time history for right sidewall
(d) Closer view of deformation time history for right sidewall
Fig. 5.16: Sidewall horizontal deformation time histories for Test 7

(a) Deformation time history for left sidewall
(b) Closer view of deformation time history for left sidewall
(c) Deformation time history for right sidewall
(d) Closer view of deformation time history for right sidewall
Fig. 5.17: Sidewall horizontal deformation time histories for Test 8    a) Deformation time history for left sidewall  
b) Closer view of deformation time history for left sidewall  
c) Deformation time history for right sidewall  
d) Closer view of deformation time history for right sidewall
Fig. 5.18: Sidewall horizontal deformation time histories for Test 9 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.19: Sidewall horizontal deformation time histories for Test 10 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.20: Sidewall horizontal deformation time histories for Test 11 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.21: Sidewall horizontal deformation time histories for Test 12 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.22: Sidewall horizontal deformation time histories for Test 13
(a) Deformation time history for left sidewall
(b) Closer view of deformation time history for left sidewall
(c) Deformation time history for right sidewall
(d) Closer view of deformation time history for right sidewall
Fig. 5.23: Sidewall horizontal deformation time histories for Test 14 a) Deformation time history for left sidewall b) Closer view of deformation time history for left sidewall c) Deformation time history for right sidewall d) Closer view of deformation time history for right sidewall
Fig. 5.24: Horizontal deformation recorded by reciprocal extensometers during Test 3
Fig. 5.25: Horizontal deformation recorded by reciprocal extensometers during Test 4
Fig. 5.26: Horizontal deformation recorded by reciprocal extensometers during Test 5
Fig. 5.27: Horizontal deformation recorded by reciprocal extensometers during Test 6
Fig. 5.28: Horizontal deformation recorded by reciprocal extensometers during Test 7
Fig. 5.29: Horizontal deformation recorded by reciprocal extensometers during Test 8
Fig. 5.30: Horizontal deformation recorded by reciprocal extensometers during Test 9
Fig. 5.31: Horizontal deformation recorded by reciprocal extensometers during Test 10
Fig. 5.32: Horizontal deformation recorded by reciprocal extensometers during Test 11
Fig. 5.33: Horizontal deformation recorded by reciprocal extensometers during Test 12
Fig. 5.34: Horizontal deformation recorded by reciprocal extensometers during Test 13

Fig. 5.35: Horizontal deformation recorded by reciprocal extensometers during Test 14
Fig. 5.36: Maximum horizontal deformations recorded on sidewalls
a) Test 3 b) Test 4 c) Test 5 d) Test 6
Fig. 5.37: Maximum horizontal deformations recorded on sidewalls
a) Test 7  b) Test 8  c) Test 9  d) Test 10
Fig. 5.38: Maximum horizontal deformations recorded on sidewalls
a) Test 11  b) Test 12  c) Test 13  d) Test 14
5.5 Racking Deformations

Wang (1993) defined the racking deformation of a rectangular underground structure, R, as follows:

\[ R = \frac{\Delta_{str}}{\Delta_{ff}} \]  \hspace{1cm} (5.1)

where, \( \Delta_{str} \) is the racking deformation of underground structure, \( \Delta_{ff} \) is the free-field deformation structure. Penzien (2000) proposed the following relationship to estimate the racking ratio:

\[ R = \frac{4(1-\nu_s)}{1+\alpha_s} \]  \hspace{1cm} (5.2)

\[ \alpha_s = (3-4\nu_s) \frac{k_{il}}{k_{si}} \]  \hspace{1cm} (5.3)

where, \( \nu_s \) is the Poisson’s ratio of the soil, \( k_{si} \) and \( k_{l} \) are soil and lining stiffness coefficients, respectively. Racking ratios were calculated by using Penzien (2000) approach and compared with the results obtained from extensometer measurements and free field strains (Table 5.2). As seen from Table 5.2, Penzien’s approach overestimates racking deformations of model #1. On the other hand, the approach underestimates racking deformations of model #2 and #3. As the rigidity of structure increases, Penzien’s solution is more likely to underestimate the racking deformations. There are two possible explanations why the approach has a tendency to underestimate the racking ratios for relatively rigid structures:

- Inertia of the structure is neglected,
- Normal stresses acting on the underground structure is neglected,

when assessing the racking deformation of underground structures.

Racking ratios were also calculated by using the recorded acceleration time histories on the culvert model. First, strain of the culvert sidewalls was computed from the displacement of top and bottom slab of structure model and then, obtained results divided with the free-field strains obtained at the mid-depth of culvert model (Eq 5.1). The results are presented in Table 5.3. As
seen from Table 5.3, racking ratios obtained from accelerometers are higher than the extensometers. The main reason for that result may be the existence of rocking motion or spurious accelerations generated by the vibration of system in X and Z directions.

Table 5.2: Racking ratios obtained from centrifuge tests and Penzien (2000) solution

<table>
<thead>
<tr>
<th>Culvert Model #</th>
<th>Test #</th>
<th>Racking ratio (Centrifuge tests)</th>
<th>Racking ratio (Penzien, 2000)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3</td>
<td>0.64</td>
<td>1.04</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
<td>1.16</td>
<td>1.28</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>0.50</td>
<td>0.57</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
<td>0.72</td>
<td>0.93</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>0.28</td>
<td>0.21</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>0.51</td>
<td>0.31</td>
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<td>9</td>
<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>0.28</td>
<td>0.18</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>0.16</td>
<td>0.08</td>
</tr>
<tr>
<td>3</td>
<td>12</td>
<td>0.21</td>
<td>0.10</td>
</tr>
<tr>
<td>3</td>
<td>13</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>3</td>
<td>14</td>
<td>0.12</td>
<td>0.06</td>
</tr>
</tbody>
</table>

Table 5.3. Comparison of racking ratios obtained from extensometers with racking ratios obtained from accelerometers

<table>
<thead>
<tr>
<th>Culvert Model #</th>
<th>Test #</th>
<th>Racking ratio obtained from extensometers</th>
<th>Racking Ratio obtained from accelerometers</th>
</tr>
</thead>
<tbody>
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<td>1</td>
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</tr>
<tr>
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<td>4</td>
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</tr>
<tr>
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<td>6</td>
<td>0.72</td>
<td>1.07</td>
</tr>
<tr>
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</tr>
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<td>0.40</td>
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</tr>
<tr>
<td>3</td>
<td>14</td>
<td>0.12</td>
<td>0.38</td>
</tr>
</tbody>
</table>
5.6 Lateral Earth Pressures acting on Sidewalls

Wang (1993) recommended a simplified pressure distribution so as to assess the seismic design of shallow rectangular underground structures. Fig. 5.39 shows dynamic the pseudo-triangular pressure distribution which is the simplified form of shear and normal stresses acting on the buried underground structure. Based on this assumption, a dynamic lateral soil pressure coefficient, \(k_{sd}\), is defined as follows:

\[
k_{sd} = \frac{P_{sd}}{\sigma_{v,m}}
\]  

(5.4)

where, \(P_{sd}\) is the maximum pressure value of the triangular pressure distribution and \(\sigma_{v,m}\) is the vertical stress at the mid-depth of underground structure.

Wang (1993) defined relative stiffness between the underground structure and soil by the flexibility ratio, \(F\), as follows:

\[
F = \frac{G_m x W}{S x H}
\]  

(5.5)

where, \(G_m\) is the degraded shear modulus, \(S\) is the force required for unit racking deformation, \(W\) and \(H\) is the width and height of the underground structure, respectively. Since \(G_m\) is very sensitive to shear strain, an initial flexibility ratio, \(IF\), is defined by replacing \(G_m\) with \(G_{max}\) in Eq. (5.5). Thus, \(IF\) is given by:

\[
IF = \frac{G_{max} x W}{S x H}
\]  

(5.5)

Sidewall deformations obtained from centrifuge tests were used to calculate \(P_{sd}\) values by applying simplified frame analysis proposed by Wang (1993). Values of \(k_{sd}\) were computed from Eq. 5.4 and plotted against \(IF\) (Fig. 5.40). As seen in Fig. 5.40, \(k_{sd}\) values vary between 0.4 and 2. Values of \(k_{sd}\) increases with the increase of the rigidity of structure as expected. The figure
may help engineers to make reasonable estimates in the preliminary design of rectangular underground structures.

![Diagram of dynamic earth pressures acting on the underground structure with theoretical and simplified distributions.](image)

Fig. 5.39: Dynamic earth pressures acting on the underground structure a) Theoretical distribution of shear and normal stress b) Simplified distribution proposed by Wang (1993)

![Graph showing variation of dynamic lateral coefficient, $k_{sd}$, with respect to IFR.](image)

Fig. 5.40: Variation of dynamic lateral coefficient, $k_{sd}$, with respect to IFR
6 Summary & Conclusions

The study was carried out through within the aim of enhancing the experimental data regarding seismic behavior of underground structures. Box shaped buried culvert, one of the most built underground structures, was explored in the study. As being a demanding problem seismic response of embedded culverts was examined by performing centrifuge tests. There existed 14 tests conducted under a centrifugal field of 40g. 2 out of 14 tests were performed to examine free field response. Three culvert models, each has cross-section with a particular thickness, were imposed to harmonic excitation in rest of the tests. The main results of the experimental study were presented in detail.

The main conclusions of the study can be summarized as follows:

- Maximum horizontal acceleration at the base of model amplified with an increasing rate from bottom to the surface of model ground. Besides, the amplification factor decreases with the increase of acceleration amplitude.
- There was a slight difference between the recorded motion at the shaking table and base of the model ground. This difference indicated that there might be small amount of sliding at the soil-ESB container interface.
- During the dynamic tests, ESB box could not successfully move in accordance with the soil at relative high strains larger than 2%. Since, boundaries were far from the area of interest, undesirable boundary effects were minimized.
- Measured sidewall deformations in centrifuge tests were consistent with the theoretical racking displacement profile. There is a 180° phase shift between the reciprocal extensometer recordings having almost the same amplitude.
- Racking ratio values obtained from accelerometer recordings on the culvert model were higher than those obtained from extensometer measurement. The most likely reasons for
this result are the rocking motion of culvert and spurious accelerations arising from mechanical system.

- Racking ratios estimated from the Penzien (2000) approach were compared with those obtained in dynamic centrifuge tests. Penzien approach overestimated racking ratios for the most flexible model #1 whereas it underestimated racking ratios for relatively rigid structures; models #2 and model #3. The approach ignores the normal stresses exerted on the underground structure and structural inertia. This may lead to the underestimation of racking deformation for relatively rigid underground structures.

- A dynamic lateral soil pressure was defined based on the pseudo triangular pressure distribution acting on the simplified frame recommended by Wang (1993). Variation of this coefficient was given with respect to relative stiffness between soil and underground structure. This coefficient might be used by design engineers to make preliminary estimates for the underground culverts buried in dry sand.
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