Escola de Camins
Escola Tècnica Superior d'Enginyeria de Camins, Canals i Ports
UPC BARCELONATECH

TESINA D'ESPECIALITAT

Títol

Soil Steel Composite Bridges.

An international survey of full scale tests and comparison with the Pettersson-Sundquist design method

Autor/a

Alberto Moreo Mir

Tutor/a

Gonzalo Ramos Schneider & Lars Pettersson

Departament

Enginyeria de la construcció

Intensificació

Structural design methods

Data

Juny 2014

Preface

First I would like to express my gratitude to my supervisors. First to Professor Lars Pettersson of Skanska Sverige AB for giving me the opportunity of doing this thesis under his supervision and the effort and time he has spent helping me with my research. And second to Gonzalo Ramos for helping with my thesis from Barcelona and for solving all my doubts during the time I was working in this project.

I would also like to thank Amer H. H. Wadi for the time, the dedication and the discussions we had about my research and also I really appreciate all the knowledge he shared with me.

I also want to say thank you to all my friends for the support and the company during this time working on this thesis. Especially to Álvaro, Elena, Jaume, Jose Javier ("Jota") and René.

Finally but not lease, I would like to thank my family for their support during all the time and for the confidence and the force they gave to me since the beginning.

Barcelona, June 2014

Alberto Moreo Mir

Resumen

Hoy en día, se están estudiando diferentes soluciones eficientes para resolver los problemas relacionados con la ingeniería. Dentro de este grupo de soluciones se encuentran los Soil Steel Composite Bridges (SSCB) que se presentan como una alternativa a los puentes tradicionales. Los SSCB se usan de cada vez más y muestran ser unas estructuras muy fiables y duraderas. Esta tesina busca conseguir principalmente dos objetivos. El primero es crear una base de datos general acerca de los SSCB incluyendo ejemplos de estas estructuras de diferentes partes del mundo y el segundo es comparar y discutir el método de Pettersson-Sundquist (SDM) para el diseño de estas estructuras.

Para crear la base de datos y las consiguientes comparaciones, se han utilizado 25 ejemplos de tests a escala real realizados sobre este tipo de estructuras.

Una vez creada la base de datos, se continua con la discusión y comparación de los diferentes tests. Estas discusiones y comparaciones están relacionadas con el módulo de resistencia del suelo usado en la construcción de estas estructuras. Se usa el SDM (Swedish design method) para calcular el valor de dichos módulos del suelo. Dentro de los valores calculados, se pueden diferenciar dos clases. Primero tendremos los valores teóricos calculados usando las fórmulas descritas en el método de diseño que dependen totalmente de las características del suelo. Y por otro lado, tendremos los valores del módulo del suelo calculados realizando un análisis inverso utilizando la información de los tests a escala real.

La tesina continua con la explicación de las diferentes conclusiones obtenidas del estudio de los resultados obtenidos. Caben destacar, dentro de estas conclusiones, la importancia que tiene incluir toda la información sobre el tipo de suelo y el tipo de estructura en los informes para posibles investigaciones en un futuro y también es destacable el efecto que crea el uso de losas de hormigón sobre la estructura ya que se consigue una mayor distribución de las cargas que actúan sobre la estructura y en consecuencia, se consigue una mayor resistencia.

Por último la tesina acaba con unas propuestas sobre varios temas relacionados con los SSCB sobre las que se podrían realizar investigaciones en un futuro.

Palabras clave: Soil steel, culvert, Pettersson-Sundquist design method, soil modulus, database, live load test and full scale tests.

Abstract

Nowadays, many different efficient solutions are being studied to solve engineering problems. Inside this group of solutions we can find the Soil Steel Composite Bridges (SSCB) as an alternative to traditional bridges. SSCB are being used more often every day and they are showing themselves as competitive structures in terms of feasibility and constructability. This project was started to achieve two different goals. The first one was to create a general database of SSCB including few selected tests all around the world and the second one was to compare and discuss full scale tests using the Pettersson-Sundquist design method.

To create the database and the following comparisons, twenty-five different full scale tests were used. From this tests all the necessary information was extracted and used to create the database.

After creating the database, the project continued with the discussion and comparison of the full scale tests. Specifically those discussions and comparisons were related to the resistance of the soil (the soil modulus) used in the construction of the SSCB. All the values of the different soil modulus of each full scale test used in the comparisons were calculated using the Swedish Design Manual (SDM). Two different types of soil modulus were calculated in this project using SDM, ones are the soil modulus back calculated using the values reported from the live load tests performed on the culverts and the others are theoretical soil modulus calculated using the detailed information of the soil.

The report continues with the explanation of the different conclusions ended up with during this project. It can be highlighted within this group of conclusions, the one related to the importance of reporting all the necessary information from the full scale tests including the soil parameters, the measures of the culvert, the cross sectional parameters and the vehicle dimensions among others. Another important conclusions are the effect of using the slabs over the top of the culvert and how it would effect to the sectional forces over the culvert and also the limitations using method B of the SDM regarding the type of soil used as backfilling

Finally, the project finishes explaining some proposals for future research about other fields of the study of SSCB.

Keywords: Soil steel, culvert, Pettersson-Sundquist design method, soil modulus, database, live load test and full scale tests.

Contents

Preface ... **i**

Resumen ... **iii**

Abstract ... **v**

Notations ... **ix**

1 Introduction ... **11**

1.1 Background ... 11

1.2 Types of culvert and nomenclature ... 12

1.3 Purpose ... 17

1.4 Limitations and challenges ... 17

1.5 Contents of the thesis ... 19

2 Explanation of the SDM (Swedish design method) ... **20**

2.1 General ... 20

2.2 Principles for analysis ... 20

2.2.1 General calculation principles ... 20

2.2.2 Loads and the characteristics of the structural backfill ... 21

2.3 Influence of loads ... 21

2.3.1 Reduction of the effective height of cover ... 21

2.3.2 Determination of normal forces ... 22

2.3.3 Determination of design bending moments ... 24

3 Methodology ... **28**

3.1 Literature study ... 28

3.2 Data extraction from full scale papers ... 29

3.3 Database ... 31

3.4 Calculation routines ... 32

3.4.1 Live load ... 32

3.4.2 Back calculations of soil modulus ... 33

3.4.3 Theoretical soil modulus ... 34

4 Full scale tests ... **37**

4.1 General 37
4.2 Abstracts 37

5 Calculations 57

5.1 Soil modulus calculated 57
5.1.1 Back calculated soil modulus 57
5.1.2 Theoretical soil modulus 63

6 Discussions 66

6.1 Comparison between back calculated soil modulus and soil modulus using the soil suggested in the design manual 66
6.1.1 Soil modulus back calculated compared to soil modulus calculated using crushed rock 68
6.1.2 Soil modulus back calculated compared to soil modulus calculated using base course material 68
6.2 Comparison between back calculated soil modulus and calculated soil modulus using method B 69
6.3 Variation of soil modulus with respect to the parameter h_c/D 71
6.4 Effect of slabs or pavement over the top of the culvert 72
6.5 Effect of the parameter Cu in respect to the calculations using method B 73

7 Conclusions 76

7.1 General 76
7.2 Database 76
7.3 Soil modulus 77
7.4 Future research 78

Bibliography 79

A. Appendix. Data base 82

B. Appendix. $P_{traffic}$ loading curves 108

C. Appendix. Sample of back calculated soil modulus using SDM 113

Notations

A_s	Cross section area of steel profile	[mm^2/mm]
c_u	Soil uniformity coefficient	[-]
D	Diameter or span of the culvert	[m]
d_{10}	Aggregate size at 10% passing	[mm]
d_{50}	Aggregate size at 50% passing	[mm]
d_{60}	Aggregate size at 60% passing	[mm]
e_1	Soil void ratio	[-]
E_{sd}	Design soil tangent modulus	[MPa]
E_{sk}	Characteristic soil tangent modulus	[MPa]
f_1, f_2, f_3, f_4	Functions used as a mean of simplification	[-]
H	Vertical distance between crown and spring opening	[m]
$h_{.c.red}$	Reduced height of soil cover	[m]
h_c	Soil cover	[m]
I_s	Moment of inertia for steel profile	[mm^4/mm]
M	Modulus number	[-]
M_t	Moment due to live load	[kNm/m]
P_a	Reference pressure	[kPa]
$P_{traffic}$	Equivalent line surface load	[kN/m]
Q	Distributed pressure from traffic	[kN/m]
RP	Degree of compaction according to standard Proctor	%
R_c	Corner radius	[m]
R_s	Side radius	[m]
R_t	Top radius	[m]

S_{ar}	Reduction factor for load from the soil overburden used in conjunction with arching calculations	[-]
SDM	Swedish design method developed by Pettersson and Sundquist	[-]
SDM manual	Swedish design method manual developed by Pettersson and Sunquist, 4th edition, 2010	[-]
SSCB	Soil steel composite bridge	[-]
t_s	Steel profile thickness	[mm]
W_s	Elastic section modulus of steel profile	[mm^3/mm]
Z_s	Plastic section modulus of steel profile	[mm^3/mm]
β	Stress exponent used in conjunction with soil stiffness	[-]
φ	Angle of internal friction	[°]
γ_m	Soil material safety factor	[-]
γ_n	Safety class factor according to Eurocode	[-]
λ_f	Stiffness number	[-]
ρ	Weight density of the soil above the crown	[kN/m^3]
ρ_{opt}	Optimum density determined according to the standard or modified Proctor method	[kN/m^3]
ρ_s	Solid weight density of the soil material in the back fill	[kN/m^3]
σ_1	Major principal soil stress	[kPa]
σ_3	Minor principal soil stress	[kPa]

1 Introduction

1.1 Background

The Soil Steel Composite Bridges (SSCB) are structures that are being used more often nowadays all around the world because those structures can be erected in an easier and faster way than for example a concrete bridge and are much cheaper than other structures that could carry the same functions.

Soil steel bridge is a structure based on corrugated steel plates bolted together creating a pipe embedded in well compacted engineering soil surrounding the structure.

Those flexible steel shell structures began to be used in bridge design at the end of the nineteenth century. Initially, such bridge structures were constructed as riveted pipes with circular cross-section and a relatively short span.

Figure 1. 1: Example of circular pipe section.

The second generation of that type of structures had a span in the range of 8 to 16 m. Their development was ensued in Canada around 1960. It was encouraged by a better understanding of principles of interaction of soil and steel shell. In that period, an intensive research in these structures resulted in the development of a number of analytical methods for determining the distribution of internal forces and stresses in structure. The progress in computational methods based on FEM contributed to development of the third generation of soil–steel structures which were characterized by: a) span being longer than 16 m, b) a further introduction of the box section profiles of span up to 8 m and c) utilization of techniques combining different materials, i.e. longitudinal and transverse stiffening ribs (corrugated steel plate with reinforced concrete) and reinforced soil (soil and geotextiles) and cement or lime stabilized soil. In the 90's of the twentieth century, the corrugated plates with large corrugation (380x140 mm and 400x150 mm) were introduced to bridge engineering which enabled us to construct the box section structures significantly exceeding the span length of 12 m at the overburden depth of 0,45–1,5 m. Nowadays, the range of application of flexible soil–steel structures covers not only the building of new structures but also the strengthening of existing ones (C. Machelski and G. Antoniszyn, 2004)

Figure 1. 2: Example of a large span box culvert

Firsts soil steel composite bridges appeared in Sweden around mid 1950's with moderate spans and different height of cover, varying between high or low depending on the structure. Those culverts were designed using simple diagrams and standard drawings valid for culverts with spans up to 5 m (Pettersson, 2007).

During the following years the interest on large span steel culverts with low height of cover increases a lot because designers realized that it could be a more efficient alternative against other types of bridges. For this reason it was necessary to create a design method capable of predicting the structural behaviour of the culverts. This design method, often referred as Swedish design method SDM, was developed by Professor Lars Pettersson.

1.2 Types of culvert and nomenclature

One of the advantages of the SSCB is the variety of solutions regarding to the shape of the culvert. Designers can choose between close and open profiles and then can decide which type of culvert exactly they need within each group. Even the huge variety of profiles within the SSCB we can organize mostly of them in 6 main groups. Following images are representative schematic images of the different groups:

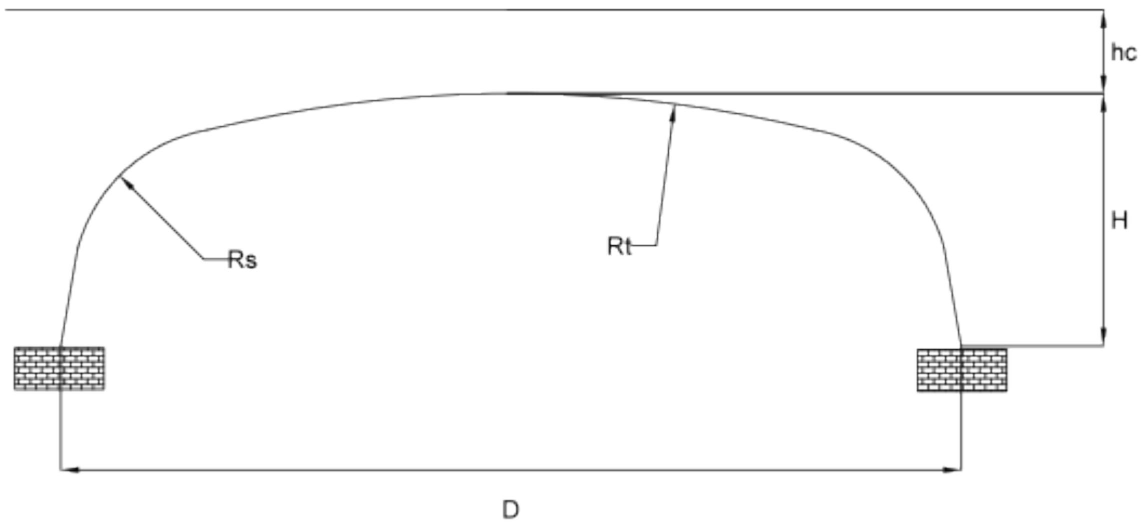

Figure 1.3: Box culvert profile

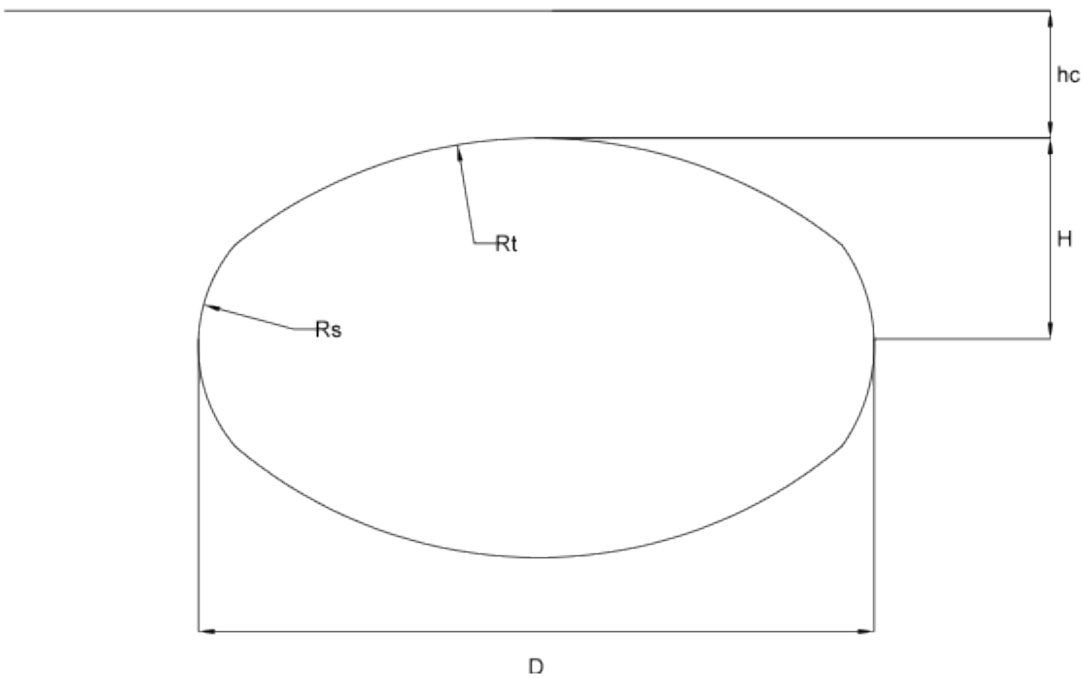

Figure 1.4: Horizontal ellipse prifile

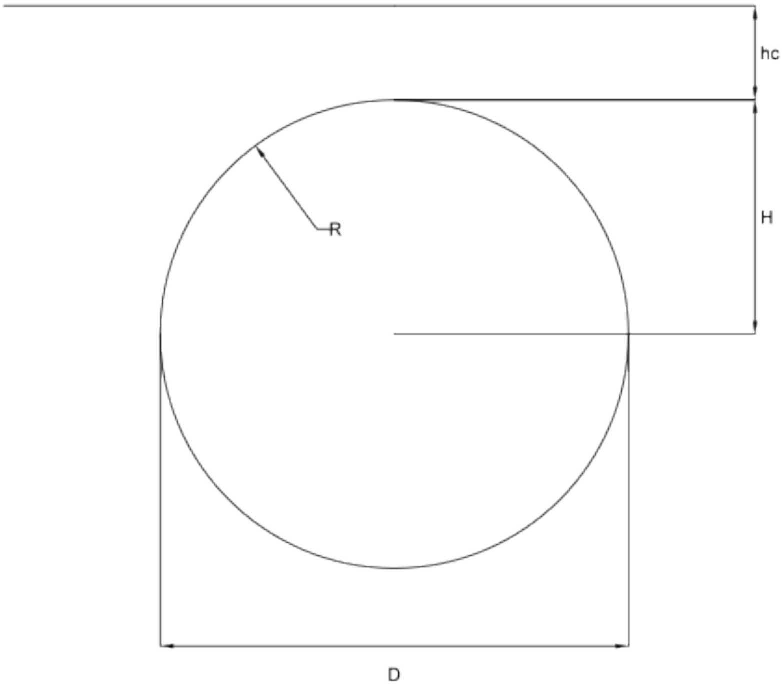

Figure 1.5: Circular profile

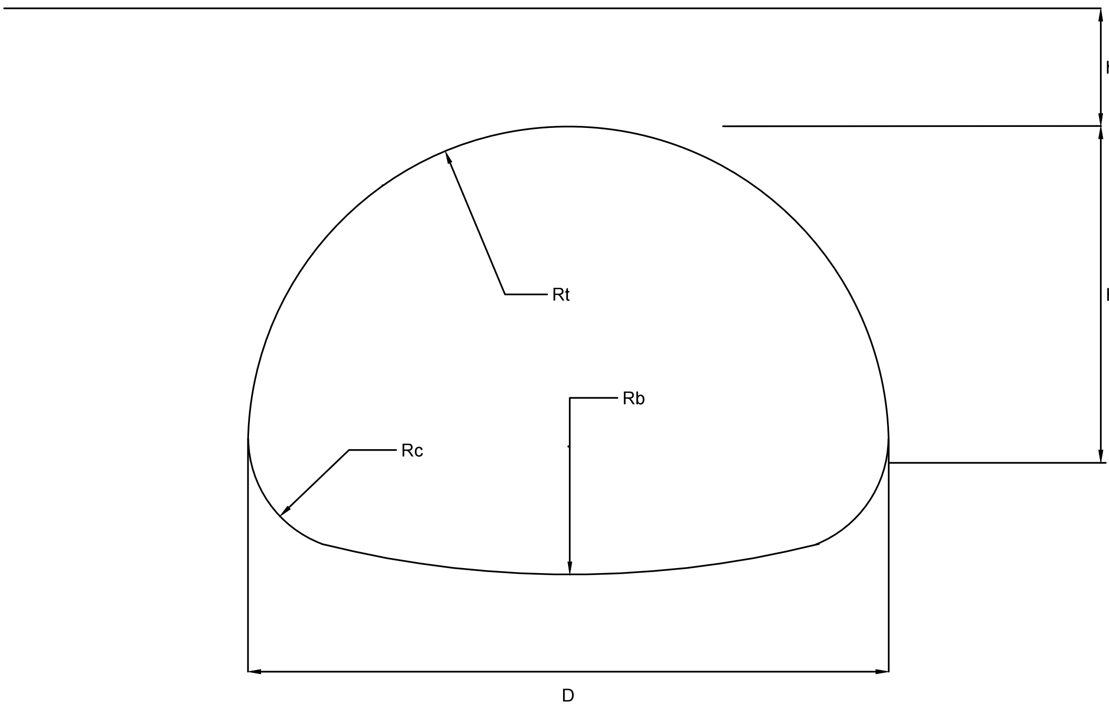

Figure 1.6: Pipe arch

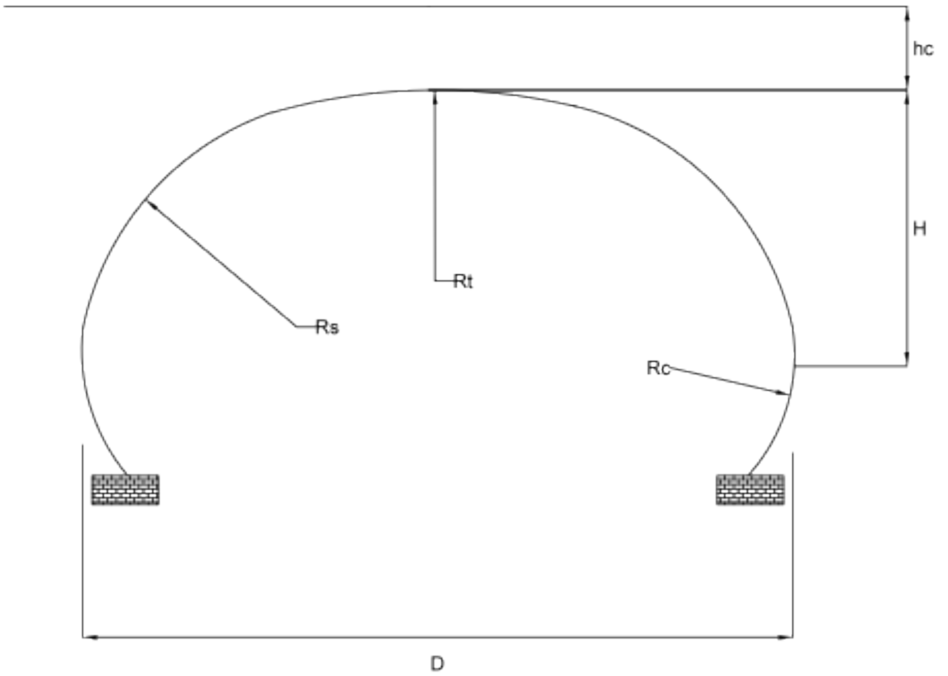

Figure 1.7: Three radius arch profile

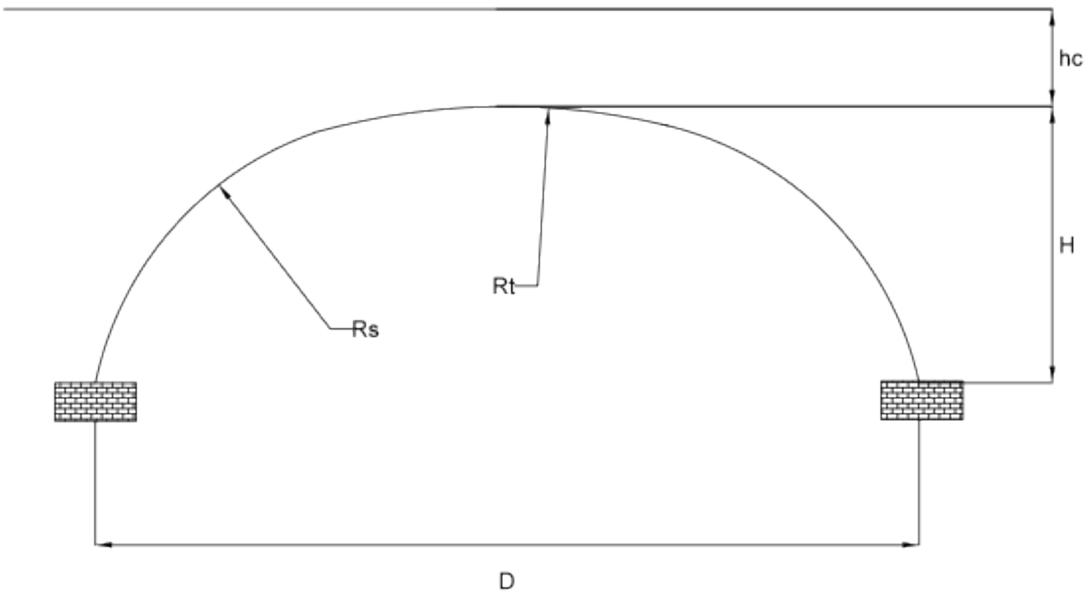

Figure 1.8: Two radius arch profile

The different profile cross sections are defined by some parameters, some of the most important are:

- *Crown*: is the highest point of the cross section of the conduit.

- *Rise*: is the vertical distance between the crown point and the lowest point of the profile.

- *Span*: is the maximum horizontal distance inside the conduit.

- *Height of cover*: is the vertical distance between the crown point and the top of the corrugation and the road surface.

Another important detail for the culvert are the corrugations. The stiffness of the culvert depends mostly on the corrugation profile. The first attempt at making a corrugated metal pipe was developed by James H. Watson (sheet metal worker in Crawfordsville, Indiana) and E. Stanley Simpson (city engineer of Crawfordsville, Indiana) but it didn't work well because the corrugations were running along the longitudinal axis instead of running along the circumference and it caused that the pipe couldn't resist radial pressures of the soil. When they found out the problem they changed it and patented the corrugated metal pipe under U.S. Patent No. 559.642 (Abdel-Sayed, Bakht and Jaeger, 1994).

Nowadays exists different types of corrugation, the most common are showed in figure 1.3. Companies have also started using profiles with higher corrugations like deep corrugated Ultra-Cor structural plate with a corrugation pitch and depth of 500x237 mm and thickness ranged from 7 to 12 mm (Williams, Mackinnon and Newhook, 2012).

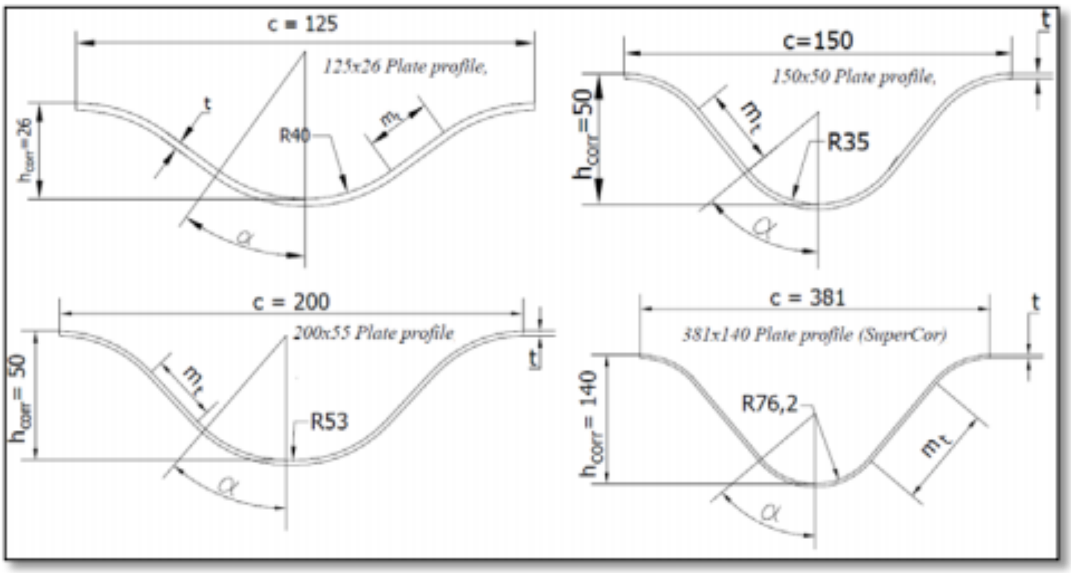

Figure 1. 9: Common types of corrugation. Source: (Pettersson, 2007)

Once defined the profile of the culvert designers need to choose the soil they will use around the structure, called structural backfill, but this is not the only type of soil used during the construction. Those are the different soil used in the construction of a culvert:

- *Bedding*: is the engineered soil used just under where the culvert is placed and it must have the shape as the invert of the closed profiles.

- *Backfill*: is the selected engineered soil placed around the conduit and compacted until a high degree of compaction using different compaction methods.

- *Fill*: is the rest of soil used to finish the structure. It doesn't need to be compacted as much as the backfill and it can be less quality soil. Normally the natural soil of the place where the structure is erected is used for this purpose.

- *Foundation*: is the soil or rock material underlying the engineered soil (bedding, backfill and fill).

1.3 Purpose

The aim of this thesis is first to achieve a general database of full-scale soil steel composite bridge tests where researchers could obtain a first impression and some specific data without the necessity of reading all the paper.

The second objective of the thesis is to compare and discuss full scale test with the Pettersson-Sundquist design method, which is often referred to as the Swedish design method (SDM), to back-calculate soil modulus for the design of SSCB. Further details are explained in chapter 4.

1.4 Limitations and challenges

The main limitations came from the fact that number of tests studied was limited to 25 while there are more constructions of this type which should also need to be studied. But for this thesis it was decided to pick only 25 as a representative number of the international SSCB constructed.

The main challenge of this project was the lack of information in the different papers. Mostly of the information missing is related to the back-calculations of soil modulus using the SDM. The main information that is not reported is:

- *Live load test performed:* in some papers live load test is not performed on the culvert. For those cases was not possible to back-calculate any soil modulus of the soil. To write the thesis, 25 different cases were studied and in 19 of this 25 cases live load test was performed.

- *Moment at the crown (M_t) reported:* to be able to back-calculate the soil modulus using the SDM the moment at the crown measured during the live load test is needed. Some papers only reported the normal forces and no bending moments and then it was not possible to back calculate the soil modulus.

- *Load configuration:* the information about the vehicle like tyre pressure, vehicle description, weight, etc… is necessary to calculate the $p_{traffic}$ equivalent load used in the equations of the SDM. $P_{traffic}$ is an equivalent linear distributed load and is explained with further details in section 2.4.1.

- *Information about the soil used as backfilling:* as it was explained in section 1.1 the structure is based on the interaction the soil-culvert. The characteristics of the soil are important to calculate the quality of it and the load that can be carried by the soil. Normally the gradation of the soil was not reported, instead of it, the type of soil was reported but it was not enough information for the calculations in the SDM because the numerical values d_{60}, d_{50} and d_{10} are needed for the equations included in the design method.

A part from the limitations regarding the information reported in the papers there are also some limitations due to the Swedish design method. Those limitations are:

- *Culvert profiles:* there are different cross sections profiles regarding the SSCB but some profiles limited by relationship between the different radii. Those relationships are:

- for horizontal ellipses: $R_t/R_s \leq 4$ and $R_b/R_S \leq 4$ (normally the top and the bottom radius are the same).
- for pipe-arch profiles: $R_t/R_c \leq 5{,}5$ and $R_b/R_c \leq 10$.
- for multi-radii arches: $R_t/R_s \leq 4$ and $1 \leq R_c/R_s \leq 4$.
- for box culverts : $R_t/R_s \leq 12$.

- *Structural soil:* the soil used as backfilling around the culvert must provide some friction between the culvert and structural soil. Normally the friction angle is the range of 35-45º and is calculated from the different soil parameters.

- *Stiffness number:* the stiffness number relates the stiffness of the soil and the conduit. Its a very important factor in the design of the culverts.

$$10^2 < \lambda_f < 10^5$$

- *Minimum height of cover:* the required minimum height of cover depends on the load that the structure has to carry. Even though, in some cases, due to an adequate compaction of the soil this minimum height of cover may vary. Therefore in the Swedish design method a minimum height of cover of 0,5 m is selected, that number is based on several full scale tests having soil of cover of 0,5 m or less.

- *Longitudinal slope of roadway above the culvert:* for the cases where the longitudinal slope is higher than 10% the cross sectional forces of the culvert should be recalculated taking into account the slope of the road or railway.

- *Special measures during backfilling:* The SDM assumes that no special measures are adopted during backfilling for the calculation of cross sectional forces. There is no influence in the equation for normal forces and bending moments. In case that such special measures would be adopted the analysis of the cross sectional forces of the culvert must take into consideration the new conditions.

1.5 Contents of the thesis

This thesis includes six chapters plus two different appendixes. The first chapter includes a brief introduction to the soil steel composite bridges (SSCB) where the characteristics and the different types of culverts are explained. This chapter includes also an explanation of the main goals of this project.

The second chapter explains all the methodology I have followed to be able to create the database and to calculate the different values of the soil modulus from the different full scale tests studied.

In the third chapter abstracts are presented to describe each one of the twenty-five full scale tests. The abstracts include all the relevant information with regard to every test plus a representative image to help future researchers to get a general idea of the culvert.

The fourth chapter contains the results regarding the soil modulus calculated using the Swedish design method (SDM) for the tests where the necessary information for the calculations was available.

In the fifth chapter all the results presented in chapter four are studied and discussed.

Finally the sixth chapter, the main conclusions derived from this study are summarized.

2 Explanation of the SDM (Swedish design method)

2.1 General

The need for a Swedish design method for soil steel composite bridges has emerged to cover the on-going demand of market investment in building these structures. The Swedish road administration - Vägverket (SRA) saw a need for more accurate design method, although international design methods were available, but it was unclear how they should be applied to the Swedish conditions.

The challenges to build structures with larger spans and small cover heights were encountered by the limitations and restrictions existing in the international methods. Moreover, these design methods had some drawbacks like the effect of higher degree of compaction or even effect of soil grading, and in order to have better understanding of the behaviour of these structure, an investigation has been made (beginning of 1980s) for developing a new design method composed together with the international experience and suitable for Swedish conditions.

2.2 Principles for analysis

2.2.1 General calculation principles

When designing a culvert, it is assumed that the culvert has a uniform section over a long distance in the pipe's longitudinal direction. Further, the calculation model assumes that it is possible to consider a strip with a length of one meter subjected to loading forces acting perpendicularly to the axis of the pipe. If the culvert changes section in any part, then each section must be checked. This also applies if the depth of cover or backfilling materials varies along the culvert.

When subjected to a traffic load, the upper part of the culvert profile can be considered separately. This "top arch" has elastic supports whose characteristics are defined by the amount of lateral support that can be provided by the soil surrounding the pipe. The arch is also continuously elastically supported with the aid of the mass of soil lying above the arch. The most important calculation is directed to the treatment of this top-arch, as it is mainly this area that is affected by the traffic load.

In the upper part of the profile (the top arch), the normal force can from a calculation point of view, be considered constant, while the bending moment due to the traffic load is such that the negative value is roughly half the positive value.

Culverts where surrounding backfill consists entirely, or in part, of light-weight filling material are not directly covered by this manual.

2.2.2 Loads and the characteristics of the structural backfill

The loads are assumed to arise from different types of moving loads or traffic loads. The surrounding soil material is assumed to be compacted friction material.

It is necessary to characterise the properties of the soil at the sides and above the culvert. Information relating the soil material in the different backfilling regions is necessary in order to calculate the capacity of the pipe walls.

With this information of the soil it is possible to calculate the theoretical soil modulus used in the design method. There are two different methods, Method A and Method B. Method A gives more conservative values and it needs less information about the soil. On the other hand, Method B gives more realistic values of the soil modulus but it need mucho more soil parameters then Method A.

The equations used in both methods are given in section 3.4.3.

2.3 Influence of loads

The forces and moments that arise as a result of loads on the pipe are calculated using the equation below. The SDM, which is based on extensive FEM calculations, is used as the starting point for these calculations. The method makes it possible to estimate the section forces arising both from soil loads and from live traffic loads.

2.3.1 Reduction of the effective height of cover

During the installation of a culvert the crown rises during the back-filling process because of the pressure of the soil acting against the sides of the pipe. This results in a reduced depth of cover for any given positive height between the bottom of the pipe and the road surface.

The reduced cover depth to be used in the calculations is therefore

$$h_{\mathrm{c,red}} = h_c - \delta_{crown} \qquad (2.1)$$

The increase in height of the pipe during the back-filling operation may be assumed to be approximately

$$\delta_{crown} = 0{,}015 \bullet D \tag{2.2}$$

2.3.2 Determination of normal forces

The calculation model assumes that the maximum normal forces and the moments are calculated. The load-bearing capacity is checked with respect to both the normal force and to the combination of normal force and moment using interaction formula.

Loads due to the surrounding soil

The normal force caused by the load due to the soil in its permanent position is determined as

$$N_s = 0{,}2\frac{H}{D}\rho_1 D^2 + S_{ar}\left(0{,}9\frac{h_{\mathrm{c,red}}}{D} - 0{,}5\frac{h_{\mathrm{c,red}}}{D}\frac{H}{D}\right)\rho_{cv}D^2 \tag{2.3}$$

The coefficient S_{ar} takes into account the effect of arching of the soil above the culvert which occurs with large cover depths. If the culvert is placed in an excavation in natural soil or rock, this effect may be calculated as follows

$\varphi_{cv,d}$ design angle of internal friction for the cover material, $\tan\varphi_{cv,d} = \dfrac{\tan\varphi_{cv,k}}{\gamma_n\gamma_m}$ (2.4)

In Eq. (2.4), the value of γ_m is normally 1,3. The angle of internal friction to be used refers to the soil above the culvert.

$$S_v = \frac{0{,}8\tan\varphi_{cv,d}}{\left(\sqrt{1+\tan^2\varphi_{cv,d}} + 0{,}45\tan\varphi_{cv,d}\right)^2} \tag{2.5}$$

$$k = 2S_v\frac{h_{c,red}}{D} \tag{2.6}$$

$$S_{ar} = \frac{1-e^{-k}}{k} \tag{2.7}$$

Distributed loads

Distributed loads give rise to both normal forces and bending moments. The cross-sectional combinations with and without a distributed load shall be calculated in accordance with the instructions below and the load-bearing capacity shall be checked for these combinations.

Concentrated loads

The load distribution from point loads arising from e.g. wheels of vehicles and axle loads should be calculated according to Boussinesq's method. The reason why this method is used is that, for example, the 2:1 method is considered to be too conservative while distribution 1:1 yields non-conservative values. In addition, the simplified load distribution method yields discontinuities in the vertical pressure from the traffic loads when it is expressed as a function of depth of cover. Axle and bogie loads are converted to equivalent line loads at the roadway level, p_{traffic}.

Calculation of the equivalent line load, p_{traffic} and the normal force due to traffic

The SDM is based on the concept that the actual traffic load is converted, with the aid of the stress distribution in a semi-infinite body according to Bousinesq, to the equivalent line load which yields the same vertical stress at the level of the crown of the pipe as the traffic load itself.

Bousinesq gives the following relationship fot the vertical stress at a depth z (vertically beneath the load) caused by a line load p applied to a semi-infinite elastic body:

$$\sigma_v = \frac{2 \cdot p}{\pi \cdot z} \tag{2.8}$$

In the same manner for a point load, the expression:

$$\sigma_v = \frac{3 \cdot P \cdot h_{c,red}^3}{2\pi \cdot s^5} \tag{2.9}$$

is obtained where s is the sloping distance between the point load and the calculation point at depth $h_{c,\text{red}}$. Point and line loads are converted to an equivalent line load p_{traffic}. In the absence of more precise method, equations (2.8) and (2.9) are used so that the vertical stress at the point concerned is calculated according to Bousinesq, and the equivalent line load at the point which is subjected to the greatest vertical stress is thereafter calculated using the equation:

$$p_{traffic} = \frac{\pi \cdot h_{c,red}}{2} \sigma_v \tag{2.10}$$

On the basis of this information, the forces in the walls of the pipe are calculated as follows.

$$\text{if } h_{c,\text{red}} / \text{D} \le 0{,}25\ ; \quad N_t = p_{\text{traffic}} + (\text{D}/2)q \tag{2.11}$$

$$\text{if } 0{,}25 < h_{c,\text{red}} / \text{D} \le 0{,}75\ ; \quad N_t = (1{,}25 - h_{c,}\text{red}/\text{D})\, p_{\text{traffic}} + (\text{D}/2)q \tag{2.12}$$

if $0{,}75 < hc{,}red / D$; $Nt = 0{,}5\ p_{traffic} + (D / 2)q$ (2.13)

Design normal forces

The design normal force in the serviceability limit state is given by

$$N_{d,s} = (\psi\gamma)_{s,s} \cdot N_s + (\psi\gamma)_{t,s} \cdot N_t \left(\frac{R_t}{R_s}\right)^{0,25} \tag{2.14}$$

And in the ultimate limit state by

$$N_{d,u} = (\psi\gamma)_{s,u} \cdot N_s + (\psi\gamma)_{t,u} \cdot N_t \tag{2.15}$$

And in fatigue limit state by

$$\Delta N_{d,f} = (\psi\gamma)_{s,f} \cdot Nt \tag{2.16}$$

Where the partial safety factors are chosen according to the appropriate standard for the respective limit states.

2.3.3 Determination of design bending moments

General

The bending moment in the wall of the pipe depends on the relationship between the stiffness of the soil and the stiffness of the pipe. This relationship is denoted by λ_f, and is given by:

$$\lambda_f = E_{sd} \cdot D^3 / (EI)_s \tag{2.17}$$

where E_{sd} is the design tangent modulus of the soil calculated using Methods A and B and $(EI)_s$ is the bending stiffness of the pipe. The tangent modulus of the soil material is dependent on the prevailing stress distribution within the soil. In the design of culverts with different soil materials a weighted average value of E_s shall be used in the calculation of λ_f. In the simplified method, only the knowledge of the degree of compaction is required. This can be determined either according to the Standard Proctor, RP^{std} method or the Modified Proctor, RP^{mod} method. The relationship between the Standard Proctor and the Modified Proctor for friction material is assumed to be $RP^{std} = RP^{mod} + 5\%$.

The moments which arise in a culvert are built up in three different stages:

1. Firstly, a moment distribution is created by the lateral pressure which arises before any filling material is placed in top of the culvert.

2. When the culvert is later covered, i.e. material is packed above the level of the crown, the moment will change.

3. Traffic and other loads will subsequently give rise to other moment distributions.

This process and the equations describing it are given in the following sections.

Loads due to the surrounding soil

The bending moment caused by the soil load can, for both the serviceability limit state and the ultimate limit state, be expressed by Eq. (2.18).

The equation is based on the observation that when the backfill is compacted around the flexible structure, the structure is pressed inwards at the sides and a negative moment is created at the crown. This reaches a maximum when the level of filling reaches the level of the crown, and this is represented by the first term on the right-hand side of Eq. (2.18). When the work of filling is continued above the level of the crown, the structure is pressed down and the negative moment is reduced. If the height of cover is large the moment in the crown can change and become a positive moment.

$$M_s / D^3 = M_{s,surr} / D^3 + M_{s,\mathrm{cover}} / D^3 = -\rho_1 f_1 f_3 f_{2,surr} + S_{ar} \rho_{cv} \frac{h_{c,red}}{D} \left(\frac{R_t}{R_s} \right)^{0,75} f_1 f_{2,\mathrm{cover}} \qquad (2.18)$$

The function f_1 is calculated as:

$$\text{if } 0{,}2 < H / \mathrm{D} \le 0{,}35 \text{ ; } f_1 = \left[0{,}67 + 0{,}87 (H/D - 0{,}2) \right] \qquad (2.19)$$

$$\text{if } 0{,}35 < H / \mathrm{D} \le 0{,}5 \text{ ; } f_1 = \left[0{,}8 + 01{,}33 (H/D - 0{,}35) \right] \qquad (2.20)$$

$$\text{if } 0{,}5 < H / \mathrm{D} \le 0{,}6 \text{ ; } f_1 = 2 \cdot (H/D) \qquad (2.21)$$

f_2 is calculated as:

$$\text{if } \lambda_f \le 5000 : \ f_{2,surr} = 0{,}0046 - 0{,}0010^{10} \log\left(\lambda_f \right) \qquad (2.22)$$

if $\lambda_f > 5000$: $f_{2,surr} = 0{,}0009$ (2.23)

and f_3 is calculated as

$$f_3 = 6{,}67\frac{H}{D} - 1{,}33 \qquad (2.24)$$

For the backfilling above the crown of the culvert, the following applies

if $\lambda_f \leq 5000$: $f_{2,\text{cover}} = 0{,}018 - 0{,}004^{10}\log\left(\lambda_f\right)$ (2.25)

if $\lambda_f > 5000$: $f_{2,\text{cover}} = 0{,}0032$ (2.23)

Load due to traffic

The moment arising from the traffic load due to the equivalent line load p_{traffic} and en evenly distributed traffic load q is given by the equations:

$$M_t = f_4^{'} \cdot f_4^{''} \cdot f_4^{'''} \cdot f_4^{IV} D \cdot p_{traffic} + S_{ar}\left(\frac{R_t}{R_s}\right)^{0,75} f_1 \cdot f_{2,\text{cov}\,er} \cdot q \cdot D^2 \qquad (2.24)$$

$$f_4^{'} = 0{,}65 \cdot \left(1 - 0{,}2 \cdot {}^{10}\log\left(\lambda_f\right)\right) \qquad (2.25)$$

$$f_4^{''} = 0{,}120 \cdot \left(1 - 0{,}15 \cdot {}^{10}\log(\lambda_f)\right) \qquad (2.26)$$

$$f_4^{'''} = 4 \cdot \left(0{,}01^{(h_c/D)} + 0{,}1\right) \qquad (2.27)$$

$$f_4^{IV} = \left(\frac{R_t}{R_s}\right)^{0,25} \qquad (2.28)$$

In addition, the condition $f_4^{'} \cdot f_4^{'''} < 1{,}0$ shall always apply.

In the case of profiles where $R_t / R_S \geq 1$ the moment on the side plates is calculated as 1/3 of the moment calculated according to Eq. (2.24).

Design bending moments

The design bending moments due to the soil and traffic loads have different directions at different points and checks must therefore be performed according to the following formula.

The design moment in the serviceability limit state is determined according to:

$$M_{d,s} = \left(\psi\gamma_{s,surr,s}\right)M_{s,surr,s} + \left(\psi\gamma_{s,\mathrm{cover},s}\right)M_{s,\mathrm{cover},s} + \left(\psi\gamma_{t,s}\right)M_{t,s} \tag{2.29}$$

(Order of index: soil, surround/cover, serviceability). The design moment for a traffic load in the serviceability limit state is determined according to:

$$M_{t,s}^{\max} = \left(\psi\gamma\right)_{t,s} \cdot M_t$$
$$M_{t,s}^{\min} = \left(\psi\gamma\right)_{t,s} \cdot (M_t/2) \tag{2.30}$$

In the ultimate limit state, the design moment is calculated according to

$$M_{d,u} = \left(\psi\gamma\right)_{s,surr,u} M_{s,surr} + \left(\psi\gamma\right)_{s,\mathrm{cover},u} M_{s,\mathrm{cover}} + \left(\psi\gamma\right)_{t,u} M_t \tag{2.31}$$

3 Methodology

3.1 Literature study

To write my thesis I have used some papers in which there are reports about construction of twenty-five SSCB and some test performed on them. Normally there is one culvert reported per paper but there are some in which two, three or even four culverts are described. Those are the papers used to extract the data I will present in chapter 3:

Number of culvert	Name of the paper	Culvert profile	Span (m)
1	*Field verification of ring compression conduit design*	Pipe arch	6,3
2	*Instrumentation of a corrugated steel-soil arch overpass at Leigh creek, South Australia*	Two radius arch	12
3	*Measurement of soil arching above a large diameter flexible culvert*	Three radius arch	15,5
4	*Measurements and analyses of compaction effects on a long-span culvert*	Two radius arch	12
5	*Measured performance of Newport creek culvert*	Three radius arch	7,9
6	*Loading tests on an ARMCO pipe arch culvert*	Pipe arch	4
7	*Long-span reinforced steel box culverts*	Box culvert	12
8	*The effect of pavement layers on the behaviour of corrugated steel culverts*	Pipe arch	3,7
9	*The effect of pavement layers on the behaviour of corrugated steel culverts*	Pipe arch	6
10	*Loading tests on two long span corrugated steel culverts*	Horizontal ellipse	9,8

Number of culvert	Name of the paper	Culvert profile	Span (m)
11	*Loading tests on two long span corrugated steel culverts*	Three radius arch	7,5
12	*Live-load response of a soil-steel structure with a relieving slab*	Horizontal ellipse	8,8
13	*Performance of a large corrugated steel culvert*	Horizontal ellipse	8,2
14	*Deflection of flexible culverts due to backfill compaction*	Horizontal ellipse	7,7
15	*Performance and analysis of a long-span culvert*	Two radius arch	11,5
16	*Behaviour of aluminium structural plate culvert*	Pipe arch	8,7
17	*In-situ load testing of corrugated steel pipe-arch culverts*	Pipe arch	3,8
18	*In-situ load testing of corrugated steel pipe-arch culverts*	Pipe arch	3,4
19	*In-situ load testing of corrugated steel pipe-arch culverts*	Pipe arch	4,3
20	*In-situ load testing of corrugated steel pipe-arch culverts*	Pipe arch	3,7
21	*Full-scale field tests on flexible pipes under live load application*	Circular	0,93
22	*Measured field performance and computer analysis of a large-diameter multiplate steel pipe culver installed in Ohio*	Circular	6,4
23	*Research advancing the design of large span deep corrugated metal culverts*	Box culvert	10
24	*Static load tests of a corrugated steel plate arch with relieving slab*	Two radius arch	10
25	*Field tests of a large-span metal culvert*	Two radius arch	9,5

Table 3.1: Papers titles and corresponding authors

3.2 Data extraction from full scale papers

To create the database was, it necessary to extract all the information needed from all the different tests. The extraction of the data started by reading the different papers but there was a problem because obviously not all the papers reported the same information, maybe because the information was irrelevant for the author or simply because the data was not able to be reported because the specific tests were not performed.

Is important to remark that not all the papers were written at the same time. Within the 25 tests the oldest ones were performed around 1970's and the new ones were performed around 2000's. It is obvious that the equipment to perform the tests and to measure the different data are not the same and furthermore this equipment has been improved during the following years.

Within the 25 tests, there are some data reported in all or almost all the different papers. Normally the measures of the section of the culvert are reported in all the papers. At least the span, the rise, the height of cover, the corrugation profile and the material used to build the culvert are reported in all of them. On the other hand there are some section measures like inside radius (top, bottom, side and corner) that are reported only in some papers. It is important to say that the Viacon catalogues (Multiplate and SuperCor) are used to take some values regarding the section profile of the culverts for the cases in which the data was not reported. Viacon is an specialized company in construction of SSCB.

Another cross-section parameters like the area A (mm^2/mm), the moment of inertia I (mm^4/mm), the section modulus W (mm^3/mm) and the plastic section modulus Z (mm^3/mm) are also reported in most of the papers. For the papers in which those values are not reported I used the most exact values from the design tables (Pettersson and Sunquist, 2007) depending on the type of corrugation and the thickness of the plate.

t	***A***	***I***	***W***	***Z***	***Z/W***
mm	mm^2/mm	mm^4/mm	mm^3/mm	mm^3/mm	-
2,00	2,36	898	31,5	41,4	1,31
3,00	3,54	1353	46,7	62,3	1,34
4,00	4,73	1811	61,4	83,3	1,36
5,00	5,92	2273	75,8	104,5	1,38
6,00	7,10	2739	89,8	125,8	1,40
7,00	8,29	3208	103,5	147,3	1,42

Table 3.2: Example of cross-section parameters for a 200×55mm corrugation profile. Source: (Pettersson, 2007)

Another important group of data are the soil parameters like the type of soil used as backfilling, the degree of compaction, the sieve analysis of the soil or the soil density in the field. Those parameters are very important for the back calculations explained in chapter 4 but not all the papers report this specific information, for example only 6 of 25 tests include sieve analysis of the soil they use as backfilling. For the tests that don't include this information or only some parameters some assumptions are made on similar examples found in which the same type of soil is used to obtain the parameters left.

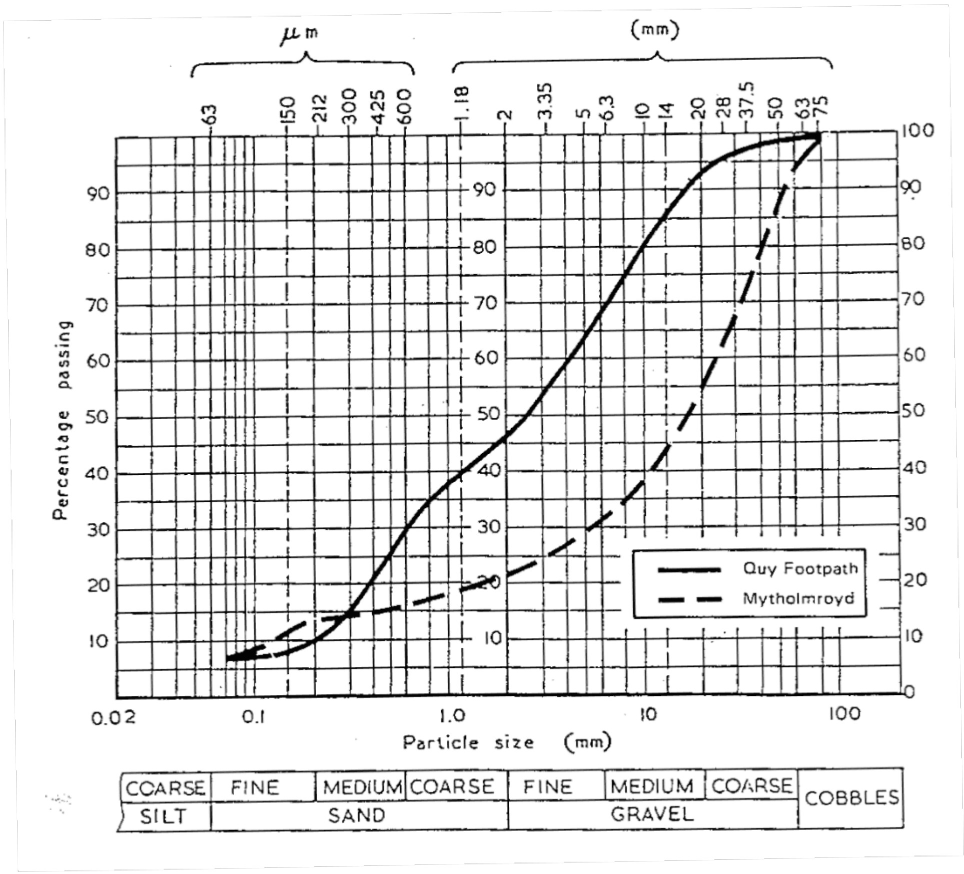

Figure 3.1 Example of particle size distribution for the soil used in tests number 8 and 9. Source: (Johnson, Temporal and Watts, 1989)

The measures reported in the different tests also vary a lot within the 25 papers. The main difference between all tests for the purpose of the thesis is if they performed a live-load test on the culvert or not and then is also important what values they report in case the perform the live-load test. There are tests in which deflections, thrust, moments or strains are reported however there are others where none of those values are presented.

3.3 Database

The database was created while reading the different papers. The routine to put all the information in the database was always the same. The first step consisted of reading the full paper where details of the culvert and the different tests performed on it were explained and get a general idea.

After that the paper was read another time and maybe more if it was necessary to get as much information to fill the database. As it was explained before not all the papers reported the same information, in consequence there were some data impossible to extract depending on the test.

The full database created for this thesis is included in the Appendix A.

3.4 Calculation routines

The first step in the calculations is to extract all the information needed from the report of the different tests. That information concerns points like height of cover (h_c), moment measurements or vehicle information. Once that data is extracted the back calculations of the soil modulus can start using the SDM equations.

3.4.1 Live load

The first point that we need to calculate is the equivalent $p_{traffic}$ load for the tests where a live load test was performed. The SDM propose a method to convert live load, both road live load as railway load, to an equivalent line load and this value become the value used in the different equations within the SDM.

The $p_{traffic}$ line load is based on the stress distribution in a semi-infinite body according to Bussinesq. This concept helps to calculate and convert the live load to an equivalent line load that yields the same vertical stress at the level of the crown of the pipe as the live load itself.

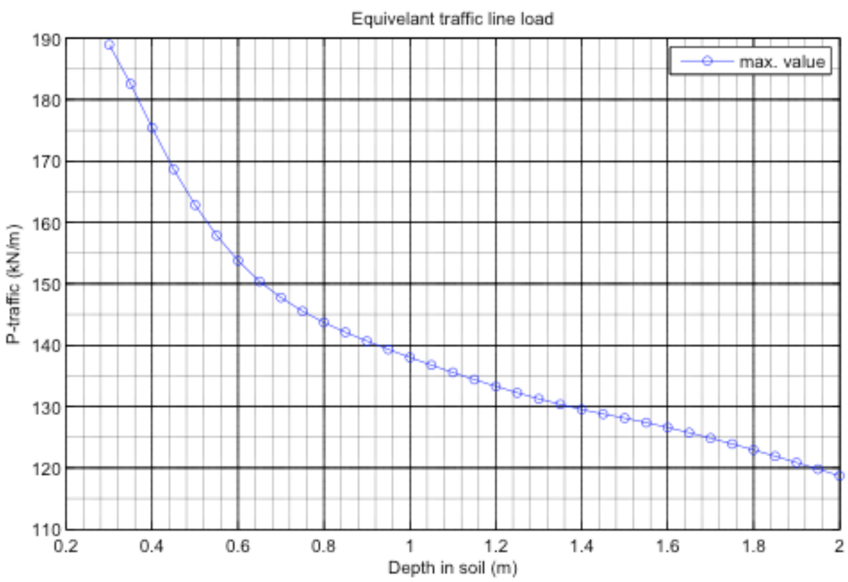

Figure 3.2: Example of $p_{traffic}$ line load of culvert 6 “Newport”

To calculate the equivalent line load the most important information needed is related to the vehicle dimensions. The vehicle load is represented as a group of distributed loads one per tyre and those distributed loads represents how the weight of the vehicle is transferred to the soil. As the different distributed loads corresponds to the contact between the soil and the tyre, tyre pressure and detailed measurements of the axles are necessary to be able to calculate the equivalent line load with higher accuracy. Only in some papers of the 25 this information was reported, for the rest of papers either some assumptions needed to be done to calculate the $p_{traffic}$ line load or there was no possibility to calculate it due to lack of information about the vehicle.

As we can see in figure 2.2 the equivalent $p_{traffic}$ line load decreases while increasing the height of cover (h_c). Once the line load was calculated for the papers where the information needed was available, next step consist on select the exact value in the graph according to the height of cover of every test. The following step is to use this value in the equations from the SDM and back calculate the soil modulus.

3.4.2 Back calculations of soil modulus

The SDM based the calculations on the relationship between the stiffness of the soil and the stiffness of the pipe. Allgood and Tagahashi (1972), proposed this relationship:

$$\frac{M \cdot D^3}{(EI)_s} \tag{3.1}$$

where M is the soil tangent modulus on one-dimensional compression and $(EI)_s$ is the bending stiffness of the culvert.

Duncan (1978) related the bending moments on the culvert to the relationship between the stiffness of the conduit and the soil and based on observations using FEM he introduced the soil tangent modulus at constant confining pressure, E_s. Duncan ended up with the following expression:

$$\lambda_f = \frac{E_s \cdot D^3}{(EI)_s} \tag{3.2}$$

SDM uses the value of this relationship (λ_f) in the equation to calculate the bending moments. The equation is the following one:

$$M_t = f_4^{'} \cdot f_4^{''} \cdot f_4^{'''} \cdot f_4^{IV} D \cdot p_{traffic} + S_{ar} \left(\frac{R_t}{R_s} \right)^{0,75} f_1 \cdot f_{2,\text{cov}\,er} \cdot q \cdot D^2 \tag{3.3}$$

The formula consists of two parts, the first one makes reference to the effect of the vehicle load converted to the equivalent $p_{traffic}$ line load and the other one consider the effect of any evenly distributed load over the culvert.

The factors $f_4^{'}; f_4^{''}; f_4^{'''}\, and\ f_4^{IV}$ varies for each different culvert and take into consideration different aspects like the stiffness relationship (λ_f), the span (D), the height of cover (h_c) and the cross section dimensions (R_t *and* R_s). Those factors are defined by the following equations:

$$f_4^{'} = 0{,}65 \cdot \left(1 - 0{,}2 \cdot {}^{10}\log\left(\lambda_f\right)\right) \tag{3.4}$$

$$f_4^{''} = 0{,}120 \cdot \left(1 - 0{,}15 \cdot {}^{10}\log(\lambda_f)\right) \tag{3.5}$$

$$f_4^{'''} = 4 \cdot \left(0{,}01^{(h_c/D)} + 0{,}1\right) \tag{3.6}$$

$$f_4^{IV} = \left(\frac{R_t}{R_s}\right)^{0,25} \tag{3.7}$$

For all the 25 tests included in this thesis, no distributed load is considered acting over the culvert, that means that $q = 0$ and then the equation of bending moment of the SDM stays like this:

$$M_t = f_4^{'} \cdot f_4^{''} \cdot f_4^{'''} \cdot f_4^{IV} D \cdot p_{traffic} \tag{3.8}$$

next step is, knowing the value of the moment at the crown reported (M_t), to back calculate using the equation above the value of λ_f. The last step is to calculate the back-calculated soil modulus using the formula defined by *Duncan*:

$$E_{sd} = \frac{\lambda_f \cdot (EI)_s}{D^3} \tag{3.9}$$

That is the methodology used to calculate the different soil modulus for the different tests that will be presented and commented with further details in chapter 4 and chapter 5.

3.4.3 Theoretical soil modulus

The SDM manual includes two different empirical methods to calculate the soil modulus. Using *Method A* designers doesn't need information about the soil parameters, they only need the relative degree of compaction (RP) and they will end up with lower and then more conservative soil modulus. Using *Method B,* contrary, designers will get higher values of soil modulus but the sieve analysis of the soil used as backfilling is needed. Next points describe the equations used in each method:

Method A

Designers can use the following equation to calculate the soil modulus. The information needed is the type of material, the degree of compaction and the overburden pressure from the soil cover. Equation used in method A is the following:

$$E_{sd} = \frac{1,2}{\gamma_n} 1,17^{(RP-95)} \left[1,25 \ln(h_c + H/2) + 5,6\right] \tag{3.10}$$

This equation is based on Duncan (1978), and it is applicable to the soil materials classified as SP (= poorly graded gravel), SW (= well-graded sand), GP (= poorly graded gravel) and GW (= well-graded gravel) according to the “Unified Soil Classification System”. The term RP is related to the degree of compaction according to the standard Proctor value. In the cases where the degree of compaction is calculated using the modified proctor the value of RP is calculated as follows: RP = RP (modified Proctor) + 5 %.

Method B

Method B is based on a detailed investigation of the structural backfill material where the following input data are required:

- Particle size distribution
- Dry density and maximum dry density
- $RP = 100 \cdot \left(\rho / \rho_{opt}\right)$
- Stress level in the surrounding fill calculated using the passive earth pressure at a depth equal to the cover depth plus H/2

The tangent modulus is determined in the following stages:

The void ratio is calculated using the equation

$$e = \frac{\rho_s}{\rho} - 1 \tag{3.11}$$

where ρ_s is set equal to 26 kN/m^3

The modulus ratio is calculated as

$$m = 282 \cdot C_u^{-0,77} \cdot e^{-2,83} \tag{3.12}$$

where the uniformity coefficient C_u is calculated as

$$C_u = d_{60} / d_{10} \tag{3.13}$$

The stress exponent is calculated as

$$\beta = 0,29 \cdot {}^{10}\log\left(\frac{d_{50}}{0,01}\right) - 0,065^{10}\log\left(C_u\right) \tag{3.14}$$

where the values of d_{50} is inserted in mm. The characteristic angle of internal friction of the structural backfill material is calculated as

$$\varphi_k = 26^\circ + 10 \cdot (RP - 75)/25 + 0{,}4 \cdot C_u + 1{,}6 \cdot {}^{10}\log(d_{50}) \tag{3.15}$$

In the above equation the relationship between the relative density and degree of compaction is assumed to follow the relationship $I_D = (RP - 75)/25$ with the degree of compaction determined as the Standard Proctor.

$$\rho = \rho_{opt} \frac{RP}{100} \tag{3.16}$$

The tangent modulus is calculated according to:

$$E_{s,d} = \frac{1}{\gamma_n \gamma_m} \left[1 - \frac{R_f \cdot (1 - \sin\varphi_k) \cdot (\sigma_1 - \sigma_3)}{2 \cdot \sigma_3 \cdot \sin\varphi_k} \right]^2 k_v \cdot m \cdot p_a \cdot \left(\frac{\sigma_3}{p_a} \right)^{1-\beta} \tag{3.17}$$

$$k_v = \frac{1 - \nu - 2\nu^2}{1 - \nu} = \frac{\sin\varphi_k \left(3 - 2\sin\varphi_k\right)}{2 - \sin\varphi_k} \tag{3.18}$$

After insertion of $R_f = 0{,}7$, $p_a = 100$ kPa and the stress situation equivalent to the passive earth pressure at the level immediately under the quarter-points of the culvert, we obtain

$$E_{s,d} = \frac{1}{\gamma_n \gamma_m} 0{,}42 \cdot m \cdot 100kPa \cdot k_v \left(\frac{\left(1 - \sin\varphi_k\right) \cdot \rho_2 \cdot S_{ar}\left(h_c + H/2\right)}{100kPa} \right)^{1-\beta} \tag{2.19}$$

The equation above is the one has been used to calculate the theoretical soil modulus presented in Chapter 5and 6.

4 Full scale tests

4.1 General

Abstracts are created for each one of the full scale tests studied in this thesis. The abstracts were created to help future researchers to understand and get a general idea of the test without necessity of looking into the details of the report. Obviously not all the information reported is included in the abstract. The abstracts are complemented by the database created which includes more details about the full scale tests.

The abstracts can be divided in two separate parts. One is the written part where information about the author, the culvert, the place where the test was performed, the measurements the measure during the test, etc… are included. The other part is an schematic image to represent the type of culvert and also include information about the measures of the culvert, the height of cover and the soil.

In the following section the twenty-five different abstracts of full scale tests are presented.

4.2 Abstracts

1. FIELD VERIFICATION OF RING COMPRESSION CONDUIT, USA, 1981

This article was written by J. Demmin and talks about a test performed in July 1963 by Armco-Thyssen. The aim of the article is to test a pipe arch of 6,3 m span, 3,965 m rise, 4,8 mm thickness and 150×50 mm corrugations under a live-load and a loading to failure tests. Strains where measured by strain gage strips placed in six different points in two sectional planes. Deformations where measured photographically at 12 different points of the section plane. The live-load test was performed using 1 m of cover high while for the loading to failure test 1,6 m of cover high was used. Railways ties 2,6 m long placed on the surface grade parallel to the pipe-arch axis supported the load covering an area of 8,19 m^2 that increases until reach the value of 15 m^2 due to the settlement of the ties and the sag of the slabs. Soil pressure was measured by 3 gage points placed 101,6 mm above the crest. One was placed in the load centre directly above the pipe-arch crest, the other two were located one on each side at a distance of 2 m from the centre.

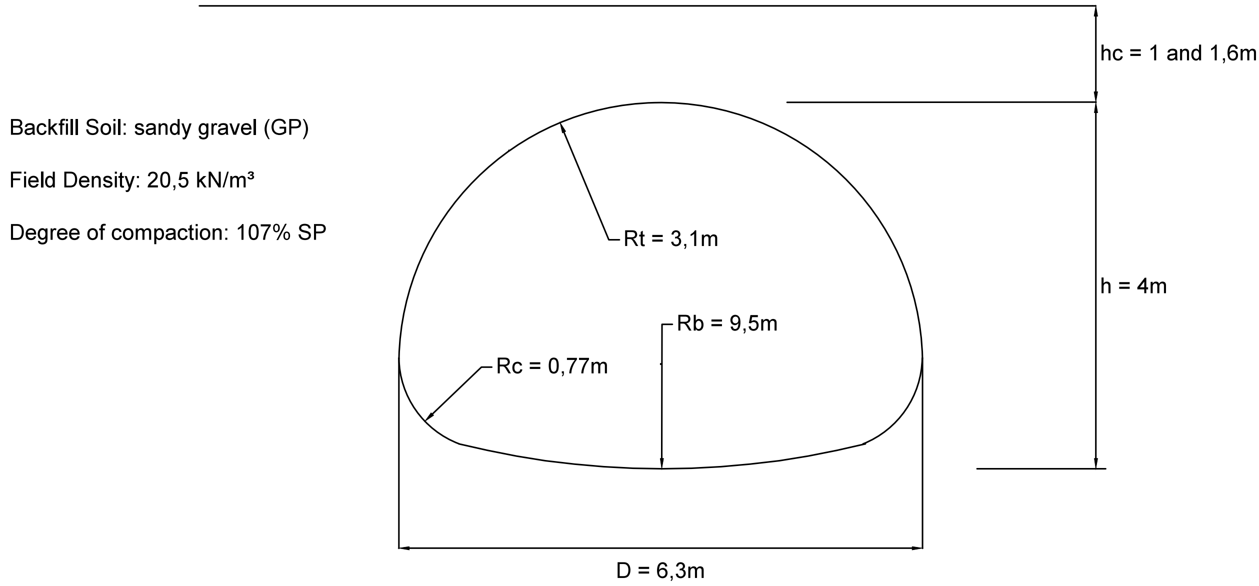

Figure 4. 1: Cross section of culvert 1

2. HEAVY-VEHICLE LOADING OF ARCH STRUCTURES METAL AND SOIL, USA, 1981.

In this article a three circular radius corrugated steel arch of 12m span was tested with a live-load test. The article was written by J. N. Kay, D. K. Avalle, R. C. L. Flint and C. F. R. Fitzhardinge. Electricity Trust of South Australia constructed the arch on the Hawker-Maree highway. The culvert had a corrugation of 150x50 mm. The live-load test consists on two trucks, one of 50.000 kg and other of 170.000 kg, crossing the culvert. Measurements were taken during backfilling and in different positions of the two trucks. The University of Adelaide installed all the instruments and they measure vertical deflections in the arch, axial strains in the corrugated steel wall, slip between plates at the lap joints and settlements in the soil adjacent to the structure. The vertical deflections were measured at three points in two different sections along the axis. To measure the strains in the steel they used electrical resistance strain gauges placed in 14 points, 7 at each of the two different sections. A torpedo containing a mercury chamber in which the pressure was measured by an electrical pressure transducer measured the soil settlement but finally they couldn't get any results due to the high temperatures which provoke a flow of mercury in the system and in consequence errors around 3 mm appeared and therefore the accuracy was not enough and the results were not reported.

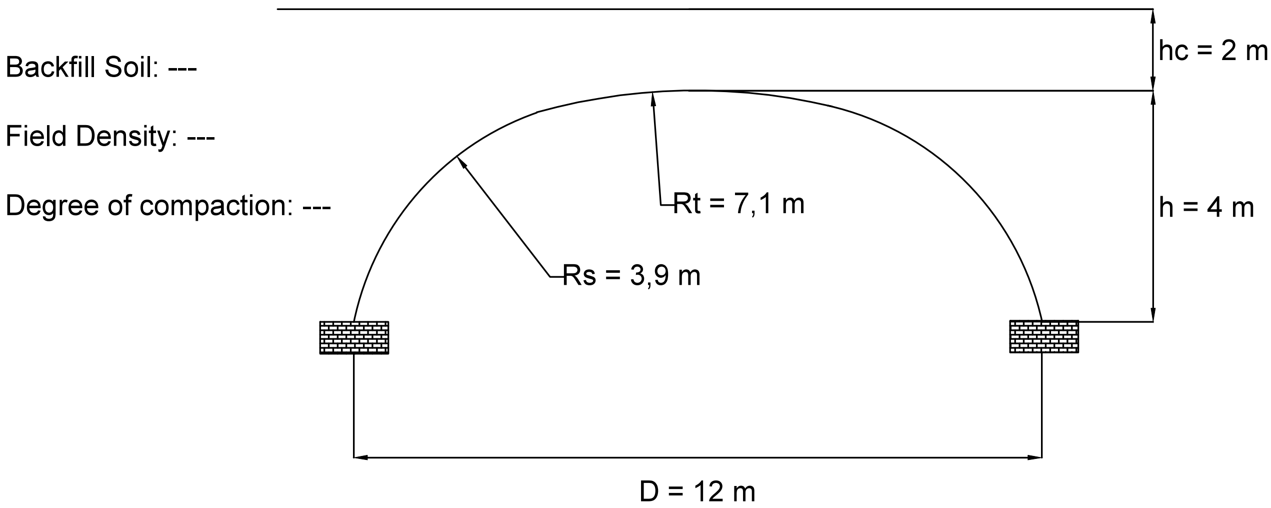

Figure 4. 2: Cross section of culvert 2

3. MEASUREMENTS OF SOIL ARCHING ABOVE A LARGE FIEMETER FLEXIBLE CULVRT, CANADA, 1975

In this test a 15,5 m in span multiple flexible steel arch covered by 13,4 m high embankment was tested to see the influence of the arching effect in the structure. Guy Lefebvre, Maurice Laliberté, Liguori M. Lefebvre, Jean Lafleur and C. L. Fisher wrote this article. The test took place in Vieux Comptoir river on the James Bay access road, 467 km north of Matagami.

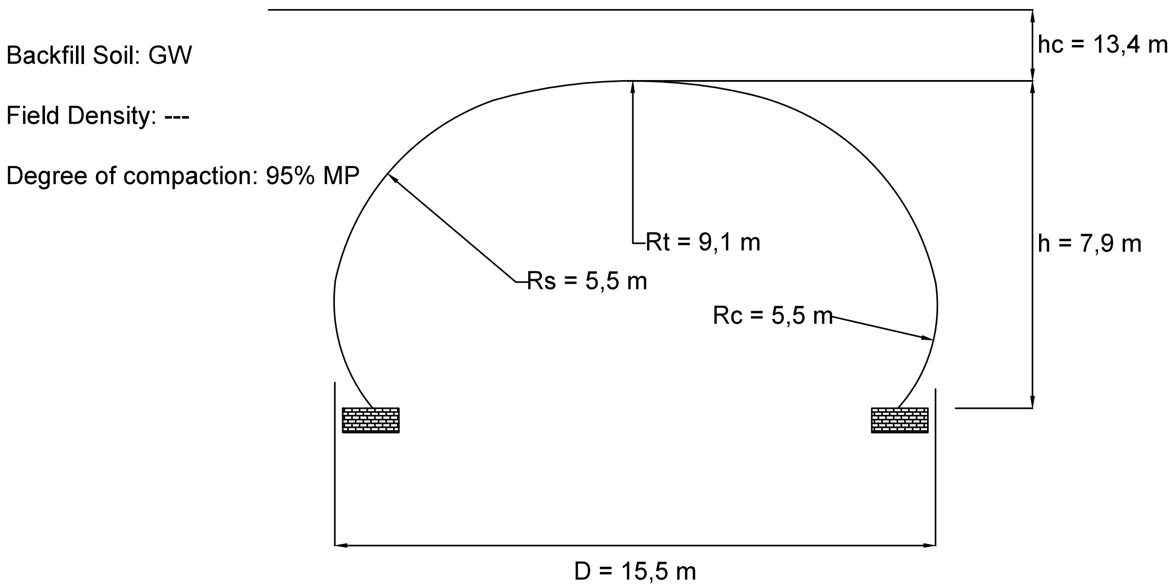

Figure 4. 3: Cross section of culvert 3

They used a corrugated steel of 6,86 mm thickness and 152x51 mm corrugations. Horizontal thrust beam and stiffener ribs embedded in the concrete thrust beams were constructed along the axis of the culvert. The backfill material consists of compacted well graded natural gravel placed around the culvert. In the test they measured vertical and horizontal displacements, earth pressures and stresses in the steel. The arching effect was measured by earth pressure measurements at different points in the embankment. Strains in the steel have been measured at seven different points in two different sections but only the results in the central section are reported.

4. MEASUREMENTS AND ANALYSES OF COMPACTATION EFFECTS ON A LONG-SPAN CULVERT

In this report, a 38 ft 5 in (12 m) span flexible metal culvert with 9x2,5 in (228,6x63,5 mm) corrugation was tested at different positions of backfilling and at different stages of compaction. The Culvert was located in Mesa, California. The paper was written by Raymond B. Seed and Chang-Yu Ou. The deformations of the culvert were measured and then compared with the results of finite elements analyses to see the influence of the compaction of backfilling on culvert stresses and deformations. The culvert was designed as a bridge in the intersection of two roadways. The values of deformation were measured at 13 different points placed in two different sections separated 19,7 ft (6 m). In the test they used different construction equipment for compacting. The material used as backfill was angular silty sand placed to a minimum width of 4 ft (1,2 m) at both sides of the culvert and continued to the final fill surface.

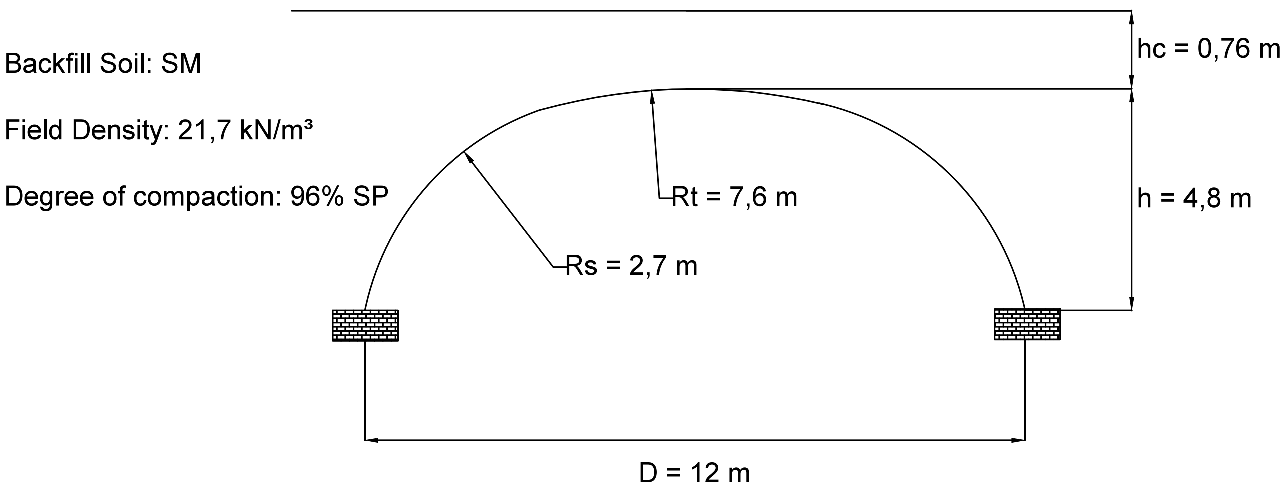

Figure 4. 4: Cross section of culvert 4

5. MEASURED PERFORMANCE OF NEWTOWN CREEK CULVERT, USA, 1979

In this report a 26 ft (7,9 m) span arch with a 23 ft (7 m) compacted-soil cover and 6x2 in (150x50 mm) corrugation was tested. The structure was developed as a highway crossing over Newton Creek in Bucks Country, Pa. The test was performed by Ernest T. Selig, Charles W. Lockhart and Richard W. Lautensleger. Two different types of backfilling were used, one immediately surrounding the structure called structural fill and other one used for the rest of the embankment which consists of sandy and clay called embankment soil. Soil measurements, like strains and stresses, and structure measurements, like deflection of culvert and strain in steel, are reported in the document. In the test they didn't perform any live-load test, they only measurement they took were during construction depending on the high of fill. Arching observations around the culvert are also reported.

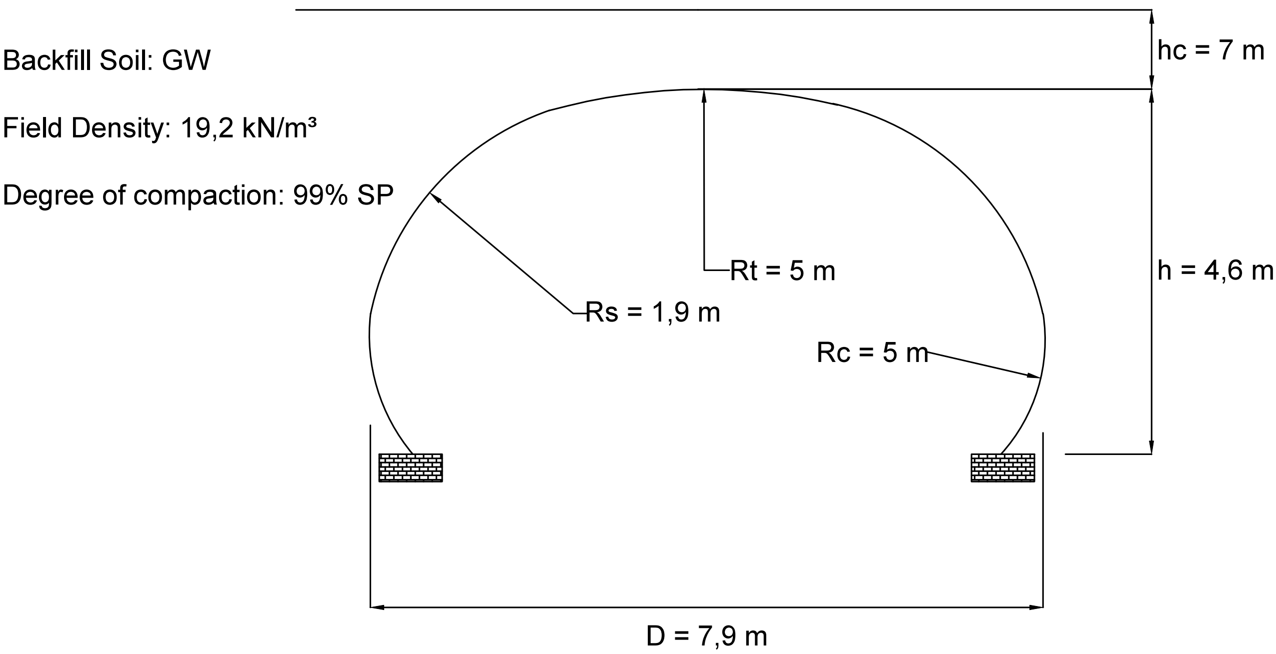

Figure 4. 5 Cross section of culvert 5

6. LOADING TESTS ON AN ARMCO PIPE ARCH CULVERT, UK, 1985

In this report Armco Ltd and the Transport and Road Research Laboratory conducted a test on a 4 m span corrugated steel culvert with corrugations of 100x20 mm. The report was written by J. Temporal, D. A. Barratt and B. E. F. Hunnibell. They performed some loadings tests using a single axle trailer and starting with a high of cover of 1,5 m down to 0,36 m. The weight of the axle trailer also changed from 12.200 kg up to 48.800 kg. During the test they measured ring compression and bending strains at 19 locations in the central section of the culvert, deformation and soil pressures at each depth of cover and for five different positions of the loaded axle, spaced at 0,45 m intervals. First position of the trailer was over the crown and the last position was offset by 1,8 m. Soil pressures were measured by 20 pressure cells installed over the culvert. Finally the culvert failed with a 0,36 m of depth of cover and with a 48.800 kg load applied on the axle.

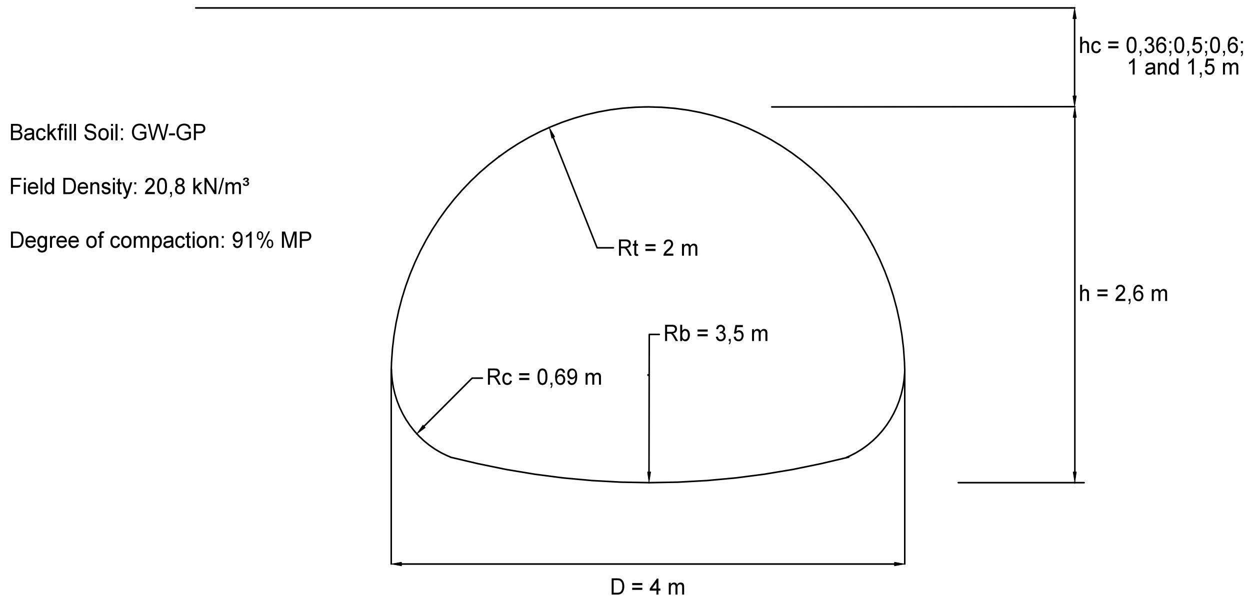

Figure 4. 6: Cross section of culvert 6

7. LONG SPAN REINFORCED STEEL BOX CULVERT, USA, 1998

In this report two box culverts of 12 m span, 3,2 m rise with 380x140 mm corrugations were tested under a live-load test using a test vehicle following six load paths. Thomas C. Mc Cavour, Peter M. Byrne and Timothy D. Morrison wrote this article.

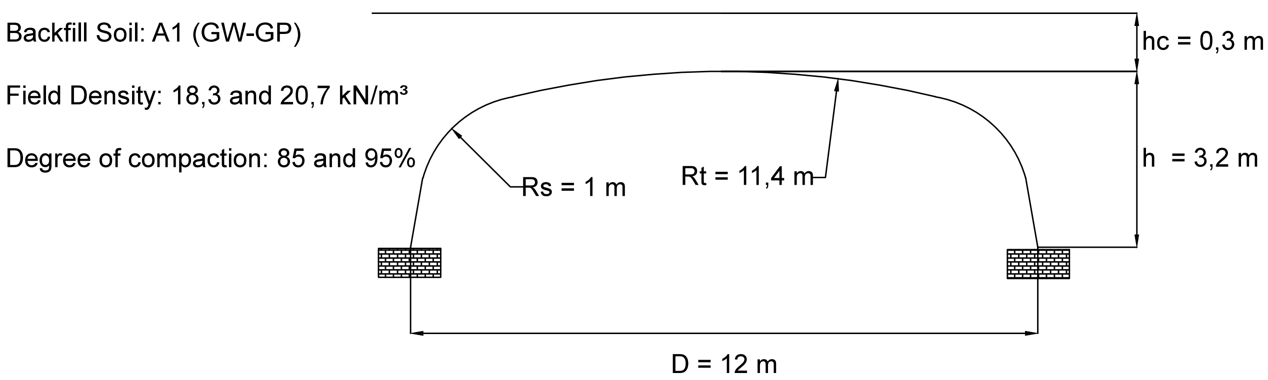

Figure 4. 7 Cross section of culvert 7

During the test strains and deflections were measured by 64 strain gauges placed at 14 stations and 10 displacement transducers for the six different positions of the vehicle on the surface of the structure. The two box culverts were erected at Dorchester, New Brunswick separated by 51 mm gap. Both culverts were tested using two different backfilling densities, 85% and 95% of Standard Proctor density. One of the culverts was reinforced with continuous deep-corrugated crown stiffeners and the other one was intermittently reinforced with concrete metal-encased stiffeners. After the test the measures were compared with a nonlinear soil-structure interaction program (NLSSIP) and then discussed by the authors of the report.

8. - 9. THE EFFECT OF PAVEMENT LAYERS ON THE BEHAVIOUR OF CORRUGATED STEEL CULVERTS, QUY FOOTPATH AND MYTHOLMROYD, UK, 1989

In this report two different examples of corrugated steel culverts were studied. One case is the Quy footpath and the other one is the Mytholmroyd's culvert. The first one is a 3,7 m span high profile pipe arch with corrugations of 152,4x50,8 mm that carries a pedestrian footpath in the A45 Cambridge Northern Bypass. The second one is a 6 m span high profile pipe arch with 200x55 mm corrugations constructed to carry an access road over Rochdale Canal at Moderna Bridge, Mytholmroyd, West Yorkshire. The test and the report were performed by P.E. Johnson, J. Temporal and G.R.A. Watts. Load tests were carried in both culverts using a single axle load with a maximum load of 455 kN.

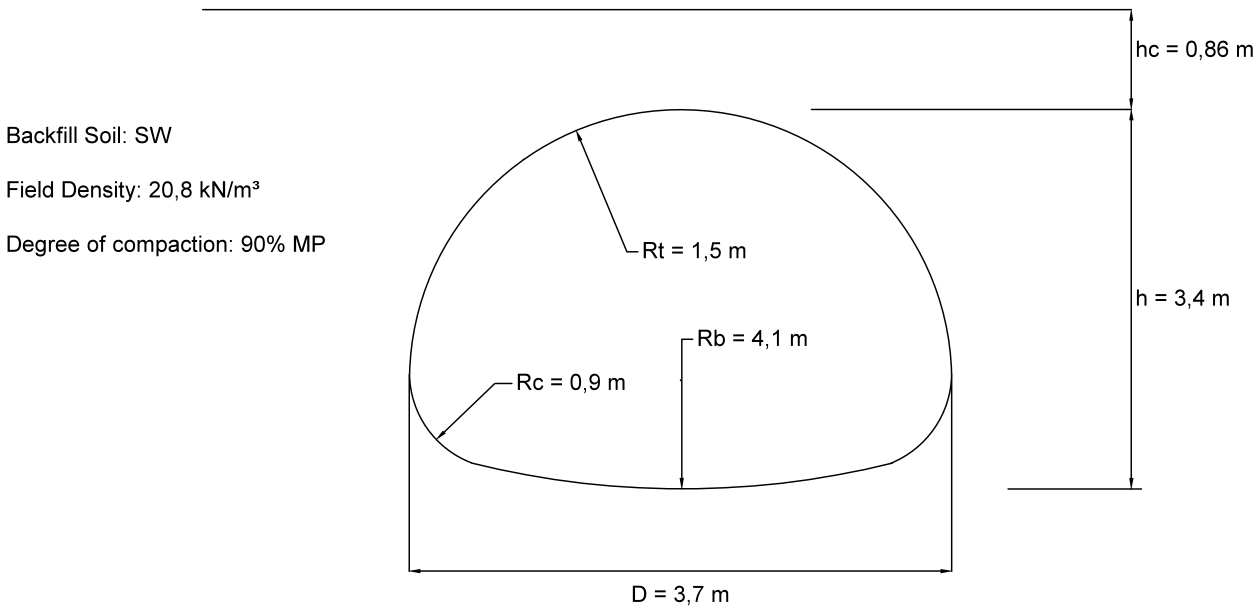

Figure 4. 8: Cross section of culvert 8

The high of cover was different for each case, in the first one a 0,86 m of high of cover was used and the second case was tested with 0,7, 1,2 m height of cover unpaved and 1,2 m including pavement. In both case they measured the deflection of the culvert, the bending strains and the ring compression. It's important to say that the results of the measures for the second culvert with 1,2 m and 1,2 m (including the pavement) high of cover were quite different. The effect of the pavement reduced the values of the deflection and the strains of the culvert. The deflections and the strains of the culvert of 1,2 m height of cover including the pavement were 27,8% and 43,4% less than the values measured in the 1,2 m high of cover without pavement. They used 3 measuring point in the Quy footpath and 7 measuring points in the Mytholmroyd's culvert.

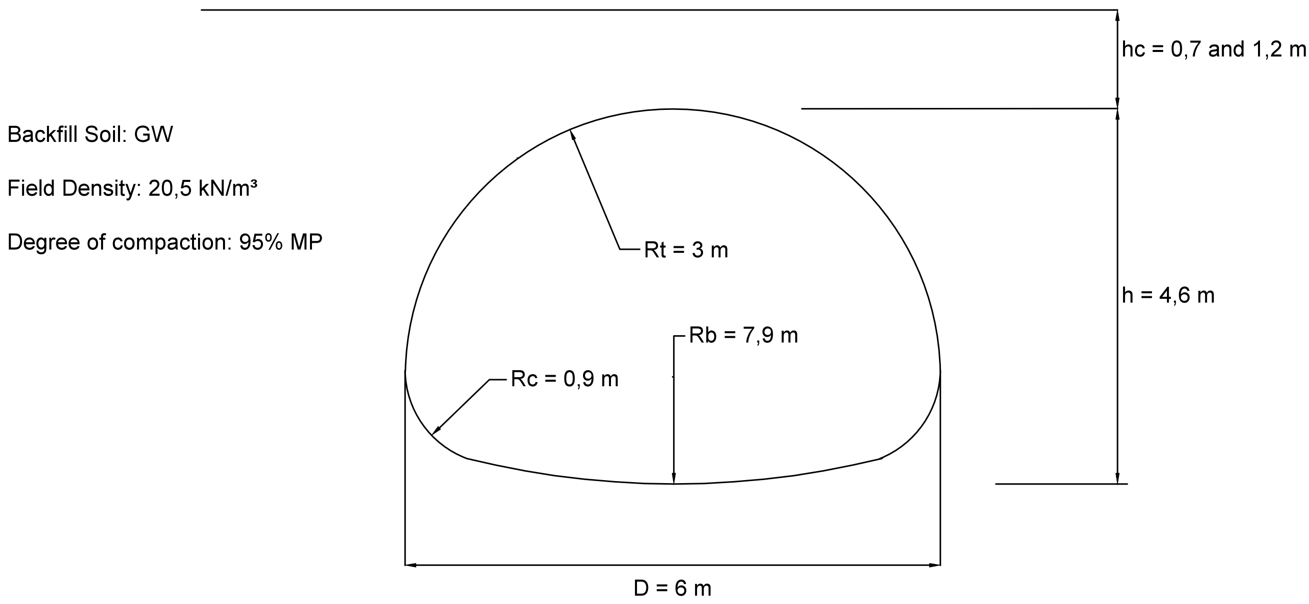

Figure 4. 9: Cross section of culvert 9

10. – 11. LOADING TESTS ON TWO LONG SPAN CORRUGATED STEEL CULVERTS, QUY WATER AND STONE HILL, UK, 1988

In this report two long span corrugated steel culverts have been tested under a live load test using a single axle loads up to 455 kN. One of the culverts is a 9,8 m span horizontal ellipse with 152,4x50,8 mm corrugations placed in the intersection between Quy Water stream and a section of the A45 Cambridge Northern Bypass. The other culvert is a 7,5 m span high-profile arch with 152,4x50,8 corrugations constructed in the intersection between a branch of the Little Avon river and a section of the old A38 road between Bristol and Gloucester close to Stone. The authors of the paper are J. Temporal and P. E. Johnson. During the tests deflections of the culvert and steel strains were measured at different measuring points of each culvert and for different positions of the loaded single axle. The load of the single axle was increased from 122 kN progressively up to 455 kN. They used 7 measuring points for both culverts. Finally the compared and discussed the values obtained from the loading tests with the design methods available for long span corrugated steel culverts.

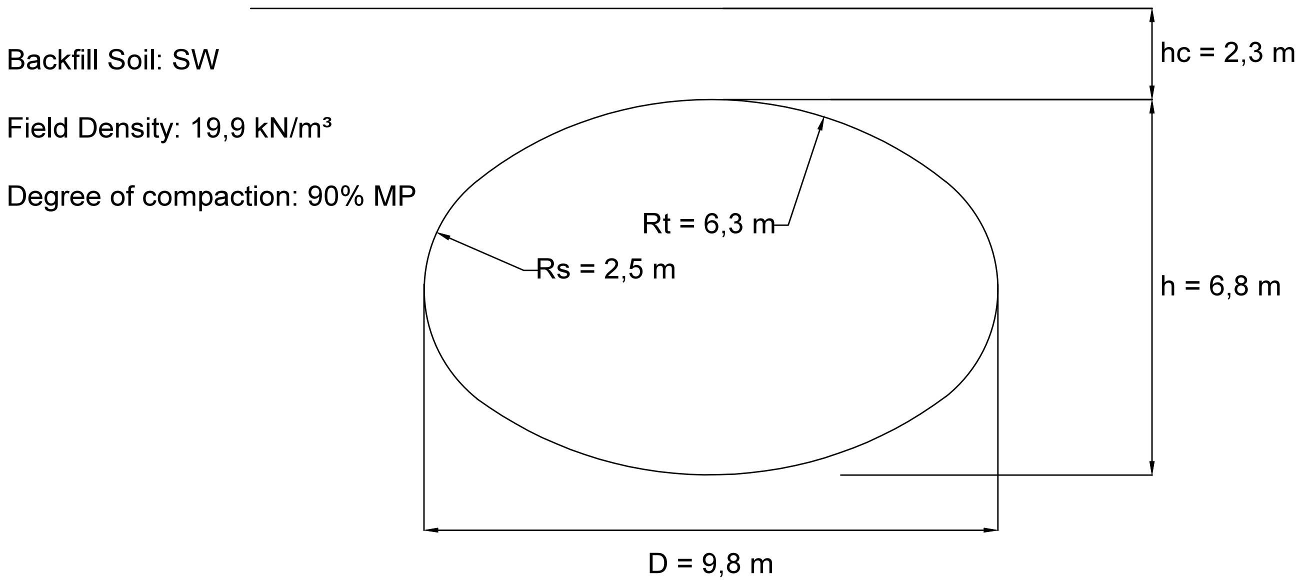

Figure 4. 10: Cross section of culvert 10

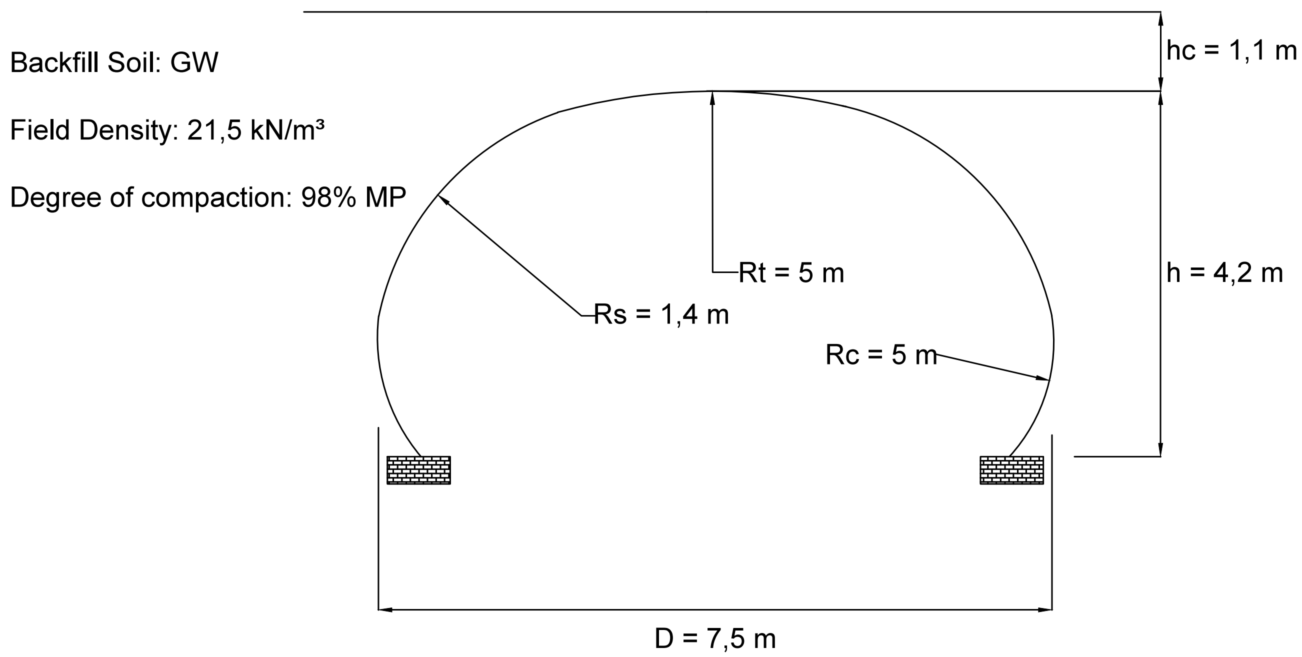

Figure 4. 11: Cross section of culvert 11

12. LIVE-LOAD RESPONSE OF A SOIL-STEEL STRUCTURE WITH A RELIEVING SLAB, THUNDER BAY, ONTARIO, CANADA, 1983

This report talks about a loading test done on a horizontal elliptical culvert with a reinforced-concrete relieving slab placed on the top of the structure. The culvert was 8,8 m span and 4,9 m rise with corrugations of 152x51 mm. This article was written by Baidar Bakht. The structure is called the McIntyre River Bridge and is located in Thunder Bay, Ontario, Canada. During the test strains and deflections of the culvert were measured. The aim of the test was to compare the behaviour of two similar culverts but one with a relieving slab on the top of the cover and the other without it. The structure they choose for the comparison was the Adelaide Creek structure. It was also a horizontal elliptical culvert of 7,2 m span and 4,1 m rise. The same vehicle and vehicle positions were used in both cases to see the influence of the relieving slab on the strains and deflections of the culverts. The fact of using a relieving slab reduced maximum thrust by about 50%. On the other hand the values of moment were quite similar.

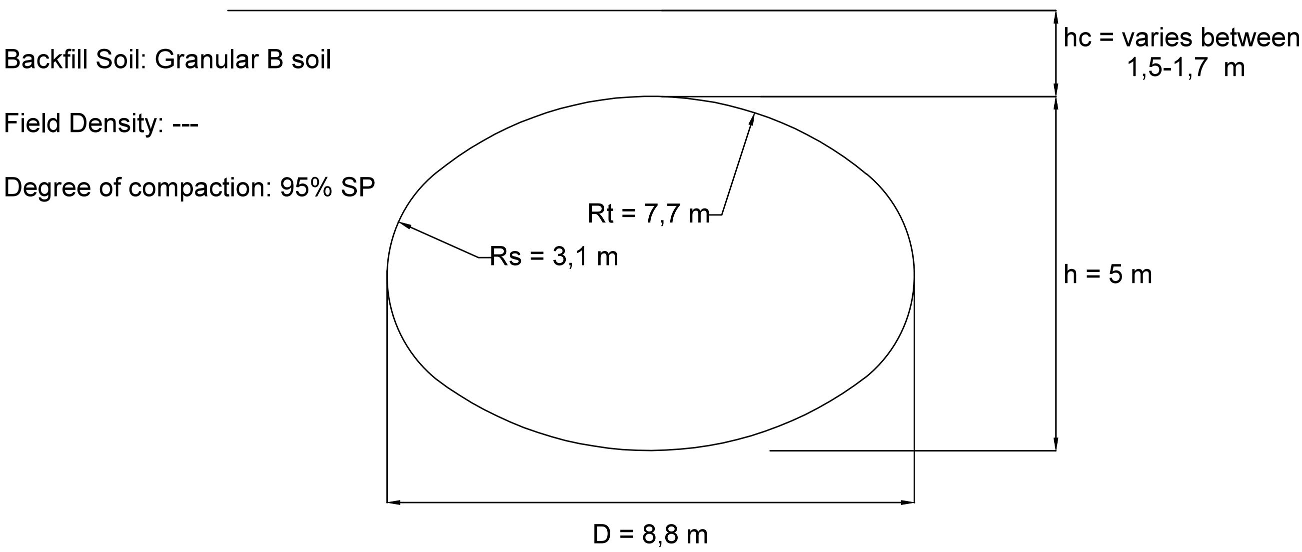

Figure 4. 12: Cross section of culvert 12

13. PERFORMANCE OF A LARGE CORRUGATED STEEL CULVERT, THUNDER BAY, ONTARIO, CANADA, 1975

In this report a 27 ft (8,2m) span and 16 ft (4,9m) height elliptical corrugated steel culvert is tested. The test was placed in Thunder Bay, Ontario, Canada and was performed and reported by Ernest T. Selig, State University of New York and Salvatore J. Calabrese. The height of cover was around 20% of the span including a concrete relief slab placed in the top. Soil pressures, strains in the backfilling and structural deformation of the culvert were measured. All of these values were measured using embedded stress gauges, horizontal and vertical extensometers and radial extensometers respectively. Measurements were taken during construction and under live load tests. The live load test was conducted by using a truck with a gross weight of 32.800 lb (14.900 kg) and measures were taken for different positions of it. Finally the culvert was tested under a 162.000 lb (73.500 kg) live load using three gravel trucks positioned side by side on the slab. The results with the slab indicated that the slab was effective for the distribution of the load created by the vehicle and in stiffening the soil-steel structure.

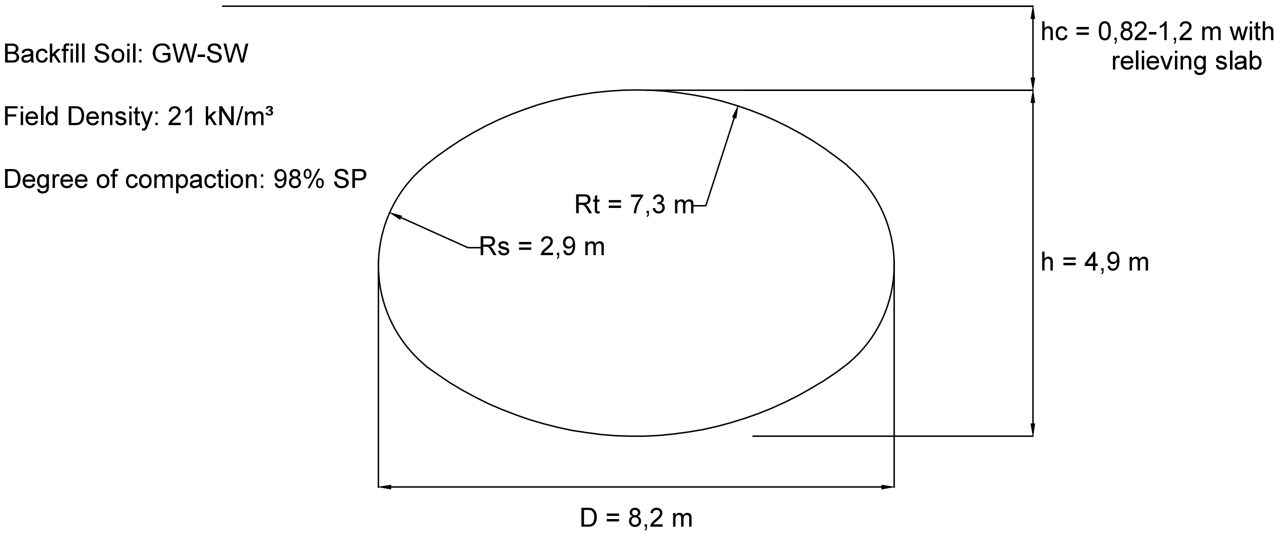

Figure 4. 13: Cross section of culvert 13

14. DEFLECTION OF FLEXIBLE CULVERTS DUE TO BACKFILL COMPACTION, WALNUT CREEK, CALIFORNIA

In this report they tested a 25 ft 1 in (7,7 m) span 12 ft 11 in (3,9 m) rise horizontal ellipse culvert. The culvert was constructed in Tice Valley in Walnut Creek, California, about 20 miles (32,2 km) east San Francisco. The authors of this article are J. M. Duncan and J. K. Jeyapalan. Measurements were taken during backfilling and no live load tests were carried through. During the test they measure change in span, change in rise, deflection of quarter points relative to invert and vertical movements of quarter points relative to haunch. Measuring instruments consisted on gauges installed at two different sections of the culvert. Finally they compared the values of the different measures they took with a finite-element program and some conclusions are reported.

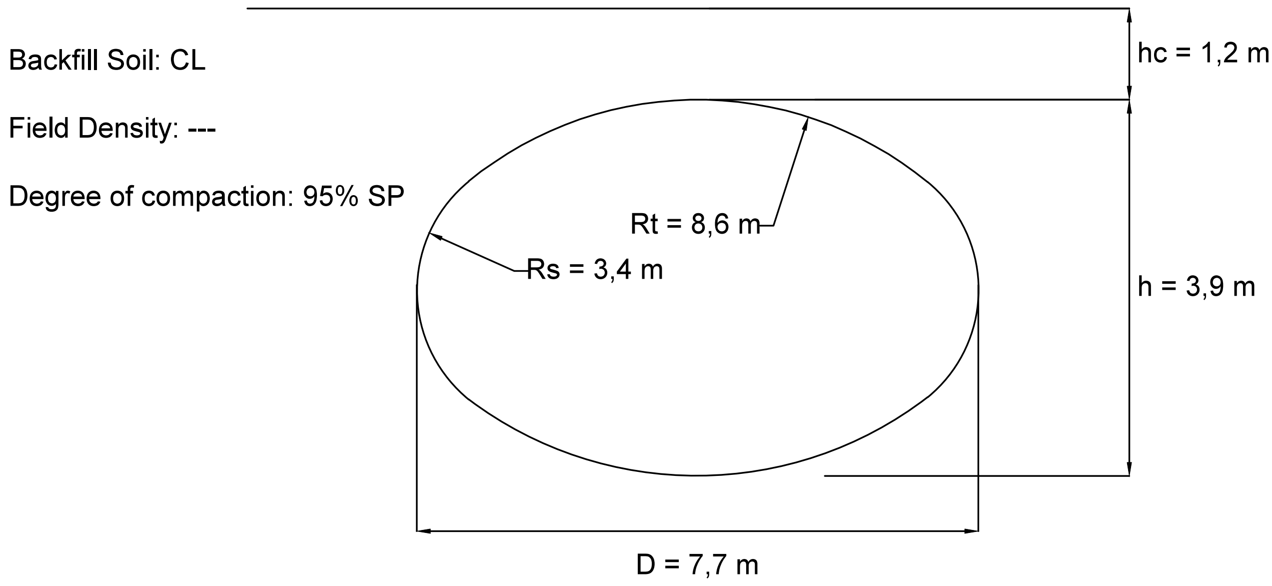

Figure 4. 14: Cross section of culvert 14

15. PERFORMANCE AND ANALYSIS OF A LONG-SPAN CULVERT, BUCKS COUNTRY, PENNSYLVANIA, USA, 1978-1979

In this report long-span flexible corrugated-steel low profile-arch culvert with 6x2 in (150x50 mm) corrugations is tested. The culvert is 37 ft 10 in (11,5 m) span, 15 ft 8 in (4,8 m) rise and 11 ft (3,4 m) of high of cover. Michael C. Mc Vay and Ernest T. Selig wrote this article. The structure was constructed in Bucks Country, Pennsylvania, as a bridge-replacement structure.

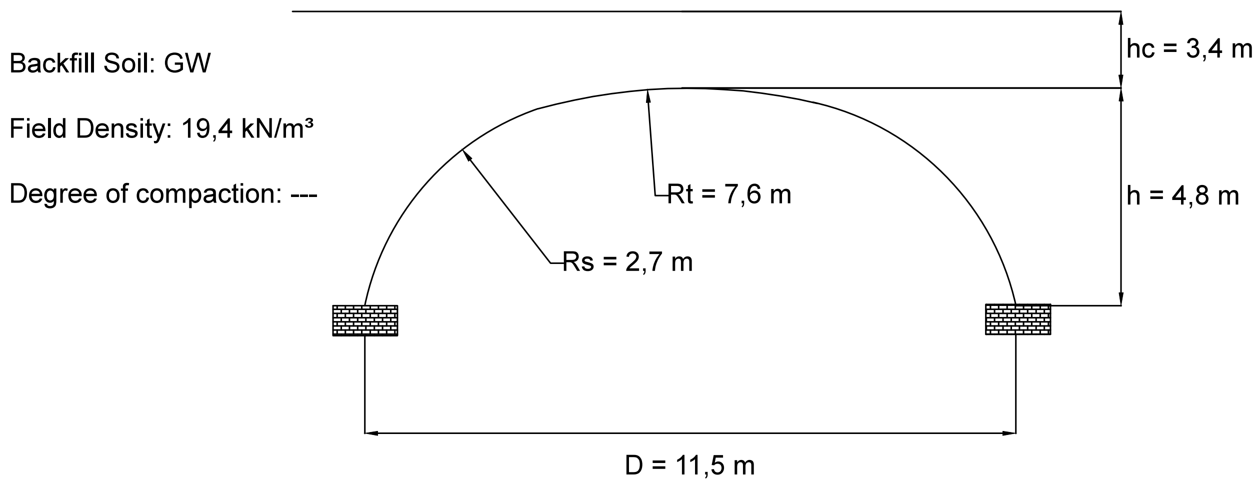

Figure 4. 15: Cross section of culvert 15

The construction began in September 1978 and ended in January 1979. Strains in the steel, displacement of the culvert and soil strains were measured during backfilling. No live-load tests were carried through. Comparison between measured values and predicted values from a computer model of culvert displacement, soil stress, soil strain, culvert thrust and bending stress are reported.

16. BEHAVIOR OF ALUMINUM STRUCTURAL PLATE CULVERT, FRIENDSHIP, NEW YORK, USA

In this report a 28 ft 6 in (8,7 m) span and 11 ft 9 in (3,8 m) rise with a 9x2,5 in (228,6x63,5 mm) corrugations is tested and measurements are taken during backfilling and during a live load test.

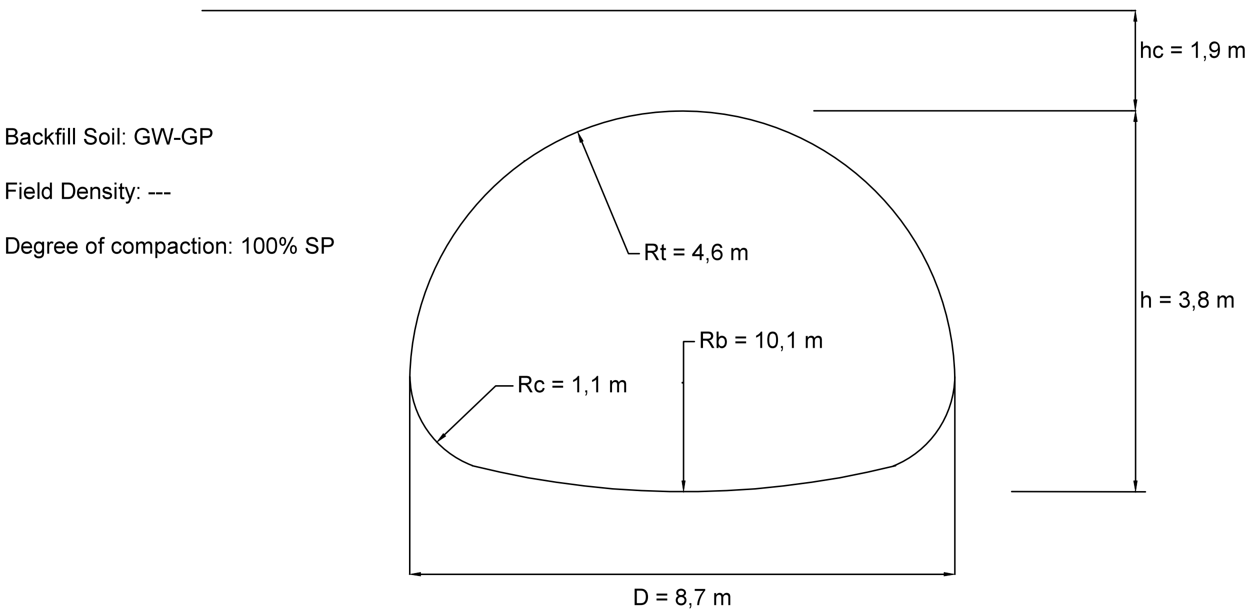

Figure 4. 16 Cross section of culvert 16

The structure was constructed to carry the Van Campen Creek under State Route 275 in the town of Friendship, New York. The test and the article were performed by David B. Beal. Changes in culvert shape, slippage at the joints and steel strains were measured over the longitudinal centreline at two sections 13 ft 6 in (4,1 m) on either side of the centreline. Steel and air temperatures were also monitored during the tests. The vehicle used for the live load test consisted on a truck of 20 kip (9071,9 kg) rear axle and it was located at different positions over the top of the culvert.

17. 18. 19. 20. IN-SITU LOAD TESTING OF CORRUGATED STEEL PIPE-ARCH CULVERTS, OHIO, USA, 2008

In this report four steel pipe arch culverts with 152x51 mm corrugations are tested under a live load test using a truck of approximately 32000 kg weight. The four culverts are situated on State Route 19 near Mt. Gilead in Morrow County, Ohio; on State Route 38 near London in Madison County, Ohio; on State Route 95 near Chesterville in Morrow County, Ohio and in Bradysville, Ohio.

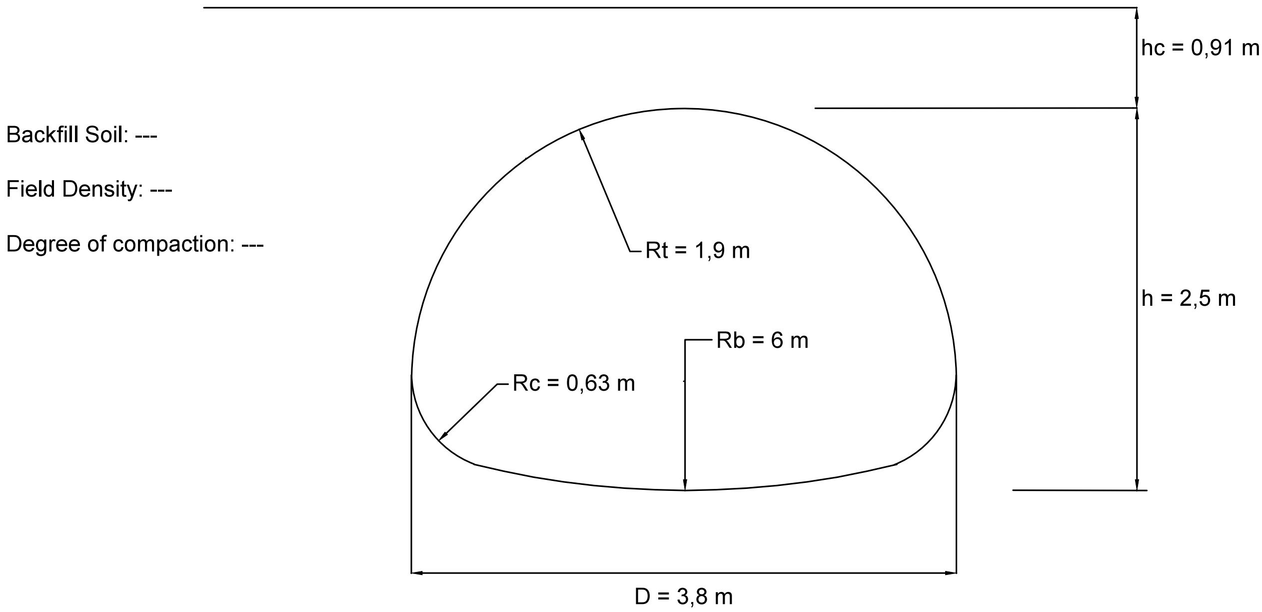

Figure 4. 17 Cross section of culvert 17

The authors of this article are Halil Sezen, Kyong Y. Yeau and Patrick J. Fox. The culverts have spans of 3,8; 3,4; 4,3 and 3,7 m respectively.

During tests deflection of the culvert and strains in the steel were measured for ten different positions of the truck. Statics and dynamics tests were performed in the four culverts. Dynamic tests were conducted at speed varying from 8 to 64 km/h.

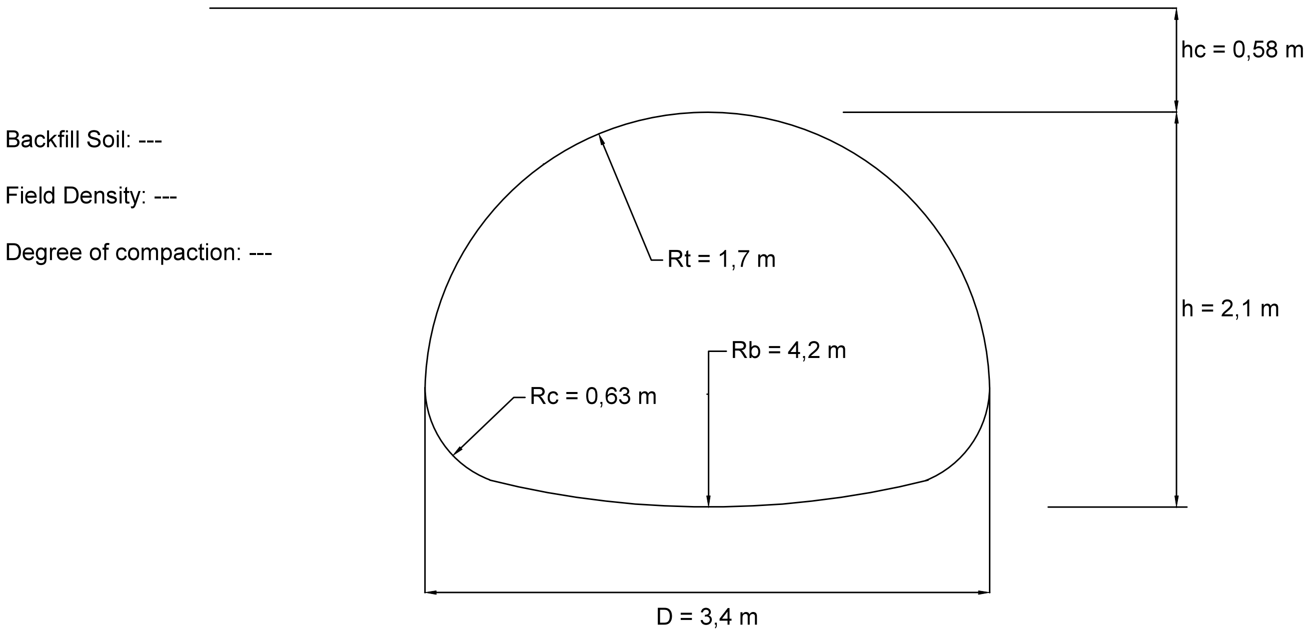

Figure 4. 18 Cross section of culvert 18

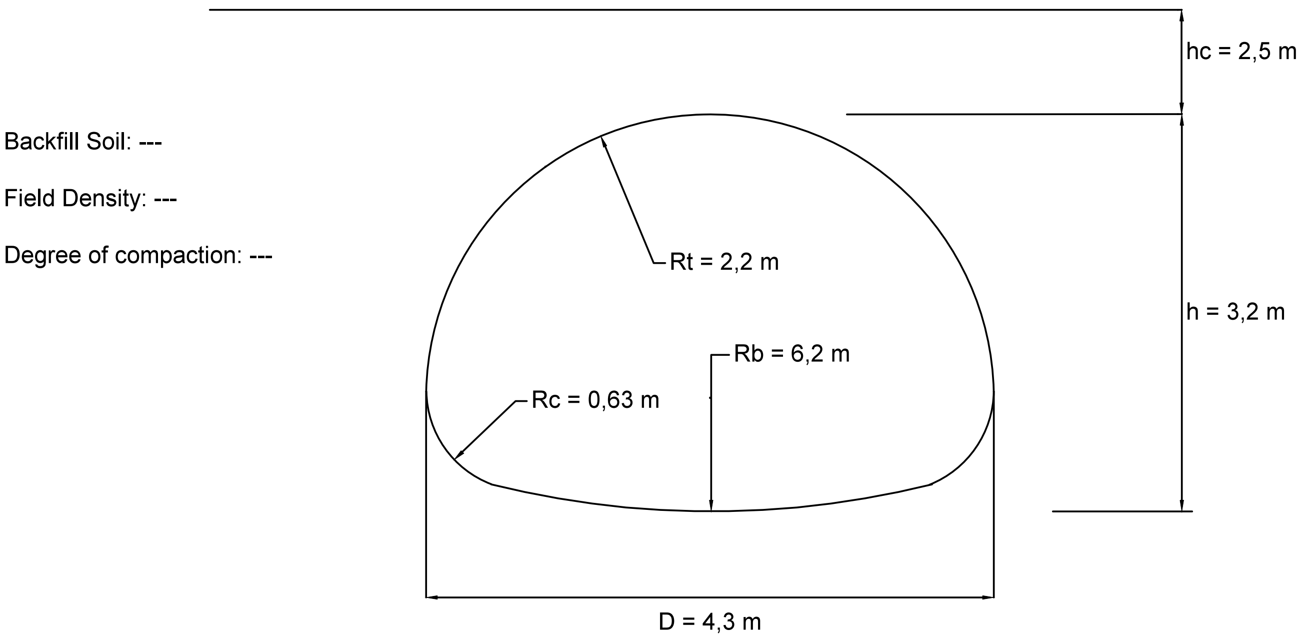

Figure 3. 19 Cross section of culvert 19

Backfill Soil: ---

Field Density: ---

Degree of compaction: ---

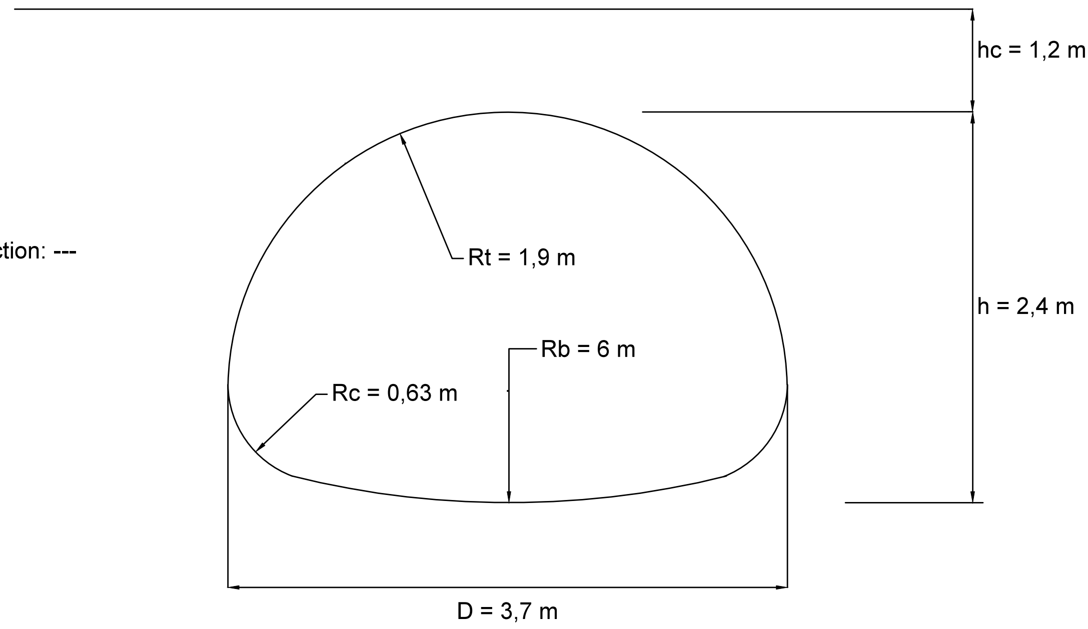

Figure 4. 20 Cross section of culvert 20

21. FULL-SCALE FIELD TESTS ON FLEXIBLE PIPES UNDER LIVE LOAD APPLICATION, FLORIDA, USA, 2006

In this report six flexible pipe types were tested under a live load test. The tests were performed at Florida DOT (FDOT) maintenance yard on Pringhill Road in Tallahassee during December 2001 to May 2002.

The report was written by Madasamy Arockiasamy, Omar Chaallal and Terdkiat Limpeteeprakam. The pipes were three high-density polyethylene pipes (PE36a, PE36b and PE48), one polyvinyl chloride pipe (PVC36), one aluminium pipe (Al36) and one steel pipe (St36). The important one for the report is the last one because is made by steel with 191x19 mm corrugations.

It has a nominal pipe diameter of 0,9 m and the modulus of elasticity is 200 MPa. During the test they measure vertical deflections, horizontal deflections, soil pressures and steel strains. They used a FDOT truck for the loading tests. They finish the report with a comparison of the measured values and the predicted values of a finite element method.

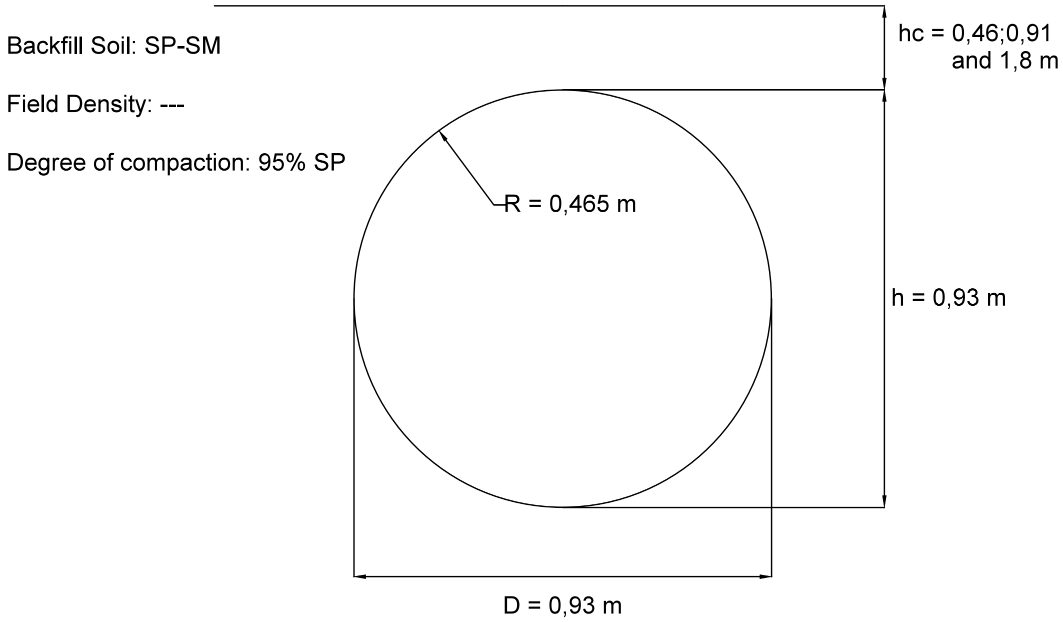

Figure 4. 21 Cross section of culvert 21

22. MEASURED FIELD PERFORMANCE AND COMPUTER ANALYSIS OF LARGE-DIAMETER MULTIPLATE STEEL PIPE CULVERT, OHIO, USA, 2008

In this report a 6,4 m diameter corrugated steel pipe with 152x51 mm corrugations was constructed under a 22,9 m height of cover. The structure was located on Nease Creek, 4,8 km southeast of the junction of State Routes 7 and 124 in Meigs County, Ohio. Ohio Departament of Transportation (ODOT) choose to construct this culvert instead of a conventional bridge structure because it was 3,4$ million cheaper.

The authors of this article are Shad Sargan, Teruhisa Masada and Andrew Moreland. The field measurements were taken during construction of the culvert. Soil pressure and deflections of the pipe culvert were monitored using soil pressure cells and telescoping rod or magnetic tape respectively. Finally the real field values were compared with predicted values from a finite element program.

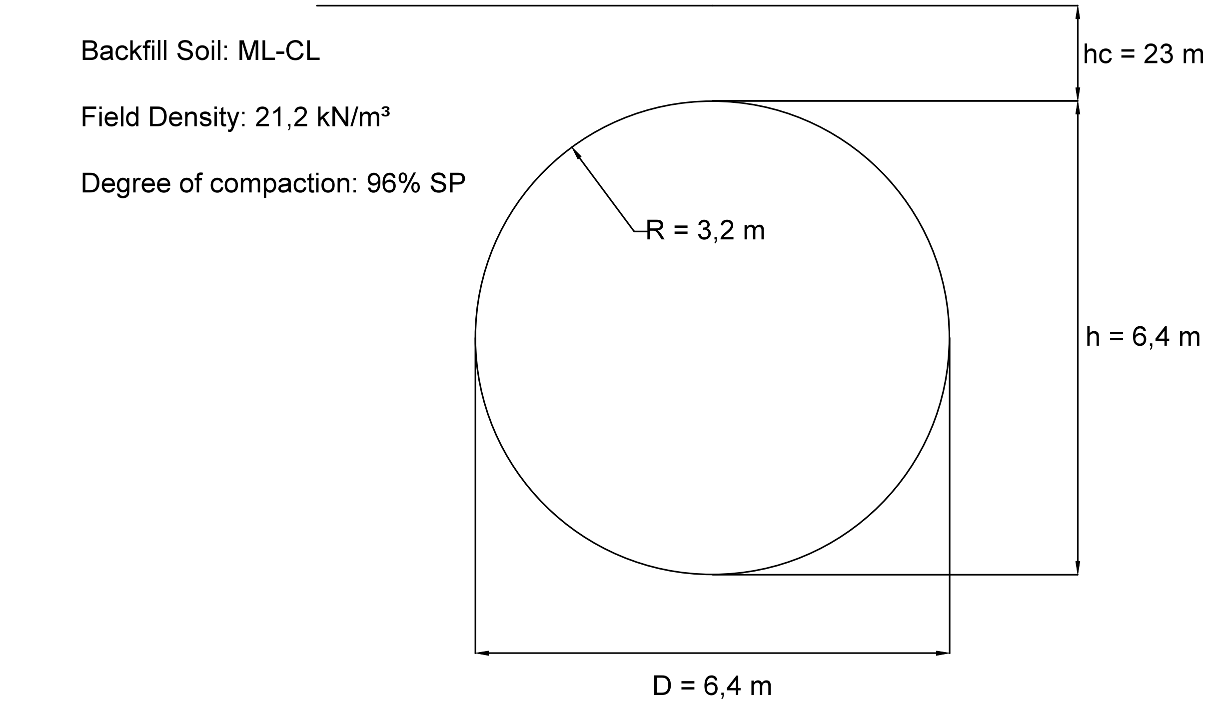

Figure 4. 22 Cross section of culvert 22

23. RESEARCH ADVANCING THE DESIGN OF LARGE SPAN DEEP CORRUGATED METAL CULVERTS, ONTARIO, CANADA, 2012.

For this report a 10 m span steel box culvert with 400x150 corrugations is erected to perform some loading tests and monitor the values of the deformations of the structure. That structure was located at Queen's University, Kingsdom, Ontario, Canada and the author of this report are Ian Moore and Richard Brachman.

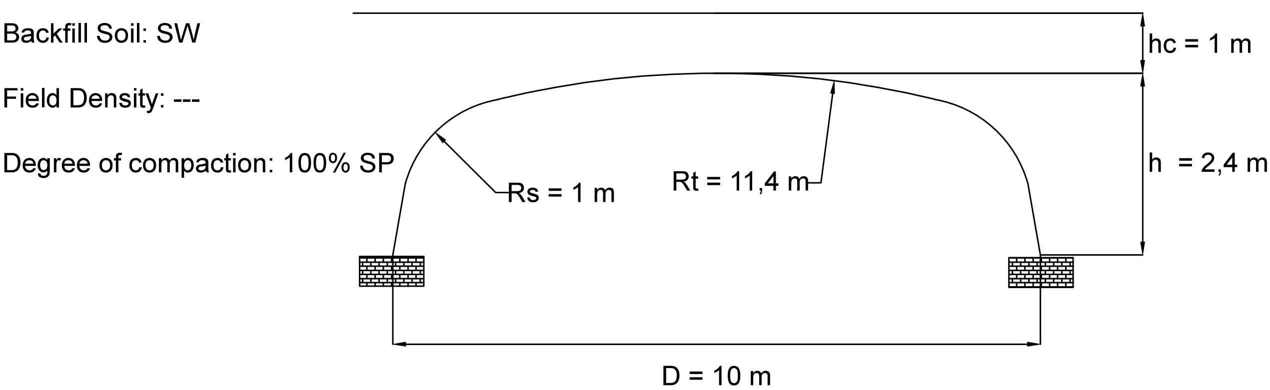

Figure 4. 23 Cross section of culvert 23

The test was performed in a laboratory and during the load tests they monitored the vertical and horizontal deformation and the strain steel of the culvert. They performed live load test using a truck and ultimate strength test using an actuator centred over the structure. They used three different high of cover during the tests. Finally they compared the real measurements taken from the tests with predicted values from a finite element program.

24. STATIC LOAD TEST OF A CORRUGATED STEEL PLATE ARCH WITH RELIEVING SLAB, STARY WALISZOW, POLAN, 2008

In this report a steel arch with 150×50 mm corrugations is tested under a live load test. The structure is located over the Plawna Stream on a local road between Bystrzyca Klodzca and Ladek Zdroj in Stary Waliszow, Poland. D. Beben and Z. Manko wrote this article. The culvert has a span of 10 m and a height of 4 m. During the live load test they measured deflections of the culvert, strains in the steel and soil settlement around the structure. For the live load test they used two different trucks.

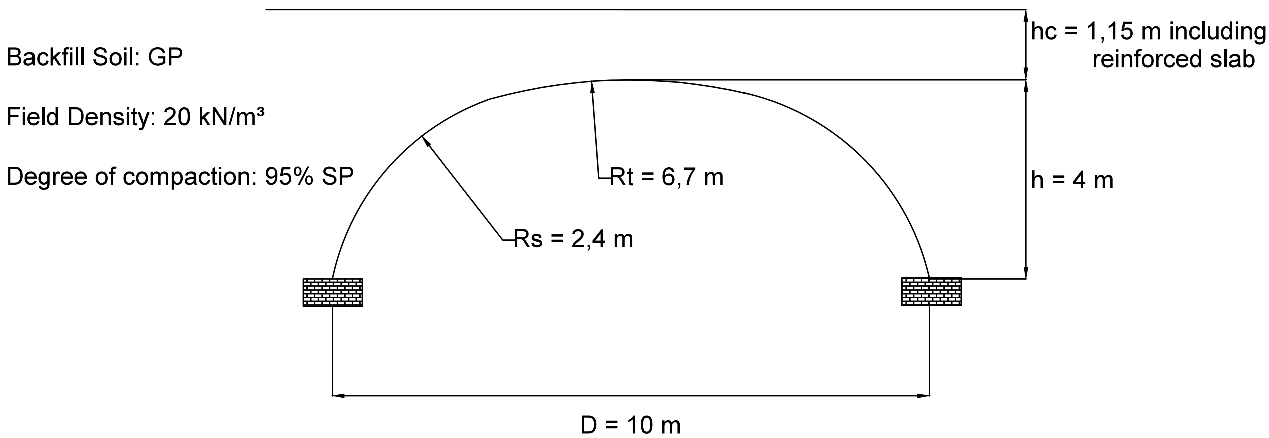

Figure 4. 24 Cross section of culvert 24

They performed three different tests. One consists of both trucks positioned at one curb on the upstream side, another with the two trucks positioned in such a way that their longitudinal axes coincided with the longitudinal axis of the bridge and their rears touched at half of the bridge's effective span and the last one consist on both trucks positioned on both sides of the bridge's longitudinal axis so that their rear axles were situated at half the bridge's effective span. The report ends with a comparison of the measured values and the predicted values of a computer program in 2D and 3D.

25. FIELD TESTS OF A LARGE-SPAN METAL CULVERT, MASSACHUSETTS, USA 1999

In this report a 9,5 m span low-profile arch culvert with 152x51 mm corrugations is tested under a live-load test. The authors of this article are Mark C. Webb, Ernest T. Selig, Jeanne A. Sussmann and Timothy J. Mc Grath. Two different tests were performed. One was conducted using 0,9; 0,6 and 0,3 m of height of cover and with 92% of degree of compaction of the soil. The other one was conducted using a 1,4 m height of cover and without compaction during construction achieving a degree of compaction of 87%. The University of Massachusetts in Amherst carried through the tests in 1997. During the test they measured deformations of the culvert, steel strains and culvert-soil interface pressure. Deformations and steel strains were measured at 26 and 25 points inside the culvert respectively. The truck used for the live load test had tandem axles with dual tires. The spacing between the tandem axles was 1,4 m. The center-to-center spacing between the wheels of the tandem axles was almost 2 m. The width of one set of dual wheels was about 0,58 m. The total weight of the truck was around 37.000 kg.

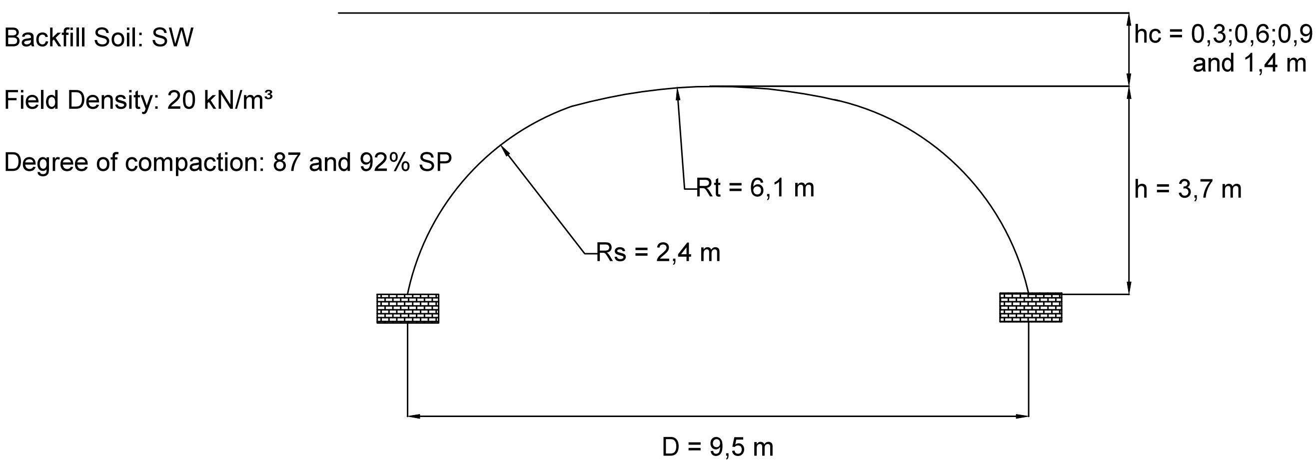

Figure 4. 25 Cross section of culvert 25

5 Calculations

5.1 Soil modulus calculated

In this chapter, the results of back calculated soil modulus are presented for the tests where information was available. There are two different types of results. In section 4.1.1 are presented the back-calculated soil modulus using the equations of SDM explained in section 2.4.2. The soil modulus presented in section 4.1.2 are calculated using the theoretical method (method B) from the SDM.

5.1.1 Back calculated soil modulus

The back calculated soil modulus from this section are calculated using the methodology explained in section 4.1.1 and can be seen that the soil modulus vary from lowest value 11,8 MPa to the highest 681,7 MPa depending on the case. Following points present all the cases in which live load tests were performed and the measured moments were reported and then it was possible to back calculate the soil modulus. There are some cases included in the table where the soil modulus have been calculated for the same culvert using different height of cover (h_c) or degree of compaction. All the $p_{traffic}$ line loads used for the back calculations are included in the Appendix B. The following tables summarize the results:

- *Culvert number 1*

It is necessary to say that in the case of culvert number 1 the live load test was performed using metal slabs placed over the culvert instead of using a vehicle like in the rest of cases. Also in case number 1 we can see that different heights of cover are tested and the load also vary. For a height of cover of 1m a load of 151.000 kg was used. In the other case (h_c=1,57 m) a load of 690.000 kg was used to perform the live load test.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
1	0,16	107	1,2	117
1,57	0,25	107	3,5	122,8

Table 5. 1: Back calculated soil modulus for Culvert 1

- *Culvert number 6*

This paper is the one that gives more controlled values of back-calculated soil modulus because all the information needed was reported and therefore no assumptions were done. As we can see in the test different height of cover are tested. The load test was a loading to failure test starting with a height of cover of 1,5 m and decreasing progressively until reach a h_c=0,36 m that is not reported because it has been considered that due to the failure of the structure the measurements reported were not valid for the calculations. The vehicle for the loading test consisted of a 4 tyres single axle loads up to 48.800 kg. Dimensions and characteristics of the trailer are defined in the paper.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,5	0,13	96	6,1	12,2
0,6	0,16	96	3,4	19,2
1	0,26	96	0,36	46,3
1,5	0,39	96	0,17	48,8

Table 5. 2: Back calculated soil modulus for culvert 6

- *Culvert number 7*

In this paper two box culverts were tested, one reinforced using deep-corrugated crown stiffener and the other one using concrete-filled composite metal-encased stiffener. For this thesis only the first one was considered because the one reinforced with concrete would have much lower stresses at the crown due to the high stiffness and then the back-calculated soil modulus would have been much bigger than the real one. As we can see in the table they performed two tests using the same height of cover but varying the degree of compaction. The vehicle used for the live load test simulated the first three axles of a CL 625-kN truck and the characteristics of it are explained in the paper.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,3	0,025	85	60	34,9
0,3	0,025	95	38	73,4

Table 5. 3: Back calculated soil modulus for culvert 7

- *Culvert 8*

As we can see in the table for this case the back calculated soil modulus is too high. One reason can be that a pavement layer was placed on the top of structure. Pavements has a tangent modulus higher than the soil and therefore it increase the load carrying capacity of the structure but in this case the increment is extremely high so probably the measures were not taken correctly. The vehicle for the loading test is the same as in *Culvert 6*.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,86	0,23	95	0,35	681,7

Table 5. 4: Back calculated soil modulus for culvert 8

- *Culvert 9*

Loading test on culvert 9 is quite interesting because it is the only one where they test the same height of cover but one using only compacted soil and the other one including a pavement layer. It is evident the effect of the pavement on the result of the soil modulus. In this case they test 3 different height of cover and in all of three the soil is compacted to the same degree of compaction. The vehicle used in the live load test is the same as in *Culvert 6*.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,7	0,11	100	21,9	11,8
1,2	0,2	100	8,8	28,9
1,2 with slab or pavement	0,2	100	4,3	71,5

Table 5. 5: Back calculated soil modulus for culvert 9

- *Culvert 10*

For this test they only used one height of cover compacted until 95% of degree of compaction. The vehicle used for the loading test was the same as in *Culvert 6* and it was loaded up to 455 kN. A pavement consisting of 210 mm of lean mix concrete overlaid by 160 mm of bituminous-bound material was constructed on top of the backfill.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
1,9	0,19	95	0,13	57,2

Table 5. 6: Back calculated soil modulus for culvert 10

- *Culvert 11*

This culvert was tested using the same vehicle as in *Culvert 6*. The culvert was constructed on a 600 mm thick reinforced concrete slab which provided a foundation for the base of the arch and also formed a paved invert for the structure. As can be seen in the table the backfill of the structure was compacted to a high density. A pavement consisting of 120 mm of bituminous-bound material was constructed on top of the backfill. This gave a total depth of cover to the crown of the culvert of 0,95 m.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,95	0,13	103	3,3	74,2

Table 5. 7: Back calculated soil modulus for culvert 11

- *Culvert 12*

For this case live-load moments were less than the instruments could measure accurately. The only information available was that the maximum flexural stress in the conduit wall was less than 2,0 MPa. To be able to back calculate the soil modulus it was decided to consider the maximum flexural conduit wall stress equal to 2 MPa.

This case shouldn't be considered as a reliable case due to the big assumption taken to calculate the M_t using the stress value of the conduit wall assumed as it was explained above. The live load test was performed using two vehicles which ones are described in the paper.

One possible explanation for this low strain values measured on the culvert could be that a relieving reinforced slab 0,3 m thick was placed over the top of the crown and then the load effects on the culvert were reduced a lot.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
1,45	0,17	95	0,234	84,1

Table 5. 8: Back calculated soil modulus for culvert 12

- ***Culvert 18***

The culvert was built in 1961 and it presented some problems due to disrepair in the structure. The steel was corrugated in some parts and near the top of the culvert, the connections between plates were detached and created gaps at several locations. The first point to highlight is the high value of the back-calculated soil modulus and also that the degree of compaction is not reported. The vehicle used for the loading test was a truck loaded up to a total weight of 310,3 kN. The paper report the name of the model of the truck used, the information about the axle loads and the total load of the vehicle and the information about the distances between axles and tyres.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,58	0,17	-	1,258	227

Table 5. 9: Back calculated soil modulus for culvert 18

- ***Culvert 23***

The structure of this test was built in the laboratory. They only test one h_c with the soil compacted until 100 % Standard Proctor. The vehicle used for the live load test is a tandem axle dump truck fully loaded (240 kN) which dimensions are included in the paper.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,85	0,1	100	12	92,2

Table 5. 10: Back calculated soil modulus for culvert 23

- *Culvert 24*

For this case the loading test was performed using two different models of trucks: Jelcz Steyr and Kamaz 5511. The trucks were positioned at different positions over the culvert. Three different load schemes were tested and the position that created the highest moments on the culverts was the one where the two trucks were positioned in the middle of the roadway (looking across the bridge) and in consequence this load configuration was chosen for the back calculations. This position consisted on the two trucks positioned in such a way that their longitudinal axes coincided with the longitudinal axis of the bridge and their rears touched at half of the bridge's effective span. Only one height of cover was tested with the soil compacted until 95 % Standard Proctor. A reinforced concrete slab 0,2 m thick was laid over 0,1 m over the corrugated plates in the bridge's crown to relieve the steel structure.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
1,15	0,115	95	0,133	22,3

Table 5. 11: Back calculated soil modulus for culvert 24

- *Culvert 25*

For this case the test was performed using four different height of cover compacted to 95 % Standard Proctor. The vehicle used for the live-load test had tandem axles with dual tyres. The spacing between the tandem axles was 1,4 m. The centre-to-centre spacing between the wheels of the tandem axles was 1,96 m. The width of one set of dual wheels was about 0,58 m.

h_c (m)	h_c/D	DoC (%) SP	M_t (kN·m/m)	E_{sd} (MPa)
0,3	0,031	95	5,3	30,1
0,6	0,062	95	3,3	31,8
0,9	0,093	95	1,5	37,7

Table 5. 12: Back calculated soil modulus for culvert 25

5.1.2 Theoretical soil modulus

In this section, the soil modulus was calculated using the method B explained in the SDM. Method B is a theoretical method and was created to calculate conservative values of the soil secant modulus. Equations used in Method B are explained in details in section 2.3.

The required data is the following:

- Particle size distribution (d_{10}, d_{50} and d_{60})
- Degree of compaction (dry density and maximum dry density)
- $RP = 100 \cdot \left(\rho / \rho_{opt}\right)$ and
- Stress level in the surrounding fill calculated using the passive earth pressure at a depth equal to the cover depth plus *H/2*.

Within the 25 tests only 6 included all the above information needed to calculate the soil modulus. The main problem was the particle size distribution, normally they don't report this information which is very important for this design method. For SSCB the type of soil used affects the behaviour of the structure increasing or decreasing the load capacity of it.

The following table represents the results of the calculations of the soil modulus using the Method B obtained for the those cases:

Culvert number	h_c (m)	Doc (%) SP	Type of soil	E_{sr} (MPa)
6	0,5	96	GW-GP	35,9
6	0,6	96	GW-GP	36,2
6	1	96	GW-GP	37,4
6	1,5	96	GW-GP	38,6
8	0,86	95	SW	16,1
9	0,7	100	GM	4,7
9	1,2	100	GM	4,9
10	1,9	95	SW	16,5
11	0,95	103	GW	45,2
23	1	100	SW	64,7

Table 5. 13: Calculated soil modulus using method B for tests where sieve analysis was reported

In this table are presented some cases and we can notice for example the variation of the calculated soil modulus depending on the height of cover over the crown. The values back calculated using SDM range from 10,8 to 674,7 MPa while the results using the theoretical method range from 4,7 to 64,7 MPa.

For the rest of the cases the information about the soil was not given but anyway there were at least some cases that they report the type of soil they used without including the corresponding sieve analysis. For those cases the theoretical soil modulus was calculated making assumptions about sieve analysis, for example using sieves analysis from other reports where the same type of soil was used according to the "Unified Soil Classification System". Following table summarize the soil modulus calculated for the cases where the sieve analysis were not included in the report.

Culvert number	h_c (m)	Doc (%) SP	Type of soil	E_{sr} (MPa)
1	1	107	GP	104,6
1	1,57	107	GP	111,4
3	13,4	100	GW	23,1
4	0,76	96	SM	83,8
5	7	99	GW	12,6
7	0,3	85	GW-GP	9,4
7	0,3	95	GW-GP	40,6
12	1,7	95	Granular B soil	16
15	3,35	99	GW	13,2
16	1,86	100	GW-GP	61,6
24	1,15	95	GP	61
25	0,3	96	SW	11,5
25	0,6	96	SW	12,1
25	0,9	96	SW	12,6

Table 5. 14: Calculated soil modulus using method B for the test where no sieve analysis was reported

Obviously the results obtained are not the most reliable because, in fact, the soil even it is classified in the same group the particle size distribution may vary a lot. Anyway it was found important to include them just to have an idea about the values ended up with using the method B. In the following chapter are discussed all the results regarding soil modulus presented in this section.

6 Discussions

6.1 Comparison between back calculated soil modulus and soil modulus using the soil suggested in the design manual

The design manual proposes three different types of soil as backfilling of the culvert to design the structure. The Swedish standard specifications for road construction, *ATB Väg 2005,* consider two types of soil to be used when backfilling against bridges, crushed rock and base course material, and of course those soils are suitable to be used as backfilling against flexible culverts. The author of the SDM proposed to also include the sub base material inside this group of soils because this soil had already been used in different tests like Skivarspån railway bridge designed by the author (Flener, 2003, 2004 and 2005). The characteristics of the three soils are described in the following table extracted from the design manual:

Soil type	Optimum w. density (kN/m^3)	Weight density (kN/m^3)	Angle of friction (^{o})	At-rest lateral soil coeff, K_0	$C_u(d_{60}/d_{10})$	d_{50} (mm)
Crushed rock	19,6	19	45	0,29	15	70
Base course material	20,6	20	40	0,36	10	20
Sub base material	21,7	21	43	0,32	15	10

Table 6. 1: Soil parameters for soil proposed in the design manual

Next step was to calculate the different soil modulus using the soils explained before for the test where the back calculated soil modulus were available to calculate and then compare the results. Next table summarize the results including the soil modulus back calculated and soil modulus calculated using method B using the types of soil described above:

Culvert number	E_s back calculated (MPa)	E_s using crushed rock (MPa)	E_s using base course material (MPa)	E_s using sub base material (MPa)
1	117	23,2	41,1	50,6
1	122,8	23	42	52,5
6	12,2	23,8	38,4	44,3
6	19,2	23,7	38,6	44,8
6	46,3	23,6	39,5	46,8
6	48,8	23,4	40,3	48,7
7	34,9	23,7	38,9	45,5
7	73,4	23,6	39,4	46,6
8	681,7	23,5	39,7	47,2
9	11,8	23,3	40,7	49,5
9	28,9	23,1	41,4	51,3
9	71,4	23,1	41,4	51,3
10	57,2	23	42,1	52,9
11	74,2	23,3	40,8	49,8
12	84,1	23,2	41,2	50,8
18*	227	23,9	38,1	43,6
23	92,2	23,4	40,2	48,5
24	22,3	23,1	41,6	51,7
25	30,1	23,4	40	48
25	31,8	23,3	40,6	49,2
25	37,7	23,2	41,1	50,4

Table 6. 2: Summary table of soil modulus back calculated and calculated using soil parameters explained in Table 5.1

The case of culvert 8 hasn't been included in the following comparison due to the extremely high value of the back calculated soil modulus which is much bigger compared with the results obtained for the rest of the cases. It could be possible as it was explained in section 4.1.1 that the pavement layer used on the top of the culvert had reduced the load transmission to the culvert and then the moment at the crown created by the load was reduced which caused that soil modulus ended with was much higher than the rest of the results presented in the table above.

For the case of culvert 18* the degree of compaction was not reported. Consequently it was assumed a degree of compaction of 95 % standard Proctor for the calculations using the three different types of soil. As the results obtained were not reliable due to the lack of information they are not included either in the following comparison.

6.1.1 Soil modulus back calculated compared to soil modulus calculated using crushed rock

Soil modulus calculated using crushed rock gives normally lower values than the ones back calculated. Only in 4 of the 19 cases presented above method B using crushed rock achieves higher values.

Highest difference between both soil modulus is found in *Culvert 1* with a h_c = 1,57 m, the soil modulus back calculated was almost five times bigger than the one calculated. On the other hand the closest value was found for the case of *Culvert 24* where the soil modulus calculated were almost the same. Soil modulus calculated using crushed rock don't vary a lot. The lowest soil modulus calculated is 23 MPa for *Culvert 10* and the highest is 23,8 MPa for *Culvert 6* with a h_c=0,5 m. The range of values obtained using crushed rock is around 23-24 MPa.

Average difference between the two soil modulus is 33 MPa. Using crushed rock lower soil modulus are obtained therefore is a conservative method to calculate the soil modulus for this cases in particular.

6.1.2 Soil modulus back calculated compared to soil modulus calculated using base course material

Using base course material higher soil modulus were calculated compared with the ones using crushed rock. This is normal because the characteristics of the base course material are better than the crushed rock and then it causes that the soil capacity is better regarding the soil modulus.

Comparing back calculated and calculated soil modulus values for each case we can see that there are more cases were back calculated soil modulus are lower than the ones calculated using base course material. In particular there are 9 cases of 19 (almost 50% of the cases) where base course material achieved higher soil modulus than back calculated ones.

Highest difference between the soil modulus compared is found in *Culvert 1* with a h_c=1,57 m like in the comparison using crushed rock but in this case using base course the back calculated soil modulus was around three times bigger than the one calculated and not five like in the previous comparison. On the other hand the closest soil modulus were found in *Culvert 7* with a *Doc* of 85%.

The lowest and highest calculated soil modulus were 38,4 MPa and 42,1 MPa respectively and it can be seen that the range of values using base course material are higher compared with the range of values using base course material. The average difference between soil modulus is 27 MPa.

Soil modulus back calculated compared to soil modulus calculated using sub base course material

The results using base course material showed that 10 of 19 cases the soil modulus calculated was higher than the one back calculated. Most similar soil modulus was found in *Culvert 6* with a h_c=1,5 m and the most different soil modulus was calculated in *Culvert 1* with a h_c=1,57 m.

Lowest value of soil modulus calculated using base course material was 44,3 MPa and was found in *Culvert 6* with a h_c=0,5 m and the highest value was 52,9 MPa and was calculated in *Culvert 10*. The average difference between soil modulus was 26,2 MPa, a little bit less than using base course material.

6.2 Comparison between back calculated soil modulus and calculated soil modulus using method B

Next table present the values back calculated and calculated using method B for the culverts where sieve analysis was reported.

Culvert number	Height of cover (m)	E_s back calculated (MPa)	E_s calculated from sieve analysis (MPa)	Highest soil modulus
6	0,5	12,2	35,9	Calculated
6	0,6	19,2	36,2	Calculated
6	1	46,3	37,4	Back calculated
6	1,5	48,8	38,6	Back calculated
8	0,86	681,7	16,1	Back calculated
9	0,7	11,8	4,7	Back calculated
9	1,2	28,9	4,9	Back calculated
10	1,9	57,2	16,5	Back calculated

Culvert number	Height of cover (m)	E_s back calculated (MPa)	E_s calculated from sieve analysis (MPa)	Highest soil modulus
11	0,95	74,2	45,2	Back calculated
23	1	92,2	64,7	Back calculated

Table 6. 3: Summary of soil modulus calculated from sieve analysis and soil modulus calculated using the soils defined in the design manual

The rest of cases are not included due to the fact that the soil parameters were not reported and in consequence the results calculated using method B were not reliable or simply it was not possible to calculate them.

As it was said in this chapter soil modulus back calculated from field measurements (moments) and soil modulus calculated using method B (theoretical) are compared. Method B is an empirical method used for the design of SSCB. The designers to calculate the soil modulus before constructing the structure use it and normally, as it is a design method, the soil modulus calculated using this method should be lower than the one back calculated from field measurements.

For this comparison no safety coefficients have been used to calculate the results using method B but normally the designers would use the following coefficients 1,3 and 1,1 to reduce the calculated soil modulus.

The results are logical for all the cases except for *culvert 6* and *8*. In the first case, as we can see in the table above, four different height of cover were tested. When the test was performed using 1,5 and 1 m of height of cover the results obtained were good, it means that the soil modulus back calculated were higher than the soil modulus calculated using method B.

On the other side, results obtained in the test performed using 0,6 and 0,5 m of height of cover were just the opposite, soil modulus using method B were higher than the back calculated ones. It means that the design method overestimates the real soil modulus in the field.

But there is a reasonable explanation about why did it happen. In *culvert* 6 a loading to failure test was performed. Loading to failure tests create extreme conditions in the soil and sometimes, when the shear capacity is exceed, the soil loses the loading capacity which means that the soil somehow got disturbed that causes a decreasing of the resistance of the soil, that's the soil modulus. The soil above the culverts can fail either by wedge sliding or through tension. This is an explanation why the soil modulus back calculated are much lower than the ones using method B. Another cause to take into account is the shallow height of cover used. The SDM limit the lowest height of cover to 0,5 m not less and it means that in *Culvert 6* the two lowest height of cover tested (0,5 and 0,6 m) are very close to this limit and it could also explain the low values of soil modulus back calculated in those cases.

For the case regarding *culvert 8* we can say that the result is logical from a design point of view because the soil modulus back calculated is higher than the soil modulus calculated using method B. However, from an engineering point of view, it can be seen that the value of back calculated soil modulus. Back calculations gave a value of soil modulus of 681,7 MPa.

This value of the soil modulus is the highest of the ones back calculated with difference but it can be explained looking at the details of the culvert cross section.

In the report of *culvert 8* it is stated that the height of cover is 0,86 m but not whole cover height is made by soil. The structure includes a 0,21 m lean-mix concrete and a 0,16 m road surface within the 0,86 m of height of cover. Those layers helped to carry and distributing almost all the load and this fact decreased the load supported by the culvert and the soil. It caused that the moments measured on the loading test were very small (M_t=0,35 kN·m/m) and then the soil modulus back calculated using equation explained in section 2.4.2 was too high. It can be said that this was not the real soil modulus of the soil in fact, it was overestimated due the two different layers named before that helped to carry the load.

6.3 Variation of soil modulus with respect to the parameter h_c/D

The height of cover is an important factor to consider when designing SSCB. The fact of using higher or lower height of cover will affect the behaviour of the whole structure. Normally higher heights of cover will cause a positive arching effect and on the other hand in low height of cover situations, the soil arching would be negative. It means that in low height of cover situations the steel structure will support higher stresses and the designers must take it into account.

Within the twenty-five tests included in this thesis, it was found that in some of them different heights of cover were tested for the same culvert and it was found important to report the variation of the soil modulus back calculated depending on the height of cover.

Soil modulus back calculated against the value of h_c/D (height of cover/span) are presented in the following figure:

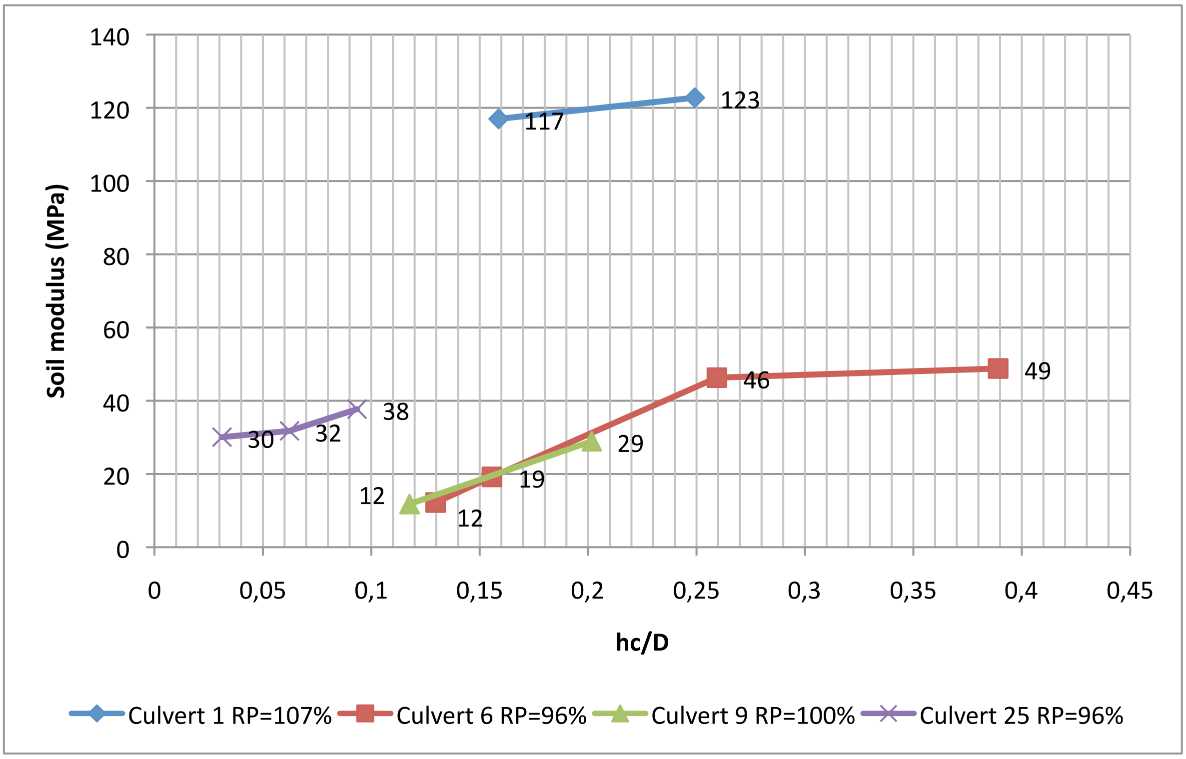

Figure 6.1: Variation of back calculated soil modulus with respect to h_c/D

In the figure 5.1 it can be seen how increasing the height of cover, the soil modulus back calculated also increase. Those results are logical because the fact of increasing the height of cover means that there will be more soil over the culvert to distribute the load and then the vertical pressure created by this load on the steel culvert will get reduced.

Only four cases have been studied in this section, obviously not enough to get any general conclusion about the effect of increasing the height of cover on SSCB. However, it's interesting that for all of those cases the back calculated soil modulus increased when increasing the height of cover.

6.4 Effect of slabs or pavement over the top of the culvert

Sometimes in the construction of the SSCB, reinforced concrete slabs are placed over the top of the culvert just under the riding surface. Those slabs are used to increase the load capacity of the structure or simply to reduce the sectional forces of the steel culvert.

Within the twenty-five tests there is one case, *culvert* 9, where the effect of the slab in the calculation of the soil modulus can be checked. For this test two different height of cover were tested, 0,7 and 1,2 m and the last one was tested with and without slab and in the next figure we can see the difference in the soil modulus:

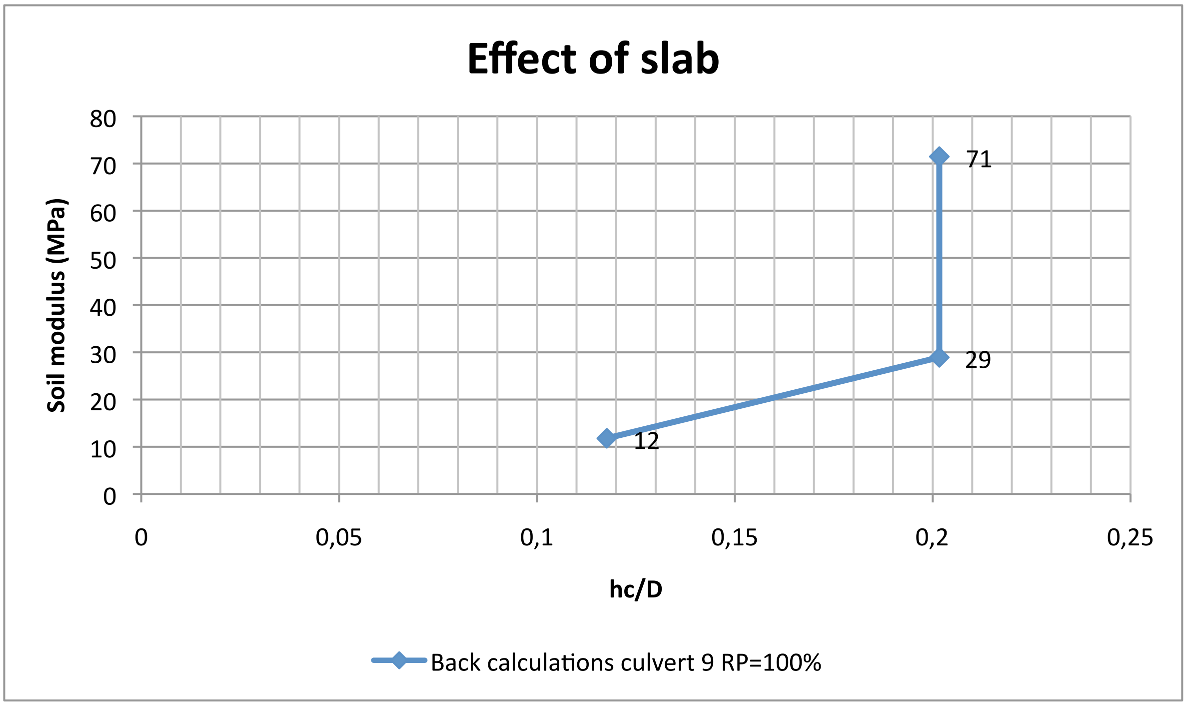

Figure 6. 2: Variation of back calculated soil modulus with respect to h_c/D

It can be seen in figure 5.2 that using a height of cover of 1,2 m without slab, the soil modulus back calculated is 29 MPa but for the same height of cover using a 0,3 m thick reinforced concrete relieving slab, the soil modulus back calculated is 71 MPa. It is obvious the effect of the slab in this case.

6.5 Effect of the parameter Cu in respect to the calculations using method B

The coefficient of uniformity (C_u) is defined by the relationship between the parameters d_{60} and d_{10} using the following equation:

$$C_u = \frac{d_{60}}{d_{10}} \tag{6.1}$$

This parameter is used in the calculations of soil modulus using equations of method B from SDM. Those equations are presented in section 2.4.3.

Following table summarize values of soil modulus calculated using method A and method B with the corresponding value of C_u calculated for each case. In this table are only compared the full scale tests where the information of the soil was reported and in consequence reliable results could be obtained.

Culvert number	h_c (m)	C_u	E_{sd} method A (MPa)	E_{sd} method B (MPa)
6	0,5	13,3	8,78	35,9
6	0,6	13,3	8,88	36,2
6	1	13,3	9,2	37,4
6	1,5	13,3	9,5	38,6
8	0,86	15	7,9	16,1
9	0,7	233,3	17,8	4,7
9	1,2	233,3	18,3	4,9
10	1,9	18,9	8,6	16,5
11	0,95	28	28,5	45,2
23	1	22,5	27,7	64,7

Table 6.4: Comparison of soil modulus calculated using method A and B

As it was explained in section 2.4.3 using method A, the designers will get lower values of the soil modulus than the ones obtained using method B. Looking at the table above we can see that this rule fits perfectly except for one case, culvert number 9, where the soil modulus calculated using method A is four times bigger than the soil modulus calculated using method B.

This fact was not normal using the SDM so it was necessary to find an explanation to this. Looking into the details of the calculation of the different culverts, there was one parameter that varied a lot in comparison with the same value for the rest of culverts. This parameter was the coefficient of uniformity (C_u).

In the case of culvert 9, the value of C_u was 233,3 while the values for the rest of culverts had a range between 13 and 28. The coefficient of uniformity of culvert 9 was almost 10 times bigger than the second highest value of C_u.

Recalculating the soil modulus using lower values of C_u gave more reasonable results as we can see in the following table:

h_c (m)	C_u	E_{sd} method A (MPa)	E_{sd} method B (MPa)
0,7	10	17,8	60,3
1,2	10	18,3	61,1
0,7	20	17,8	35,2

h_c (m)	C_u	E_{sd} method A (MPa)	E_{sd} method B (MPa)
1,2	20	18,3	35,8
0,7	30	17,8	25,6
1,2	30	18,3	26,1

Table 6. 5: Soil modulus calculated using lower values of C_u

Results presented in table 5.5 show how the variation of the coefficient of uniformity affects the value of the soil modulus calculated. Those results are more reasonable because soil modulus obtained using method B are higher than the ones using method A as it is expected using SDM.

As a conclusion it can be said that the SDM can not be used for all the soils. Soils with very high value of C_u will cause that the designers will end up with really low values of soil modulus using method B and in consequence a limitation for the value of C_u should be stated.

7 Conclusions

7.1 General

Soil Steel Composite Bridges (SSCB) are considered to be a competitive alternative to design short span bridges under rail or road traffic. Those types of structures are being used more and more often during the last years. SSCB are cheaper and the construction is faster compared with other bridges.

Nowadays SSCB with higher corrugation profiles and higher spans start to be used as an alternative to conventional bridges.

Another important point is the information about the SSCB included in the reports. In those types of structures, the soil used surrounding the culvert is very important and all the information related to it is necessary to know for a good design. As it was explained before only six of the twenty-five tests, the sieve analysis of the soil was reported and in consequence only for those culverts the soil modulus back calculated and calculated using method B could be compared.

Future reports about other full scale tests on SSCB have to include:

- All the information about the soil: sieve analysis, degree of compaction, dry density and optimum dry density.

- The information about the vehicle used for the life load test. That is the vehicle measures, the tyre pressure and the load of each axle.

- All the cross section measures of the culvert. That is the inside radius, the height of cover, the span and the rise.

7.2 Database

The database presented in this thesis is a starting point to create a big and complete database of SSCB. As it was explained before, SSCB are being used more often as an alternative to conventional bridges and it would be a great advantage to collect all the information from lot of different SSCB and put all of this information together in a database. Future researchers will use it to find and take information in a faster and easier way from the tests they would be interested in. Of course for deeper research about one specific test the whole report should be read.

7.3 Soil modulus

Back calculated soil modulus are shown in section 4.1.1 for the culverts where a live load test was performed on them and also for the ones the values of moments on the culvert where reported during the live load test.

From the twenty-five tests, in nineteen of them a live load test was performed. From these nineteen tests it was possible to back calculate the soil moduli in twelve of them due to the fact that the values of the moment during the live load test were not reported.

Using method B, only six test reports had all the necessary information to calculate the soil modulus. For some of the remaining tests, the soil modulus were also calculated using this method but it was necessary to make assumptions about the soil and therefore the results are not reliable to compare with the ones back calculated.

As it was shown in section 5.2 six different tests have been able to compare. In five of the six tests the back calculated soil modulus were higher than the one calculated using method B. However in culvert 6 the results varied. For the two highest height of cover the results were similar to the other culverts, back calculated soil modulus were higher. But for the two lowest height of cover the results changed. The back calculated soil modulus were lower than the theoretical one (method B). It was explained that it happened because the soil over the structure started to fail due to the heavy pressures caused by the loading to failure test performed on this case.

The soil modulus back calculated increased when increasing the height of cover as shown in section 5.3.

Even if it is not included in the report, it is important to say that a comparison between method A and method B was done for the test where sieve analysis was reported. Normally using method A designers will end up with much lower values of soil modulus than using method B. Method A is very conservative. But it was found that in c*ulvert* 9 it didn't happen, the soil modulus calculated using method A was higher.

Checking the soil parameters it was found that the value of C_u in that case was very high, much more than the rest of the cases. Recalculating the soil modulus using a lower C_u, similar to the ones of the rest of the test it could be seen that the result changed, now the soil modulus from method B was higher. So it can be concluded that the method B might not be possible to be used for all the soils. Soils with high values of C_u will get really low values of soil modulus.

7.4 Future research

The proposals for future researchers are the next ones:

- Increase the number of tests included in the database. It would be a great help for the study of SSCB to continue adding tests to the database to finally create a comprehensive one where almost all of the SSCBs around the world and the characteristics of them would be included.

- Study more full scale culvert where live load tests have been performed.

- Study other fields regarding SSCB like the arching effect or the soil pressures around the culvert

- Deeper research about the effect of using reinforced slabs over the top of the culvert

- Deeper study about the soils with high values of C_u using method B of the SDM.

Bibliography

Abdel-Sayed, G., Bakht, B. and Jaeger, L. G. (1994) *Soil-steel bridges: design and construction*, New York: McGraw-Hill.

Allgood, J.,R., and Takahashi, S., K., "Balanced Design and Finite Element Analysis of Culverts, Highway Research Record 413", Transportation Research Board, Washington D.C., 1972.

Arockiasamy, M., Chaallal, O. and Limpeteeprakarn, T. (2006) '*Full-scale field tests on flexible pipes under live load application*'.

Bakht, B. (1983) '*Liv-load response of a soil-steel structure with a relieving slab*'.

Beal, D.B. '*Behaviour of aluminium structural plates culvert*'.

Beben, D. and Manko, Z. (2008) '*Static load tests of a corrugated steel plate arch with relieving slab*'.

Demmin, J. (1970) '*Field verification of ring compression conduit design (Klöppel and Glock test)*'.

Duncan, J.M. and Jeyapalan, J.K. '*Deflection of flexible culverts due to backfilling compaction*'.

Flener, E.B., *Field Testing of a Long-span Arch Steel Culvert Railway Bridge over Skivarspån,* Sweden. Part I, Trita-BKN, Rapport 72, KTH, Stockholm, 2003.

Flener, E.B., *Field Testing of a Long-span Arch Steel Culvert Railway Bridge over Skivarspån,* Sweden. Part II, Trita-BKN, Rapport 84, KTH, Stockholm, 2003.

Flener, E.B., *Field Testing of a Long-span Arch Steel Culvert Railway Bridge over Skivarspån,* Sweden. Part III, Trita-BKN, Rapport 84, KTH, Stockholm, 2003.

Johnson, P.E., Temporal, J. and Watts, G.R.A (1989) '*The effect of pavement layers on the behaviour of corrugated steel culverts*'.

Kay, J. N., Avalle, D.L, Flint R.C.L. and Fitzhardinge, C.F.R., (1980) '*Instrumentation of a corrugated steel soil arch overpass at Leigh creek, south Australia*'.

Lefebvre, G., Laliberté, M., Lefebvre, L.M, Lafleur, J. and Fisher, C.L., (1975) '*Measurement of soil arching above a large diameter flexible culvert*'.

Machelski, C. and Antoniszyn, G. (2004) '*Influence of live loads on the soil-steel bridges*'.

McCavour, T.C., Byrne, P.M. and Morrison, T.D. (1998) '*Long span reinforced steel box culvert*'.

McVay, M.C. and Selig, E.T. '*Performance and analysis of a long-span culvert*'.

Moore, I. and Brachman, R. (2012) '*Research advancing the design of large span deep corrugated metal culverts*'.

Pettersson, L., (2007) *Full scale tests and structural evaluation of soil steel flexible culvertswith low height of cover*. Doctoral thesis in Civil and Architectural Engineering, Division of Structural Design and Bridges, Royal Institute of Technology, KTH, Stockholm, Sweden.

Pettersson, L., and Sunquist, H., (2010) *Design of soil-steel composite bridges.* TRITA-BKNRep. No. 112, Dept. for Architectural and Civil Engineering, Royal Institute of Technology, KTH, Stockholm, Sweden.

Raymond, B., Seed and Chang-Yu Ou (1986) '*Measurements and analysis of compaction effects on a long span culvert*'.

Sargan, S., Masada, T. and Moreland, A. (2008) '*Measured field performance and computer analysis of large-diameter multiplate steel pipe culvert installed in Ohio*'.

Selig, E.T. and Calabrese, S.J. (1975) '*Performance of a large corrugated steel culvert*'.

Selig, E.T., Lockhert, C.W. and Lautensleger, R.W. (1979) '*Measured performance of Newtown creek culvert*'.

Sezen, H., Yeau, K.Y. and Fox, P.J. (2008) '*In-situ load testing of corrugated steel pipe-arch culverts'*.

Temporal, J. and Johnson, P.E. (1988) '*Loading tests on two long span corrugated steel culverts*'.

Temporal, J., Barratt, D.A. and Hunnibell B.E.F. (1985) '*Loading tests on an Armco pipe arch culvert (Newport)*'.

ViaCon company, ViaCon catalogues, URL: http://viacon.pl/en/download.html

Wadi, Amer H. H., (2012) *A comparison between the Pettersson-Sundquist design methodand the Klöppel & Glock design method including finite element modelling.* Master thesis in Civil and Architectural Engineering, Division of Structural Design and Bridges, Royal Institute of Technology, KTH, Stockholm, Sweden.

Webb, M.C., Selig, E.T., Sussmann, J.A. and McGrath, T.J. (1999) '*Field tests of a large-span metal culvert*'.

Williams, K., Mackinnon, S. and Newhook, J. (2012) '*New and innovative developments for design and installation of deep corrugated buried flexible steel structures*'.