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Static and Seismic Soil Culvert Interaction

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A thesis submitted in partial fulfillment of the requirements for the degree in Doctor of Philosophy

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STATIC AND SEISMIC SOIL CULVERT INTERACTION
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by

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Graduate Program
in
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A thesis submitted in partial fulfillment of the requirements for the degree of
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ABSTRACT

Failures of box culverts under static and earthquake loads can cause significant economic loss. Therefore, it is important to investigate the soil-culvert interaction of box culverts to understand their responses to such loads. The response of buried box culverts is a complex soil-structure interaction problem, where the relative stiffness between the soil and the structure is a critical factor. Soil arching is an important aspect of the soil-culvert interaction problem, and results in the redistribution of free-field stresses due to the presence of buried structures and leads to an increase or decrease in the loading around box culverts.

A series of static and seismic scaled physical model centrifuge tests were performed to investigate the soil culvert interaction. Two different box culvert thicknesses and two Nevada sand relative densities were used to explore the interaction between the sand and box culverts under a wide range of different conditions. The static loading consisted of the soil self-weight of and the surcharge from a surface foundation, while the seismic loading considered the application of seven earthquake shaking events for each test. Several sensors were used in these tests, including tactile pressure sensors, LVDTs, accelerometers and strain gauges. A newly developed method for installing the strain gauges inside the box culvert model is introduced. The responses of the box culvert have been compared for all of the loading conditions.

It was observed that the kinematic soil culvert interaction due to the presence of a box culvert, as well as the surface foundation, had a significant effect on reducing the peak ground acceleration at the surface when compared to the free-field peak ground acceleration. The kinematic interaction can provide up to a 50% reduction and is dependent on the amplitude of the input motion at the base of the model. Small values for the rocking of the box culvert and surface foundation were also observed, and their values changed with the amplitude of the input motion. The values observed for the foundation were higher than those for the culvert, due to the soil confinement. The lateral movement of the foundations increased as the peak ground acceleration at the base of the model increased. The racking deformation ratio of the culvert was found to change with the thickness and therefore the relative stiffness of the culvert and the soil density.

Soil pressures measured by different methods were in good agreement and those obtained from the tactile sensors can be considered to bound the expected behaviours. The soil pressure
observed on the culvert top slab had a parabolic shape, i.e., higher values at the edges and lower at the center than the theoretical vertical soil pressure. On the side wall, the horizontal soil pressure increased with depth. The soil-culvert interaction factors decreased at the center and increased at the edges of the top slab, as the thickness and the relative stiffness of the culvert decreased. The seismic analysis showed that the seismic bending moment increased as the peak ground acceleration at the model base and the relative stiffness of the culvert increased.

The static and seismic responses of the box culvert were analyzed using the finite difference code FLAC 2D and the results matched the experimental responses. The validated numerical model was then used to perform a parametric study, to evaluate the effects of: culvert geometric parameters, foundation locations and soil properties for the static loading and only the culvert geometric parameters for the seismic loading. The results have been evaluated for bending moment, soil pressure and soil culvert interaction factors. Based on these analyses, charts and equations are presented to help in assessing the design values of the static soil pressure, static bending moment, and the seismic bending moments around box culverts.

Keywords
Soil arching, Soil Culvert Interaction (SCI), box culvert, foundation, Nevada sand, centrifuge modeling, finite difference modeling, kinematic interaction, rocking of structures, racking of culverts, bending moment, soil pressure, soil-culvert interaction factors, geometry effect, foundation effect, soil properties.
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CHAPTER ONE
INTRODUCTION

1.1 INTRODUCTION

Culverts are an important life lines in society, and are considered a critical part of transportation infrastructure all over the world. They serve a significant role in modern infrastructure conveying water, sewerage, and sometimes pedestrians beneath roads, railways, span highways and other obstacles. They are also used to control water flow, storm runoff, divert municipal services, allow vehicular access and for other related activities. Culverts are largely unknown and unseen by the public, but are vital in managing waste, providing irrigation, and regular water levels. Often, culverts are buried beneath a substantial depth of overburden soil.

To design culverts for engineering purposes, most manufacturers use design methods based on formulas that assume simplified behaviors and often rely on considerable empiricism. Culverts can be built using different types of material, such as steel, aluminum, and concrete. They can take various geometric shapes like pipes, arches, ellipses, and boxes. Culverts of all shapes, sizes and materials are installed in new developments. Most drainage systems utilize concrete as the construction material, because of its inherent strength and durability.

Box culverts are constructed from short sections of reinforced concrete, which are joined together to form the final desired cross-section. The geometry of these structures is usually square or rectangular in cross-section and they can have single or multi-celled openings. The shape of a box culvert is designed to support loads above and around it, but they may be required to resist loads that are considerably higher, from other structures and construction activities.

Construction of reinforced concrete box culverts can be either cast-in-place or precast concrete units. Nowadays, 80% of the single culvert installations are precast. They are considered to be efficient, since they reduce project execution time, and they are ideal when the concrete batch plant is not near the construction site.

Box culverts, which are the subject of this research, are subject to different types of loads that may be subdivided into two categories: static or seismic loads. Buried box culvert problems
are a complex example of soil-structure interaction, where the relative stiffness between the backfill soil and the culvert materials is a critical factor in the load carrying capacity of culverts. The soil-structure interaction of buried box culverts is difficult to solve theoretically. No closed form solutions have been developed that adequately approximate the actual behavior. The loads attracted to buried structures, from both overburden and surcharge loads, are governed by the characteristics of soil and the geometry and stiffness of the structural components. Incompatibility in stiffness between the structure and the surrounding soil can lead to the redistribution of free-field stresses around the box culvert. This will result in a decrease in loading over deflecting or yielding areas of the structure and an increase over adjoining rigid or stationary parts. This transfer of load due to soil-structure interaction is known as “Soil Arching”. Soil arching around box culverts can lead to either increased or decreased loading on the structure. Detailed studies show the effect of soil-structure interaction mechanisms around box culverts under the effect of static and seismic loading are generally absent in the existing literature. Present design codes also include the effect of this interaction in a limited capacity, ignoring a number of important aspects.

Whilst there have been experimental and field studies to investigate stress distribution and arching (e.g. Lefebvre et al., 1976), the exact conditions required for this phenomenon to occur are still unclear and arching is often ignored in engineering design due to a lack of experience, and inclusion in codes of practice is rare. A range of problems such as underground conduits, tunnels, trapdoors, retaining walls and braced cuts can all experience significant arching action and theoretical analyses have been published on these subjects. These approaches have considered soil arching from both elastic and plastic soil states. Iglesia et al. (1999) presented a model for estimation of loads on underground structures based on the ground reaction curve (Einstein & Schwartz, 1979). Karinski et al. (2003) tried to develop an analytical model to evaluate the static soil pressure on the buried structures and they chose the box section as an example to find the pressure on the top and bottom slabs. They also tried to evaluate the positive and negative arching. More recently, arching around buried structures has been studied in centrifuge model tests (e.g. Iglesia et al., 1999; Stone & Newson, 2002).

In recent years, research has concentrated on the behaviour of flexible circular culverts and large diameter pipelines (e.g. Valsangkar & Britto, 1979). However, knowledge of arching around rectangular and square culverts is currently limited. Numerical study of negative and
positive arching around deeply buried rigid box culverts was conducted by Kim and Yoo (2005), with emphasis on the imperfect trench method of construction. The current edition of the AASHTO (2002) standard specifications for highway bridges takes some account of arching by changing vertical stresses over box culverts based on Marston-Spangler theory. However, this approach is quite conservative compared to more sophisticated numerical techniques, such as finite difference and finite element analysis.

For the case of relatively flexible buried structures, the soil-structure interaction is even more complicated and the problem is difficult to solve theoretically or analytically. However, it is relatively simple to examine the problem experimentally and in particular with a geotechnical centrifuge. Centrifuge facilities can offer the chance of building more realistic scaled physical models to represent the field situation and to investigate them under different conditions.

Failure of box culverts can cause large economic penalties, especially if the emergency replacement and the user daily costs which can be millions of dollars taking into consideration. The soil structure interaction can play an important role in defining the actual static and seismic loads that the culvert will be subjected to and that will help the engineer to design a more safe and economic structure.

1.2 RESEARCH OBJECTIVES

The main aim of this research is to investigate the Soil-Culvert-Interaction (SCI) mechanisms of buried reinforced concrete box culverts and their influence on the loads attracted to the structure. This investigation will lead to a better understanding of the behavior of the interaction between the soil and box culverts and its effect on the soil pressures attracted to them and the bending moments in their structural members. This investigation will be partly achieved by exploring the behavior and performance of model box culverts with different thicknesses supported by dry cohesionless soil under static and seismic loading in a series of centrifuge tests. To achieve this goal, the specific objectives of this study are:

1. To study the arching effect of soil around scaled model box culverts under static and seismic loading taking into consideration the effect of soil density, surface foundations and culvert wall thickness.
2. To study the seismic response of scaled model box culverts under the effect of different earthquake loadings with different amplitudes and frequencies.

3. To compare between the effect of static and seismic loading on scaled model box culverts.

4. To develop a numerical model at the prototype scale, to be calibrated and validated by the experimental results, to describe the effect of soil arching under static and seismic loadings in more detail by performing parametric studies.

5. To develop static and seismic design equations, procedures, guidelines and recommendations for consideration of soil pressures on box culverts and resulting bending moments.

1.3 ORGANIZATION OF THE THESIS

This research is divided into seven main chapters that include details of the work performed. The contents of each chapter are as follows:

Chapter 1: provides a description of the problem, objectives of the research and the layout of the thesis.

Chapter 2: includes a comprehensive literature review of previous research work related to box culverts and current design approaches. This chapter begins by describing the phenomenon of arching around box culverts, and then it is divided into two parts; one for static loading and the other for the seismic loading. The static part includes the previous field instrumented box culvert tests, the equations derived for the soil structure interaction, and previous numerical and centrifuge tests. Descriptions of methods from codes and standards such as AASHTO and CHBDC for box culverts are also presented. In the seismic portion, a review of the performance of box culverts under the effect of previous earthquakes was reported, and then a description of previous shaking table tests on box culverts is presented. The concept of “racking” of rectangular buried structures is introduced since it is the only available closed-form solution that can be applied for pseudostatic analysis. Nothing applicable to seismic loading on box culverts is reported in any standards and codes, but some remarks and recommendations are also presented.

Chapter 3: describes the centrifuge modelling program that was conducted for the experimental part of this research. In this chapter, the properties of Nevada sand that was used in the tests is presented, and the design of the box culvert and foundation models are explained. All
of the sensors and the instruments used in the centrifuge facility are explained in terms of their usage, calibration, and collection of the data. A newly developed technique for installing strain gauges inside box culvert tubes is also described in detail. A description of the one dimensional shaker that was used to excite the model using three different earthquakes (with different amplitudes and frequencies) is also presented.

Chapter 4: reports the results of the centrifuge tests. Results from varies sensors that were connected to the box culvert and foundation, or placed on the sand surface or imbedded inside it are presented. Analysis of bending moment and soil pressure attracted to the culvert under static and seismic loads are also presented for different cases of relative densities and culvert thicknesses. Ground motion parameters, as well as dynamic soil properties of sand produced using the centrifuge results are analysed and presented, and compared to some well published data. Analysis of soil-structure factors and kinematic soil-structure interaction based on centrifuge test results are also discussed in this chapter. The effects of ground shaking on the culvert in terms of racking and the rocking of culvert and the surface foundation are also investigated.

Chapter 5: presents the numerical analysis of the experimental results of the centrifuge tests, using the finite difference method implemented in the FLAC 2D software. Details of the static and seismic models that include the mesh generation, boundary conditions, sand and structure properties, loads and earthquakes used are presented. Results of the numerical model were verified and calibrated, and compared to the results of the centrifuge tests. The effect of g level and frequency of earthquakes was also discussed in terms of its effect on the bending moments. The predicted and measured seismic bending moments are compared to the recommendations in CHBDC, (2006).

Chapter 6: includes a comprehensive parametric study for static and seismic numerical models. In this parametric study, different parameters related to the soil properties, culvert thicknesses, depth of the soil cover, and foundation location on the surface, are investigated and described in this chapter.

Finally, Chapter 7: discusses the results that have been obtained from this research and presents conclusions from the findings of the experimental and numerical studies, and provides design guidance with recommendations for future research in the area.
CHAPTER TWO
LITERATURE REVIEW

2.1 INTRODUCTION

Box culverts are a critically important life line for society. They form an important part of the transportation infrastructure and their failure can cause significant economic losses. The primary usages of box culverts are for carrying roadways and railways over water courses, to allow for the storm runoff and sewerage to flow without affecting the above infrastructure. It can also be used to carry electrical and telephone lines or to allow vehicles and pedestrian to access. The geometry of the culverts can take different shapes, but research in previous years has focused on the behaviour of flexible circular and arch culverts. However, the behaviour of box culverts, which can be square or rectangular, has not received much attention to date.

Box culverts can be classified based on the type of material or based on the installation type. Based on the material used, the culvert can be either rigid or flexible; rigid culverts can be made of reinforced concrete, which can be precast or cast-in-place, while flexible culverts can be made of steel, cast iron, aluminum, or plastic or other materials. Based on the installation method, the culvert can be constructed under embankments, trenches or induced/imperfect trenches. Box culverts can be installed as single cell, or multiple cells depending on the use.

Depending on the installation condition, soil culvert interaction behaviour can be examined to show the effect of soil arching on the pressures attracted to the culvert. This arching effect can cause increases or decreases in the soil pressure on the culvert due to stress redistribution. Arching is usually ignored in engineering design due to lack of knowledge, which may lead to damage or collapse in the culvert.

This chapter presents up to date research in the literature for soil culvert interaction under static and seismic loading. The chapter is divided into two main parts: static and seismic. The first part covers studies on static soil culvert interaction, including definitions and explanations for arching effects around box culverts and field studies of measurements of soil pressure on box culverts, as well as numerical and centrifuge modelling studies conducted are summarised.
Lastly, it presents some provisions of codes and standards such as AASHTO and CHBDC. The second part is focused on previous studies of seismic soil culvert interaction. It reviews the performance of box culverts during past earthquakes, and in limited tests using shaking tables. This is followed by describing the concept of culvert racking, which is widely used in the literature. Although there is no clear seismic treatment in the standards and codes, a few remarks related to this area are also presented.

2.2 PREVIOUS STUDIES RELATED TO STATIC LOADING:

2.2.1 Arching around Box Culverts

The redistribution of ‘Free Field’ stresses as the result of the presence of buried structure will result in a decrease in loading over the deflecting or yielding areas of the structure, and an increase over adjoining rigid or stationary parts. This transfer of load is termed arching (Terzaghi, 1943). Shear resistance tends to keep the yielding mass in its original position resulting in a change of the pressure for both the yielding surface and the adjoining area of soil. If the yielding part moves downward, the shear resistance will act upward and reduce the stress at the base of the yielding mass as shown in Figure 2.1. On contrary, if the yielding part moves upward, the shear resistance will act downward to cause increase in the stress at the support of the yielding part (Bjerrum et al., 1972).

The experimental study by Terzaghi (1943) is the most famous one for studying arching in sand using a yielding horizontal trap door for plane strain conditions. The vertical stress on the trap door up to the surface resulting from his experiment is shown in Figure 2.2. The sand cover \(H\) is more than four times the width of the trap door \(b\) with the distance \(h\) measured upward from the trap door. Reduction in the vertical stress \(\sigma'_v\) was observed starting from \(h/b = 2.5\) down to the trap door. At \(h = H\), the vertical stress (i.e. immediately above the trap door) is 10% of the vertical stress if there is no arching \(\sigma'_{vh}\).
Figure 2.1: Stress distribution in the soil above yielding base (After Bjerrum et al., 1972)

Figure 2.2: Terzaghi’s trapdoor experiments (note: z measured upwards from trapdoor). (After Bulson, 1985).
Janssen (1895) assumed that the vertical pressure on the yielding element of width $b$ and at depth $z$ as shown in Figure 2.3 is equal to the difference between the soil pressure due to the weight of the soil prism $ABCD$ above the element and the frictional resistance along the sides of the element. The shear resistance can be determined by:

$$\tau = c' + \sigma' \tan \phi'$$  \(2.1\)

where, $c'$ is the effective cohesion of the soil, $\phi'$ is the effective angle of internal friction of the soil and $\sigma'$ is the normal effective stress on the plane of shearing. The ratio between the horizontal and vertical stress is an empirical constant $K$. Further experiments by Terzaghi (1943) suggested that $K$ increased from unity immediately over the yielding surface (of width $b$) to 1.5 at an elevation of $b$ above the centerline of the surface. At elevations greater than $2.5b$, the movement of the trapdoor did not alter the state of stress.

![Figure 2.3: Janssen’s analysis (After Bulson, 1985).](image)

By resolving the vertical equilibrium on a unit length of the section, the following relation can be obtained:

$$b\gamma \ dz = b \left( \sigma'_v + d\sigma'_v \right) - b\sigma'_v + 2c'dz + 2K\sigma'_v dz \tan \phi'$$  \(2.2\)

where, $\gamma$ is the unit weight of the soil. With a uniform surcharge $q$ at the surface, this equation can be solved for $\sigma'_v$. 

The negative exponential indices in Equation 2.3 imply ‘active arching’, which occurs due to the downward movement of the trapdoor. In the case of ‘passive arching’, the door will move upwards and the indices will be positive (Bulson, 1985).

Arching can be either active or passive depending on the relative stiffness between the soil and the structure. Active arching occurs when the structure is more compressible than the surrounding soil as illustrated in Figure 2.4a. When the system subjected to loads, the resulting stress distribution on plane AA and BB is similar as shown in Figure 2.4b, where the stresses on the structure are less than the soil. If the structure deforms uniformly on plane AA and BB, the stresses on it tend to be lower at the edges and that is because of the mobilized shear stresses in the soil (Evans, 1984).

\[
\sigma'_v = b \left( \frac{\gamma - 2 c' / b}{2 K \tan \phi'} \right) \left[ 1 - \exp \left( -K \tan \phi' \frac{2z}{b} \right) \right] + q \exp \left( -K \tan \phi' \frac{2z}{b} \right)
\]  
(2.3)
Passive arching occurs when the soil is more compressible than the structure as shown in Figure 2.5a. In this case, large deformations and mobilized shear stresses will happen in the soil and causes the total pressure on the structure to increase while the pressure decreases in the adjacent soil as illustrated in Figure 2.5b. If the structure deforms uniformly, the stresses will be high at the edges and low at the center (Evans, 1984).

![Passive arching diagram](image)

(a) Displacements under pressure $P_s$ when structure is less compressible than surrounding soil

(b) Stress distribution across Plane AA or BB

Figure 2.5: Passive arching (After Evans, 1984)

Buried structures usually do not have uniform deformations and that cause a more complex stress distributions than active and passive stresses shown above. As an example, a structure that has more flexible spans towards the center can result in a deformation pattern as presented in Figure 2.6. The horizontal and vertical stress distributions suggest that the faces of the structure experience active and passive arching at the same time.
Allen and Russ (1978) analyzed loads on box culverts under high embankments and presented the active and passive arching in the diagrams illustrated in Figure 2.7. The culvert dimensions were 1.90 m × 1.90 m and the height of fill was 21.3 m. The height of equal settlement $H_e$ was equal to $2.5B_C$. In the active case, the failure lines were drawn tangent to the line of $(45^\circ + \phi' / 2)$ from the horizontal line, located at the top of the culvert, up to the plane of the equal settlement and vertically thereafter. For a high embankment they determined that the case was a passive arching condition because the failure lines were tangent to the line of $(45^\circ - \phi' / 2)$ from the horizontal to the elevation of the plane of equal settlement and vertical thereafter.

Allen and Russ (1978) show the result of a photoelastic model that was constructed to simulate the positive projection box culvert under high fill using gelatin material around it as shown in Figure 2.8. The lines indicate that the arching occurs in such elastic material, and the pressure above the culvert is greater than the weight of the material above it. The darkest areas in the photograph are areas of zero strain, and from this observation, it is apparent that two large strain bulbs formed at the corners of the culvert and reached the surface.
Figure 2.7: Active and Passive arching conditions over a box culvert (after Allen and Russ, 1978)

ASSUMPTIONS:
\[ \phi = 30^\circ \]
\[ h_e = (2 \frac{1}{2}) B_c \]
ACTIVE ARCHING CONDITION (IMPERFECT TRENCH)

Figure 2.8: Photoelastic modeling of arching and stress bulb above the box culvert (after Allen and Russ, 1978)
To examine the mechanism of arching, Stone and Newson (2002) performed centrifuge tests on the rectangular culvert shown in Figure 2.9. Two tests were performed under a soil cover of 200 mm; Test A: stiff sides and flexible top and base, and Test B: flexible sides and stiff top and base. The results were collected at three g levels: 5g, 10g, and 20g.

For Test A, the results of the loads are consistent with active arching due to the inward deflection of the top slab. The loads attracted to the top slab were about 45% of the theoretical soil load. Since the ratio of the measured to prism loads were almost constant over all the g levels, it can be deduced that the same soil volume is involved in the loading of the top slab and the attracted loads to the top slab is independent of its deflection. For Test B, the load ratio increased with g level. At the 5g level, the load ratio was the same as in Test A, but after that started increasing until it exceeded 100% at 20g. As shown in Figure 2.9, it is assumed that if sufficient inward movement of the side wall occurs, then active Rankine zones will form right beside them. As a result of that, an active soil wedge (a-b) extending toward the free surface will develop. This wedge will extend to (a-b-c), which results in increasing the load attracted to the culvert. This indicates that the volume of soil involved in the loading on the top slab increases as the deflection of the side wall increases and explains the reason behind the increase in the load ratio as the g level increases.

![Figure 2.9: Schematic diagram illustrating the arching mechanism for Test A and Test B (after Stone and Newson, 2002)](image-url)
Newson et al. (2006b, 2007) revisited the tests shown above, but with a 100 mm soil cover. Similar conclusions were observed for Test A, and the expected positive arching occurred above the culvert. The culvert top slab attracted fewer loads than expected based on the calculation of the overburden due to the rectangular prism of soil above. On the top slab of the culvert, the shaded area on the left-hand side shows the volume of soil applying loads to the top slab in the absence of arching, while the right-hand side shows the reduced volume of soil involved in loading when arching occurs. This occurs due to the transfer of load to the stationary mass of soil as shear stresses are mobilized on the vertical shear plane.

For Test B, the assumed soil arching mechanism was as presented in Figure 2.10. Above the top slab of the culvert, the self weight of the prism (iklm) is partially transferred to the next prism (fmih) and to the corners of the culvert causing positive arching over the top slab as it deflects under the loading. When the sidewall deflects laterally, the soil in (acfd) will move in the same deflection direction but the loads are supported partially by the shear stresses on the surfaces (ac) and (df). This will increase the loads experienced by the upper parts of the culvert sidewalls.

Figure 2.10: Assumed soil stress and movements around culvert (after Newson et al., 2007)
2.2.2 Investigation of Soil Structure Interaction for Box Culverts

2.2.2.1 Box culvert installation methods

Different circular culvert installation techniques were described by Marston and Anderson (1913), and then presented in a more detailed way by Clarke (1967), and later by Spangler (1950). The same installation methods can be applied to box culverts as well. Marston and Spangler showed that the main factor influencing the loads associated with the installation technique is the magnitude and direction of the settlement of soil prism over the culvert (interior prism) relative to the settlement of the immediately adjacent soil prisms (exterior prisms). Friction forces or shear stresses can be generated due to the relative settlement between the interior and exterior soil prisms; these can be added to or subtracted from the soil pressure in the central prism and applied to the culvert as shown in Figure 2.11. Three main installation techniques that can be applied to box culvert are briefly described as follows:

1. **Embankment installation (Positive projecting condition):** In this case, the settlement of the soil prisms adjacent to the culvert is greater than the settlement of the soil prism right above the culvert. This will cause the layers of soil in the central soil prism to deform and take an arch shape. Therefore, the pressure of the soil layers above the culvert will increase, which referred to as **negative arching** or **passive arching** as shown in Figure 2.11a.

2. **Trench installation (Negative projecting condition):** In this condition, the settlement of the soil prisms adjacent to the culvert is less than the settlement of the central soil prism above the culvert. This will lead the soil layers in the central prism to deform in a reverse arch shape as shown in Figure 2.11b. As a result of that, the soil pressure above the culvert will be reduced by the amount of the acting up shear stress exerted on the central soil prism, which means that part of the weight of the central soil prism will be transferred to the exterior prisms. This condition referred to as **positive arching** or **active arching**.

3. **Imperfect/Induced trench installation (ITI):** The purpose of this installation method is to reduce the soil pressure from high embankments on the culvert. In this method, a large relative vertical displacement of soil prism above the culvert is induced by replacing the above soil with light weight material such as straw, leaves, compressive soil, or expanded polystyrene (EPS) as shown in Figure 2.12. It was expected that this material will cause a reverse arch in the soil prism above the soft zone as the case in the trench installation method, and hence a reduction in the soil pressure above the culvert. More details about this

Figure 2.11: Pressure transfer within soil structure system: (a) embankment installation, (b) trench installation (after Kim and Yoo, 2005)

Figure 2.12: Imperfect trench installation (after Kim and Yoo, 2005)
2.2.2.2 Previously instrumented box culverts

Many field tests were performed on circular culverts under deep and shallow fills, but their results are not applicable to box culverts because the soil structure interaction is different for box culverts. Circular culverts experience more significant soil arching effect, and the soil pressures on the sides tends to support them. Thus, the results of circular culverts are of limited importance to study the reinforced concrete box culverts (James et al., 1986). Even though box culverts are used in many places around the world, few of them have been tested to find the actual soil pressures they are subjected to in the field.

Binger (1947) reported pressure measurements at the center of a box culvert that has the dimensions of 2.74 m wide by 3.30 m high and was built under a 15.2 m of sandstone fill. The foundation soil under the culvert consisted of weathered rock and compacted residual clay. Two different pressure cell types (friction and stress-meter) were used to measure the soil pressure on the culvert. The cells were covered with 0.6 m of red brown clay layers by hand, and then the sandstone fill was compacted. The ratio of the fill height to the culvert width (H/Bc) is 5.6 and the pressure measured was 1.8 times the overburden pressure (i.e. soil structure interaction (SSI) factor).

Spangler (1950) documented soil pressure measurement tests of nine box culverts. The box culvert nominal sizes range between 0.61 m × 0.61 m to 2.44 m × 2.44 m. The pressure was measured using friction ribbons and the results show the average pressure on the top slab of the culvert. Ratios of soil fill height to culvert width (H/Bc) ranges from 1.8 to 6.18. The soil structure interaction (SSI) factors for all of these culverts range from 1.0 to 1.62.

Girdler (1974) presented the pressure measurement for three box culverts. The first culvert was at Station (89+20), where the culvert dimension was 2.44 m × 2.44 m and the fill height was 14.6 m. The second culvert was at Station (203+20) and its dimensions were 1.5 m × 1.5 m under a fill height of 22 m. The third culvert was at Station (210+50) and has dimensions of 1.5 m × 1.8 m and the fill height for this culvert was 29.3 m. All the three culverts were built using the imperfect trench method. The soil fill was sand and the material used for the imperfect trench was loose straw. The pressure cells were distributed diagonally on top slab and side walls to reduce the possibility of creating weakened planes as shown in Figure 2.13. The ratio of the soil fill height to the culvert width (H/B) for the three culverts was 6.1, 14.7, and 16.3 respectively. The material used for imperfect trench proved to be efficient in reducing the
pressure on the culvert. Most of the SSI factors for the different locations measured were less than 1 by considerable amount.

Russ (1975) continued the work started by Girdler (1974) but this time, he used two box culverts without imperfect trenching. The first box culvert was at Station (123+95) and it has the dimensions of 1.22 m × 1.22 m and under a fill height of 23.5 m. It was designed using rigid frame analysis, which means that it has a moment resisting joints; this will not allow joint translation. The second box culvert was at Station (268+30), this culvert has a rectangular dimension as 1.22 m × 1.52 m and the height of the soil fill was 11.4 m. This culvert was designed as a continuous beam structure without moment constraint. The soil fill used for both culverts was silty clay. The only difference between the two culverts is that the first one was built on yielding foundation and the second culvert built on an unyielding foundation, which was bedrock. The ratio of the soil fill to culvert width (H/B) for the two culverts was 19.3 and 7.5 respectively. Carlson pressure cells were used to measure the pressure on top and bottom slabs as well as the side wall as shown in Figure 2.14. The SSI factors for the yielding foundation culvert at the top is in the range of 1.5 which is less than the unyielding foundation case where the SSI factor is in the range of 1.7.
Figure 2.14: Pressure cell locations for the two culverts (after Russ, 1975)
Sato and Iwasaki (1981) reported the use of a 1 m thick layer of expanded polystyrene (EPS) above a large rectangular concrete culvert under embankment of sand fill. The EPS was placed 1 m above the culvert. The height of the embankment is 14 m and the external dimensions of the culvert are 7.6 m high and 10.7 m width. The thickness of the top slab was 0.9 m. The unit weight of the EPS is 17 kN/m$^3$ and its yield strength was 80 – 90 kPa. Earth pressure measurements were taken with and without EPS. The results show that the soil structure interaction factors increase with height until it reaches 1, when using the EPS up to a fill height of 5 m and after that the pressure is nearly constant, while without EPS, the SSI factor is 1.2.

Katona and Vittes (1982) reported the measured pressure for a box culvert that had dimensions of 1.22 m × 1.22 m founded on a dense granular bed within bedrock. The embankment fill was of silt and placed to a height of 23 m above the top slab of the culvert. The ratio of fill height to culvert width ($H/B$) is 18.9. Eight pressure cells were installed around the buried box culvert. The SSI factors for the top and bottom slabs were 1.48 and 1.61, respectively, while the lateral pressures were 0.35 and 0.18 of the overburden pressures on the right and left side walls, respectively. The study indicated that shear traction on the side walls produced significant downward force and must be accounted for in increased pressure on the bottom slab.

James et al. (1986) investigated a heavily instrumented box culvert. The dimensions of this culvert were 2.4 m × 2.4 m with a uniform thickness of 0.23 m. The subsoil was a river sedimentary deposit consists of thin layers of gravel, medium sand and silts, inter-lain by a thick layer of fine clayey sand. The excavated soil used was as a backfill material over the top slab of the culvert to a height of 2.4 m. This shows that the ratio of the fill height to culvert width is equal to unity. Twenty pressure cells of three kinds were installed on the culvert, four on each side and twelve on the top. The registered vertical pressures on the top of the culvert indicated that the SSI factor is in the range of 1.20.

Tadros (1986) presented the results of pressure measurements of 1.22 m × 1.22 m box culvert under a fill height of 5.8 m. The backfill material was a soil classified as CL-ML (silty clay), but in these tests a comparison was made between yielding and unyielding foundation material under the culvert. The ratio of fill height to culvert width ($H/B$) is 4.75. The culvert and its instrumentation under the soil fill are presented in Figure 2.15. The soil pressure measurement continued for more than 2000 days and a continuous record of pressure versus time was
obtained. The SSI factors obtained were 1.5 and 2.0 for yielding and unyielding foundation respectively.

Tadros et al. (1989a) instrumented a double box culvert; each cell had internal dimensions of 3.7 m × 3.7 m. The thickness of the top slab is 0.30 m and the bottom slab is 0.25 m, while the side wall thickness is 0.28 m and the center wall is 0.23 m. The total external dimensions of the culvert were 4.3 m × 8.1 m. The culvert was placed on a glacial till and the silty clay was used as a backfill with a height of 2.6 m. The ratio of the fill height to the culvert width ($H/B_c$) is 0.6. Twenty-eight vibrating-wire earth pressure cells, 14 on the top, 4 on each side and 6 in the bottom slab were used to measure the soil pressure as shown in Figure 2.16. Figure 2.16 shows pressure cell readings 80 days after completing backfill and compared to the soil load predicted by AASHTO (1987). AASHTO determines the vertical pressure by an equivalent fluid pressure (18.8 kN/m$^3$) without using soil structure interaction correction. The SSI factors on the top and bottom slabs show an increase at the edges and decrease at the center of the slab. The range of SSI factors is between 1.25 and 1.75.
Figure 2.15: Instrumentation of box culvert (after Tadros, 1986)
Figure 2.16: Pressure cell locations and distributions around box culvert (after Tadros et al., 1989)
Dasgupta and Sengupta (1991) performed a large scale model test on a reinforced concrete box culvert. The internal dimension of the culvert was 1.20 m ×1.20 m and had a uniform thickness of 75 mm for top and bottom slabs as well as side walls. Dry sand was used under the box culvert as soil foundation, and also used as the fill over the culvert. The side fill was compacted using wood tampers, and the height of the fill over the top slab of the culvert was 2.4 m. The ratio of the soil column height above the top slab to the culvert width (\(H/B_c\)) is 1.78. Twelve deflecting diaphragm pressure cells with a capacity of 200 kPa were placed at the central section of the culvert as shown in Figure 2.17. As shown in Figure, the pressure distribution on the top and bottom slabs takes the shape of a parabola. This shows that the stiffer edges attract more pressure than the center of the slab. The soil structure interaction factor increases at the edge by 1.90 and reduces to 0.6 at the center. If a parabola is fitted to the measured pressures, the average pressure on the top slab can be calculated as the area under the pressure diagram divided by the culvert width. The average pressure was 32% higher than the overburden pressure, and the SSI factor was 1.32.

![Figure 2.17: Pressure cell locations for the model box culvert (after Dasgupta and Sengupta, 1991)](image)
Vaslestad et al. (1993) presented the pressure measurements of three cells placed at different positions with respect to a cast in place reinforced concrete box culvert. The box culvert has a width of 2 m and a height of 2.55 m. The thickness of the culvert was 0.3 m. The culvert was under a 9.8 m of silty clay fill. The subsoil consisted of over-consolidated silty clay. The ratio of fill height to culvert width ($H/B_c$) was 4.9. To investigate the effect of imperfect trench, EPS with a thickness of 0.50 m and width of 2 m was placed over the culvert for a length of 20 m. This section was instrumented with two Glotzl type earth pressure cells (Cell 1 and 2) as shown in Figure 2.18. To compare the earth pressure of the imperfect trench with the conventional section, one earth pressure cell (Cell 3) was placed above the culvert. The measured pressure from cell 3 was 24% higher than the overburden pressure, which means a 1.24 soil structure interaction factor.

![Figure 2.18: Pressure cell locations for the model box culvert (after Vaslestad et al., 1993)](image)

Yang (2000) reported the results of pressure measurements on a double box culvert used as replacement to a failed one. Each cell had an internal dimension of 3.06 m high and 4.57 m
wide. The total width of the culvert was 9.9 m. The thickness of the top and bottom slabs was 0.32 m and the side walls 0.25 m. The overburden height was about 12 m above the culvert roof. The ratio of the fill height to culvert width ($H/Bc$) was 1.18. The culvert was supported by a 3 m of shaley clay. Limestone gravel was used around the culvert for a 0.6 m above the roof. Above the gravel, the backfill consisted of clayey weathered shale. In order to minimize the vertical and lateral stresses on the culvert, the backfill with 2 m of the culvert was not compacted. The recorded pressure was 26% higher than the overburden pressure which gives a soil structure interaction factor of 1.26. It is interesting to note in this culvert that even with the large layer of uncompacted fill, the SSI factor is still greater than 1.

Bennet et al. (2005) examined the pressure measurements on the top slab and side wall of a double-cell reinforced concrete box culvert. The typical inside dimensions of each cell were 2.4 m high ×3.0 m wide. The thickness of the top and bottom slab was 0.78 m, while for the side walls was 0.41 m and for the center wall was 0.28 m. The outside dimensions of the culvert were 4 m high and 7 m wide. Pressure measurements were made on the top slab and side wall of the culvert using three different methods at two sections A and B as shown in Figure 2.19. The culvert was under a backfill of 0.6 m of gravel and silty clay soil fill to the height of 18.9 m at section A and the ($H/Bc$) ratio for this culvert was 2.7, while at section B the height of soil fill was 11.7 m and therefore, the ($H/Bc$) ratio was 1.7. Firstly, six pressure cells were used to give direct measurements of the pressure on the roof. Secondly, strain gauges in the wall were used to determine the axial forces in the wall and pressure was back calculated, and thirdly, strain gauge on the roof were used to obtain bending moment and the pressure was obtained from the second derivative of the bending moment function. The SSI factors from the first method ranged from 1.48 to 2.09 on the roof at section A, while at section B, SSI factors ranged from 0.95 to 1.42 with the increase at edges and reduction at cell center. For the side walls, the SSI factors from the second method gives a SSI factor ranged from 1.79 at the bottom to 2.21 at the top for section A, while for section B, the SSI factors ranged from 0.25 at the top and 1.20 at the bottom. It is interesting to note that for section A the SSI factor decreased with depth, while for section B, the opposite happened.
Pimentel et al. (2009) investigated a box culvert that had an interior dimension of 2.0 m high and 2.0 m wide. The thickness of the top and bottom slabs had the same thickness of 0.25 m, and the wall thickness varied from 0.15 to 0.16 m. Above the instrumented box culvert, a 9.5 m high embankment was constructed. Four vibrating wire strain pressure cells were used, three of them on the top slab and one on the side wall as shown in Figure 2.20. The soil used for the backfill was clayey sand. The ratio of the fill height to culvert width ($H/B_c$) is 4.09. The resulting pressures indicated that there is an increase in pressure near the edges and decrease at the center. The SSI factors on the top slab ranged from 1.99 at the edge to 0.86 at the center.
Sun et al. (2011) performed a five year monitoring of pressure measurements on the top slab and side walls of a box culvert installed using the imperfect trench method. The material used to reduce the soil pressure on the culvert was a compressible fill (Geofoam). The 112.78 m long cast in place box culvert used had the inner dimensions of 2.74 m × 2.43 m, with a top slab thickness of 0.64 m and bottom slab thickness of 0.66 m, while the thickness of the side walls was 0.30 m. The culvert was rested on an unyielding foundation of fossiliferous limestone with many interbedded shale laminations. The upper half of the embankment was a mixture of limestone rock and red residual clay, and the lower part consisted of compacted red residual clay. The total height of the backfill above the culvert was 16.46 m. The ratio of the fill height to culvert width was \( H/B = 4.9 \). The geofoam had low stiffness and elastic plastic behavior. The maximum compressive strength was 143.64 kPa and elastic modulus was 6368.04 kPa, while its density was 21.62 kg/m\(^3\). Three section of the culvert were monitored; the first one had 0.61 m thick geofoam with a width equal to \( B_C \), the second had the same thickness, but a width of 1.5\( B_C \), and the third one had no geofoam. Twelve vibrating wire pressure cells were used to measure the soil pressure by placing two of them on top slab and the other two on the side wall at each section. A layer of sand had a thickness of 0.30 m placed between the top slab and the geofoam. The average pressure measured on the top slab at the 1BC and 1.5BC were 8.9% and 11.2% respectively of the pressure measured without geofoam, which is a significant reduction. On the side walls, the average pressure was 13.6% of the pressure measured on the top slab.

A summary of the results reported in the literature for the soil-structure interaction factors for instrumented single and double box culverts are presented in Table 2.1. The soil-structure interaction factors are given as functions of the soil fill height to culvert width ratio \( H/B_C \) and when possible to thickness to width of culvert \( t/B_C \). Most of the soil-culvert interaction factors \( F_e \) obtained and collected from previous field experiments clearly show that there is considerable increase in the soil pressure attracted to the culverts, especially for the embankment installation, and a reduction in the soil pressure for imperfect trench installation. Also, for box culverts that have instruments near edges and centers, differences are found between the measurements, which indicate that there will be non-uniform distribution of soil pressure on the culvert (as proposed theoretically). These results will be used later in this chapter, to compare the relations derived from numerical analysis and presented in codes and standards. Also, these will be used in Chapter 6 to compare with the results obtained from the numerical analysis of this thesis.
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2.2.2.3 Relations of soil structure interaction for box culverts

The theoretical vertical soil pressure $\sigma_v$ is usually obtained by multiplying the unit weight of the soil column above the box culvert by the height. Taking the effect of soil arching into consideration, the actual vertical soil pressure on the top slab of a box culvert is determined as the theoretical vertical pressure times the soil structure interaction factor $F_e$. This factor can be referred to as a soil modification factor because it is numerically equal to the factor by which the soil unit weight can be multiplied to obtain the equivalent soil unit weight used to calculate the vertical pressure acting on the culvert. So the actual vertical soil pressure can be obtained from the following equation:

$$\sigma_v = (F_e \times \gamma_S)H \quad (2.4)$$

where $H$ is the height of soil fill over the culvert and $\gamma_i$ is soil unit weight.

Marston and Anderson (1913) were the first to study the effect of soil pressure coming from the prism of soil right on top of circular culverts and pipes. They developed a theory for this effect depending on the installation type of the culverts either embankment or trench installations. Spangler (1947) presented the soil deflection compared to the culvert deflection in a form of chart. The design pressure was a function of the projection condition; various soil properties and the soil fill height above the culvert $H$ to culvert width $B$ ($H/B$) ratio. Clarke (1967) presented all of the previous work in addition to more complicated cases and developed formulas and charts for design of circular sections of buried conduits. The soil structure interaction factor relations were as follows:

For embankment installation:

$$F_e = C_C = \frac{e^{2K\mu(H/B)}}{2K\mu} - 1 \quad (2.5)$$

where $K\mu = 0.19$

$$F_e = C_C = \frac{e^{0.38(H/B)}}{0.38} - 1 \quad (2.6)$$
For trench installation:

\[ F_e = C_c = \frac{1 - e^{2K\mu(H/B)}}{2K\mu} \] (2.7)

where \( K\mu = 0.13 \)

\[ F_e = C_c = \frac{1 - e^{0.26(H/B)}}{0.26} \] (2.8)

Based on typical values for design and formulas developed by Clarke (1967), Bennett et al. (2005) modified the soil structure interaction factors \( F_e \) to be used for the embankment condition of box culverts as:

\[ F_e = e^{0.38(H/B)} \frac{-1}{0.38(H/B)}, \quad H/B \leq 2.42 \] (2.9)

\[ F_e = 1.69 - \frac{0.12}{H/B}, \quad H/B > 2.42 \] (2.10)

In Equation 2.9, the \( H/B \) ratio of 2.42 corresponds to the plane of equal settlement being at the ground surface. Spangler (1947) defined the plane of equal settlement as the horizontal plane in the embankment at which the settlements of the interior soil prism above the culvert and the exterior soil prism outside the culvert are equal. For embankment heights greater than the plane of equal settlement, the soil structure interaction factor is essentially constant. In Equation 2.10 the \( F_e \) values varies from 1.64 at \( H/B = 2.42 \) and increase to 1.69 at \( H/B = \infty \).

Katona et al. (1981, 1982) used CANDE-1980 to model the Kentucky installation of a box culvert with dimensions of 1.20 m × 1.20 m, under a 23.5 m of fill (\( H/B = 19.6 \)). The soil pressure was in good agreement with the pressure measured at specific locations on the top and bottom slabs and less agreement on the side walls. They found that the soil pressure does not have a uniform shape, but it takes a parabolic shape with increase at the edges and decrease at the center.

Tadros et al. (1989b) numerically modeled a box culvert with dimensions of 2.7 m × 2.7 m with a backfill height of 4.6 m above the top slab (\( H/B = 1.7 \)). Two soil types, silty sand and silty clay, were used to obtain the CANDE predictions. These two types were selected to
represent the extremes of a wide range of soils. The results show that the pressures for the silty sand model are smaller at mid span and larger at the corners than for the silty clay model. This is to be expected, since the relative stiffness of silty sand to the culvert is greater than that of silty clay.

Tadros et al. (1989b) modeled another box culvert with dimensions of 1.5 m × 2.1 m under a backfill height of 8.2 m of silty sand \((H/B = 5.5)\), and considered two soil models; a linear elastic model and Duncan hyperbolic soil model, to examine the effect of soil arching. The results of the CANDE models show that the differences between the two soil models were small. The predictions for the average pressure on the top slab are 9% higher than weight of the soil column (i.e. \(F_e = 1.09\)). Additionally, the results show that the soil pressure is not uniform on the top and bottom slabs, with higher values at edges and lower at center.

CANDE-1980 was used by Tadros et al. (1989b) to perform a parametric study to predict the soil pressure on all four sides of the box culvert. Twelve box culverts ranging in size from 1.20 m ×1.20 m to 3.70 m ×3.70 m were modeled using both soils silty sand and silty clay. Calculated pressures were found to be higher at the more rigid corners and less at the middle of the top and bottom slabs. Side pressures were of the form that can be approximated as a trapezoid. The SSI factors were derived as a function of the fill height \((H)\). These factors for top slab were found to increase consistently as the fill height increase, and at low fill heights the SSI factor is approximately 1 and increases to about 1.15. That could be due to the soil at the sides of the culvert settling more than the column of soil directly above the top slab, which induces downward drag. The SSI factor for the side pressures was generally about 0.6, which was in consistent with the at-rest \(K_o\) conditions against non-yielding walls. The SSI factors for the bottom slab were found to be higher for relatively low fills and approached 1.0 as the fill increased. Also, it seems that as the box culvert had taller sides, the soil pressure on the bottom slab would be higher to three or four times the pressure from the soil prism above the culvert. This could be due to the increase in differential settlement of the adjacent soil relative to the box and the soil directly above. Based on the above parametric study, Tadros et al. (1989b) presented a list of equations that can be used to obtain the soil pressure on all culvert sides. The original equations were presented to give pressure in lb/ft², and it was modified to give pressures in kPa as follows:
For top slabs:

\[ P_T = (0.984 + 0.0019H)\gamma_s H \]  \hspace{1cm} \text{(Silty Clay)} \hspace{1cm} (2.11)

and

\[ F_e = 0.984 + 0.0019H \]  \hspace{1cm} \text{(Silty Clay)} \hspace{1cm} (2.12)

\[ P_T = (0.970 + 0.0020H)\gamma_s H \]  \hspace{1cm} \text{(Silty Sand)} \hspace{1cm} (2.13)

and

\[ F_e = 0.970 + 0.0020H \]  \hspace{1cm} \text{(Silty Sand)} \hspace{1cm} (2.14)

For side walls:

\[ P_S = (0.600)\gamma_s H \]  \hspace{1cm} \text{(Silty Clay)} \hspace{1cm} (2.15)

and

\[ F_e = 0.600 \]  \hspace{1cm} \text{(Silty Clay)} \hspace{1cm} (2.16)

\[ P_S = (0.567)\gamma_s H \]  \hspace{1cm} \text{(Silty Sand)} \hspace{1cm} (2.17)

and

\[ F_e = 0.567 \]  \hspace{1cm} \text{(Silty Sand)} \hspace{1cm} (2.18)

For bottom slabs:

\[ P_B = P_T + (2.73 + 0.38H_C)\frac{2H_C}{B_C} \]  \hspace{1cm} \text{(Silty Clay)} \hspace{1cm} (2.19)

\[ P_B = P_T + (5.46 + 0.24H_C)\frac{2H_C}{B_C} \]  \hspace{1cm} \text{(Silty Sand)} \hspace{1cm} (2.20)

where \( P_T \) is the pressure on the top slab, \( P_S \) is the pressure on the side wall and \( P_B \) is the pressure on the bottom slab. \( H \) is the fill height and \( H_C \) is the culvert height and \( B_C \) is the culvert width. \( \gamma_s \) is soil unit weight.

Kim and Yoo (2005) examined the soil foundation under the culvert and its effect on the soil structure interaction factors. They defined the soil structure interaction factor as ‘effective
density’ in their numerical modeling study. After some trials for the effect of soil width to culvert width ratio, the width of the soil was taken as six times the culvert width and the depth of the soil foundation below the culvert was adopted as four times the culvert height. The size of the culvert used in their analysis was 2.4 m × 2.4 m with a uniform thickness of 0.305 m. In their study, the height of soil fill varied from 15.2 m to 61 m (H/B = 6.3 to 25.4) for the embankment installations, and from 15.2 m to 45.7 m (H/B = 6.3 to 19) for the trench installations. The soil structure interaction factors or the effective density was found to be most sensitive to the foundation characteristics. Therefore, based on their regression analysis, the following set of equations for effective density $D_E$ was proposed based on the soil foundation type either yielding or unyielding. For embankment installations the relations are a function of the fill height $H$, while the trench installations are a function of the of fill height to trench width ($H / B_d$). In both cases, the width of culvert was ignored.

For embankment installation:

$$F_e = D_E = 1.047 H^{0.055}; \quad \text{yielding foundation} \quad (2.21)$$

$$F_e = D_E = 1.200 H^{0.059}; \quad \text{unyielding foundation} \quad (2.22)$$

For trench installation:

$$F_e = D_E = \exp[0.012(H / B_d)^2 - 0.288(H / B_d) + 0.375]; \quad \text{yielding foundation} \quad (2.23)$$

$$F_e = D_E = \exp[0.011(H / B_d)^2 - 0.273(H / B_d) + 0.465]; \quad \text{unyielding foundation} \quad (2.24)$$

It was noticed that the effective density values increase as the fill height increases for the embankment installation, while in the trench installations, the effective density reduces as the ratio of the $H / B_d$ increases. However, in both cases, the effective density from an unyielding foundation is higher than the yielding foundation. Kim and Yoo (2005) also investigated the imperfect trench method numerically and reported that the preferred width of the compressible layer should not exceed $1.5B_C$ and the ratio of the thickness of the compressible layer to the height of the culvert should not be greater than 1.5. The maximum load reduction rate is
achieved when the compressible layer is placed directly on the top slab of the culvert, and concluded that the imperfect trench method reduced the soil structure interaction factors.

Kang et al. (2008) employed the finite element method to study the side friction of a box culvert with dimensions of 3.6 m × 3.6 m and a uniform thickness of 0.36 m under different soil fill heights. The soil properties used in the analysis represented the cases of compacted and uncompacted side fills, and also the foundation soils were varied to represent the cases of yielding and unyielding foundations. Soil structure interaction factors were developed as a function of the ratio of the fill height $H$, and the culvert width $B_C$ ($H/B_C$) for yielding and unyielding foundations. The SSI relations for the embankment installation were derived based on analytical values by means of regression analysis. Therefore, the proposed relations are as follows:

For Embankment installation:

For top slab:

$$F_e = -0.005(H / B_C) + 1.304,$$

on compacted side fill

(2.25)

$$F_e = -0.012(H / B_C) + 1.407,$$

on uncompacted side fill

(2.26)

For bottom slab:

$$F_e = 0.004(H / B_C)^2 - 0.105(H / B_C) + 2.105,$$

on compacted side fill

(2.27)

$$F_e = 0.006(H / B_C)^2 - 0.175(H / B_C) + 2.685,$$

on uncompacted side fill

(2.28)

Kang et al. (2008) also studied the imperfect trench installation method and proposed an earth load reduction rate ($R$) based on the elastic modulus ($E_S$) of the light weight material used in the trench and the coefficient of friction ($\mu$). The soil structure interaction factor for imperfect trench was recommended to be calculated using the following equation:

$$F_{ei} = F_e (1 - R/100)$$

(2.29)
where \( F_{ei} \) is the soil structure interaction factor in ITI, and \( F_e \) is the soil structure interaction factor given by Equations (2.25 to 2.29) for embankment installation. The reduction rate \( R \) is as follows:

For yielding foundation

\[
R = 66.46 e^{-0.0006E_s} \mu^{-0.0517}, \quad \text{compacted sidefill} \tag{2.30}
\]

\[
R = 61.94 e^{-0.0005E_s} \mu^{-0.0808}, \quad \text{uncompacted sidefill} \tag{2.31}
\]

For unyielding foundation

\[
R = 76.37 e^{-0.0005E_s} \mu^{-0.0131}, \quad \text{compacted sidefill} \tag{2.32}
\]

\[
R = 69.48 e^{-0.0004E_s} \mu^{-0.0574}, \quad \text{uncompacted sidefill} \tag{2.33}
\]

Kang et al. (2008) raised a concern in the imperfect trench installation method about the effect of downward frictional forces acting along the side walls on the increase of the contact pressure on the bottom slab.

Pimentel et al. (2009) used nonlinear finite element model to investigate the behavior of a box culvert with same dimensions mentioned earlier. In this analysis, the soil was modeled with plane strain state, while the culvert was modeled assuming plane stress conditions, by using plane stress elements instead of beam elements to produce failures. A nonlinear material model was used for the reinforced concrete and an elastic-plastic model for the soil and soil structure interface. To evaluate the effect of nonlinear behavior on the soil structure interaction, a comparative study was presented where the isolated effect of nonlinearity was studied. The analysis with material set A1 was performed up to culvert failure. The results obtained were compared to the same analysis, but assuming an elastic behavior for the box culvert. The summary of the results are shown in Figure 2.21. The results are presented in terms of three interaction factors: global interaction factor, \( F_e \), shear interaction factor, \( F_V \), and bending moment interaction factor, \( F_M \) as a function of the embankment height \( H \). Both the results of \( A1_{\text{confined}} \) and \( A1_{\text{elastic}} \) enabling the perception of the consequence of nonlinearity on the load that reaches the structure. Figure 2.21 show four stages: In Stage 1, both analyses are very similar, because both...
of them are still displaying elastic deformations. Starting from Stage 2, the effect of nonlinearity appears in the divergence in the results, where the elastic analysis continues to increase, while the nonlinearity causes a reduction in the interaction factors. It can be noted that the shear and moment factors appear always to be less than the global interaction factor in all stages. Also comparing the results with AASHTO results, it can be concluded that both analyses lie between the AASHTO provisions.

Joa et al. (2003) investigated numerically the effect of a strip footing right above the box culvert. In their analysis, they considered the change in culvert size (0.9, 1.8, 2.7 and 3.7 m), location and wall thickness (25.4, 76.2, and 152.4 mm). The footing analyzed was a reinforced concrete strip footing embedded in the soil to a depth of 0.3 m and its width of 0.9 m. Both concrete and soil were modeled as a nonlinear elastic perfectly plastic material. The results of this study demonstrated that the footing induced soil pressure distributions around box culverts

Figure 2.21: Evolution of global interaction factor, $F_e$, shear interaction factor, $F_V$, and bending moment interaction factor, $F_M$, for Analyses A1\textsubscript{confined} and A1\textsubscript{elastic} (after Pimentel et al., 2009)
are strongly influenced by the interaction between the culvert and the surrounding soil. Also, the soil pressure distribution strongly depends on culvert size and thickness. More data is still required to establish a database for the development of rational methods for design of box culverts overlain by strip footings.

2.2.2.4 Centrifuge and numerical modeling of soil structure interaction for box culverts

The soil structure interaction factors predicted from previous research either experimentally or numerically strongly indicate that there is soil arching occurring around box culverts. Most of the previous instrumented and studied box culverts were for embankment installations, while the trench installations of box culverts are very limited. As an example of that, Chen et al. (2010) presented an experimental and numerical simulation for concrete box culverts with a top slab taking an arch shape installed using the trench method.

Stone et al. (1991) performed centrifuge tests on a rectangular box culvert to replicate a prototype dimension of 3 m wide by 2 m high. Three different thicknesses (0.5, 1.0, and 1.62 mm) were used to investigate the effect of relative stiffness between the culvert and the surrounding soil on the loads attracted to the culvert. Due to the difficulty of constructing micro concrete model for the culvert, mild steel was used for the model. The culvert was instrumented by forty miniature strain gauges on both faces to enable calculating the bending moment and axial force. To calibrate the strain gauges, the culverts ends were capped and vacuum applied to the inside of the culvert. This produces loading condition equivalent to the uniform external pressure. Well graded quartz sand was rained inside the centrifuge box until the \( H/B \) ratio approximately 3.3. The centrifuge spun to 75g and 150g and the data were collected.

The response of the culvert to the applied loads was best represented by bending moment plots where the mode of deformations was possible to determine. It was noted that while the response of the top slab is similar for all the models, the response of the side walls and base of flexible culvert is different to that observed for stiff culverts. The flexible culvert (0.5 mm) sides, bending moment at the edges were positive and at the center were negative and this led to two points of inflection observed in the side walls and similar effects in the base. However, in the other two culverts (1.0 and 1.62 mm) all of the bending moments were on the positive side. The total loads attracted to the culvert can be obtained from the strain gauges mounted on the side walls. The ratios of the measured loads to the assumed uniform theoretical loads were found to
be 1.23, 1.66 and 2.77 for the 0.5, 1.0 and 1.62 mm culverts respectively. This shows that stiff culverts attract more loads due to the passive nature of the soil arching above the stiff culverts. The authors found that fitting the bending moment data with one cubic equation can be applicable for stiff culverts, but it is not applicable for flexible culverts and can underestimate the loads at edges by some 50%. Therefore, they suggested using a cubic equation for the edge side and a quadratic equation for the center of the top slab as shown in Figure 2.22.

![Figure 2.22: Possible loading and associated bending moment distribution in the culvert top slab (after Stone et al., 1991)](image)

Stone and Newson (2002) performed further centrifuge tests on a non-uniform 300 mm long rectangular box culvert model to investigate the contribution of the structural elements (top slabs and side walls) of the buried structure to the attracted loads to it. The model culvert used was made from aluminum and had an external dimension of 101.6 mm and initial thickness of
6.35 mm as shown in Figure 2.23. Two opposite sides were machined to a thickness of 2 mm, and the other sides remained the same original thickness. Pairs of miniature strain gauges were used on the external and internal sides of the model to be used to obtain the bending moment and axial force. Congleton Sand was used in the centrifuge tests and the height of sand for the reported results was 200 mm above the model \((H/B = 2)\). The centrifuge was accelerated and then held at 5g, 10g and 20g and data of strains were recorded. The results presented were for two tests; Test A: stiff sides and flexible top and base, and Test B: flexible sides and stiff top and base.

For Test A, the normalized bending moments from the three g levels were fitted with a parabola. Therefore, the second derivative gave a uniformly distributed load on the top slab. The measured load \((w_{\text{meas}})\) was 42, 45, and 47% of the load derived from the soil prism \((w_{\text{prism}})\) for the g levels 5g, 10g and 20g respectively (Newson and Stone, 2006a). This indicates that there is no effect for the deflection of the top slab on the loads attracted to it. For Test B, The ratio of the measured to theoretical loads on the top slab \((w_{\text{meas}} / w_{\text{prism}})\) was 0.42, 0.97 and 1.16 for the g levels 5g, 10g and 20g respectively. Newson et al. (2006b, 2007) revisited the same tests shown above but with a 100 mm soil cover \((H/B = 1)\), and came to similar conclusions.
Abuhajar et al. (2009) analyzed numerically using finite element method two of the centrifuge test models performed by Stone and Newson (2002) as explained above. Both culverts used in this analysis had a 2 mm uniform thickness. In these two cases, one of the culverts was in the normal parallel position and the other rotated 45° and the soil and culvert properties used are shown in Figure 2.24. The centrifuge tests were performed up to 80g with increments of 5, 10, 20, 40, 60, and 80g. The numerical analysis was done for the case of 20g only. The soil was modeled using elastic perfectly plastic model, while the culvert used a linear elastic model. Due to symmetry, only half the model was considered. The loads attracted to the top slab of both culverts were calculated using the prism load of overburden pressure and the resulted load obtained from the finite element analysis. Both culverts show reduced loads attracted to the upper slabs of the structure. The load ratios \((\frac{w_{meas}}{w_{prism}})\) were 0.90 and 0.37 for the parallel and rotated culverts respectively.

Figure 2.24: Schematic diagram showing the orientation of the two culverts and the properties used for analysis (after Abuhajar et al., 2009)
Okabayashi et al. (1994) performed a series of centrifuge tests on a box culvert buried inside a dry sand to examine both the positive projection installation and the imperfect trench installation using flexiable materials. They studied the effect of width and location of a thin layer of the flexible material (EPS) above and under the culvert on soil pressure on top slab and side walls of a model box culvert. The main observations are: increasing the width of the compressible material resulted in a slight increase in the soil pressure on the top slab; higher pressures occur close to the edges compared to the center of the top slab; and the most reduction in soil pressure happens when the flexible material was located closer to the top slab of the culvert rather than higher inside the embankment.

Li and Qubain (2004) numerically investigated the effect of foundation yielding on design loads. Yielding, partially yielding and unyielding foundations were analyzed and compared with conventional methods. Using 2D FEM program (SIGMA/W), a precast concrete box culvert with dimensions of 3.35 m × 3.81 m with a total fill height of 11.58 m was analysed ($H/B = 3.45$). The soil structure interaction factor for the unyielding foundation was 1.30 and for yielding foundation was 1.0, while for the partially yielding foundation, the pressures were similar to the unyielding foundation. Li and Qubain (2004) observed that the bending moment on the top slab was significantly reduced at the center and slightly increased at the edges. The SSI factors for the side walls was 0.27 and 0.41 for the unyielding and yielding foundations respectively.

Bourque (2002) used the University of New Branswick (UNB) geotechnical centrifuge to evaluate the vertical and horizontal soil pressures acting on single and twin box culverts under an imperfect trench method. He performed a parametric study using numerical modeling to investigate the effect of culvert spacing, width of compressible zone, culvert geometry, and backfill material. He observed that the soil pressure on the side walls is higher than the vertical pressure on the top slab for both single and twin box culverts. Results from this study were addressed later in details by McGuigan and Valsangkar (2011).

MacLeod (2003) investigated the soil pressure around circular and box culverts in an imperfect trench method of installation using centrifuge and numerical modeling and compared the prototype results from the literature. He addressed the compressible zone width, thickness, stiffness and location on the induced trench conduits. Results from the centrifuge tests were incorporated in a later study by more details by McGuigan and Valsangkar (2010).
McAffee (2005) performed centrifuge tests on a square box culvert to study different $H/B_C$ ratios of field structures installed using the imperfect trench method, for both single and twin culvert shapes, and compared the results to the case of positive projection method. Results confirmed that there is a reduction in the vertical soil pressure and increase in lateral soil pressure. Important factors that affect the results were concluded to be width and height of the compressible layer. Results from this study were addressed later in details by McGuigan and Valsangkar (2011).

McAffee and Valsangkar (2008) monitored the soil pressure in the field on a circular culvert under imperfect trench installation for two years. Then a box culvert model with the same soil cover and pipe width as well as the thickness of the compressible layer was tested in the centrifuge and followed by numerical model using FLAC, to simulate the soil pressures on the circular culvert. The box culvert model used had the dimensions of $38 \text{ mm} \times 38 \text{ mm}$ and used air pluviated uniform silica sand and was run to 30g as shown in Figure 2.25. Kyowa BE-10KC soil pressure transducers were used to measure the pressure on the vertical pressure on the top slab and then rotated to measure the horizontal pressure on the side wall. The centrifuge results showed that the soil culvert interaction factors for vertical pressure on top slab were 1.16 and 0.24, while for horizontal pressures on side wall were 0.45 and 0.49 for positive projection and induced trench methods respectively. Numerically, the SCI for vertical pressure on top slab were 1.23 and 0.18, while for horizontal pressure on side wall were 0.52 and 0.57 for positive projection and induced trench methods respectively. It was observed from numerical modeling of the imperfect trench method that the average vertical pressure on the bottom slab is 9% higher than the vertical pressure on the top slab plus the dead load of the culvert, and 30% higher for positive projection installation because of the downward drag forces on the side walls.

McGuigan and Valsangkar (2010) presented a centrifuge and numerical modeling on a single box culvert in addition to a parametric study to evaluate the soil pressures on top and bottom slabs, side walls of box culverts in induced trenches under yielding and unyielding foundations. Two of centrifuge tests considered the unyielding foundation was done by MacLeod (2003) and other tests on yielding foundation was done by the first author. In all tests, positive projection and induced trench installations were used. The box culvert model used had the dimensions of $38 \text{ mm} \times 38 \text{ mm}$ with the same pressure cells. The model was placed in air pluviated uniform silica sand and run to 70g as shown in Figure 2.26. The fill height above the
top slab of the culvert was 152 mm and 162 mm for yielding and unyielding foundations respectively. The centrifuge results showed that the soil culvert interaction factors for vertical pressure on top slab were 1.33 and 0.24, while for horizontal pressures on side wall were 0.43 and 0.46 for positive projection and induced trench methods respectively. The contact pressure on the bottom slab compared to the weight of soil prism plus the dead load of the culvert, and the soil culvert interactions were 1.13 and 0.75 for positive projection and induced trench methods respectively. They observed that the increased pressure on the bottom slab is due to downward drag forces on the side walls. The results of the parametric study identified a preferred compressible zone geometry having width of $1.2B_C$ and thickness of $0.5B_C$.

Figure 2.25: Schematic of centrifuge tests (After McAffee and Valsangkar, 2008)
McGuigan and Valsangkar (2011) used a numerical modeling and parameteric study on the centrifuge tests done before by Bourque (2002) and McAffee (2005) to evaluate the culvert spacing and compressible zone geometry for twin positive projection and imperfect trench box culverts. Bourque (2002) performed centrifuge tests on twin 38 mm square rigid box culverts on an unyielding foundation as shown in Figure 2.27. The height of silica sand was 162 mm above the culvert and the centrifuge was run to 70g. The centrifuge tests were performed for positive projection and induced trench methods with culvert spacings of $0.5B_C$ and $1.0B_C$. Two individual compressible zones were used with $1.0B_C$ in width, as well as a single compressible zone for both culverts. McAffee (2005) performed two series of centrifuge tests on a 38 mm square twin box culverts to simulate the field tests at two sites, where the soil pressure measured on a circular culvert under yielding foundation as shown in Figure 2.28. In the first one, the spacing between culverts was $1.6B_C$, and height of sand above the culvert was 432 mm and run at 48g, while for the second one the spacing was $0.4B_C$ and the soil height was 380 mm and run at 115g. For the

Figure 2.26: Centrifuge strongbox dimensions and general setup for yielding foundation tests
(After McGuigan and Valsangkar, 2010)
parametric study of the positive projection and the induced trench installations, two parameters were investigated: the culvert spacing ($0.5B_C$, $1.0B_C$ and $1.5B_C$) and the compressible zone geometry. Four compressible zone geometries: (i) two zones of $w = 1.0B_C$ (ii) two zones of $w=1.2B_C$ (iii) two zones of $w = (2.0B_C + s)$ and (iv) two zones of $w = (2.2B_C + s)$ where $s$ is the spacing between culverts. It was observed that twin culverts installed by the positive projection method gave lower soil pressures than single culverts. Soil pressure on the top slab of twin induced trench culverts were observed to be higher than the soil pressures of single culverts. However, horizontal pressures were observed to be low in twin culverts compared to a single one, while the lower pressure on the bottom slab was observed for twin culverts with $0.5B_C$ spacing than for single culverts. A single compressible zone spanning both culverts $w = (2.0B_C + s)$ was proposed for culverts spaced at $0.5B_C$ and $1.0B_C$, while for a spacing of $1.5B_C$, two individual compressible zones of $1.2B_C$ width were preferred.

![Figure 2.27: Geometry of centrifuge tests performed by Bourque (2002)](after McGuigan and Valsangkar, 2011)
Oshati et al. (2012a) presented soil pressure measurements from field test results and centrifuge tests for a cast in place reinforced concrete double cell rectangular box culvert with dimensions of 6.75 m wide and 4.65 m high under a 25.10 m of well graded granular soil fill height. The internal dimensions of each cell was 2.60 m width and 3.60 m high with a top and bottom slab thicknesses of 0.45 m and 0.60 m respectively, while the outside and inside wall gave thicknesses of 0.55 m and 0.45 m respectively. The thickness of the compressible zone was 2.50 m and its width is 8.0 m. To measure the soil pressure, 32 Geokon vibrating wire earth pressure cells were used in the field at two different sections A and B, 8 m apart. In the centrifuge tests, a scaled aluminum model culvert had the dimensions of 76 mm wide by 52.4
mm high and 195 mm long was used and the centrifuge run to 89g as shown in Figure 2.29. The Kyowa BEC-A-500 kPa soil pressure cell that cover 60% of the culvert width was used to measure the pressure on top slab, bottom slab and side walls. Silica sand was air pluviated and used as backfill of 281.7 mm height, and 5.6 mm thick between the top of the culvert the base of the EPS compressible layer, and as a yielding foundation. The soil culvert interaction factors from centrifuge results were 1.16, 0.21 on the top slab, 0.36, 0.54 on the side walls and 1.61, 0.70 on the bottom slab for the positive projection and imperfect trench installations respectively. Results from field tests show that the soil culvert interaction factors for induced trench installation were (0.41 – 0.46) and (0.35 – 0.39) on the top slab, (0.40 – 0.47) and (0.48 – 0.65) on the side walls, while (0.53 – 0.58) and (0.52 – 0.57) on the bottom slab at sections A and B respectively.

Oshati et al. (2012b) monitored the soil pressure from two double cell cast in place reinforced concrete box culverts as shown in Figure 2.30. One of them was constructed using the positive projection installation and the other using the induced trench installation. The induced trench culvert was described above (Oshati et al., 2012a). The 193.50 m long positive projection culvert has the dimensions of 7.30 m wide by 4.53 m high under a 14.10 m of soil fill. 13 pneumatic-type pressure cells were used to measure soil pressure. Centrifuge tests were done as described above to account for the two installations (Oshati et al., 2012a). For the case of positive projection the centrifuge run to 50g to represent a 14.10 m of fill height. The soil culvert interaction factors resulted from the centrifuge tests were (1.12 – 1.23) and (0.21 – 0.24) on the top slab, (0.28 – 0.37) and (0.47 – 0.56) on side walls and (1.56 – 1.77) and (0.56 – 0.82) on bottom slab for positive projection and induced trench installations respectively. Results of field tests show that the soil culvert interaction factors were 0.94 and 0.56 on the top slab, 0.32 and 0.46 on side walls and 0.72 and 0.66 on bottom slab for positive projection and induced trench installations respectively. It was found that the soil culvert interaction factors reduced as the ratio $H/B_C$ increased, and the relationship is dependent on the settlement rate which can be influenced by several factors such as compaction, bedding, and installation method.

It should be noted that all centrifuge tests performed at UNB and the corresponding numerical models were performed on a rigid solid square aluminium section, which would definitely give responses that diverged from the real shape of the hollow box culvert. The real shape of box culvert will allow for some flexibility in the structural elements and fixity at the
joints. This was very clear from the pressure distribution on the top and bottom slabs where the pressure is almost a straight line (uniformly distributed) and this is expected from a very rigid structure. Also, McAfee and Valsangkar (2008) compared their field pressure results for circular culvert to a square box culvert, which are completely different structures. The effect of the structural members are investigated in this thesis, which will show clearly their effects on the results of the soil culvert interaction factors.

Figure 2.29: Centrifuge model set-up (units: mm) (After Oshati et al. 2012)
Figure 2.30: Culvert dimensions and pressure cell locations (units: mm)

(After Oshati et al. 2012)
2.2.3 Soil Structure Interaction for box culverts in Codes and Standards

2.2.3.1 AASHTO

Several versions of the American Association of State and Highway Transportation Officials (AASHTO 1994, 1996, 2002, 2005, 2007) proposed calculating the vertical and horizontal soil pressure around box culverts using the principles of soil mechanics, and based on Article 6.2. In this article the density of soil for vertical and horizontal pressures are assigned as shown in Table 2.2.

For any type of box culvert construction method either the case-in-place or precast concrete units, the soil structure interaction factors can be multiplied by the calculated soil pressure obtained using the Article 6.2 in AASHTO. Then, the total earth load $W_E$ on the box culvert section can be obtained by:

$$W_E = F_e \gamma_s g B_c H$$  \hspace{1cm} (2.34)

where: $W_E$ is the total un-factored earth load, $F_e$ is the soil structure interaction factor, $B_c$ is the outside width of culvert, $H$ is depth of backfill, $g$ is the acceleration of gravity, and $\gamma_s$ is the density of backfill, as shown in Figure 2.31.

Up until the 12th edition of the AASHTO specification (AASHTO, 1977), the vertical loading was essentially 70% of the weight of the earth prism above the top slab. Starting from the 13th edition of AASHTO, the soil structure interaction factors have different notations depending on the version of ASSHTO used. The older versions use the notation $F_e$ for embankments installation and $F_t$ for trench installation method. While new versions use $F_{el}$ for embankment installation and $F_{e2}$ for trench installation technique. The values of soil structure interaction factors depend on the type and installations of box culverts. ASSHTO (2002) requirements are based on the Marston-Spangler Theory to determine the soil structure interaction factors $F_e$.

For embankments installation the soil structure interaction factor is:

$$F_{el} = 1 + 0.20 \frac{H}{B_c}$$  \hspace{1cm} (2.35)
$F_{e1}$ need not be greater than 1.15 for installations with compacted fill at the sides of the box section, and need not be greater than 1.4 for installations with un-compacted fill at the sides of the box section.

For trench installation the soil structure interaction factor is:

$$F_{e2} = \frac{C_d B_d^2}{H B_c} \leq F_{e1}$$  \hspace{1cm} (2.36)

where: $B_d$ is the horizontal width of the trench, $C_d$ is a coefficient specified in Figure 2.32 for normally encountered soils. The maximum value of $F_{e2}$ need not exceed $F_{e1}$. It should be noted that the factor $F_e$ only considers the overall vertical load attracted to the culvert, and not the actual distribution of pressure against the different structural elements.

Table 2.2: Density of Soil according to AASHTO Article 6.2

<table>
<thead>
<tr>
<th>Culvert in trench, or culvert un-trenched on yielding foundation</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(A) Rigid culverts except reinforced concrete boxes</td>
<td></td>
</tr>
<tr>
<td>(1) For vertical earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$)</td>
<td></td>
</tr>
<tr>
<td>For lateral earth pressure 30 pcf (480.6 Kg/m$^3$ = 4.71 KN/m$^3$)</td>
<td></td>
</tr>
<tr>
<td>(2) For vertical earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$)</td>
<td></td>
</tr>
<tr>
<td>For lateral earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$)</td>
<td></td>
</tr>
</tbody>
</table>

| (B) Reinforced concrete boxes                                  |                  |
| (1) For vertical earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$) |                  |
| For lateral earth pressure 30 pcf (480.6 Kg/m$^3$ = 4.71 KN/m$^3$) |                  |
| (2) For vertical earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$) |                  |
| For lateral earth pressure 60 pcf (961.1 Kg/m$^3$ = 9.42 KN/m$^3$) |                  |

| (C) Flexible Culverts                                          |                  |
| For vertical earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$) |                  |
| For lateral earth pressure 120 pcf (1922.2 Kg/m$^3$ = 18.86 KN/m$^3$) |                  |

Culvert un-trenched on unyielding foundation

A special analysis is required.
Figure 2.31: Box culvert installation:
(a) Embankment installation and (b) Trench installation

Figure 2.32: Coefficient $C_d$ for Trench installation.
2.2.3.2 CHBDC

The Canadian Highway Bridge Design Code (CHBDC, 2000, 2006) specifies the vertical and horizontal arching factors as shown in Table 2.3. To calculate the vertical and horizontal earth loads, the weight of the earth over the top of the box culvert should be multiplied by the vertical and horizontal arching factor $\lambda_v$ and $\lambda_h$ respectively. Earth pressures on the box culvert are assumed to be uniformly distributed vertical pressures $\sigma_v$ and varying linearly horizontal pressures $\sigma_h$ as follows:

$$
\sigma_v = \lambda_v W_c \\
\sigma_h = \lambda_h W_c
$$

(2.37) \hspace{1cm} (2.38)

where $W_c$ is the weight of a column of unit area of fill above the reference point. The maximum and minimum values of $\lambda_h$ should be used to obtain the maximum positive and negative moments in the culvert walls. The reaction pressure on the bottom of the box assumed to be uniformly distributed.

The commentary of the Canadian Highway Bridge Design Code (CHBDC, 2006) shows that these soil structure interaction factors for box culverts are based on previous practice and limited soil-structure interaction analyses by the finite element method.

The soil structure interaction factors depend on the two standard installation methods B1 and B2 for box culverts are defined as shown in Table 2.4, and the soil groups are classified in Table 2.5.

Table 2.3: Arching factors for box sections in standard installations (from CHBDC, 2006)

<table>
<thead>
<tr>
<th>Installation Type</th>
<th>Vertical Arching Factor, $\lambda_v$</th>
<th>Horizontal Arching Factor, $\lambda_h$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
</tr>
<tr>
<td>B1</td>
<td>1.20</td>
<td></td>
</tr>
<tr>
<td>B2</td>
<td>1.35</td>
<td></td>
</tr>
</tbody>
</table>
### Table 2.4: Soils and compaction requirements for standard installations for concrete boxes (from CHBDC, 2006)

<table>
<thead>
<tr>
<th>Installation type</th>
<th>Soil group</th>
<th>Equivalent minimum Standard Proctor compaction in sidefill and outer bedding zones</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>I</td>
<td>90 %</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>95 %</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>Not permitted</td>
</tr>
<tr>
<td>B2</td>
<td>I</td>
<td>80 %</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>85 %</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>95 %</td>
</tr>
</tbody>
</table>

### Table 2.5: Classification of placed soils (from CHBDC, 2006)

<table>
<thead>
<tr>
<th>Soil group</th>
<th>Description</th>
<th>Unified Soil Classification Symbols</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Sand and Gravel</td>
<td>SW, SP, GW, GP</td>
</tr>
<tr>
<td>II</td>
<td>Sandy Silt</td>
<td>GM; SM; ML; GC and SC with less than 20% passing #200 sieve</td>
</tr>
<tr>
<td>II</td>
<td>Silty clay</td>
<td>CL; MH; GC and SC with more than 20% passing #200 sieve</td>
</tr>
</tbody>
</table>
2.2.4 Comparison of Soil Structure Interaction Factors given in the Standards and the Literature

Both AASHTO and CHBDC deal only with positive projecting box culverts. These documents recommend uniform earth pressures over the top and bottom slabs of positive projecting box culverts, with the contact pressure on the bottom equal to the sum of the top pressure and the pressure due to the culvert dead load. (McGuigan and Valsangkar, 2010).

AASHTO did not define the conditions to classify the soil as compacted or uncompacted. In AASHTO, there is no reference concerning the shape of the pressure diagram, and the results provided by many researchers (Bennett et al. 2005; Dasgupta and Sengupta. 1991; Kim and Yoo, 2005; Tadros et al. 1989) show that the pressure is lower at the center of the slab and higher at the edges. The validity of the interaction factors proposed in AASHTO (2002) is limited to the cases of yielding foundation. Kim and Yoo (2005) discussed this subject and concluded that the interaction factor for unyielding foundations can take higher values.

Comparing the CHBDC to AASHTO, it is clearly noticeable that the CHBDC specify a constant arching factor that does not rely on the change in the ratio of $H/B$. The arching factors change only due the installation type. Even though the CHBDC talks about the embankment and trench installation methods, no distinction has been made between them in terms of the arching factors.

Average soil structure interaction factors of the soil pressure measured on the top slabs of the instrumented culverts were compared to the available relations to calculate $F_e$ and all are shown in Figures 2.33 and 2.34 as a function of $H/B$ and $H$ receptively. All of the available relations, except the one by Bennett et al. (2005) which is considered to be too conservative, fall between the upper and lower limits of ASSHTO, and the CHBDC values are constant for any depth of soil cover. ASSHTO and CHBDC $F_e$ values show that they are unconservative. Importantly, the scatter in the test data indicate that the factor $F_e$ may not be a unique function of $H/B$, however, the relative stiffness between the soil and the culvert is not taken into consideration, which could be a significant factor as suggested by Einstein and Schwartz, (1979) and Karinski et al. (2003).
Figure 2.33: Comparison between different methods for calculating the soil structure interaction factors $F_e$ as a function of $H/Bc$.

Figure 2.34: Comparison between different methods for calculating the soil structure interaction factors $F_e$ as a function of $H$. 
2.3 PREVIOUS STUDIES RELATED TO THE SEISMIC LOADING:

2.3.1 Box Culvert Behavior during Past Earthquakes

Youd and Beckman (1996) documented the performance of twenty nine cast in place reinforced concrete box culverts that were subjected to past earthquakes. These box culverts were evaluated through field inspection or review of earthquake reports. None of them required major repairs or replacement, however, only minor damage when subjected to earthquake accelerations in excess of 0.4g. Severe damage and failure occurred in a number of structures where permanent ground displacement was significant. The performance of the precast concrete box culverts is not likely to vary from the performance noted below. Much of the damage was concentrated at construction joints and wall to ceiling joints, likely areas of weakness in pre-cast construction. Nishioka and Unjoh (2003) and Wood (2005) stated that underground structures were thought to be relatively safe during earthquakes until the 1995 Kobe earthquake in Japan where six out of 21 subway tunnel stations suffered severe damages.

The performance of cast in place box culverts and the failure mechanisms under seismic conditions can be summarized as follows:

1- **Penetration of highway fill into foundation material**, which can cause fractures as a consequence of slight penetration of the culvert and overlying embankment into the underlying foundation material. This penetration could happen due to the liquefaction of the foundation soil.

2- **Lateral Spreading**, during earthquakes, lateral spreading can cause large values of ground displacement, which will affect the function of the culvert especially if it was used for power cables or sewerage drainage. This also may cause each part of the culvert to move laterally, vertically or both and cause separation at joints. Lateral displacement pulls the culvert apart at the joints, while associated transverse movements and differential settlements offsets joints laterally and vertically. Ground oscillation, associated with spreading movements could cause joints to open and close with impacts intensifying the damage, including the breaking of the reinforced concrete bars, in addition to the longitudinal and transverse fractures across culvert sections. The longitudinal fractures indicate that the wall may be subjected to larger lateral soil pressure than the wall can withstand.

3- **Surface Faulting**, earthquake damage varied from hair cracks to major shear and moment failure in walls, and roofs. The most severe damage occurs at the zone of surface rupture. The cracks can cause the culvert to be replaced completely. They can be vertical or
horizontal, which is produced by lateral, vertical, and longitudinal forces. Tectonic deformations can cause uplift and lateral displacement.

4- **Increased Lateral Earth Pressure**, Seismic waves can cause compaction in the soil around the culvert, which may increase the pressure on the walls and roofs and lead to longitudinal and transverse cracks.

5- **Ground Shaking**, The above four points indicate that ground shaking in the absence of ground failure or ground deformation is generally insufficient to cause damage to lightly loaded reinforced concrete box culverts. Youd and Beckman (1996) inspected eleven box culverts in area within 10 km from the epicenter subjected to PGA range from 0.5g to 1.0g. There was no visible damage to any of these culverts, since they were well designed and constructed and covered by compacted fill up to few meters thick.

### 2.3.2 Soil Structure Interaction for Box Culverts in Research

The seismic performance of box culverts has attracted little attention mainly due to the lack of reported seismic failures. Consequently, only a few studies focussed on this topic and a few design guidelines are included in general provisions on the seismic design of such structures. Some of these studies are related directly to box culverts and others to other shapes of culverts such as arch culverts. The main reason for the good performance of culverts and buried structures is that they are constrained by the surrounding soil. It is unlikely that they could move independently of the surrounding soil to any significant extent due to vibration amplification or resonance. Typical specifications for static design attempt to control the backfill to ensure acceptable performance under gravity loads and avoiding settlements, which leads to good seismic performance (Anderson et al, 2008). Underground structures cannot move independently, so are not generally subjected to significant dynamic amplification effects. They are affected by the deformation of the surrounding soil and not the inertial forces acting on the structure (Wood, 2005). Hashash et al. (2001) stated that the dynamic earth pressure for rectangular buried structure under plane strain conditions using Mononobe-Okabe method leads to unrealistic results and is not recommended for underground structures. That is because this method was developed for retaining walls and assumes the wall structure to move yielding an active wedge. However, buried structures will move together with soil making the formation of the wedge difficult.
Chen (1988) used the finite difference method to investigate the dynamic soil structure interaction of reinforced concrete life line structures. The effect of embedment of life line structures that have a rectangular shape was studied under earthquake excitation. The thickness of the structure was 0.30 m and its dimensions were 4.0 m × 4.0 m. Three embedment depths were used which are $0H_C$, $0.5H_C$, $2H_C$ and the height of the soil profile was 34.8 m. Chen (1988) concluded that the peak structural motions were found to be influenced by the following system parameters: soil cohesion, embedment, material nonlinearity, structural shape and frequency of the accelerogram used. The peak structural responses occurred at the corners of the structure, and the occurrence time for peak structural motions changed with the embedment and the shape of the structure. The fully buried structure experienced the earliest peak motions. Circular structures can suffer less damage under severe earthquake effects than the rectangular structure. Chen (1988) proposed a transfer function and design spectra to be used in design and depending on the soil cohesion, a ductility factor between 2.5 and 10 can be employed for seismic design of the structural system.

Byrne et al (1996) presented results of a numerical study for seismic analysis of large arch culvert structure. Three types of analyses were applied: static, pseudodynamic and fully dynamic using the finite element method. In the pseudodynamic analysis, earthquake loads were simulated by additional static loads specified by seismic coefficients $k_h$ and $k_v$, applied to the elements. The forces on each element increased by $kW$, where $W$ is the weight of the element and $k = A/g$ where $A$ is the peak ground acceleration and $g$ is the acceleration of gravity. In the fully dynamic analysis, the loads were applied using the acceleration time history from the San Fernando earthquake. Byrne et al (1996) found that for the horizontal seismic loading, the surrounding soil is much stiffer than the arch and the seismic loads were taken by the soil. Under vertical seismic loads, the arch is stiffer than the soil and attracts more loads. The increased forces and moments from shaking arise from both horizontal and vertical components of accelerations. Such analyses show that the increase in thrust is largely controlled by the vertical component of the earthquake, while the increase in moment is largely controlled by the horizontal component of the earthquake. The moments are very sensitive to the backfill and surrounding soil stiffness properties and less sensitive to the foundation soil beneath the arch (Wood and Jenkins, 2000).
Turan and El Naggar (2011) used the finite difference method to study the seismic behaviour of an arch culvert. The study included the effects of soil structure interaction (SSI) on the ground motion as well as the effect of dynamic loading amplitudes on the arch’s moments and thrusts under different compaction arrangements. The dynamic analysis was performed using the Ricker wavelet acceleration time history at the base of the model. The amplification of ground motion in the free field was about 40% higher than at the arch structure. As the level of compaction reduced, the PGA of 0.3g was amplified at the free field and the amplification factor increased by 259%. Seismic moments and thrusts were found to increase as the PGA increased. For example, at 0.3g, the maximum seismic moment is 3.9 times the static moments. The seismic moments were sensitive to the backfill compaction, as a result of that the seismic moments were 3.9, 5.1, and 8.5 times the static moment for symmetric backfilling with different levels of compaction.

Luzhen et al. (2010) described a series of shaking table tests performed on a scaled box section tunnel model for a newly built tunnel in China. This tunnel has a square section of 3000 mm × 3000 mm and uniform thickness of 300 mm. These tests were done to explore its performance under earthquake excitation. A finite element analysis then followed to simulate the soil structure dynamic interaction, and the confinement effect of the laminar box. Sensors used in the shaking table tests were accelerometers, strain gauges, earth pressure cells, and wired displacement transducers as illustrated in Figure 2.35. The El-Centro earthquake was used as input excitation with PGA values of 0.1g, 0.4g, and 1g. Figure 2.36 shows the distribution of the soil pressure when the time of the maximum story drift of the laminar box was reached. It was found that the soil pressure is symmetrically distributed on the left and right side walls, which means similar values with negative on the left and positive on the right. On the top and bottom slabs, the soil pressure took a triangular shape. From these tests, it was observed that, the acceleration response of the structure is less than the soil; the maximum internal forces appear at the corners and increase with the increase of PGA; earth pressure increases as PGA increases; and the displacement drift between the top and bottom slabs is produced under horizontal earthquake excitation accompanied with the rotation of the tunnel cross section.
Figure 2.35: Layout of measuring sensors (After Luzhen, et al., 2010)
Liu and Song (2005) numerically investigated the seismic response of a two storey subway underground structure in liquefiable soil. Effective stress based, fully coupled dynamic finite element modeling was used. The soil profile was assumed to be a homogenous liquefiable soil and modeled using a generalized plasticity model. Horizontal as well as vertical excitation was used in addition to different buried depths. The countermeasure against uplifting was also studied. It was found that the effect of vertical motion depends on the characteristics of the excitation. Also, it was found that the increase of the buried depth improves the safety of the underground structure against earthquake damage. Using injected grout proved to be effective against structure floatation.

Wang et al. (2005) performed effective stress analysis in a centrifuge as well as numerical modeling to study the seismic response of a box culvert model made of reinforced
concrete. At 30g in the centrifuge, the culvert was 9 m high and 12 m wide under a 16 m dry and saturated soil. The model was subjected to PGA of 0.8g to 0.9g in prototype scale. It was found that the soil pressure acting on the side wall increased after shaking in both dry and saturated models. These pressures together with the lateral displacement due to shaking lead to yielding of side walls. The lateral displacement of soil is larger than the culvert in both dry and saturated models and they were almost identical.

Wang et al. (2006) reported results of centrifuge and numerical modeling of box culvert as described by Wang et al. (2005) to investigate its seismic response to liquefiable soil in nuclear plants. Similar findings were observed, but with the use of sine wave input, the displacements in the case of frequency 0.8 Hz under 0.35g were larger than those of the case with frequency 1.2 Hz under 0.50g. Also, by using earthquake records, it was shown that the displacements were less than those using sine waves. The earth pressure acting on the side wall increased gradually to the value of total vertical stress in the free field on average.

2.3.2.1 Racking of rectangular/box culverts

Anderson et al. (2008) stated that the general effects of earthquakes on culverts can be generated by either ground shaking or ground failure. Ground shaking refers to the vibration of ground produced by seismic wave (Body and Surface waves) propagation through the earth’s crust. The shaking or wave travelling induced ground deformations are called transient ground deformations/displacements (TGD). Three types of deformations that can happen due to TGD which are: Axial deformations, Curvature deformations, Ovaling (Circular section) or Racking (Rectangular section) deformations. Axial and curvature deformations are unlikely to happen in culverts due to its limited length. Ovaling and racking deformations may develop when the waves propagate in perpendicular or nearly perpendicular directions to the longitudinal axis of the structure, resulting in a distortion of the shape of the structure. The vertically propagating shear wave is the predominant form that governs the ovaling/racking because of: (1) ground motion in the vertical direction is less severe than in the horizontal direction, (2) vertical ground strains are generally smaller than shear strains, and (3) the amplification of vertically propagating shear wave is much higher than vertically propagating compression waves.

Ground failure includes different types of ground instability such as faulting, landslides, liquefaction, lateral spreading, settlement, flotation, tectonic uplift and subsidence. These types
of ground deformations are called permanent ground deformations (PGD). Characteristics of permanent ground deformations and its effect of culverts are extremely complex and must be dealt with on a case-by-case basis.

Wang, (1993) developed closed form and analytical solutions for the determination of ovaling/ racking deformations and the corresponding internal forces on tunnel structures based on a theory by Peck et al. (1972). This procedure is also applicable to culvert structures. Hashash et al. (2001) and Anderson et al. (2008) presented a good summary for the procedure developed by Wang (1993) as is described in the next paragraphs.

Racking deformations are defined as the differential sideways movements between the top and bottom elevations of rectangular structures, shown as “Δs” in Figure 2.37. The internal forces can be obtained by imposing the racking deformation on the structure using a simple frame analysis.

![Diagram of racking deformations](image)

Figure 2.37: Racking deformations of rectangular structure (After Anderson et al, 2008)

The general procedure developed by Wang (1993) for determining Δs and the corresponding structural internal forces, taking into account the soil-structure interaction effects, are presented below:
**Step 1:** Estimate the free-field ground strains $\gamma_{\text{max}}$ (at the structure elevation) caused by the vertically propagating shear waves of the design earthquakes using the following formula:

$$\gamma_{\text{max}} = \frac{V_s}{C_{se}}$$  \hspace{1cm} (2.39)

where $\gamma_{\text{max}}$ is the maximum free field shearing strain at the elevation of the culvert, $V_s$ is the shear wave peak particle velocity at the culvert elevation, and $C_{se}$ is the effective shear wave velocity of the medium surrounding the culvert. For shallow culverts, the maximum free field shear strain can be obtained using the earthquake induced shear strain which can be estimated using the following equation:

$$\gamma_{\text{max}} = \frac{\tau_{\text{max}}}{G_m}$$  \hspace{1cm} (2.40)

where $\tau_{\text{max}}$ is the maximum earthquake induced shear stress;

$$\tau_{\text{max}} = (PGA / g) \sigma_v R_d$$  \hspace{1cm} (2.41)

where $\sigma_v$ is the total overburden pressure at the invert of the culvert, and equal to:

$$\sigma_v = \gamma_t (H + D)$$  \hspace{1cm} (2.42)

where $\gamma_t$ is the total unit weight, $H$ is the height of soil cover, and $D$ is the height of culvert. $R_d$ is depth-dependent stress reduction factor and equal to:

$$R_d = 1.0 - 0.00233 z$$ \hspace{1cm} \text{For } z < 30 \text{ feet} \hspace{1cm} (2.43)$$

$$R_d = 1.174 - 0.00814 z$$ \hspace{1cm} \text{For } 30 < z < 75 \text{ feet} \hspace{1cm} (2.44)$$

where $z$ is the depth to mid point of culvert. $G_m$ is effective strain compatible shear modulus of the soil.
Alternatively, shear strains can be estimated by conducting a SHAKE analysis. Once the maximum shear strain is obtained, the differential free-field relative displacements $\Delta_{\text{free-field}}$ corresponding to the top and the bottom elevations of the rectangular/box structure can be determined by:

$$\Delta_{\text{free-field}} = H \gamma_{\text{max}}$$  \hspace{1cm} (2.45)

where $H$ is the height of the culvert.

**Step 2:** Determine the racking stiffness $K_s$ of the structure from a simple structural frame analysis. The racking stiffness can be obtained by applying a unit lateral force at the roof level, while the base of the structure is restrained against translation, but with the joints free to rotate. The structural racking stiffness is defined as the ratio of the applied force to the resulting lateral displacement.

**Step 3:** Derive the flexibility ratio $F_{\text{rec}}$ of the rectangular structure using the following equation:

$$F_{\text{rec}} = \left( \frac{G_m}{K_s} \right) \left( \frac{L}{H} \right)$$  \hspace{1cm} (2.46)

where $L$ is the width of the culvert.

The flexibility ratio is a measure of the relative racking stiffness of the surrounding soil to the racking stiffness of the structure. The derivation of $F_{\text{rec}}$ is schematically shown in Figure 2.38. For one-barrel frames (one box), the flexibility ratio can be obtained without computer analysis. Wang (1993) suggest using the following relations:

$$F_{\text{rec}} = \frac{G_m}{24} \left( \frac{H^2 L}{EI_w} + \frac{HL^2}{EI_R} \right)$$  \hspace{1cm} (2.47)

where $E$ is plane strain elastic modulus of the frame, $I_R$ is the roof slab moment of inertia, and $I_W$ is the side wall moment of inertia.
**Step 4:** Determine the racking ratio $R_{rec}$ for the structure using Figure 2.39 or the following expression:

$$R_{rec} = \frac{2F_{rec}}{1 + F_{rec}} \quad (2.48)$$

The racking ratio is defined as the ratio of actual racking deformation of the structure to the free-field racking deformation in the ground. As expressed by Wang (1993):

$$R_{rec} = \frac{\Delta_{structure}}{\Delta_{free-field}} = \frac{\gamma_{structure}}{\gamma_{free-field}} \quad (2.49)$$

where $\gamma$ is angular distortion, and $\Delta$ is the lateral racking deformation.

Finite element analyses by Wang (1993) showed that the flexibility ratio has the most significant effect on the distortion of the structure as follows:

- $F \rightarrow 0.0$, the structure is rigid, so it will not rack and it will take all the load.
- $F < 1.0$, the structure is considered stiff relative to the soil and will therefore deform less.
- $F = 1.0$, the structure and soil have equal stiffness, so the structure will undergo approximately free-field distortions.
- $F > 1.0$, the racking distortion of the structure is amplified relative to the free field. This is not due to dynamic amplification but because the soil now has a cavity, providing lower shear stiffness than free field.
- $F \rightarrow \infty$, the structure has no stiffness, so it will undergo deformations identical to the soil.

**Step 5:** Determine the racking deformation of the structure $\Delta_s$ using the following relationship:

$$\Delta_s = R_{rec} \cdot \Delta_{free-field} \quad (2.50)$$
Figure 2.38: Relative stiffness of soil versus rectangular frame (After Anderson et al, 2008)

Figure 2.39: Racking ratio between structure and free field (After Anderson et al, 2008)
**Step 6:** The seismic demand in terms of internal forces as well as material strains can be calculated by imposing $\Delta_s$ upon the structure in a frame analysis as illustrated in Figure 2.40.

![Figure 2.40: Simple frame analysis for racking deformations (After Anderson et al, 2008)](image)

Penzien (2000) presented an analytical procedure for evaluating the racking of rectangular structures during seismic events. This procedure is for a homogenous isotropic soil medium subjected to uniform shear strain field. Penzien (2000) showed that the deformations of the structure depend on the relative stiffness or the flexibility ratio between the soil and structure. The relative stiffness is defined with the parameter $k_{str}/k_{soil}$, where $k_{str}$ is the stiffness of the structure and $k_{soil}$ is the stiffness of soil. $k_{str}$ is equal to the magnitude of shear stress applied to the perimeter of the structure that produces unit displacement of the structure; and $k_{soil}=G/H$, where $G$ is the shear modulus of the soil and $H$ is height of the structure. The ratio $R$ between the structure deformation $\Delta_{str}$ and the free field soil deformation $\Delta_{ff}$ can be obtained by:

$$R = \frac{\Delta_{str}}{\Delta_{ff}} = \frac{4(1-\nu_s)}{1+\alpha_s}$$  \hspace{1cm} (2.51)

in which

$$\alpha_s = (3-4\nu_s) \frac{k_{str}}{k_{soil}}$$  \hspace{1cm} (2.52)

where $\nu_s$ is the Poisson’s ratio of the soil.
Hou et al. (2006) presented a closed form solution for rectangular tunnels. This solution adds to the previous solutions the consideration of normal and shear stresses at the interface as well as the actual deformations of a rectangular opening. Complex variable theory and conformal mapping were used assuming a plane strain deep rectangular structure in a homogenous, isotropic and elastic medium. This solution can be used for pseudo-static analysis, where the seismic deformations of the soil and structure can be approximated by far-field shear stress or strain. A summary of this analytical solution as follows:

**Step 1:** Determine the internal dimensions of the opening, width $a$ and height $b$. Use $a > b$.

**Step 2:** Find the aspect ratio $\lambda = a/b$, and factor $k$ from:

$$\lambda = \frac{a}{b} = \frac{1 + \cos 2k\pi - \frac{1}{6} \sin^2 2k\pi - \frac{1}{20} \sin 2k\pi \sin 4k\pi}{1 - \cos 2k\pi - \frac{1}{6} \sin^2 2k\pi + \frac{1}{20} \sin 2k\pi \sin 4k\pi}$$

(2.53)

**Step 3:** Find the relative stiffness ratio $\Omega$.

$$\Omega = \frac{E_s I_s}{Gb^3}$$

(2.54)

where $G$ is the shear modulus of the soil. If the structure is much more rigid than the surrounding soil, $\Omega$ approaches infinity and the deformation of the structure approaches zero. If the structure is much more flexible than the soil, $\Omega$ approaches zero and the normalized deformation of the structure becomes equal to the deformation of a rectangular opening.

**Step 4:** Use structural analysis to find $\Delta_{ri}$ and $\Delta_{piz}$, the deformation of the structure due to shear and normal stresses distributions. For simple structures with equal stiffness $E_s I_s$ use:

$$\Delta_{ri} = \frac{(1 + \lambda)}{24E_s I_s} \lambda b^4$$

(2.55)

$$\Delta_{piz} = \frac{(1 + \lambda)}{60E_s I_s} b^4$$

(2.56)
Step 5: Find parameters $M$, $N$, and $L$, from Figure 2.41.

![Figure 2.41: Parameter $M$, $N$, and $L$ values (After Hou et al., 2006)](image)

Step 6: Find normalized structural distortions from:

$$\frac{\Delta_{sf}}{\Delta_{ff}} = \frac{GA_{sf}}{\tau_{ff} b} = \left(1 - v_s^2\right) \left[ N_{A_{p_2}} + \left( M_{A_{p_2}} + \Delta_{p_2} \right) L \right] \frac{G}{b}$$

(2.57)

where $v_s$ is the Poisson’s ratio of the soil.

Hou et al. (2006) stated that in the free field approach, the structure must accommodate the free field deformations without loss of integrity and this may not be correct. This is because for a structure that is more rigid than the soil, the structure will reduce the deformations from the surrounding soil. If the structure is more flexible than the soil, the linear distortions are larger than the free field deformations.

Bobet, et al. (2008) used the analytical solution by Hou et al. (2006) and proposed a procedure to incorporate the soil stiffness degradation through an iterative process, where the soil shear modulus is changed in each iteration based on the shear strain of the soil obtained in the
previous iteration. The process ends when the shear modulus used in the last iteration corresponds to the soil deformation.

Nishioka and Unjoh (2003) proposed a simplified method based on the shear deformation capacity. The shear deformation capacity studied using nonlinear finite element analyses of five types of standard boxes. In the evaluation method, the seismic performance is checked by the difference between the soil strain and the peak soil strain at the structure level. The results show that the boxes have enough ductility with respect to the shear deformations. It was noted also that as the thickness of the structure increases, the shear strains decrease.

Wood (2005) used the method proposed by Wang (1993) to analyze single and double barrel structures on soils and rocks. Wood (2005) compared the results obtained for the racking ratio and flexibility ratio with the simplified method proposed by Nishioka and Unjoh (2003) and the analytical method proposed by Penzien (2000). There was a good agreement between all the methods particularly for a flexibility ratio less than 2.0.

Amiri et al. (2008) used the analytical method proposed by Penzien (2000) and performed a numerical FEM parametric study to assess the effect of structure geometry and embedment depth (h/H), where h is the height from ground surface to the mid-side of the structure height and H is the height of the structure. Results show that the racking deformations are insensitive to the structure geometry, while for the embedment depth, the racking deformation is independent of the depth, for burial ratios h/H > 2. For stiffer structures than the soil and h/H < 2, the racking distortion decreases as the burial depth decreases, while for flexible structure with h/H < 2, the racking increases as the depth decreases.

Katona (2010a, 2010b) presented a step by step methodology for analyzing and evaluating the structural integrity of a buried structure under the combined influence of static and seismic loading. The analysis was a combination between the racking procedure proposed by Wang (1993) and the CANDE-2007 software developed by the author. Using CANDE-2007, a plane strain finite element program, the soil structure problem can be characterized. In the static design, loads are applied with a series of incremental load steps. Then, the seismic loading is simulated by specifying quasistatic displacements at the peripheral boundaries of the soil envelope, to produce shear racking distortion equivalent to the maximum free field seismic shear strain from the design earthquake. The procedure applies to any culvert shape, size, material, and
the design can be assessed either by working stress (WS) or load reduction factor design (LRFD).

2.3.3 Earthquake loads for box culverts in Codes and Standards

2.3.3.1 AASHTO

Even though the American Association of State and Highway Transportation Officials (AASHTO 1994, 1996, 2002, 2005, 2007) has a complete section on the seismic design, nothing is related to the calculation of earthquake loads on box culverts. The only reference in that section is associated with the seismic design of bridges. Article 3.6.2.2 is the only section in AASHTO that mentions dynamic loads for culverts and buried structures. The dynamic loads here are related to the vibrations that may happen due to trucks, centrifugal or braking forces. In this case, a dynamic load allowance IM is added as a percentage to the static loads, as follows:

\[ IM = 33 \left( 1.0 - 4.1 \times 10^{-4} D_E \right) \geq 0\% \]  

(2.58)

where \( D_E \) is the minimum depth of earth cover above the structure (mm)

2.3.3.2 CHBDC

The Canadian Highway Bridge Design Code (CHBDC, 2000, 2006) introduced the force effect due to the earthquake loads on concrete box culverts by multiplying the force effects due to self weight \( W \), and earth load \( W_P \) by the vertical acceleration ratio, \( A_V \). The vertical component of the earthquake acceleration ratio, \( A_V \), can be taken as two-thirds the horizontal ground acceleration ratio, \( A_H \). The vertical component of earthquake acceleration is going to increase the soil density from \( \gamma \) to \( \gamma (1+A_V) \). Amplification of ground motion should be considered where a significant thickness of soil overlies rock or firm ground.

Other than concrete structures, the CHBDC presented the focus on the additional axial force due to earthquake loading on soil-metal structures, \( T_E \). Analysis showed that horizontal acceleration has little effect on axial force, which is the basis for design of soil-metal structures. The axial force due to earthquake \( T_E \) should be calculated as:

\[ T_E = T_D \cdot A_V \]  

(2.59)
where \( T_D \) is the thrust caused by dead loads. While for metal box structures the additional moment due to the effect of earthquake, \( M_E \), should be calculated as:

\[
M_E = M_D \cdot A_v
\]

(2.60)

where \( M_D \) is the moment caused by dead loads.

The dynamic analysis indicates that additional significant moments are induced by the horizontal component of the earthquake. The vertical component is less important, but there is no simple way to incorporate it into design formulas. Axial load due to seismic loads is not currently considered in the design of box structures (Commentary of CHBDC, 2006).

### 2.4 SUMMARY

Box culverts are a critically important life line structure. They are considered to form an important part of the transportation infrastructure. As there are large number of these structures in service and their failures can have potentially significant economic implications, more research should be conducted on their behaviour and issues related to their design under static and seismic loadings.

Arching of soil around box culvert shows the complexity of soil-culvert interaction, since the behaviour is also a function of the deflection of their structural members, and that has not been studied extensively. The soil-culvert interaction factors obtained from various field, experimental and numerical research projects indicates two things: the distribution of the soil pressure is not equal to the theoretical soil pressure; and the shape of the soil pressure assumes a parabolic shape, not a uniform shape (especially on the top slab of the box culvert). Taking this into consideration, the values proposed in AASHTO and CHBDC consider only the overall vertical soil pressure attracted to box culverts, and not the actual pressure distribution on different structural elements. The soil-culvert interaction factors do not take into consideration the relative stiffness between the culvert and the surrounding soil, and therefore can not accurately determine the degree of soil pressure attracted to the culvert in several cases. Some research has tried to develop fitting relations for the soil-culvert interaction factors, based on experimental or numerical studies, and these relations when compared to the results of field measurements, show either a good fit with the measured data or very conservative results. The
soil culvert system of buried box culverts is difficult to study theoretically or analytically, however, it is much easier to study it experimentally using a geotechnical centrifuge, or numerically using the finite element/difference method. Numerical methods can provide a convenient way to study the redistribution of soil pressure around buried box culverts and that can help understand the mechanism of soil-culvert interaction.

Even though many box culverts have recently suffered huge damage during earthquake loading, studies to investigate their responses is very limited. AASHTO does not show a procedure to determine the effect of earthquake loads on box culvert, while the only thing mentioned in CHBDC is to use the vertical acceleration component to determine the moment and axial forces. The only method proposed to account for earthquake effects is the racking method. This method is based on determining the racking deformation based on the free-field deformations, which can be used in finite element analysis to determine the internal forces of the culvert. In this method the racking deformation is applied at the top corner of the culvert, and that might not be the case always, since the effect of the relative stiffness between the culvert and the surrounding soil has an effect on the relative ratio. Therefore, the racking method needs more investigation to be adopted for seismic analysis.

Finally, soil-culvert interaction is still a subject that needs more investigation to develop reliable methods that can be used in both static and seismic analysis and design for box culverts. In this thesis, the topic of soil-culvert interaction under static and seismic loads is investigated extensively both experimentally and numerically, to understand the interaction behaviour and create design guidelines for box culverts.
CHAPTER THREE
CENTRIFUGE MODELLING TESTS

3.1 INTRODUCTION

Centrifuge modelling is an important tool that can be used to investigate the arching effects due to the interaction between the soil and the buried structure. A centrifuge test program was conducted to study the arching effects of a box culvert buried in cohesionless soil under static and seismic loading conditions. Four primary centrifuge model tests were performed involving two box culverts with two different wall thicknesses and embedded in sand with two different relative densities. The tests were conducted using the geotechnical centrifuge at the Rensselaer Polytechnic Institute (RPI) in Troy, New York, USA.

The static centrifuge model tests were focussed on the response of the box culverts to the self-weight of sand and additional surface pressures, such as strip and rectangular foundations. The loadings were applied by accelerating the centrifuge gradually from 1g to the required g level to the increase in static loading, while continuously monitoring all sensors. The seismic loading was applied as earthquake shakings at the required g level. The data generated and collected from all sensors during the centrifuge tests were interpreted to understand the main features of soil-culvert interaction. The experimental data were also used to calibrate advanced numerical models using the computer code FLAC (Itasca, 2005) and to validate the results and the abilities of the models to represent the soil structure interaction (SSI) problems.

This chapter will describe in detail the centrifuge model test facility, material tested, model sand preparation, test instrumentation and calibration, and provide a description of the overall test plan and procedures followed for the centrifuge model tests. It will also describe some challenges faced during the centrifuge tests and how they were solved. In addition, a clear explanation of methods for simulating earthquake shakings using one dimensional shaker is also provided.
3.2 CENTRIFUGE MODELLING

The RPI centrifuge used in this research was manufactured by Acutronic in France and is shown in Figure 3.1. The centrifuge nominal radius (i.e. the distance between the center of payload and the centrifuge axis) is 2.7 m, while the distance between the platform and the centrifuge axis is 3 m. The maximum centrifugal rotational speed is 265 r.p.m. The capacity of this centrifuge is 150 g-tons and its acceleration range is between 1g to 200g. The data acquisition system has the ability to provide 128 channels of data sampling during testing (www.nees.rpi.edu).

![Figure 3.1: Centrifuge facility at RPI, Troy, NY, USA (www.nees.rpi.edu)](image)

The mechanical principle of the centrifuge modeling is well described by Taylor (1995) and Wood (2004). If a mass \( m \) is rotating at a constant radius \( r \) about an axis with steady speed \( v \) as shown in Figure 3.2, then it would experience a constant radial acceleration \( \frac{v^2}{r} \) or \( r\omega^2 \) (\( \omega \) is the circular velocity). Owing to this acceleration, the mass would be subjected to a centrifugal
force equal to $mra^2$ directed toward the axis. This acceleration can be normalized by the gravity acceleration, $g$, and from that it can be said that this mass is subjected to acceleration equal to $Ng$, where $N = ra^2/g$.

![Figure 3.2: The basic concept of the centrifuge modeling (after Wood, 2004)](image)

The key advantage of using the geotechnical centrifuge to model soil problems is the ability of the centrifuge to correctly simulate the linear increase in the effective stresses with depth through the soil profile. Centrifuge testing involves a scaled model based on the enhanced gravity field of the geotechnical centrifuge, hence allowing the simulation of the actual field behaviour of the soil structure system with a small physical model. Centrifuge modeling is based on a number of scaling laws, illustrated in the example in Figure 3.3 and presented in Table 3.1. Figure 3.3 shows a comparison between the vertical stress values in a model and prototype scales by applying model scaling laws. At the prototype scale, if a material has a density $\rho$, and height $H$, then the vertical stress at the prototype scale $\sigma_{vp}$ is equal to $\rho gH$. At the model scale, if the same material is subjected to an acceleration equal to $Ng$, its height after applying the scaling law for the length ($l/N$), will be $H/N$. The vertical stress at the model scale $\sigma_{vm}$ is then equal to $\rho Ng \frac{H}{N} = \rho gH$. Thus the stresses at the model and prototype scale are identical, i.e. $\sigma_{vp} = \sigma_{vm}$.

The same concept applies to strain, which leads to a 1:1 scale for the soil stress-strain curve mobilized in the model, which will be identical to that of the prototype.
Figure 3.3: Stresses in centrifuge modeling

Table 3.1: Key Scaling relationships for the centrifuge modeling (Taylor, 1995)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Field</th>
<th>Centrifuge model (Ng)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>( l )</td>
<td>( l/N )</td>
</tr>
<tr>
<td>Area</td>
<td>( A )</td>
<td>( A/N^2 )</td>
</tr>
<tr>
<td>Mass</td>
<td>( m )</td>
<td>( m/N^3 )</td>
</tr>
<tr>
<td>Density</td>
<td>( \rho )</td>
<td>( \rho )</td>
</tr>
<tr>
<td>Stress</td>
<td>( \sigma )</td>
<td>( \sigma )</td>
</tr>
<tr>
<td>Strain</td>
<td>( \varepsilon )</td>
<td>( \varepsilon )</td>
</tr>
<tr>
<td>Force</td>
<td>( F )</td>
<td>( F/N^2 )</td>
</tr>
<tr>
<td>Moment</td>
<td>( M )</td>
<td>( M/N^3 )</td>
</tr>
<tr>
<td>Time (Consolidation)</td>
<td>( T_v )</td>
<td>( T_v/N^2 )</td>
</tr>
<tr>
<td>Dynamic time</td>
<td>( t )</td>
<td>( t/N )</td>
</tr>
<tr>
<td>Displacement</td>
<td>( x )</td>
<td>( x/N )</td>
</tr>
<tr>
<td>Velocity</td>
<td>( v )</td>
<td>( v )</td>
</tr>
<tr>
<td>Frequency</td>
<td>( f )</td>
<td>( Nf )</td>
</tr>
<tr>
<td>Acceleration</td>
<td>( a )</td>
<td>( Na )</td>
</tr>
<tr>
<td>Energy</td>
<td>( E )</td>
<td>( E/N^3 )</td>
</tr>
</tbody>
</table>
The basics of centrifuge modeling can be applied to seismic SSI by considering the scaling laws related to the time dependent events. As shown in Table 3.1, dynamic time in the prototype is N times the time period of the centrifuge model and this means that the dynamic time is N times faster in the model. This results in frequency and accelerations higher by a factor of N in the model, while the velocity stays the same.

Even though scaling laws are helpful for creating models in the centrifuge, some concern may still arise regarding soil particle size effects. Considering the scaling laws, sand particle diameter at high g levels may be in the range of gravel or boulders. Some researchers have developed simple guidelines for the critical ratio between the major dimensions of the model and the average grain diameter. This approach was adopted by Ovesen (1979), who investigated the performance of circular foundations on sand by using different sized models at different g levels. His data showed that the ratio of the foundation diameter to the grain size should be less than 15. It may also be necessary to consider the ratio of particle size to the shear band width as suggested by Tatsuoka et al (1991).

### 3.3 MATERIAL TESTED

#### 3.3.1 Nevada Sand

The sand used in the centrifuge tests was the 120-Nevada Sand, which is fine, uniform, and clean. The sieve analysis to determine the sand grain size distribution was performed according to the ASTM D-421 and the result is shown in Figure 3.4. The particle size was in the range of 0.075 to 0.550 mm. According to the Unified Soil Classification System (USCS), the sand is classified as poorly graded sand (SP). The maximum and minimum unit weight and densities of the sand were determined according to the ASTM D-4253 and D-4254, respectively. The maximum unit weight $\gamma_{\text{max}}$ was 16.77 kN/m$^3$ and the minimum unit weight $\gamma_{\text{min}}$ was 14.85 kN/m$^3$. The general properties of 120-Nevada Sand are shown in Table 3.2.
Table 3.2: General properties of 120-Nevada Sand

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_s$</td>
<td>2.67</td>
</tr>
<tr>
<td>Effective diameter</td>
<td>$D_{10} = 0.080 \text{ mm}$</td>
</tr>
<tr>
<td></td>
<td>$D_{30} = 0.115 \text{ mm}$</td>
</tr>
<tr>
<td></td>
<td>$D_{50} = 0.145 \text{ mm}$</td>
</tr>
<tr>
<td></td>
<td>$D_{60} = 0.160 \text{ mm}$</td>
</tr>
<tr>
<td>Uniformity coefficient</td>
<td>$C_u = 2.00$</td>
</tr>
<tr>
<td>Curvature coefficient</td>
<td>$C_c = 8.98$</td>
</tr>
<tr>
<td>Maximum void ratio $e_{\text{max}}$</td>
<td>0.764</td>
</tr>
<tr>
<td>Minimum void ratio $e_{\text{min}}$</td>
<td>0.562</td>
</tr>
<tr>
<td>Maximum density $\rho_{\text{max}}$</td>
<td>1.71 g/cm$^3$</td>
</tr>
<tr>
<td>Minimum density $\rho_{\text{min}}$</td>
<td>1.51 g/cm$^3$</td>
</tr>
<tr>
<td>*Critical State Friction angle $\phi_{cs}$</td>
<td>$40^\circ$</td>
</tr>
<tr>
<td>*Peak Friction angle $\phi_p$</td>
<td>$45^\circ$</td>
</tr>
<tr>
<td>*Peak Dilation angle $\psi_p$</td>
<td>$5^\circ$</td>
</tr>
</tbody>
</table>

* Direct shear tests performed by RPI.

Figure 3.4: Grain size distribution of 120-Nevada Sand
3.3.2 Box Culvert Model

In engineering practice, box culverts are generally constructed from short sections of reinforced concrete, which are joined together to form the final desired cross-section. Due to the difficulty involved in constructing model culverts from a micro-concrete aggregate with appropriate reinforcement in the laboratory, aluminum material is used instead (e.g. Stone and Newson, 2002). Before choosing the required size of the aluminum tubes to be used as model culverts in the centrifuge tests, thorough research into engineering practice was performed to explore the typical sizes of box culverts. There are several factors controlling the choice of the culvert aluminum tube that can be used in the centrifuge tests. These factors are: the culvert sizes and wall thickness available, the range of possible relative stiffnesses, the height of the centrifuge box and the g-level that the centrifuge test will run at. These factors are discussed below.

1. ASTM C1433-10 (2010) presents many design tables for each precast reinforced concrete box culvert size. The square box culvert size ranges from (0.91 m × 0.91 m) to (3.66 m × 3.66 m) and the thickness of the slabs and walls range from 0.10 m to 0.30 m.

2. The size of square box culverts in practice ranges from (0.3 m × 0.3 m) to (1.2 m × 1.2 m) for small culverts and from (1.5 m × 1.5 m) to (6.0 m × 6.0 m) for large culverts (Wembley Cement, 2003; Carr Concrete; E-Rete, Banagher Concrete, 2001; Humes, 2006 and 2007; Rocla, n.d).

3. The range of thickness of culvert walls and slabs s is from (0.067 m) to (0.200 m) for small culverts and from (0.200 m) to (0.600 m) for large culverts (Wembley Cement, 2003; Carr Concrete; E-Rete, Banagher Concrete, 2001; Humes, 2006 and 2007; Rocla, n.d).

4. The height of the centrifuge box is 35.56 cm. As 2.54 cm at least has to be used for the surface instruments, the total height of the sand model is 33.02 cm. A significant amount of sand is required above and below the culvert, and therefore 12.7 cm of sand above and below the culvert is adopted.

5. At 60g, the prototype thickness of the sand above and below the culvert will be 7.62 m.

Based on these factors, a hollow square box aluminum tube that has an external dimension of 7.62 cm was chosen with two different thicknesses to examine the effect of change in culvert thickness/flexibility on the SSI behaviour. The two thicknesses are 6.35 mm, termed here as a ‘thick’ culvert and 3.18 mm, which is termed as a ‘thin’ culvert, as shown in Figure 3.5.
At 60g, these dimensions will be equivalent to 4.572 m external dimension and the thickness values are 0.533 m and 0.267 m. Table 3.3 summarizes the properties of the material used in designing the box culvert models.

The centrifuge model material can be different from that of the prototype provided the correct scaling law is used to ensure proper modeling of structural deflection. Different materials have been used to model the behaviour of reinforced concrete box culverts, such as mild steel (Stone et al., 1991) and aluminum (Stone and Newson, 2002). The scaling law for stiffness is given by:

\[
E_m I_m = \frac{E_p I_p}{N^4}
\]

(3.1)

where, \(E\) = Young’s modulus of the material, \(I\) = second moment of area per unit length of the material and \(N\) = scaling factor. The subscripts ‘\(m\)’ and ‘\(p\)’ refer to model and prototype, respectively. The relationship between the model and prototype wall thickness can therefore be evaluated using:

\[
t_p = N t_m a^{1/3}
\]

(3.2)

where \(t\) = wall thickness, and \(a = E_m/E_p\).

Figure 3.6 shows the relative stiffness values derived using Equation 2.54 between the box culvert and the surrounding soil. Two different elastic moduli (10 and 30 MPa) of the soil representing two relative densities were used to compare the relative stiffness for different EI and culvert thickness values. The model thin and thick culvert cases are also plotted on the figures to show the range of relative stiffnesses that can be achieved using these two sections. The graphs show that the two chosen sections have up to one order of magnitude difference in relative stiffness in terms of EI values.

<table>
<thead>
<tr>
<th>Part</th>
<th>Block Shape</th>
<th>Thickness (mm)</th>
<th>Length (mm)</th>
<th>Width (mm)</th>
<th>Height (mm)</th>
<th>Volume (mm³)</th>
<th>Density (kg/m³)</th>
<th>Mass (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thick</td>
<td>Hollow</td>
<td>6.35</td>
<td>369</td>
<td>76.2</td>
<td>76.2</td>
<td>627414.9</td>
<td>2700.7</td>
<td>1662.4</td>
</tr>
<tr>
<td>Thin</td>
<td>Hollow</td>
<td>3.18</td>
<td>369</td>
<td>76.2</td>
<td>76.2</td>
<td>335262.6</td>
<td>2700.7</td>
<td>907.1</td>
</tr>
</tbody>
</table>
Figure 3.5: Box Culvert Model
Figure 3.6: Relative stiffness between the box culvert and the surrounding soil.

(a) Relative stiffness as a function of culvert EI

(b) Relative stiffness as a function of culvert thickness
3.3.3 Foundation Model:

Two types of foundation model were used in the centrifuge tests: a strip foundation and a rectangular foundation as shown in Figure 3.7 and both were made of aluminum. As shown in Figure 3.6, the strip foundation consisted of two identical solid aluminum bars, while the rectangular foundation was made of one piece. Table 3.4 shows the properties of the rectangular foundation model and one piece of the strip foundation model at model scale. Each piece of the strip foundation was chosen to give a 50 kPa bearing pressure at its base at 60g with a total of 100 kPa for both pieces, while the rectangular foundation gives 100 kPa by itself. The final dimensions of both foundations were restricted by the size of the centrifuge box, to separate the free field effects from the structure field effect (ideally 5 to 7 times the width of the foundation).

<table>
<thead>
<tr>
<th>Part</th>
<th>Block Shape</th>
<th>Length (mm)</th>
<th>Width (mm)</th>
<th>Height (mm)</th>
<th>Volume (mm$^3$)</th>
<th>Density (kg/m$^3$)</th>
<th>Mass (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td>Solid</td>
<td>76.2</td>
<td>31.8</td>
<td>63.5</td>
<td>153139.1</td>
<td>2700.7</td>
<td>0.42</td>
</tr>
<tr>
<td>Strip</td>
<td>Solid</td>
<td>367.2</td>
<td>76.2</td>
<td>31.8</td>
<td>892582.4</td>
<td>2700.7</td>
<td>2.42</td>
</tr>
</tbody>
</table>
Figure 3.7: Foundation models. (a) Strip foundation, (b) Rectangular foundation
3.4 TEST INSTRUMENTATION

The instrumentation used in the model tests consisted of accelerometers, LVDTs, strain gauges and tactile pressure sensors. These instruments were suitably positioned around the box culvert, inside the sand and on the sand surface to characterize the soil, culvert and foundation responses under the effect of static and seismic loading. The locations of these instrumentations were kept consistent in all the tests, so that results can be compared easily.

3.4.1 Accelerometers:

Sixteen accelerometers were placed inside the sand, around the culvert and the foundation to measure the acceleration time history in the free field and in the structure field, to compare the effect of soil structure interaction. They were also placed in different positions around the centrifuge box and the shaker. The accelerometers are manufactured by PCB Piezotronics (Model 353 B17) and shown in Figure 3.8. Each accelerometer was calibrated before installation and the range of accelerations that can be recorded is ± 500g. The weight of accelerometer is 1.7g, and has a height of 15 mm. The sensitivity of the accelerometers is ± 10% (10 mV/g). (www.pcb.com)
3.4.2 LVDTs:

Three Linear Variable Differential Transducers (LVDT), were used to measure the settlement of the surface of the sand due to the static loading and seismic shakings. These LVDTs were mounted on cross bars in three different locations. Figure 3.9 shows a cross section of an LVDT and shows its main components. The LVDTs used (shown in Figure 3.10) were manufactured by Schaevitz (model Schaevitz 500 MHR). The LVDT has a weight of 17 g and a length of 83.8 mm, while the core has a weight of 1.6 g and a length of 50.8 mm. The outer and inner diameters of the LVDT are 9.5 mm and 3.18 mm, respectively. The range of this LVDT is 25.4 mm. All three LVDTs were calibrated by moving the rod inside the body at equally spaced displacement increments and recording the output voltage. A linear relationship between the resulting displacement and the output voltage was established. The slope of this relationship was the calibration factor that was applied to change the output voltage readings of the LVDTs to settlement. Another LVDT was placed in the shaker to measure the displacement time history of the shaker during the seismic phase of the test (www.meas-spec.com).

![LVDT cross section](image1)

Figure 3.9: LVDT cross section

![LVDT](image2)

Figure 3.10: LVDT
3.4.3 Strain Gauges

Strain gauges were used to measure the change in strain values on the outside and inside faces of the box culvert. These strain measurements were then converted to bending moments and axial forces using calibration factors. The type of strain gauges used was manufactured by Vishay Precision Group as shown in Fig 3.11. These strain gauges are typically used for static and dynamic stress analysis. It has a resistance of 350 Ohms, which indicates that the readings of such strain gauges are going to be more accurate than 120 Ohms.

![Figure 3.11: Strain gauges](image)

3.4.4 Tactile Pressure Sensors

Two tactile pressure sensors (Tekscan model number 5101) were used for each culvert model to measure the soil pressure around the culvert. One of the pressure sensors was placed on the culvert top surface to measure the vertical pressure of the soil and the adjacent side to measure the horizontal pressure. The tactile pressure sensor used (shown in Figure 3.12) can measure up to 150 psi (1034 kPa) and can record up to 225 frames per second (i.e. it is only suitable for static pressure). For seismic loading, at least 2000 fps is required to be able to
capture the change in stress time history. It should be noted that this sensor has the ability to measure the normal stress only (i.e. it does not account for shear stresses). The square sensing area is $111.8 \times 111.8$ mm and has 1936 sensels (44 columns crossed by 44 rows). To protect sensor from damage due to friction with sand particles, it was laminated with a plastic sheet and then covered with Teflon sheet, using vacuum grease to protect it as shown in Figure 3.13.
Figure 3.13: (a) Protected Tactile pressure sensor, (a) with lamination, (b) with Teflon sheets
3.5 INSTRUMENT CALIBRATION

3.5.1 Accelerometer Calibration

The accelerometers were already calibrated by the RPI centrifuge team following their standard procedures. The calibration device is a Gilchrist Technology Model 4000. This device has a small vibrating table, where the accelerometer has to be connected and vertically shaken as shown in Figure 3.14. The DAQ hardware was used to connect the accelerometer wires to the computer. Using special readings and formulas, the calibration factors were calculated. The resulting calibration factors were used in the DAQ system of the centrifuge to convert the readings directly to accelerations.

Figure 3.14: Accelerometer calibration: (a) Calibration device, (b) DAQ hardware
3.5.2 LVDT Calibration

The LVDTs were calibrated using the device shown in Figure 3.15. By mounting the LVDT as shown in Figure 3.15 and connecting it to the DAQ, the core was moved inside the body of the LVDT for known displacements and the output reading from the DAQ was recorded. This procedure was repeated for different displacement readings. A relationship between the displacements and output readings was established as a linear function. The slope of this line represents the calibration factor for the LVDT, which was used in the centrifuge DAQ system.

Figure 3.15: LVDT calibration device
3.5.3 Strain Gauge Calibration

The strain gauge calibration factors provided by the vendor were used along with the strain gauge resistance to convert the voltage readings recorded by the DAQ to strain readings. To convert the strain readings to bending moment and axial force, two different methods were used. The first method involved applying known loads to the culvert and recording the strain reading from each strain gauge. The beam theory was used to evaluate the bending moment at the locations of the strain gauges. In the second method, the finite element method was employed to analyze the culvert subjected to the different load configurations used in the first method.

During testing, the culvert model would be surrounded by sand from all four directions and the sand pressure may not be uniform on all sides. However, the pressure was assumed to be uniform in the calibration procedures described above and there was a lack of pressure confinement and joint fixity. To alleviate these problems, elastic structural analysis was adopted to calculate the stress \( \sigma \) at a point in the structure using the following equation:

\[
\sigma = E \varepsilon = \frac{F}{A} \pm \frac{M y}{I}
\]  

(3.3)

where \( F \) = axial force, \( M \) = bending moment, \( A \) = area of cross section, \( I \) = moment of inertia, \( y \) = distance from neutral axis, \( E \) = modulus of elasticity, and \( \varepsilon \) = strain.

As the strain was measured on both surfaces of each side of the culvert, the external strain on the outside surface (\( \varepsilon_e \)) and the internal strain on the inside surface (\( \varepsilon_i \)) were used in Equation (3.3) to derive the calibration factors for bending moment and axial force. The calibration factors are derived from the following relations:

\[
M = \frac{E I (\varepsilon_i - \varepsilon_e)}{2y}
\]  

(3.4)

\[
F = \frac{E A (\varepsilon_i + \varepsilon_e)}{2}
\]  

(3.5)

From these relations, the calibration factors for bending moment and axial force are:
\[ CF\ (BM) = \frac{E.I}{2.y} \]  

\[ CF\ (AF) = \frac{E.A}{2} \]  

(3.6) 

(3.7)

These equations were used to calculate the calibration factors for both thick and thin culverts accounting for the effect of culvert thickness on the values of \( A, I, \) and \( y. \)

### 3.5.4 Tactile Pressure Sensor Calibration

The process of calibration for a tactile pressure sensor consisted of three stages. These processes are saturation, equilibration and calibration. Saturation pressure is the point at which the sensor output no longer varies with applied pressure, i.e., the calculated digital output becomes incorrect because increases in pressure on a saturated sensel yield no increase in pressure on those cells (I-Scan, 2006). To avoid this problem during testing, the sensors were saturated to a pressure higher than the expected pressure for one hour. The instrument shown in Figure 3.16 is the instrument used to saturate the pressure sensors by inserting them from the side and applying air pressure on both faces of the sensor.

The processes of equilibration and calibration are performed together. Equilibration means that all the sensor cell points read the same pressure value at the same time. The calibration for the sensors has two procedures: linear calibration, and a 2-Points Power Law calibration. In the linear calibration, a known load is applied to the sensor and the I-Scan software will interpolate between the zero and the known calibration load. In the 2-Point Power calibration, two different known loads are applied to the sensor and the I-Scan software will perform a power law interpolation between zero and the two known calibration loads. The 2-Points Power Law calibration is the method used in the calibration for the sensors used in the centrifuge tests, since this method is preferred if the measured loads vary during the testing. This method gives more accurate results for the pressure of the soil on the culvert.
Figure 3.16: Tekscan calibration machine
The Tekscan calibration machine shown in Figure 3.16 is used to calibrate the tactile pressure sensors. In this machine, the sensor is inserted between two steel plates and subjected to the calibration pressure using air pressure. During centrifuge testing, however, the sensors experience two conditions that are different than the calibration machine: the test is run at 60g level and not at 1g; and the test pressures would occur between two different materials (i.e. sand and aluminum) and not two steel plates. Therefore, the calibration was conducted inside the centrifuge box. This involved covering the sensing area of the pressure sensor was by double-sided tape and placing it at the base of the centrifuge box as shown in Figure 3.17. The sensors were covered by 127 mm of sand at 80% relative density (same amount of sand applied during the centrifuge tests). The centrifuge box was then placed on the centrifuge platform and the tactile handles were connected to the centrifuge DAQ.

To define the required loads and the g levels to achieve equilibration and calibration of the sensors, expected soil pressures were calculated considering both the 90 % and 50 % relative densities and foundation pressures from the foundations on the culvert during the centrifuge tests. Since only two load values are required for calibration, the minimum and maximum calculated pressures were used as calibration points. Two extra g levels were selected to add more equilibration pressure points. As the calculated pressures for the top and side sensors were different, the centrifuge was run twice, one for the vertical stresses on the top sensors and the other for the horizontal stresses on the side sensors as shown in Figure 3.18. After running the centrifuge and at the selected g levels, the equilibration pressures were applied first to make sure that all the cells in the sensors measured the same pressure, and then the calibration loads were applied. The I-Scan software was used in the calibration process to establish the sensor actual contact area between the sand and base of the centrifuge box. By multiplying the calculated pressures with the actual contact area, the calibration and equilibration loads were determined and applied to the sensors. At the end, each data set was saved in a separate file, one for equilibration and the other for calibration. During the real tests, all data are recorded continuously from 1g to 60 g and then by applying the equilibration and calibration files, the actual soil pressures on the culvert top and side would be obtained.
Figure 3.17: Tactile pressure sensors placed at the base of the centrifuge box
Figure 3.18: Tactile pressure sensor calibration
3.6  CHALLENGES DURING CENTRIFUGE TESTING:

3.6.1  LVDT Cross Bar:

Two types of cross bars were used to hold the LVDTs during the centrifuge tests (Figure 3.19) to allow some space for the middle LVDT when the foundation is placed. The orange ended cross bars (used for LVDTs 1 and 3) had to be attached on both sides of the centrifuge box and hence there would be no space for the foundations. The hollow cross bar (used for LVDT 2) is flexible and could be adjusted to the height required. Both cross bars caused no problems when used during the static stage of the centrifuge tests. However, when the earthquake acceleration time history was applied, the hollow cross bar had a clear effect on the shape of the displacement time history recorded from the LVDT at the sand surface.

Figure 3.19: LVDTs cross bars
To illustrate this effect, Figure 3.20 shows the difference between the displacement time history recorded using the LVDTs connected on both types of cross bars. In this research, these results do not affect any of the conclusions, since it is used only to record how much residual settlement happened during shaking. If the shape of the displacement time history is important for any other purpose or research, then it is important to take this effect into consideration. The change in the shape of the displacement time history is due to the vibrations that occur between the sides of the hollow cross bar.

Figure 3.20: Displacement results. (a) Static, (b) Seismic (model scale)
3.6.2 Strain Gauge Installation:

The strain gauges were installed on the outside (top and side) surfaces following the standard procedures proposed by the manufacturer. This procedure involves: cleaning the surface using Carbird 400 grit paper then with conditioner and neutralizer; marking the location of the strain gauge on the surface; and attaching the strain gauge to a PCT-2M tape and placing it precisely in the marked position. Once the strain gauge location was achieved, the tape stuck to the strain gauge was removed from one side only and fast acting glue was used to attach the strain gauge in position, and finger pressure was applied to the strain gauge for one minute. After installing the strain gauges, the lead wires were soldered to them. The strain gauges resistance was checked before and after wiring using Ohm meter to ensure that the strain gauges were functioning as expected.

Installing the strain gauges on the inside (top and side) surface of the culvert model paused a challenge. Since the side of the culvert model was 76.2 mm, it was not possible to insert the strain gauge by hand as described above. Therefore, different strain gauges were used inside the culvert. They had the lead wires already soldered to them and were glued using slow acting (24 hours) adhesive. Although the use of different types of glue to install the strain gauges on the outside and the inside surfaces may have some influence on the readings, it is anticipated that this effect is relatively small. A special technique was used to apply the pressure to the strain gauges, which involved an inflated test ball as shown in Figure 3.21. This ball has a diameter of 50.8 mm and can be inflated using air pressure up to 275.79 kPa, which is greater than that required to install the strain gauges (about 140 kPa or less).

Some tools were specially developed to insert the strain gauges and position them in their correct location, including: strain gauge transfer plate and light weight plastic rubbing plate as shown in Figure 3.22. Sketches of these two tools are shown in Figure 3.22. The dimensions shown in Figure 3.23 depend on the thickness of the culvert model tube. The detailed procedure for installing the strain gauges inside the culvert model is presented in Appendix (A).
Figure 3.21: Method of applying the pressure using test ball
Figure 3.22: Strain gauge installation tools inside the culvert model
Figure 3.23: Sketch of strain gauge installation tools:

(a) Strain Gauge Transfer Plate, (b) Light Weight Plastic Rubbing Plate
3.6.2.1 Strain gauge locations

Figure 3.24 and 3.25 show the location of each strain gauge on both thick and thin culvert models, respectively. The numerical values in both directions (x,y) for each strain gauge are shown in Tables 3.5 and 3.6.

Table 3.5: Strain gauge locations on the thick culvert

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<th>Thick Culvert (Side Wall)</th>
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Table 3.6: Strain gauge locations on the thin culvert

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Figure 3.24: Schematic diagram for strain gauge locations for the thick culvert (Not to scale)

Figure 3.25: Schematic diagram for strain gauge locations for the thin culvert (Not to scale)
3.7 EARTHQUAKE SIMULATION

The Rensselaer Polytechnic Institute (RPI) centrifuge facility utilizes a servo-hydraulically controlled system (Figure 3.26) to produce one dimensional horizontal shaking. This shaker can run different shaking types, periodic or random. The maximum payload weight is 250 kg and the maximum centrifugal acceleration is 100g, with a frequency range between 0 and 350 Hz. This shaker produces earthquake signals as an applied input voltage signal and simulates the earthquake shakings by applying forces to the model base.

Figure 3.26: One Dimensional Box Shaker
3.7.1 Earthquake Calibration

The shaker has a displacement controlled actuator and does not directly accept acceleration time histories of earthquake records as input. The earthquake records are scaled to voltage and sent to the shaker as an electric signal. The response of the shaker to this signal will be in the form of displacement that can be measured using an LVDT. To ensure that the voltage signal sent to the shaker matches well the earthquake record, an accelerometer was connected to the shaker to monitor and record the acceleration time history and then compares it to the original earthquake record as shown in Figure 3.27. Additionally, the displacement recorded by the LVDT was compared to the displacement time history calculated by double integrating the acceleration time history recorded from the shaker as shown in Figure 3.27. It is also important to compare the acceleration time history recorded from the shaker and that of the base of the centrifuge box, which is assumed to be the earthquake record applied to the test model base.

To ensure that all earthquake records used in the centrifuge tests have the best match in terms of amplitude and frequency, a dummy test was conducted before starting the actual tests. In the dummy test, an equivalent model was built and subjected to all earthquake records with different amplitudes. Since the centrifuge box used in all tests was rigid, the effect of centrifuge box boundaries was investigated during the dummy test. Several accelerometers were distributed inside the sand at the same elevation (in the middle height of the sand model) and at different distances from the boundary to examine the boundary effects. Another series of accelerometers were used outside the centrifuge box to measure the acceleration time history of the shaker and the base of the centrifuge box. In addition, a Duxseal (Duct seal) material was used on the other side of the box to assess any changes in the acceleration readings and to investigate if this material is able to absorb reflections from the boundary.

The recorded acceleration time histories from all accelerometers within the soil bed were checked and compared. It was ascertained that there was no effect of the boundary on the results. The acceleration time history recorded from the accelerometers that were positioned at the same elevation and at different distances from the box side gave almost the same results. It should be noted that the closest accelerometer to the box side was placed at 3 mm from the box side, and that the box walls were only 7 mm.
Figure 3.27: Comparison between the input and output motion for Western Canada Earthquake from dummy test (a) Acceleration time history, (b) Displacement time history, and (c) Shaker acceleration versus Base acceleration. (in prototype scale)
3.7.2 Earthquakes used in the Centrifuge Tests

A process of filtering with trial and error was applied on the dummy model until a good match was found between the filtered records and the response at the base of the centrifuge box. The results of the dummy test were used to establish a relationship between the voltage values and the amplitudes recorded to establish the values of voltage that give the required level of shakings. This relationship varies from an earthquake to another, and is based on the capacity of the shaker.

Three different earthquakes with different amplitudes and frequencies were adopted for use in the centrifuge tests. The three earthquakes were: the Kobe earthquake (North-East component of the Port Island down hole array -79 m record), the Western Canada, and the Vancouver Cascadia Subduction (Artificial records corresponding to 2% probability of occurrence in 50 years). The predominant frequencies of these earthquakes are 1.453, 0.647, and 0.464 Hz, respectively. The original records as well as the final shapes of the filtered earthquakes that were used in all tests (prototype scale) are shown in Figures 3.28 to 3.33. Earthquake input motions in the entire centrifuge tests are summarized in Table 3.7.
Figure 3.28: Time histories and frequency content for the actual Kobe earthquake

Figure 3.29: Time histories and frequency content for the modeled Kobe earthquake
Figure 3.30: Time histories and frequency content for the actual Western Canada earthquake

Figure 3.31: Time histories and frequency content for the modeled Western Canada earthquake
Figure 3.32: Time histories and frequency content for the actual Cascadia earthquake

Figure 3.33: Time histories and frequency content for the modeled Cascadia earthquake
Table 3.7: Earthquake input motion in all centrifuge tests

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<th>Predominant Frequency (Hz)</th>
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<tr>
<td></td>
<td>KEQH</td>
<td>0.27</td>
<td>1.453</td>
<td>16.17</td>
<td>87.18</td>
</tr>
</tbody>
</table>

*VC = Vancouver Cascadia, WC = Western Canada, KEQ = Kobe Earthquake, L = Low, M = Medium, H = High
3.8 MODEL CONFIGURATION AND PREPARATION

3.8.1 Model Container

The centrifuge model testing requires using a box to contain the sand bed and the box culvert. This container will impose boundary conditions that do not exist in the prototype situation. It is desirable to have a container that allows the physical model to perform in a similar manner to the real life situation either under static loading or seismic shaking. The centrifuge box containers available at RPI include the laminar box and rigid box types. For the static loading, both boxes will give the same conditions; for the seismic shakings, the laminar box is more suited to simulate seismic lateral deformations of the soil model. However, it has to be designed in accordance of the earthquake signal, which is not practical for the current testing program. Hence, a rigid box container was used for all tests performed. To avoid the problem of wave reflections due to the rigid sides of the box, a series of accelerometers were used in the dummy models (as explained in the previous sections) to explore this boundary effect.

Figure 3.34 shows the large centrifuge rigid box container that was used in all tests, which has the dimensions: 876.3 mm (L) × 368.3 mm (W) × 355.6 mm (H). This box made of aluminum and all of its sides have a waffle shape. The actual thicknesses of the sides of the aluminum box are 7 mm, with a web thickness of 13 mm. The web squares are 50 mm by 50 mm. One of the long sides was replaced by a Plexiglas window for easy viewing of the model contents. The thickness of this Plexiglas is 50 mm. This container has the ability to fit over the shaking table to apply the earthquake shakings.
3.8.2 Model Sand Preparation

Nevada Sand was used in all centrifuge tests. The target relative densities for the tests were 50% and 90%. Different methods can be used to achieve these relative densities, such as tamping and raining (air pluviation) techniques. Eid (1987) reviewed the development of the raining technique and stated that this method not only provides homogenous samples with desired density, but also closely simulates the fabric of in situ soils formed by sedimentation. Factors affecting the density of rained sand, such as the falling height, the deposition intensity, diffuser sieve size and the shutter-hole pattern were studied by Rad and Tummy (1987). They concluded that the shutter porosity or the deposition intensity has the strongest effect on the relative density. The falling height, the diffuser sieves size and the shutter-hole pattern has less pronounced effects. To achieve the required relative densities, the centrifuge box height was divided into layers of 25.4 mm thickness. As the maximum and minimum densities for Nevada
Sand are known and the volume of each sand layer can be calculated, the amount of sand required for each layer was determined for both 50% and 90% relative densities. To make sure that the required density can be obtained, a process of calibration based on trial and error has been performed using the pocket shown in Figure 3.34 before placing the sand in the actual tests. Based on this calibration, it was concluded that placing the sand into layers with the raining technique (air pluviation) alone can achieve the 50% relative density, while for 90% relative density, each sand layer was additionally tamped after air pluviation as shown in Figure 3.35.

Figure 3.35: Model sand preparation using raining and tamping techniques
3.8.3 Model Configurations

Figure 3.36 shows the general configurations of the test models. The four main test configurations and associated test cases are presented in Tables 3.9 and 3.10. Tests 1 and 2 are for the thick culvert with sand density 90% and 50 %, respectively. Each test included three cases: Case A with sand only; Case B involved a surface strip foundation with 50 kPa positioned right over the box culvert; and Case C same as Case B but the strip foundation pressure was 100 kPa. Tests 3 and 4 are for the thin culvert and the sand density was 50% and 90%, respectively. In each of these tests, there were four cases. Case A: with sand only, Case B and Case C: with a surface strip foundation positioned right over the box culvert location, with strip foundations pressure 50 and 100 kPa, respectively. Case D: involved a surface rectangular foundation centrally positioned right over the box culvert location, applying a pressure of 100 kPa. The total height of the sand model is 330.2 mm, simulating 19.812 m at prototype scale of 60g.

All models were instrumented to measure the free field and structure field acceleration time history by placing accelerometers inside the sand and around the culvert and the foundation structures. As shown in Figure 3.36, the accelerometers Ac2, Ac3, Ac4, Ac5 and Ac6 were used to measure the horizontal acceleration time history inside the sand body along a vertical section away from the structure (box culvert). This was assumed to be the Free Field (FF) condition. On the other hand, the accelerometers Ac7, Ac8, Ac9, Ac12, and Ac13 were used to measure the horizontal acceleration time history along a vertical section in the area of the box culvert, and therefore, were defined as the Structure Field (SF) condition.

Accelerometer Ac1 was placed near the sand surface to measure the vertical acceleration time history, while Ac10 and Ac11 were used to measure the vertical acceleration time history on both sides of the culvert to investigate any rocking that might happen due to shaking. Three accelerometers Ac14, Ac15, and Ac16 were attached to the foundations to record both the horizontal and the vertical acceleration of the structure during earthquake excitation.

In each of the tests performed, a set of accelerometers were placed outside the centrifuge box and on the shaker as shown in Figure 3.36, to ensure that the actual and required accelerations at the base of the centrifuge box model were the same. Figure 3.36 and Table 3.8 show the accelerometers locations and the distances between them. Additionally, the box culvert was instrumented with strain gauges and tactile pressure sensors to measure the bending moment
and the contact pressure between the sand and the culvert. LVDTs were placed on the surface to measure the settlement of the sand surface and the foundation.

Figure 3.36: Schematic diagram for centrifuge tests
(a) No foundation, (b) With foundation. (All units are in mm)
Table 3.8: Location of accelerometers placed within the sand

<table>
<thead>
<tr>
<th>Accelerometer</th>
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<th>y (mm)</th>
<th>z (mm)</th>
</tr>
</thead>
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<tr>
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<td>184</td>
<td>317.5</td>
</tr>
<tr>
<td>Ac2</td>
<td>219</td>
<td>184</td>
<td>317.5</td>
</tr>
<tr>
<td>Ac3</td>
<td>219</td>
<td>184</td>
<td>279.4</td>
</tr>
<tr>
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<td>219</td>
<td>184</td>
<td>203.2</td>
</tr>
<tr>
<td>Ac5</td>
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</tr>
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<td>Ac7</td>
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<td>Ac8</td>
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</tr>
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<td>200</td>
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</tr>
<tr>
<td>Ac12</td>
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</tr>
<tr>
<td>Ac13</td>
<td>438</td>
<td>184</td>
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Table 3.9: Centrifuge Tests:

<table>
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</tr>
<tr>
<td>Test 2 (T2)</td>
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</tr>
<tr>
<td>Test 3 (T3)</td>
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<td>50</td>
</tr>
<tr>
<td>Test 4 (T4)</td>
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<td>90</td>
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Table 3.10: Centrifuge Test Cases:

<table>
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<th>Test No.</th>
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<tr>
<td>T1A, T2A, T3A, T4A</td>
<td>Sand surface alone</td>
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<tr>
<td>T1B, T2B, T3B, T4B</td>
<td>Strip foundation on surface (50 kPa)</td>
</tr>
<tr>
<td>T1C, T2C, T3C, T4C</td>
<td>Strip foundation on surface (100 kPa)</td>
</tr>
<tr>
<td>T3D, T4D</td>
<td>Rectangular foundation on surface (100 kPa)</td>
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</table>
3.8.4 Sequence for Building the Model

Building the models started with adopting the procedure to achieve the required relative density for each model as shown above. The models were built after placing the empty centrifuge box on the one-dimensional shaker on the centrifuge platform to avoid any disturbance in the sand model that might cause any changes in the density of the sand (Figure 3.37). Once the first five sand layers were placed, the tactile pressure sensors were attached to the box culvert model using double sided tape as shown in Figure 3.38. The culvert model was then placed and levelled in its position. A thin layer of vacuum grease was used to prevent leaking the sand between the box culvert and the Plexiglas side. After placing the sand layers on both sides of the culvert model and on top for the required height, the accelerometers were placed at their required levels inside the sand as shown in Figure 3.39; five cross bars were used to strengthen the box and to protect the Plexiglas from breaking at high g levels. Also, thin layers of blue painted sand were placed between sand layers just behind the Plexiglas to monitor any movement that might occur during testing as shown in Figure 3.40. For Case A, the three LVDTs were connected to the cross bars and placed on the sand surface, while for Cases B, C, and D, the middle LVDT was mounted on top of the foundation as shown in Figure 3.41. All sensors used in the model were checked and connected to the data acquisition system. The centrifuge was then accelerated incrementally and held at the following acceleration levels, 10g, 20g, 30g, 40g, 50g and 60g to check stability of the sensor readings. The earthquake signals were sent to the shaker at 60g. Data from all sensors were recorded continuously during the test. The entire procedure was repeated for all test cases by de-accelerating the centrifuge to 1g and stopping it to make any changes related to each case and re-running the centrifuge again.
Figure 3.37: Placing the centrifuge box on the shaker and the centrifuge platform

Figure 3.38: Attaching and levelling the tactile pressure sensors to the box culvert model
Figure 3.39: Sequence of building the centrifuge model
Figure 3.40: Photo of completed model

Figure 3.41: Final model with all the cases
3.9 SUMMARY

A detailed description of the centrifuge modeling conducted to study the soil-culvert interaction behaviour under static and seismic loading was presented. All the materials used along with instrumentations and their calibration were explained in detail.

Specific challenges related to application of some instrumentation are highlighted along with the solutions proposed to resolve them. A newly developed procedure for installing the strain gauges inside the box culvert and the parts manufactured for this purpose were presented. Model configuration and preparation of the model container was described. The sand preparation method used to achieve the required sand density was also explained. The earthquake shakings were applied using a one dimensional box shaker, and the earthquake signals were calibrated and filtered in dummy tests, to obtain the appropriate signals that the shaker can produce.

Although there were certain challenges and difficulties that have been described in this chapter, scaled physical experimental modeling still represents an effective way to investigate soil-culvert interaction.
CHAPTER FOUR
CENTRIFUGE TEST RESULTS AND INTERPRETATIONS

4.1 INTRODUCTION

This chapter presents the results and the analysis of the data collected from a series of centrifuge tests performed as described in Chapter 3. The volume of data produced from these tests is considerable and therefore, it is not feasible to present all data obtained in this chapter. Hence selections of the data are included in this chapter and further data is summarized in Appendices. The data collected was in the form of settlement using the LVDTs, strain data using strain gauges, pressure data using tactile pressure sensors and acceleration data using accelerometers. Data processing and filtering were applied to all acceleration records to remove electronic drift from the records. Filtering is necessary to obtain the correct shape of the velocity and displacement time histories that starts and ends with zero values.

The Free Field (FF) is a well known term for the zones where the soil movement is not influenced by the presence of a structure either placed on the surface or buried inside it. The zones where there is a structure are termed as “Structural Field” (SF). To investigate the soil culvert interaction under the effects of static and seismic loading, analyses were performed to examine different aspects that involve both factors. For static loading the analyses, included the static bending moment and soil pressures. Under the effect of earthquake loading, analyses were conducted to explore the effect of the Free Field versus Structural Field responses in terms of dynamic soil properties, rocking of the box culvert and foundations, racking of the box culvert, kinematic soil culvert interaction, lateral movement of the foundations, as well as the envelope seismic bending moment.

Comparisons were also performed to show the effect of the box culvert thickness, soil density, and surface foundations. This chapter also compares the observed results from the centrifuge tests with estimates based on theoretical and/or empirical relationships. All results are presented at prototype scale unless otherwise noted.
4.2 MODEL SETTLEMENTS

Settlements of the soil surface and the foundations were measured using LVDT1, LVDT2 and LVDT3 supported by cross bars and extending downward to pads placed either on the sand surface or the foundation surface. In case A (sand only) of all four tests, three LVDTs were used to measure the soil surface settlements, while in cases B, C, and D, LVDT2 was used to measure the settlement of the foundations. Measurements of the LVDTs were recorded during increasing the accelerations of the centrifuge “spin up” from 1g to 60g and then during the shaking. The largest settlement measured during each case occurred through the spinning from 1g to 60g, while the residual settlement measured from the displacement time history recorded during each shaking was generally small. The “spin up” settlement curves from 1g to 60g for the Free and Structure Fields for all tests are presented in Figures 4.1 and 4.2, as a function of the spin up time at model scale. Each relation shows a number of steps, and each step represents 10g increase in the acceleration during the spin up. A large amount of settlement data during shaking was recorded and these show typical shapes and results. Therefore, only the results for the displacement time history of shakings in Test 1, for cases A and C are shown in Figures 4.3 and 4.4 at prototype scale and the reminder are presented in Appendix (B).

The surface settlement of the 90% relative density tests (Tests 1 and 4) was less than the settlement of the 50% relative density tests (Tests 2 and 3); this is expected since the looser sand will tend to settle more. The results show that the rate of settlement decreases as the centrifuge spins up, and the settlement was large at the beginning and decreases approaching 60g. This indicates that the sand densified as the model acceleration field increased from 1g to 60g creating higher self-weight stresses. This is more apparent in the 50% relative density models. The target starting relative densities in the four tests were 50% and 90% respectively and the settlements were seen to change in relative density during each test. The 90% relative density tests increased up to 100% by the end of the tests, while the 50% relative density models increased to 66% and 68% for Tests 2 and 3 respectively. This was due to the cumulative settlement during all of the test cases. In Figure 4.1(d), LVDT2 in case A of Test 2 shows strange behaviour after 40g and that was observed several times from other LVDTs at high g levels for unknown reason.

For each test, all test cases were run consecutively in the same model, and this caused an increase in the relative density for each case. Due to the fact that the difference between the
maximum and minimum densities of the sand was not large, the densification for each case did not cause a large change in the relative density between test cases.

The residual settlement measured after each shaking period defined as the difference between settlements before and after shaking was found to be a function of the peak ground acceleration (PGA) of the shaking. As PGA increases, the residual settlement increases. The results presented in Figures 4.3 and 4.4 show the difference between the displacement time histories for the shakings in Cases A and C. For Case A, the LVDTs were placed on the soil surface, and the results were similar, while in Case C, the LVDT2 was placed on the foundation, and the settlements were generally higher than the other LVDT measurements due to the inertial interaction effect of the foundation.
Figure 4.1: Settlement recorded from 1g to 60g for Tests 1 and 2 (Model Scale): (a) Test 1 Case A, (b) Test 1 Case B, (c) Test 1 Case C, (d) Test 2 Case A, (e) Test 2 Case B, (f) Test 2 Case C.
Figure 4.2: Settlement recorded from 1g to 60g for Tests 3 and 4 (Model Scale): (a) Test 3 Case A, (b) Test 3 Case B, (c) Test 3 Case C, (d) Test 3 Case D, (e) Test 4 Case A, (f) Test 4 Case B, (g) Test 4 Case C, (h) Test 4 Case D.
Figure 4.3: Settlement recorded at 60g for Test 1 Case A (Prototype Scale): Western Canada earthquake (a) low, and (b) medium, Vancouver Cascadian earthquake (c) low and (d) medium, Kobe Earthquake (e) low, (f) medium and (g) high
4.3 GROUND MOTION PARAMETERS

Ground motion parameters include the peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), frequency content, and response spectra. The measured data were only in the form of acceleration time histories, using accelerometers distributed inside and outside the centrifuge box as explained in Chapter 3. Velocity and displacement time histories were derived by integrating the acceleration time history. All the acceleration time histories collected were filtered to get the correct shape of velocity and
displacement time histories. A large amount of data were collected, and only typical results, an example of the acceleration, velocity, and displacement time histories and their frequency content and response spectra are presented below. The PGA, PGV, PGD profiles for all data are then reported and lastly, comparisons between the PGA, PGV, and PGD around the box culvert are illustrated.

4.3.1 Acceleration, Velocity, and Displacement Time Histories and their Frequency Content and Response Spectrum

MATLAB code was developed to filter the acceleration time histories and calculate the correct shape of velocity and displacement time histories. This code produces the frequency content in the form of frequency amplitude FA versus the frequency of the record, and also the response spectrum at 5% damping in the form of spectral value versus the period. The code provided the shape of acceleration, velocity, and displacement time histories and their corresponding frequency content and response spectrum shown in Figure 4.5. In Figure 4.5, the first column shows the acceleration, velocity, and displacement time histories, the second column shows the frequency content of the acceleration FA(A), velocity FA(V), and displacement FA(D), and in the third column shows the spectral acceleration Sa, spectral velocity Sv, and spectral displacement Sd. The results presented in Figure 4.5 are for KEQH event, which was considered to be the maximum event as an example, and the reminder of the results for the seven earthquake events are shown in Appendix (C). Due to the large amount of acceleration time histories recorded and due to the repeatability of the results, the resulting figures from Test 2 case C only are presented in Appendix (C). Only the results from Ac2, Ac7 and AcBaseH1 are shown. Ac2 and Ac7 represent the acceleration time history recorded at the surface for the Free Field (FF) and Structural Field (SF), respectively, while AcBaseH1 represents the acceleration time history recorded at the base of the model.

The acceleration time histories show that there is a difference between the FF and SF, and both acceleration histories are higher than that at the base. This shows that the PGA at the FF is higher than that of the SF by considerable amount and depends on the PGA at the base of the model. The difference between FF and SF is reduced for the velocity time history and reduces even more for the displacement time history.
The frequency spectrum of the FF acceleration shows more content in the high frequency range compared to the SF data, which is in the same range as that of the model base data. This effect reduces in the velocity spectrum and essentially disappears for the displacement frequency spectrum.

Figure 4.5: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake KEQH: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
The predominant frequencies of the acceleration time histories requested of the shaker were 1.453, 0.647 and 0.464, while the results show some variations at 1.437, 0.646, and 0.472 for the KEQ, WC, and VC acceleration time histories, respectively. This slight shift in frequency is attributed to the influence of the dynamic characteristics of the shaker. The predominant frequencies for the velocity frequency content are 0.999, 0.646, and 0.472 for the KEQ, WC and VC, respectively, while for the displacement frequency content these are 0.375, 0.646, and 0.352 for the KEQ, WC, and VC. These results illustrate that for WC, there is no change in the predominant frequency for all of the three frequency contents, while for VC the change happens only in the displacement frequency content. For the KEQ, the predominant frequency decreases from acceleration to velocity to displacement frequency contents.

The response spectra for Sa and Sv show that there is some difference between the FF and SF spectral values, especially for the low period range, and beyond this the results for the FF and SF are the same. The slight reduction of the SF relative to FF is attributed to the kinematic SSI effects of the relatively rigid culvert. For Sd, the FF and SF results are identical. In all three spectral plots, the amplitudes from the FF and SF motions are higher than those at the base, signifying amplification cross the profile.

### 4.3.2 Peak Ground Acceleration, Velocity, Displacement Profiles

Figure 4.6 presents the PGA, PGV, and PGD profiles from the three earthquakes for Test 1, Case A. The results shown in Figure 4.6 consist of nine sub figures; the PGA, PGV, and PGD are shown in rows 1, 2, and 3 respectively. Columns 1, 2 and 3 display the results for earthquakes WC, VC, and KEQ. The remainder of the results is presented in Appendix (D).

The black and blue lines in Figure 4.6 represent the Free Field (FF) and the Structure Field (SF) results, respectively. It is clear from the results shown that there is an amplification occurring in the PGA, PGV, and PGD as waves propagate from base to top and this intensifies close to the soil surface. It is also clear that the FF values are higher than the SF values and this is due to the presence of the box culvert. The results in Figure 4.6 show the difference between the FF and SF values for case A, where both of them have a sand surface. This difference appears to increase in cases C and D, because of the surface foundation. This will be explained in greater detail in Section 4.5.
Figure 4.6: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 1 Case A, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
4.3.3 PGA, PGV, PGD around Culvert

Four accelerometers were used around the box culvert to measure the acceleration time histories at different locations/direction. The accelerometers Ac9 and Ac12 were used to measure the horizontal acceleration time history above and below the culvert, respectively, while the accelerometers Ac10 and Ac11 were installed on the left and right sides of the box culvert in order to measure the vertical acceleration time history. The results presented in Figure 4.7 are for Test 1 case A, and the reminder of the results for all tests is presented in Appendix (E).

The results shown in Figure 4.7 include three rows for PGA, PGV, and PGD for the WC, VC, and KEQ events. In general, all amplitudes of PGA, PGV, and PGD across the soil profile increase as the input motion at the base of the model increases. The peak amplitudes of horizontal motion are much larger than the peak vertical values. It is noted that the peak horizontal values above the culvert are slightly higher than the values below it. The peak vertical values on the left and right of the culvert are almost the same.
(i) Results of horizontal accelerometers
(ii) Results of vertical accelerometers

Figure 4.7: PGA, PGV, and PGD resulted from the horizontal and vertical accelerometers on both sides of the box culvert for all the shakings in Test 1 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively, while Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
4.4 DYNAMIC SOIL PROPERTIES

Several researchers have studied the shear modulus and damping ratio parameters for different soil types under the effect of cyclic loading. (e.g. Kokusho, 1980; Rollins et al., 1998; Seed et al., 1986; and Ishibashi and Zhang, 1993). For clean sand, Hardin and Drenvich (1972b) found that the shear strain, effective stress level, and void ratio are the main factors affecting the shear modulus and damping ratio. Field studies have also been carried out to investigate stiffness nonlinearity based on earthquake motions (Chang et al. 1989; Zeghal et al. 1995). Zeghal et al. (1995) developed a method to establish the shear modulus and damping ratio from measured acceleration time history based on evaluating shear stress-strain cycles obtained at different depths in the soil for instrumented test sites.

Dynamic centrifuge test results can be used for investigating soil behaviour (Brennan, et al. 2005). The method of Zeghal et al. (1995) has been applied recently to the acceleration time history recorded in centrifuge models (Ellis et al. 1998; Zeghal et al. 1999; Brennan et al. 2005; Elgamal et al. 2005; Rayhani and El Naggar, 2008; Lanzano, et al. 2010). Ellis et al. (1998) derived shear modulus and damping ratio of very dense sand saturated with different pore fluids based on centrifuge experiments. Conti and Viggiani (2012) proposed another method to evaluate shear modulus and damping ratio from centrifuge tests. This method is based on the fit of the experimental transfer functions with an analytical expression of the amplification of a viscoelastic layer on a rigid base, and the corresponding shear strain level as a function of the particle velocity and shear wave velocity.

In this section, the data collected from accelerometers located in the Free Field are used to evaluate the shear modulus and damping ratio of Nevada sand. Shear stress and shear strain responses are back calculated using the recorded acceleration time history at different depths. The earthquake event chosen for this analysis was the Western Canada (WC) and at different amplitude levels (WCL and WCM), in order to investigate different ranges of shear strain. The resulting shear stress-strain data was used to determine the shear modulus and damping ratio of Nevada sand under two relative densities (50% and 90%). The analysis was applied to Case A from two tests (Test 1 and Test 2). The variations of shear modulus and damping ratio with shear strain obtained were compared to the well established Seed and Idriss (1970) ranges.
4.4.1 Evaluation of Shear Stress Strain History

The shear stress and shear strain time histories were evaluated using the method proposed by Zeghal et al. (1995). They used one-dimensional shear beam idealization to describe the site seismic lateral response assuming 1D vertically propagating shear waves. By applying integration, the shear stress at any level \( z \) and time \( t \) may be expressed as

\[
\tau(z,t) = \int_0^z \rho \ddot{u} \, dz
\]  

(4.1)

where \( \rho \) is the mass density in kg/m\(^3\) of the soil, \( \ddot{u} \) is the horizontal acceleration and \( \tau \) is the horizontal shear stress. Utilizing linear interpolation between downhole accelerations, the shear stress at level \( z_i \) can be calculated by

\[
\tau_i(t) = \sum_{k=1}^{i-1} \rho \cdot \frac{\dddot{u}_k + \dddot{u}_{k+1}}{2} \cdot \Delta z_k \quad i = 2, 3, \ldots
\]  

(4.2)

where \( i \) refers to level \( z_i \) and \( \Delta z_k \) is the spacing interval as shown in Figure 4.8. The shear stress estimated using Equation 4.2 is second-order accurate.

The shear strain calculations are based on the displacement time history derived by double integrating the acceleration time history. The shear strain obtained is then evaluated using the displacement data and the distance between the accelerometers. A second-order accurate shear strain \( \gamma_i \) at level \( z_i \) and time \( t \) can be expressed as

\[
\gamma_i(t) = \frac{1}{\Delta z_{i-1} + \Delta z_i} \left[ (u_{i+1} - u_i) \frac{\Delta z_{i-1}}{\Delta z_i} + (u_i - u_{i-1}) \frac{\Delta z_i}{\Delta z_{i-1}} \right]
\]  

(4.3)

where \( u_i = u(z_i,t) \) is the absolute displacement. The resulting shear stress and shear strain histories are directly related to the soil shear stiffness at each accelerometer level (Zeghal et al. 1995).
The acceleration time history from Ac2, Ac3, Ac4, Ac5, Ac6, and AcBaseH1 were used to produce the shear stress and shear strain at different depths. The results presented in Figure 4.9 show an example of the shear stress and shear strain time histories at four different depths (3.0, 7.6, 12.2, and 16.8 m). Using these time histories, the shear stress-strain hysteresis loops can be obtained for each depth. The results shown in Figure 4.9 are from Test 2, Case A as a result of shaking during the WCM event. It is found that the maximum shear stresses increase with depth, while the maximum shear strains are found to be smaller at the base and increase close to the surface. As noted, the slope of the loops decreases with increasing shear strains. This leads to a reduction in the shear modulus of the sand with cycling.
Figure 4.9: Shear stress and shear strain time history and their cyclic loops for Test 2 Case A at different depths under shaking WCM
4.4.1.1 Effect of data filtering

To get the correct shape of the displacement time history from the double integration of the recorded acceleration time history, a filter to the original record should be applied. Brennan et al. (2005) recommended filtering the data at high frequency to eliminate noise and at low frequency to eliminate drift errors during integration. The shear stresses and shear strains derived from the acceleration and displacement time histories are very sensitive to the range of filter applied to the original acceleration record. A large number of bandpass filters were applied and their effects on the shear stress-strain loops were noted. As an example of the unfiltered frequency content of the acceleration time history recorded for the WCL event at Ac2 in Test 1 Case A is shown in Figure 4.10. To show the effect of filtering on the shear stress-strain loops, two ranges of bandpass filter were applied to this frequency content; one filtered between 0.2 – 25 Hz, and the other between 0.3 – 1.5 Hz, as shown in Figure 4.11 and Figure 4.12. Using both filter ranges produced a correct shape of the displacement time history, but filtering in the 0.3 – 1.5 Hz range can be considered to be heavily over-filtered, because removing the high frequency components can remove the details of actual load paths (Brennan et al., 2005).

The shear stress-strain loops presented in Figures 4.13 and 4.14 use filtered and unfiltered accelerations to calculate the shear stresses for the bandpass filter of 0.3 – 1.5 Hz. The results show very smooth loops when using both filtered acceleration and filtered displacement to get the shear stress and shear strain. This may be challenged because the acceleration time history is over-filtered and causes reduction in the actual values of shear stresses and consequently a reduction in the shear modulus values. Also, the shape of the loops indicates a single frequency record, while earthquakes are multi-frequency records. When using the unfiltered acceleration with the filtered displacement for a bandpass filter of 0.3 – 1.5 Hz, the shape of the loops will be different and will compress and cause the area of the loop to be very small. The damping ratio is a function of the area of the loop, so this is going to produce small values of damping ratios for such shear strain levels. The shear stress-strain loops for the bandpass filter of 0.2 – 25 Hz is presented in Figures 4.15 and 4.16 for filtered and unfiltered accelerations, respectively. The filtered accelerations show very compressed loops while the unfiltered accelerations produce a very good shape of loops. The unfiltered accelerations will produce more accurate shear modulus as they use the actual shear stresses and more appropriate shapes for the loops, to get more reasonable damping ratios as will be explained later. Therefore, it is proposed to use the
unfiltered acceleration with the filtered displacement time histories to produce the shear stress and shear strain time histories to determine the shear stress-strain hysteresis loops. These loops are shown in Figures 4.16 to 4.19 for WCL and WCM of Tests 1 and 2. It is also recommended to use the minimum bandpass filter that can give the correct shape of the displacement time history, since it is the main purpose to filter the original acceleration time history records.

Figure 4.10: Unfiltered frequency of the acceleration at Ac2

Figure 4.11: Filtered frequency of the acceleration at Ac2 between 0.2 and 25 Hz
Figure 4.12: Filtered frequency of the acceleration at Ac2 between 0.3 and 1.5 Hz

Figure 4.13: Shear stress-strain cycles at different depths during WCL shaking in Test 1 Case A using filtered accelerations and filtered displacements between 0.3 and 1.5 Hz.
Figure 4.14: Shear stress-strain cycles at different depths during WCL shaking in Test 1 Case A using unfiltered accelerations and filtered displacements between 0.3 and 1.5 Hz.

Figure 4.15: Shear stress-strain cycles at different depths during WCL shaking in Test 1 Case A using filtered accelerations and filtered displacements between 0.2 and 25 Hz.
Figure 4.16: Shear stress-strain cycles at different depths during WCL shaking in Test 1 Case A using unfiltered accelerations and filtered displacements between 0.2 and 25 Hz.

Figure 4.17: Shear stress-strain cycles at different depths during WCM shaking in Test 1 Case A using unfiltered accelerations and filtered displacements between 0.2 and 25 Hz.
Figure 4.18: Shear stress-strain cycles at different depths during WCL shaking in Test 2 Case A using unfiltered accelerations and filtered displacements between 0.2 and 25 Hz.

Figure 4.19: Shear stress-strain cycles at different depths during WCM shaking in Test 2 Case A using unfiltered accelerations and filtered displacements between 0.2 and 25 Hz.
Shear stress-strain hysteresis shows that the shear strain level increased as the amplitude of the earthquake shaking increased from WCL to WCM events, and their areas and thus the soil damping increases. On the other hand, the shear modulus of the soil decreased as the strain level increased. These behaviours are consistent with the results reported in the literature using element tests for the same soil.

4.4.2 Calculation of Shear Modulus and Damping Ratio

The shear modulus and damping ratio of the sand were evaluated as a function of the shear strain based on the shear stress-strain cycles of the WCL and WCM earthquake shakings at different depths through the soil profile. These two shaking levels provide dynamic soil properties for different confinements and shear strain ranges from 0.02 and 0.4%.

The procedure for evaluating the shear modulus and damping ratio from shear stress-strain cycle is presented in Figure 4.20. For each stress-strain hysteresis shown in the Figures above, there is a large number of loops. Each loop was separated and the following procedure was applied using a MATLAB code developed to calculate the secant shear modulus and damping ratio. The multi-frequency loadings resulted in non-uniform loop shape, therefore the procedure suggested by Brennan et al. (2005) was followed. The secant shear modulus $G_{sec}$ calculated based on the difference between the maximum and minimum shear stresses and shear strains as follows:

$$G_{sec} = \frac{\Delta \tau}{\Delta \gamma} = \frac{\tau_2 - \tau_1}{\gamma_2 - \gamma_1}$$  \hspace{1cm} (4.4)

where $\tau_1$ and $\tau_2$ are the maximum and minimum shear stresses and $\gamma_1$ and $\gamma_2$ are the maximum and minimum shear strains, respectively. The damping ratio was calculated from the selected stress-strain hysteresis loop using the actual area of loop $A_{loop}$ as shown in the following relation:

$$\xi = \frac{W_D}{4\pi \cdot W_S} = \frac{1}{4\pi} \cdot \frac{A_{loop}}{A_{triangle}}$$  \hspace{1cm} (4.5)
where $\zeta$ is the damping ratio, $W_D$ is the dissipated energy and $W_S$ is the maximum strain energy. The area of the triangle can be obtained using:

$$A_{\text{triangle}} = \frac{1}{2} \times \frac{\Delta \tau}{2} \times \frac{\Delta \gamma}{2}$$  \hspace{1cm} (4.6)

By substituting the area of the triangle in Equation 4.6, the damping ratio can be determined by:

$$\xi = \frac{1}{2\pi} \cdot \frac{A_{\text{loop}}}{0.25 \times \Delta \tau \times \Delta \gamma}$$  \hspace{1cm} (4.7)

![Figure 4.20: Evaluation of shear modulus and damping ratio from stress-strain loop.](image)

It is noted that the stress strain loops were not symmetrical about the shear stress and shear strain axes. In some cases, most of the loop falls within the positive side and in other cases on the negative side of the axes. Therefore, using the above procedure was found to be more...
appropriate than other procedures that only use one side of the axis to determine the shear modulus and damping ratio. Other procedures can be applied for single frequency records, such as sine wave excitations where there loop shape is uniform and the values of shear stress and shear strain are identical on both sides of the axes.

### 4.4.3 Assessment of Shear Modulus

Figures 4.21 and 4.22 present the results of the normalized secant shear modulus $G_{sec}$ to the maximum shear modulus $G_{max}$ obtained from the backbone curves for Case A of Tests 1 and 2 under the earthquake shakings WCL and WCM. The results are plotted against the curves given by Seed and Idriss (1970) for dry fine sand. Each point in these Figures represents the result from a single hysteresis loop. The shear strain ranged between 0.005 and 0.2% for Test 1; while for Test 2 it ranged from 0.007 to 0.32%, which is attributed to the difference in sand relative density. It is expected the less dense sand experienced higher shear strain, especially close to the surface. The effect of the shaking amplitude is clearly shown from the results, since the magnitude of strain produced by the WCM is larger than that of the WCL. The centrifuge results are in good agreement with the standard curve results from element tests by Seed and Idriss (1970). The shear modulus ratio $(G/G_{max})$ decreased with shear strain amplitude and increased with depth, indicating increase with confining pressure.

### 4.4.4 Assessment of Damping Ratio

The damping ratios values were estimated from the shear stress-strain loops as a function of the shear strain during dynamic centrifuge tests. Figures 4.23 and 4.24 illustrate the results of the damping ratio degradation with shear strain for case A of Tests 1 and 2 under the earthquake shakings WCL and WCM. The damping ratios obtained were compared to the curves proposed by Seed and Idriss (1970) for dry fine sand and the agreement is very good. The damping ratios increased as the shear strain increased. The effect of soil density is also observed in terms of the shear strain as explained above. Finally, it is noted that adapting the filtering procedure recommended above resulted in significant reduction in the damping ratio scatter as noted by Brennan et al. (2005).
Figure 4.21: Shear modulus degradation of sand under WCL and WCM shakings in Test 1 Case A

Figure 4.22: Shear modulus degradation of sand under WCL and WCM shakings in Test 2 Case A
Figure 4.23: Damping degradation of sand under WCL and WCM shakings in Test 1 Case A

Figure 4.24: Damping degradation of sand under WCL and WCM shakings in Test 2 Case A
4.4.5 Evaluation of Shear Velocity

The maximum shear modulus $G_{\text{max}}$ calculated using the backbone curve of the shear stress-strain hysteresis loops as shown in Figure 4.20, and the results were compared to the following two empirical equations:

$$
G_{\text{max}} = \frac{3230 \left(2.97 - e\right)^2}{1 + e} \cdot \bar{\sigma}_{\rho}^{1/2}
$$

(4.8)

$$
G_{\text{max}} = 1000 \cdot K_{2,\text{max}} \cdot \left(\sigma_m^\prime\right)^{0.5}
$$

(4.9)

where $e$ is the voids ratio, $K_{2,\text{max}}$ is a factor that depends on the void ratio, and $\bar{\sigma}_o$ and $\sigma_m^\prime$ are the mean principle stress in kPa (Kramer, 1996). The shear wave velocity $V_s$ was calculated using the maximum shear modulus $G_{\text{max}}$ and the soil mass density $\rho$, i.e.:

$$
V_s = \sqrt{\frac{G_{\text{max}}}{\rho}}
$$

(4.10)

The results of $G_{\text{max}}$ and $V_s$ obtained are in good agreement with the results of the empirical equations. The values for $G_{\text{max}}$ and $V_s$ increased with depth due to the effect of overburden pressure. The average values of $G_{\text{max}}$ and $V_s$ through the soil profiles are 106 MPa and 250 m/s.

4.5 KINEMATIC SOIL CULVERT INTERACTION

Kinematic interaction of underground structures can alter the ground input motion parameters. Kinematic interaction results from the inability of the structural system to conform to the deformations of the free field motion. The kinematic interaction causes the motion of the structure to deviate from the free field motion. This concept generally applies when comparing the free field motion with the motion of soil underneath an embedded foundation or a structural system. If the structural system is buried in the soil profile at some depth from the ground surface, its interference with seismic motion (i.e. kinematic interaction) may cause the ground
input motion to a foundation above the buried structure to be different from that of the free motion (Kramer, 1996).

The kinematic effect of the box culverts on the ground input motion parameters has been investigated through evaluating the peak ground acceleration (PGA) in the Free Field (FF) and Structural Field (SF). The results from the Kobe earthquake (KEQM) for Test 1, Case C are presented in Figure 4.25 to demonstrate the effect of the box culvert on the values of peak ground acceleration (PGA) with depth. It is noted from Figure 4.25 that the PGA values of the SF decreased in comparison with the FF condition. The results also demonstrate that the de-amplification of the PGA values due to the presence of structure is a function of the earthquake input motion amplitude at the model base. As the PGA of the earthquake at the model base increased, the effect of structure is more pronounced in reducing the PGA at the soil surface. Same results were obtained from other tests, which confirm the kinematic effect due to the presence of the rigid culvert structure inside the sand body.

The FF sand response is consistent with the well established behaviour of soil profiles demonstrating an amplified response (i.e. higher PGA values) as the seismic waves propagate towards the surface (Kramer, 1996). Meanwhile, as the propagating seismic waves hit the relatively rigid structure of the box culvert, the amplitude of the seismic wave is decreased. Hence, the amplitude of the seismic wave in the SF was reduced relative to that of the FF, leading to the observed reduction in the PGA values.
4.5.1 Effect of Soil Density

To investigate the effect of soil density on the ground motion amplification, the PGA values at the base and soil surface of Tests 1 and 4 (Dr. = 90%) and Tests 2 and 3 (Dr. = 50%) are compared. Figures 4.26, 4.27 and 4.28 display the PGA values of the FF and SF conditions for KEQ, WC, and VC shakings. The SF PGA values are reduced compared to the PGA for the FF condition, and the percentage decrease was larger as the PGA of the base increased.

The reduction in PGA values for Cases A and C is similar for all ground shakings, but the reduction in Case C (with strip foundation) is larger than that for Case A (no foundation). This clearly demonstrates the effect of kinematic interaction of the strip foundation. In Case D, the kinematic interaction effects were less significant owing to the smaller size of the footing. The SF PGA values in Cases A and C were close for the 50 and 90% sand relative densities, while the FF PGA values for Dr = 50% were higher than for Dr = 90%. This is expected since the ground motion amplifies more in looser soil. For Case D, the SF PGA values for the 50% relative density were higher than PGA values for Dr = 90%. Even though FF PGA values for Dr = 50% are higher than for Dr = 90%, the opposite is noted for the SF PGA values, which confirms the combined kinematic interaction effects for the foundation and the box culvert.

Figure 4.25: Free Field and Structure Field PGA profiles for Test 1 Case C in KEQM shaking
Figure 4.26: Free Field (FF) and Structure Field (SF) PGAs due to KEQ: (a) Tests 1 and 2 - Case A; (b) Tests 1 and 2 - Case C, (c) Tests 3 and 4 - Case A, (d) Tests 3 and 4 - Case C, and (e) Tests 3 and 4 - Case D.
Figure 4.27: Free Field (FF) and Structure Field (SF) PGAs due to WC: (a) Tests 1 and 2 - Case A, (b) Tests 1 and 2 - Case C, (c) Tests 3 and 4 - Case A, (d) Tests 3 and 4 - Case C, and (e) Tests 3 and 4 - Case D.
Figure 4.28: Free Field (FF) and Structure Field (SF) PGAs due to VC: (a) Tests 1 and 2 - Case A, (b) Tests 1 and 2 - Case C, (c) Tests 3 and 4 - Case A, (d) Tests 3 and 4 - Case C, and (e) Tests 3 and 4 - Case D.
4.5.2 Effect of Culvert Thickness

To investigate the effect of culvert thickness on the ground motion amplification, the PGA values at the base and soil surface for Tests 1 and 2 (thick culvert) and Tests 3 and 4 (thin culvert) are compared. The FF and SF PGA values for KEQ, WC, and VC are presented in Figures 4.29, 4.30, and 4.31, respectively. The results show that the SF PGA values were smaller compared to the FF PGA values, and the percentage decrease in the SF PGA values was larger as the base input motion is increased. The SF PGA values in Cases A and C were similar for thick and thin culverts, which indicate that the effect of culvert thickness is small. This is attributed to the relative rigidity of the culvert for both thickness values.

Figure 4.29: Free Field (FF) and Structure Field (SF) PGAs due to KEQ: (a) Tests 1 and 4 - Case A, (b) Tests 1 and 4 - Case C, (c) Tests 2 and 3 - Case A, (d) Tests 2 and 3 - Case C.
Figure 4.30: Free Field and Structure Field PGAs due to WC (a) Tests 1 vs. 4 for Case A, (b) Tests 1 vs. 4 for Case C, (c) Tests 2 vs. 3 for Case A, (d) Tests 2 vs. 3 for Case C.
Figure 4.31: Free Field and Structure Field PGAs due to VC: (a) Tests 1 vs. 4 - Case A, (b) Tests 1 vs. 4 - Case C, (c) Tests 2 vs. 3 - Case A, (d) Tests 2 vs. 3 - Case C.

4.6 ROCKING OF STRUCTURES:

To investigate the rocking behaviour of the buried culvert and the surface foundation during earthquake shakings, readings of the four vertical accelerometers (Ac10 and Ac11 on either side of the culvert, Ac14 and Ac15 on the foundation top sides) were analyzed. It should be noted that the input motion applied at the box base was in the horizontal direction.
Rocking of structures was estimated using the vertical peak ground displacements \((PGD)\) derived from the vertical acceleration time histories. The rocking angle \(\psi_R\) was calculated as follows:

\[
\Delta PGD_{\text{Culvert}} = PGD_{10} - PGD_{11} \tag{4.11}
\]

\[
\Delta PGD_{\text{Foundation}} = PGD_{14} - PGD_{15} \tag{4.12}
\]

\[
\psi_R = \tan^{-1}\left(\frac{\Delta PGD}{B}\right) \tag{4.13}
\]

where \(B\) is either the width of the box culvert or foundation.

The resulting rocking angles for the box culvert are presented in Table 4.1 for the three Cases A, C, and D, and Table 4.2 shows the foundation rocking angle for the Cases C and D. The results presented in Tables 4.1 and 4.2 show small rocking angles. However it is hard to derive a clear conclusion, since in some cases the rocking angle increases as PGA increases, while in other cases the rocking angle decreases as the PGA increases. The rocking angles for the box culverts range from 0.0005 to 0.0917, and the rocking angle for the foundations range from 0.0007 to 0.1375. The smaller rocking angles for the culvert are expected due to the soil confinement of the culvert.
Table 4.1: Rocking angles of the box culvert

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Table 4.2: Rocking angles of the foundations

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<th>PGA Base (g)</th>
<th>Rocking Angle $\psi_R$°</th>
<th>PGA Base (g)</th>
<th>Rocking Angle $\psi_R$°</th>
<th>PGA Base (g)</th>
<th>Rocking Angle $\psi_R$°</th>
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<th>Test 4 Case D</th>
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<td>VCL</td>
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<td>0.0069</td>
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<tr>
<td>VCM</td>
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<td>0.0030</td>
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<tr>
<td>WCL</td>
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<td>0.0021</td>
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<tr>
<td>WCM</td>
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<td>KEQL</td>
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<td>0.0133</td>
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<tr>
<td>KEQM</td>
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<td>0.0717</td>
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<tr>
<td>KEQH</td>
<td>0.301</td>
<td>0.0568</td>
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</table>
4.7 LATERAL MOVEMENT OF FOUNDATIONS

The recorded horizontal acceleration time history from accelerometer Ac16 at the top of the foundation is used to investigate its lateral movement. The PGD is evaluated through the double integration of the acceleration time history and is used to investigate the lateral movement of foundations. The results can be used to explore the relation between the PGA values at the model base and both the PGA and PGD values at the top of the foundation.

Figure 4.32 presents the results of the PGA and its corresponding PGD values for Case C. Both PGA and PGD at the top of the foundation increased as the PGA at the model base increased. The PGA and PGD values from all tests are close, with the exception of the PGD of Test 1, which was lower than other PGD values. The PGD value from the VC shaking was highest.

Figure 4.33 compares the PGA and PGD values for the strip foundation in Case C and rectangular foundation in Case D of Tests 3 and 4. The results show that the PGA values obtained from the rectangular foundation were larger than those of the strip foundation, and the difference for Test 3 was larger than that for Test 4. This difference also varied with the amplitude of the input motion. For KEQ with PGA = 0.3g at the model base, the difference was up to 47% in Test 3, while for Test 4 it was only 11%. This difference may be attributed to the different Dr values in both tests. It is interesting to note that PGD values for strip and rectangular foundations were almost the same.
Figure 4.32: Base and foundation top PGA and PGD values for Case C: (a) PGA and (d) PGD for KEQ, (b) PGA and (e) PGD for WC and (c) PGA and (f) PGD for VC.
Figure 4.33: Base and foundation top PGA and PGD values for Cases C and D of Tests 3 and 4. (a) PGA and (d) PGD for KEQ, (b) PGA and (e) PGD for WC and (c) PGA and (f) PGD for VC.
4.8 RACKING OF BOX CULVERTS

Wang (1993) proposed a procedure to determine the racking deformations of the differential movement of the top and bottom slabs of the culvert using the racking ratio. In order to investigate the validity of this procedure, the peak ground displacements resulting from the double derivative of the horizontal acceleration time histories of Ac4, Ac5, Ac9 and Ac12 were used to compare the Free Field $PGD$ to the Structure Field $PGD$ at the levels of the top and bottom slabs of the culvert. The difference in $PGD$ was calculated as follows:

$$\Delta PGD_{FF} = PGD_4 - PGD_5$$ (4.14)
$$\Delta PGD_{SF} = PGD_9 - PGD_{12}$$ (4.15)

The racking ratio $R$ is calculated as:

$$R = \frac{\Delta PGD_{SF}}{\Delta PGD_{FF}}$$ (4.16)

The results obtained for the racking ratio for Tests 1, 2, 3, and 4 are presented in Tables 4.3, 4.4, 4.5, and 4.6. Generally, most of differences in $PGD$ either in FF or SF are positive, which indicates that the $PGD$ at the level of top slab is larger than that at the level of the bottom slab. Only some shaking cases in Test 3 exhibited a negative sign in the FF indicating the opposite. The results presented in the tables show that the thickness of the culvert and sand density can affect the raking ratio (i.e. cumulative effect).

4.8.1 Effect of Soil Density

The results of Tests 1 and 2 clearly show the raking ratios of the thick culvert for cases of $Dr = 90\%$ and $50\%$. The racking ratios for Test 1 are in the range of 0.1 and 0.7, while for Test 2 they are in the range of 1.5 to 1.9, depending on the test case and level of shaking. The results of Test 1 are less than 1.0, and this indicates that the racking deformation of the culvert for the SF is less than that for the FF; however, the results of Test 2 are greater than 1.0, which indicates an increase in the culvert deformations for the SF than those for the FF.
Comparing the results of Tests 3 and 4, which are for the thin culvert at Dr = 50% and 90%, shows that combining the lower density with a thin culvert in Test 3 produces a high racking ratio ranging from -8 to 156, while for Test 4 the range was between 5 and 13 depending on the test case and level of shaking. In both tests, the racking deformations of the SF are higher than those for the FF.

4.8.2 Effect of Culvert Thickness

The results of Tests 1 and 4 (as well as Tests 2 and 3) are compared to investigate the effect of culvert thickness. The results of Tests 1 and 4 for Dr = 90%, and Tests 2 and 3 for Dr = 50%, show that the thick culvert experienced very small racking deformations compared to the thin culvert. It may be concluded that the level of racking deformations in the thick (i.e. more rigid) culverts are less than for thin (i.e. less rigid) culvert.

Comparing the extreme cases of Test 1 and Test 3, it is noted that culvert racking in Test 1 is the lowest. This is because of the culvert high rigidity and soil high density, the culvert and soil move together during shaking, which reduces the racking deformations of the culvert. On the other hand, the racking in Test 3 is highest. This is because the culvert is flexible and the soil is not dense, the culvert elements will move with shaking resulting in high racking deformations. In addition, the racking ratio for the FF in some shakings in Test 3 were negative, which indicates that using FF deformations can sometime correctly predict the culvert behaviour under seismic loading in similar situations. The results show that, as expected, the culvert racking deformations increase as the level of shaking increase and that the relative stiffness between the culvert and soil appears to have a great effect.
Table 4.3: Racking of the box culvert in Test 1

<table>
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<th>Test</th>
<th>Shaking Type</th>
<th>PGA (Base) (g)</th>
<th>$\Delta PGD_{FF}$ (cm)</th>
<th>$\Delta PGD_{SF}$ (cm)</th>
<th>$\Delta PGD_{SF}$/$\Delta PGD_{FF}$</th>
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<td>T1A</td>
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<td>0.13</td>
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<tr>
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<td>0.12</td>
<td>0.07</td>
<td>0.56</td>
</tr>
<tr>
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<td>WCM</td>
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Table 4.4: Racking of the box culvert in Test 2

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Table 4.5: Racking of the box culvert in Test 3

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<td>T3A</td>
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</tr>
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Table 4.6: Racking of the box culvert in Test 4

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<th>Test</th>
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<th>PGA (Base) (g)</th>
<th>$\Delta PGD_{FF}$ (cm)</th>
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<th>$\frac{\Delta PGD_{SF}}{\Delta PGD_{FF}}$</th>
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<td>1.81</td>
<td>9.52</td>
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<td>0.17</td>
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4.9 SOIL CULVERT INTERACTION PARAMETERS:

The soil culvert interaction was investigated under static and seismic loadings. This section focuses on the bending moment and soil pressure, since they are important for engineering design. The interaction between the box culvert and the sand was evaluated considering the results obtained from the strain gauges and tactile pressure sensors. The strain readings were used to capture the interaction performance between the culvert and the sand under static and seismic loadings, while the tactile pressure sensors recorded the contact soil pressures under static loadings only.

4.9.1 Static Bending Moment

The static bending moment was derived using the strain gauge data recorded at 60g. The strain data was measured on the top slab and side wall. At each strain gauge location, the strain was recorded on the inside and outside faces of the culvert. Equations 3.4 and 3.6 were applied to calculate the calibration factors. Based on the measured strains on both faces and the calibration factor, the bending moment values were obtained at each strain gauge location. Due to the symmetry of the bending moment on the top slab and since there was a limited number of strain gauges used, mirror points of each strain gauge location were applied to produce double the number of bending moment points. That was not possible on the side wall since there was no symmetry in the bending moment. As the amount of strain data collected was very large and for the purpose of drawing conclusions, the results are presented in the form of comparisons to show the effect of the presence of the surface foundation, sand density, and culvert thickness as illustrated in the following sections.

4.9.1.1 Effect of surface foundation

Figures 4.34 and 4.35 demonstrate the effect of surface foundation on the bending moment at the top slab and side wall in Test 4. The results clearly show the effect of the foundation on the culvert bending moments, i.e., as the foundation load increased in Case B (50 kPa) and Case C (100 kPa), the values of bending moment on top slab and side wall increased, and they were higher than the moments for Case A (no foundation). The bending moments from all other tests have similar shape and trends and are provided in Appendix (F).
Figures 4.34 and 4.35 demonstrate the effect of strip foundation on the soil pressure. Even though the theoretical foundation pressure calculated for strip foundations (Cases C) and rectangular foundations (Case D) is almost the same, the measured pressures on the box culvert were different, which also affected the shape of the corresponding bending moments. In Case D of Tests 3 and 4, the bending moment values on top slab and side wall were very close to Case A where there is no foundation on the surface. This indicates that the effect of rectangular foundation pressure on the box culvert was very small. Even if it has the same pressure as the strip foundation, it does not have the same effect on box culvert in terms of the amount of soil pressure transferred to the box culvert. This behaviour may be attributed to the small foundation size relative to the culvert depth, hence its pressure bulb reduced with depth minimizing the pressure transferred to the culvert.
Figure 4.34: Comparison of bending moment on the top slab for all cases of Test 4

Figure 4.35: Comparison of bending moment on the side wall for all cases of Test 4
4.9.1.2 Effect of soil density

The effect of soil density on the bending moment at the culvert top slab and side wall is demonstrated by comparing the results of Tests 1 and 2 (and Tests 3 and 4) since they have the same culvert thickness but different soil density. Only results of Tests 3 and 4 for Case A are presented herein, and the remainder results are shown in Appendix (G).

Figures 4.36 and 4.37 show that for thin culvert (Tests 3 and 4), the bending moment decreased as the soil density (and hence elastic modulus) increased. As the soil elastic modulus increased, the soil pressure on the culvert decreased and consequently the bending moment decreased. This further investigated numerically as will be explained in Chapter 5. On the other hand, the bending moments for Tests 1 and 2 (i.e. thick culvert) are almost the same at the top slab and there was a small difference between them at the center of side wall. This indicates that for thick culverts, it is relatively rigid regardless of the soil stiffness and hence the soil pressure and bending moment values are not affected by the soil density.

Figure 4.36: Comparison of bending moment on the top slab for Case A of Tests 3 and 4
4.9.1.3 Effect of culvert thickness

Figures 4.38 and 4.39 illustrate the effect of culvert thickness on the bending moments at the culvert top slab and side wall. Tests 1 and 4 (and Tests 2 and 3) are compared since they have the same $Dr$ values but different culvert thicknesses. Only results for Tests 1 and 4 for Case A are presented here, and the remainder are shown in Appendix (H).

Generally, the bending moments at the top slab decreased as the culvert thickness decreased. This is attributed to the soil arching effect, i.e., as the culvert thickness decreases, its displacement increases and hence reducing the loads attracted. The soil prisms on either side of the culvert will take some of the soil pressure and therefore, reduces the pressure on the culvert. The bending moment of side walls for the thick culvert was positive throughout the height. As the culvert thickness decreased, the bending moment decreased, resulting in some negative moment near the centre, which is also attributed to soil arching.
Figure 4.38: Comparison of bending moment on the top slab for Case A of Tests 1 and 4

Figure 4.39: Comparison of bending moment on the side wall for Case A of Tests 1 and 4
4.9.2 Static Pressure

The actual static soil pressure acting on the culvert is important for determining the soil culvert interaction factors. Therefore, soil contact pressures were evaluated using strain gauge data and tactile pressure sensors.

The tactile pressure sensors measured the soil pressure on the top slab and side wall of the culvert. Two files were recorded for each tactile pressure sensor; one during the process of equilibration and one during calibration. By applying the equilibration and calibration files to the recorded data by using the I-Scan software (I-Scan, 2006), the data was converted to pressures depending on the pressure units used during calibration. Each tactile pressure sensor consists of 44 columns crossed by 44 rows, i.e., there were 44 pressure readings in each line. The soil pressure measured by the sensors increased gradually as acceleration increased from 1g to 60g. Three data points from each line were collected at 60g: the maximum, the minimum, and the average of all 44 data points. Due to the large amount of data collected, the average results from each case were used to explore the effects of: presence of surface foundation, sand density, and culvert thickness.

The measured strain values were converted to bending moments by applying calibration factors, and the resulting bending moments were curve fitted with several functions. The equations of the fitted curves were then subjected to double derivation in order to obtain the soil pressure.

Figure 4.40 presents data obtained from different methods to fit the measured bending moment values. The experimental bending moment data points shown as yellow diamond points, and they were fitted with a 3rd order polynomial, a 4th order polynomial, a cubic spline interpolant and the bending moment resulting from a numerical model using FLAC 2D (as will be explained in details in Chapter 5). All curve fitting results for the bending moments were similar and therefore only the results of Case A are presented here and the reminder of the results are shown in Appendix (I). The results from Test 1 Cases and their corresponding soil pressures are compared to the vertical overburden pressure and the soil pressure directly measured through tactile pressure sensors only for the top slab.
Figure 4.40: Fitting of bending moment resulted from strain data on the top slab
for Test 1 Case A

The soil pressures obtained from the double derivative of each curve fitting function are compared to the vertical overburden pressure on the top slab as well as the range of soil pressures measured using tactile pressure sensors. The resulting soil pressures from the 3rd order polynomial, 4th order polynomial, cubic spline interpolant and FLAC 2D analyses are shown in Figures 4.41, 4.42, 4.43, and 4.44, respectively.

The different curve fitting methods resulted in similar bending moment distributions except at the points near the edges. At the edges, the FLAC 2D analyses led to the lowest bending moment values (close to values predicted for 3rd order polynomial), while the 4th order polynomial resulted in the highest bending moment values.

Curve fitting the bending moment on the top slab with 4th order polynomial leads to parabolic soil pressure distribution since the double derivation yields 2nd order polynomial. The tactile pressure sensors measurements showed maximum soil pressures close to the edges, which were higher than the theoretical overburden pressure. In addition, Dasgupta and Sengupta (1991) among others reported that the soil pressure distribution on the top slab takes a curved shape like a parabola, with higher pressure close to the edges and lower pressure towards the center. Thus, the 4th order polynomial fit resulted in acceptable shape of soil pressure distribution, but the
pressure values were much higher than the measured values except at points close to the center. On the other hand, the 3\textsuperscript{rd} order polynomial resulted in uniform soil pressure distribution, but the values were comparable to the theoretical overburden pressure at the elevation of the top slab and the measured pressure using the tactile sensors. The cubic spline resulted in an unacceptable shape of soil pressure distribution. Finally, the bending moment diagram obtained from the FLAC 2D model was fitted with a 4\textsuperscript{th} order polynomial, which resulted in the right shape of soil pressure distribution and good match with the measured maximum tactile soil pressure values.

It is clear from the discussion above that the only curve fitting method that produced good match with the measured soil pressure on the top slab in terms of distribution and values was through the bending moment results obtained from the FLAC 2D analyses. It should be noted that the FLAC 2D models were calibrated using the experimental data to simulate the observed behaviour during the experiments. Therefore, this was adopted in processing all test data obtained as will be discussed in detail in Chapter 5. For the side wall, the best curve fitting of moment data was achieved through a 3\textsuperscript{rd} order polynomial, resulting in soil pressure that increases linearly from top to bottom.

The soil pressures on the side wall measured based on strain gauges (3\textsuperscript{rd} order polynomial fit) and tactile pressure sensors data obtained from Test 1 Case A are compared to the at rest and active horizontal theoretical earth pressures as shown in Figures 4.45.

Generally, the slope of the horizontal tactile soil pressures is slightly different than the strain gauge and theoretical horizontal soil pressures, showing lower values at the top and higher values at the bottom compared to the theoretical horizontal soil pressures. This can be attributed to the effect of shear stresses. It is noted from Figure 4.45 that the horizontal pressures evaluated from the strain gauge readings are close to the maximum horizontal tactile soil pressure, while the theoretical at rest horizontal soil pressures are close to the average horizontal tactile soil pressure. It is also noted that the active theoretical horizontal soil pressures are close to the minimum horizontal tactile soil pressure. It may be concluded that the horizontal soil pressure on the culvert side walls lie between the at rest and active lateral earth pressure values. It may also be concluded that the tactile pressure sensors readings provide bounds for the expected soil pressures.
Figure 4.41: Comparison of pressure resulting from 3\textsuperscript{rd} order polynomial fitting of moment on the top slab for Test 1 Case A

Figure 4.42: Pressure on top slab using 4\textsuperscript{th} order polynomial fitting of moment for Test 1 Case A
Figure 4.43: Pressure on the top slab using Cubic Spline fitting of moment for Test 1 Case A

Figure 4.44: Pressure results on the top slab from FLAC 2D for Test 1 Case A
4.9.2.1 Effect of foundation

Figures 4.46 and 4.47 present the average vertical and horizontal soil pressures measured during Test 1 using the tactile pressure sensors on the top slab and side wall. The effect of the surface foundation was clearly visible in the top slab pressure, and less so on the side wall. The soil pressures for Cases B (50 kPa), and C (100 kPa) are compared to Case A (no foundation). As expected, the soil pressure on the top slab for Case A was the lowest and the pressure for Case C was highest. It is also noted that the distribution of soil pressure on the top slab was parabolic, and varied with the culvert thickness. The thicker culvert attracted higher soil pressure. Similar results were observed from the other tests, which are shown in Appendix (J).

The horizontal soil pressure on the side wall increased with depth in all tests. In Tests 1 and 2 (thick culvert), the soil pressure decreased as the foundation pressure increased. This is due to increased vertical pressure on the top slab, increasing its deflection. Thus, the side walls moved outward resulting in reduced horizontal pressure. This effect was more pronounced for the thick culvert and the strip foundation, compared to thin culvert and rectangular foundation.
Figure 4.46: Average tactile pressure on the top slab for Test 1

Figure 4.47: Average tactile pressure on the side wall for Test 1
4.9.2.2 Effect of soil density

Figures 4.48 and 4.49 illustrate the effect of soil density on the average soil pressures on the culvert top slab and side wall obtained from the tactile pressure sensors through comparing results from Tests 1 and 2, and Tests 3 and 4. Only results of Case A of Tests 1 and 2 are presented here, and other results are shown in Appendix (K).

The soil pressures on the top slab and side wall obtained from Test 1 were higher than that obtained from Test 2, i.e., the soil pressure increased as the soil density increased, as expected. However, the effect of soil density was more pronounced in the vertical soil pressure on the top slab than the horizontal soil pressure on the side wall.

Figure 4.48: Average tactile pressure on the top slab for Case A of Tests 1 and 2
4.9.2.3 Effect of culvert thickness

Figures 4.50 and 4.51 illustrate the effect of culvert thickness on the vertical and horizontal soil pressures for the top slab and side wall through comparing results of Tests 1 and 4 (and Tests 2 and 3). As the results from all tests were similar, only results of Tests 1 and 4 for Case A are shown here, and the reminder are presented in Appendix (L).

Generally, the vertical soil pressure on the top slab increased as the culvert thickness increased, while the horizontal soil pressure on the side wall increased as the culvert thickness decreased. This is attributed to increased deflection of thin culvert, hence reducing the pressure attracted to the culvert due to the soil arching effect. In this case, the soil prisms on both sides of the culvert would carry some of the pressure and therefore culvert carries less. The thick culvert attracted more pressure causing larger deflection of the top slab, which resulted in outward movement of the side wall and hence the horizontal soil pressure decreased.
Figure 4.50: Comparison of the average tactile pressure on the top slab for Case A of Tests 1 and 4

Figure 4.51: Comparison of the average tactile pressure on the side wall for Case A of Tests 1 and 4
4.9.2.4 Soil Culvert Interaction Factors:

The soil culvert interaction factor $F_e$ is defined as the ratio between the measured soil pressure and the theoretical soil prism pressure. The theoretical vertical soil pressure on the top slab $\sigma_v'$ is usually obtained by multiplying the unit weight $\gamma_s$ of the soil column above the culvert by its height $h$. The soil culvert interaction factor for the side wall is defined as the ratio between the horizontal measured soil pressure and the theoretical horizontal soil pressure $\sigma_h'$. The vertical and horizontal soil pressures increase with depth and they are related through the lateral earth pressure coefficients, either for at-rest, $K_o$, or active, $K_a$, conditions. Thus, the interaction factor for the top slab is given by:

$$F_e = \frac{(\sigma_v')_{\text{measured}}}{(\sigma_v')_{\text{theoretical}}} = \frac{(\sigma_v')_{\text{measured}}}{\gamma_s \cdot h} \quad (4.17)$$

and the interaction factor for the side wall is given by:

$$F_e = \frac{(\sigma_h')_{\text{measured}}}{(\sigma_h')_{\text{theoretical}}} = \frac{(\sigma_h')_{\text{measured}}}{K_o \cdot \gamma_s \cdot h} \quad (4.18)$$

where

$$K_o = 1 - \sin \phi' \quad (4.19)$$

$$K_a = \frac{1 - \sin \phi'}{1 + \sin \phi'} \quad (4.20)$$

The measured soil pressure distribution on the top slab has higher values at the edges and lower values at the center, and both are considered in evaluating the interaction factors. Similarly, the side wall pressure distribution shows lower values at the top and higher values at the bottom and both are considered herein. Therefore, the top and bottom values were considered as shown in the following sections.

For test cases involving surface foundation (Cases B, C and D), the additional pressure on the culvert top slab and side walls due to the foundation was calculated using the approximate trapezoidal 2:1 method as shown in Figure 4.52.
The following two sections provide a summary of the soil culvert interaction factors obtained from the soil pressures measured with the tactile pressure sensors and those evaluated from strain gauges readings. The results presented in Appendix (M) provide further explanation for the variation of the interaction factors for the different cases.

4.9.2.4.1 Tactile pressure sensors data

Table 4.7 presents the soil culvert interaction factors based on the tactile pressure sensors readings. The $F_e$ values are provided for the top slab edge and center; and for the side wall top and bottom, and relative to the maximum, minimum, and average measured pressures. Interaction values greater than 1.0 signify measured soil pressure that is higher than the theoretical soil pressure.

For the edge of top slab, the maximum $F_e$ value, $F_{e\text{ max}} > 1.0$, while at the center, $F_{e\text{ max}} < 1.0$ for Tests 3 and 4 and $F_{e\text{ max}} > 1.0$ for Test 1 and Cases B and C of Test 2 (but still lower than $F_{e\text{ max}}$ at edges). This shows that for the thick culvert, $F_{e\text{ max}}$ is always $> 1.0$, while for the thin
culvert, $F_e$ is below 1.0 at the center and above 1.0 at the edges. The higher values for the thick culvert because it deforms less than the thin culvert and attracts higher soil pressure. $F_{e\,\text{max}}$ values for Test 1 are higher than for Test 2 (and $F_{e\,\text{max}}$ for Test 4 are higher than for Test 3), which indicates the effect of soil density; as the soil density increased (Tests 1 and 4), values of $F_{e\,\text{max}}$ increased. The minimum values of $F_e$ ($F_{e\,\text{min}}$) for the top slab are all below 1.0, which means that all values are less than the theoretical soil pressure. The average values of $F_e$ ($F_{e\,\text{ave}}$) for the top slab in Tests 1 and 2 are $> 1.0$ at the edges and $< 1.0$ at the center, while for Tests 3 and 4, $F_{e\,\text{ave}} < 1.0$ for all cases (due to culvert thickness as explained).

The values of $F_e$ for the side wall are lower at the top and increase at the bottom as observed for Tests 1, 2 and 3. For Test 4, the soil pressure distribution demonstrated a curved shape with soil pressure increase at the mid-height, and in some cases a peak value at the top. In most cases, $F_{e\,\text{max}} > 1.0$, $F_{e\,\text{min}} < 1.0$, while $F_{e\,\text{ave}} < 1.0$ for the top and $> 1.0$ for the bottom for Tests 1, 2, and 3. For Test 4, all $F_e$ values are $< 1.0$, indicating same behaviour observed for the top slab (including effects of soil density and culvert thickness).

### 4.9.2.4.2 Strain gauge data

Table 4.8 shows the soil culvert interaction factors based on soil pressures established from the strain gauges readings. It is observed that almost all $F_e$ values for the top slab are $> 1.0$ for the edge and the center for Tests 1 and 2. In Tests 3 and 4, $F_e > 1.0$ for the edge and $F_e < 1.0$ for the center (due to effect of culvert thickness). The effect of soil density resulted in $F_e$ of Test 1 $> F_e$ of Test 2 (also $F_e$ for Test 4 $> F_e$ for Test 3).

It is interesting to note that the values of $F_e$ obtained from Tests 1 and 4 are higher at the edge and lower at the center, but the Test 1 $F_e$ values at the edge are lower than those from Test 4, while at the center, the values of $F_e$ from Test 1 are higher than those from Test 4 (and same observations for values of $F_e$ obtained from Tests 2 and 3). This is because the thin culvert will attract less soil pressure at the center and higher pressure at the edge. For the side wall, the slope of the horizontal soil pressure depends on the shape of the bending moment. Generally, all results show small values at the top and larger values at the bottom.
Table 4.7: Soil culvert interaction factors resulted from tactile pressure data

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<th>Side Wall ($F_w$) at rest</th>
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<td>Min</td>
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</tr>
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<td></td>
<td>Center</td>
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</tr>
<tr>
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<td>Center</td>
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<tr>
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</tr>
<tr>
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</tr>
<tr>
<td></td>
<td>Center</td>
<td>0.93</td>
</tr>
<tr>
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<td></td>
<td>Edge</td>
<td>1.23</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>0.86</td>
</tr>
<tr>
<td>T4A</td>
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<td></td>
</tr>
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<td>Edge</td>
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</tr>
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<td>Center</td>
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<tr>
<td>T4B</td>
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<td></td>
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<tr>
<td>T4C</td>
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</tr>
<tr>
<td></td>
<td>Center</td>
<td>0.87</td>
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<tr>
<td>T4D</td>
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<td></td>
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<td></td>
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<td>1.45</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>0.83</td>
</tr>
</tbody>
</table>
Table 4.8: Soil culvert interaction factors resulted from strain data

<table>
<thead>
<tr>
<th>Test</th>
<th>Top Slab ($F_e$)</th>
<th>Side Wall ($F_e$) at rest</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Edge</td>
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</tr>
<tr>
<td>T1A</td>
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</tr>
<tr>
<td></td>
<td>Center</td>
<td>Top</td>
</tr>
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<td>T1B</td>
<td>1.19</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
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</tr>
<tr>
<td>T1C</td>
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<td>0.85</td>
</tr>
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<td></td>
<td>Center</td>
<td>Top</td>
</tr>
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<td>T2A</td>
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<td>1.09</td>
</tr>
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<td></td>
<td>Center</td>
<td>Top</td>
</tr>
<tr>
<td>T2B</td>
<td>1.13</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>Top</td>
</tr>
<tr>
<td>T2C</td>
<td>1.11</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>Top</td>
</tr>
<tr>
<td>T3A</td>
<td>1.29</td>
<td>1.13</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>Top</td>
</tr>
<tr>
<td>T3B</td>
<td>1.29</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
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<td>Top</td>
</tr>
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<td>T4A</td>
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<td>T4C</td>
<td>1.58</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>Center</td>
<td>Top</td>
</tr>
</tbody>
</table>
4.9.3 Seismic Bending Moment

Seismic bending moments were calculated based on strain time histories recorded at 60g. There are two possible methods to select strain values to be used for this calculation. The first method is to select the strains at the time step of the maximum PGA of the shaking. This method produces a smooth but un-symmetric bending moment shape. The strain values in this case may not represent the peak strains at all locations. The second method is to select the peak strain at each location. This method would produce the maximum bending moment at each measuring point, i.e., envelope of maximum bending moment. The second method is adopted herein to evaluate seismic bending moments that may be considered for design purposes.

The strains were recorded during flight from 1g to 60g and then during the shakings applied at 60g. Therefore, the static strain data at 60g were subtracted from the strains measured during the shakings to separate the static strains from the seismic strains, and consequently, static bending moment from the seismic bending moment. Inspecting all records of strain time histories, it was found that peak strains occurred near the peak acceleration of the shaking. An example for the seismic strain time history is shown in Figure 4.53(a), while the acceleration time history is shown in Figure 4.53(b).

The seismic bending moment on the culvert top slab is not symmetrical, and hence the number of strain points was not enough to provide a complete picture of the bending moment distribution during seismic loading. However, the data collected were used to calibrate numerical models that were used to evaluate the bending moments more fully as will be explained in detail in Chapter 5. In the mean time, the envelopes of seismic bending moments discussed herein to investigate the effects of earthquake characteristics and culvert thickness.
4.9.3.1 Effect of peak ground acceleration (PGA)

Figures 4.54, 4.55, and 4.56 present the envelopes of the seismic bending moment obtained at the culvert top slab for the WC, VC, and KEQ shakings. These results are presented here as an example and to be used as basis for discussion. The results obtained for other tests and for the side walls have similar trends, and are included in Appendix (N).

The figures show that the seismic bending moments increased as the PGA increased. It is also noted that there was no universal shape of envelop of seismic bending moment.
Figure 4.54: Effect of PGA amplitude on seismic bending moments (WC, Case A of Test 1)

Figure 4.55: Effect of PGA amplitude on seismic bending moments (VC, Case A of Test 1)
4.9.3.2 Effect of culvert thickness

Figures 4.57, 4.58, and 4.59 illustrate the envelope of top slab seismic bending moment diagrams for WC, VC, and KEQ, respectively, during Tests 1 and 4. The results for the side walls and other tests have similar trends, and are shown in Appendix (O).

Similar to the static bending moments on the top slab, the values of seismic bending moment envelope decreased as the culvert thickness decreased. Similar behaviour is noted for the envelope of seismic bending moments on the side wall. This could be attributed to the larger seismic displacement of culvert, resulting in higher horizontal soil pressures and consequently larger bending moments.
Figure 4.57: Effect of culvert thickness on seismic bending moments (WCL, Tests 1A and 4A)

Figure 4.58: Effect of culvert thickness on seismic bending moments (VCL, Tests 1A and 4A)
4.10 SUMMARY

The data collected from four static and seismic centrifuge tests on box culverts are presented in this chapter. The surface settlements and displacement time histories were measured during flight as the centrifuge acceleration increased from 1g to 60g and during shaking at 60g. The majority of surface settlement occurred during the spin-up (static phase), whilst only a small residual settlement was observed between the start and the end of the seismic loading.

The seismic loading results showed amplification in the PGA, PGV and PGD values from the bottom to the top of the model. The PGA, PGV, and PGD at different locations at the same elevation were comparable. As the amplitude of the PGA of the base motion increased, these values increased. The dynamic soil properties of the soil bed were investigated by evaluating the shear stress and shear strain time histories obtained from the acceleration time histories at different elevations. The effect of data filtering was studied and it was concluded that: the minimum filter range produced the correct shape of the displacement time history; and the unfiltered acceleration should be used to calculate the shear stress while the filtered displacement should be used to calculate the shear strain.
The kinematic soil culvert interaction was found to have significant impact on the PGA values at the surface. The PGA at the surface for the Structural Field decreased considerably compared to the Free Field, especially at high PGA of input motion. This difference in PGA can be up to 50% reduction at 0.3g for the KEQ, while at 0.1g this effect decreased significantly. This effect is most significant for the low period range, and was almost the same for the range of soil density and culvert thickness considered in this study. On the other hand, the Free Field motion was affected by the soil density significantly, as the PGA amplified more for loose soil.

The results indicated very small values of rocking occurred during shaking. The rocking values increased as the PGA of the base motion increased. In addition, the surface foundation experienced rocking values higher than those for the box culvert. The lateral movement observed for the strip and rectangular foundations were similar and increased as the amplitude of the base motion increased. The peak acceleration for the rectangular foundation was higher than the strip foundation and the difference was larger for the sand with Dr = 50%. Racking ratios were less than 1.0 for the case of thick culvert in dense soil, which indicated that the Free Field deformations were higher than those for the Structure Field. The racking ratio was larger than 1.0 for loose sand. The seismic bending moments increased as the amplitude of the input motion and the culvert thickness increased.

The bending moment and soil pressure distributions were investigated to evaluate the soil culvert interaction under static condition, including consideration of surface foundations. The strip foundation loading increased the bending moment and soil pressure values. The rectangular foundation effect was very small. The bending moment and soil pressure values increased as the culvert thickness increased. The culvert in dense soil experienced lower bending moments due to the higher elastic modulus of the soil. The soil pressure distribution on the top slab was parabolic and increased linearly on the side wall. The soil culvert interaction factors on the top slab were higher at the edges and lower at the center. Also, the effect of soil density on the culvert interaction factors was relatively small compared to the effect of the culvert thickness.
CHAPTER FIVE
NUMERICAL MODELLING RESULTS

5.1 INTRODUCTION

Physical modeling provides an opportunity for gaining general understanding of the soil structure interaction system and can be used to explore and identify fundamental mechanisms of behaviour. On the other hand, numerical modeling is advantageous for simulating complex systems under controlled conditions. Model test results are often used to calibrate numerically simulated systems and predict prototype responses, and provide a database for the validation of the numerical models under either static or seismic conditions.

Limited numerical modeling research has been dedicated to study soil culvert interaction (SCI) problems. Most of these research studies focused on the static part. Katona et al. (1981, 1982) conducted a finite element study on box culverts and reported that the soil pressure distribution on the culvert top slab is parabolic (high at the edges and low at the centre). Tadros et al. (1989b) proposed analytic solution in terms equations to calculate the soil pressure on the culvert side walls as a function of the height of fill above the culvert. Kim and Yoo (2005) examined foundation soil under the culvert and its effect on the soil culvert interaction factors. They proposed equations to determine the soil culvert interaction factors considering the type of culvert installation. Kang et al. (2008) studied the effect of side friction of a box culvert on the SCI factors and related them to the ratio of fill height to culvert width. Pimentel et al. (2009) investigated the nonlinear behaviour of a box culvert and proposed three SCI factor relations; global, shear and moment. Joa et al. (2003) investigated the effect of a surface strip footing above the box culvert. They demonstrated that the footing induced soil pressure distributions around box culverts are strongly influenced by the interaction between the culvert and the surrounding soil, i.e. the soil pressure distribution depends on culvert dimensions.

A few centrifuge tests has been performed on box culverts, which were used to calibrate numerical models for further numerical studies (e.g. Stone and Newson, 2002; McAffee and Valsangkar, 2008; McGuigan and Valsangkar, 2010 and 2011; and Oshati et al., 2010a and b).
Amiri et al. (2008) and Katona (2010a, 2010b) studied the seismic soil culvert interaction numerically to investigate the racking effect proposed by Wany (1993).

The current study examines SCI under static and seismic loading. In this study, a 2D FLAC numerical model was established to investigate the soil culvert interaction. The numerical models were calibrated/validated using the results of the static and seismic centrifuge tests involving the box culverts and surface foundation in dry sand as explained in Chapter 4. This chapter presents the details of the numerical models of the centrifuge tests. Several factors were explored in this study, including: soil pressure, static and seismic bending moment, and kinematic soil culvert interaction. The effects of amplitude of the ground input motion and its frequency on the seismic bending moment were studied, and a comparison with some standards such as CHBDC is also reported.

5.2 NUMERICAL SIMULATION

Numerical analysis has been carried out to investigate several static and seismic parameters affecting the soil culvert interaction problem. The model consisted of three main parts: the box culvert, the surface foundation, and the soil. The box culvert and the foundation were modeled as linear elastic material represented by their actual elastic modulus and Poisson’s ratio, while the sand was modeled as an elastic perfectly plastic continuum material that deforms plastically according to the Mohr-Coulomb criteria, represented by its actual strength and stiffness characteristics.

5.2.1 Numerical Approach

The program FLAC 2D (Itasca, 2005) was used to develop the numerical models for the centrifuge tests and simulate the response of box culverts under static and seismic loading. FLAC 2D (Finite Difference Lagrangian Analysis Code – 2D) is a two-dimensional explicit finite difference program that has the ability to simulate the behaviour of soils and structures considering soil-structure interaction that occurs under different kinds of loads covering a range from elastic to plastic deformations. FLAC uses small time steps to ensure stability of the model, ensuring computational efficiency. It does not require iterations to follow the nonlinearity of a constitutive law, and no significant numerical damping is introduced for dynamic solutions. The
material behaviour is modeled by using elements that obey assigned linear or nonlinear stress-strain behaviour in response to the forces and boundary conditions.

The box culvert and the surface foundations were assumed to behave linearly and therefore the linear elastic model was used to simulate their response. The linear elastic model requires the parameters that include unit weight, elastic modulus, and Poisson’s ratio. The soil response was expected to cover a range of elastic and plastic deformations; hence the Mohr-Coulomb model was used to simulate its nonlinear behaviour. The model is based on plane strain conditions, and is formulated in terms of effective stresses. The Mohr-Coulomb material model requires conventional sand parameters that include unit weight, friction angle, dilation angle, cohesion intercept, elastic modulus, Poisson’s ratio, shear modulus, and bulk modulus.

5.2.2 Model Mesh and Boundary Conditions

The FLAC model was built to simulate the centrifuge tests at a prototype scale. Therefore, all dimensions of the centrifuge model were scaled to represent the experimental tests in prototype dimensions. The soil was modeled using continuum zones and each zone divided into small grids. The finite difference grids used around the box culvert were square in shape; and were rectangular elsewhere as shown in Figure 5.1. The density of the grid was increased around the box culvert to improve accuracy. Several trials were performed to refine the grids until there was no noticeable change in the results. Numerical computations were conducted with the small strain mode in the first stage, where there was no structure inside the sand. After the box culvert was placed, large strain mode computations were performed to ensure sufficient accuracy.

The box culvert and the foundations were modeled using structural elements. The liner element was used to model the box culvert as recommended for buried structures, and the beam element was used to model the foundation. Liner elements, like beam elements, are two-dimensional elements with three degrees of freedom (x-translation, y-translation and rotation) at each end node, and these elements can be joined with the grid. The primary difference between liner elements and beam elements is that liner elements include bending stresses to check for yielding, whereas beam elements only base the yielding criterion on axial thrust (Itasca, 2005). As there are two uniform thicknesses of box culvert, the center to center dimensions were used,
and the thickness was applied according to the actual dimensions. The thickness of the beam element was equal to the height of the foundation to apply the same pressure.

The boundary conditions for the numerical model simulated the same conditions in the centrifuge tests. In the static analysis, the soil culvert foundation system was under gravity loading only, and therefore the base of the model was fixed in x and y directions, while the side boundaries were fixed in the x direction only. In the seismic analysis, the acceleration time history was applied to the entire base of the model in the horizontal direction. As the rigid centrifuge box enclosed the soil, the base and sides of the model were fixed in both x and y directions to represent the centrifuge container.

![Figure 5.1: Numerical grid and model component](image)

**Figure 5.1: Numerical grid and model component**

### 5.2.3 Model Parameters

It is necessary that the numerical analysis is able to produce results that are in good agreement with the recorded responses during testing. The box culvert and the foundations were modeled using linear elastic elements, with mass density of 2548.4 kg/m$^3$, elastic modulus of 25.2 GPa (25200 MPa) and Poisson’s ratio of 0.2, which are usually used for reinforced concrete structures. Two box culvert thicknesses were used to simulate the box culverts used in the centrifuge tests: 0.533 m and 0.267 m for the thick and thin culverts, after applying Equation 3.2 to determine the prototype dimension based on the scaling factors and the elastic modulus ratio.
To obtain the required pressure (50 and 100 kPa) from the foundation on the soil surface, the height of the model foundations at prototype scale was used as the thickness of the beam elements used to model the foundations.

The nonlinear elastic-plastic material using Mohr-Coulomb failure criterion with non-associated flow rule was used to model the dry sand. Table 5.1 presents the main soil parameters used to model the dry Nevada sand with Dr = 50% and 90%. The mass densities shown in Table 5.1 are the initial values; during centrifuge tests, the mass density changed throughout each test. Therefore, when the FLAC model was applied to a specific test case, the measured density for that case was used. The dry sand modeled is a fine sand, and as it was reported in the literature (e.g. Hunt, 2005) the elastic modulus for fine sand is 10 MPa for medium dense and 30 MPa for dense sand; the same Poisson’s ratio of 0.28 was used for both densities. To avoid any numerical instability, a small value of cohesion (1 kPa) was used as recommended by Kanungo (2008).

Table 5.1: Main modeling parameters for the dry Nevada sand

<table>
<thead>
<tr>
<th>Model parameters</th>
<th>Medium dense</th>
<th>Dense</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relative density Dr (%)</td>
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<td>90</td>
</tr>
<tr>
<td>Mass density ρ (kg/m³)</td>
<td>1605.7</td>
<td>1687.7</td>
</tr>
<tr>
<td>Elastic modulus Es (MPa)</td>
<td>10</td>
<td>30</td>
</tr>
<tr>
<td>Shear modulus G (MPa)</td>
<td>3.91</td>
<td>11.7</td>
</tr>
<tr>
<td>Bulk modulus K (MPa)</td>
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<td>22.7</td>
</tr>
<tr>
<td>Poisson’s ratio ν</td>
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<td>0.28</td>
</tr>
<tr>
<td>Friction angle φ (°)</td>
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<td>40</td>
</tr>
<tr>
<td>Dilation angle ψ (°)</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Cohesion c (kPa)</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

During the seismic stage of FLAC modeling, the average shear velocity (250 m/sec) and shear modulus (106 MPa) obtained from the backbone curves of the dynamic soil properties calculated from centrifuge test results were used to update the shear modulus of the model. Also the hysteretic damping results obtained from the dynamic soil properties was used to match the average hysteretic damping parameters for the nonlinear analysis in the FLAC model.
5.3 SOIL-CULVERT AND SOIL-Foundation INTERFACE INTERFACE

The box culvert-sand and surface foundation-sand interface conditions in the numerical models were simulated using interface elements. The interface elements were modeled with a linear spring slider system denoted “glued interface” in FLAC, which does not allow slipping or gap opening. It allows only elastic displacement according to the specified stiffnesses: normal stiffness \(k_n\) and shear stiffness \(k_s\) between the two planes representing the structures and soil. Itasca (2005) recommends a rule-of-thumb to estimate the maximum interface stiffness values \(k_n\) and \(k_s\) to be set to ten times the equivalent stiffness of the stiffest neighbouring zone. The apparent stiffness (expressed in stress-per-distance units) of a zone in the normal direction is:

\[
k_n(\text{max}) = k_s(\text{max}) = 10 \times \max \left[ \frac{K + \frac{4}{3}G}{\Delta z_{\text{min}}} \right]
\]

(5.1)

where \(K\) and \(G\) are the bulk and shear modulus, respectively; and \(\Delta z_{\text{min}}\) is the smallest width of continuum zone adjacent to the interface in the normal direction.

This procedure was followed to estimate the preliminary values for the interface normal and shear stiffness. These values were adjusted by refining the magnitude of \(k_n\) and \(k_s\) to obtain a good match with the centrifuge data. After several calibrations, the values adopted to use for both stiffness values are \(k_n = k_s = 5.68 \times 10^3\) MPa/m.

5.4 NUMERICAL MODEL CALIBRATION AND VERIFICATION

To investigate the effect of static and seismic loading on the soil culvert interaction, it was important to ensure that the numerical model used is capable of results that are in very good agreement with the results observed from the centrifuge tests. In order to achieve this goal, a model was built considering the soil properties listed in Table 5.1 and the structures properties stated above, and the analysis was repeated numerous times to examine the effect of different soil and interface parameters on the results in comparison with the static and seismic centrifuge results of Test 1. The model that achieved best fit with the experimental results was verified by applying it to all cases of the static centrifuge tests along with all seven earthquake shakings of Test 1 Case A. The calibrated/verified model was then employed to conduct an extensive
parametric study of static and seismic performance of box culverts in sand as will be discussed in detail in Chapter 6.

5.4.1 Modeling of Static Tests

In modeling static tests, two main parameters were considered in the comparison between the measured and computed results. These parameters are the static bending moment and static soil pressure as discussed below.

5.4.1.1 Static bending moment

The static bending moments measured during centrifuge tests using the strain gauges on the top slab and side wall were compared with values obtained from the FLAC 2D model. Figures 5.2 and 5.3 compare the calculated static bending moments with those obtained from centrifuge results at the top slab and side wall of Test 1, Case A. The numerical model that resulted in best fit with the measured data was used to analyze the reminder of tests. The good agreement between the calculated and measured responses as shown in Appendix (P) verified the model. In particular, the agreement between the measured and computed static bending moments on the top slab and side wall were excellent.

5.4.1.2 Static soil pressure

The FLAC 2D model was used to calculate the soil pressure at the interface elements used between the soil (grid) and the box culvert (liner elements) to determine the contact pressure between them. The calculated pressures were compared to the soil pressures measured directly through the tactile pressure sensors and indirectly through the double derivatives of the measured static bending moment (obtained from strain gauges measurements). Figures 5.4 and 5.5 illustrate the comparison between the measured and calculated soil pressures for top slab and side wall of Test 1, Case A. The agreement is similar for the other tests as shown in Appendix (Q). At the top slab, soil pressures from interface elements were slightly less than values obtained from strain gauges readings, but follow same trend; while on the side wall they are in close agreement, especially at the corners. The calculated soil pressures at the top slab confirm that the soil pressure distribution at the top slab is parabolic, while the pressure on the side wall increases with depth.
Figure 5.2: Measured versus computed bending moment at the top slab - Case A of Test 1

Figure 5.3: Measured versus computed bending moment on the side wall - Case A of Test 1
Figure 5.4: Measured versus computed pressure at the top slab - Case A of Test 1

Figure 5.5: Measured versus computed pressure on the side wall - Case A of Test 1
5.4.2 Seismic Model

During the centrifuge tests, seven earthquake shakings were applied to the centrifuge model during different cases of each test, for a total number of 70 seismic loading cases as shown in Table 3.7. Due to the large amount of data collected, and it is not possible to simulate all these load cases in the numerical modeling because of time constraints. Therefore, only Test 1 Case A was considered for seismic modeling, which involved seven seismic loading cases to verify the validity of the FLAC 2D seismic model. Two parameters were considered in the comparison between the measured and computed results: the kinematic soil culvert interaction and the seismic bending moment.

5.4.2.1 Kinematic soil culvert interaction

To investigate the kinematic soil culvert interaction, acceleration time histories were calculated at the same locations as the points of measurement for Free Field and Structural Field and the calculated and measured PGA values were compared. Figures 5.6 and 5.7 illustrate the comparisons for the Kobe earthquake and Western Canada earthquake loading events. The measured PGA values from accelerometers Ac2 and Ac7 for the FF and SF are compared to the calculated values from FLAC 2D model at the same locations. As can be noted from Figures 5.6 and 5.7, the calculated and measured PGA values for the FF and SF locations are in good agreement, confirming the ability of the numerical model to reproduce the observed behaviour.

5.4.2.2 Seismic bending moment

As mentioned earlier, the seismic bending moment diagrams obtained from the centrifuge results are the “envelope bending moments”, which represent the maximum values of bending moment at each strain gauge location on the top slab and side wall. Similarly, the seismic envelope bending moments are obtained from the numerical simulations; the maximum seismic bending moment at each node in the liner element was noted and the diagram of the envelope of seismic bending moment was established. Figures 5.8 and 5.9 compare the seismic bending moment diagrams obtained from the centrifuge tests and numerical simulations for the KEQL loading event. The comparisons for other earthquake events, using the results from the same numerical model, produced equally good agreement as shown in Appendix (R), hence verifying the numerical model.
Figure 5.6: Measured and calculated PGA values for the Free and Structural Fields for Kobe Earthquake

Figure 5.7: Measured and calculated PGA values for the Free and Structural Fields for Western Canada Earthquake
Figure 5.8: Measured and computed seismic bending moments due to KEQL on the top slab for Case A of Test 1

Figure 5.9: Measured and computed seismic bending moments due to KEQL on the side wall for Case A of Test 1
5.5 NUMERICAL MODEL RESULTS AND DISCUSSION

The soil culvert interaction is evaluated from the numerical results and is discussed in this section. In particular, the effects of soil density on the static bending moment, the static soil pressure and the soil culvert interaction factors are evaluated. The seismic bending moments will be investigated in comparison with the recommended procedure by CHBDC (CHBDC, 2006), static and total bending moments. The effects of amplitude and frequency content of the ground input motions are also presented.

5.5.1 Effect of Soil Density

The centrifuge testing results demonstrated an increase in the static bending moment values for the medium dense sand and a decrease in bending moment values for the dense sand. The numerical results were investigated to further explore the reason for this aspect. The data presented in Table 5.1 was used to model all tests, taking into consideration the actual measured density in each case. As can be noted from Table 5.1, the only different input between models for Dr = 50% and 90% is the value of the elastic modulus, which was 10 MPa and 30 MPa respectively.

5.5.1.1 Static Bending Moment

The results obtained for the thick culvert did not show clear effect for the soil density on the bending moment values. On the other hand, for thin culverts soil density has a consistent, albeit small, effect of the culvert bending moments as demonstrated in Figures 5.10 and 5.11 for Case A of Tests 3 and 4 (and similar results shown in Appendix (S) for other tests).

The bending moment values at the edges of the top slab from Test 4 were close to those of Test 3, similar to what was observed from the centrifuge test results. At the center of top slab, the values of bending moment from Test 4 are smaller than those from Test 3, which demonstrated a moderate decrease in the bending moment values as the soil density increased. This is attributed to the smaller vertical soil pressure acting on the top slab in Test 3 compared to Test 4. Similar observations can be made for the side wall moments, negligible effect at the edges and relatively smaller bending moments at the centre in Test 3 compared to Test 4. This suggests that the amount of horizontal soil pressure attracted to the side walls increases as the soil density decreases.
Figure 5.10: Bending moments on the top slab for Case A of Tests 3 and 4

Figure 5.11: Bending moments on the side wall for Case A of Tests 3 and 4
5.5.1.2 Static Soil Pressure

Figures 5.12 and 5.13 illustrate the comparison between the soil pressure diagrams on the top slab and side wall respectively. Similar results were obtained from the other comparisons and therefore only the results from Case A of Tests 3 and 4 are shown here and the others are presented in Appendix (T). On the thick culverts of Tests 1 and 2, there was no difference in vertical soil pressure values observed at the center of the top slab while there is noticeable difference at the edges. On the side wall, the horizontal soil pressure from Test 1 is larger than those from Test 2 as would be expected. The effect of soil density on the vertical and horizontal soil pressures is clear for the thin culvert (Tests 3 and 4).

The elastic modulus of the dense sand is higher than that of the medium dense sand. The higher stiffness of dense sand resulted in higher vertical soil pressures at the centre of top slab in Test 4 compared to Test 3. On the side wall, the horizontal soil pressure in Test 3 throughout the side is less than the horizontal soil pressure in Test 4. These changes in the vertical and horizontal soil pressures explain the observed bending moments diagrams discussed above.

Inspecting the deflected shape of the box culvert in Tests 3 and 4, it is noted that the settlement of the soil prisms on both sides of the culvert was higher than the settlement of the culvert itself. In Case A of Test 3 (Dr = 50%), the bottom slab deflected inward, causing the top slab and side walls to deflect outward. This indicates that the soil pressure on the culvert top slab was not enough to prevent the outward deflection. In contrast, in Case A of Test 4 (Dr = 90%) and due to the dense soil (with higher stiffness), the soil pressure on the top slab caused it to deflect inward, even though the bottom slab and side walls experienced the same deflection shape as in Test 3. This explains the increase in the vertical soil pressures on the top slab in Test 3 and its decrease in Test 4. On the other hand, the side wall deflected outward in both cases resulting in an increase of soil pressure as the soil density increases.
Figure 5.12: Soil pressures on top slab and the deflected shape for Case A of Tests 3 and 4

Figure 5.13: Soil pressures on side wall for Case A of Tests 3 and 4
5.5.1.3 Static soil culvert interaction factors

The verified results of soil pressure values clearly show that the distribution of vertical soil pressure on the top slab is parabolic with higher pressures at the edges and lower pressure at center. On the side wall, the horizontal soil pressure increases with depth, with some large values close to the top and bottom corners. Table 5.2 presents the soil culvert interaction on the culvert top slab and side wall for all tests obtained from the verified FLAC 2D model. Two extreme points were selected for comparison: the edge and center of the top slab; and top and bottom of the side wall.

The top slab interaction factor, $F_e$, is higher than 1.0 (i.e. vertical soil pressure is higher than the theoretical value) at the edges and is less than 1.0 (i.e. vertical soil pressure is less than the theoretical value) at the center. The soil density effect is insignificant in Tests 1 and 2, since the culvert was thick and therefore the values of $F_e$ are similar, but in Tests 3 and 4 (thin culvert) there are clear differences. In this case, the dense sand produces higher values of $F_e$ at the edges and lower values at center, than the medium dense sand. On the side wall, all $F_e$ values at the top are less than those at the bottom, which indicates an increase in horizontal soil pressure with depth, with $F_e$ values varying between 0.79 and 1.32. The values of $F_e$ increase as the sand density increases and decrease as the thickness of the culvert increases.

The above results indicate that, in general, all the soil behavior remained within the elastic range. The results also show the importance of soil arching and how it affects the soil pressure distribution (i.e. increase in soil pressure at the edges compared to those at the center). This confirms that the soil pressure distribution on the top slab is parabolic, unlike the assumed theoretical overburden distribution (i.e. uniform). The effect of soil density on the $F_e$ values is found to be small, while the effect of culvert thickness on $F_e$ is large. No significant nonlinear behaviour was observed, hence, the linear elastic model used to simulate the culvert was sufficiently accurate.
Table 5.2: Soil culvert interaction factors resulting from FLAC 2D data

<table>
<thead>
<tr>
<th>Test</th>
<th>Top Slab ($F_e$)</th>
<th>Side Wall ($F_e$) at rest</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Edge</td>
<td>Top</td>
</tr>
<tr>
<td>T1A</td>
<td>1.09</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>0.94</td>
<td></td>
</tr>
<tr>
<td>T1B</td>
<td>1.08</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>0.94</td>
<td></td>
</tr>
<tr>
<td>T1C</td>
<td>1.08</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>0.94</td>
<td></td>
</tr>
<tr>
<td>T2A</td>
<td>1.06</td>
<td>1.08</td>
</tr>
<tr>
<td></td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>T2B</td>
<td>1.05</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>T2C</td>
<td>1.04</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>T3A</td>
<td>1.22</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>T3B</td>
<td>1.25</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>T3C</td>
<td>1.25</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td>T4A</td>
<td>1.42</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>T4B</td>
<td>1.47</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>T4C</td>
<td>1.39</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>0.63</td>
<td></td>
</tr>
</tbody>
</table>
5.5.2 Seismic Bending Moment

The seismic bending moment data points described in Chapter 4 were not enough to represent the bending moment distribution. The seismic bending moments obtained from the verified FLAC 2D will therefore be used to explore the effect of input motion characteristics (such as PGA and predominant frequency) on the seismic bending moment.

5.5.2.1 Comparison between seismic bending moment and CHBDC

In the Canadian Highway Bridge Design Code (CHBDC, 2006) provisions (see Equation 2.60), the seismic bending moment is evaluated by multiplying the static bending moment times the vertical component of the earthquake acceleration ratio, $A_V$, which can be taken as two-thirds of the horizontal ground acceleration ratio, $A_H$. As the CHBDC does not specify which vertical acceleration component to be used, either that for the base level (Base) or at the culvert level (Culvert), both of them were used to calculate the seismic bending moment.

Figures 5.14 and 5.15 compare the calculated seismic bending moments obtained from the numerical models and those based on the CHBDC provisions for the culvert top slab and side wall in Test 1 Case A during the KEQL seismic event. Similar results were observed from other shaking events as shown in Appendix (U). The CHBDC appreciate that the dynamic analysis indicates that there are significant bending moments induced by the horizontal component of earthquake and the method proposed, which accounts for vertical accelerations only, is a simplified form that can be used for engineering design. Figures 5.14 and 5.15 show that the seismic bending moments due to vertical accelerations are negligible compared to the obtained seismic bending moments considering the horizontal excitation. For KEQL event the bending moment values at the center of top slab and side wall, the values are close, but the calculated bending moment from the numerical analyses at the edges are up to 16 times (1500 %) that calculated using the CHBDC method. As the earthquake shaking increases, the difference is expected to increase.
Figure 5.14: Computed seismic bending moment from FLAC 2D and from CHBDC due to KEQL on the top slab for Case A of Test 1

Figure 5.15: Computed seismic bending moment from FLAC 2D and from CHBDC due to KEQL on the side wall for Case A of Test 1
5.5.2.2 Comparisons of static, seismic and total bending moments

The static and seismic, and their summation (total) bending moments are compared for each earthquake shaking event. Figures 5.16 and 5.17 present some examples for this comparison for the bending moment diagrams on the top slab and side wall of Test 1 Case A due to KEQL earthquake event. Similar results are obtained for other test cases as shown in Appendix (V).

The static bending moment on the top slab displays positive values at the edges and negative values at the center. The seismic bending moment, on the other hand, varies almost linearly from a high positive value on the left edge to a high negative value at the right edge. The total bending moment (i.e. summation of static and seismic bending moments) displays a high positive value at the left edge and negative values at the right edge, with curvy distribution in between.

On the side wall, all static bending moment values are on the positive side of the axis while the seismic bending moment displays negative values at the top and positive values at the bottom. Thus, the total bending moment displays a negative value at the top and very high positive value at the bottom.

These observations suggest that the seismic bending moments are significant and the total bending moments should be considered in the design, along with the static bending moments. This is particularly important for culverts situated in areas with high seismicity since the seismic bending moment will increase as the PGA increases.
Figure 5.16: Computed static, seismic and total bending moments from FLAC 2D due to KEQL on the top slab for Case A of Test 1

Figure 5.17: Comparison of computed static, seismic and total bending moments from FLAC 2D due to KEQL on the side wall for Case A of Test 1
5.5.2.3 Effect of g-level on the seismic bending moment

Different levels of earthquake shaking events were applied to the numerical model (see Table 3.7), which are characterized by different amplitude of ground input motion. Figures 5.18 to 5.23 compare the seismic bending moments resulting from different levels of KEQ, WC, and VC shaking events on the top slab and side wall.

The results show that as the PGA value increases, the seismic bending moment values increase and may also change from positive to negative and vice versa. The shape of the seismic bending moment diagram is not unique. It is therefore important to conduct the seismic analysis for the culvert considering an earthquake signal representative of the seismicity of culvert location and its seismic design should be performed considering the envelop bending moment covering a range of different levels of the ground motion.

Figure 5.18: Computed seismic bending moment from FLAC 2D due to KEQ on the top slab for Case A of Test 1
Figure 5.19: Computed seismic bending moment from FLAC 2D due to KEQ on the side wall for Case A of Test 1

Figure 5.20: Computed seismic bending moment from FLAC 2D due to WC on the top slab for Case A of Test 1
Figure 5.21: Computed seismic bending moment from FLAC 2D due to WC on the side wall for Case A of Test 1

Figure 5.22: Computed seismic bending moment from FLAC 2D due to VC on the top slab for Case A of Test 1
5.5.2.4 Ratio of seismic to static bending moment

As shown in the previous sections, the seismic bending moments are much higher than the static bending moments, which clearly demonstrate the significant effect of the horizontal component of the peak ground acceleration on box culverts. This is further demonstrated through calculating the ratio of seismic and static bending moments at selected points on the top slab and side wall to represent the extreme values of this effect. The effects of g-level and predominant frequency of the earthquake on this ratio is observed considering the different earthquakes.

5.5.2.4.1 Effect of g-level on the seismic to static bending moment ratio

Figures 5.24 to 5.29 present the effect of PGA on the ratio of seismic to static bending moments for KEQ, WC, and VC seismic events. Three points are selected on the top slab representing the centre and right and left edges. On the side wall, three points are also selected representing the top, bottom and center. Generally, all points experience increase in the ratio of seismic to static bending moment values.
The edges of the top slab have similar values, while the ratio at the center is small. The ratios obtained for the KEQ and WC earthquakes are similar, while the VC earthquake gave higher ratio values, even though it is characterized by lower PGA values. It is interesting to note that all ratios at both edges are greater than 1.0 and can be as high as 3.0, while the ratio is less than 0.25 at the center.

On the side wall, all ratios are greater than 1.0, with the ratios at the top and bottom points being similar. It is also noted that the ratio at the centre is higher than the ratios at the top and bottom points. The ratios at center can be as high as 5.0, while the ratio at the edges are between 2.0 and 3.0.

Figure 5.24: Effect of g-level on the ratio of seismic to static bending moments due to KEQ on the top slab for Case A of Test 1
Figure 5.25: Effect of g-level on the ratio of seismic to static bending moments due to KEQ on the side wall for Case A of Test 1

Figure 5.26: Effect of g-level on the ratio of seismic to static bending moments due to WC on the top slab for Case A of Test 1
Figure 5.27: Effect of g-level on the ratio of seismic to static bending moments due to WC on the side wall for Case A of Test 1

Figure 5.28: Effect of g-level on the ratio of seismic to static bending moments due to VC on the top slab for Case A of Test 1
5.5.2.4.2 **Effect of predominant frequency on seismic to static bending moment ratio**

Earthquake events are multi-frequency events, which can have a range of frequencies. For the purpose of comparison, the predominant frequency $F_P$ was chosen to represent each earthquake. The predominant frequency $F_P$ values for the VC, WC and KEQ earthquake signals considered are 0.464, 0.647, and 1.453 Hz, respectively. The results from earthquakes with similar g-level (low (L) and medium (M)) are compared to explore the effect of frequency on the ratio.

Figures 5.30 and 5.31 illustrate the effect of frequency on the seismic bending moment values on the top slab, while Figures 5.32 and 5.33 show the effect on the seismic bending moments of the side wall. The results show that noticeable decrease in the moment ratios as the $F_P$ increased from 0.464 to 0.647 Hz, and a smaller decrease as $F_P$ increased to 1.453. The moment ratios at the edges (for both top slab and side wall) are similar. For the top slab, the ratio decreased from 2.2 to 1.2 for L level, and from 2.8 to 1.8 for M level. The ratios for the center were generally less than 1.0. At the side wall, the ratios decreased for the L level from about 5.0 at the center and 2.2 at the top and bottom points, to around 1.1 at the high frequency value. Lower decrease was observed for the M level, as the ratio at center point decreased from 4.5 to 4.0 and the ratio at top and bottom points decreased from 2.8 to 1.8.
Figure 5.30: Effect of earthquake frequency for low g-level on the ratio of seismic to static bending moments on the top slab for Case A of Test 1

Figure 5.31: Effect of earthquake frequency for medium g-level on the ratio of seismic to static bending moments on the top slab for Case A of Test 1
Figure 5.32: Effect of earthquake frequency for low g-level on the ratio of seismic to static bending moments on the side wall for Case A of Test 1

Figure 5.33: Effect of earthquake frequency for medium g-level on the ratio of seismic to static bending moments from FLAC 2D on the side wall for Case A of Test 1
5.6 SUMMARY

Numerical models were developed using a finite difference code (FLAC 2D) to calculate the static and seismic responses of box culvert considering different culvert thicknesses and soil densities. The numerical models were calibrated/verified by comparing their predictions with the measured responses of the static centrifuge model tests and seven earthquake shaking events. The calculated responses agreed well with the centrifuge test results. The following conclusions can be drawn from this study.

The results demonstrate that the higher elastic modulus of sand with higher relative density affected the bending moment distribution on the top slab. The soil pressure distribution on the top slab is parabolic, characterized by high values at the edges and low values at center. The soil-culvert interaction factors followed the same trend. On the side wall, the soil culvert interaction factors were low at the top and high at the bottom.

The seismic bending moment was investigated considering different earthquake events with varying amplitudes and predominant frequencies. The seismic bending moment obtained from the horizontal excitation was compared to the seismic bending moment obtained using the CHBDC procedure. The results show that the CHBDC gave very small seismic bending moment values compared to the computed moments. The results obtained show that the seismic bending moment distribution varies depending on the PGA values and frequency content. It is recommended that the seismic analysis of the culvert is performed considering an earthquake signal representative of the seismicity of the area and the design to be performed considering different levels of the earthquake intensity.
CHAPTER SIX
PARAMETRIC STUDY AND DESIGN GUIDELINES

6.1 INTRODUCTION

The validated numerical model presented in Chapter 5 has been used to perform a comprehensive parametric study to examine several factors that affect the static and seismic soil culvert interaction (SCI). For the static SCI, the parameters investigated include: the soil fill height to culvert width ratio ($H/B_c$); the culvert thickness to width ratio ($t/B_c$); the effect of foundation location on the sand surface; and the change in soil properties, such as soil relative density, friction angle, dilation angle, elastic modulus, and Poisson’s ratio. For the seismic SCI, the parametric study focussed on the effects of $H/B_c$ and $t/B_c$ ratios.

Current design codes (e.g. the CHBDC 2006) recommend a range for the minimum compaction values depending on the type of soil as shown in Table 2.4. The range for soil relative density is usually between 80 to 90% and this means that the soil density is close to the densest state. Therefore the soil parameters presented in Table 5.1 for the 90% relative density were adopted as the base case for the parametric study. A wide range of box culvert thickness values are used in practise. Therefore, the thick culvert case (i.e. $t/B_c = 0.12$) is selected as the base case for all the models in the current study. In case where the effect of different culvert thickness is important, two different thicknesses similar to centrifuge tests were used and referred to as thin ($t/B_c = 0.06$) or thick ($t/B_c = 0.12$) culverts.

Different approaches have been used to define the ratio $H/B$ (i.e. soil height above the culvert, $H$, to culvert width, $B$). The soil height above the culvert is a relatively clear concept, but the culvert width is defined differently by different researchers. Three different width values are used: the external width $B_c$, the center to center width $B_{c-c}$ and the internal width $B$. In the experimental study, the center to center $B_{c-c}$ width was used to model the box culvert. In this numerical study, all three width values were initially considered, but the external width $B_c$ (i.e. $H/B_c$ and $t/B_c$) was considered for the remaining part of the study. In all analysis, the foundation width, $B_F$, is given by its external width.
6.2 STATIC PARAMETRIC STUDY

6.2.1 Effect of $H/Bc$ Ratio:

To investigate the effect of $H/Bc$ ratio on SCI, eight different values covering a range of culvert embedment were considered, i.e. $H/Bc$ ratios of 0, 0.08, 0.38, 0.77, 1.53, 2.30, 3.07, and 6.13. Two different culvert thicknesses (0.267 m and 0.533 m) were used to explore its effect on the SCI factors for the same $H/Bc$ ratio. Several analyses are conducted to demonstrate the effects of $H/Bc$ ratio and culvert thickness on the culvert bending moment, soil pressure distribution and the SCI factors, including comparisons with SCI factors obtained from previous research and design codes.

6.2.1.1 Bending moment

Figures 6.1 and 6.2 present the effects of $H/Bc$ ratio on the bending moment diagrams on the top slab and side wall for culvert thickness of 0.533 m, while Figures 6.3 and 6.4 are for culvert thickness of 0.267 m. The bending moment values of the top slab increase as the $H/Bc$ ratio or culvert thickness increases for both thickness values. For the side wall, as the thickness of the culvert increases the bending moment values increase in the positive direction. The bending moment of the thin culvert is characterized by positive bending moment at the top and bottom corners and negative bending moment at the center, except for very low $H/Bc$ ratios, and the bending moment values increased as the culvert thickness increased.

6.2.1.2 Soil pressure

Soil arching influences the soil pressure distribution on the culvert top slab and side wall, which is in turn affected by the $H/Bc$ ratio. Figures 6.5 and 6.6 present the contour lines of the vertical soil pressure and Figures 6.7 and 6.8 display the horizontal soil pressure contours. The vertical pressure contours on the top slab show some stress concentrations at the culvert edges. The results indicate that soil arching is more pronounced for the thin culvert for all $H/Bc$ ratios. For the thick culvert, the effect of soil arching decreases as the $H/Bc$ ratio increases. The horizontal soil pressures on the thick culvert are less than the case for the thin culvert, which reflects the increased soil arching in this case.

The effects of $H/Bc$ ratio on the soil pressure diagrams on the top slab and side wall are presented in Figures 6.9 to 6.12. Generally, as the $H/Bc$ ratio increases, the soil pressure values
increase for the top slab and side wall. The soil pressure values are affected by the culvert thickness; as the culvert thickness decreases and as the $H/B_c$ ratio increases, the horizontal soil pressure distribution takes a curved shape reflecting a reduction in the horizontal soil pressure at the center of the side wall. This behaviour was observed in the thin culvert, while the thick culvert showed a uniform increase in the horizontal soil pressure with depth.
Figure 6.1: Effect of $H/B_c$ ratio on the bending moment on the top slab ($t = 0.533$ m)

Figure 6.2: Effect of $H/B_c$ ratio on the bending moment on the side wall ($t = 0.533$ m)
Figure 6.3: Effect of $H/Bc$ ratio on the bending moment on the top slab ($t = 0.267$ m)

Figure 6.4: Effect of $H/Bc$ ratio on the bending moment on the side wall ($t = 0.267$ m)
Figure 6.5: Effect of $H/Bc$ ratio on the vertical stresses around box culvert ($t = 0.533$ m) (Legend units in Pa)
Figure 6.6: Effect of $H/Bc$ ratio on the vertical stresses around box culvert ($t = 0.267$ m) (Legend units in Pa)
Figure 6.7: Effect of $H/Bc$ ratio on the horizontal stresses around box culvert ($t = 0.533$ m)

(Legend units in Pa)
Figure 6.8: Effect of $H/B_c$ ratio on the horizontal stresses around box culvert ($t = 0.267 \text{ m}$) (Legend units in Pa)
Figure 6.9: Effect of $H/Bc$ ratio on the soil pressure on the top slab ($t = 0.533$ m)

Figure 6.10: Effect of $H/Bc$ ratio on the soil pressure on the side wall ($t = 0.533$ m)
Figure 6.11: Effect of $H/Bc$ ratio on the soil pressure on the top slab ($t = 0.267$ m)

Figure 6.12: Effect of $H/Bc$ ratio on the soil pressure on the side wall ($t = 0.267$ m)
6.2.1.3 Soil Culvert Interaction factors

Figure 6.13 presents the effect of $H/Bc$ ratio on the SCI factors defined at the edge and center of the culvert top. The ratio $F_e = 1.0$ represents the state at which the soil pressure exactly equals the theoretical soil pressure. The results show that all $F_e$ values for the edge are greater than 1.0 (i.e. soil pressures are larger than the theoretical values). At the center, all $F_e$ values are less than 1.0 (i.e. soil pressures are less than the theoretical soil values). The $F_e$ values at the edge from thin culvert are larger than those from thick culvert, while at the center, the $F_e$ values for the thin culvert are less than for the thick culvert. The SCI factors at the edges increase as the $H/Bc$ ratio increases up to $H/Bc = 0.38$, after that the $F_e$ values decrease as $H/Bc$ increases up to $H/Bc = 1.53$. There is no change in the $F_e$ values for $H/Bc > 1.53$, especially for the thick culvert. The $F_e$ values at the center decrease for $H/Bc$ ratios up to 0.77 and remain almost constant for $H/Bc > 0.77$ in both thick and thin culverts.

Figures 6.14, 6.15, and 6.16 illustrate the effect of $H/Bc$ ratio on the SCI factors on the side wall under the conditions of at rest, active and passive soil pressures. Generally, the effect of $H/Bc$ ratio on the $F_e$ values is only important for $H/Bc < 1.53$, with the thin culvert experiencing higher $F_e$ values than the thick culvert. For the at rest pressure condition, $F_e < 1.0$ at the top corner, except at $H/Bc = 0.77$ where $F_e \approx 1.0$. At the bottom corner, $F_e > 1.0$ for $H/Bc$ up to 1.2 and less than 1.0 for $H/Bc > 1.2$. For the culvert top and bottom corners, $F_e < 1.0$ and increases for $H/Bc \leq 0.77$ and decreases slightly for $H/Bc > 0.77$. Similar trends are observed for the other soil pressure conditions. As expected, the $F_e$ values for the at-rest conditions are lower than the active cases and higher than the passive cases as expected.
Figure 6.13: Effect of the thickness and the ratio $H/Bc$ on the soil culvert interaction factors $F_e$ on the top slab

Figure 6.14: Effect of the thickness and the ratio $H/Bc$ on the soil culvert interaction factors $F_e$ on the side wall at rest pressure $K_o$
Figure 6.15: Effect of the thickness and the ratio $H/Bc$ on the soil culvert interaction factors $F_e$ on the side wall at active pressure $K_a$.

Figure 6.16: Effect of the thickness and the ratio $H/Bc$ on the soil culvert interaction factors $F_e$ on the side wall at passive pressure $K_p$. 
6.2.1.4 Comparison of Soil Culvert Interaction factors with AASHTO and CBDHC

Figure 6.17 compares the SCI factors obtained from the FLAC 2D numerical model at the edge of the top slab with those proposed by AASHTO 2002 and CHBDC 2006. Both codes assume uniform soil pressure on the culvert top slab, while the numerical results demonstrate parabolic distribution with $F_e > 1.0$ at the edges and $F_e < 1.0$ at the center. The comparison is presented herein in terms of $F_e$ at the edges.

Generally, the $F_e$ values proposed by AASHTO and CHBDC are bounded by the calculated $F_e$ values for the thick and thin culverts. All $F_e$ values are presented as a function of $H/Bc$ ratio. It should be noted, however, that the $F_e$ values from CHBDC are constant (i.e. not a function of $H/Bc$), while the $F_e$ values proposed by AASHTO increase linearly for $H/Bc \leq 0.77$ to a maximum of 1.15 and 1.4 for the compacted and uncompacted side fill cases, and remain constant for $H/Bc > 0.77$. The calculated $F_e$ values for the thin culvert are similar to the AASHTO guidelines for $H/Bc \geq 2.4$. At $H/Bc < 0.38$, the calculated $F_e$ values for the thick culvert case are in good agreement with those proposed by AASHTO.

![Figure 6.17: Comparison between the soil culvert interaction factors $F_e$ determined from parametric study with AASHTO and CHBDC](image-url)
6.2.1.5 Comparison of Soil Culvert Interaction factors with other researchers

Several researchers proposed soil culvert interaction factors for different soil height above culvert (e.g. Bennett et al. 2005; Kang et al., 2007; Tadros et al., 1989; and Kim et al. 2005). Figure 6.18 compares the current numerical results with the relationship proposed by Bennett et al. (2005) and Kang et al. (2007) as a function of $H/Bc$, while Figure 6.19 compares the results with the relationship proposed by Tadros et al. (1989) and Kim et al. (2005) as a function of $H$. The relationship proposed by Kang et al. (2007) for compacted and uncompacted side fills shows a reduction in the $F_e$ values as the $H/Bc$ ratio increases, while the relationship suggested by Bennett et al. (2005) shows an increase in the $F_e$ values for $H/Bc \geq 2.42$, and very small increase afterwards. The Bennett et al. (2005) $F_e$ values are overly conservative, especially for $H/Bc > 2.0$. The $F_e$ values provided by the Tadros et al. (1989) and Kim et al. (2005) methods increase as $H$ increases, but the Tadros et al. (1989) method (proposed for silty clay) gives low values for $F_e$ values that fall well below the range obtained in this study for the thick and thin culverts. Kim et al. (2005) relationships for yielding and unyielding foundations provide $F_e$ values that fall between those for thick and thin culvert.

Figure 6.20 compares the results obtained from the current study and some field test data. Several factors may affect the $F_e$ values obtained from the field tests, such as accuracy of pressure transducers used or the soil type and culvert thickness. Despite these factors, most of the test data fall in the range between the thick and thin culverts, and follow similar trends in terms of the increase or decrease with $H/Bc$ ratio.
Figure 6.18: Comparison between the soil culvert interaction factors $F_e$ determined from parametric study with Bennett et al (2005) and Kang et al (2007)

Figure 6.19: Comparison between the soil culvert interaction factors $F_e$ determined from the parametric study with Tadros et al. (1989) and Kim et al (2005)
6.2.2 Effect of \( t/Bc \) Ratio:

Both experimental and numerical modeling results suggest that the culvert thickness affects the bending moment, soil pressure and soil culvert interaction factors significantly. Six different culvert thickness values to cover a range of culvert thicknesses used in practice were considered in the parametric study, i.e. \( t = 0.1, 0.2, 0.3, 0.4, 0.5 \) and 0.6 m, normalized by the culvert width, \( Bc \) to give \( t/Bc = 0.02, 0.04, 0.07, 0.09, 0.11, \) and 0.13. These thicknesses cover a wide range of relative stiffnesses (Figure 3.6). The effects of \( t/Bc \) ratio on the bending moment, soil pressure, and soil culvert interaction factors are explored in the following sections.

6.2.2.1 Bending moment

The effects of \( t/Bc \) ratio on the top slab and side wall bending moment diagrams are presented in Figure 6.21 and Figure 6.22, respectively. Figure 6.21 shows that the top slab bending moments (both positive values at edges and negative values at centers) increase as the \( t/Bc \) ratio increases. The effect of culvert thickness is even greater on the bending moment of the side wall as can be noted from Figure 6.22.
Inspecting Figures 6.21 and 6.22 shows that the bending moments on the culvert top slab and side wall increase with $t/Bc$ but at a decreasing rate. The bending moment diagrams suggest that the ratio of $t/Bc \approx 0.1$ may be considered to be a limiting ratio beyond which very little to no further increase in moment is expected. Similar observation can be made on the effect of $t/Bc$ on soil pressures on the culvert top slab and side wall as discussed below. Hence, it can be suggested that $t/Bc \geq 0.1$ represents the condition of relatively rigid (thick) culvert. On the other hand, at very low $t/Bc$ ratios of 0.02, the bending moment diagram is slightly different than other cases. Inspecting the deformed shape of culverts with different thicknesses can shed some light on that behaviour. It appears that for very low $t/Bc$ ratio all sides of the box culvert deformed inwards (i.e. within the culvert). However, as $t/Bc$ increases the top slab deforms inwards but at a decreasing rate, while the side wall deformation changes gradually from inwards to outwards. These observations suggest that the case of $t/Bc = 0.02$ represents a flexible culvert, which is obvious in the amount of horizontal soil pressure attracted to it as will be shown in next section.

6.2.2.2 Soil pressure

To explore the effect of culvert thickness on the soil arching, different $t/Bc$ ratios were considered. Figure 6.23 present the vertical soil pressure contour lines, while Figure 6.24 display the horizontal soil pressure contour lines. The results show that the soil arching is greater for the thinner culvert. As the thickness of the culvert increases, the effect of soil arching decreases. To have a clearer picture of the effect of soil arching on the soil pressure distribution on the top slab and side wall, Figure 6.25 presents a horizontal section throughout the model above the top slab of the culvert, while Figure 6.26 presents the vertical section passing beside the side wall of the culvert. The obtained results display the effect of soil arching in terms of large increase in the soil pressure (i.e. stress concentration) at the edges and corners and followed by a reduction in the soil pressure towards the center of top slab or side wall. The results also show that the effect of soil arching decreases as the $t/Bc$ ratio increases.

The effect of $t/Bc$ ratio on the soil pressure diagrams on the top slab and side wall are illustrated in Figure 6.27 and Figure 6.28, respectively. The results show that the vertical soil pressure on the top slab decreases at the edges as the ratio of the $t/Bc$ increases and the opposite occurs at the center where the vertical soil pressure increases as the ratio of $t/Bc$ increases. As
the thickness of the culvert increases especially for the ratios of $t/Bc$ higher than 0.09, the vertical soil pressure shows very similar values as the $t/Bc$ ratio increases, while as the thickness of the culvert decreases (i.e. $t/Bc < 0.09$), the difference between the vertical soil pressure increases. Generally, all vertical soil pressure diagrams show that the soil pressure distribution on the top slab takes a parabolic shape with increases at the edge and decreases at the center. This parabolic shape is a function of the thickness of the culvert, i.e., as $t/Bc$ increases, the difference between the vertical soil pressure values at the edges and at the center decreases and vice versa. On the side wall, the horizontal soil pressure for all $t/Bc$ ratios increases with depth and peak values occur at the top and bottom corners. The general trend is that as $t/Bc$ decreases, the horizontal soil pressure increases at the top and bottom corners, while at the center the opposite behaviour is observed, and the horizontal soil pressure decreases.

The effect of $t/Bc$ ratio can be summarized in two distinguishing behaviours that can be separated at the ratio $t/Bc \approx 0.1$. For $t/Bc > 0.1$, all horizontal soil pressures increase with depth, and their values reduce as $t/Bc$ increases. For $t/Bc < 0.1$, large increases in the horizontal soil pressures are observed at the top and bottom corners, while at the center of the side wall the horizontal soil pressure distribution decreases as $t/Bc$ decreases. For very low $t/Bc = 0.02$, the soil pressures on the top slab and side wall are large at the edges, top and bottom corners, and small at the center.
Figure 6.21: Effect of $t/B_c$ ratio on the bending moment on the top slab

Figure 6.22: Effect of $t/B_c$ ratio on the bending moment on the side wall
Figure 6.23: Effect of $t/Bc$ ratio on the vertical stresses around box culvert (Legend units in Pa)

Figure 6.24: Effect of $t/Bc$ ratio on the horizontal stresses around box culvert (Legend units in Pa)
Figure 6.25: Effect of $t/Bc$ ratio on the vertical stresses at the level of the top slab

Figure 6.26: Effect of $t/Bc$ ratio on the horizontal stresses at the level of the side wall
Figure 6.27: Effect of $t/Bc$ ratio on the soil pressure on the top slab

Figure 6.28: Effect of $t/Bc$ ratio on the soil pressure on the side wall
6.2.2.3 Soil Culvert Interaction factors

Figure 6.29 presents the effect of $t/Bc$ ratio for different $H/Bc$ values on the soil culvert interaction factors for the culvert top slab. The results show that for the edge, $F_e > 1.0$ and for the center, $F_e < 1.0$. Generally, $F_e$ for the edge decreases as $t/Bc$ increases, while $F_e$ for the center increase as $t/Bc$ increases. Also, $F_e$ for the edge is much larger than $F_e$ for the center for $t/Bc < 0.1$, and this difference diminishes for $t/Bc > 0.1$.

Figure 6.30 shows the effect of $t/Bc$ ratio on the soil culvert interaction factors for the top and bottom corners of the culvert side wall for different $H/Bc$ values. The results show that for shallow embedment depth ($H/Bc = 0.08, 0.38$) $F_e < 1.0$, while for large embedment ($H/Bc \geq 1.67$) $F_e > 1.0$. At $t/Bc = 0.02$, $F_e$ values for the bottom corner are lower than the top corner. Generally, $F_e$ decreases as $t/Bc$ increases and for thick culverts ($t/Bc > 0.11$), $F_e$ approaches unity.
Figure 6.29: Effect of the thickness ratio $t/Bc$ on the soil culvert interaction factors $F_e$ on the top slab
Figure 6.30: Effect of thickness ratio $t/Bc$ on soil culvert interaction factors $F_e$ on side wall at rest pressure $K_o$. 

(a) $F_e$ values at the top corner of the side wall

(b) $F_e$ values at the bottom corner of the side wall
6.2.2.4 Comparison of Soil Culvert Interaction factors with published literature

Figure 6.31 compares the numerical results with some field test data in terms of $F_e$ values at the edge of the top slab. Even though there is some scatter in the field data, but in general, the numerical and experimental data have approximately the same trend. It is observed though that the experimental $F_e$ values decrease as $t/B_c$ increases. This could be attributed to either the accuracy of the pressure transducers used or the soil type/state.

![Figure 6.31: Comparison between the soil culvert interaction factors $F_e$ determined from parametric study with previous test data](image)

6.2.3 Effect of Foundation Location

To investigate the effect of location of the surface foundation on the culvert behaviour, four different locations were explored. These locations are referred to as $0B_F$, $1B_F$, $2B_F$ and $4B_F$, where $B_F$ is the width of the foundation, which is equal to the width of the box culvert. The $0B_F$ location represents a foundation on the surface right above the location of the box culvert, while $1B_F$, $2B_F$ and $4B_F$ represent a foundation shifted by 1, 2, or 4 times the foundation width. Three values of soil fill height, $H$, were used as a ratio of the foundation width $B_F$ ($H/ B_F = 0.5, 1.0, 1.67$) to investigate the effect of soil embedment. Same foundation pressure was applied at all
locations, equivalent to 100 kPa at the soil surface. The effects of foundation location on the bending moment, soil pressure, and soil culvert interaction factors are discussed below.

6.2.3.1 Bending moment

The effect of foundation location on the bending moment diagrams of the culvert top slab and side wall are presented in Figures 6.32 and 6.33 for the $H/B_F$ ratio of 0.5, Figures 6.34 and 6.35 for the $H/B_F = 1.0$, and Figures 6.36 and 6.37 for the $H/B_F = 1.67$. As expected, the effect of foundation location on the culvert bending moment is most significant for $H/B_F = 0.5$. On the top slab, the bending moment resulting from the $0B_F$ case shows a uniform bending moment distribution with equal values at right and left edges, while for the $1B_F$ case, the bending moment values increased slightly at the left edge and decreased to negative values at the right edge. For foundation locations $2B_F$ and $4B_F$, this effect decreased and the right and left edge bending moment values were positive, but with the left edge having higher values than the right edge. In all three foundation locations, the bending moment at the center was reduced by a considerable amount and the peak value shifted to the right. For $H/B_F = 1.0$, similar results were observed on the top slab, but bending moments at the edges were positive with the left edge values being higher than the right edge values. For $H/B_F = 1.67$, the variation of the bending moment was insignificant, and the peak value shifted to the right. However, the magnitudes of bending moment from all cases were close, which indicates that the effect of foundation location on the culvert is relatively small for deep embedment.

The effect of foundation location on the bending moment of the side wall is significant for $H/B_F = 0.5$; the top corner experienced negative bending moment for $1B_F$, while the bottom corner experienced large positive moment. For $2B_F$ and $4B_F$, varying the foundation location had a small effect on the bending moment. Similar observations are made for $H/B_F = 1.0$ and 1.67, but all bending moments became positive. In all cases, the effect of foundation location is more evident in the upper half of the side wall.
Figure 6.32: Effect of foundation location \((H=0.5B_F)\) on the bending moment on the top slab

Figure 6.33: Effect of foundation location \((H=0.5B_F)\) on the bending moment on the side wall
Figure 6.34: Effect of foundation location \((H=B_F)\) on the bending moment on the top slab

Figure 6.35: Effect of foundation location \((H=B_F)\) on the bending moment on the side wall
Figure 6.36: Effect of foundation location \( (H=1.67B_F) \) on the bending moment on the top slab

Figure 6.37: Effect of foundation location \( (H=1.67B_F) \) on the bending moment on the side wall
6.2.3.2 Soil pressure

The effect of foundation location on the soil pressure on the culvert top slab and side wall are presented in Figures 6.38 and 6.39 for $H/B_F = 0.5$, Figures 6.40 and 6.41 for $H/B_F = 1.0$, and Figures 6.42 and 6.43 for $H/B_F = 1.67$. It is noted that the vertical soil pressure on the top slab increases as $H/B_F$ increases. It is interesting to note that for the case $0B_F$, the vertical soil pressure on the top slab has a uniform distribution for $H/B_F = 0.5$ and 1.0, while for the case of $H/B_F = 1.67$, it is parabolic due to soil arching effects. As expected, cases $2B_F$ and $4B_F$ resulted in the lowest soil pressures, while case $1B_F$ resulted in a noticeable increase in the vertical soil pressure at the right edge (i.e. below the foundation) relative to case $0B_F$ values. For a foundation located at $2B_F$ and $4B_F$, the soil pressures decreased as expected. Similar behaviours were observed for $H/B_F = 1.0$ and 1.67. For $H/B_F = 1.67$, vertical soil pressure diagrams were approximately the same for all foundation locations indicating a marginal effect of foundation location for deep culvert embedment.

On the side wall, all $H/B_F$ cases show an increase in the horizontal soil pressure with depth. As expected, the highest soil pressures for $H/B_F = 0.5$ occurred when the foundation was at $0B$s. For other foundation locations, the horizontal pressures are characterized by a large peak at the top corner and rapid reduction to a lower value, followed by a gradual pressure increase with depth. Similar behaviour was observed for the cases of $H/B_F$ of 1.0 and 1.67, but with another peak at the bottom corner. Again, the effect of foundation location on the horizontal soil pressures is marginal for large culvert embedment ($H/B_F = 1.67$).
Figure 6.38: Effect of foundation location \((H=0.5B_F)\) on the soil pressure on the top slab

Figure 6.39: Effect of foundation location \((H=0.5B_F)\) on the soil pressure on the side wall
Figure 6.40: Effect of foundation location ($H=B_F$) on the soil pressure on the top slab.

Figure 6.41: Effect of foundation location ($H=B_F$) on the soil pressure on the side wall.
Figure 6.42: Effect of foundation location \((H=1.67B_F)\) on the soil pressure on the top slab

Figure 6.43: Effect of foundation location \((H=1.67B_F)\) on the soil pressure on the side wall
6.2.3.3 Soil Culvert Interaction factors

The $F_e$ values were investigated for different $H/B_F$ ratio values. Figures 6.44, 6.45, and 6.46 present the effect of foundation location on the $F_e$ values on the top slab and Figures 6.47, 6.48, and 6.49 show their effect on the side wall for the $H/B_F$ ratios of 0.5, 1.0, and 1.67, respectively. Figures from 6.50 to 6.54 show the effect of foundation location by considering the effect of $H/B_F$ ratio.

Generally, the $F_e$ values on the top slab and side wall increase as $H/B_F$ increases. For $H/B_F = 0.5$ and 1.0, $F_e < 1.0$, but for deep embedment ($H/B_F = 1.67$), $F_e > 1.0$. For all $H/B_F$ values, $F_e < 1.0$ except for deep culvert embedment where $F_e > 1.0$.

Figure 6.44: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the top slab ($H/B_F = 0.5$)
Figure 6.45: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the top slab ($H/B_F = 1.0$)

Figure 6.46: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the top slab ($H/B_F = 1.67$)
Figure 6.47: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the side wall at rest pressure $K_o$ ($H/B_F = 0.5$)

Figure 6.48: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the side wall at rest pressure $K_o$ ($H/B_F = 1.0$)
Figure 6.49: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the side wall at rest pressure $K_o$ ($H/B_F = 1.67$)

Figure 6.50: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the top slab (Left Edge)
Figure 6.51: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the top slab (Right Edge)

Figure 6.52: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the top slab (Center)
Figure 6.53: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the side wall at rest pressure $K_o$ (Top)

Figure 6.54: Effect of the Foundation location on the soil culvert interaction factors $F_e$ on the side wall at rest pressure $K_o$ (Bottom)
6.2.4 Effect of Soil Density

Three different relative densities were considered in the analysis to investigate the effect of soil density on the bending moment, soil pressure and soil culvert interaction. These three relative densities are 40, 70 and 100%. The results obtained showed that the effect is marginal on bending moments, soil pressures and SCI factors.

6.2.5 Effect of Soil Elastic Modulus

The elastic modulus, $E_s$, of fine sand (similar to sand used in the centrifuge tests) ranges from 8 to 30 MPa, and for coarser sands it may range from 10 to 80 MPa depending on the density level (Hunt, 2005). To investigate the effect of $E_s$ on the culvert behaviour, five values were considered in the analysis, i.e., $E_s = 10, 20, 30, 40, \text{ and } 100$ MPa.

6.2.5.1 Bending moment

Figure 6.55 shows the effect of $E_s$ on the bending moment diagrams. Generally, the effect of $E_s$ on the bending moment diagrams is noticeable at the center of the top slab and side wall and the bending moment decreases as $E_s$ increases. At the edges, the bending moments are almost the same for all $E_s$ values.

6.2.5.2 Soil pressure

Figure 6.56 shows the effect of $E_s$ on the soil pressure diagrams. The vertical soil pressure distribution on the top slab is parabolic and the values at the edges increase and the values at the centre decrease with an increase in $E_s$. On the side wall, all horizontal soil pressures increase with depth and increase as $E_s$ increases.

6.2.5.3 Soil Culvert Interaction factors

Figure 6.57 shows the effect of $E_s$ on the SCI factors. It is noted that $F_e$ values increase at the edge and decrease at the center linearly as $E_s$ increases. Therefore, linear functions were used to fit these relations. Equations 6.1 and 6.2 describe the variation of $F_e$ with $E_s$, for the edge and centre, respectively.

\[
F_e = 0.0011 E_s + 1.0428 \tag{6.1}
\]

\[
F_e = -0.002 E_s + 0.999 \tag{6.2}
\]
On the side wall, $F_e$ for both the top and bottom corners increase linearly as $Es$ increases. Therefore, linear functions were used to fit these relations yielding Equations 6.3 and 6.4 for $F_e$ at top and bottom corners, respectively.

$$F_e = 0.0008 Es + 1.0242$$ (6.3)

$$F_e = 0.0007 Es + 1.0459$$ (6.4)

The above analyses was repeated for different $H/Bc$ ratios and yielded same results for different culvert embedment ratios.

(a) top slab

(b) side wall

Figure 6.55: Effect of soil elastic modulus on the bending moment on the top slab and side wall

(a) top slab

(b) side wall

Figure 6.56: Effect of soil elastic modulus on soil pressure on the top slab and side wall
6.2.6 Effect of Poisson’s Ratio

To investigate the effect of Poisson’s ratio on culvert behaviour, four values of Poisson’s ratio were used, covering the range of Poisson’s ratio for sand, i.e. 0.2, 0.25, 0.3, and 0.35.

6.2.6.1 Bending moment

Figure 6.58 shows the effect of Poisson’s ratio on the bending moment diagrams for top slab and side wall. The results show that as Poisson’s ratio increases, the positive bending moment at the edges decreased and the negative bending moment at the center increased. Similar shift of the bending moment on the side wall was observed.

6.2.6.2 Soil pressure

Figure 6.59 shows the effect of Poisson’s ratio on the soil pressure diagrams for top slab and side wall. Generally, Poisson’s ratio has no effect on the vertical soil pressure on the top slab, but has a significant effect on the horizontal pressure on the side wall. As Poisson’s ratio increases, the horizontal soil pressure increases. It is important to note that FLAC 2D uses Poisson’s ratio to calculate the horizontal earth pressure coefficient.

6.2.6.3 Soil Culvert Interaction factors

Figure 6.60 shows the effect of Poisson’s ratio on the soil culvert interaction factors for top slab and side wall. Generally, Poisson’s ratio effect on $F_e$ values for the top slab is moderate,
but its effect is strong on $F_e$ values for the side wall. On the top slab, $F_e$ at the edge decrease linearly as Poisson’s ratio increases, while at the center $F_e$ increases linearly as Poisson’s ratio increases. The Poisson’s ratio-$F_e$ relationships at the edge and center can be represented by linear function as shown in Equations 6.5 and 6.6.

\[ F_e = -0.1585 \nu + 1.1414 \]  

(6.5)

\[ F_e = 0.1316 \nu + 0.8811 \]  

(6.6)

The $F_e$ values on the side wall increase as Poisson’s ratio increases. For Poisson’s ratios = 0.2 and 0.25, $F_e < 1.0$, while for Poisson’s ratios = 0.3 and 0.35, $F_e > 1.0$. Second order polynomial functions gave the best fit for the Poisson’s ratio-$F_e$ relationships at the top and bottom corners, i.e. Equations 6.7 and 6.8.

\[ F_e = 6.923 \nu^2 + 1.5278 \nu + 0.1406 \]  

(6.7)

\[ F_e = 7.4664 \nu^2 + 1.2559 \nu + 0.1921 \]  

(6.8)

Figure 6.58: Effect of Poisson’s ratio on the bending moment on the top slab and side wall
6.2.7 Effect of Shear Strength Parameters

Four different friction angle values were used to investigate the effect of friction angle on the culvert behaviour, i.e., 30°, 35°, 40° and 45°. In addition, the analysis was repeated for 3 different dilation angles, i.e., 0°, 5° and 10°. The results indicated that there is no effect for the shear strength parameters on the bending moment values or soil pressure distribution indicating that the behaviour remained within the elastic range.
6.3 SEISMIC PARAMETRIC STUDY

A liner element was used to model the structural parts, i.e., the culvert. These elements are capable of simulating the bending moment history, but are unable to produce the soil pressure history at each node during the earthquake shaking. In addition, there was no soil pressure data collected during the seismic centrifuge tests to calibrate the numerical model. It also appears that there is no method available to calculate the seismic soil pressure on a box culvert. However, the CHBDC (2006) recommends evaluating the seismic bending moment as the product of static bending moment and the vertical component of peak ground acceleration.

The strongest Kobe earthquake event with a peak ground acceleration (PGA) of 0.31g and predominant frequency of 1.453 Hz (KEQH) was used in the FLAC 2D models to investigate the seismic behaviour of box culverts. The effects of $H/B_c$ and $t/B_c$ ratios on the static, seismic and total bending moment values were evaluated. Only the effect of $H/B_c$ ratio on the kinematic soil culvert interaction was investigated because the centrifuge tests results indicated that the culvert thickness had a minor effect.

6.3.1 Effect of $H/B_c$ Ratio:

The effect of $H/B_c$ ratio on the seismic bending moment obtained from the numerical models was evaluated in comparison with the CHBDC (2006) provisions for seismic moments.

6.3.1.1 PGA for Free Field (FF) and Structure Field (SF)

Figure 6.61 presents the peak ground acceleration PGA values for the Free Field (FF) and Structure Field (SF) at the same locations as Ac2 (FF) and Ac7 (SF), which were used in centrifuge tests. Generally, the results show that the PGA values for FF and SF decrease as $H/B_c$ increases. For shallow culvert embedment ($H/B_c < 1.67$), the PGA for FF was much higher than for SF, and that PGA values for FF decrease sharply as the $H/B_c$ ratio increases. For higher $H/B_c$ ratios, a small reduction was observed in the PGA as the $H/B_c$ ratio increased. For SF, there was a gradual decrease in the PGA values as $H/B_c$ increased. For $H/B_c \geq 2.30$, the effect of kinematic interaction is very small (i.e. difference between PGA for FF and SF is very small).
6.3.1.2 Comparison between seismic bending moment and CBHDC

Figures 6.62 and 6.63 compare the seismic bending moments obtained from the CHBDC provisions (considering vertical acceleration at the culvert level (Culvert) and at the model base (Base)), with those obtained from the FLAC 2D seismic model for the top slab and side wall, respectively. Similar results were observed for all \( H/Bc \) ratios, therefore only the results for \( H/Bc = 0 \) are presented here and the remainder results are shown in Appendix (W).

The seismic bending moments obtained considering the Culvert and Base vertical accelerations using the CHBDC procedure are close. The seismic bending moments calculated from the numerical model (which is based on the horizontal acceleration) are much higher than those obtained from the CHBDC provision, and the shape of the bending moment envelop is different. The CHBDC seismic bending moment has the same distribution as the static bending moment, but the seismic bending moment obtained from the FLAC 2D model on the top slab has a positive value at the left edge and negative value at the right edge with some irregular changes at the centre. The same behaviour was observed for seismic bending moments of the side wall. The ratio between the seismic bending moments on the top slab obtained numerically and those from CHBDC varied between 130 for \( H/Bc = 1.53 \) and 60 for \( H/Bc = 0 \). At the left edge of the top slab, this ratio is 25 for \( H/Bc = 0 \) and 4 for \( H/Bc = 6.13 \).
Figure 6.62: Comparison of computed seismic bending moment from FLAC 2D and CHBDC due to KEQH on the top slab for $H/Bc = 0$

Figure 6.63: Comparison of computed seismic bending moment from FLAC 2D and CHBDC due to KEQH on the side wall for $H/Bc = 0$
6.3.1.3 Comparison between static, seismic and total bending moment

The static, seismic and their summation (total) bending moments are required for design purposes. All three values were plotted in Figures 6.64 and 6.65 for the top slab and side wall, respectively. Depending on the $H/Bc$ ratio, the bending moment values may change, but they follow the same trend. Therefore, only the case of $H/Bc = 0$ is presented herein and other cases are shown in Appendix (X).

Figures 6.66 and 6.67 show that the shapes of seismic bending moments are a function of the $H/Bc$ ratio. The seismic bending moment on the top slab is characterized by a positive value at one end and a negative value at the other end. At the center, the seismic bending moment approaches zero as $H/Bc$ increases, except for $H/Bc = 0.08$, where the shape of the seismic bending moment drops to the negative side and then increases to the positive side. In most $H/Bc$ cases, the maximum positive and negative seismic bending moments at the left and right edges respectively control the shape of the total bending moment and that causes the positive bending moment to be greatest at the left edge and decrease to the negative side at the right edge. This occurs for all of the $H/Bc$ cases except $H/Bc = 6.13$.

On the side wall, all of the $H/Bc$ cases show a positive static bending moment for the values throughout the side wall depth. From all of the $H/Bc$ cases, it was observed that seismic bending moment started with negative values at the top corner and then increases towards the positive side at the bottom corner. All of the $H/Bc$ cases gave similar shapes for the seismic bending moment, except the cases of $H/Bc$ of 0 and 0.08, which demonstrates some irregular changes near the centre. The shape of total bending moment on the side wall displays a negative value at the top corner and a positive value at the bottom corner, except for $H/Bc = 6.13$ where the total bending moment at the top corner is also positive.

Figures 6.68 and 6.69 illustrate the effect of $H/Bc$ ratio on the total bending moment obtained on the top slab and side wall, respectively. The shape of the total bending moment follows the shape of the seismic bending moment. The results show that the total bending moment at the left edge increases as $H/Bc$ increases. At the right edge, the total bending moment was negative except for $H/Bc = 6.13$, which had a small positive value. The total bending moment along the slab is negative and increases as the $H/Bc$ ratio increases. The total bending moment at the top corner of side wall was negative except for $H/Bc = 6.13$, which had a small positive value. At the bottom corner, as $H/Bc$ increases the total bending moment increases.
Figure 6.64: Computed static, seismic and total bending moments on the top slab for $H/Bc = 0$

Figure 6.65: Computed static, seismic and total bending moments on the side wall for $H/Bc = 0$
Figure 6.66: Effect of $H/Bc$ ratio on the seismic bending moments on top slab

Figure 6.67: Effect of $H/Bc$ ratio on the seismic bending moments on side wall
Figure 6.68: Effect of $H/Bc$ ratio on the total bending moments on the top slab

Figure 6.69: Effect of $H/Bc$ ratio on the total bending moments on the side wall
6.3.1.4 Static, seismic and total bending moment design observations

Structural design usually considers bending moment at representative points. The representative points selected are the top slab left and right edges and the center point. For the side wall, the representative points are the top and bottom corners and the center (mid-height).

Figures 6.70 and 6.71 present the effect of the $H/Bc$ ratio on the static bending moment at the representative points on the top slab and side wall, respectively. The static bending moment values of the left and right edges of the top slab are exactly the same, and they increase as $H/Bc$ increases. Similarly, the magnitude of the negative static bending moment at the top slab centre increases. On the side wall, top, bottom and center bending moments increase as $H/Bc$ increases.

Figures 6.72 and 6.73 present the effect of $H/Bc$ on the seismic bending moments at the top slab and side wall, respectively. As $H/Bc$ increases, the seismic bending moment values at the top slab edges increase, but the rate of increase is lower for $H/Bc > 2.30$. At the centre, the seismic moment increases with $H/Bc$ for $H/Bc \leq 2.30$, then decreases slightly afterwards. On the side wall, as $H/Bc$ increases the seismic bending moments at the top and bottom corners increase linearly for $H/Bc \leq 2.30$ and remains almost constant afterwards.

Figures 6.74 and 6.75 present the effect of $H/Bc$ ratio on the total bending moments on the top slab and side wall, respectively. On the top slab, the total bending moments at the left edge and at the centre increase as $H/Bc$ increases. At the right edge, the total bending moments increase for $H/Bc \leq 1.53$, and decreases afterwards and even changes sign for deep embedment. On the side wall, the total bending moments at the bottom corner increase as the $H/Bc$ ratio increases. At the top corner, the total bending moment initially increases then decreases and change sign for higher embedment ratios. At the center, the total bending moments are small and the effect of $H/Bc$ is moderate.

Figures 6.76 and 6.77 present the effect of $H/Bc$ the ratio of seismic to static bending moments ($BM_{dy}/BM_{st}$) at the representative points on the top slab and side wall, respectively. This ratio can be helpful in determining the seismic bending moment considering the static bending moments, similar to the approach incorporated in the CHBDC (2006).

On the top slab, $BM_{dy}/BM_{st}$ ratios for the left and right edges are close. As $H/Bc$ increases, $BM_{dy}/BM_{st}$ decreases rapidly for as $H/Bc \leq 0.38$, after which $BM_{dy}/BM_{st}$ continues to decrease but at a lower rate. At the center, $BM_{dy}/BM_{st}$ decreases gradually as $H/Bc$ increases. The $BM_{dy}/BM_{st}$
ratio at the center ranges from 0.80 at $H/Bc = 0$ to almost zero at $H/Bc = 6.13$. These results indicate the large effect of soil height above the top slab of box culvert. At low $H/Bc$ ratios, the seismic bending moment is much higher than the static bending moment and may reach five times its value. However, as the soil height increases $BM_{dy}/BM_{st}$ can decrease to below 1.0. This can explain the reduced effect of earthquakes on deeply buried box culverts, in which case the seismic bending moment values are less than static bending moment.

On the side wall, the $BM_{dy}/BM_{st}$ ratios for top, bottom and center decrease as $H/Bc$ ratio increases. However, $BM_{dy}/BM_{st}$ for the top and bottom corners is much less than that at the center. The $BM_{dy}/BM_{st}$ ratio of the center is higher than that for the top and bottom corners. At very low $H/Bc$ ratio, there was an increase in $BM_{dy}/BM_{st}$, followed by a steep reduction in $BM_{dy}/BM_{st}$ ratio. At very low $H/Bc$, the seismic bending moment at the mid-section of the side wall reaches approximately ten times the static bending moment. During previous earthquakes, there was large cracks observed on side walls (e.g. Youd and Beckman, 1996), which could be explained by the large seismic moments. For deeply embedded culverts, the $BM_{dy}/BM_{st} \leq 1.0$, i.e., the seismic bending moment could be less than the static bending moment. Hence, deeply buried box culverts designed for static bending moment only could survive the seismic bending moment, due to the large safety factors involved in the structural design. It is important to note that the $BM_{dy}/BM_{st}$ ratio can be positive or negative depending on the actual sign of the static and seismic bending moment values.
Figure 6.70: Effect of $H/Bc$ ratio on the static bending moment from FLAC 2D on the top slab

Figure 6.71: Effect of $H/Bc$ ratio on the static bending moment from FLAC 2D on the side wall
Figure 6.72: Effect of $H/Bc$ ratio on the seismic bending moment on the top slab

Figure 6.73: Effect of $H/Bc$ ratio on the seismic bending moment on the side wall
Figure 6.74: Effect of $H/B_c$ ratio on the total bending moment from on the top slab

Figure 6.75: Effect of $H/B_c$ ratio on the total bending moment on the side wall
Figure 6.76: Effect of $H/Bc$ ratio on ratio of seismic to static bending moment on the top slab

Figure 6.77: Effect of $H/Bc$ and ratio of seismic to static bending moment on the side wall
6.3.2 Effect of $t/Bc$ Ratio:

The effect of $t/Bc$ ratio on the seismic bending moment diagrams was investigated in comparison with the seismic bending moments obtained from CHBDC. This investigation involved evaluating static, seismic and total bending moment as well as the ratio of seismic to static bending moments.

6.3.2.1 Comparison between seismic bending moment and CHBDC

Figures 6.78 and 6.79 compare the seismic bending moment obtained from CHBDC procedures with those obtained from the FLAC 2D seismic model for a culvert with $t/Bc = 0.02$ subjected to the KEQH. The results for other $t/Bc$ ratios are shown in Appendix (Y).

The CHBDC results obtained considering the Culvert and Base vertical accelerations are close. The seismic bending moment obtained numerically was found to depend on the culvert thickness ratio ($t/Bc$). The seismic bending moments obtained numerically for $t/Bc = 0.02$ and 0.04 have similar trends to those obtained from CHBDC. The results for $t/Bc = 0.02$ for the top slab and side wall are in good agreement with those obtained from the CHBDC. As discussed before, the bending moment increases as the thickness increases. For large $t/Bc$ ratios, the seismic bending moment increases significantly and becomes larger than that obtained from the CHBDC procedure. For $t/Bc > 0.04$, the shape of the seismic bending moment changes. The seismic bending moments on the side wall become all positive for $t/Bc > 0.04$. The ratio of seismic bending moments obtained numerically and those from CHBDC for the left edge on the top slab increases as $t/Bc$ increases. This ratio can be 2.5 to 10 for $t/Bc = 0.13$. 
Figure 6.78: Computed seismic bending moments on the top slab for $t/B_c = 0.02$

Figure 6.79: Computed seismic bending moment on the side wall for $t/B_c = 0.02$
6.3.2.2 Comparison between static, seismic and total bending moment

The effect of $t/Bc$ ratio on the static, seismic and total bending moment values are plotted in Figures 6.80 and 6.81 for the top slab and side wall, respectively, for $t/Bc = 0.02$. The bending moment values may change as $t/Bc$ increases, but all cases investigated displayed the same trends. Therefore, only the case of $t/Bc = 0.02$ is presented herein and the other cases are shown in Appendix (Z).

The top slab static bending moment diagram is symmetrical and has two equal positive values at the edges and negative value at the center. The seismic bending moment distributions for $t/Bc = 0.02$ and 0.04 are similar to the static bending moment distribution. For $t/Bc > 0.04$, the seismic bending moment at the right edge changes sign, while the moment at the center is positive. Accordingly, the total bending moment at the left edge is higher than the static moment, while the right edge total seismic moment may increase or decrease. At the center, the static bending moment is higher than the seismic bending moment and hence, the total bending moment remains negative.

On the side wall, the static bending moment has two positive bending moment values at the top and bottom corners and negative bending moment at the center for $t/Bc = 0.02$ and 0.04. For $t/Bc > 0.04$, all static bending moment values are positive. The total positive bending moments at top and bottom corners and total negative bending at the mid-height of the side wall increase. For $t/Bc > 0.04$, the seismic bending moment at the top corner changes sign and the total bending moment at top corner also changes sign.

Figures 6.82 and 6.83 present the effect of $t/Bc$ ratio on the top slab and side wall seismic bending moments, respectively. The seismic bending moment at the left edge of top slab increases as $t/Bc$ increases. At the right edge, the seismic bending moment values for $t/Bc = 0.02$ and 0.04 are positive, while for $t/Bc > 0.04$, all seismic bending moment values are negative. At the left edge, the seismic bending moment is positive and changes sign as $t/Bc$ increases. On the side wall, the seismic bending moment increases as $t/Bc$ increases. For $t/Bc = 0.02$ and 0.04, the seismic bending moment at the top and bottom corners are positive. For $t/Bc > 0.04$, the seismic bending moment at the top decreases with depth.

Figures 6.84 and 6.85 show the effect of $t/Bc$ ratio on the total bending moment obtained on the top slab and side wall, respectively. The top slab total bending moment shows high positive values at the left edge and their values increase as $t/Bc$ increases. At the center, all total
bending moment values were negative and also increase as \( t/Bc \) ratio increases. At the right edge, for \( t/Bc = 0.02 \) and 0.04, the positive total bending moment increases, while for \( t/Bc > 0.04 \) the bending moment values increase as the \( t/Bc \) ratio increases. On the side wall, the top corner total bending moment is lower than the seismic bending moment. At the bottom corner, the values of the total bending moment are very high, since both of the static and seismic bending moments are positive.

![Figure 6.80: Computed static, seismic and total bending moments on the top slab for \( t/Bc = 0.02 \)](image)

Figure 6.80: Computed static, seismic and total bending moments on the top slab for \( t/Bc = 0.02 \)

![Figure 6.81: Computed static, seismic and total bending moments on the side wall for \( t/Bc = 0.02 \)](image)

Figure 6.81: Computed static, seismic and total bending moments on the side wall for \( t/Bc = 0.02 \)
Figure 6.82: Effect of $t/B_c$ ratio on the seismic bending moments on the top slab

Figure 6.83: Effect of $t/B_c$ ratio on the seismic bending moments on the side wall
Figure 6.84: Effect of $t/Bc$ ratio on the total bending moments on the top slab

Figure 6.85: Effect of $t/Bc$ ratio on the total bending moments on the side wall
6.3.2.3  Static, seismic and total bending moment design observations

Figures 6.86 and 6.87 show the effect of t/Bc ratio on the static bending moment on the top slab and side wall, respectively. The static bending moment at the left and right edges are equal and increase as t/Bc increases. The center static bending moment also increases as t/Bc increases. On the side wall, the static bending moment values for the top and bottom corners for t/Bc = 0.02 are almost identical. For t/Bc > 0.02, the static bending moments increase. At the mid-height of the side wall, the static bending moment values are negative for t/Bc < 0.08. For t/Bc > 0.08 static bending moments are positive and increase.

Figures 6.88 and 6.89 show the effect of the t/Bc ratio on the seismic bending moment on the top slab and side wall, respectively. On the top slab, the seismic bending moment at the left edge increases as t/Bc increases. At the right edge, the seismic bending moment increases only for t/Bc = 0.02 and 0.04. For t/Bc > 0.04, the seismic moments decrease. The seismic bending moment at the center start with small negative values at t/Bc = 0.02 and then change to positive value and continue to increase as the t/Bc ratio increases. On the side wall, the seismic bending moments at the top and bottom corner have the same trend as those of the top slab. At the mid-height of the side wall, the seismic bending moment decreases as the t/Bc ratio increases.

Figures 6.90 and 6.91 show the effect of the t/Bc ratio on the total bending moment at on the top slab and side wall, respectively. On the top slab, the total bending moment values at the left edge are positive and increase as t/Bc increases. At the right edge, the total bending moment increases up to t/Bc = 0.04 and then decreases and becomes zero at t/Bc ≈ 0.06. All total bending moments at the center are negative and increase as t/Bc increases. On the side wall, the total bending moments are high at the bottom corner and increase as t/Bc increases.

The effects of t/Bc ratio on BM_{dy}/BM_{st} are evaluated for the top slab and side wall for representative points and are presented in Figures 6.92 and 6.93. The relationship between t/Bc and BM_{dy}/BM_{st} can be helpful in determining the seismic bending moment based on the static bending moment.

On the top slab, the BM_{dy}/BM_{st} ratios for the left and right edges have the same trend and increase as t/Bc increases. For t/Bc ratios < 0.07, BM_{dy}/BM_{st} < 1.0. For t/Bc > 0.07, BM_{dy}/BM_{st} > 1.0. At the center, all BM_{dy}/BM_{st} ratios are less than 0.30, which indicates that the seismic bending moments are small.
On the side wall, the $BM_{dy}/BM_{st}$ ratios of the top and bottom corner have the same trend and increase as $t/Bc$ ratio increases. At the center of the side wall, $BM_{dy}/BM_{st} < 1.0$ for $t/Bc = 0.02$ and 0.04. For $t/Bc > 0.04$, $BM_{dy}/BM_{st}$ increases sharply as $t/Bc$ increases. The $BM_{dy}/BM_{st}$ ratio can be as high as eight times the static bending moment. It is important to note that the $BM_{dy}/BM_{st}$ ratio can be positive or negative.

Figure 6.86: Effect of $t/Bc$ ratio on the static bending moment on the top slab

Figure 6.87: Effect of $t/Bc$ ratio on the static bending moment from FLAC 2D on the side wall
Figure 6.88: Effect of $t/Bc$ ratio on the seismic bending moment from FLAC 2D due to KEQH on the top slab

Figure 6.89: Effect of $t/Bc$ ratio on the seismic bending moment from FLAC 2D due to KEQH on the side wall
Figure 6.90: Effect of $t/Bc$ ratio on the total bending moment on the top slab

Figure 6.91: Effect of $t/Bc$ ratio on the total bending moment on the side wall
Figure 6.92: Effect of $t/Bc$ ratio on ratio of seismic to static bending moment on the top slab

Figure 6.93: Effect of $t/Bc$ ratio on ratio of seismic to static bending moment on the side wall
6.4  STATIC AND SEISMIC DESIGN GUIDELINES FOR BOX CULVERT

The results of the static and seismic centrifuge and numerical models presented in this research underscore several factors that may affect the design of square box culverts. In this section, some potential design guidelines interpreted from these results are suggested to aid in the design of box culverts under the effect of static and seismic loads.

6.4.1  Static Design Guidelines

For the static design of a box culvert, the main focus is on the static soil pressure, which can be used to determine the internal forces of the structural members of the box culvert. These internal forces are required for designers to have a safe and economic design. Several factors affect the shape and value of the soil pressures including: soil properties, \( \frac{H}{B_c} \) and \( \frac{t}{B_c} \) ratios, as well as the elastic modulus of the culvert material. In addition, the presence of any surface foundation in the vicinity of the culvert can affect the design parameters. The proposed static design guidelines are based on these factors and are summarised in the following sections.

6.4.1.1  Soil pressure distributions

The actual shape of the vertical soil pressure on the culvert top slab is parabolic. By defining the pressures at the edges and centre, the resulting parabola can describe the actual vertical soil pressure diagram on the top slab. The horizontal soil pressure diagram on the side wall generally shows an increase in their values with depth. The increase in the horizontal soil pressure is linear for the thicker culverts, while for the thinner culverts the horizontal soil pressure diagram takes a curvy shape at the mid-section similar to a parabola. This parabolic shape increases with increases in the soil height above the culvert. In general for simplicity, by defining the top and bottom values of the horizontal soil pressure, the horizontal soil pressure diagram on the side wall can be considered to linearly increase with depth as the curvy shape in the mid-section of the side wall appears more under high soil pressure columns above the culvert or very small culvert thicknesses are not normally used in design.

6.4.1.2  Soil pressure values

To determine the soil pressure on the top slab, the theoretical vertical soil pressure \( \sigma_v \) is evaluated based on the soil unit weight \( \gamma_s \) and the height of soil column \( H \) above the culvert.
The earth pressure coefficient $K$ is required to evaluate the theoretical horizontal soil pressure $\sigma_h$ on the side wall as shown in Equations 6.9 and 6.10, respectively.

\[
(\sigma_y)_{\text{Theoretical}} = \gamma_s \cdot H
\]  
(6.9)

\[
(\sigma_h)_{\text{Theoretical}} = K \cdot \gamma_s \cdot H
\]  
(6.10)

Once the soil culvert interaction factor $F_e$ is determined, the design vertical and horizontal soil pressures can be calculated using Equations 6.11 and 6.12, respectively.

\[
(\sigma_v)_{\text{Actual}} = F_e \times (\sigma_v)_{\text{Theoretical}}
\]  
(6.11)

\[
(\sigma_h)_{\text{Actual}} = F_e \times (\sigma_h)_{\text{Theoretical}}
\]  
(6.12)

### 6.4.1.3 Soil culvert interaction factors

Several factors that affect the soil culvert interaction factor $F_e$ can be divided into three main groups:

**Group 1**: factors related to the geometry and material of the box culvert and the surrounding soil, which determine the relative stiffness of the culvert and surrounding soil. These factors include the ratios of the height of soil fill above the culvert to its width ($H/B_c$) (i.e. external pressure) and the ratio of the thickness of the culvert to its width ($t/B_c$) (i.e. relative stiffness of culvert). It should be noted that the ratio $H/B_c$ was investigated for two different thicknesses, while the $t/B_c$ ratio was investigated for several $H/B_c$ ratios.

**Group 2**: factors related to the extra surcharge from the soil surface on the box culvert structural members. These factors include different locations and pressure of surface foundation. Even though moving loads such as live loads (from vehicles) are beyond the scope of this study, a moving point or distributed load on the surface should be considered to evaluate the effect of these loads on the soil pressure. All foundation models were investigated for single $t/B_c$ and $H/B_c$ ratios.
**Group 3:** factors related to the soil properties around the box culvert. These factors include soil elastic modulus \((Es)\), Poisson’s ratio \((\nu)\), soil density \((\rho)\), friction angle \((\phi)\), and dilation angle \((\psi)\). All soil properties models were investigated for single \(t/Bc\) and \(H/Bc\) ratios except soil elastic modulus \((Es)\) and Poisson’s ratio \((\nu)\) where the effect of deep soil fill is also investigated.

The soil culvert interaction factor \(F_e\) has been investigated and several charts have been provided for the edge and center of the top slab, as well as the top and bottom corners of the side wall to determine the soil culvert interaction factor \(F_e\).

It was found that the \(F_e\) value is a function of \(H/Bc\) ratio (external pressure) and \(t/Bc\) ratio (i.e. culvert relative stiffness). Therefore, a three dimensional plots were produced for the top slab (edge, center) and side wall (top and bottom) to show the combined effect of \(H/Bc\) and \(t/Bc\) ratios on \(F_e\) values and the location where \(F_e\) values can be expected to be high or low as presented in Figure 6.94.

### 6.4.2 Seismic Design Guidelines

Throughout this study, the seismic soil pressure was not possible to obtain, and therefore the focus was only on determining the seismic bending moment either experimentally or numerically. The analysis was performed using three different earthquakes having different amplitudes and frequencies. The three earthquakes used were Kobe earthquake, where the earthquake signal used is identical to a real earthquake, while the Western Canada and Vancouver Cascadian are artificial. Therefore the one used for the parametric study is the one representing a real earthquake. The results from all earthquakes show that there is no single shape for the seismic bending moment that can be observed from all the earthquakes and therefore it is recommended to run a complete nonlinear dynamic numerical analysis to get the real behaviour of the box culvert under specific conditions. The seismic parametric study was performed under certain conditions such as the PGA of the acceleration time history applied at the base of the model which was in the range of 0.3g for the Kobe earthquake. However, the seismic design guidelines can be summarised in the following sections.
Figure 6.94: Three Dimensional representation of the $F_e$ values as a function of $H/Bc$ and $t/Bc$ ratios.

### 6.4.2.1 Shape of the seismic bending moment

The shape of the seismic bending moment was investigated from the centrifuge and numerical models. The results show that there is no single shape for the seismic bending moment. The shape of the seismic bending moment is changing with the earthquake signal used and depends on its PGA value. As the PGA levels increase, the shape of the seismic bending moment will change and their values increase and in some cases may transition from positive to negative. However, all the seismic bending moment shapes obtained from all the earthquakes show large moment values either positive or negative at the edges and corners, while in between these points the seismic bending moment values are less.
6.4.2.2 The seismic bending moment values

The seismic bending moment values obtained from horizontal earthquake shakings seems to be very large compared to the procedure recommended by the CHBDC, and therefore it is not recommended to rely on that procedure for seismic design.

The results show that seismic bending moment values are having high values close to the edges or corners and reduce in between them. The $BM_{dy}/BM_{st}$ ratios increase with the increase in PGA values from all the earthquakes, and their values are higher than 1.0 which indicates that the seismic bending moment is higher than the static bending moment. This behaviour is observed more close to the culvert edges and corners, while at the center, the $BM_{dy}/BM_{st}$ ratios are less than 1.0 which shows the opposite behaviour. The relation between the $BM_{dy}/BM_{st}$ ratios and the frequency of the earthquake show that the increase in the frequency causes a reduction in the seismic bending moment.

The $BM_{dy}/BM_{st}$ ratios obtained from the seismic parametric study are very useful in defining the seismic bending moment based on the static bending moment value and the ratios of $H/Bc$ and $t/Bc$. Those charts can be helpful in two ways: one way is by using the defined $H/Bc$ and $t/Bc$ ratios directly to find the $BM_{dy}/BM_{st}$ ratio and the other way is by giving overview picture of their behaviour and allowing the designer to choose the right values for the height of soil fill and culvert thickness. The general behaviour of the change in $BM_{dy}/BM_{st}$ ratios with the $H/Bc$ ratio indicates that as $H/Bc$ ratio increases, there is a clear reduction in the $BM_{dy}/BM_{st}$ ratio. This indicates that as the height of the soil fill above the culvert increases, the seismic bending moment will decrease to a level where the static bending moment has the highest value. The behaviour of the change in $BM_{dy}/BM_{st}$ ratios with the $t/Bc$ ratio is the opposite of $H/Bc$. As the thickness of the culvert increases, the $BM_{dy}/BM_{st}$ ratios increase too, and that lead to the conclusion of an increase in the seismic bending moment values as the thickness of the culvert increases.

6.4.3 Design Example

A simple step by step example has been included in this chapter to demonstrate the procedure for obtaining the static soil pressure, static bending moment and seismic bending moment on the top slab of box culvert using the soil culvert interaction factors. For the purpose
of this example, a box culvert width $Bc = 4.572$ m, its thickness $t = 0.533$ m and the height of soil fill $H = 7.62$ m is assumed.

**Step 1**: Calculate the $H/Bc$ and/or $t/Bc$ ratios, and use Figure 6.13 and/or Figure 6.29 to determine the soil culvert interaction factor value $F_e$. The results obtained are $H/Bc = 1.67$, $t/Bc = 0.12$ and the $F_e$ values at the left and right edges of the top slab is 1.08 and at the center is 0.92.

**Step 2**: Calculate the theoretical vertical soil pressure using Equation 6.9. Using a soil unit weight of 16.56 kN/m$^3$ the theoretical vertical soil pressure is 126.2 kPa.

**Step 3**: Calculate the actual soil pressure on the top slab using Equation 6.11. The actual soil pressure at both edges is 135.87 kPa and at the center is 116.09 kPa.

**Step 4**: Use the three static soil pressures calculated at the edges and center to fit a 2nd order polynomial. The fitted equation can be integrated twice to obtain the actual static shear force and static bending moment diagrams on the top slab as shown in Figure 6.95. The resulting fitted and integrated equations are:

$$W = 4.9505x^2 + 3 \times 10^{-13}x + 116.09 \quad (6.13)$$

$$SF = 1.65017x^3 + 1.5 \times 10^{-13}x^2 + 116.09x \quad (6.14)$$

$$BM = 0.4125x^4 + 5 \times 10^{-14}x^3 + 58.045x^2 - 132.06 \quad (6.15)$$

**Step 5**: Use Figures 6.76 and 6.92 to obtain the ratio of the seismic to static bending moment ratio $BM_{dy}/BM_{st}$ based on $H/Bc$ and/or $t/Bc$ ratios. The value obtained for the $BM_{dy}/BM_{st}$ ratio at the left edge is 2.0 and at right edge is -2.05, while at the center is -0.28. Multiply these ratios by their equivalent static bending moment values, the seismic bending moment can be obtained. Based on these ratios, the resulted seismic bending moment is 221.8 kN.m/m at the left edge, -227.34 kN.m/m at the right edge and 37.15 kN.m/m at the center of the top slab.

The same procedure can be used to determine the static and seismic bending moments on the side wall. For the side wall the horizontal soil pressure distribution should be fitted to linear equation in order to obtain static bending moment as 3rd order polynomial fit. It is also important to note that the $F_e$ values obtained from the $H/Bc$ and $t/Bc$ charts are for $Es = 30$ MPa and $\nu = 0.28$. The effect of different soil parameters has been examined and it was found that there is no
effect for the $H/Bc$ ratio. Therefore, Equations 6.1 to 6.8 can be used to adjust the $F_e$ value to different values of soil elastic modulus and Poisson’s ratio.

![Graph of vertical soil pressure, shear force, and bending moment vs. distance](image)

Figure 6.95: The resulting static soil pressure, shear force and bending moment diagrams.
6.4.4 **General Recommendation**

Based on the results of the total bending moment diagrams obtained from the static and seismic parametric studies, it is recommended to design for each of the static and seismic bending moment separately and then combine both designs. This is because the total bending moment depends on the sign of each one, which may lead to a large reduction in the total bending moment values at some locations and huge increase in their values at other locations. This might affect the design especially for the static cases where the total bending moment can show a large reduction in their static bending moment values.

6.4.5 **Kinematic Soil Culvert Interaction**

The centrifuge results illustrate the kinematic interaction effect of the box culvert by comparing the Free Field PGA with the Structural Field PGA values. The results show that as the PGA at the base of the model increases, the reduction in the PGA at the Structure Field increases. This has led to about 50% reduction in the PGA at the Structure Field compared to those at the Free Field. This observation is very important and it can be very helpful in assessing the seismic hazard of buildings where large underground structures exist.

6.4.6 **Racking of Box Culvert**

Using the racking deformations for seismic design of box culverts was the only method available in literature (Wang, 1993) and this procedure can be used for pseudo-static analysis. The centrifuge results of the racking deformations indicate that the racking deformation is not always as suggested by the racking method. In the case of a very dense soil with a thick culvert, the racking ratio was less than 1.0, which indicates that the deformations at the free field are higher than those at the structure field. For the other cases where the soil is medium dense with a thick culvert or the culvert is thin (irrespective of density), the racking ratio is larger than 1.0 and in some cases very high. This proves that the racking deformations at the structure field are much higher than the free field. In some cases the difference between the deformations at the top and bottom slabs of the culvert and in the free field are negative, which indicate that the values at the top is less than those at the bottom, therefore this method should be used with caution.
6.5 SUMMARY

The validated model presented in Chapter 5 was used to perform a static and seismic parametric study to investigate the effect of several factors that may affect the soil culvert interaction factors. The geometrical parameters were used in both static and seismic parametric studies as they have large effects on the results, while the different locations of surface foundations as well as the different soil properties were used in the static parametric study only. The following conclusions can be drawn from these analyses.

The effect of the geometric parameters is presented in the form of dimensionless numbers $H/Bc$ and $t/Bc$. The ratio $H/Bc = 1.53$ can be considered to be a limiting ratio for the top slab and side wall because there is no change in the $F_e$ values after that, while before that it is variable. The $F_e$ values obtained from the $H/Bc$ ratio increases at the edges and decreases at the center of the top slab as the culvert thickness decreases. Similar observation was made for $H/Bc$ ratio on the side wall. Similar observations to the $H/Bc$ ratio were made in terms of the thickness effect. Even though the $F_e$ values at the center of the top slab are less than 1.0, it was observed that the $F_e$ values increase as the $t/Bc$ ratio increase. At the edge of the top slab and the top and bottom corners of the side wall, it was observed that the $F_e$ values decrease as the $t/Bc$ ratio increase.

The change in the foundation location on the soil surface and the culvert depth shows an influence on the $F_e$ values. The $F_e$ values increase as the $H/BF$ ratio increases and as the depth of soil cover increases. The soil density, friction angle, and dilation angle have no effect on the $F_e$ values. The $F_e$ values increase as the elastic modulus increases at the edges of the top slab and top and bottom corners of the side wall, while at the center of the top slab, the $F_e$ values decrease as the elastic modulus values increase. The $F_e$ values decrease at the edge of the top slab as Poisson’s ratio increase, while at the center of the top slab as well as the top and bottom corners of the side wall, the $F_e$ values increase as Poisson’s ratio values increase. Several charts showing the effect of these factors were produced to show their effect on the $F_e$ values.

Kinematic soil culvert interaction appears to be more pronounced for $H/Bc < 3.07$. The FF and SF PGA values were observed to decrease as the $H/Bc$ ratios increase up to 3.07 and after that there is very little difference. This indicates that the kinematic interaction appears to be more significant when the height of the soil fill above the culvert is less than three times the width of the culvert.
The seismic bending moment based on the CHBDC provides very small values compared to the values obtained from the numerical analysis and that is because of the direction of the earthquake shaking (i.e. vertical component for CHBDC and horizontal for the FLAC). This was investigated for different $H/Bc$ and $t/Bc$ ratios and the only exception that was noticed is for $t/Bc = 0.02$. At that thickness, the results gave a good agreement with the CHBDC results.

The shape of the seismic bending moment is not unique but changes with the earthquake excitation and its amplitude which can change its values from positive to negative. The results show that the seismic bending moment values are high at the edges of the top slab and corners of the side wall, while in between, the values decrease at the center of the top slab and side wall with more fluctuation on the side wall.

The shape of the total bending moment is controlled by the seismic bending moment shape. If the sign of the static and seismic bending moments are the same, this will increase the total bending moment and vice versa.

The ratio between the seismic to static bending moment $BM_{dy}/BM_{st}$ ratio is proposed to be used to determine the seismic bending moment. This ratio was investigated to see the effect of $H/Bc$ and $t/Bc$ ratios. It was found that the $BM_{dy}/BM_{st}$ ratio increases as the $t/Bc$ ratio increases, and decreases as the $H/Bc$ ratio increases.

Using the charts obtained from the static and seismic effects of the different $H/Bc$ and $t/Bc$ ratios, preliminary design guidelines were provided considering the shape and values of static soil pressure which can be used to produce the static bending moment. Using the static bending moment and the $BM_{dy}/BM_{st}$ ratio, the seismic bending moment can be obtained. These guidelines were followed by design example to clarify a step by step procedure.
CHAPTER SEVEN
SUMMARY AND CONCLUSION

7.1 RESEARCH SUMMARY AND FINDINGS

This dissertation describes the results of a comprehensive experimental and numerical study of the effect of soil arching for static and seismic soil culvert interaction. The research consisted of two major components: (1) a series of static and seismic scaled physical centrifuge model tests for box culverts with two different wall thicknesses, located within a dry cohesionless soil (in a “wished in place condition”) with two different relative densities; (2) numerical modeling of the static and seismic centrifuge model tests, as well as comprehensive static and seismic parametric studies. A summary of the research and the main findings are described in the following sections.

7.1.1 Centrifuge Modeling

A series of centrifuge tests were conducted at the RPI centrifuge facility in the USA to examine soil culvert interaction under static as well as seismic loadings. The results from these tests show that the soil culvert interaction during static loading is a function of the self-weight of the soil, any extra surcharge from foundations on the surface and the relative stiffness of the culvert compared to the soil. The effect of two different culvert thicknesses and two different soil relative densities were investigated. A total of seventy earthquake shaking events were applied for ten cases using a one dimensional shaker to apply these shakings in the horizontal direction in-flight. Several instruments and measuring devices were used to fully instrument each centrifuge test. Generally, the instruments were used to capture the response of the soil and the culvert during the static and seismic phases of each test; these were strain gauges, LVDTs, tactile pressure sensors and accelerometers. One of the biggest challenges during the installation of these instruments was choosing the appropriate method to install the strain gauges on the inside faces of the top slab and side wall of the box culvert. Therefore, a viable method was developed and is introduced in this thesis. Details of each step in the procedure have been provided.
7.1.1.1 **Dynamic soil properties**

Proper evaluation of the shear modulus and damping ratio is a key factor for accurate seismic analysis. The centrifuge acceleration time histories recorded for the free-field location for two shaking levels of the Western Canada earthquake (under two soil relative densities) were investigated to examine the shear modulus and damping ratio of the soil and their degradation with shear strain. The effect of filtering of the acceleration time history records on the cycling loops and thereafter its effect on the shear modulus and damping ratio values were also investigated. Several filtering ranges were examined to obtain the correct shape of the displacement time history to be used for shear strain calculations. The conclusion from these analyses has lead to the adaption of using the minimum filtering range that can produce the correct shape of displacement time history. That shape is what is recommended to use to calculate the shear strain time history, while the original (unfiltered) acceleration time history is recommended for use to calculate the shear stress time history. Following this procedure has the advantage of obtaining the actual shear stress values and a more reasonable shape of the cyclic loops, as well as a reduction in the damping scatter that was observed by researchers in the past.

7.1.1.2 **Kinematic soil culvert interaction**

The effect of kinematic soil culvert interaction during the earthquake shakings (due to the box culvert buried inside the soil) is to decrease the ground input motion by a considerable amount. This is due to the interference of the relatively rigid structure with the propagation of the seismic waves and its inability to conform to the soil movements. This led to reductions in the peak ground acceleration (PGA) values for the Structure Field, in comparison to the Free Field. The effect of sand density on the amplification of PGA values was clearly observed for the Free Field condition, while it was not as significant for the Structural Field condition. Also, there was no significant effect of the thickness of (relative stiffness) the culvert on the PGA values for the Structural Field.

The amplitude of earthquake at the model base is an important factor in determining how much reduction occurs in the PGA at the ground surface for the Structure Field compared to the Free Field. As the amplitude of an earthquake increases, the reduction in PGA increases. The results from the Kobe earthquake at PGA = 0.3g show that the reduction in the PGA values can reach up to 50%. Also, the kinematic interaction effect appears as the amplitude of the
earthquake exceeds 0.1g. This means that the presence of buried underground structures will have minimal kinematic interaction effect if the earthquake has a PGA less than 0.1g.

The observations made in this study may be helpful when assessing the seismic hazard for existing buildings overlying significant underground structures (e.g. box culverts or tunnels). It may also be helpful when evaluating the input ground motion for the purpose of performance based design of buildings overlying significant underground structures.

7.1.1.3 **Response of structures to earthquake excitations**

The rocking of structures was investigated by calculating their rocking angles. The rocking angle for the foundation and box culvert was found to be very small and changes with the PGA of the ground input motion. Even though the rocking angles were very small, the values for the foundation are higher than those of the box culvert. This behaviour is expected due to the confinement of the surrounding soil around the box culvert which is not the case of the foundations on the surface.

The lateral movement of the surface foundations was investigated through the displacement time history that was derived from the acceleration time history recorded at the top of the foundation. The results show that the peak ground displacement (PGD) and the PGA at the top of the foundations increased as the PGA of the ground input motion at the base of the model increased.

The racking ratio of the box culvert (which represents the ratio between the differential movements at the top and bottom slabs for the Free Field and Structural Field) was investigated based on the centrifuge results. The racking deformation obtained indicates that for the case of the very dense soil with thick culvert, the racking ratios are less than 1.0, which indicates that the deformations for the Free Field are higher than those of the Structure Field. For the case of medium dense soil with thick culvert the racking ratio is larger than 1.0. On the other hand, the thin culvert with different soil densities shows a racking ratio also larger than 1.0 but with very high values. In some cases the difference between the deformations of the top and bottom slabs of the culvert and the Free Field are negative, which indicate that the values of the top are less than those at the bottom.
7.1.1.4 Soil culvert interaction parameters

The static results have been presented in terms of bending moment and soil pressure, and these have been used to investigate the effect of foundation, soil density, and culvert thickness. The effect of the foundation shows that the bending moment as well as the soil pressure increase as moving from case A (with no foundation on the surface) to cases B and C (where the strip foundation is placed on the surface). The case D (where the rectangular foundation is placed on the surface) shows little effect as the strip foundation and their results appear similar to case A. This can be attributed to the culvert depth, the foundation contact surface area, and the pressure bulb under the foundation. The effect of soil density appeared to be opposite to that expected. As the density increased, the bending moment and soil pressure obtained from the strain gauges decreased. This behaviour was investigated numerically and the reason appears to be the change in the elastic modulus of the soil from a medium dense to very dense state. It was found also that as the culvert thickness decreased the soil pressure and bending moment also decreased.

The soil pressures measured using the tactile pressure sensors were similar to the strain gauge result when assessing the effect of a foundation. This showed the distribution of the vertical soil pressure on the top slab to be parabolic with high values at the edges and low values at the center. The parabolic shape is found to be a function of the thickness of the culvert and height of soil column above the culvert, as well as the size of the box culvert. The horizontal soil pressure on the side wall shows an increase in value with depth and this reduces as the foundation pressure on the surface increases. The soil pressure measured from case D (where there was a rectangular foundation on the surface) show similar results to case A, where there was no foundation on the surface. The soil pressure also increases as the soil density and the culvert thickness increases.

Most of the soil pressures obtained from the strain gauge and tactile pressure sensors show good agreement. For the thick culvert, the strain gauge soil pressures range between the maximum and minimum soil pressures from the tactile sensors, while for the thin culvert the soil pressure from strain gauges close to the maximum at edges and close to the minimum at center. This suggests that the tactile pressure sensors provide bounds for the soil pressures, even though they do not account for any shear stress effects.

The soil culvert interaction factors indicate the variation between the measured soil pressures from the theoretical soil pressures. These were investigated at certain points and it was
found that they increase at the edges and decrease at center of the top slab, while on the side wall their values were low at top and high at bottom. It was also noticed that a decrease in culvert thickness causes a greater increase in the soil culvert interaction factors at the edges and further decreases at center of the top slab.

The seismic results were shown only in terms of the seismic bending moment, as it was not possible to measure the soil pressure directly. The results obtained lead to the conclusion that the seismic bending moment increases as the PGA of the input motion increases and the thickness of the culvert increases.

7.1.2 Numerical Modeling

A numerical model was developed using the two dimensional finite difference code (FLAC 2D), to predict the static and seismic responses of the soil culvert interaction. The numerical model was calibrated and then verified by comparing its predictions with the measured responses of all static centrifuge tests and all earthquake shaking events for one case of the centrifuge tests. Excellent agreement was found between the measured and computed static results, while very good agreement was obtained between the numerical simulations of the seismic results and those obtained from the centrifuge model. The validated models were then used to perform static and seismic parametric studies to investigate the effect of several factors that may affect the soil culvert interaction factors under static loads and the ratio of seismic to static bending moments under the effect of earthquake shakings. The following conclusions can be drawn from these analyses.

7.1.2.1 Static analysis

Several factors were investigated during the static parametric study. These parameters include geometric parameters, foundation locations, and different soil properties. The investigation includes bending moment, soil pressure, and soil culvert interaction factors. The effect of the geometric parameters is presented in the form of dimensionless numbers $H/Bc$, and $t/Bc$. The results show that the ratio $H/Bc = 1.53$ can be considered to be a limiting ratio for the top slab and side wall, where the $F_e$ values after that is constant and before that is variable. It was observed that the $F_e$ values obtained from the $H/Bc$ ratio increases at the edges of the top slab as the culvert thickness decreases, while at the center, the $F_e$ values decrease as the culvert
thickness decrease. Similar observation was made for $H/Bc$ ratio on the side wall. Even though the $F_e$ values at the center of the top slab are less than 1.0, it was observed that the $F_e$ values increase as the $t/Bc$ ratio increase. Looking at the edge of the top slab and the top and bottom corners of the side wall, it was observed that the $F_e$ values decrease as the $t/Bc$ ratio increase.

There are clear differences in the $F_e$ values depending on the foundation location on the soil surface and the depth of soil cover over the culvert. The $F_e$ values increase as the $H/Bc$ ratio increases and as the depth of soil cover increases. The soil density, friction angle, and dilation angle have no noticeable effect on the $F_e$ values and have a constant $F_e$ value for the $H/Bc$ and $t/Bc$ conditions that were used in the investigation. Changing elastic modulus shows an increase in the $F_e$ values at the edges of the top slab and top and bottom corners of the side wall as the elastic modulus values increase, while at the center of the top slab, the $F_e$ values decrease as the elastic modulus values increase. Changing Poisson’s ratio shows a decrease in the $F_e$ values at the edge of the top slab as Poisson’s ratio increase, while at the center of the top slab as well as the top and bottom corners of the side wall, the $F_e$ values increase as Poisson’s ratio values increase.

Based on the static analysis performed for the above factors, several charts were created and equations were derived to help define the $F_e$ values at specific extreme points on the box culvert for different cases.

7.1.2.2 Seismic analysis

The effect of $H/Bc$ ratio on the kinematic soil culvert interaction was investigated and it was found that the PGA values for both the Free Field and Structural Field decrease as the $H/Bc$ ratio increases. Reductions in PGA were observed for all of the $H/Bc$ ratios less than 3.07, while for larger values, there is very little difference between them. This suggests that the effect of kinematic interaction effect appears to be more significant when the height of the soil fill above the culvert is less than three times the width of the culvert.

The seismic bending moment results from the entire earthquake shaking events show that the procedure recommended by the CHBDC based on the vertical acceleration component provides very small seismic bending moment values compared to the bending moment obtained from a horizontal earthquake excitation. The only exception that was noticed is when the
thickness of the culvert is very small ($t/Bc = 0.02$). At that thickness, the results gave a good agreement with the CHBDC results.

There is no single characteristic shape for the seismic bending moment and the shape of the seismic bending moment was found to change with the earthquake excitation and its amplitude which, can change values from positive (at specific PGA values) to negative for different cases. Despite this fact, all of the results show that the seismic bending moment values are high at the edges of the top slab and corners of the side wall, while in between, the values decrease at the center of the top slab and side wall with more fluctuation on the side wall.

It was observed from the comparison between the static, seismic and total bending moment diagrams that the shape of the total bending moment is controlled by the seismic bending moment shape. If the sign of the static and seismic bending moments are the same, this will increase the total bending moment while if they are different, then it will decrease the total bending moment; in some cases this will be lower than the static bending moment. This can lead to mistakes in design, and therefore it is recommended to design for each case of static and seismic bending moments separately and then combine both designs.

For seismic design, it was proposed to use the ratio between the seismic to static bending moment. This ratio was investigated to see the effect of PGA, predominant frequency, $H/Bc$ ratio and $t/Bc$ ratio. It was found that the $BM_{dy}/BM_{st}$ ratio increases with the PGA of the input motion, as well as the $t/Bc$ ratio increase, and decreases with the predominant frequency and $H/Bc$ ratio increase.

Based on the static and seismic parametric studies, several design guidelines were proposed. These design guidelines may be helpful for determining the actual soil pressure distribution around box culverts under static loads. Once these have been obtained, the static bending moment can be easily calculated and using the seismic to static bending moment ratio, the seismic bending moment due to horizontal shaking can also be determined. All of these aspects are shown in a step by step design example.

7.2 RECOMMENDATIONS

The geotechnical centrifuge facility has been shown to be a very useful tool for investigating small scale physical models for a range of engineering problems. Centrifuge tests can also be used to investigate the behaviour of the soils and for soil culvert interactions as well.
The main four test series presented in this thesis consisted of fourteen static tests and ten seismic tests, which included seventy earthquake shakings for sandy soils and instrumented structures (box culverts and foundations).

The results obtained from these tests strongly support further centrifuge testing and numerical modeling to investigate more cases related to box culverts that were not studied. The box culverts used in this work had a uniform thickness for all structural members; however, this is not always the case in practice. Therefore it would be interesting to perform similar static and seismic tests on box culverts that have the top and bottom slabs thicker or thinner than the side walls. Also, in some cases, the side walls have the same thickness, while the top slab is thinner than the bottom slab, which also needs further investigation. In practice, box culverts are not only placed as single cells, but also installed as twin, double or triple box culverts cells depending on their use. Investigation of such combinations is still very limited in the literature, at a field experimental level or for centrifuge and numerical modeling; further work is therefore recommended.

The soil used in the research was dry sand with different densities. It would be interesting to investigate the effect of saturated and partially saturated sand, since these are more realistic materials available during the construction of box culverts. Also, investigating different types of soil, such as clay or mixed soils would give a broader prospective of their effects on the behaviour of box culverts under static and seismic loads. On a practical level, the foundation soil under the box culvert may be different than the soil used as the backfill on the side walls and above the top slab, and therefore investigating layered soils that have different properties may be useful to investigate the response of the box culvert and the soil culvert interaction under the effect of different soil configurations.

One of the uses of box culverts is for construction under highways, where many traffic loads can pass above them, and these can cause effects in terms of extra static loads, as well as vibrations and cyclic loads. In this research, the moving of a foundation on the sand surface shows the effect on the soil culvert interaction, and this shows the importance of investigating the effect of both the surcharge loads on the soil surface and the travelling loads, such as live loads of vehicles on the soil surface. Even though investigating such loads was outside the scope of this research, it is important to investigate them experimentally and numerically.
The relative stiffness between the box culvert material and the surrounding soil may have significant effects on the soil culvert interaction behaviour. Investigating this relative stiffness is very important for culvert construction using different materials, such as steel, concrete, and aluminum, as well as different soil types having different stiffness values. This will give more insight into the effect of relative stiffness on the responses of box culverts.

The method of installing the box culvert also has important effects on the soil culvert interaction. Most of the box culverts investigated in the literature focus on embankment installation, while the trench method has had little attention. In practice, both methods are used, as well as the imperfect trench method. Therefore, investigating all three methods and providing conclusions showing their effects on soil culvert interaction factors are of great interest.

For the seismic case, more research needs to be performed on the centrifuge and in numerical models, to study liquefaction, lateral spreading, and fault location, as these can cause very large deformations and lead to box culvert and other related infrastructure failures.

The results of the kinematic soil culvert interaction obtained from the centrifuge tests of this thesis provided a promising approach for decreasing the effect of peak ground accelerations for the Structural Field compared to those of the Free Field. This observation needs more investigation in terms of the shape of the buried structure, its width, its height, and either hollow or solid structures on reducing the PGA of the Structure Field. In addition, the kinematic effect of the surface foundation should also be taking into consideration as it increases the reduction in PGA values. During these studies, geotechnical software that can produce contours of the PGA values throughout the soil profile at both Free and Structural Fields would be helpful.

The tactile pressure sensors used in this research provided good representation of the soil pressure around box culvert. Using high speed tactile pressure sensors that can capture the seismic soil pressure time history over very short time periods would also provide a good understanding of the seismic soil pressures around box culverts. If such sensors can give trustworthy dynamic soil pressures, their results can be used to derive equations that may be used to calculate the dynamic soil pressures, since no relations are currently available in the literature, especially for deeply buried structures.
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APPENDIX (A)

DEVELOPED PROCEDURE FOR STRAIN GAUGE INSTALLATION INSIDE A SQUARE BOX CULVERT TUBE

This appendix presents a step by step the newly developed procedure for installing the strain gauges inside the box culvert. It shows the parts that were manufactured and the method of using them to insert and place the strain gauges in their correct position. Also, the type of glue used to stick the strain gauges and the procedure for applying the required pressure.
STRAIN GAUGE INSTALLATION PROCEDURE

The developed procedure for installing the strain gauges inside the culvert model is explained in further detail in the following steps:

1. Prepare the inside surface of the square culvert model tube using Carbird 400 grit paper on either sides (top and side) of strain gauge locations. Then complete cleaning the surfaces with conditioner and neutralizer as shown in Figure A.1.

2. Attach a lip under the transfer plate to fix the plate to the tube when ready to install gauges as shown in Figure 3.23 (a). Note that this lip edge of plate has to meet the thickness of the aluminum tube.
3. Place the transfer plate on a clean glass plate and attach 30 cm lengths of PCT-2M tape to the transfer plate by overlapping the transfer plate by 5 cm. Overlap the PCT-2M tape to form a flexible sheet and at the same time attach the tape to the glass surface as shown in Figure A.2.

Figure A.2: Strain gauge transfer plate attached to a glass plate using PCT-2M tape

4. Mark the location of all of the strain gauges on the surface of the tape using permanent ink pen showing one common centerline and individual gauge centerlines 90 degrees to the common line as shown in Figure A.3.
Figure A.3: Marking strain gauge locations
5. Put the 15 mm thick Light Weight Plastic Rubbing Plate on the transfer plate and lift the tape carefully and turn it upside down with the transfer plate onto the glass as shown in Figure A.4. Fold the tape over the rubbing plate carefully.

![Figure A.4: Positioning the Light Weight Plastic Rubbing Plate over the Strain Gauge Transfer Plate](image)

6. Mount the strain gauges to the tape as shown in Figure A.5, with the dull bonding surface of strain gauge facing away from the tape. The transfer plate aligns and sets distance of the gauges from the end of the tube. To install the strain gauges on the tape at any location, round nose tweezers can be used, and then rub down the lead wires.
Figure A.5: Mounting and checking the strain gauges on the tape
7. Re-clean the gauge area and 100 mm beyond the centerline away from the plate with conditioner and neutralizer.

8. Prepare one mix of AE-10 adhesive as per manufacture instructions. Use a new soft toothbrush with an extended arm. Transfer adhesive with the cleaned mixing stick to the toothbrush and apply to the gauging area. Ensure full coverage and 2 cm beyond gauge towards brush end.

9. Hold the transfer plate and the rubbing plate, with the PCT-2M tape and the gauges against the curved end of the plate and flat on top surface of the rubbing plate.

10. Remove the glass plate and locate the transfer plate inside the culvert model tube.

11. Lift rubbing plate and PCT-2M tape together ensuring no slack between the tape and the curved end of the plate, and hold at 20-30 degree angle.

12. Hold the tape with one hand and slide the rubbing plate along the surface allowing strain gauges to slip around the radius gradually onto the surface of the tube as shown in Figure A.6. Rub down the PCT-2M tape to bring the lead wires down onto the surface of the tube all the way to the end of the tube and allow epoxy to saturate the lead wires. Hold plate at an angle to keep pressure on the lead wires.

Figure A.6: Positioning the strain gauges inside the culvert model
13. Install a strip of TFE-1 Teflon tape crosswise over the gauge area and a portion of the lead wire to protect the gauges as shown in Figure A.7.

14. Apply small rubber pieces over each gauge and a one inch strip of SGP-2 silicone rubber crosswise over all the strain gauges as shown in Figure A.7.

15. Tape down and apply a backup block of 6.35 mm thick aluminum strip.

16. Apply about 137.89 kPa air pressure on the strain gauges using the test ball as shown in Figure A.8 and leave it for 24 hours.
17. After 24 hours remove the test ball, backup plate, silicone strip and Teflon tape. Peel back PCT-2M tape carefully from lead wire end. Pull the tape parallel to tube surface to expose the strain gauges and release the transfer plate.

18. Inspect gauge area with mirror and lighting to ensure integrity of installation. There should be no bubbles under the gauges as shown in Figure A.9. Then, check the resistance of the strain gauges to make sure that they are all working and no damage has happened to them.
Figure A.9: Inspecting strain gauges after installation
19. When all gauges are installed, the process of soldering the lead wires to the outside gauges is started as shown in Figure A.10.

Figure A.10: Soldering the lead wires to the outside strain gauges
20. After soldering all of the lead wires, connect the inside and outside wires to the DAQ as shown in Figure A.11.
21. Once all of the gauges are installed and checked inside and outside the tube, clean the gauge area with rosin solvent and apply the protective coating and then recheck each strain gauge again as shown in Figure A.12.

Figure A.12: Protective coating applied to the strain gauges
APPENDIX (B)

LVDT SETTLEMENT MEASUREMENTS

This appendix presents the LVDT measured settlements during the spin up of the centrifuge from 1g to 60g which represent the static part of the centrifuge tests. The appendix also shows the LVDT measurements during each single earthquake shaking in all the four tests and their entire cases, which represent the displacement time history at the soil surface and foundation top.
Figure B.1: Settlement recorded from 1g to 60g for Tests 1 and 2 (Model Scale): (a) Test 1 Case A, (b) Test 1 Case B, (c) Test 1 Case C, (d) Test 2 Case A, (e) Test 2 Case B, (f) Test 2 Case C.
Figure B.2: Settlement recorded from 1g to 60g for Tests 3 and 4 (Model Scale): (a) Test 3 Case A, (b) Test 3 Case B, (c) Test 3 Case C, (d) Test 3 Case D, (e) Test 4 Case A, (f) Test 4 Case B, (g) Test 4 Case C, (h) Test 4 Case D.
Figure B.3: Settlement recorded at 60g for Test 1 Case A (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.4: Settlement recorded at 60g for Test 1 Case C (Prototype Scale) Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.5: Settlement recorded at 60g for Test 2 Case A (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.6: Settlement recorded at 60g for Test 2 Case C (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.7: Settlement recorded at 60g for Test 3 Case A (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.8: Settlement recorded at 60g for Test 3 Case C (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.9: Settlement recorded at 60g for Test 3 Case D (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high.
Figure B.10: Settlement recorded at 60g for Test 4 Case A (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
Figure B.11: Settlement recorded at 60g for Test 4 Case C (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high.
Figure B.12: Settlement recorded at 60g for Test 4 Case D (Prototype Scale): Western Canada earthquake (a) low, and (b) medium; Vancouver Cascadian earthquake (c) low and (d) medium; Kobe Earthquake (e) low, (f) medium and (g) high
APPENDIX (C)

ACCELERATION, VELOCITY, AND DISPLACEMENT TIME HISTORIES, WITH THEIR FREQUENCY CONTENT AND RESPONSE SPECTRUM FOR TEST 2 - CASE C

Due to the large amount of acceleration time history data collected from all the accelerometers used in all the four tests and their entire cases, the results presented in this appendix are only for Test 2 Case C. The results presented here are only from three accelerometers Ac2, and Ac7 which represent the Free Field and Structural Field motions close to the soil surface and AcBaseH1 which represent the base motion. For each earthquake shaking event, the results present the acceleration, velocity and displacement time histories as well as their frequency content and response spectrum.
Figure C.1: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake WCL: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
Figure C.2: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake WCM: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
Figure C.3: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake VCL: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
Figure C.4: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake VCM: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
Figure C.5: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake KEQL: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
Figure C.6: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake KEQM: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
Figure C.7: Acceleration, velocity, and displacement time histories with their frequencies and spectral responses for Test 2 Case C and the earthquake KEQH: Time histories of (a) acceleration, (b) velocity, and (c) displacement; Frequency content of (d) Acceleration, (e) velocity, and (f) displacement; and Response spectrum of (g) Acceleration, (h) velocity, and (i) displacement. Ac2 represent Free Field, Ac7 represent Structural Field and AcBaseH1 represent the base of the model.
APPENDIX (D)

PEAK GROUND ACCELERATION, VELOCITY, AND DISPLACEMENT PROFILES

This appendix presents the peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD) profiles. Each figure represents one test case and shows the effect of different earthquake shaking events on the amplification of acceleration, velocity and displacement through out the soil profile. Two profiles were compared that represent the Free Field and Structural Field and how the presence of the box culvert affects this amplification.
Figure D.1: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 1 Case A, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.2: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 1 Case C, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.3: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 2 Case A, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.4: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 2 Case C, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.5: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 3 Case A, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.6: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 3 Case C, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.7: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 3 Case D, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.8: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 4 Case A, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.9: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 4 Case C, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
Figure D.10: Profile of PGA, PGV, and PGD for all earthquakes considered in Test 4 Case D, Free Field (FF) and Structural Field (SF) profiles from Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement.
This appendix illustrates the peak ground acceleration, velocity and displacement obtained around the box culvert in all the tests and their cases. Four accelerometers were used around the box culvert; two of them were placed in the horizontal direction above and below the culvert and referred to as Ac9 and Ac12 respectively; and the other two were installed vertically to record the vertical acceleration time history resulted from horizontal excitation, and those referred to as Ac10 and Ac11 on the left and right side of the culvert respectively. The results of each pair were compared to explore the effect of the culvert on the results as well as their results compared to the base motion.
Figure E.1: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 1 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.2: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 1 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.3: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 1 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.4: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 1 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.5: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 2 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.6: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 2 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.7: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 2 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.8: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 2 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.9: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 3 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.10: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 3 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.11: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 3 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.12: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 3 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.13: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 3 Case D. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.14: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 3 Case D. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.15: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 4 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.16: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 4 Case A. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.17: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 4 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.18: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 4 Case C. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
Figure E.19: PGA, PGV, and PGD resulted from the horizontal accelerometers on both sides of the box culvert for all the shakings in Test 4 Case D. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac9 and Ac12 are the horizontal accelerometers above and below the culvert respectively.
Figure E.20: PGA, PGV, and PGD resulted from the vertical accelerometers on both sides of the box culvert for all the shakings in Test 4 Case D. Western Canada earthquake (a) acceleration, (b) velocity, and (c) displacement; Vancouver Cascadian earthquake (d) acceleration, (e) velocity, and (f) displacement; Kobe Earthquake (g) acceleration, (h) velocity, and (i) displacement. Ac10 and Ac11 are the vertical accelerometers on the left and right sides of the culvert respectively.
APPENDIX (F)

STATIC BENDING MOMENT FROM CENTRIFUGE RESULTS
EFFECT OF STRIP AND RECTANGULAR FOUNDATION

This appendix presents a comparison between the static bending moments obtained from the different cases of each test of the four tests. To show the effect of foundation on the bending moment of the box culvert, Case A where there was no foundation on the surface compared to Case B (50 kPa) and Case C and D (100 kPa). The only difference between Cases C and D is the type of foundation used, as Case C represent the strip foundation and Case D represent the rectangular foundation.
Figure F.1: Comparison of bending moment on the top slab and side wall for all the test cases.
This appendix presents a comparison between the static bending moments obtained from the different cases of each test of the four tests. To show the effect of soil density on the bending moment of the box culvert, each similar test cases that have different soil densities and similar culvert thicknesses are compared. To show that, the static bending moments obtained from Test 1 are compared to those from Test 2, and the static bending moments obtained from Test 3 are compared to those from Test 4.
Figure G.1: Comparison of bending moment on the top slab and side wall
Figure G.2: Comparison of bending moment on the top slab and side wall
This appendix presents a comparison between the static bending moments obtained from the different cases of each test of the four tests. To show the effect of culvert thickness on the bending moment of the box culvert, each similar test cases that have different culvert thicknesses and similar soil densities are compared. To show that, the static bending moments obtained from Test 1 are compared to those from Test 4, and the static bending moments obtained from Test 2 are compared to those from Test 3.
Figure H.1: Comparison of bending moment on the top slab and side wall
Figure H.2: Comparison of bending moment on the top slab and side wall
This appendix presents a comparison between different fitting methods for the static bending moment diagrams for the top slab of Test 1 Cases A, B and C. Each case fitted with 3rd order polynomial, 4th order polynomial, cubic spline interpolant and FLAC 2D bending moment. The results presented to show the effect of different fit methods on the shape of the soil pressure diagrams resulted a double derivative of the bending moment diagram.
Figure I.1: 3rd order polynomial fitting of moment resulted from strain data and comparison of the resulted pressure with the measured pressure on the top slab for Test 1 Cases A, B and C.
Figure I.2: $4^{th}$ order polynomial fitting of moment resulted from strain data and comparison of the resulted pressure with the measured pressure on the top slab for Test 1 Cases A, B and C.
Figure I.3: Cubic Spline fitting of moment resulted from strain data and comparison of the resulted pressure with the measured pressure on the top slab for Test 1 Cases A, B and C
Figure I.4: FLAC 2D results fitting the moment resulted from strain data and comparison of the resulted pressure with the measured pressure on the top slab for Test 1 Cases A, B and C
APPENDIX (J)

STATIC TACTILE PRESSURE FROM CENTRIFUGE RESULTS
EFFECT OF STRIP AND RECTANGULAR FOUNDATION

This appendix presents a comparison between the average static tactile soil pressures obtained from the different cases of each test of the four tests. To show the effect of foundation on the soil pressure of the box culvert, Case A where there was no foundation on the surface compared to Case B (50 kPa) and Case C and D (100 kPa). The only difference between Cases C and D is the type of foundation used, as Case C represent the strip foundation and Case D represent the rectangular foundation.
Figure J.1: Comparison of the average tactile pressure on the top slab and side wall for all the Test cases
APPENDIX (K)

STATIC TACTILE PRESSURE FROM CENTRIFUGE RESULTS
EFFECT OF SOIL DENSITY

This appendix presents a comparison between the average static tactile soil pressures obtained from the different cases of each test of the four tests. To show the effect of soil density on the soil pressure of the box culvert, each similar test cases that have different soil densities and similar culvert thicknesses are compared. To show that, the static soil pressure obtained from Test 1 are compared to those from Test 2, and the static soil pressure obtained from Test 3 are compared to those from Test 4.
Figure K.1: Comparison of the average tactile pressure on top slab and side wall
(a) Tests 1 & 2 Case C (top slab)  

(b) Tests 1 & 2 Case C (side wall)

(c) Tests 3 & 4 Case C (top slab)  

(d) Tests 3 & 4 Case C (side wall)

(e) Tests 3 & 4 Case D (top slab)  

(f) Tests 3 & 4 Case D (side wall)

Figure K.2: Comparison of the average tactile pressure on top slab and side wall
APPENDIX (L)

STATIC TACTILE PRESSURE FROM CENTRIFUGE RESULTS
EFFECT OF CULVERT THICKNESS

This appendix presents a comparison between the average static tactile soil pressures obtained from the different cases of each test of the four tests. To show the effect of culvert thickness on the soil pressures of the box culvert, each similar test cases that have different culvert thicknesses and similar soil densities are compared. To show that, the static soil pressures obtained from Test 1 are compared to those from Test 4, and the static soil pressures obtained from Test 2 are compared to those from Test 3.
Figure L.1: Comparison of the average tactile pressure on top slab and side wall
(a) Tests 2 & 3 Case B (top slab)  
(b) Tests 2 & 3 Case B (side wall)  
(c) Tests 1 & 4 Case C (top slab)  
(d) Tests 1 & 4 Case C (side wall)  
(e) Tests 2 & 3 Case C (top slab)  
(f) Tests 2 & 3 Case C (side wall)  

Figure L.2: Comparison of the average tactile pressure on top slab and side wall
APPENDIX (M)

COMPARISON OF STATIC PRESSURE FROM STRAIN GAUGE AND TACTILE PRESSURE SENSORS IN CENTRIFUGE TESTS

This appendix presents a comparison between the different methods used to measure the static soil pressure on the top slab and side wall of the box culvert from all the centrifuge tests and their cases. The methods used are the strain gauges and the tactile pressure sensors. The soil pressure obtained from the tactile pressure sensors presented in three values: maximum, minimum and average. The theoretical vertical and horizontal soil pressures were used in comparison to the measured pressures.
Figure M.1: Comparison of pressures on the top slab and side wall for Test 1
Figure M.2: Comparison of pressures on the top slab and side wall for Test 2
Figure M.3: Comparison of pressures on the top slab and side wall for Test 3
Figure M.4: Comparison of pressures on the top slab and side wall for Test 4
APPENDIX (N)

SEISMIC BENDING MOMENT FROM CENTRIFUGE RESULTS
EFFECT OF G-LEVEL

This appendix illustrates the effect of g-level (the amplitude of ground motion at the base PGA) produced from each earthquake shaking events on the seismic bending moment on the top slab and side wall of box culvert for all the centrifuge tests and their Cases A, C and D. The results represent the maximum bending moment values obtained from the strain gauges around the box culvert. To show the effect of g-level, the seismic bending moment obtained from specific earthquake with its different shaking amplitudes are compared together.
Figure N.1: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Test 1
Figure N.2: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Test 1
Figure N.3: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Test 2
Figure N.4: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Test 2
Figure N.5: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Test 3
Figure N.6: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Test 3
Figure N.7: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case D of Test 3
Figure N.8: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Test 4.
Figure N.9: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Test 4
Figure N.10: Effect of g-level: Comparison of measured seismic bending moments on the top slab and side wall for Case D of Test 4
APPENDIX (O)

SEISMIC BENDING MOMENT FROM CENTRIFUGE RESULTS
EFFECT OF CULVERT THICKNESS

This appendix illustrates the effect of culvert thickness on the seismic bending moment on the top slab and side wall of box culvert for all the centrifuge tests and their Cases A, C and D. The results represent the maximum bending moment values obtained from the strain gauges around the box culvert. To show the effect of culvert thickness on the bending moment of the box culvert, each similar test cases that have different culvert thicknesses and similar soil densities are compared. To show that, the static bending moments obtained from Test 1 are compared to those from Test 4, and the static bending moments obtained from Test 2 are compared to those from Test 3. Each comparison was for the same earthquake shaking event.
Figure O.1: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Tests 1 and 4
Figure O.2: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Tests 1 and 4
Figure O.3: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Tests 1 and 4
Figure O.4: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Tests 1 and 4
Figure O.5: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Tests 2 and 3.
Figure O.6: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case A of Tests 2 and 3
Figure O.7: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Tests 2 and 3
Figure O.8: Effect of Thickness: Comparison of measured seismic bending moments on the top slab and side wall for Case C of Tests 2 and 3
This appendix presents a comparison between the static bending moment measured from centrifuge tests and the static bending moment computed from FLAC 2D models on the top slab and side wall of the box culvert. This comparison was applied to all the tests and their cases.
Figure P.1: Comparison of measured versus computed bending moment on the top slab and side wall for Test 1
Figure P.2: Comparison of measured versus computed bending moment on the top slab and side wall for Test 2
Figure P.3: Comparison of measured versus computed bending moment on the top slab and side wall for Test 3
Figure P.4: Comparison of measured versus computed bending moment on the top slab and side wall for Test 4
This appendix presents a comparison between the static soil pressure measured by strain gauges and tactile pressure sensors (maximum, minimum, and average) from centrifuge tests and the static soil pressure computed from FLAC 2D models on the top slab and side wall of the box culvert. This comparison was applied to all the tests and their cases.
Figure Q.1: Comparison of measured versus computed pressure on the top slab and side wall for Test 1
Figure Q.2: Comparison of measured versus computed pressure on the top slab and side wall for Test 2
Figure Q.3: Comparison of measured versus computed pressure on the top slab and side wall for Test 3.
Figure Q.4: Comparison of measured versus computed pressure on the top slab and side wall for Test 4.
APPENDIX (R)

SEISMIC BENDING MOMENT
CENTRIFUGE VS FLAC 2D RESULTS

This appendix presents a comparison between the seismic bending moment measured from centrifuge tests and the seismic bending moment computed from FLAC 2D models on the top slab and side wall of the box culvert. This comparison was applied to Test 1 Case A only under the effect of seven earthquake shaking event (KEQL, KEQM, KEQH, WCL, WCM, VCL, and VCM).
Figure R.1: Comparison of measured versus computed seismic bending moment on the top slab and side wall for Case A of Test 1
Figure R.2: Comparison of measured versus computed seismic bending moment on the top slab and side wall for Case A of Test 1
This appendix presents a comparison between the static bending moments obtained from FLAC 2D models for the different cases of each test of the four tests. To show the effect of soil density on the bending moment of the box culvert, each similar test cases that have different soil densities and similar culvert thicknesses are compared. To show that, the static bending moments obtained from Test 1 are compared to those from Test 2, and the static bending moments obtained from Test 3 are compared to those from Test 4.
Figure S.1: Comparison of FLAC 2D bending moments on the top slab and side wall
Figure S.2: Comparison of FLAC 2D bending moments on the top slab and side wall
APPENDIX (T)

STATIC SOIL PRESSURE FROM FLAC 2D RESULTS
EFFECT OF SOIL DENSITY

This appendix presents a comparison between the soil pressures obtained from FLAC 2D models for the different cases of each test of the four tests. To show the effect of soil density on the soil pressure of the box culvert, each similar test cases that have different soil densities and similar culvert thicknesses are compared. To show that, the static soil pressure obtained from Test 1 are compared to those from Test 2, and the static soil pressure obtained from Test 3 are compared to those from Test 4.
Figure T.1: Comparison of the FLAC 2D soil pressures on top slab and side wall
Figure T.2: Comparison of the FLAC 2D soil pressures on top slab and side wall

(a) Tests 3 & 4 Case B (top slab)  
(b) Tests 3 & 4 Case B (side wall)

(c) Tests 1 & 2 Case C (top slab)  
(d) Tests 1 & 2 Case C (side wall)

(e) Tests 3 & 4 Case C (top slab)  
(f) Tests 3 & 4 Case C (side wall)
This appendix presents the comparison between the seismic bending moment obtained from FLAC 2D and the seismic bending moment obtained using the procedure recommended by the CHBDC which requests multiplying the vertical component of the acceleration by the static bending moment. To show this comparison two acceleration values were used in the CHBDC procedure: at the base of the model and at the culvert level. The results presented for the top slab and side wall of the box culvert represents Test 1 Case A and for each single earthquake shaking event (KEQL, KEQM, KEQH, WCL, WCM, VCL, and VCM).
Figure U.1: Comparison of computed seismic bending moment from FLAC 2D and from CHBDC on the top slab and side wall for Case A of Test 1
Figure U.2: Comparison of computed seismic bending moment from FLAC 2D and from CHBDC on the top slab and side wall for Case A of Test 1
APPENDIX (V)

COMPARISON OF STATIC, SEISMIC AND TOTAL BENDING MOMENT FROM FLAC 2D RESULTS

This appendix presents the comparison between the static, seismic and total bending moment diagrams obtained from FLAC 2D. The results presented for the top slab and side wall of the box culvert represents Test 1 Case A and for each single earthquake shaking event (KEQL, KEQM, KEQH, WCL, WCM, VCL, and VCM).
Figure V.1: Comparison of computed static, seismic and total bending moments from FLAC 2D on the top slab and side wall for Case A of Test 1
Figure V.2: Comparison of computed static, seismic and total bending moments from FLAC 2D on the top slab and side wall for Case A of Test 1
APPENDIX (W)

SEISMIC BENDING MOMENT FROM FLAC 2D VS CHBDC
RESULTS FOR DIFFERENT $H/Bc$ RATIOS

This appendix presents the comparison between the seismic bending moment obtained from FLAC 2D and the seismic bending moment obtained using the procedure recommended by the CHBDC which requests multiplying the vertical component of the acceleration by the static bending moment. To show this comparison two acceleration values were used in the CHBDC procedure: at the base of the model and at the culvert level. The results presented for the top slab and side wall of the box culvert represents Test 1 Case A. The results show the effect of different $H/Bc$ ratios under the earthquake shaking event KEQH on the seismic bending moment diagrams.
Figure W.1: Comparison of computed seismic bending moment from FLAC 2D and from CHBDC due to KEQH on the top slab and side wall.
Figure W.2: Comparison of computed seismic bending moment from FLAC 2D and from CHBDC due to KEQH on the top slab and side wall
APPENDIX (X)

COMPARISON OF STATIC, SEISMIC AND TOTAL BENDING MOMENT FROM FLAC 2D RESULTS FOR DIFFERENT $H/B_c$ RATIOS

This appendix presents the comparison between the static, seismic and total bending moment obtained from FLAC 2D. The results presented for the top slab and side wall of the box culvert represents Test 1 Case A. The results show the effect of different $H/B_c$ ratios under the earthquake shaking event KEQH on the static, seismic and total bending moment diagrams.
Figure X.1: Comparison of computed static, seismic and total bending moments from FLAC 2D due to KEQH on the top slab and side wall
Figure X.2: Comparison of computed static, seismic and total bending moments from FLAC 2D due to KEQH on the top slab and side wall.
This appendix presents the comparison between the seismic bending moment obtained from FLAC 2D and the seismic bending moment obtained using the procedure recommended by the CHBDC which requests multiplying the vertical component of the acceleration by the static bending moment. To show this comparison two acceleration values were used in the CHBDC procedure: at the base of the model and at the culvert level. The results presented for the top slab and side wall of the box culvert represents Test 1 Case A. The results show the effect of different $t/Bc$ ratios under the earthquake shaking event KEQH on the seismic bending moment diagrams.
Figure Y.1: Comparison of computed seismic bending moment from FLAC 2D and from CHBDC due to KEQH on the top slab and side wall
(a) $\frac{t}{Bc} = 0.09$ (top slab)  

(b) $\frac{t}{Bc} = 0.09$ (side wall)

(c) $\frac{t}{Bc} = 0.11$ (top slab)  

(d) $\frac{t}{Bc} = 0.11$ (side wall)

(e) $\frac{t}{Bc} = 0.13$ (top slab)  

(f) $\frac{t}{Bc} = 0.13$ (side wall)

Figure Y.2: Comparison of computed seismic bending moment from FLAC 2D and from CHBDC due to KEQH on the top slab and side wall
This appendix presents the comparison between the static, seismic and total bending moment obtained from FLAC 2D. The results presented for the top slab and side wall of the box culvert represents Test 1 Case A. The results show the effect of different $t/Bc$ ratios under the earthquake shaking event KEQH on the static, seismic and total bending moment diagrams.
Figure Z.1: Comparison of computed static, seismic and total bending moments from FLAC 2D due to KEQH on the top slab and side wall.
Figure Z.2: Comparison of computed static, seismic and total bending moments from FLAC 2D due to KEQH on the top slab and side wall.

(a) $t/Bc = 0.09$ (top slab)
(b) $t/Bc = 0.09$ (side wall)

(c) $t/Bc = 0.11$ (top slab)
(d) $t/Bc = 0.11$ (side wall)

(e) $t/Bc = 0.13$ (top slab)
(f) $t/Bc = 0.13$ (side wall)
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- University of British Columbia Entrance Scholarship, Uni. of British Columbia 2003  
- Libyan Education Program Scholarship, Ministry of High Education, Libya 2002-2008

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- Over 10 years of inter-disciplinary consulting experience in geotechnical and structural planning, design, construction and evaluation of civil engineering infrastructure projects.  
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