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Stiffness variations on railway tracks over culvert underpassings

- *A comparison between rigid concrete culverts with flexible steel-soil composite culverts*

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Abstract

For many years have railway transition zones between embankment and bridge, both in Norway and internationally been described as a problem area where extra maintenance required. It is often thought that these problems relate to a combination of differential settlements and the sudden change in track stiffness, a deterioration loop which leads to an increase in track interaction forces and eventually creates differential settlements as a result.

Comprehensive FEM with PLAXIS 2D was used in this thesis to model four railway transition zones in an attempt to quantify the stiffness variations seen over flexible and rigid culverts. The model involves train loads passing over a railway culvert where displacements are studied.

From using the principles of plane-strain, it was possible to model the entire problem from skratch by building a culvert, backfilling, building a track and modelling moving train loads. The procedure purposed in this thesis was then used to study four specific scenarios of railway transition zones, but also for investigating the behaviour of these culverts. The results from these models suggests that the magnitude of the problem is insignificant as long as certain conditions are met.

Data from the norwegian measuring car «ROGER 1000» has also been used for assessing six selected railway culverts were it was investigated if there was any correlation between the apparent stiffness variations at these sites and vertical track deviations measured by the veichle. For the six culvert cases, no clear trend in vertical track deviation over them was found when comparing to the adjecent free track.

The general conclusion is that stiffness variations over these culverts are to small to create any obvious deterioration problem seen in vertical track geometry when comparing to a normal track.

Keywords:

1. Railway transition zones and culverts
2. FEM using PLAXIS 2D
3. ROGER 1000 measurements
4. Stiffness variations and maintenance

Preferance

This Master Thesis has been written as a mandatory closing part of the Master of Science programme, Geotechnics & Geohazards. The thesis make up for 30 out of total 120 credits in the programme. The work behind this thesis was carried out during the spring semester of 2019 at the Norwegian University of Science and Technology (NTNU), Trondheim.

The topic for the project was a continuation from the autumn's specialization project regarding railway transition zones in Norway, purposed by Geir Svanø and Juan Barrera from Bane NOR.

Trondheim, 14.06.19

Dan Sergei Sukuvara

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During this project I've been consulting with several people, but my biggest thanks goes to my main supervisor Prof. Steinar Nordal and Dr. Albert Lau from the road and transport department.

Their constant support during this project has been exceptional, Steinar for always making room for discussions and sharing his broad, yet outstanding knowledge. Albert for helping me whenever I was stuck on topics regarding railways. I would've never be able to keep such standard on so many broad topics without their constant support through the entire project.

I also want to show my gratitude to Prof. Gustav Grimstad for interesting discussions and learning experiences regarding FEM in PLAXIS. His supervision was very helpful whenever I was stuck on parts with the model.

A thanks goes to the seniors from Bane NOR, Geir Svanø and Alf-Helge Løhren for providing me with necessary information and sharing their experiences on topics related to railways. I also want to thank Trine-Lise Lorentsen for extracting and organizing ROGER 1000 data for me.

Lastly, I want to thank Peder Hembre from ViaCon Norway and Jan Vaslestad from Statens Vegvesen for supporting me with litterature and discussions related to flexible steel-soil composite bridges.

Peder provided me with alot of background information on several culverts used as a reference in this thesis, and Jan provided me with litterature and participated in several discussions where he shared his experiences.

Summary

Railway transition zones, i.e the transition between embankment and bridge has over many years been reported as a problem area that requires additional maintenance. These problems are often thought to be related to differential settlements but also the sudden change in track stiffness, a deterioration loop which gradually worsens from train traffic as time passes by, which deteriorates the tracks geometry and causes higher rate of wear in all elements of this area.

In this project the bridge transition problem was investigated and compared between rigid and flexible steel-soil composite culverts. In Norway, all new bridges expect for flexible ones are instrumented with transition slabs, a short concrete plate which extends out from the bridge a couple of meters into the fill. This is a fixed requirement independent of axle loads, bridge type or train speed.

When comparing the transition slab to international solutions, questions arise from its relative short lenght compared 20-30 meter long transition zones seen at other high speed lines. In addition to this questionnaire comes the confusion from flexible steel-soil composite bridges which has no requirement set for the transition zones.

From a comprehensive FEM with PLAXIS 2D was it possible model the entire problem in plane-strain from scratch in an attempt to quantify the problem over these culverts. The procedure involves building a culvert, constructing a railway track and modelling moving train loads which was used in the analysis of four specific cases created for this project. The general conclusion is that the irregularities seen at the track over these culverts from the model is relative small compared to a normal track.

The pressumed correlation between stiffness variations and increased deterioration rate at transition zones was also investigated with ROGER 1000 data, a measuring car which rutinely registers vertical track deviations among other things for the entire railway network in Norway. In general, no trend were found for the six preselected culverts between the presumed stiffness variations over them and vertical track geometry deviations measured by ROGER 1000, when comparing them to the measurements at the adjecent free track.

The conclusion from this project is that the stiffness variations seen at these kind of culverts are relative small as long as certain conditions are met, and there is no direct correlation between stiffness varations, differential settlements and increased deterioration in the transition zones of these culvert underpassings.

Sammendrag

Overgangen mellom bru og fylling («overgangssone») har over mange år blitt beskrevet som et problemområdet der det er et større behov for vedlikehold enn i et vanlig spor. Problemet er ofte tenkt å være relatert til differansesetninger men også stor variasjon i stivhet sett fra jernbanesporet, en ond sirkel som gradvis forværres av togtrafikk ettersom tiden går som vil forårsake geometriske avvik og økende slitasje på sporkomponentene i dette området.

I dette prosjektet ble problemet med overgang bru-fylling undersøkt for stive og fleksible samvirke kulverter. I Norge stilles det et minimumskrav til at alle nye bruer med unntak for fleksible stålrør skal instrumenteres med over-gangsplate, en kort betongplate som strekker seg ut fra bruen et par meter inn mot fylling. Dette er et fastsatt krav uavhengig av aksellast, brutype eller toghastighet.

Når overgangsplata sammenlignes med internasjonale løsninger stilles det spørsmål til dens relative korte lengde til sammenligning med 20-30 meter lange overgangssoner sett på andre høyhastighets-baner. I tillegg til dette spørsmålet skapes det desto større forvirring til at det ikke er stilt noen krav til overgangssonene for fleksible samvirkekonstruksjoner.

Fra en omfattende FEM analyse med PLAXIS 2D var det mulig å modellere hele problemet overgangs-soner i plan-tøyning fra scratch i et forsøk om å tallfeste problemet. Prosedyren omhandler bygging av kulvert, bygging av jernbanespor og modellering av bevegelige toglaster. Denne metodikken ble benyttet for å analysere fire spesifikke scenarioer laget for dette prosjektet. Den generelle konklusjonen etter disse modellene er at de uregelmessighetene (forskjellene) i forskyvning sett på et jernbanespor som følge av disse stivhets-variasjonene er relativt små til sammenligning med et vanlig jernbanespor.

Den ofte antatte korrelasjonen mellom stivhetsvariasjoner på et jernbanespor og økt nedbrytningsrate i overgangssonene ble også undersøkt med ROGER 1000 data, målevognen som tar rutinemessige målinger over hele jernbanenettverket i Norge for blant annet geometriske vertikal avvik. Det ble generelt ikke funnet noen trend for de seks utvalgte kulvertene mellom de antatte stivhetsvariasjonene som følger dem og geometriske vertikalavvik målt med ROGER 1000, når disse sammenlignes med målingene tatt ved nærtliggende fritt spor.

Konklusjonen fra dette prosjektet er at stivhetsvariasjonene sett over slike kulverter er relativt små, og så lenge gitte betingelser er oppfylt er det ikke funnet noen direkte korrelasjon mellom stivhets-variasjoner, differansesetninger og økt nedbrytningsrate på jernbanesporet i overgangssonene over disse kulvertunderganger.

Notations

Common notations used in railways

L_c	characteristic lenght	[m]
EI	the bending stiffness of a rail	[kNm^2]
k	foundation modulus under a railway track	[N/m^2]
C	ballast coefficient	[N/m^3]
a	sleeper spacing	[m]
A_s	sleeper area	[m^2]
y	rail deflection	[mm]
M	bending moments	[kNm^2/m]

Common notations used for flexible steel-soil composite bridges

p	soil pressure at any section of a buried, circular pipe	[kPa]
T	circumferential (axial) thrust force	[kN/m]
R	radius of the pipes curvature	[m]
λ	the flexibility number for soil-culvert interactions (in SDM)	[-]
n	the flexibility number soil-culvert interactions (other)	[-]
E_s	charateristic tangent modulus of a backfill surrounding a culvert	[MPa]
D	span lenght	[m]
EI	a pipes cross-sectional bending stiffness	[kNm^2/m]
K	the horisontal earth pressure coefficient for a buried pipe	[-]
K_A	active earth pressure coefficient	[-]

Common notations used in geotechnics

E_{50}	the secant stiffness from a standard triaxial test	[kPa]
σ_3'	horisontal effective stress	[kPa]
a	attraction	[kPa]
p_{ref}	atmospheric pressure, constant at 100 kPa	[kPa]
m	power modulus for defining the characteristic stress distribution	[-]

Other

I	Second moment of inertia	[m^4]
f	frequency	[Hz]
v	wave speed	[m/s]
λ	wave lenght	[m]

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The transition zone of Gouda railway culvert

1 Introduction

1.1 Transition zones

For many decades has the transition from embankment to bridge been known to be a problem area which often leads to extra maintenance and renewal work at road and railway infrastructures. In the US, the problem has often been refered to as «the bump at the end of the bridge» from the fact that differential settlements accumulates at the bridges end over time, increasing the need for maintenance.

From a study performed in 2002 by the Kentucky transportation center in the US, from more than 600 000 roadbridges was it found that more than 25% had problems with bumps at their ends. In the study it was stated that the cause of the bump was differential settlements, often occouring due to low quality of the backfill materials in combination with poor compaction work close to the bridge, but could be tied to the structural properties of the bridge aswell [1].

In 1999 the European Rail Research Institute (ERRI) compiled the first state of the art report on the problem for railway bridges, describing a railway transition zone as both a differential settlement and a stiffness transition problem. The bump in the transition between embankment and bridge can be created due to the sudden change in track stiffness which leads to higher dynamic impact forces in the veichle-track interaction [2]. Similar descriptions has later been used by several authors on the subject [3-7].

Similar to the findings from the road bridges in the US, the root cause for the differential settlements occouring at railway bridges is considered dispersed. Some authors has suggested that yet a cause for the bump could be the inherent settlement rate of the ballast itself in the backfill opposed to the bridge, to a degree leading to adverse dynamics in the train-track interaction which increases the deterioration rate of the transition zone and contributing to the problem [4,6,8].

In Norway the measuring department from the norwegian railway state company Bane NOR, regulary registers vertical deviations in track geometry when passing over railway bridges with their measuring coach ROGER 1000. This is the only clear available evidence that shows the same tendencies at norwegian transition zones as described internationally [1-7].

From Bane NORs measuring departments yearly track controll is it from experience well known that the type of transition zone which clearly stands out is the type where the track is fastened directly to the bridge deck while being founded on ballast in the adjecent embankment, see figure 1.1.

The suddent transition from a «free track» founded on ballast to a fixed track on the bridge deck makes it a challenge to maintain an even track allignment over time, often leading to a dip in the transition zone. The following transition zone is well known both internationally and in nationally to induce maintenance problems and is therefor avoided [2].

In Norway it has been found that the track on alot of the older railway bridges are fixed directly to their decks in combination with having no transition slab, a concrete plate which extends into the backfill to smoothen the transition from embankment to bridge. This has caused alot of maintenance problems for the bridge division in Bane NOR, consequently leading to higher tamping frequencies and even yearly reconstructions of short span bridges, completely replacing short span bridges where the track is fixed to ballasted track.

This was for example the case for Trangrud bridge shown in figure 1.1, which was completly rebuilt and replaced with trough to enable the track to be funded on ballast.

Figure 1.1 – Illustration of a fixed track: Trangrud railway bridge in Norway during reconstruction from fixed to ballasted track on the bridge with through, autum 2018 (Source: Rallar Service AS)

Internationally, ERRI stated that railway transition zones can have up to five times as high maintenance frequency compared to a normal track with the unit cost for the maintenance being twice as high [2]. In Norway, no considerations has been made for the maintenance frequency but experience from the measuring department in Bane NOR suggests that it is primarily the older bridges where the track is fixed to their decks that causes problems, especially in cases where transition slabs has not been used.

The various variables involved in a transition zone makes it hard to distinguish a failed transition zone from a successful one. According to a study from 2005 in the US, it was found for four different types of transition zones that the track settlement and degradation rate always was higher in the transition zone compared to the free track. The transition zones were built by the principles of cement stabilized back-fills, geocells or underlaid asphalt at the bridge abutments for enhancing the transition [6].

The complex variables in the interacting nature of a railway track, soil and bridge makes it difficult to point out a general root cause of the increased degradation at the transition zones. One of ERRIs concluding factors for the transition problem was that there was to poor coordination between structural, geotechnical and railway regulations which often led to the effect of each regulation not taking into account their effect on other regulations [2].

As problems in the transition zones arise to the surface level and becomes visible in the track, the end result will often be similar for all cases. Over time it accumulates a «dip» in the transition zone which contributes to increasing the axle loads, accelerating the deterioration rate further, see figure 1.2.

According the ERRI, track irregularities in the transition zone usually disperse over a lenght of 5 meters. After the vertical track irrugeluraities has reached a depth of a few millimeters, its depth will continue to increase exponentially untill maintenance has been performed [2].

Figure 1.2 – Illustration: An extreme case of vertical track irregularities in the transition zone, clearing showing the characteristic «dip» before the bridge (Source: www.railwaygazette.com)

In Norway the severity of the problem which can be seen in figure 1.2 would rarely occour as the yearly track inspections performed by ROGER 1000 would register such deviation, and report it further to the mainte-nance department. Depending on the severity of the deviation, a request for aligning the track by tamping or adding ballast will be registered, followed up by the means of maintenance.

1.1.1 Distributions on norwegian railway bridges

In order to define a «general norwegian transition zone» for this master thesis, the following section provides some general background information on all the railway bridges registered in Norway.

In Norway, the railway network consist of ca. 4200 km of track where the majority is built in the southern region of the country. According the Bane NORs railway database *Banedata*, there is in total 3594 bridges at the norwegian railway network in year 2019, 74% of which carries railway traffic and is hereafter to be defined as a railway bridge. Figure 1.3 shows the distribution of building principles, span lenghts and track execution for all railway bridges in Norway.

Figure 1.3A) shows that there are five types which commonly occours, that is through, plate, beam, bow and culvert while ca. 8% have dispersed bridge types. Ca. 75% of the railway bridges are short span bridges, ranging in between 2-10 meter while only 4% is longer than 100 meter, see figure 1.3B).

When it comes to track design on railway bridges there is only registered if they are fixed or funded on ballast, for about 50% of the bridges. As figure 1.3C) shows, even if only half of all bridges has their track design documented, this proves that alot of bridges in Norway are designed with the principle of fixing the track to the bridge, something which is well known to induce maintenance problems.

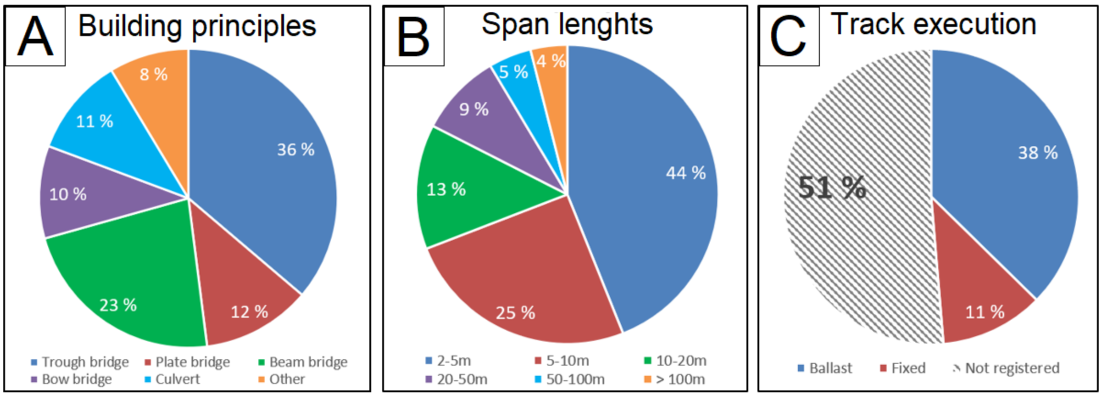


Figure 1.3 – Distribution of A) Building principles, B) Span lenghts and C) Track execution for all railway bridges according to the norwegian railway database, Banedata (Source: Bane NOR)

1.2 Culvert underpassings at railways

The type of bridge which are to be investigated further in this project are the bridge type culvert. A culvert is per definition a collective term for a burried tunnel used either as a guideway for water, technical installations or traffic. According to Bane NORs railway normal *Teknisk Regelverk*, if a culvert is bearing the railway traffic and has a span lenght over 2.0 meter, it is defined as a bridge [9].

This section gives an introduction for railway culverts in Norway. Two types of culverts are presented in this section, that is the rigid concrete bearing variety with transition slab, and the flexible steel-soil composite variety, see figure 1.4A) and 1.4B).

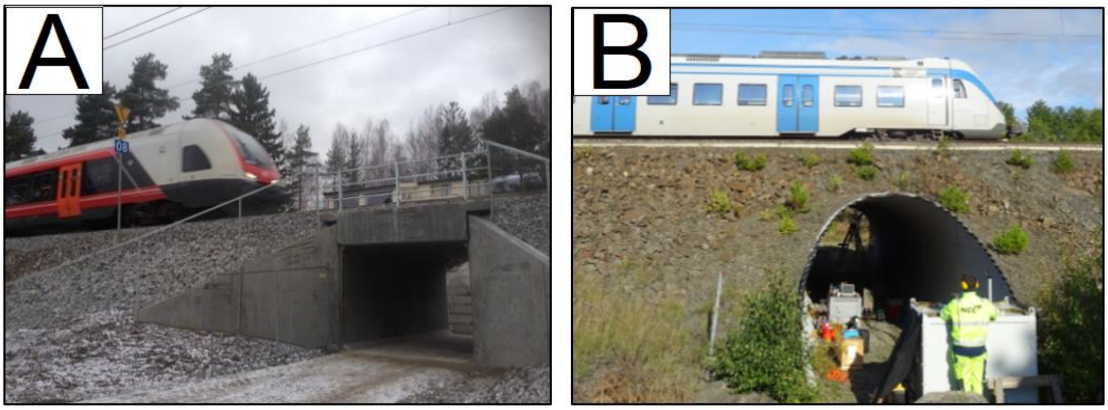


Figure 1.4 - Illustration of a train passage over A) a rigid concrete bearing culvert from Norway and B) a flexible steel-soil composite culvert from Sweden (Source: http://www.oatek.no/ and via Andreas Andersson)

Culverts is in general considered a good alternative to other bridge types as they in comparison are cheap, easy to build and reliable. Railway culverts in Norway of both the stiff and flexible variety has both been installed down to 24 hour short traffic breaks on existing railway lines.

According to the norwegian railway database Banedata, a little more than 11 % of all railway bridges is of the type culvert. Approximately a little more than 90% of these culverts is designed with concrete and a little less than 10% with flexible steel.

For norwegian transition zones all concrete culverts (fig. 1.4A) are as a minimum to be designed with transition slabs, a concrete plate which extends from the bridge into the backfill [7]. This requirement is a fixed requriement set for all railway bridges in Norway with the exception of flexible steel-soil composite bridges (fig. 1.4B), where Bane NOR refers to the road bridge normal HandBook N400 which states that bridges with «tunnel cross-sections» are exceptions from the requriement [10].

The following two sections provides general technical background information of both these culvert types based of current norwegian regulations, field practices and structural principles.

1.2.1 Rigid concrete bearing culverts with transition slabs

Rigid concrete culverts are built as stiff structures with their intended bearing capacity to be completely independent of how the structure interacts with its adjecent soil. This leads to the bridge being less dependent on the backfill procedure but with the cost of being more massive. Concrete culverts may fall into Bane NORs general category of stiff bridges where it is a fixed requirement to use transition slabs.

There are in general two types concrete culverts used at the norwegian railways, that is the precast fully integrated variety and the element variety, see figure 1.5A) and 1.5B). Even if some variations in each of these categories exists, only this general distinction will be discussed further.

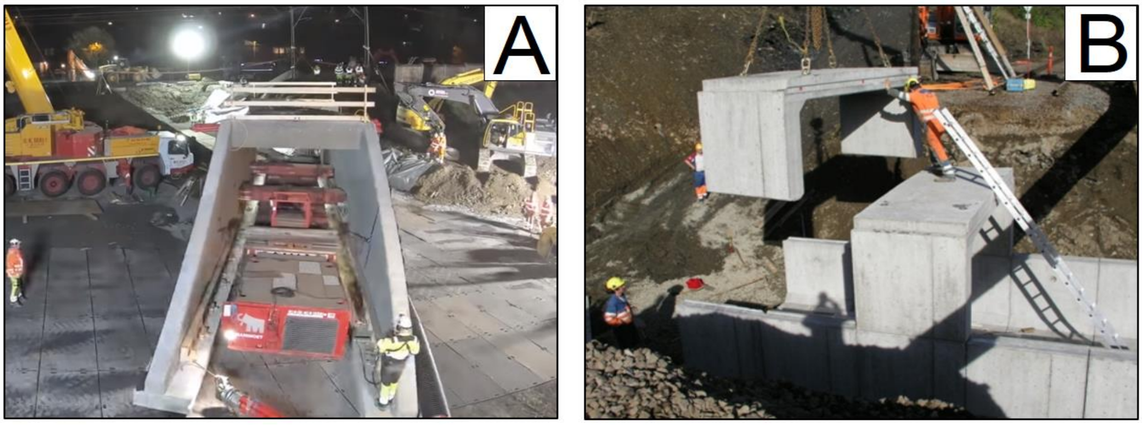


Figure 1.5 - Principles of installation: Showing A) the installation of the cast-in-place, fully integrated culvert at Hans Edges Vei in 2018 with a transportation transformer trailer, while B) shows installation of the element culvert at «Bergensbanen» railway line in 2007 with a mobile crane (Source: www.PEAB.no and www.Banenor.no)

Both varieties has been installed during short traffic breaks (24-72 hours) in Norway. The installation method however, is not the same. The integrated variety is so massive that using a convential mobile crane for installation is not feasible, and special trailers such as transportation transformers are often used. Transportations transformer makes integrated culverts an competitive alternative to the element variety on existing railway lines where construction time is a dominating factor. In 2015 a eight meter span, integrated culvert weighing over 900 tonn were driven into place with this method at Follobanen.

For existing lines in Norway, both varieties has regulary been used. But as the designed speed of the line reaches 200 km/h, it has been a preferance to avoid the element variety. This is according to Bane NOR related to the jointed variety being much harder to document are within the tolerable limits of not causing additional vibrations or resonance of the track as Eurocode 1: *Actions on structures – Part 2*, requires that all railway bridges designed for speeds over 200 km/h are assessed with dynamic analyses.

After a concrete culvert has been installed at its location the procedures tied to backfilling and building remains similar for both culvert varieties. The backfill must consist of permeable material, preferably 20/120 mm norwegian crushed rock which is to be compacted with light equipment close to the culvert (min. 1 meter from the bridge) and medium equipment further out, see figure 1.6A).

Figure 1.6 – Backfilling stages for Svenningdal culvert built in 2016, showing A) compacting a 20/120 mm crushed rock material with a light vibroplate, B) installation of the transition slab and C) construction finish (FOTO: Farbu & Gausen AS)

When the backfill has reached the same level as the transition slab, a smaller corrective layer is built with the same inclination as the slab, see figure 1.6B). After installing the slab, the culvert is covered by alternately layers of backfill material untill the formation plane of the railway track is reached.

The transition slab should according to Bane NOR be minimum 3.2 meter long and slope 1:5 down into the backfill. For smaller culverts with fill heights less than 3.5 meter, an even shorter slab of 2.5 meters lenght is allowed. The transition slab is to be constructed on a integrated corbel, which ties the slab to the culvert and prevents it from displacing away from the bridge over time.

The norwegian railway normal Teknisk Regelverk has stated that as a function of the bridges height, it is as a minimum required the use of a slab. This means that the requirements set for the slab is fixed and does not accout for train speed.

With respect to a railway superstructure the slabs intended function is to hinder differential settlements to accumulate create track irregularities over time, but also provide a gradual stiffness transition for the passing trains. This should in principle account for the two classic problems described by ERRI in 1999 [2]. A principal sketch summarizing the slabs main functions are presented in figure 1.7.

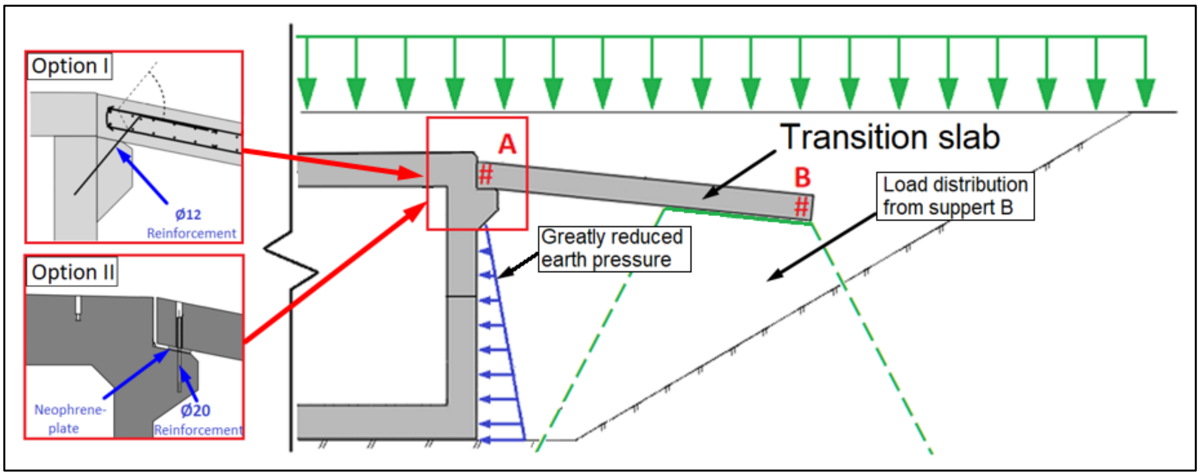


Figure 1.7 – Principal sketch showing the main functions of the transition slab, including two common solutions for the connection between transition slab and corbel used in Norway

The plate is connected to the corbel with a relatively slender reinforcement, ensuring that the plate cant displace off the corbel. The moment capacity provided by this reinforcement however may be considered neglectible, and it can be assumed that this connection rotates freely. This means that if settlement would occour at the end of the slab, it would rotate. In Norway two types of connections has regulary been used, labeled as option I and option II in the figure. Contractors has reported that option II is most practical during short traffic breaks since option I is more time consuming.

From assuming that support A rotates freely, the plate will distribute a payload about 50% to the corbel at support A and about 50% at the slabs toe at support B. Depending on the distance between the contact lenght at support B to the culvert, this distribution will ensure that little to no earth pressure from the payload reaches the bridge where the compaction quality is often lower.

When a new transition slab is built it can be expected that some change in the plates contact lenght (support B) will accumulate over time. From a case study in Netherland, it was found from static Brinch Hansen solutions that a contact lenght of 1-2 meter under a transition slab in general is sufficient to carry a railway loads [8].

More practical information on concrete culverts regarding installation principles, backfilling and construction of the slab are provided in chapter 4.1 appendix A.

1.2.2 Flexible Steel-Soil Composite Bridges (SSCB)

Flexible Steel-Soil Composite Bridges (SSCB) requires less materials than convential stiff culverts by exploiting the strenght of the surrounding backfill material. Even as the structure is flexible in bending, is it very stiff in ring compression compared to soil, an attribute which enables it to exploit the stiffness of the surrounding soil. The low material use in this structure causes it to be cheaper and clearly beneficial in terms of carbon footprint [11] compared to convential concrete culverts.

As a part of Du. G doctoral thesis from 2014 the enviromental aspect of LCA for four concrete culverts where comparted to four cases of SSCB. It was found that aspects such as the Global Warming Potential (GWP) and Cumulative Energy Demand (CED) in general was 40% lower in all cases of SSCB [11].

In 2004, construction of the Whitehorse Creek steel-soil composite bridge was executed in Alberta, Canada [12]. The arch was built with a corrugation the type «SuperCor» and steel thickness 7.1 mm. The backfill consisted of mechanically stabilized soil and the soil cover of regular soil material.

The Whitehorse Creek bridge at its time broke several world records by sustaining a span lenght of 24 meter while being able carry extremely heavy mining equipment, weighing about 15 times more than the heaviest highway veichles. In figure 1.8 a passage of a 1144 tonn mining veichle passing the bridge is shown.

Figure 1.8 - Passage of 1144 tonn mining veichle over the 24 meter long Whitehorse creek bridge (Bakht. B, 2007)

Flexible steel-soil composite bridges are rarely built over 8 meters on norwegian railways. Typically they're built with span lenghts between 2-8 meter and often with the corrugation type MP 200x55 mm.

In figure 1.9 two typical SSCB built recently at the norwegian railways are presented, showing A) the 4 meter long pipe arch «Sjånes» built in 2016 and B) the 3 meter circular pipe «Solheim» built in 2009. Solheim culvert was built with only 1.2 meter soil cover and is daily exposed to axle loads up to 32.5 tonn from the heavy ore trains traveling between Kiruna – Narvik at the «Ofotbanen» railway line.

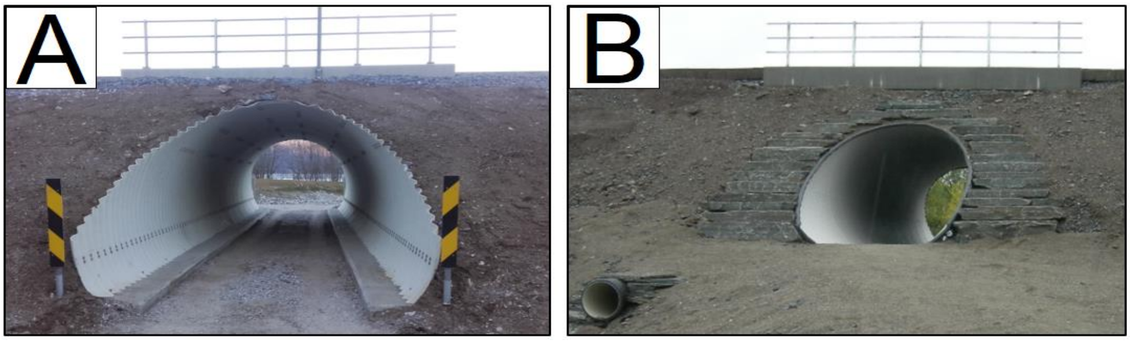


Figure 1.9 – Two typical profiles used in Norway, showing A) Sjånes and B) Solheim culvert (FOTO: ViaCon Norway As)

According to Bane NOR, flexible SSCB on railways are to be built with minimum 1.1 meter height of cover seen from the top of a sleeper [9]. This is similar to what is described by the Swedish Design Method (SDM), which requires 1.0 meter as a minimum height of cover [13]. It should be mentioned that this requirement is tied to factors associated with maintenance of the track and not the capacity of the bridge. For roads, a height of cover down to 0.5 meter is allowed according to the swedish design.

In general, the general flexible behaviour of an SSCB is little affected by the structures cross-sectional profile with the exeption of box-profiles. In Poland, two full-scale field measurements were performed on half-arches founded on beams which showed that it is the structures top shelf that governs this structures elastic behaviour when it is being exposed by surface loads [14,15]. The exception is box culverts which behaved slightly different from the other profiles.

The design in the SDM-manual covers in total eight profiles. The profiles are defined on the basis of relationships between the sectional radiuses, where five of them are closed pipes and three are open arches, see figure 1.10.

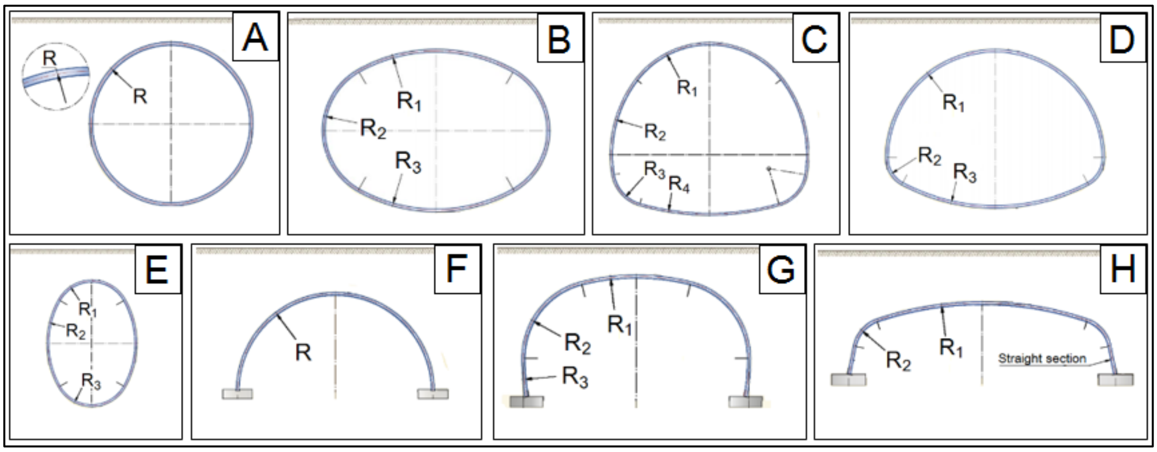


Figure 1.10 – The eight profiles covered in the Swedish Design Manual (SDM)

With respect to a railway transition zone, it is the profiles with the lowest radiuses at the top shelf which will be the stiffest ones with respect to vertical deflection, as for example profile A), C) and E), while latter cases such as profile B) and D) will be softer. The stiffer types often leads to a higher cut depth while the softer ones exploits the cross section better but with the cost of being softer.

Independent of what profile is selected for design, the structural capacity of the steel it self is rarely an issue after the structure has been installed. According to Duncan, the stiffness of flexible metal culverts in ring compression is at least 25 times as great as the stiffness of the backfill under equal all-around pressure loading [16]. Most strain on this structure occours during construction which is widely accepted to be the most critical phase for these structures [16-18].

The Swedish Design Method (SDM) does not apply for railways with line speeds exceeding 200 km/h. Similar to the jointed variety of concrete culverts, this limitation is rather set by the european codes than the structure itself, which requires dynamic analyses to prove that the structure are within the tolerable limits in terms of vibrations or resonances. Methods for doing such analyses does currently not exist [19].

In 2012 the vertical ellipse «Märsta» on the Arlanda express line in Sweden was studied with acceler-ometers. The line is daily used by commuter trains which travels with speeds up to about 175 km/h, see figure 1.11.

Figure 1.11 – An X52 commuter train passing the «Rörbro i Märsta» Culvert in 2012 (From Andersson. A, et.al 2012)

The general conclusion was that the structures interacting nature soil made dynamic analyses based of beam theory less applicable and con-trolling acceleration levels was more likely tied to soil dynamics than structural dynamics [19]. Flexible SSCB for highspeed railways are curr-ently a part of ongoing research at the university of Stockholm, KTH.

Backfilling and construction of flexible SSCB has been performed in 24-72 hour traffic stops on the norwegian railways. In figure 1.12 an example of installation during a traffic stop is presented, showing the installation of Solheim culvert at Ofotbanen from 2009. The figure shows A) installation of the culvert, B) backfilling and compaction, and C) the finished structure.

Figure 1.12 – Installation and construction of Solheim culvert in 2009 during a short traffic stop (Source: ViaCon Norway)

Steel-soil composite bridges are often delivered in boundles which are bolted together at the site, ready to be driven into place. The relative light weight of the structure keeps the need for machinery to a minimum, as demonstrated in figure 1.12A).

Similar to concrete culverts, the backfill of a flexible steel-soil composite bridge are compacted in layers, where the closest meter to the culvert are threated with more caution. What cant be adressed enought is that the backfill of this structure is not to be threated on the basis of regular backfill regulations as would be the case for a rigid culvert, but rather as a part of the bridge itself [13,17].

For Solheim culvert (fig. 1.12), the inner meter was built of sand but using a rougher 20/120 mm crushed rock material further out, see figure 1.11B). More practical examples on installation, backfilling, steel corru-gations and the Swedish Design (SDM) are provided in chapter 4.2, appendix A.

1.2.3 Ring compression theory, earth pressure distribution and arching principles

The interactions between steel and soil for flexible pipes can be better understood from highlighting some of the background theory used in the swedish design [13], which includes Duncans original Soil-Culvert-Interaction method (SCI-method) from 1979 for calculating sectional forces on a pipe [16], and arching theory developed by Vaslestad in 1990 [18].

Duncans SCI-method is based of ring compression theory developed by White and Layer in 1960 [20], which states that once a circular pipe has been installed, its circumferential thrust (axial forces) are nearly constant. By defining the overburden pressure over a pipe as an uniform pressure (p), the soil pressure along a profile can be calculated as

$$p = \frac{T}{R} \qquad \text{(Eq. 1.1)}$$

where

p being the soil pressure at any section of a circular closed pipe
T being the circumferential thrust
R being the radius of curvature at the point under consideration

Duncan statet that ring compression theory underestimates thrust forces of a pipe in an order of 30-40% [16]. This was later verified by field measurements from Vaslestad [18], but for solely explainatory reasons using this theory for understanding how profile shapes affects earth pressure is considered valuable.

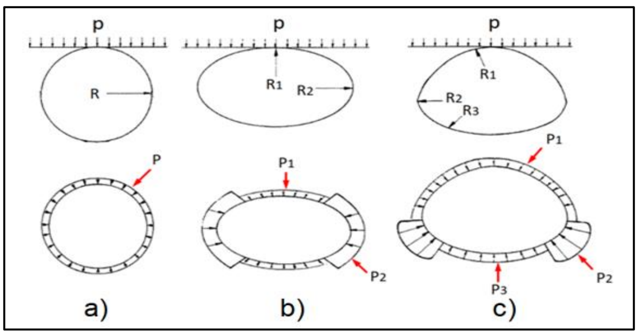


Figure 1.13 – Earth pressure distribution as a function of radius in the case of a) a circular pipe, b) a horisontal ellipse and c) a pipe arch. The figure shows profile b) and c) will undergo a earth pressure concentration (Modified after Vaslestad. J, 1990)

In figure 1.13 three closed profiles are presented, showing a) a circular pipe, b) a horisontal ellipse and c) a pipe arch. As the figure shows, based of equation 1.1 the section with the lower radius causes earth pressure concentrations.

The severe concentration in the corners of case c) gives a good explaination for why it is recommended to always ensure that the side trench of a pipe is built of well compacted, quality materials [13,17]

Another important quantity which explains the behaviour of flexible SSCBs, is the flexibility number. The flexibilty number is used in the swedish design and is based of Duncans findings in 1979, which is used to calculate forces and bending moments surrounding a flexible pipe [13].

Vaslestad later used a similar flexibility number to define the redistribution from vertical to horisontal earth pressure around a flexible pipe [18]. By knowing the soil moduli, span lenght and stiffness of the culvert, the flexibility number according to SDM is expressed as

$$\lambda = \frac{E_S * D^3}{EI} \qquad \text{(Eq. 1.2)}$$

where

E_s being the characteristic tangent modulus of the soil
D being the pipes span lenght
E being the youngs modulus of the pipe
I being the moment of intertia of the pipe

In order for a pipe to be considered flexible, a stiffness ratio of $100 \leq \lambda \leq 50000$ is required in the SDM [19]. Another way of expressing the stiffness ratio has been to reverse equation 1.2 by dividing the bending stiffness (EI) with (E_s*r^3), reported as the n-factor in Vaslestad [18].

Similar to the formulation in SDM, there is a certain limit which must be fulfilled in order for a pipe to be considered flexible, which is that the n-factor must be less than 0.1. According to Vaslestad [18], this expression can be used to define the redistribution of earthpressure around a flexible pipe as

$$K = \frac{0.18 + n * K_A}{0.18 + n} \qquad \text{(Eq. 1.3)}$$

where

K being the horisontal earth pressure coefficient for a buried pipe
K_A being the active earth pressure coefficient for soil

As equation 1.3 shows, an increase in the n-factor leads to a decrease in the pipes ability to redistribute earth pressure. Decreasing the stiffness of the pipe decreases the n-factor, which causes the pipe to distribute any surface loads better to the soil by exploiting the stiffness of the pipe in ring compression. The ability flexible pipes has with taking overburden loads in ring compression rather than bending moments demonstrates why they can be so slender compared to stiff pipes.

When the n-factor reaches an order of of 1.0, then the pipes interaction with the soil has reached a completely rigid state, causing a poor redistribution of the overburden stresses which increases the acting bending moments on the structure.

When the n-factor is under 0.1 it is normal to assume $K \approx 1$, i.e a direct redistribution from vertical to horisontal earth pressure [17,18]. In terms of long term behaviour, findings from a 7.8 meter pipe arch built under a road in Norway suggested after 21 years of field monitoring that the effect of arching remained stable while the horisontal earth pressure tends to gradually increase towards a passive state [18,21].

Yet a important aspect of flexible steel-soil composite bridges is the phenomena of positive arching, expressed by Vaslestad in 1990 [18]. The advantages of positive arching is now included in modern swedish design [13].

When a flexible pipe is compressed vertically, the soil body above the pipe tries to settle more than the adjecent soil in the free backfill (i.e on the sides). This causes vertical shear planes in a certain zone above the pipe which provides resistance from this to happend, leading to the acting earth pressure above a flexible pipe to be lower than soil pressure from the dead weight of the soil prism, see figure 1.14.

Arching relates directly to the overburden earth pressure of a pipe, and is influenced by factors such as cover height, angle of friction and service loads. A higher height of cover or increased angle of friction of the overburden material contributes more to positive arching and hence, a lower overburden earth pressure.

Arching is now a part of the Swedish design and the SDM-manual provides charts for describing the effect of positive arching on the overburden earth pressure on a flexible pipe as a function of cover height, span lenght and friction angle, see figure 1.15.

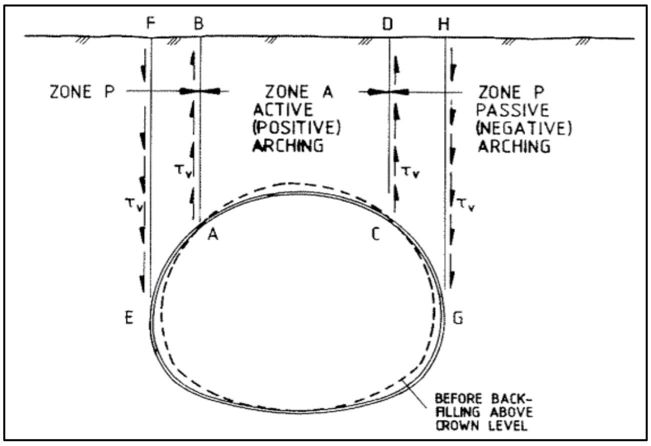


Figure 1.14 – Principles of arching (Vaslestad. J, 1990)

As the chart shows, a low height of cover leads to nearly no arching (i.e S_{ar} being 1.0) but gradually increases as the relative height of cover increases. A higher angle of friction ($\tan(\varphi_{cover,d})$) also contributes more to positive arching.

For more on construction, installation or design principles of flexible steel-soil composite culverts, see chapter 4.2 in Appendix A. More background theory are also provided in the Appendix.

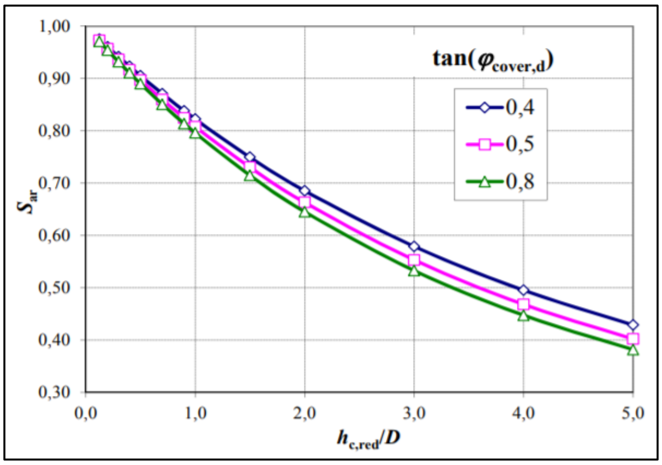


Figure 1.15 – Arching coefficient versus the relative cover depth for different angles of friction (From Petterson. L and Sundquist. H, 2014)

1.3 Problem formulation

A railway transition zone is in this project defined as the closest meters to a bridge where a railway track undergoes a change in track stiffness. The transition zone is divided into three main parts, that is the free track where the stiffness is constant, the transition zone where the track stiffness undergoes a change and the bridge where it is constant, see figure 1.16

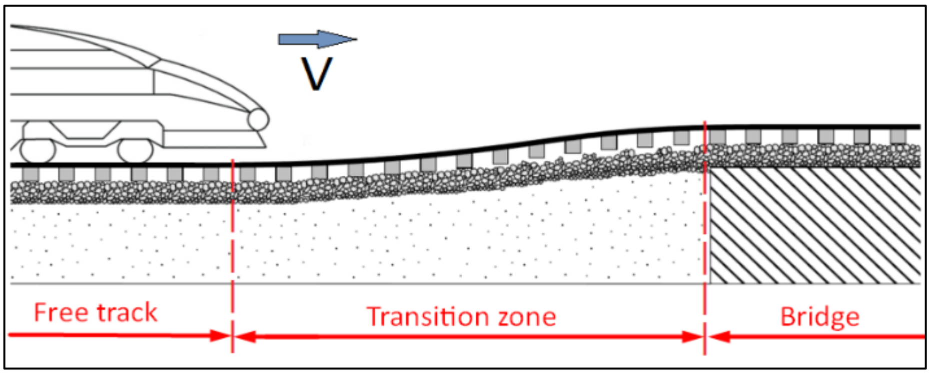


Figure 1.16 – Definition of a transition zone in this project, dividng it into three main parts: The free track, the transition zone and the bridge

Railway transition zones is according to Bane NOR a problem area where the deterioration rate of the track is normally higher and extra maintenance operations such as tamping and levelling of the track is required. But there is currently no specific explaination for what really causes these problems at the transition zones but is similar to ERRIs explaination, thought to be related to track stiffness and differential settlements at the transition from the regular embankment to the bridge.

The way of solving this problem in Norway has been to instrument all new railway bridges with transition slabs, but also avoiding fixing the railway tracks to the bridges. The transition slab is a fixed requirement set for all types of new railway bridges in Norway, and typically spans about 2.5-3.2 meter depending on the size of the bridge.

This fixed requirement applies independent of train speeds, axle load or local track conditions which are not quite understood when comparing them to international transition zones at high speed lines, where they can reach up to 20-30 meters lenght [2,22]. This raises the question if the relative short norwegian transition slab is enough to prevent the classic problems seen at transition zones from happening.

The transition slab has been reported by Bane NOR to undisputedly improve the transition zones in Norway compared to not using it, but very few knows exactly why it works. The common explaination is that it covers up the closest meter to the bridge where it is highest potential for differential settlements and that its sloping angle ensures that the railway track undergoes a gradual stiffness transition.

When thinking of stiff railway culverts the same fixed requirements described above applies, there is a minimum requirement that they must be instrumented with 2.5-3.2 meter long transition slabs if they span longer than 2.0 meter. For flexible Steel-Soil Composite Bridges (SSCB) however, Bane NOR only requires a minimum soil cover of 1.0 meter and further refers to the Norwegian Pulic Roads Administrations handbook series, which has stated that due to these structures circular geometry, no requirement is set for the transition zones [10], which rasies another question with respect to the problem.

1.3.1 Objectives

The objectives of this project is to investigate and compare the transition zones of rigid concrete culverts instrumented with transition slabs to the transition zones of flexible steel-soil composite bridges where no requirement is set. The overall goal of this comparison is to investigate how much a culvert built under a railway track and variables associated to them really affects the track stiffness, and further investigate and quantify how big of an irregularity at the track these stiffness variation causes.

The findings from this study are then compared to the yearly track inspections performed by Bane NOR, namely ROGER 1000 measurements for geometric track deviations. This part of the project will be carried out in order to investigate if there can be found any correlation between the stiffness variations over these culverts and the claimed demand for additional maintenance purposed in the litterature.

The report finishes the study with a discussion and comparison of the stiffness variations over these culverts ROGER 1000 measurements, but also compares and discusses these findings to other authors findings related to this topic.

1.3.2 Methods

The first approach is to study transition zones with the use of the FE software PLAXIS 2D, where the approach in this project is to model the entire transition zone from skratch. This involves preparing a foundation, building a culvert, backfilling, constructing a railway track and modelling moving trains.

The various aspects related to the methodology presented in PLAXIS are compared to several field measurements for validation, and the goal with model was to develop a general methodology that could be used for both kind of culverts, but also able to account for norwegian conditions such as backfill procedures. This approach is then used in the study of how variables such as the transition slabs lenght for stiff cases, or how the geometry or span lenght of a flexible pipe affects the track stiffness.

The second approach is to investigate the geometric deterioration rate of the track at the six preselected railway culverts from Norway with ROGER 1000 measurements. Vertical track deviations registered by this measureing car is the basis for the workorders sent from the measuring department in Bane NOR when maintenance is requested, and will provide some information on how severe and how often differential settlement accumulates at these culverts.

1.3.3 Prerequisites and limitations

When generalizing railway transition zones one must be aware of all variables that might affect the track stiffness. In this project the span lenghts of all analysed culverts must be in between 2-8 meter (i.e «short span»), which means that cases where special sleepers or auxiliary rail has been used in the transition zones can be neglected as such countermeasurements only applies for bridges longer than 10 meter [9].

A challenge with assessing especially older, existing transition zones is that problems arising from cases where the track is fixed to the bridges deck has taken away all the attention from the less extreme cases. A prerequisite set for the track superstructure is therefore that the track as minimum must be continously founded on ballast over the entire bridge as shown in the principle sketch in figure 1.17.

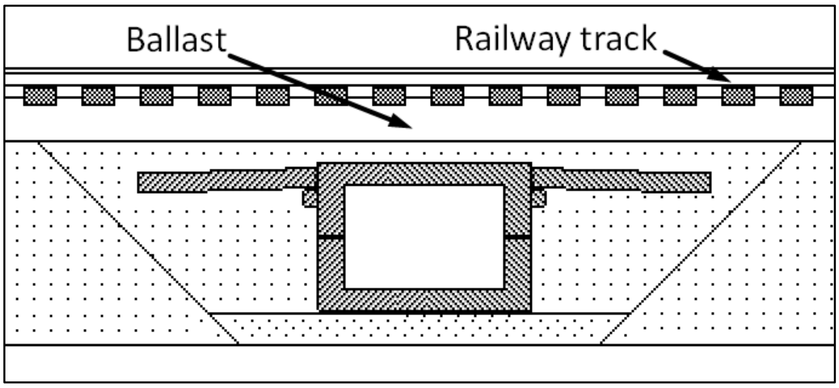


Figure 1.17 – The minimum prerequisite set for all transition zones in this project, which is that the track must be founded on a continous ballast bed over the bridge

Even though the finite element analyses from PLAXIS includes some dynamics assessments for describing the models behaviour, another condition set for this project is that it is primarily the quasi-static aspect of the track (i.e displacements and reaction forces) that are investigated in these transition zones and aspects such as acceleration levels and veichle dynamics are not covered.

For assessing the maintenance frequencies, only ROGER 1000 measurements with registrations of vertical deviations are included. This was thought to provide more valuable information than investigating data from tamping or track maintenance frequencies as it was found that there was to much uncertainty tied to these procedures with respect to the time constraint in this project.

For some of the six preselected culvert cases for the ROGER 1000 assessments, little to no background information was available for some cases (i.e design drawings, local conditions etc). It was therefore set a minimum prerequisite for these cases that design drawings was available for the flexible steel cases and that both culverts must have been built in the year of 2000 or later.

From this prerequisite it can be assumed that the preselected concrete cases has been instrumented with transition slabs based of Bane NORs suggestion that the transition slab was standardized around year 1990-2000. Only refering cases after year 2000 is also advantegous for flexible steel cases since the Swedish Design Method (SDM) was introduced in the same year, which has been the basis for several presumptions made in the model.

Using ROGER 1000 photos for evaluating the local track conditions (i.e avoiding curves, cases with auxiliary rails) were thought to be sufficient.

1.4 Structure of the thesis

This master thesis is written in total 6 chapters and is divided into four main parts. The first part introduces the topic of the thesis, and is followed by a second part which outlines a comprehensive methodology in PLAXIS 2D used for modelling four specific railway transition zones. This part is followed by a discussion and comparison to field observations in a third part to finally come to a conclusion in a fourth part.

The structure of this thesis:

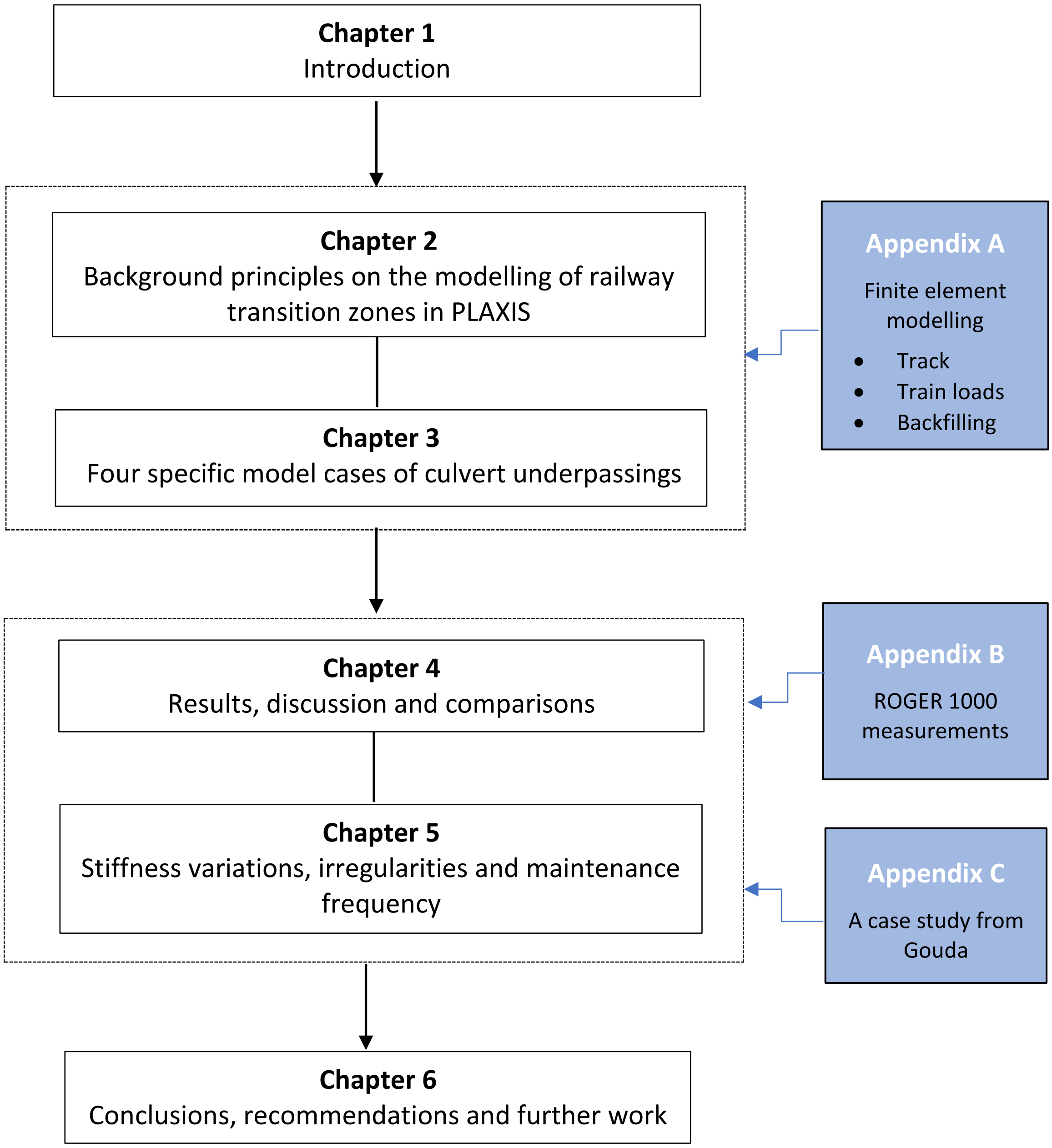


Figure 1.18 – Setup of this master thesis, showing that it is divided into four main parts

The purpose of the first part (chapter 1) is to provide all the necessary background information for the topics discussed in this master thesis. It starts with outlining some general aspects of railway transition zones, then moves to culvert underpassings at railways and finishes with a problem formulation.

The second part is written for outlining the entire background for the finite element models that was created to analyse four specific culvert cases. Chapter 2 of this part starts with outlining the entire methodology of the model, and involves modelling of a railway track, train loads and the effect of backfilling in plane-strain which is the basis for the models created in chapter 3.

From figure 1.18 it can be seen that this part refers to appendix A, a individual report written soely for explaining the finite element model on detail level. This report should be looked upon as a referance report for chapter 2 and 3, and outlines all the assumptions and decisions made for the various aspects of the model in detail. Chapter 2 on the other hand, is written more like a summary of this report which often will be refered to when it refers to simulations that was performed during the project.

The third part first validates, compares and discusses the results of the FEM simulations in chapter 4 with field measurements. These FEM simulations are used to quantify and assess how variables of the culvert and backfill affects the stiffness variations and irregularities seen at the track. Chapter 5 of this part compares and discusses these aspects further, where a discussion between stiffness variations, deterioration and maintenance are done. This chapter also compares and discusses the FEM results to the observations made with ROGER 1000 data of the six preselected culvert cases.

Figure 1.18 also shows that this part refers to appendix B and C. These reports should as appendix A, be viewed upon as individual reports where appendix B provides all the necressary background information on the six preselected culvert and the ROGER 1000 measurements that was used in evaluating these six transition zones. The second report (appendix C) refers to a comprehensive case study that was carried out in Netherland on a concrete culvert. This report provides unique insight to potential fall pits in relation to a transition slab, and is used as a reference in the discussion of chapter 5.

The last part (chapter 6) summarizes the entire project and comes to a final conclusion regarding the discussions of stiffness variations, irregularities and maintenance at culvert underpassings.

It should be noted that the main report summarizes all the findings of importance from the appendixes, and should alone be considered sufficient for understanding the entire framework behind the findings of this project. Most of the chapters in the main report is also written in such a way that they can be read individually, but it is highly recommended to read the main report from the start to get the full picture.

Some aspects of the appendixes may not be covered here, and one should bear in mind that on topics such as modelling (refered to in appendix A), new discoveries were made in the later stages of this project and it is important to treat the descriptions given in the main report as the one that applies while the appendixes should be threated more like supplementary litterature.

1.4.1 Appendix A

This individual report is written soely for finite element modelling in PLAXIS and provides all the necessary background information on the findings and calibrations that were made for the main report with the expection of the stiffness problem discussed in chapter 2.5.

The report is made out of three parts, that is modelling a railway track, moving train loads and constructing culverts with backfilling procedures. Alot of the comparisons made in this report is only summarized or refered to in the main report, so it is important to treat this appendix as a referance for the main report when certain aspects of the model are discussed.

It is important to highlight that this report was written from the beginning of this project, and may show some inconsistency when comparing the input parameters in some of the calibration examples to the input parameters used for the final FEM-simulations of chapter 3 in the main report. It is the final decisions made in the main report that stands in this project.

1.4.2 Appendix B and C

Report II and III was written more as an assessment of maintenance and field behaviour (in terms of deterioration), and should similar to appendix A be threated as referance reports. Only the most important aspects from these reports are covered in chapter 4 and 5 of the main report, but further references are made to these appendixes were it is necressary.

Appendix B outlines all the available background information and conclusions made for the six preselected transition zones. The report starts with outlining principles on how the measuring car «ROGER 1000» measures vertical track deviations and further presents each culvert case with exact coordinates. Each of these cases are presented with the available ROGER 1000 data which are discussed in individual chapters. The report is summarized in the last chapter where a general conclusion is made.

Appendix C referes to the case study «Gouda Goverwelle» which was a study on a railway culvert in Netherland. This case may be viewed upon as very special, the culvert was founded directly on piles while the free track was built on top of a sand embankment directly on top of a subsoil made of soft clay. The clay settled for about 1 mm/month which evidently led to differential settlements shortly after maintenance was performed.

Whats valuable with this study is the unique size of it, were a combination of field surveys, long-term static monitoring with dynamic measurements were carried out in a periode of two years. Especially the static field measuremetns are both rare and valuable in terms of understanding the slabs behaviour with respect to time, but also understanding potential fall pits with the solution.

2 Background principles on the modelling of railway transition zones in PLAXIS

The overall aim of the FEM models in this project has been to best catch the quasi-static behaviour of a railway track when train loads are passing over a culvert. This chapter summarizes the background principles for this model which is divided into three main parts, the railway track, moving train loads and the aspect of construction, backfilling and compaction. The principles outlined for these three parts is the basis for the four specific transition zones created in chapter 3.

The content presented in this chapter is to be considered as a summary of Appendix A – REPORT I. Both principles and results from selected simulations are presented here, as these were used for validating several aspects of the model.

The following sections of this chapter is divided into four parts, the first one explaining the principles of plane-strain, i.e how the 3D aspect of the transition problem on a railway track can be transferred into a 2D plane-strain model for PLAXIS.

The second section focuses more on the static behaviour of the railway track system which starts with describing the fundamental principles of the model, then shows a comparison to classic theory for beam on elastic foundation. The third part involves the aspect of moving train loads with dynamic assessments, showing some of the frequency domains that were located at the track and in the soil.

Lastly, a comprehensive methodology for catching the effect of backfilling and compaction in field are purposed by using a Hardening Soil model. This approach was considered a necessity in order to catch any realistic behaviour of the flexible steel-soil composite culverts during passing train loads as it is the construction and backfilling phase that governs these structures long-term behaviour.

2.1 Plane-strain

Plane-strain in PLAXIS 2D is a principle that allows for any model to only strain in only two dimension, the x- and y-axis. The remaining z-axis is the axis perpendicular to the model. This axis can't strain and has the unit of pr. effective meter plane. The plane at this axis represents an intersection of all componen-ts at this two dimensional plane (soil, structures), in and out of the z-axis pr. meter plane, see figure 2.1.

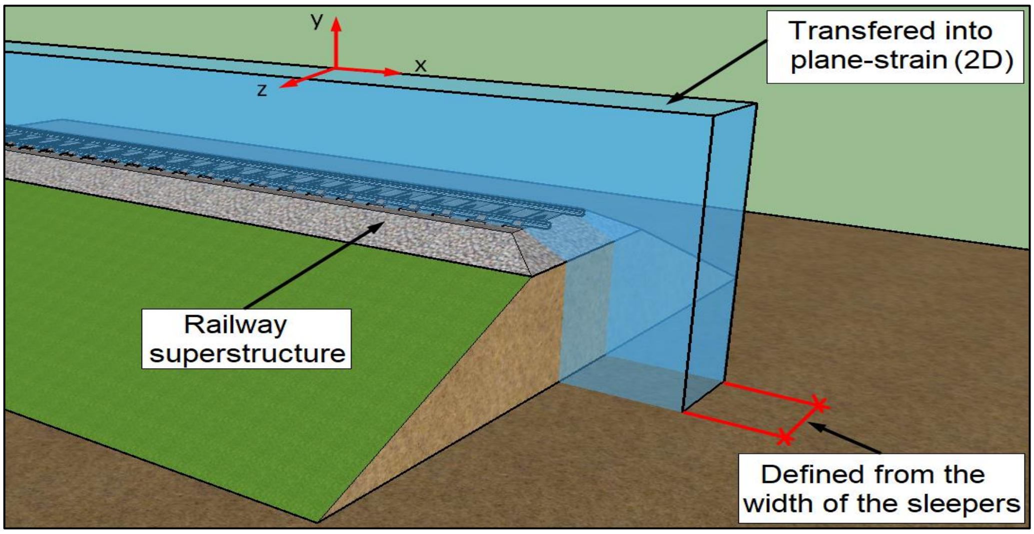


Figure 2.1 – Principal sketch of the principles of plane-strain. An effective width is taken from the basis of the sleepers, where all track components above the sleeper is transfered into plane-strain (effective width)

In order to use this principle for a railway track, these planes can only intersect within the width range of the sleepers. From assuming that any plane in this width range has the same properties pr. effective meter as an equivalent railway track, the three dimensional aspect of the problem has then been transformed into a 2D problem. A prerequisite for this transformation is that the sleeper distributes the train loads equally at any section of the sleeper and the embankment is symmetrical, see figure 2.1.

This is achieved by dividing all the track component on top of the sleepers by the width of the sleeper itself, which transform the properties of these track components into the unit pr. effective meter plane (i.e plane-strain). These properties only applies inside the square shown in figure 2.1. Any object in the plane within this width range (the square), has the equivalent properties to a three dimensional object with the only difference that the plane cannot strain along its z-axis.

2.1.1 Railway track

The standard railway sleeper in Norway is the 2.6 meter wide JBV 60 sleeper [23]. In order to use the principles of plane-strain for this sleeper, the width of it will govern the properties of all track component and train loads pr. effective meter plane in PLAXIS 2D, see figure 2.2.

To apply this principle for strenght properties of the track components (i.e their bending and axial stiffness), they are first multiplied by two (two pr. sleeper), then divided by the width of the sleeper. The properties of the track components deriven from this methodology will then represents the properties of this component in plane-strain, i.e a property that is equivalent to the property of an equivalent structure in three dimensions.

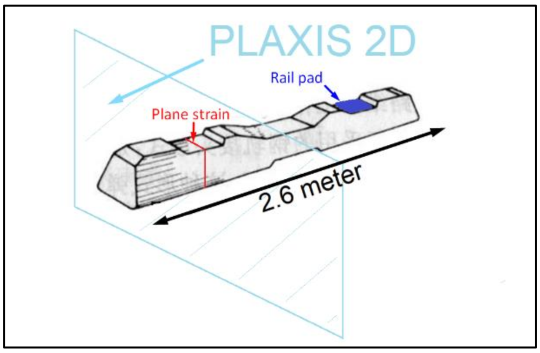


Figure 2.2 – Principles of transforming track components into plane-strain, by multiplying their strenght proper-ties by two for then to divide them by the width of the sleeper, which in plane-strain is equivalent to their original properties

Similar to the track components, the train loads must also be transfered into plane-strain. Instead of thinking in terms of wheel loads the axle load is instead transfered into force, then divided by the width of the sleeper. This means that an axle load of 22.5 tonn in only acts with 8.65 tonn in plane-strain.

2.1.2 Culvert, track and soil interaction

The shoulder width of a ballast bed measured from the end of a sleeper must as a minimum be 0.4 meter or more according to Bane NOR [9]. When thinking in terms of soil bodies such as the ballast, embankement and fundation, it is thought that these soil bodies are completely symmetrical in 3D, seen from the center of the sleepers. This means that within the width range of the sleepers (i.e «the square» in figure 2.1), the properties of the soil bodies are unaffectedly transformed into plane-strain since they're only divided by an effective width of 1.0 meter.

When introducing a concrete culvert into this system for example, the section of plane-strain governed by the sleeper width must also consider the culvert. By assuming that the culvert and transition slab is long enough, the effective width of the culverts cross-sectionion will be equivalent to an effective width pr. meter in plane-strain (similar to the soil), only having to divide them by 1.0 meter, see figure 2.3.

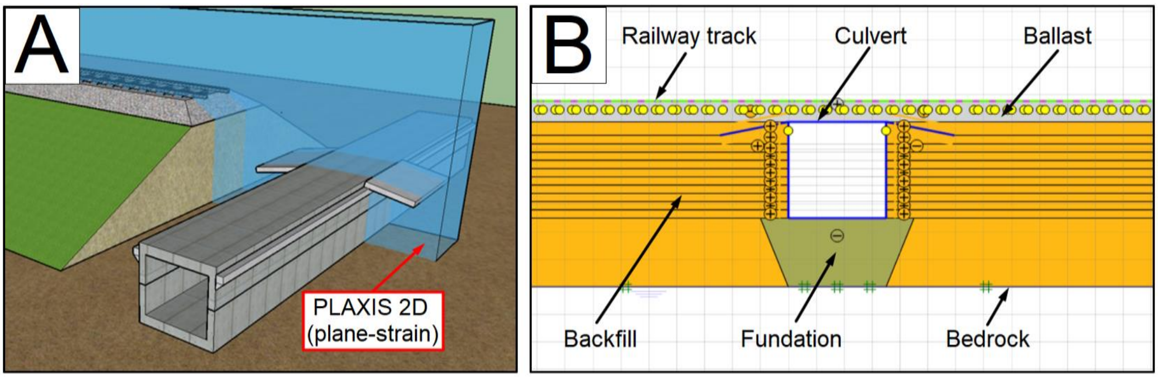


Figure 2.3 – Principles of plane-strain when introducing a concrete culvert into the system, showing A) a principal sketch of the culvert in relation to the railway structure and B) an illustration of an effective plane in PLAXIS 2D with designations

By assuming that the transition slab is centered in relation to the track, similar to the culvert and soil its properties within the effective width (the square in figure 2.3A) only needs to be divided by an effective width of 1.0 meter to be transferred into plane-strain. Figure 2.3B) shows an intersection of this plane from PLAXIS 2D. Founded on the ballast, there will be a system which represents the track structure. This superstructure is founded on a backfill and a culvert which further is founded on a bedrock. More information on each individual component of this system will be adressed later in this chapter.

Similar to a concrete culvert, the effective width of a flexible steel-soil composite bridge in the width-range of the sleepers only has to be divided by an effective width of 1.0 meter to be transfered into plane-strain, see figure 2.4.

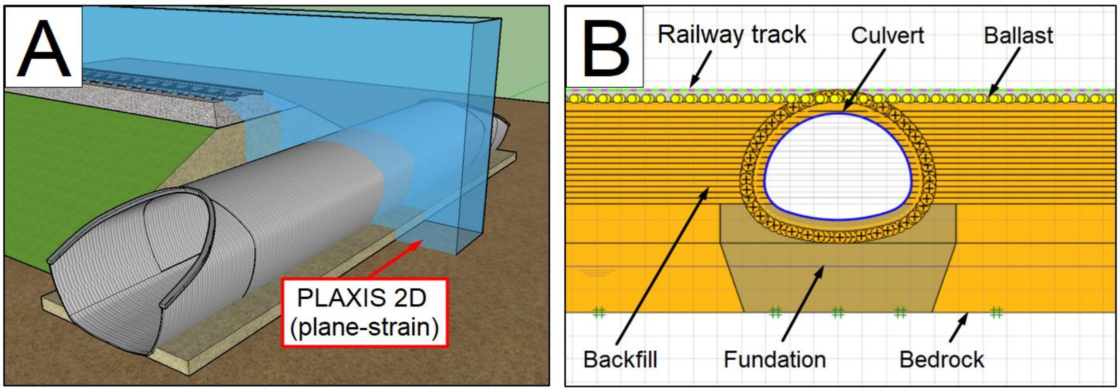


Figure 2.4 - Principles of plane-strain when introducing a steel culvert into the system, showing A) a principal sketch of the culvert in relation to the railway structure and B) an illustration of an effective plane in PLAXIS 2D with designations

In figure 2.4B) it can be seen that the other components of this intersection is identical to the case with concrete. There is a structure founded on ballast which represents a railway track, further founded on a backfill and over a culvert.

In the next section basic principles for modelling any structure in plane-strain as plate elements presented. More on transforming culverts into plane-strain can be found in chapter 5, Appendix A.

2.1.3 Modelling structures as elastic plate elements

Plate elements are two-dimensional line elements in PLAXIS that represents 3D structures by giving them the properties of an equivalent three dimensional structure. The standard option for plate elements when using a triangular soil cluster has 5-nodes, each given three degrees of freedom [24], see figure 2.5.

Whats really convenient with these plate elements is that once a structures properties has been transfered into plane-strain, their properties can be used directly in these plate elements for PLAXIS. Since these plate elements are line elements, they are modelled with zero thickness. Their weight is therefore formulated as pr. meter plate, pr. effective meter plane.

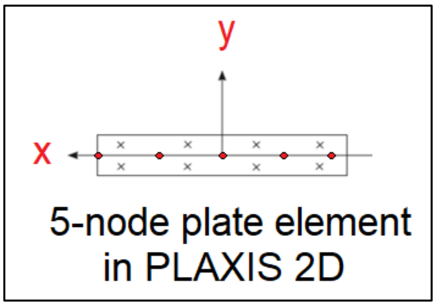


Figure 2.5 – A 5-node plate element in PLAXIS 2D pr. effective meter plate pr. effective meter plane. The axis system corresponds to the same axis system in figure 2.1

This formulation means that the weight of the plate element considers the weight of the plate pr. meter plate in PLAXIS (the x-axis in figure 2.5) while accounting for plane-strain at the same time (along the z-axis). If a structure is long enough (seen at the z-axis), the weight of this structure is deriven for PLAXIS from only multiplying its unit weight with the thickness of the real structure (the thickness of the element, seen from the y-axis, figure 2.5) as the z-axis already is in pr. effective meter.

In order to transform the weight of the sleepers and rails into pr. effective meter plane however (plane-strain along the z-axis), they must also consider the sleepers width because it is the sleepers that governs the section of plane-strain in this model.

For example, a JBV 60 sleeper weighs 285 kg, is 2.6 meters long and on average, approximately 0.2 meters thick [23]. For the weight of this sleeper to be transformed into plane-strain, its weight first has to be transferred into force then divided by the lenght (2.6 meter) and width of the sleeper (0.3 meter) which gives the weight of this plate element in PLAXIS per. effective meter. For more on transforming sleepers and rails into plane-strain see chapter 2.2.2, appendix A.

When it comes to modelling culverts as plate elements the simplest variety is the flexible SSCB. The Swedish Design Method (SDM) provides cross-sectional data in mm/mm for the various corrugation profiles as a function of steel thickness, which means that the data given by the manual already is in pr. effective meter (i.e plane-strain). The only work remaining is to multiply the unit weight of steel by the thickness (t) of the plates, see figure 2.6, also see chapter 5.2 in Appendix A for more.

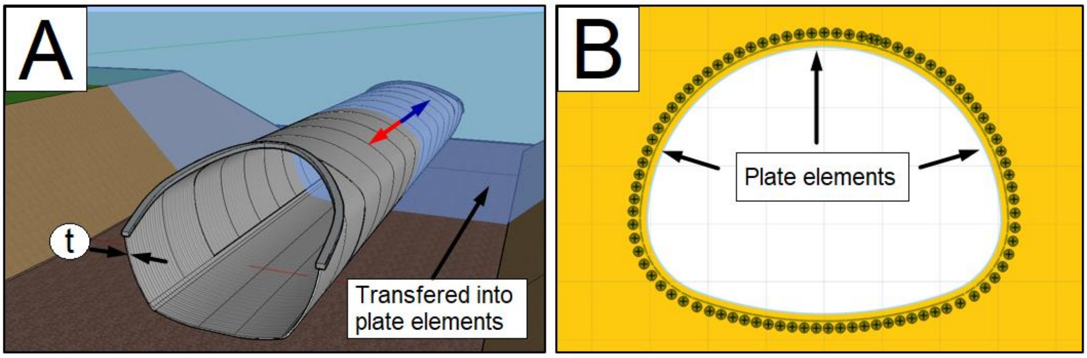


Figure 2.6 – Principle sketch of transforming the steel plates of a flexible steel culverts into plane-strain, showing A) the section which intersects the structure and B) the transformed structure with plate elements

In the case of a concrete culvert however, the cross-sectional data will vary after the thickness of the concrete. According to norwegian manufacturers of concrete elements, a typical railway culvert has a slightly thicker roof and bottom element compared to its wall thickness. Bane NOR also requires a minimum 0.3 meter thick transition slab [9]. By simplyfing these elements of the structure into three different plate elements (roof/bottom, walls and transition slab), the thickness of them only has to be multiplied with the unit weight of concrete to get the weight of each plate elements, see figure 2.7.

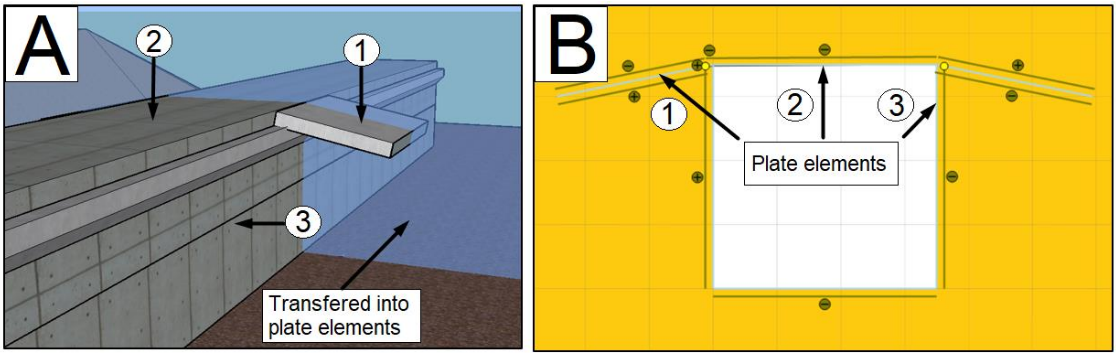


Figure 2.7 - Transforming the concrete plates of a culvert into plane-strain, showing A) the section which intersects the structure corresponding to three different plate elements, and B) after transforming the structure into plate elements

In order to transform the strenght properties of these elements as accurate as possible into plane-strain, properties of the structural connections between them can be considered. In this project however, the properties of each element has been threated individually as shown in figure 2.7 and 2.8.

Similar to calculating the weight of the plate elements, when considering the strenght properties of them, each element of the cross-sectional planes are defined from the width of the sleepers which has to be divided the culvert into three elements, the roof/-bottom, the slab and the wall elements, see figure 2.8.

By neglecting the corbel of the culvert, the only necessary data for the plate elements will be the moment of inertia, the youngs modulus and the cross-sectional area of each element, defined as shown in figure 2.8.

The second moment of inertia for any square object can be calculated as

$$I = \frac{bh^3}{12} \qquad \text{(Eq. 2.1)}$$

where

I being the second moment of inertia
b being the width of the element (x-axis)
h being the height of the element (y-axis)

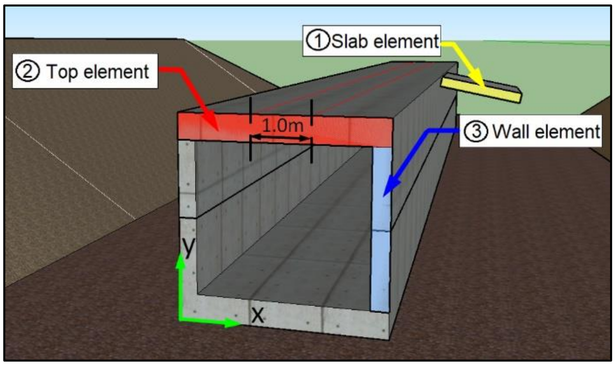


Figure 2.8 – Simplifications made for a concrete culvert when transforming it into plane-strain as elastic plate elements

In order to calculate the second moment of inertia for these wall elements, equation 2.1 can be used directly. For defining this quantity of the roof and slab elements however, the basic formulation of plane-strain must be taken into consideration. An effective strip of 1.0 meter is therefore defined along the x-axis of the top ② and slab element ③, which represents the width (b) in equation 2.1. Similar is done for the slab element where the only difference to the top element elements is the thickness (h).

For more on transforming concrete culverts into plane-strain, see chapter 5.1 in appendix A which shows a full example for calculating cross-sectional data for a 3.5x3.5 meter concrete culvert.

2.2 Railway track, static behaviour

This section concerns the aspect of modelling a railway track in PLAXIS. A purposed track model are presented, followed by a summarization of two calibrations examples of the ballast and railpad stiffness. A more detailed description of these calibrations can also be found in chapter 2, Appendix A.

The overall aim of the track model has been to account for the rails ability to distribute any load over to several sleepers realistically. This behaviour is considered a necessity in order to catch anything representative for the quasi-static behaviour of this railway track. The calibrations presented here are mostly based of experience numbers from Deutche Bahn and Bane NOR.

2.2.1 Principles of constructing a railway track

According to Bane NORs technical design basis, all new railway tracks in Norway are to be built with 60E1 rails, JBV 60 sleepers with the Pandrol Fastclip FE 1404 fastening system including Pandrol 10 mm railpads [23], see figure 2.9.

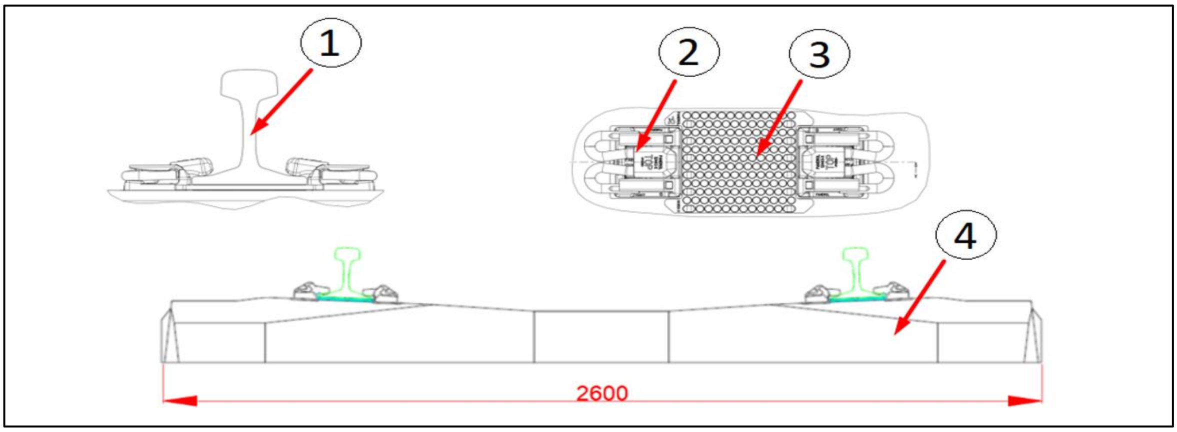


Figure 2.9 - Track components for a standard railway track in Norway based of the InterCity design basis, showing ① 60E1 rail, ② Pandrol Fastclip FE 1404 system, ③ Pandrol 10 mm rail pad and ④ the JBV 60 sleeper (Teknisk regelverk 2019)

The goal with this track model was that it as a minimum were able to account for the rails ability to distribute any load over to several sleepers, but also to account for a railpads ability to compress which governs point load concentration under a point load. From these prerequisites, the track was modeled with elastic plate elements where the sleepers were thought of as load distributors from the rail to the soil. The pad elements are modelled as vertical plate elements with the purpose of elevating the rail and with the ability to compress which govern the systems ability to distribute any point load from the rail.

As was introduced in section 2.1, this is maintained by dividing the properties of all the track components on top of the sleepers by the lenght of a JBV 60 sleeper [9], which corresponds to pr. meter effective plane over a sleeper in plane-strain. Everything above the sleepers are modelled by this principle, i.e train loads, rails, railpads and even the sleeper itself.

A section of the track model at one sleeper is shown in figure 2.10, where it can be seen that the rail ① is considered a continous plate element as if this was a Continously Welded Rail (CWR). The pad element ③ is divided into two vertical plate elements, spaced on the basis of the width of a railpad (0.15m).

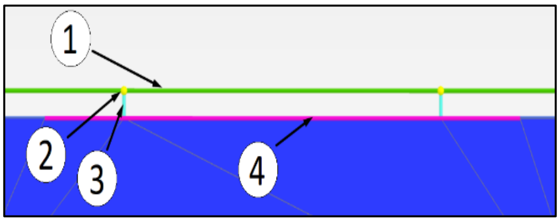


Figure 2.10 - The track structure from PLAXIS 2D, showing the interaction of all track components in the system, where element ① is the rail, element ② is the hinged connection between rail and pad, element ③ the sleeper pad and ④ the sleeper on top of the ballast

The pad elements ③ is connected by two freely rotating hinges ② to the rail ①. This was a necessity in order for the rail in this system to not be influenced by the strenght properties of the other track components. Lastly, the pad elements are connected to the sleeper element by fixed points so the two vertical pad elements remains rigid in relation to the sleeper.

This systems allows for the rail to bend unaffected in relation to all the other track components which was considered close to reality when comparing this solution to a real fastening system which shouldn't provide any additional rotational stiffness to the rails. The axial stiffness of the pad elements in this system will govern the tracks load reponse to any point load applied at the rail.

The next sections highlights the results from two calibrations, that is calibration of the railpad stiffness and calibration of the ballast stiffness. This tracks behaviour was also validated by comparing its behaviour to classic theory for beam on elastic foundation, namely Zimmermanns theory.

2.2.2 Railpad stiffness

When transforming the track components of this system into plane-strain, it was found that the railpads compressive stiffness at the vertical plate elements (the railpads in the model) was to soft.

According to Jernbanekompetanse.no, a pandrol 10 mm railpad should have an average compressive strenght of 45 kN/-mm. This means that it requires 45 kN of force to compress a railpad 1.0 mm on a sleeper [25]. This formulation is based of static load tests performed on railpads where the average value of it is calculated based of the two secants shown in figure 2.11.

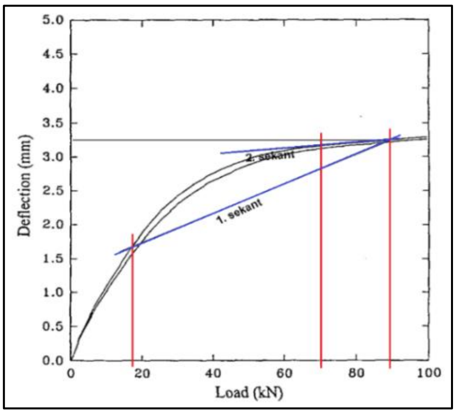


Figure 2.11 – Loading vs deformation, showing how the pad compresses as a function of load (Jernbanekompetanse, 2019)

When transforming the properties of the two vertical railpad elements at the sleeper to plane-strain, the average axial stiffness of 45 kN was first multiplied by two, divided by the sleepers width of 2.6 meter, to then again be divided by two (since the pad was defined with two plate elements), see figure 2.10. From doing so, it led to a railpad stiffness of 17.3 kN/m in plane-strain.

After running several static simulations in PLAXIS it was found that a railpad stiffness of 17.3 kN/m always led to unacceptable high deformations of the track. The pad would always compress more than 5.0 mm under a 22.5 tonn referance axle, which according to figure 2.11 is not realistic since the vertical deformation starts to flatten after it has been compressed more than 3 mm. It was concluded that the plate elements elastic behaviour in PLAXIS is not able to account for a real railpads nonlinear behaviour.

In order to calibrate a proper railpad stiffness, a practical approach was used by calibrating the railpad stiffness after expected deformations of a real track. Under the circumstances that a railway track is founded on a 75 cm thick ballast bed directly over a bedrock, Bane NOR suggested that a normal railway track being built of a E601 rails and concrete sleepers, should not compress more than 1.0 mm under a 22.5 tonn referance axle.

From this suggestion, a test model being 60 meters long and a 0.75 meter thick was created. The ballast was modelled with a linear elastic soil model and the track system modelled according to chapter 2.2.1. The model used for this calibration is shown in figure 2.12.

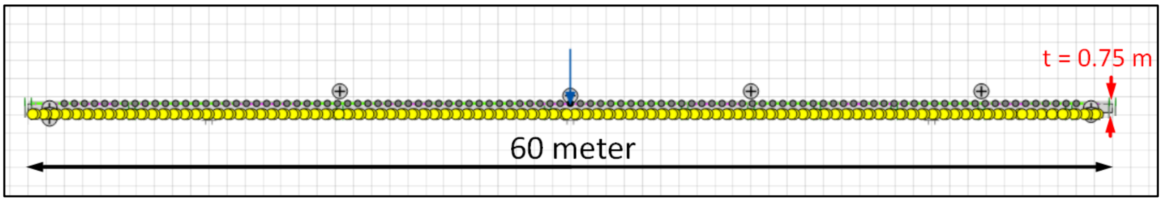


Figure 2.12 – Calibration model for the railpad, where the maximum acceptable limit was set to 1.0 mm

At this point there was two variables in the system, the ballast and the railpad. According to some authors, the ballasts elastic stiffness may range in between 100-200 MPa [26,27]. Since the soil body in this model was so thin, the influence ballast would have on track displacement compared to the railpad was insignificant, and an elastic modulus of 180 MPa was selected for this calibration.

After transforming the track components into plane-strain, the test showed that a railpad stiffness of 225 kN/m was necessary in order for the track to not displace more than 1.0 mm, see figure 2.13.

This railpad stiffness was considered necessary in order to account for the railpads nonlinear behaviour, and is for this track system considered to be valid for axle loads up to 22.5 tonn. More information on this calibration example are provided in chapter 2.3.1, appendix A.

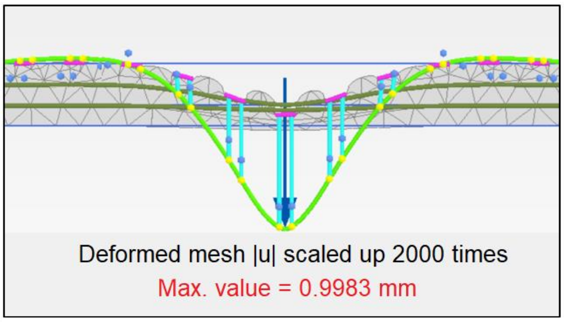


Figure 2.13 – Deformed mesh showing rail deflections for a 22.5 tonn referance axle with a pad stiffness of 225 kN/m

2.2.3 Ballast stiffness and static behaviour – Beam on elastic foundation

Zimmermanns theory states that a Continously Welded Rail (CWR) with the prerequisite of having a constant bending stiffness founded on a foundation with constant stiffness pr. sleeper, can be modelled as an Euler-Bernullis beam founded on a continous Winkler springbed, see figure 2.14.

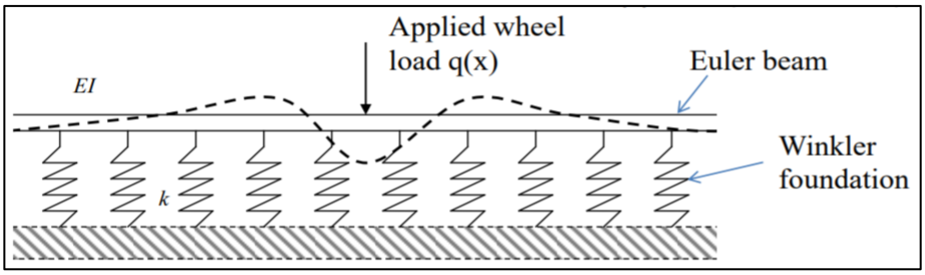


Figure 2.14 - Principle sketch: The main principles behind Zimmermanns theory, a continuously welded rail (CWR) as an Euler-bernullis beam founded on a winkler springbed

When the beam in Zimmermanns model are being subjected to an external point load, it will deflect with a characteristic shape as long as the rails bending stiffness and spring stiffness of the foundation remains constant. Despite this theory being simple, it is still considered valid by several authors for quick and easy static calculations of moments and deflections [25,26,28]. This theory was used for validating the tracks static behaviour in PLAXIS.

Zimmermanns theory states that the reaction forces from any point load is governed by the characteristic lenght of the track system. This quantity is governed by the bending stiffness of the rail and the foundation modulus (i.e the spring stiffness), and is formulated as

$$L_c = \sqrt[4]{\frac{4*EI}{k}} \qquad \text{(Eq. 2.2)}$$

where

L being the characteristic lenght
EI being the beams bending stiffness
k being the foundation modulus

To account for the discrete support of the sleepers on a real railway track (i.e sleeper spacing), Zimmermann introduced another formulation for the foundation modulus, expressed as

$$k = \frac{C*As}{a} \qquad \text{(Eq. 2.3)}$$

where

C being the ballast coefficient
A_s being the bottom contact area of the sleeper
a being the sleeper spacing

From solving the differential equation for a beam on elastic foundation, the following characteristic equations for moment and deflection are obtained

$$y = \frac{Q}{2*k*L_c} e^{-x/Lc} (\sin(x/L_c) + \cos(x/L_c)) = \frac{Q}{2*kL} * \eta(x/L_c) \qquad \text{(Eq. 2.4)}$$

$$M = \frac{Q*L_c}{4} e^{-x/Lc}(-\sin(x/L_c) + \cos(x/L_c)) = \frac{QL}{4} * \mu(x/L_c) \qquad \text{(Eq. 2.5)}$$

where

x being the distance from the applied point load
L_c being the characteristic lenght
η(x/L) being the deflection (y) at any given point (x), from the applied point load Q
μ(x/L) being the bending moment (M) at any given point (x), from the applied point load Q

Additional information on deriving these equations are given in chapter 2.3, appendix A.

Based of Zimmermanns formulation (Eq. 2.4 and 2.5), characteristic curves for distributions of bending moments and deflections along a rail at the applied point load (x = 0) can be obtained on the basis of the systems characteristic lenght, see figure 2.15.

Assuming a constant foundation modulus and bending stiffness of the rail, the theory shows that it may be is expected an uplift of 20% for the bending moments and 5% for the deflections. Only the characteristics of $\eta(x/L_c)$ and $\mu(x/L_c)$ are shown in these plots, which according to this theory is constant for any railway track.

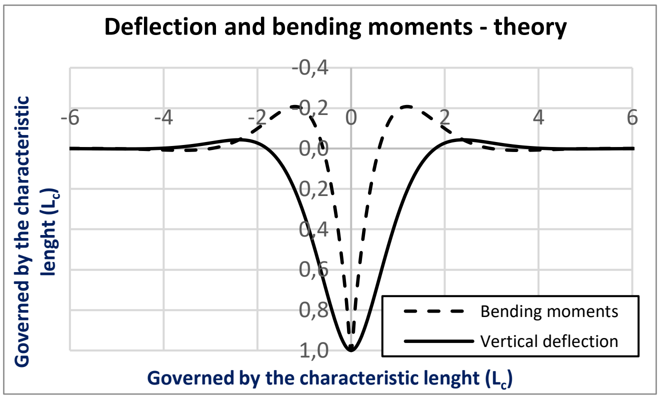


Figure 2.15 - The characteristic distribution of the rails deflection and bending moment when a point load is being applied (According to Zimmermanns theory)

Neglecting the magnitude of the point-load and thinking in terms of characteristic distributions, it can be seen from the plot that both the spread (x-axis) and magnitude (y-axis) are governed by the characteristic lenght, i.e the relationship between the bending stiffness (EI) and the foundation modulus (k).

In order to calibrate the ballast stiffness for PLAXIS, ballast coefficients based of experience numbers was used as an estimation of deformations according to Zimmermanns theory. Ranging a railway track from poor to good, typical ballast coefficients according to Esveld. C [26] are shown in table 2.1

Table 2.1 - Purposed ballast coefficients based of track quality (From Esveld. C, 2016)

Classification	E_{v2} [N/mm²]	C [N/mm³]
Poor	10	0.03
	20	0.04
Moderate	50	0.07
Good	80	0.09
	100	0.11

By using a ballast coefficient of a moderate track, sleeper spacing of 0.6 meter and an average width of 0.25 meter, the foundation modulus could be calculated after equation 2.3. From Zimmermanns theory, this led to a maximum deflection in an order of 1.6 mm with a maximum bending moment of 25 kNm/m under a 22.5 tonn referance axle.

To match the prerequisites of this theory as close as possible, the track model for these simulations was created 40 meters long and 7 meters deep, using only a linear elastic soil body. The weight of the rail and sleepers was also set to zero as the weight of these components are not accounted for in the theory.

In Esvelds formulations of the characteristic lenght [26], it is not specified if the railpad is included in a ballast coefficient or not. This led to two calibration simulations being performed, one without (pads axial stiffness was set to «infinite»), and one with a railpad stiffness of 225 kN/m, see figure 2.16.

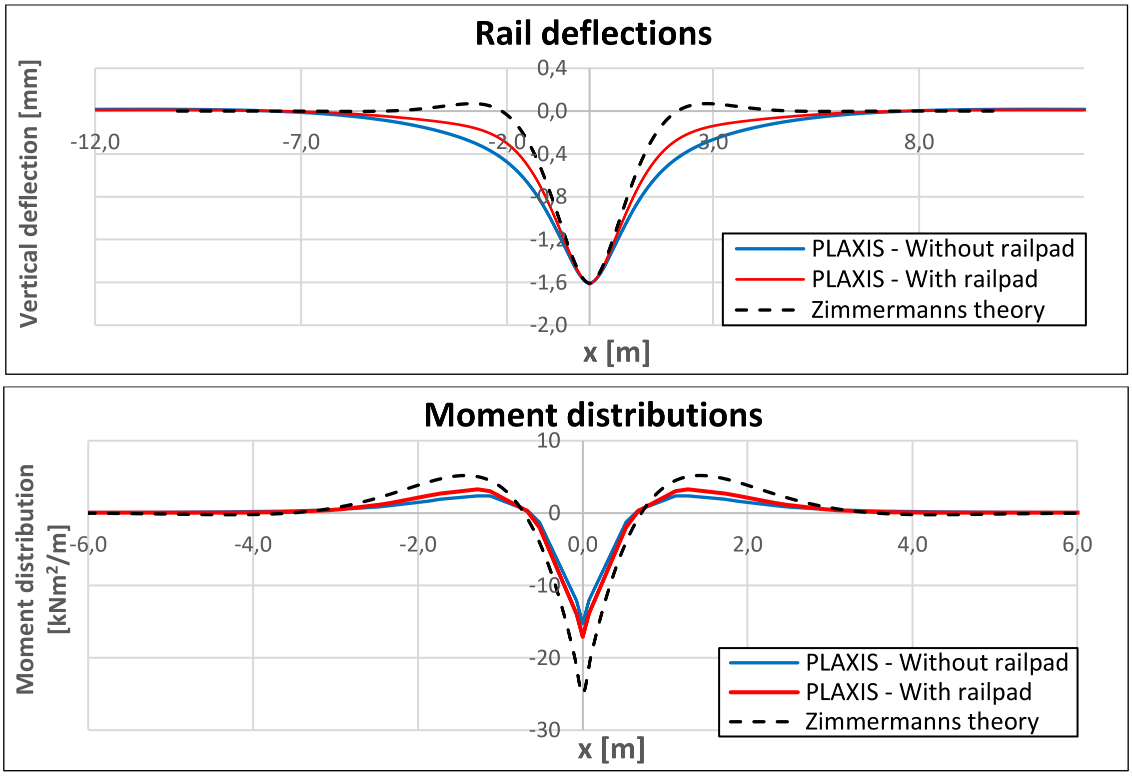


*Figure 2.16 - Comparison between Zimmermanns theory and PLAXIS 2D for deflection and bending moment distributionwhen using a ballast coefficient of 70*10E6*

From these calibrations it was found that without a railpad, an elastic modulus of 86 MPa was required as an elastic stiffness in order to match 1.6 mm deflections purposed by Zimmermanns theory, while 138 MPa for the simulation with a railpad (i.e axial stiffness set to 225 kN/m).

Judging from the characteristic shape of the deformations, it is seen that including a railpad with compressive properties, makes the distribution more concave compared to excluding it. Having a railpad also matches the downward deflection pattern from theory better than excluding it. For both similations it can be seen that the deformation pattern doesn't match the concave uplifting part from theory however. The nonuplifting behaviour in PLAXIS is thought of as more realistic than the behaviour purposed by theory based of how the sleepers are supported in PLAXIS.

The sleepers in PLAXIS supports the rail disceretely, while the theory assumes that the entire rail is supported by a continous elastic spring bed. From equation 2.3 however, it can be seen that the spring stiffness is also expressed by the sleeper spacing. It is important to be aware of the presumptions of this theory (eq. 2.4-2.5), in fact is based on a continous, and not discrete supports despite the foundation modulus also being expressed with the sleeper spacing. Even as the downward displacements from PLAXIS matches theory, this causes a difference in reaction forces under the rail where Winklers formulation leads to downward reaction forces at a certain distance from the load in order to hold the rail at its place, which to such extent is not expected to occour on a real track.

From the moment distributions it is seen that there is some deviations when comparing magni-tudes to theory. The spread of the moment distributions however, doesn't deviate that much which suggests that the characteristic lenghts of these systems are within the same range.

From these simulations it was concluded that a elastic modulus of 138 MPa is more realistic than 86 MPa as an imput for ballast stiffness, and including railpad shows a more realistic behaviour staticly, compared to excluding it. As for the uplifitng part at the displacements, it was also concluded that the nonuplifting behaviour in PLAXIS is more realistic than the uplifting behaviour suggested from theory based of the discrete spacing of the sleepers on a real track. Despite Zimmermanns formultion of the foundation modulus (Eq. 2.3) this is not accounted for in the theory in terms of reaction forces.

A more detailed description of this simulation can be found in chapter 2.3.2, Appendix A.

2.3 Moving train loads, dynamic behaviour

This section highlights the methodology and findings from chapter 3 in Appendix A. A introduction for how to model a horisontally moving point load are first presented, followed by using this principle for modelling two typical trains seen at the norwegian railways.

When running a completely undamped train simulation there has been a tendency of occouring a dominant frequency which always seemed to excite the displacement of the entire model. From Fast Fourier Transformation (FFT), was it possible to locate the source of this excitement, which made it possible to remove the unwanted noise without misusing the rayleight damping factors and ruining the dynamics.

2.3.1 Two common norwegian trains

In PLAXIS 2D, there currently does not exist any option for creating a horisontal moving point load. By using the multiplier functions in the programme however, it is possible to artificially model such behaviour with point loads. A multiplier determines how much of a point load that is acting at any given time which can be used to artifically simulate a point load that is moving horisontally at a certain speed.

As a function of the moving speed of the load and knowing the distance between a set of point loads, catching the behaviour of a horisontally moving point load is managable by calculating a certain fading time between them. By ensuring that the «moving load» always has the multipliers sum equal to 1.0, this fading manipulates the model into making the load behave as if it was moving horisontally.

As a principal example, this can be demonstrated with a section of 50 point loads, see figure 2.17. If we only consider the front axle of the «train» (A_1) which has been defined with a constant speed and by knowing the distance between the first and second pointload (P_1 and P_2), a fading time can be calculated for the load. This fading time is governed by the multiplier function which determines how much the load is working as a function of time. If the multiplier gradually increases linearly with 10% in 0.01 second segments, it will take the load 0.1 seconds to work with full force at the first point load.

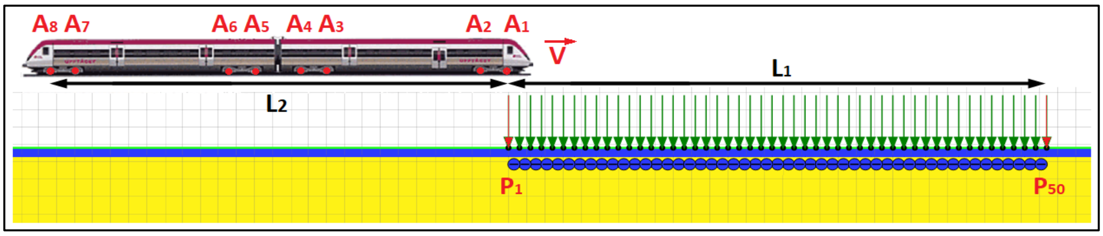


Figure 2.17 - The properties which should be known to model a set of moving train loads in PLAXIS 2D, showing the section which is to be studied (L_1) and a train with total eight axles

To get this load to move to the next point load (P_2), the same fading time is used to «fade out» from the first point load with a time segment of 0.01 seconds. This means that the load goes from working fully (i.e multiplier is 1.0), then gradually decreases linearly with 10% in 0.01 second time segments. Most importantly, in order catch the behaviour of the load moving continously with a constant force over to the next point load, when the multiplier of the first point load starts to «fade out» the multiplier of the second point load starts to «fade in», keeping the sum of the point load equal to 1.0 at all times.

What is really convenient with this method is that it allows for modelling a set of train loads from only knowing the trains traveling speed, axle load and axle spacing. By knowing the distance between each axle and assuming that the train travels at constant speed, it is only necessary to delay the «second axle» with a certain time to then use the exact same principles as for the first load. The result will be as if several point loads were traveling with a fixed distance i.e «axle spacing» with a constant force at a certain speed. For more details on this method, a comprehensive example of doing this with the configurations of a X52 commuter train is presented chapter 3.1 in Appendix A.

For this project two common norwegian trains were modelled with this principle, that is A) the BM71 passenger train and B) a freight train consisting of a CE119 front locomotive with two SNGS wagons, see figure 2.18.

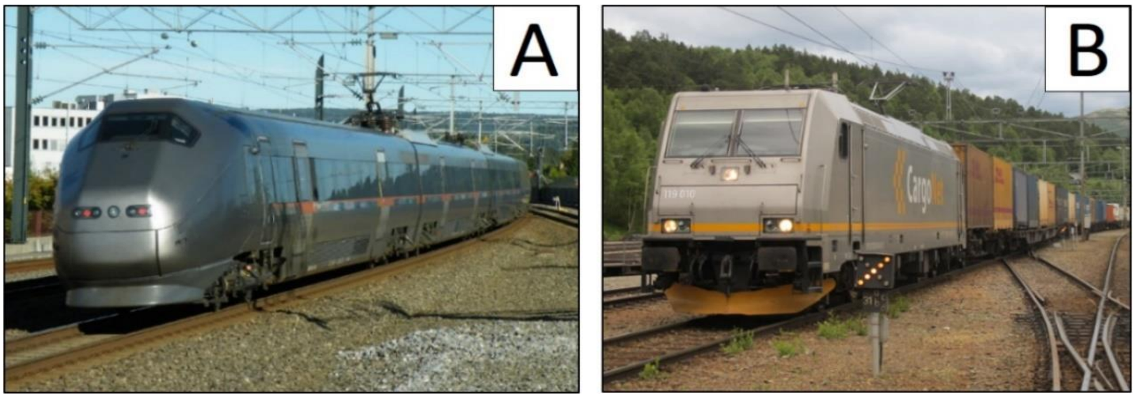
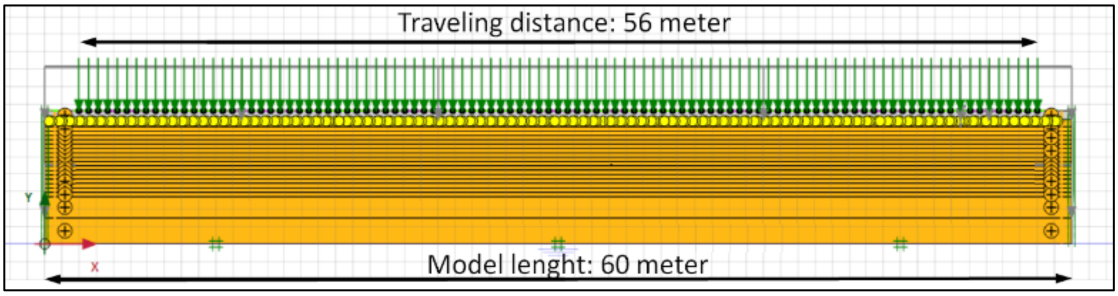


Figure 2.18 - Illustration of two common trains in Norway: A) The norwegian airport express train BM71 and B) A freight train from Cargonet consisting of an electric CE119 veichle in front (Photo: Stig Baumeyer and Andreas B. Westheimer)

For the BM 71 configuration a train speed of 56 m/s (201.6 km/h) was selected, and 28 m/s for the freight train. The traveling distance in the model was set to 56 meter with 101 point loads. This means that the distance between each point load is 0.56 meter, which is the traveling distance used for all the simulations performed with these train configurations. Exluding the «fade in» and «fade out» time of the first and last point load, it will take the front axle of the passenger train 1.0 second, while 2.0 seconds for front axle of the freight train to pass the entire model respectively, see figure 2.19.



Figur 2.19 - The generalized track section, showing the traveling distance of 56 meter which is used for all simulations

For the passenger train, a referance axle of 18 tonn is defined based of recommendations from the technical design basis from the InterCity project [23]. The passenger train consist of three veichles, resulting in a total train lenght of 74.52 meter. This means that it will require all the bogies of this «passenger train» 2.351 seconds to pass the entire model when including 0.01 seconds of fading time for the first and last point load.

In figure 2.20, train configuration of the BM71 train is presented, showing the load configurations speed, spacing and magnitude. In addition, the multiplier from PLAXIS of the first point load is illustrated, showing that the whole segment takes 2.351 seconds.

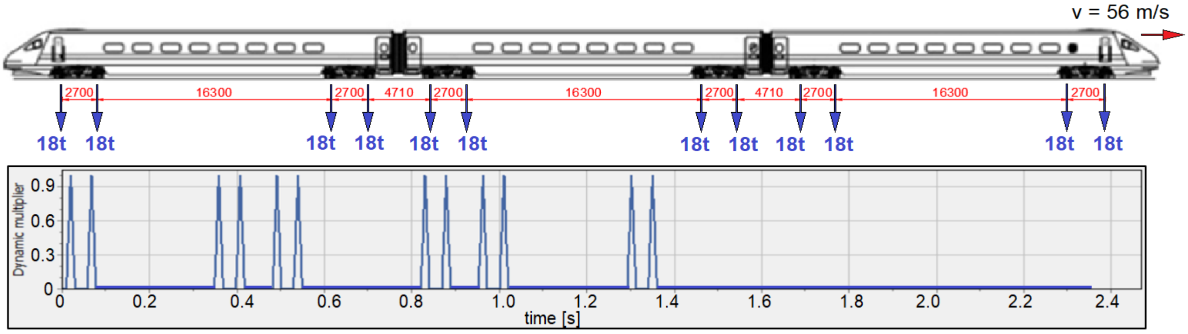


Figure 2.20 - Train specifications based of a BM type 71 train, showing its speed, axle configurations, and time increments based of the multiplier for the first point load in PLAXIS (After www.wikipedia.org)

For the freight train, it was desired to use the maximum allowable axle load for a traveling velocity of 100 km/h, which is 22.5 tonn according to the codes. The front veichle is defined based of the Bombardier Traxx type CE119, commonly used by the Cargonet on the norwegian railways. The CE119 veichle has according to the manufacturer an axle load of 21 tonn. The wagons are based of the type SNGS, which is approved for 22.5 axle loads. In total two wagons were set for this train configuration.

The load configuration and multiplier for the first point load of this train is presented in figure 2.21, showing that it has a total lenght of 53.41 meter. The speed was set to 28 m/s (100.8 km/h), meaning that it will take the whole segment 3.948 seconds (including fading). More information on these trains are provided in chapter 3.2.1-3.2.2, Appendix A.

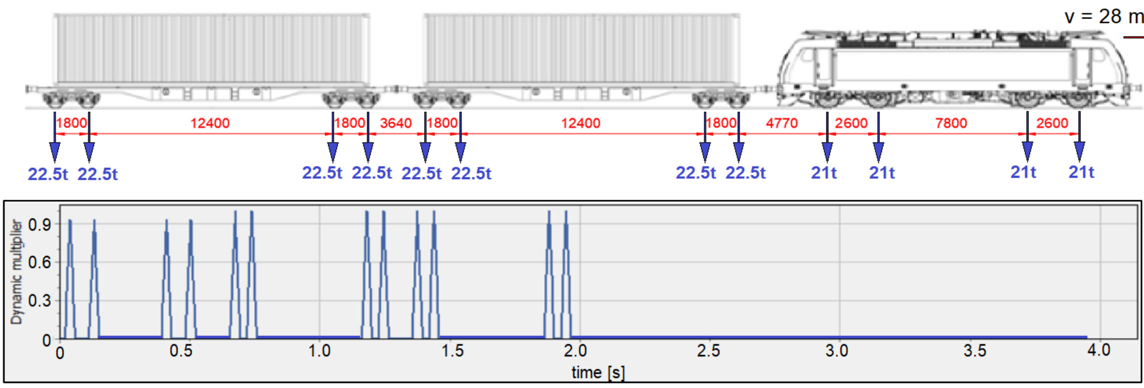


Figure 2.21 - Train specifications of a CE 119 locomotive followed by two SNGS wagons, showing its speed, axle configurations, and time increments for the first point load in PLAXIS (After www.greencargo.com and www.wikipedia.com)

2.3.2 Frequency domains

This section higlights findings from investigating the frequency domains of this model with Fast Fourier Transformations, as there had been occouring a persistent frequency domain in all the undamped simulations which excites the displacements. When thinking in terms of the quasi-static aspect of this system, one could for example damp away all accelerations by using high Rayleigh damping factors in all soil materials to get rid of this problem, but it has been a priority to avoid such app-roach as it has been a desire to enable the use of this method for external users to other dynamic problems.

When running a simulation with a full train configuration, it was found that the effect of several train loads entering the model tended to «mask» the real problem. It was therefore created a single moving point load in a «test model» for investigating the frequency domains. Only frequencies are discussed in this section and more information on the point load test can be found in chapter 3.5, appendix A.

In figure 2.22 the displacements at the rail, approximately 6 meters from the left boundary into the model are presented. It shows that after the load hits the node seen at the maximum displacement, the node keeps vibrating with 12-15 Hz. Only using one point load revealed that this frequency range was nearly constant, (i.e independent of train speed) and seem to disipate after its first oscillation.

In order to visualize the frequencies of this simulation, vertical displacements in time domain was Fast Fourier Transformed (FFT) to frequency domains. The FFT code was written by Albert Lau at NTNU.

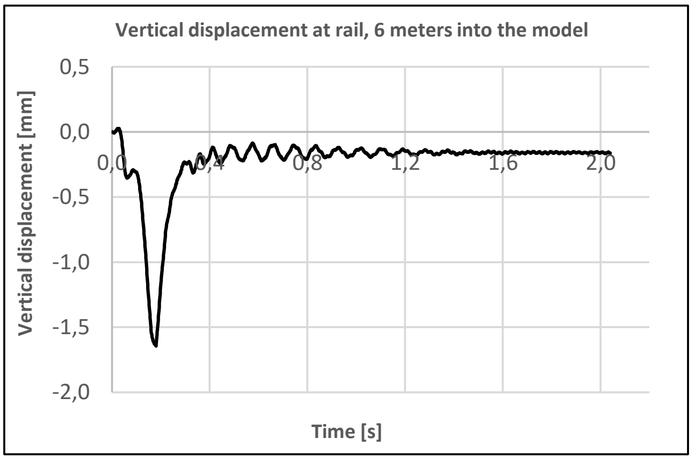


Figure 2.22 – Vertical displacement of the rail at the free track in a hardening soil model, showing that the system vibrates with a frequency of about 12-15 Hz

Two nodes was placed in the middle of the same model (approximately at x = 30 meter, corresponding to fig. 2.19), where one node was place above a sleeper and a second one in between. The displacements in time domain was then FFT to frequency domains and scaled up logaritmically. Several frequencies at the track was also located, for example the sleeper passing frequency, but this section only highlights the unrealistic freqencies associated with this model, shown in figure 2.23.

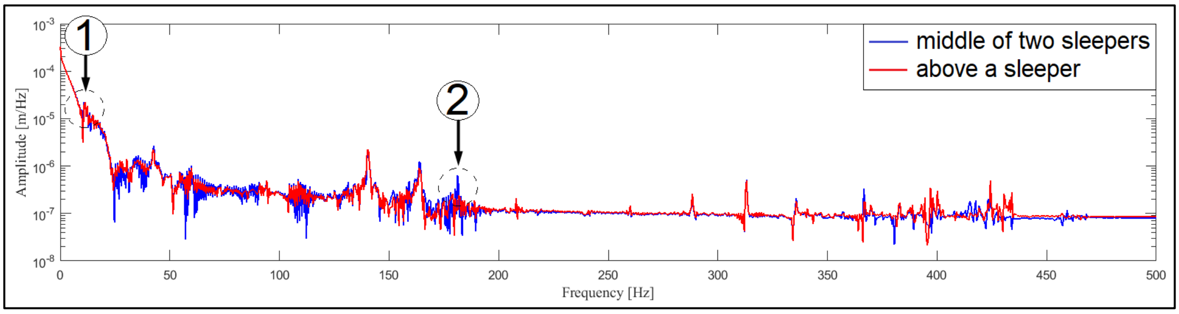


Figure 2.23 - Fast Fourier Transformed (FFT) results for a completely undamped case in frequency domain, A) when the point load is traveling at 28 m/s (The Matlab code was written by Albert Lau, NTNU)

The FFT analysis revealed that there is two frequencies in this system that should not be there, denoted as ① and ② in the figure. The first one ① was located from doing estimations of the fundamental frequencies of the soil, where the fundamental frequencies associated with shear and compression waves created by the train loads to the soil, matched suprizingly well to the frequency seen at the displacements from the FFT analysis.

The FFT analysis also revealed that the fact that the railpad is modelled as two vertical plate elements pr. sleeper, it also creates an unrealistic frequency denoted as ② in the figure 2.23. The amplitude of this frequency is relatively small compared to the frequency of the soil, but should be taken into consideration if using this track model for dynamic analyses.

According to Nordal and Kramer [29,30], based of the first mode in an oscillating soil system, the fundamental frequency of compression and shear waves can be expressed in a soil medium from the first mode of the oscillation. This mode has the characteristic oscillation as shown in figure 2.24 (i.e when n = 1).

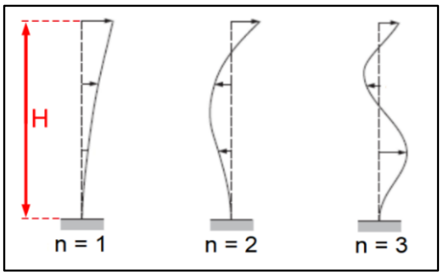


Figure 2.24 – Eigen modes of the first three modes of an oscillation in a soil layer (Modified after Nordal. S, 2017)

Based of Nordals and Kramers formulation, it was found from hand calculations that the frequency of the compression waves oscillating from the surface down to the bedrock was about 15 Hz, while about 9 Hz for the shear compression waves. This matches suprizingly well to the freqency of the oscillation seen in figure 2.22.

The second frequency which was found from the FFT analysis (denoted as ② in figure 2.23), was located at the track with using simple beam theory. By assuming that the track system consist of several simple supported beams, the different passing frequencies of any moving point load could be located, see figure 2.25.

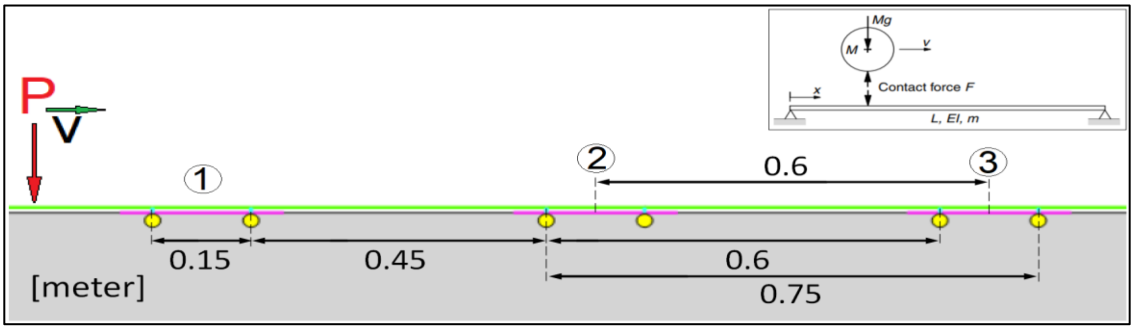


Figure 2.25 -Basis for investigating frequencies occouring at the track from the moving point load test and assuming a system of several simple supported beams

From knowing the velocity of the load, the at this track frequencies can according to some authors [26,28] be calculated as

$$f = \frac{v}{\lambda} \tag{Eq. 2.6}$$

where

f being the frequency
v being the wave speed
λ being the wave lenght

When analyzing a point load traveling at 28 m/s, the sleeper passing frequency (0.6 meter), pad to pad passing frequency between two sleepers (0.45 meter) and pad to pad passing frequency at each sleeper (0.15 meter) was located. Whats interesting here is that the FFT results revealed that some excitement in fact also is created by the pad to pad passing frequency at each sleeper (denoted as ② in figure 2.25), a unrealistic frequency which is caused by the pad being modelled as two vertical plate elements.

This excitement compared to the excitement caused by the soil oscillation is however neglectible in respect to the models quasi-static behaviour. This can be seen from the logaritmic scale in the FFT analysis (fig. 2.23), were the excitement at the track is only 1/100 – 1/1000 of the excitement created by the «pulse» seen in figure 2.22. It should be noted that the pad to pad passing frequency (with the spacing of 0.15m) should be taken into consideration if this track model is used for other dynamic problems.

With these findings it was possible to locate the vibrations seen at the displacements. When the point load «fades» into the model, the load acts with full force in a matter of one thousand of a second. This creates a pulse which propagates up and down in the soil bodies of the model, causing the entire system to vibrate. Similar is seen when the point load is leaving the model, which creates a similar oscillation.

It was therefore concluded that when a full train configuration is used, each time a load enters or leaves the model, it creates a «pulse». Knowing that the trains from section 2.3.1 is within the same lenght as the entire model (50-70 meter), a series of pulses are created continously during the simulation. This causes vibrations during the entire simulation which masks that these oscillation in fact were disipating. More assessments on frequency domains of the model are given in chapter 3.5.1-3.5.2, Appendix A.

2.3.3 Rayleigh damping – Only damping the end boundaries

One way to get rid of the dominant frequencies caused by this «pulse», is to only damp a certain section at the end boundaries. The solution purposed here damps three zones, gradually going from full damping (zone #1) to no damping (zone #4), see figure 2.26.

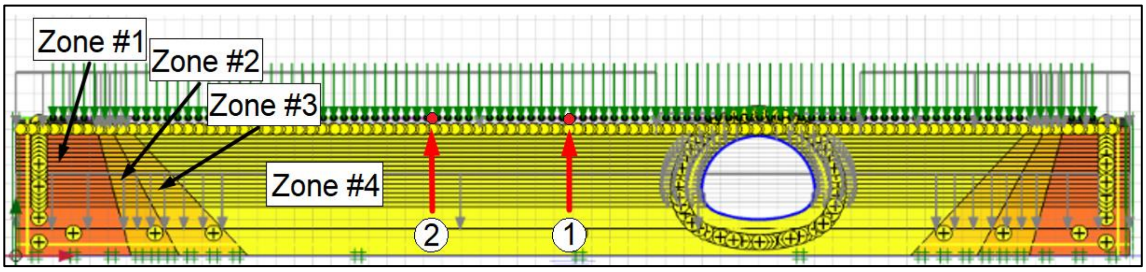


Figure 2.26 - Principles of damping the entry and exit of the model. Most of the damping is placed directly under the first point loads (zone #1) at the end boundaries of the model, gradually decreasing to no damping after leaving zone #3

In figure 2.26 two plots are presented, the first plot ① shows the displacements from a completely undamped train simulation is selected at about 27.9 meter into the model. The second one ② shows the displacements from only damping the end boundaries were the node were placed at about 20.1 meter.

In these simulations only linear-elastic soil models were used with the configurations of the freight train, see section 2.3.1. For the damped case, a frequency range of 5-50 Hz was damped with 400% in zone #1, 200% in zone #2 and 100 % in zone #3 respectively. Vertical displacements for the rail at node ① and ② is presented in figure 2.27.

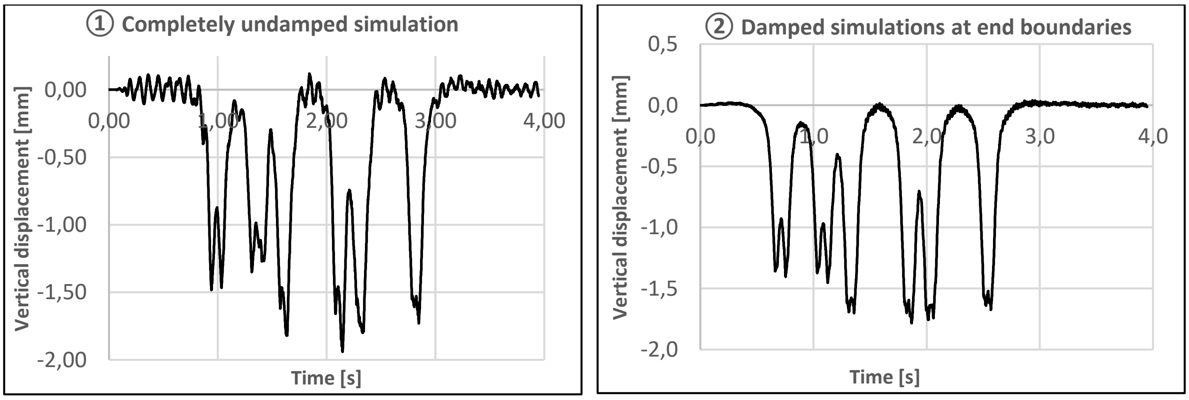


Figure 2.27 – Comparison between A) no damping and B) damping the end boundaries during the passage of a «freight train»

What these plots shows is the full passage of the «freight train» described in section 2.3.1. When the «train» closes in on a node, it can be seen that the rail is slightly lifted upwards (best seen in the damped case). The rail is then pushed down again and displaces with an order of 1.4 mm (at about 0.6 seconds) as the first bogie of the train hits this node. Both axles of the bogie can be seen by the short peaks of the displacements at 0.6-0.8 seconds.

Before the rail enables to go back to its original position, the second axle of the front bogie pushes the rail down again (at 0.8 seconds). The rail then starts to move upwards between the bogie spacing of the train loads for then to be pushed down again by the second bogie from the veichle. The difference in axle load of the front veichle compared to the wagons can also be seen as 21 tonn was used for the front veichle and 22.5 tonn for the wagons, evidently causing larger displacements.

In terms of vibrations, the undamped case ① shows that the vibrations discussed in section 2.3.2 is clearly excititing the displacements. These vibrations however, doesn't seem to gradually decrease as was seen in the simulation of the single moving point load. This suggests some kind of «pumping effect» when these point loads are entering and leaving the model, continously creating pulses which propagates through the model. Comparing the displacements of case ① to the damped case ②, it can be seen that the vibrations also tends to supinate the displacements, underlining the need to remove them.

This solution has demonstrated that when damping the end boundaries of the model, a clear improvement is found. The displacements at the rail is now more clear and consistent. Whats convenient with solving the problem this way is that no damping needs to be used in the section of study, opening up for the possibilty to use this solution for similar types of dynamic problems. For more background information on the using Rayleigh damping factors, see chapter 3.5 in Appendix A.

A important discovery in the later stages of this project has been that this solution only works in elastic soil models, while using this approach for a Hardening Soil model for example causes the displacements to plastify persistently independent of train passages. It was possible to work around this problem, which are the topic of the next section.

2.3.4 Problems with moving loads in Hardening Soil models

Whenever using a Hardening Soil model and only damping the end boundaries (i.e the section at the culvert remains undamped), it was found that the soil model doesn't plastify after any number of load cycles, resulting in persistant plastification regardless of the previous number of simulations that has been performed. This plastification first shows a tendency of convergating the first 1-5 simulations, but then improves less and less for each simulation. An example of this problem is shown in figure 2.28.

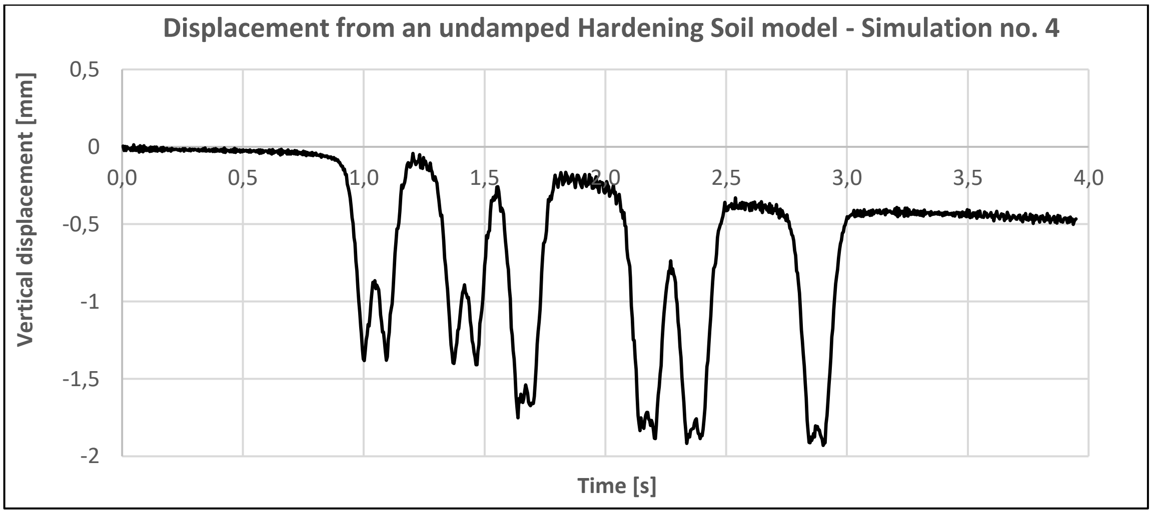


Figure 2.28 – An undamped train simulation when using Hardening Soil for the backfill, showing the persistant plastification seen at the displacements whenever a load is passing the node. The figure is taken from chapter 3.6 in Appendix A

The plot shows that each time a train load is passing the node, the undamped Hardening Soil materials plastifies slightly more for each time and creates a «stair» at the displacements. This stair gradually increases its peak in the displacement for each passing load.

It was first found that the easiest way to get rid of this problem without ruining the model, was to use Rayleigh damping in the materials that involved Hardening Soil. Using around 50% damping seemed to get rid of the majority of this problem without affecting the displacements to much. In appendix A, chapter 3.6 three comparisons were made between an undamped and a damped hardening soil simulation, but also a undamped linear elastic simulation, see figure 2.29

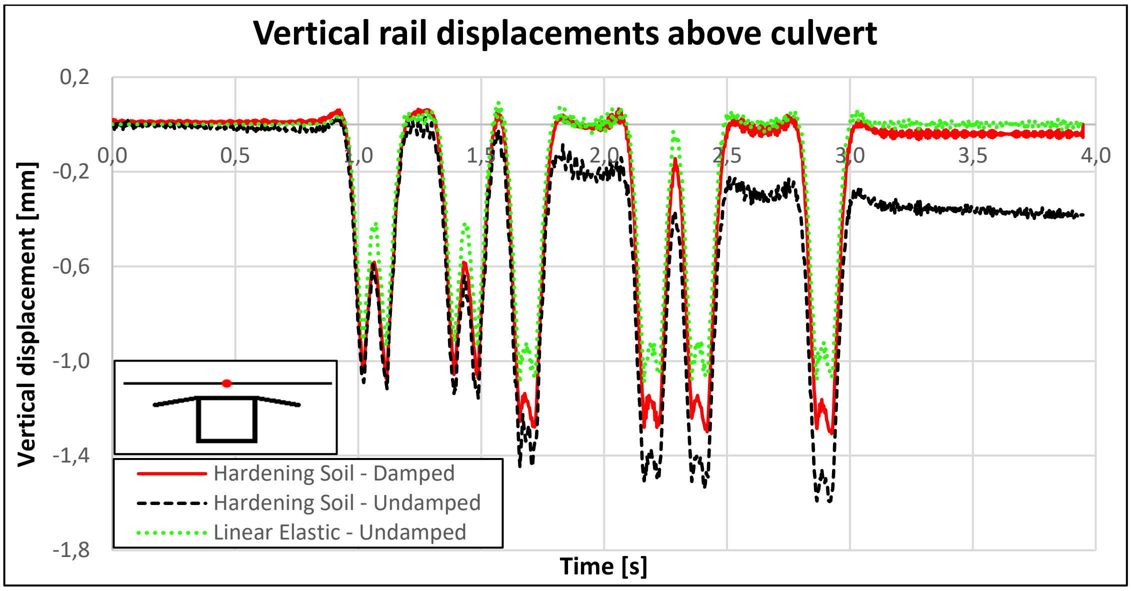


Figure 2.29 – Vertical rail displacement above a concrete culvert for demonstrating the problem in Hardening Soil

The figure shows that the problem is completely removed by introducing damping into the system. The problem of solving the issue this way however, is that it is not quite understood why damping works. Despite this projects focuse being displacements and the quasi-static aspects of transition zones, using a high damping in a problem like this without understanding why it works makes the model less reliable, and was there avoided.

At the latest stages of the project it was found that this problem more likely were related to the Hardening Soil model not being able to take tension forces. A hypotheses is that the elastic ballast layer on top of the fill with Hardening soil extends when the train loads is passing and forces the Hardening Soil material to extend which PLAXIS does not allow it to do. The exact mechanisms involved in this problem is considered complex and was not studied in detail, but another way of solving this while also keeping the model undamped, is to unmark the «Tension cut-off» option in the materials mode which seem to make it easier for the model to convergate the problem away.

Doing this allows the material to take both compression and tension forces which seems to improve this problem without ruining the dynamics of the model. This approach are not discussed in Appendix A because it was discovered at the latest stages of this project, but is considered a better way of solving this problem compared to using high damping.

2.4 Construction of culverts, backfilling and compaction

This chapter presents the methodology for modelling the effect of backfilling and compaction when constructing a culvert in PLAXIS. The level of uncertainty tied to backfilling in field has made it a challenge to create a general procedure which can be applied for both types of railway culverts. From validating the method presented here to several field measurements and after analyzing the various variables involved in the procedures tied to constructing a culvert, it was possible to create a general methodology for backfilling which applies for both type of culvert.

The methodology presented here for backfilling has been considered a necessity in this project in order to catch any realistic behaviour of flexible steel-soil composite bridges as these structures long-term behaviour is mostly governed by the backfilling stages but also in field [13,16,18,21]. From several trials and after comparing to several field measurements was it possible to catch the effect of compaction with sufficient accuracy by using a Hardening Soil model and backfilling in steps.

For concrete culverts no measurements or comparisons could be made to field behaviour. This was expected from the fact that these structures are rigid, massive and self-bearing (i.e independent of the backfill), but catching the effect of compaction underneath the transition slab is still considered important as this may affect the elastic range (i.e behaviour) of the slab in Hardening Soil.

After verifying the methodology presented in this section, it was thought that if it could be proven to be sufficient for cases of flexible SSCB, it could also be used with confidence in cases of rigid concrete culverts based of that the procedures and backfill materials are very similar.

2.4.1 Methodology

In principle, a foundation is first prepared, then the culvert is built followed by alternately backfilling in layers on both sides of the culvert. Only when the backfill reaches the top sections on these culverts is there som differences in overfilling due to construction of the transition slab in the rigid culvert. A more detailed description of the methodology presented here can also be found in chapter 6.2, appendix A.

To begin with, a minimum 3 meter thick foundation is built on top of a bedrock, see figure 2.30. From the figure it can be seen that the material bellow the culvert is separated from material bellow the free embankment (denoted as ① and ②). The reason for doing this was that after running some dynamic simulations, it was found that PLAXIS showed to soft behaviour of the soil underneath the culvert. The soil would typically compress a little more than the adjecent soil outside the foundation of the culvert, which always caused the track above any culvert to displace more to the displacement at the free track.

When comparing to fieldmeasurements on stiff bridges however, they have shown that the stiffness on such bridges normally should be higher than in the free track [3-6], which demonstrates the need to correct it. More on this stiffness problem is covered in chapter 2.5.

To solve this problem, the unloading stiffness of material ① was given twice the unloading stiffness of material ②. Both materials are then compacted in a individual phase by activating a static line load of 50 kN/m, representing a «medium vibro roller». This load is then deactiviated which finishes the layer.

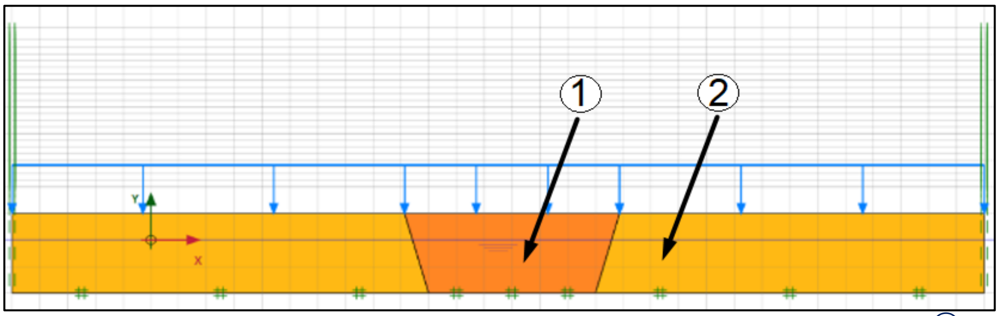


Figure 2.30 – Construction of the foundation, showing how the soil layer beneath the culvert ① is separated from the free embankment ② due to stiffness problems in PLAXIS

From a practical standpoint, it is well known that foundations under culverts is well compacted, typically by using high quality frost protective materials such as crushed rock. It was therefore thought that the foundation under the culvert should act stiffer compared to the foundation at the free embankment. Typical practices for preparing foundations in field can be found in [31], also see chapter 4 in appendix A for construction and backfilling of railway culverts.

After the foundation has been prepared, both culvert types are created simultaneosly in one single phase, see figure 2.31. The only difference here is that the concrete culvert (figure 2.31A), is founded direcly on the foundation and only one backfill material is used adjecent to the culvert. For the flexible case however, a uncompacted bedding ① is constructed simultaneously with the culvert, see figure 2.31B).

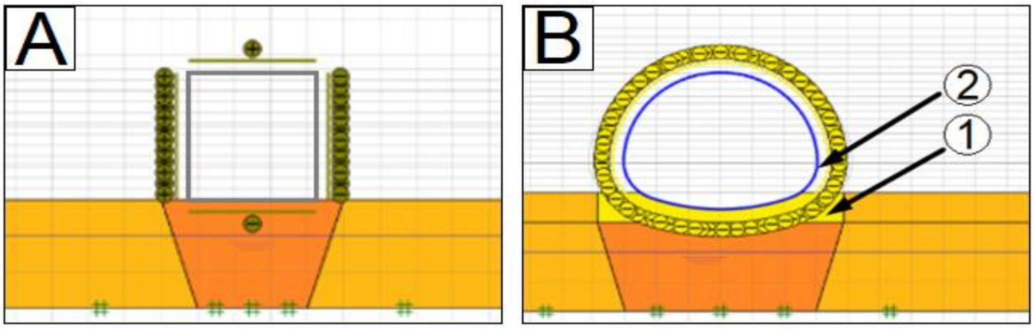


Figure 2.31 – Comparison: Constructing A) a rigid concrete culvert and B) a flexible steel-soil composite culvert

In the SDM-manual it is a minimum requirement to use a finer material for the bedding in a width of 0.5 meter around the entire circumferential cross-section of the culvert («the inner zone»). Further out («the outer zone»), rougher backfill material such as 20/120 mm crushed rock may be used. It should be noted that it is not required to use crushed rock or any different material in the «outer zone» compared to the «inner zone» [13].

This comes from that design codes such as the SDM considers the backfill of an SSCB as a part of the bridge structure, and not as a«regular backfill against a structure» as would be the case for a rigid concrete culvert [32]. In Sweden for example, there has been several cases were the entire backfill of a SSCB has been constructed exclusively with sand [33]. For more on this, see chapter 4.2.5 in Appendix A.

After these culverts has been constructed, the backfilling and compaction of the adjecent soil layers can begin. The backfill material is modelled by using a Hardening Soil model, and built alternately in layers. Each layer is built in one phase, then compacted in a second phase which is then unloaded (from the compaction) in a third phase. This procedure is used for both culvert types.

A standard backfill layer is modelled 0.3 meter thick and the compaction loads are divided into two categories, «light compaction» and a «medium compaction» (labeled as ① and ② in figure 2.32). The sentrifugal forces of the real compaction equipment (i.e in field) are modelled with static line loads. An offseth of 0.1 meter is set for these line loads close to the culvert in order to avoid nummerical problems.

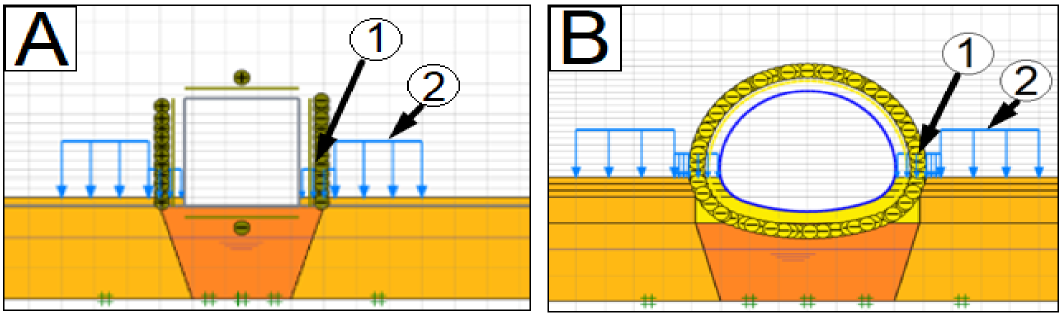


Figure 2.32 – The compaction procedure used for A) a concrete culvert and B) a flexible steel-soil composite culvert showing that the compaction loads are defined as ① «light compaction» and ② «medium compaction»

This generalized compaction scheme (illustrated in figure 2.32), was created on the basis of suggestions from contractors and current regulations [13,17,32]. Also see chapter 4.1.5 and 4.2.5, Appendix A.

Compaction forces, and most importantly in cases of flexible SSCB, makes a considerable difference in terms of the structures final bearing capacity in these models. The compaction loads should as a result be considered a variable. It was hereby thought that all these culverts are always compacted with a light vibroplate the closest meters to the culvert, and with a heavier vibro roller further out, see figure 2.32.

In this project it was found that a static line load of 25 kN/m seemed to correspond well to a light vibro-plate, while 50 kN/m was set as force for the «medium equipment». For more on compaction loads, see chapter 6.2 in Appendix A.

When the backfill has reached the crown level of an SSCB, it is made sure that only light compaction loads are used around and on top of the culverts entire circumferential cross-section. Further out, medium compaction load is used as normal. Geometrically speaking, the light compaction forces applies in a zone of about 1.5 meter around the culvert, while the medium compaction loads in a zone of about 4 meter, see figure 2.33.

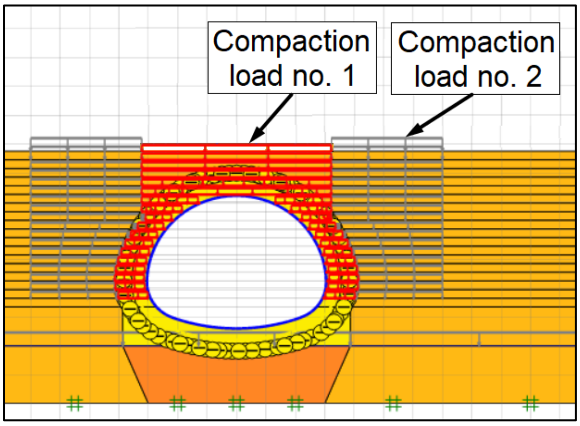


Figure 2.33 – Illustration of a typical load configuration for a flexible steel-soil composite bridge. The compaction procedure is divided into «light» and «medium» compaction force which is used for backfilling and also overfilling

For concrete culverts, when the backfill has reached the slab level a 20 cm thick corrective layer is then placed under the slab, see ① in figure 2.34A).

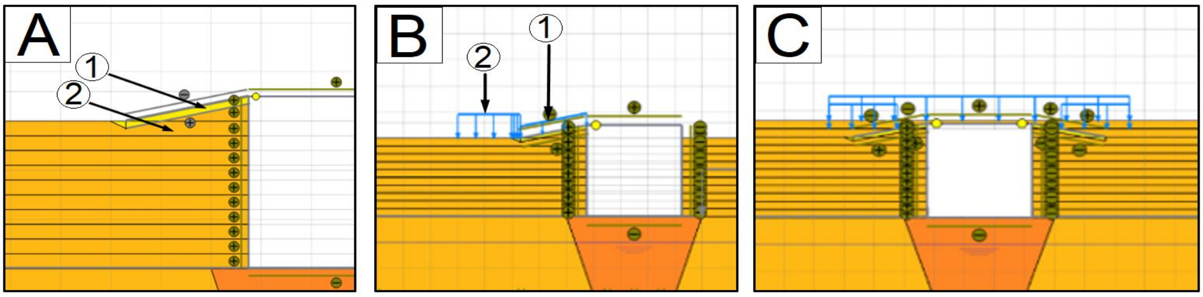


Figure 2.34 – Principles of construction the transition slab: Figure A) shows that a 20 cm corrective layer is built under the entire slab, B) shows how it is being compacted by the weight of the slab and C) overfilled and ccompaction with light equipment

According to contractors, due to the corrective layers steep sloping angle it is very hard to compact it with a convential vibro plate. A typical practice in field has therefore been to use the weight of the slab to compress this layer. Denoted as ① in figure 2.34B), a line load equivalent to the weight of the slab is placed on top of this layer. More information on this can be found in chapter 4.1.6, Appendix A.

After these layers has been constructed, the slab is then built in a individual phase and the overfilling continuous as normal in 0.3 meter thick layers. These layers are compacted similar to the backfill, but now only using light compaction loads with respect to the transition slab, see figure 2.34C).

2.4.2 Defining backfill materials and stiffness parameters

When defining backfill materials it was a desire to define materials that are applicable for both types of culverts. This was in order to limit variables associated with the backfill and rather turn the focuse on how the structures themselves affects the transition zones under a railway track. In the process of defining these materials, a combination of Finite Element (FE) simulations, reported field measurements, experiences from contractors and calculation procedures in design was used and compared.

In general, based of the formulation in the current norwegian regulations, some uncertainty tied backfill materials must be expected. For example, for concrete culverts in Norway it is allowed to use crushed rock with stone sizes up to 150 mm close to the structure, and up to 300 mm further out.

For cases of SSCB, variations in the materials are even more dispersed. Similar to Sweden, there has been several cases in Norway where the entire backfill of an SSCB has been made of only sand, for example Dovre and Tolpinrud [18,21]. In more recent cases, it has been observed incidents were the backfill has consisted of a combination of sand/gravel at the culverts inner zone, but crushed rock further out.

After consulting with contractors and investigating several referance projects, it has been found that a common practice is to use only 20/120 mm crushed rock in the backfill of concrete culverts during short traffic breaks. For flexible SSCB on the other hand, it has often been seen the use of crushed rock in the outer zones and finer material at the culverts inner zone. The finer materials are typical either sand, gravel or 4/16 mm fine crushed rock. Illustrations of two different combinations of backfill materials for flexible culverts can be found in chapter 4.2.5 in Appendix A.

In this thesis it was decided to stick with 20/120 mm crushed rock as the backfill of a concrete culvert, but a combination of sand and 20/120 mm crushed rock as the backfill of SSCB. The sand layer specified for the flexible culvert was also used as the corrective layer under the transition slab. Such material combinations happend to be the case at Kilnes and Solheim culvert from 2016 and 2009, see figure 2.35.

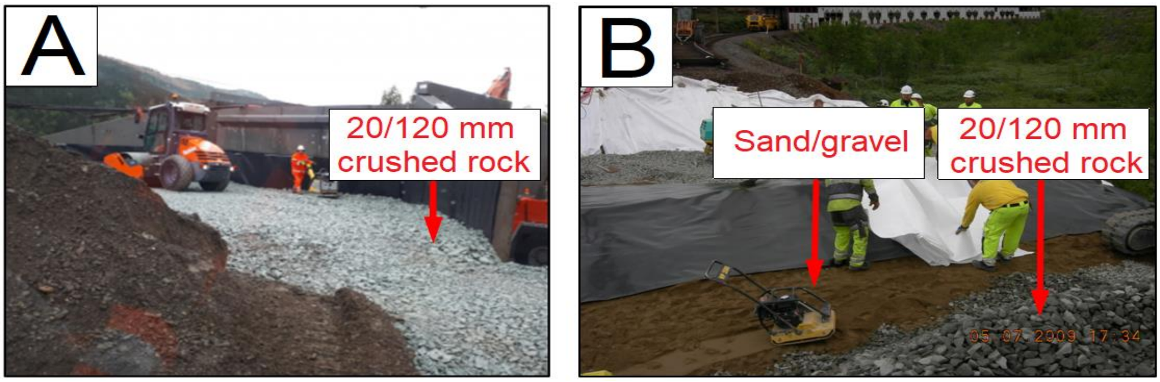


Figure 2.35 – Referance cases: A) Only using 20/120 mm crushed at Kilnes railway culvert in 2016 and B) using a combination of sand and 20/120 mm crushed rock at Solheim railway culvert in 2009 (Source: Farbu & Gausen and ViaCon Norway)

To begin with, when defining the stiffness of this «general sand» several comparisons has been made. According to Nordal [34], most sands shows a referance stiffness of 15-50 MPa in a standard triaxial test. This stiffness range gives a good basis for what ranges to expect for a sand, and is often refered to as the «secant stiffness» or «referance stiffness». This stiffness must not be misinterpreted as the elastic modulus of the material but assuming a possions ratio of 1/3, the elastic modulus of a sand is roughly 1.5 times higher than the secant stiffness. See chapter 6.1.2 in Appendix A.

In one of the validation tests for controlling the backfill methodology presented here, the stiff-ness of the sand used at the Enköping test-culvert was also estimated. The stiffness estimated for this sand were the basis behind this «general» sand defined here. What was really convenient with this field test was that there was alot of background information available on the sand used for that backfill, such as gradua-tion curves, maximum dry density, degree of compaction and even a plate loading test from the site [33].

When defining the stiffness of the Enköping sand, a comparison between Nordals purposed stiffness ranges, the plate loading test and two calculation procedures from the SDM-manual were performed. The calculation procedure used after the SDM-manual was more of an application of the purposed formulations in SDM, but these design procedures purposes a secant stiffness of the backfill which represents the average stiffness of the entire backfill, which were good for comparison.

The first and most simplified formulation in the SDM (denoted as «method A») is based of a formulation developed by Duncan in 1979, and only requires information on the backfills degree of compaction [13,16]. The second method (denoted as «method B»), is according to Petterson. L formulated after Vaslestads arching factor and a statistical reformulation of Janbus classic expression for stress dependent soil modulus [18,33,35]. This reformulation is purposedly based of laboratory work with little to no background information available [13].

The general difference between these two methods is that method A requires the least amount of infor-mation but in return yields the most conservative values. Method B requires more information on variables such as particle size distributions, degree of compaction and stress levels but in return gives higher stiffness values for deign.

In estimating the stiffness of this sand, a deformation modulus based of first time plate load testing was also calculated. Deriving the exact equation for a circular footing resulted in a slighly lower deformation modulus than what was reported by Petterson [33]. All calculations involved for the Enköping sand can be found in chapter 6.3.1 Appendix A.

In table 2.2, results from the various approaches in estimating this sands stiffness is presented.

Table 2.2 – Purposed stiffness parameters showing results from the Swedish Design Manual, the plate load test from field and the purposed stiffness ranges from Nordal

SDM - Method A	SDM - Method B	Field - Plate load test	Nordal's purposed stiffness ranges
12.42 MPa	27.36 MPa	44.73 MPa	15-50 MPa

According to Duncan [16], the values from method A should represent the average stiffness of the entire backfill. This means, after the Enköping sand had been laid out alternately and compacted to 97% standard proctor, the stiffness purposed by this method is barely 12 MPa. Its has been thought in this project that a well compacted sand is probably stiffer than most natural sands, and it was therefore concluded that the stiffness purposed by this method was to conservative to use in any PLAXIS simulations.

The stiffness obtained from method B represents something closer to what one would expect as a referance stiffness for PLAXIS. It can still be argued that 27.5 MPa as a average secant stiffness for a compacted backfill is conservative, but more reasonable to use in design than Method A.

It should be kept in mind that these stiffness quantities represents the stiffness of that sand after it has been exposed to first time loading (i.e compaction). The stiffness ranges purposed by Nordal however represents the stiffness of a virgin sand, i.e the stiffness of the material when it is founded in a neutral, untouched deposite. This means, in some cases, if a sand is founded in a deep soil deposite it may be very dense and show a higher value than this referance range and in other cases, it might be loose and show lower values [34].

Based of Method B and the plate load test, these quantities were thought to represent a range which was more to expect for a sand than method A. It was decided to use a referance stiffness of 30 MPa for the «generalized sand» for PLAXIS and from following the formulations of theory of elasticity, a referance stiffness of 30 MPa is equivalent to the oedometer stiffness (labeled E_{oed}), while the unloading stiffness (labeled E_{ur}) can typically be expected to range 3-5 times higher than the referance stiffness [34].

It was decided to stick to the friction angle obtained from using the empirical expressions of SDM [13] together with a dilatancy angle of 5°. The cohesion was set to 1.0 kPa based of the thought that the compaction procedure in field may create some cohesion in the sand. Finally, the unit weight was set to 20 kN/m^3 and the following parameters were obtained

Table 2.3 – Input parameters for a «generalized sand» for Hardening Soil in PLAXIS

	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	γ [kN/m^3]	φ [°]	ψ [°]	c [KPa]
Sandy gravel	30	30	20	36	5	1

where

E_{50}^{ref} being the secant stiffness from a standard triaxial test, labeled as the referance stiffness
E_{oed}^{ref} being the tangent stiffness for primary oedometer loading
E_{ur}^{ref} being the unloading stiffness
γ being the unit weight of the material
φ being the angle of friction at failure
c being the effective cohesion at failure

For more on the formulations used from SDM, background information or more reflections regarding the considerations made when chosing stiffness parameters, see chapter 6.3.1 in Appendix A. More infor-mation regarding the Hardening soil model is also given in chapter 6.1.2, appendix A.

A challenge with defining the stiffness of a 20/120 mm crushed rock is that little to no accurate re-search exist on this material. The large stone size involved in it makes it nearly impossible to test in a convential triaxial or oedometer apparatus. An approach based of common sense was therefore used.

The referance stiffness and the angle of friction was increased to 45 MPa and 40° since this material is expected to behave stiffer than a sand. It was also assumed that this material held no cohesion, i.e a minimum value of 0.1 kPa was used. Similar to the sand, a unit weight of 20 kN/m^3 was predefined.

It was decided to increase the dilatancy angle of this material to 15°. Increasing the dilatancy angle will make it harder for the material to climb and expand in volume, which simulates a more rigid-like beha-viour. It has been thought that a 20/120 mm crushed rock material behaves very rigid in relation to a small culvert, and over time, little to no movement of the material is expected, see figure 2.36.

Figure 2.36 – Demonstration: Backfilling at Kilnes culvert from 2016, indicating that the material may be expected to be very rigid in relation to the culvert (FOTO: Farbu & Gausen AS)

From these assessments, the following were presumed for the 20/120 mm crushed rock material

Table 2.4 – Input parameters for a Hardening Soil model with the generalized sand

	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	γ [kN/m^3]	φ [°]	ψ [°]	c [KPa]
20/120 mm crushed rock	45	45	20	40	15	0.1

More discussions on chosing stiffness parameters for this material can be found in chapter 6.4, Appendix A. Also see chapter 6.1.2 in the appendix for descriptions of the dilatalcy angle.

A final point regarding stiffness parameters is the unloading stiffness. After 1-2 simulations with the moving train loads (defined in section 2.3) the model will start to behave elastic. This elastic range is governed by the unloading stiffness which controlls the displacement ranges that occours on the surface of the railway track. It can be stated that it is the unloading stiffness that governs the stiffness of the subsoil after the some train loads has plastified the model.

Based of recommendations from Bane NOR, it has been a attempted to calibrate the unloading stiffness in such a way that the vertical displacements of the track would be less than 2.0 mm after the model reaches its elastic range. This was under the presumption of a 22.5 tonn referance axle.

Assuming that the stiffness parameters for all other aspects in the model has been predefined (i.e track, ballast) it has been found that using a unloading stiffness five times the referance stiffness for the crushed rock seemed to match Bane NORs recommendations. After the model behaves elastic, this causes the rail in the model to displace about 1.8 mm under a single static axle load. See chapter 6.4.1 in Appendix A for more on calibrating this quantity.

It has been discovered several challenges with predefining the unloading stiffness of the sand. This prob-lem is also related to the acting stiffness over and under these culverts as the train loads are passing, and is related to the inbuilt formulation of stress dependent stiffness. This aspect are discussed further in chapt. 2.5 where it can be found that it was decided to use a completely different unloading stiffness for the sand under the transition slabs compared to around the flexible culverts in the models elastic range.

Finite element calculations has shown that the surrounding forces acting on a flexible culvert as a result from backfilling is nearly unaffected by changing the unloading stiffness. It is the referance and oedometer stiffness that controlls the soil behaviour at these stages and the unloading stiffness, controlls more the behaviour of the soil when the model reaches its elastic state (i.e hardens).

The unloading stiffness of the sand during backfilling was similar to the crushed rock set to 225 MPa based of the presumption that a densily packed sand could become just as stiff as the crushed rock. But this material is also dispersed into two applications, one for backfilling and one for the elastic behaviour, see chapter 2.5 and 3.1. The following materials are obtained for backfilling, see table 2.5.

Table 2.5 – Stiffness parameters for a standardized sand and 20/120 mm crushed rock used as backfill in PLAXIS

	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	γ [kN/m^3]	φ [°]	ψ [°]	c [KPa]
Sandy gravel	30	30	225	20	36	5	1
20/120 crushed rock	45	45	225	20	40	15	0.1

Chosing the unloading stiffness of the sand at the different stages in the model were not covered in Appendix A, but is discussed further in chapter 2.5 and 3.1 in this report.

2.4.3 Comparing and validating the backfill procedure after two field cases

This chapter highlights and discusses the comparison that was made between the backfill methodology and two field cases, primarily a test culvert from Sweden and Canada [33,36].

The field test from Sweden was the pipe arch «Enköping», and from Canada a horisontal ellipse. At these tests the stepwise deformations of the culverts during backfilling was measured, development of axial forces and internal bending moments were also measured. What was really convenient with comparing these tests is that both cases used a poorly graded sandy gravel (GP-SP) exclusively in the backfill (defined after the Unified Soil Classification system) but having a completely different cross-section, which made them a good comparison.

Enköping was a pipe arch spanning about 6 meters long and was built in the years 1987-1990. The culvert was built with a steel corrugation of MP 200x55 and especially thin steel plates (2.95 mm) to particulary investigate deformations during construction, see figure 2.37A). The results from this field test was a major contributor in development of the Swedish Design Method (SDM) [13,33].

The second validation test (i.e short span horisotal ellipse from Canada) was tested in a trial pit at Queens university, see figure 2.37B). This culvert was built with a steel corrugation of 76.2x25.4 and had a steel thinkness of 1.82 mm [36].

Figure 2.37 – The two field cases used for validating the backfill methodology showing A) «Enköping» test culvert from Sweden and B) The horisontal ellipse tested at Queens university, Canada.

When modelling both cases, the extent of the compaction loads were larger in the models compared to field. This was decided in order catch the effect of the trenches side support when backfilling, see figure 2.37. A section showing both validation models are presented in figure 2.38, where model A) shows the Enköping culvert, and B) the horisontal ellipse in PLAXIS.

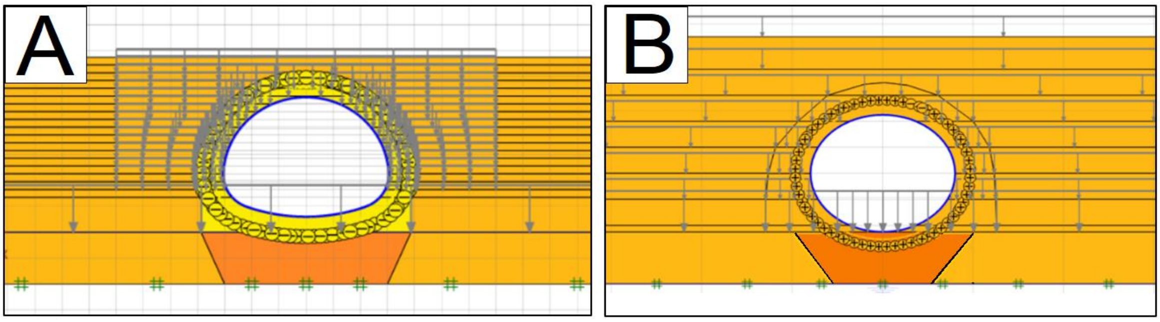


Figure 2.38 – Validations models made for A) the pipe arch at Enköping, Sweden and B) the horisontal ellipse from Canada

As was mentioned in section 2.4.1, the magnitudes of the line loads used as compaction makes a considerable influence on the deformations of flexible culverts during backfilling. According to Petterson. L [33], only small vibroplates weighing about 450 kg was used to compact the backfill at Enköping. According to Moore. I et. al [36], even lighter compaction equipment were used for the case in Canada, a small WP1550AW vibrating plate tamper.

In order to match these conditions, load configuration ① and ② (see figure 2.31) was given the same force. It has generally been found that a static line load of 25 kN/m seem to correspond well to a small vibroplate, or in other words «light compaction». For these two cases, the difference between the small compaction equipment in field were neglected, and 25 kN/m was used as compaction force for both cases. More on this is discussed in chapter 6.3.3, Appendix A.

The general behaviour under these simulations is that if the layers of the backfill are placed with the same thickness as in field, it is the magnitude of the compaction forces that dominates the deformations of the culvert rather than other variables such as stiffness of the backfill material, extent of the line loads or conditions bellow the culvert. But other variables such as the roughness interface (R_{inter} in PLAXIS) or cohesion also affects the deformations of the culvert considerably. More discussions regarding these aspects is covered in chapter 6.3.1, Appendix A.

In figure 2.39, a comparison between the vertical deformation measured at Enöping is compared to three PLAXIS simulations where the stepwise displacement of the crown are compared. These three simulations shows 1) backfilling without using any compaction (layers are only built alternately), 2) only using 25 kN/m as compaction force and 3) using 25 kN/m as compaction force close to the culvert, and 50 kN/m («medium compaction») further out.

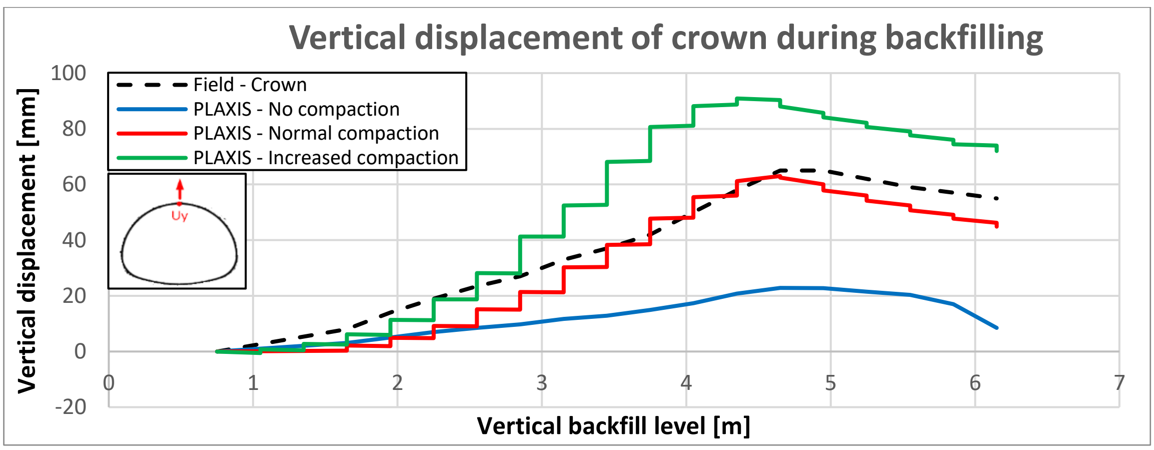


Figure 2.39 – Vertical displacement at crown of Enöping test culvert during backfilling, compared to three loading schemes

These results are presented in such a way that they show two results for vertical displacement pr. backfill level. That is the displacement of the crown when the layer is first constructed (i.e laid out), followed by when the same layer is being unloaded from the compaction force. For the uncompacted case this isn't shown from the fact that these layers were just placed out alternately.

As the figure shows, using no compaction clearly affects the deformation pattern of the crown as the deformations in that case is only a 1/3 of what was measured in field. Introducing «light compaction» to this system matches the field measurements suprizingly well. If the compaction load further out (load ② in figure 2.32) is increased to 50 kN/m however, then the crown of the culvert displaces even more.

It has been found that the deformation patterns of flexible culverts is propotional to the development of internal bending moments during backfilling. This became evident when comparing the stepwise development of internal bending moments from field to two measureing points in PLAXIS, which was at the crown (negative y-axis) and at the left quarter point of the culvert (positive y-axis), see figure 2.40.

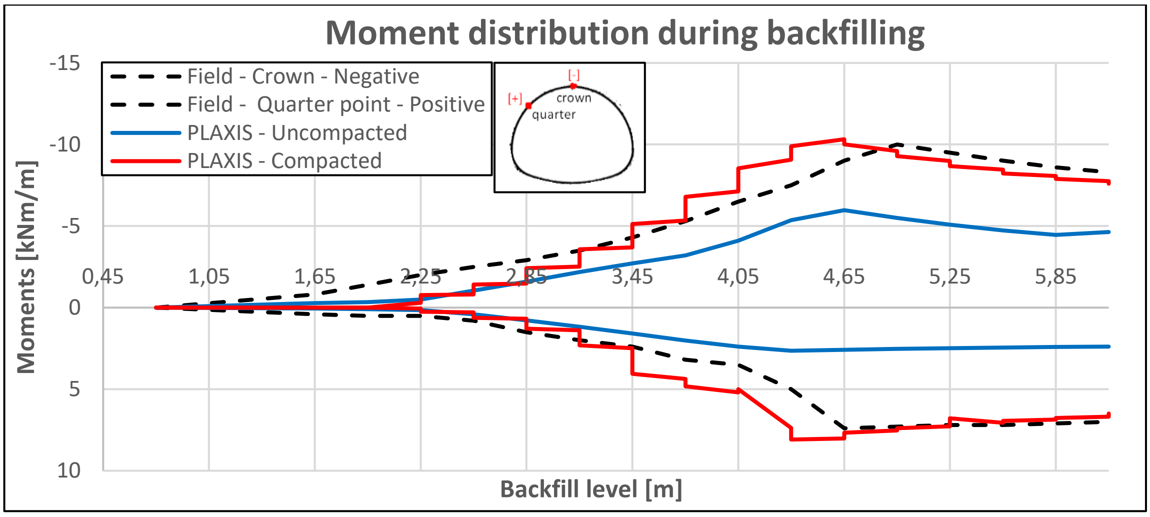


Figure 2.40 - Moment distributions from the crown level and quarter point during backfilling, compared to PLAXIS

These results shows that with a decrease in deformations follows less of a build up in internal bending moments. Another discovery was that when the culvert is being subjected to surface loads after construction finish, the bending moments that occoured from backfilling for the most parts changes sign rather than magnitude. The axial forces however, increases tremendously which suggests that this culvert carries most of the surface load in pure ring compression rather than bending, something which agrees well to field behaviour [13,16,18,33].

As a demonstration, two results from a load test that was carried out on the Enköping culvert is presented in figure 2.41. This test was carried out after the backfilling procedure of «normal compaction» were used, see figure 2.39 and 2.40.

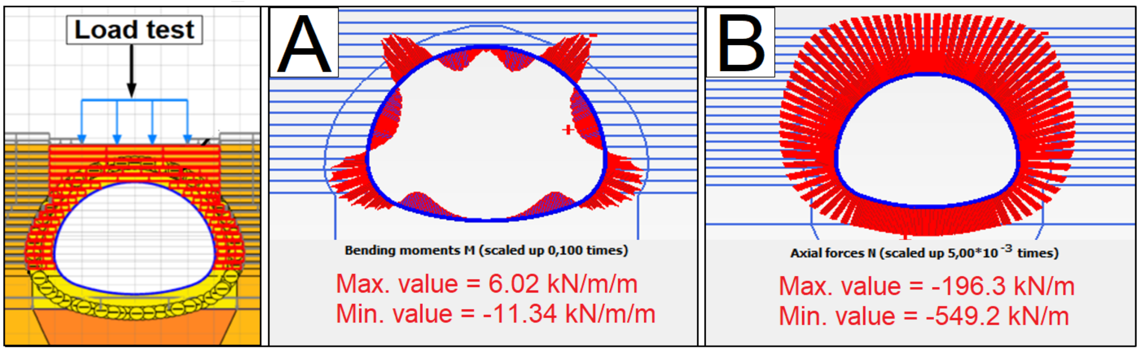


Figure 2.41 – Results from the load test, showing change in A) internal bending moments and B) axial forces (internal thrust) under a 200 kN/m surface load extending 4.0 meter above the center culvert

These results shows the change in internal bending moments and thrust forces when activating a line load with 200 kN/m force on top of the culvert, and compared to the initial axial forces of 122.4 kN/m the increase in thrust forces has increased with over 400% while the bending moments insignifcant. More about this can be found in chapter 6.3.1, appendix A.

What these results shows, is that only a small amount of the load is taken in bending (fig. A) which foremost changes sign at crown level but the axial forces however (fig. B), increases tremendously. This behaviour agrees well with White and Layers description of the classic ring compression theory (Eq. 1.1, chapter 1.3.3) which states that the internal thrust forces is propotional to a uniform overburden pressure [20]. In other words, if the overburden pressure above a flexible pipe increases, the internal thrust forces of that pipe increases proportionally to the increased overburden pressure.

One factor that doesnt correlate as well between PLAXIS and field is the magnitude of the internal thrust forces. According to Petterson. L [33], the measured thrust forces at backfill finish was about 173 kN/m, while 122.4 kN/m in PLAXIS. According to Wadi [37] this relates to 2D-softwares based of plane-strain not being able to catch the ortotropic behaviour of the corrugated steel plates and 3D-softwares such as Abaqus has proven to be better suited for such purposes.

For the backfill test in Canada, both vertical and horisontal displacements were measured using an identical backfill procedure as for Enköping. For this validation test, it was decided to stick with 25 kN/m as compaction force despite a lighter vibro tamper being used at the site. A comparison between deformations measured in field and PLAXIS is shown in figure 2.42.

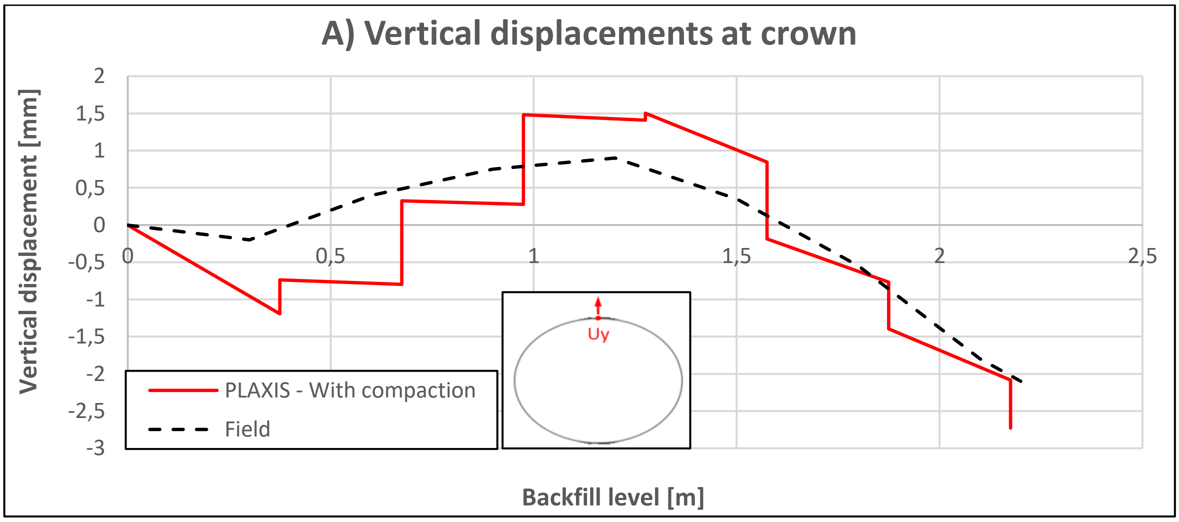


Figure 2.42 – Comparison between PLAXIS and fieldmeasurements from Canada, showing A) vertical displacements of crown

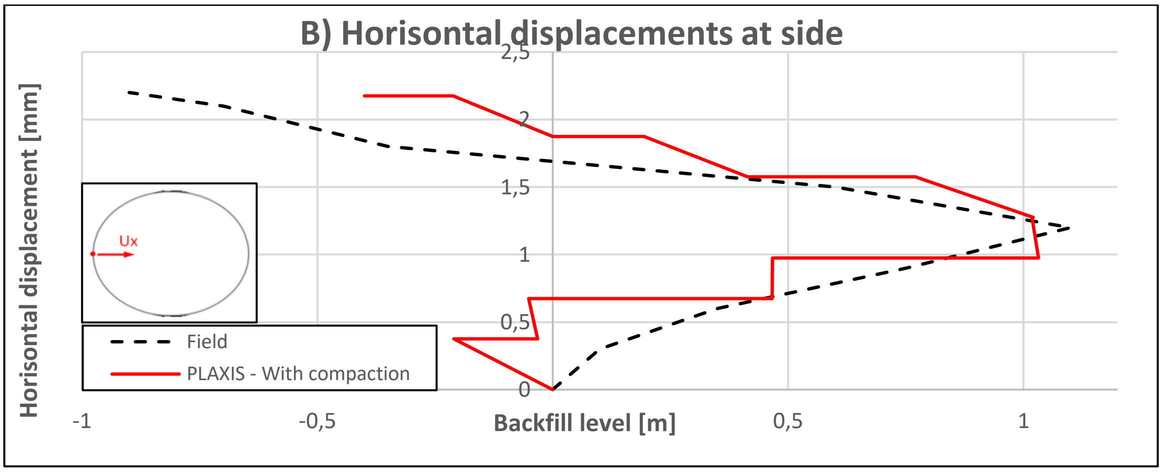


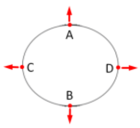
Figure 2.42 – Comparison between PLAXIS and fieldmeasurements from Canada, showing B) horisontal displacements

As this comparison shows, the results from PLAXIS correlates well to the measurements from Canada. Indeed, one could argue that the compaction load should've been decreased in this case but such minor details are not considered important compared to evaluating the overall behaviour of the soil-structure interaction of flexible pipes.

The internal bending moments on this pipe was measured to be in an order of 0.4 kN/m, and was 0.37 kN/m in PLAXIS. Similar to Enköping, this shows that the development of bending moment are propotional to the development of deformations.

The axial forces (thrust) are presented in table 2.6, and shows that in this case they also deviates some compared to field in terms of magnitudes but the results from PLAXIS shows that the model is able to catch the thrust concentrations at the lower radiis of the pipe, which agress with the formulation of ring compression theory, that a decrease in sectional radius leads to an increase in earth pressure, see Eq. 1.1 in chapter 1.1.3.

Table 2.6 – Axial thrust at cover height of 0.9 m, showing results from four nodes at the culvert

		Node A	Node B	Node C	Node D
	**Field**	18 kN/m	21 kN/m	30 kN/m	27 kN/m
	PLAXIS	12.12 kN/m	14.8 kN/m	17.14 kN/m	17.14 kN/m

What these validation tests has demonstrated is that if fundamental variables in field such as compaction forces, backfill materials and execution (layer thinknesses, the bedding, soil cover) are known, this methodology purposed with the Hardening Soil is able to catch the soil-structure interaction of flexible pipes with high accuracy. This aspect is considered the most important one when evaluating this bridges performance during passing train loads as it is this aspect that reflects the vertical stiffness above the bridge seen from the surface, which is governed by the backfilling stages.

2.4.4 Changing variables in the backfill and comparing to long-term fieldmeasurements

Other aspects of flexible pipes were also studied in PLAXIS, that is how variables such as different compaction loads, material combinations and the unloading stiffness affects the long-term soil-structure behaviour of the model after it has plastified and reached its elastic state. The purpose of this study was to get a better understanding of what it ment to the vertical track stiffness over these culverts when changing these variables.

This «long-term behaviour» in PLAXIS was also compared to observations registered at Dovre and Tolpinrud, two large span flexible steel bridges built in the 1980s in Norway. On these structures the internal forces was monitored in a period for about three years while earth pressures was continously monitored for more than 21 years with the use of Glötzl cells [18,21].

For this study, a similar model to what was used for the Enköping case was created in PLAXIS. The only difference here was that the outer compaction forces and backfill material was defined as variables. The analysis was finished with a load test performed twice for validating the structures behaviour when the soil model became elastic (i.e «long-term behaviour» with respect to the train loads), see figure 2.43.

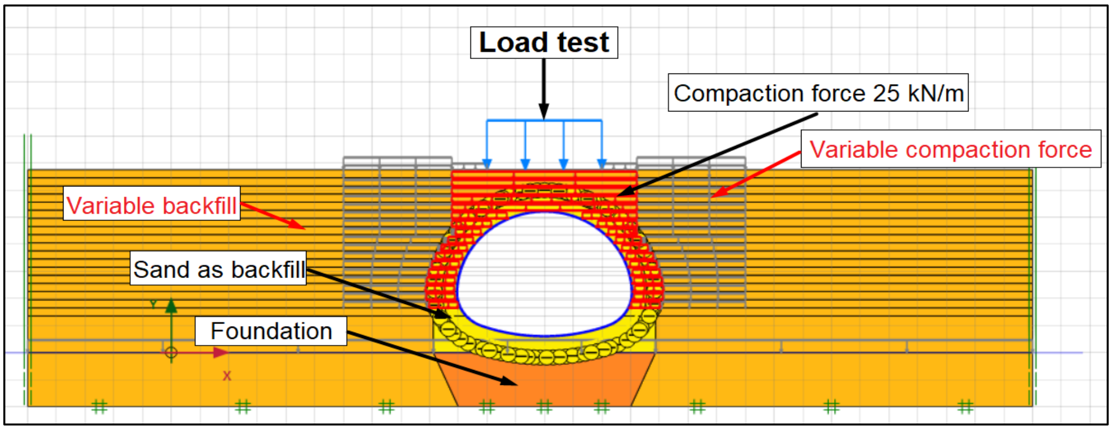


Figure 2.43 – Test model for investigating aspects of the backfills influence on surface loads

In total 5 scenarios was investigated, that is

- Using sand exclusively as backfill and only compacting with 25 kN/m of force
- Using sand exclusively as backfill and compacting with 25 kN/m and 50 kN/m of force
- Using sand and crushed rock as backfill and compacting with only 25 kN/m of force
- Using sand and crushed rock as backfill, and compacting with 25 kN/m and 50 kN/m of force
- Using sand and crushed rock, compacting with 25 kN/m and 50 kN/m, increasing the unloading stiffness to eight times the referance stiffness (oppose to 3 times which were used for the other cases)

After the backfill of these five cases had been constructed, the structure was loaded with a 4.0 meter wide line load placed on top of the culverts with 200 kN/m of force. The load test was performed two times in order to catch these structures elastic behaviour

In figure 2.44, the stepwise vertical deformation from the crown of the culvert is shown together with vertical displacement of the load test.

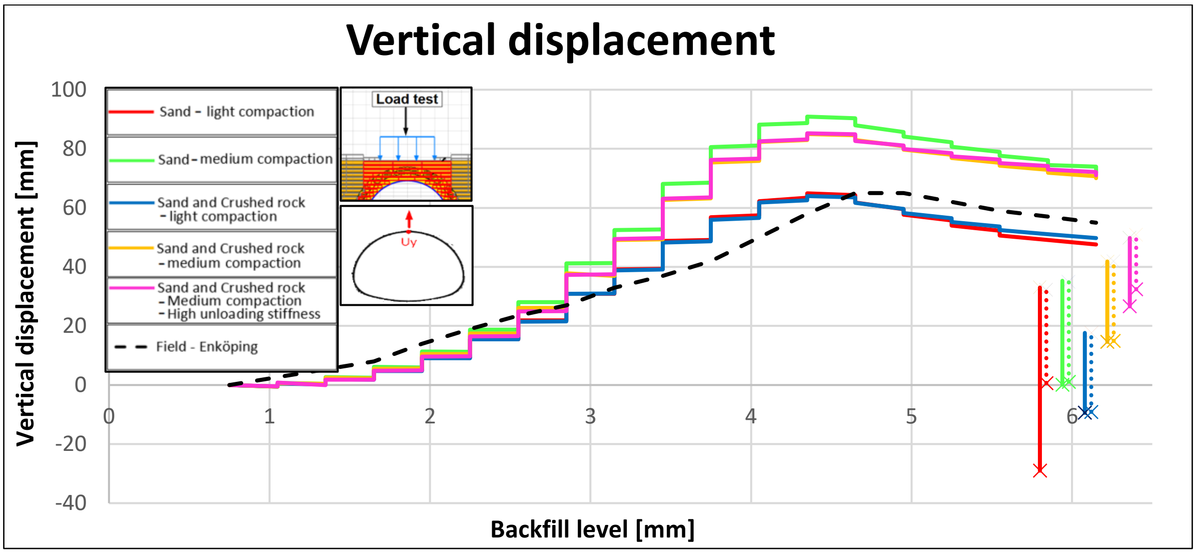


Figure 2.44 - Results from the test model for investigating how variables such as combining backfill materials and compaction forces affects the soil-culvert interaction when the soil model behaves elastic, seen from the crown of the culvert

What these results shows is that changing the backfill materials in the outer zone influences the deformations of the pipe insignificant compared to changing the compaction loads. Only the case with sand and increased compaction loads led to a slightly higher deformation of the pipe than the other cases.

Results from the load test reveals better how this really affects this culverts soil-interaction as the crown can be seen to displace differently in all five cases. For the case were sand had been used exclusively with light compaction, results from the load test suggests that the soil in that case plastified the least from the first load cycle (i.e the surface load) compared to all other four cases. This can be seen by comparing the first load test (continous red line) to the second load test (dashed red line) in the figure.

The effect of increasing compaction loads causes a bigger improvement (i.e resistance) to surface loads for the cases with crushed rock compared to the cases with only using sand. This suggests that more advantages follows the use of greater compaction loads in combination with having crushed rock in the outer zone as it seem to create more like a rigid-like support around the culverts circumferential cross-section, causing it to deform less when being subjected to vertical surface loads.

The horisontal displacements of this culvert was also investigated for this study, and the results from all five cases are presented in figure 2.45.

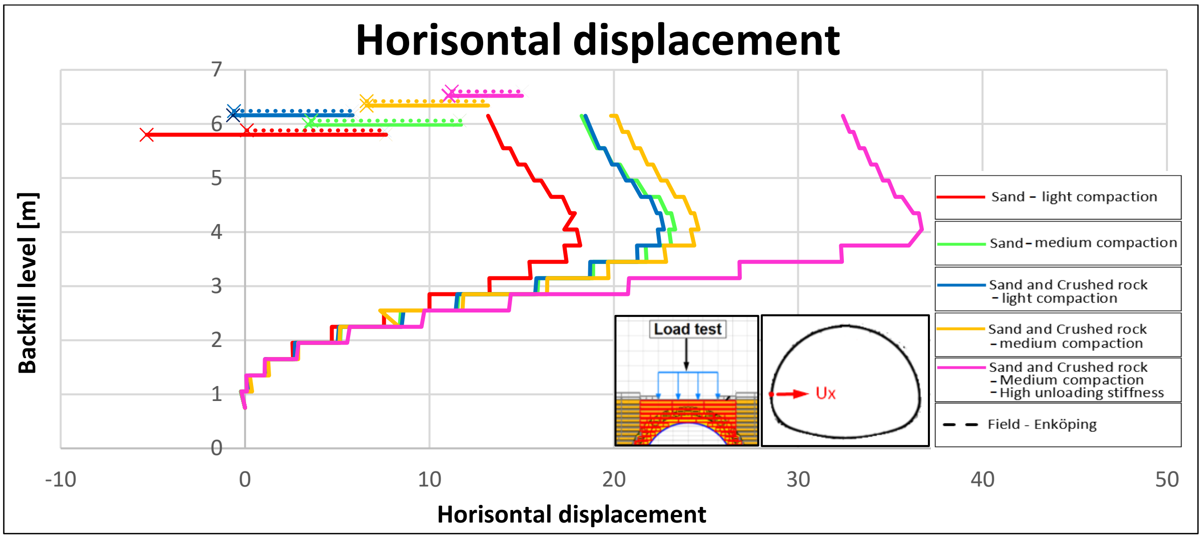


Figure 2.45 - Results from the test model for investigating how variables such as combining backfill materials and compaction forces affects the soil-culvert interaction when the soil model behaves elastic

Similar to the findings found at the crown (figure 2.43), increasing the compaction loads causes the sides of the culvert to displace more horisontally. Whats interesting here is that cases with crushed rock tends to cause the pipe to deform more horisontally during backfilling than the cases with sand. This is thought to be related to the crushed rock providing a trench like support for the sand close to the culvert, causing the horisontal forces to be greater against the pipe as a result from the compaction loads. This is even more evident when increasing the unloading stiffness as it causes the pipe to displace considerably more horisontally.

When applying the surface load of 200 kN/m, it can be seen that only the cases with crushed rock and medium compaction shows an advantage over using only sand in terms of resistance (i.e «surface stiffness»). It can also be seen that the resistance becomes greater and the elastic range shorter when increasing the unloading stiffness, suggesting that the surrounding forces of the pipe increases as the material hardens, which enhances the soil-culvert interaction seen from the surface.

When investigating internal forces of the pipe such as bending moments and axial forces, the general trend is that for all five cases where the heavier compaction loads have been used, the internal bending moments also increased. Another finding is that after performing the load test and the model plastifies (i.e it starts to behaves elastic), the bending moments were almost unaffected when unloading while the axial forces increased (compared to before applying the load). Also see chapter 6.4.1 in Appendix A.

When comparing this behaviour to the measurements at Dovre and Tolpinrud, the general trend at Dovre was that during a periode of three years, the bending moments remained almost unchanged compared to the change that developed during backfilling. But the axial forces however, changed tremendously during this periode which suggests that flexible pipes takes more of the overburden pressure in ring compression with time, as the soil rearranges [18,21].

The 21 years of earth pressure measurements for both these culverts showed the tendency of the horisontal earth pressures of the pipe tends to increase with time, suggesting that they gradually moves towards a passive state [18,21], also see chapter 6.4.2 in Appendix A.

When threating the unloading stiffness in Hardening Soil as the parameter that controlls the long-term stiffness of this model, it was shown from the load test that increasing it led to a stronger soil-structure as it became better to resist surface loads. This occours since it is the unloading stiffness that mostly controlls the elastic stiffness in Hardening soil, which agrees to the long-term earth pressure tendencies measured at Dovre and Tolpinrud.

What these findings means for the complete model is that using a combination of sand and crushed rock instead of only sand enhances the overall vertical track stiffness over a flexible culvert. Increasing the compaction loads enhance the surface stiffness even more as it causes larger internal bending moments to develop at the crown during backfilling, making the structure better suited to resist surface loads.

When the train loads has passed over the flexible culvert a couple of times, and the soil hardens (i.e the model starts to behave elastic), as has been shown in this section when this happends the «long-term» bending moments changes insignificantly compared to the changes that occoured during back-filling. The axial forces (thrust forces) however, increases. This suggests that the structure takes more of the overburden pressure in ring compression as the model hardens, a behaviour which correlates to the long-term field behaviour measured at Dovre and Tolpinrud.

2.5 Stiffness problem in the Hardening Soil model

The topic presented in this section are not covered in Appendix A since it was discovered at the later stages of this project. Whenever running these train simulation with Hardening Soil models, it was found that PLAXIS always exaggarated the displacements at the track over these culverts.

In the earlier stages of this project was it found that the track always seemed to settle a little bit more above the concrete culverts oppose to the free track, something which is uposite of what one would expect. Later in this project was it also found that when running these simulations over flexible culverts, the crown displacement would typically show 3-5 times higher deflections of whats been suggested by field meausrements. It was concluded that this was more related to the Hardening Soil models inbuilt formulation of stiffness than the models themselves.

According to Nordal [34], the inbuilt stiffness modulus in Hardening Soil models are described with expressions similar to

$$E_{50} = E_{50}^{ref} \left(\frac{\sigma 3' + a}{Pref + a} \right)^m \quad \text{(Eq. 2.7)}$$

where

E_{50}^{ref} being the input parameter for stiffness
σ_3' being the horisontal effective stress
a being the attraction
p_{ref} being the atmospheric pressure at 100 kPa
m being the power modulus

Eq 2.7 shows that the stiffness is stress dependent by multiplying the referance stiffness (E_{50}^{ref}) with a formulation that accounts for stress dependency. From how this expression is formulated, it can be seen that the stiffness will be multiplied with a quantity lower than 1.0 untill the stress passes 100 kPa, which is the referance pressure.

The quantity multiplied with the referance stiffness is also governed by the power modulus (m), a factor that controls how a stress propagates in a soil. It is this factor that makes the Hardening Soil model applicable for most types of soils since it allows for predefining how the stress should propagate.

According to Nordal [34], standard values of the power modulus is normally 0.5 for sands and 1.0 for clays. This would cause the referance stiffness to be multiplied with a square root function for a sand but a linear function for a clay. Untill the latest stages of this project was an order of 0.5 used as a standard input for all backfill materials. The thought here was that crushed rock in terms of stresses would distribute similar to a sand since nothing else regarding this had been purposed in the litterature.

The problem with this formulation in respect to these models are related to the stress level at surface where it is almost zero and hence, the stiffness will also be close to zero. This explains why the track above the concrete culverts always seemed to displace more than the free track since the stiffness bellow this culvert would never actually work at its full effect untill the train loads were present.

After constructing these culverts (i.e finishing a backfill sequence), the horisontal stress fields surrounding them would typically look as shown in figure 2.46.

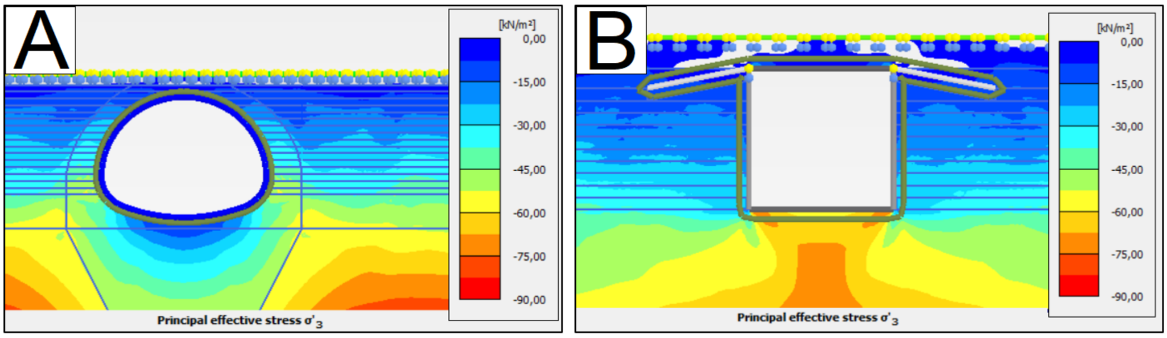


Figure 2.46 - Principal effective stress fields for A) a pipe arch flexible steel-soil composite culvert and B) a concrete culvert

What these figures shows is that the stress close to the surface or under the circular culvert is nearly zero, causing the stiffness also to be nearly zero (see fig. 2.46A). The rigid culvert however seem to absorb alot of the overburden earth pressure and concentrate it down the ground, but this stress concentration is only in an order of 60-80 kPa. This means that the stiffness will be lower than the referance stiffness untill the train loads are present.

The problem this causes for the model which are so dependent on accuracy, is that the stiffness will be to soft untill the train loads has reached the culverts, and deformations will be present before any stiffness increase occours. It was for example found with test simulations on the Enköping culvert with 1.5 meter soil cover that the crown could displace up to 10 mm during a train simulation, which is unacceptable.

For the concrete cases a similar tendency was found, but to a smaller degree. The foundations stiffness would not act fully untill the train loads were present which caused the bottom section of the culvert to displace to much, evidently exciting the displacements seen at the rail over it and causing the track to displace more above the culvert compared to the free track. This is the reason for why increasing the unloading stiffness to double the unloading stiffness of the crushed rock works for the foundation.

Ways of solving this problem can be to, increase the unloading stiffness which governs the stiffness in these models as they reach their elastic states. Another way of decreasing the problem is to decrease the power modulus number which will make the stiffness less stress dependent and higher closer to the surface. A third way of decreasing the problem is to add some cohesion in the material, which relates directly to the attraction in equation 2.7.

It was finally decided to decrease the power modulus to 0.2 for the crushed rock and 0.3 for the sand. This forces the model to behave stiffer closer to the surface. But this however, was not enought for the flexible culverts. It was also decided to separate the sand material close to the flexible culverts by dividing the material into a «short-term» and a «long-term» material, using a different unloading stiffness and cohesion during backfilling compared to the final train simulations.

3 Four specific model cases of culvert underpassings

This chapter present four specific model cases of railway transition zones over culvert underpassings, where two of these cases involves the use of flexible steel and two of them rigid concrete. For all cases the minimum required soil cover are used to represent a worst case scenario in terms of soil cover.

For the steel cases the steel thickness and geometry were selected based of available design drawings and formulations in the SDM-manual [13]. It was decided to chose two cases of steel which were very different from each other in terms of geometry, span lenght and stiffness in order to study how these aspects may affect the final track stiffness as the moving train loads passes.

For both concrete cases the culvert itself was a predefined 3.5x3.5 meter integrated square box culvert. This culvert was defined based of Bane NORs standard that allows to used a shorter transition slab, down to 2.5 meter if the culvert has a height less than 3.5 meter [9]. In the second case the exact same culvert was used but replacing the short transition slab with a 4.0 meter long slab. These cases were chosen to both investigate track stiffness but also the aspect of how increasing the slabs lenght influences the track stiffness in the transition zone.

3.1 Materials

In this chapter, every general material used in all four models are presented which includes properties of the railway superstructure (i.e rail, railpad, sleeper and ballast), the backfill (i.e sand, 20/120 mm crushed rock and foundation) but also dampingfactors.

The railway superstructure is based of Bane NORs technical design basis for the new InterCity lines which states that all new railway tracks in Norway must be made of 60E1 rails and JBV 60 concrete sleepers [23]. All these track components are modelled as elastic plate elements in PLAXIS, as described in chapter 2.2. The sleepers was ensured a sufficiently high stiffness based of that their intended function in this model is to only distributed the train loads down to the ballast layer realistically.

Transforming these track components into plane-strain is achieved by dividing them by the width of the 2.6 meter long JBV sleeper [9]. From this principle and the calibration example for the railpad (chapter 2.2.1), table 3.1 is obtained. For more on assessing of these track comp-onents, see chapter 2.2 in Appendix A.

Table 3.1 – Track properties of the elastic plate elements used for PLAXIS

	EA [kN/m]	**EI [kNm^2/m]**	**w [kN/m/m]**	**v [-]**
60E1 rail	1.239*10^6	4908	0.4543	0.3
Railpad	225	1.0*10^6	-	0.3
Sleeper	1.0*10^6	1.0*10^6	1.08	-

The ballast are modelled as a linear-elastic soil material and is given the stiffness that was calibrated via Zimmermanns theory (beam on elastic foundation) with experience numbers from Deutche Bahn on ballast coefficients, see chapter 2.2.3 or chapter 2.3.2 in Appendix A. The ballast properties used for all four models are given in table 3.2.

Table 3.2 – Properties of the linear-elastic ballast used for PLAXIS

Linear-Elastic	**E [MPa]**	**γ [kN/m^3]**	**v [-]**
Ballast	140	20	0.3

The materials used in the backfill is the predefined sand and the 20/120 mm crushed rock material from chapter 2.4.2 and 2.5. As was discussed in those chapter, the stiffness behaviour of these materials must be threated differently in the backfilling stages compared to the elastic stages of the model (i.e when the train loads has plastified the model. It was therefore decided to threat the stiffness parameters for the flexible and rigid culvert cases separately.

For both cases a predefined foundation given twice the unloading stiffness of the regular backfill is set. The power modulus number for all backfill materials are also decreased from the standard value of 0.5, down to 0.2 for the crushed rock and 0.3 for the sand respectively. This forces these materials to behave stiffer closer to the surface and being less stress dependent. This will decrease the deflections seen at the free track in an order of 0.1-0.3 mm (oppose to a power modulus of 0.5), see chapter 2.5.

For the rigid concrete cases, no distiction has been made between the sands «short-term» and «long-term» stiffness behaviour. The unloading stiffness is also set to the same unloading stiffness as the crushed rock, this in order to prevent any exaggaration of the displacements seen at the slab. This is from the presumption that a densily packed sand may behave similar to a crushed rock, see table 3.3.

Table 3.3 – Hardening Soil parameters for the sand, backfill and foundation used for models involving concrete culverts

Hardening Soil	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	γ [kN/m^3]	φ [°]	ψ [°]	c [KPa]	m [-]
Sandy gravel	30	30	225	20	36	5	1	0.3
20/120 crushed rock	45	45	225	20	40	15	0.1	0.2
Foundation	45	45	450	20	40	15	0.1	0.2

For the flexible steel-soil composite cases it was discovered several problems tied to the stiffness of the sand material as the train loads were passing the culvert. Before the train loads reaches the culvert, the stiffness of the surrounding sand at crown level is nearly zero causing the unloading stiffness to only work after a certain amount of displacements has taken place. Decreasing the modulus number helped reducing this problem, but it has in general been found that using anything bellow 600 MPa as a «long-term» unloading stiffness in combination with 25 kPa of cohesion leads to unacceptable large deformations of these culverts when the train loads are passing them.

It was decided to distinguish between a «short-term» and a «long-term» stiffness for the sand material, where a short term stiffness would reflect the sands stiffness during backfilling and construction stages. This stiffness is then changed after construction finish, increasing both the cohesion and unloading stiffness to account for the problems tied to the stiffness stress dependency in Hardening Soil, see table 3.4.

Table 3.4 – Hardening Soil parameters for the sand, backfill and foundation used for the models involving flexible steel culverts

Hardening Soil	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]	E_{ur}^{ref} [MPa]	γ [kN/m^3]	φ [°]	ψ [°]	c [KPa]	m [-]
Sandy gravel («short-term stiffness»)	30	30	225	20	36	5	1	0.3
Sandy gravel («long-term stiffness»)	30	30	600	20	36	5	25	0.3
20/120 crushed rock	45	45	225	20	40	15	0.1	0.2
Foundation	45	45	450	20	40	15	0.1	0.2

Another predefined quantity is damping factors. As was discussed briefly in chapter 2.3.4, using no damping in a Hardening Soil material causes persistent plastification of the material which can be solved by either using damping or unhooking the «Tension cut-off» option and running some train simulations. It was decided to rather unhook the tension cut-off option and let the majority of the problem convergate away instead of damping the model and runing the dynamics.

Damping the entry and exit of the model goes as following, the section closes to the end boundaries in these models are damped with 400% at 5-50 Hz, then gradually decreases to 200%, 100% and finishes with zero damping in the regular backfill, summarized in table 3.5.

Table 3.5 – Rayleigh damping factors for the ballast, backfill and foundation at the end boundaries

	Damping 5-50 Hz	**α - factor**	**β - factor**
End boundary	400%	228.5	0.02315
First transition	200%	114.2	0.01157
Second transition	100%	57.12	$5.78*10^{-3}$

3.2 Two cases of rigid concrete culverts – changing the slab lenght

For the two models involving concrete, the only difference between them is changing the lenght of the transition slab. The culvert itself is a 3.5x3.5 meter integrated square culvert having a 45 centimeter thick roof/bottom element and 30 centimer thick concrete walls. These conditions was chosen based of recommendations from norwegian manufacturers of concrete elements. The transition slab is modelled as a 30 centimeter thick, 3.2 meter wide concrete plate with a lenght of 2.5 and 4.0 meter, see figure 3.1.

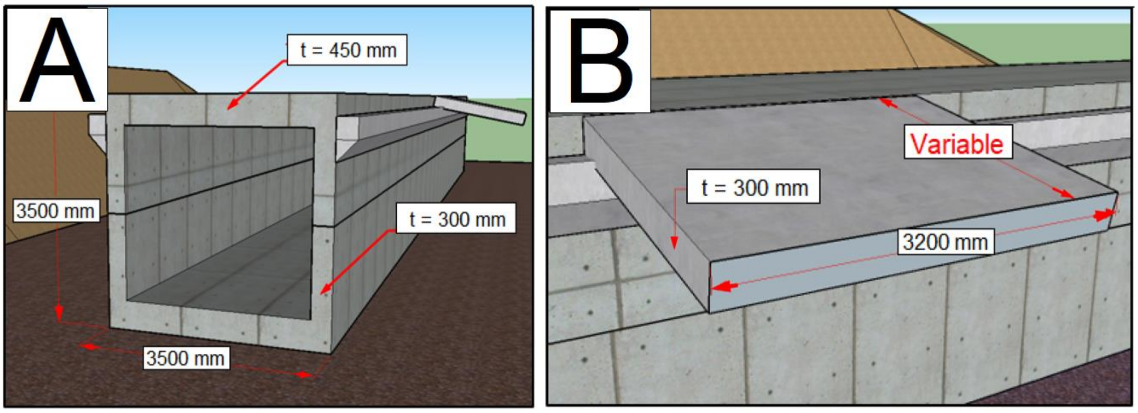


Figure 3.1 – Preconditions for both concrete cases, showing that the only variable between them is the slabs lenght

When transforming the concrete culverts cross-section into plane-strain, it is divided into three plate elements for PLAXIS based of the difference in their thicknesses. These elements are the slab/bottom element, the wall elements and the transition slab, see chapter 2.1.3. For these two model cases, the only difference regarding material properties is the prolonged transition slab in case number two.

When chosing the elastic modulus for these elements the thought here was that one should expect the culvert to behave very stiff and deform minimally during a train passage. This presumption was based of that several field cases on stiff bridges, has shown that the foundation modulus under a railway track typically is about 1.5-2.0 times higher at stiff bridges compared to a free track [3,6].

Using the purposed methods of ULS design in Eurocode 2 (EC2) suggests taking into account effects of creep and shrinkage which reduces the elastic modulus considerably. The elastic modulus from this approach will be to soft for the behaviour aimed for in this project, and the modulus was rather viewed more upon as a SLS parameter rather than ULS for these simulations. If one can assume that the concrete has a balanced reinforcement, is completely uncracket and takes both compression and tension, then characteristic values for concretes elastic modulus based of strenght classes in EC2 could, without exaggarating to much be used directly as the stiffness of the reinforced concrete elements. It was herby decided to use an elastic modulus of 35 GPa based of a C40 concrete.

After defining the elastic modulus, the only work remaining is to define the second moment of inertia for each individual element of the culvert. By using the simple formulation for any square object (Eq. 2.1, chapter 2.1.3), all the elements of the culvert could be transfered into plane-strain. By multiplying the thickness (i.e height) of each element with the unit weight of concrete, the transformed weight in plane-strain were also obtained, see table 3.5.

Table 3.6 – Material properties for both concrete culvert cases

	EA [kN/m]	EI [kNm^2/m]	w [kN/m/m]
Top/bottom elements	55.13 * 10^6	2.657 * 10^5	10.8
Wall elements	27.30 * 10^6	15.379 * 10^6	7.2
Transition slab (2.5m)	26.25 * 10^6	7.875 * 10^4	7.2
Transition slab (4.0m)	42.00 *10^6	7.875 * 10^4	7.2

In chapter 5.1.1 in Appendix A, a comprehensive example for transforming a 3.5x3.5 concrete culvert into plane-strain is given. The chapter also provides more discussions regarding the elastic modulus.

When constructing this culvert for the model, it is built according to the principles of chapter 2.4.1 to the point. First a foundation is prepared, then the culvert is built followed by alternately backfilling in 0.3 thick layers. For compaction loads, «light compaction» is used in a section about 1.5 meter out from the culvert (i.e 25 kN/m), and «medium compaction» (i.e 50 kN/m) further out, for about 4.0 meter.

When reaching the slab level, the layers are then compacted with more caution (i.e max. 25 kN/m) with respect to the concrete of the slab. The slab is constructed with an sloping angle of 1:5 [9]. The model is finished by covering the slab with regular backfill material and constructing the track, see figure 3.2.

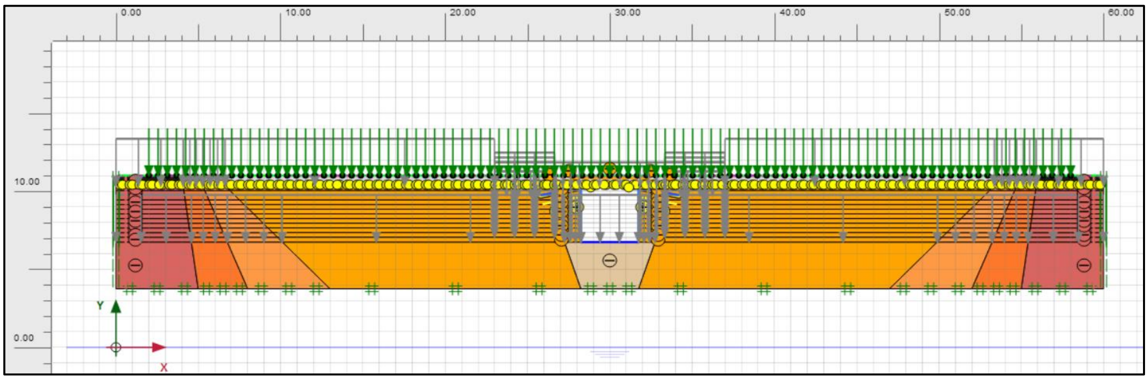


Figure 3.2 – The finished FEM model in PLAXIS 2D, ready for a dynamic train simulation

What this figure shows is the finished 60 meter long model with the culvert founded in the middel of it on top of a 3.0 meter thick foundation. At the end boundaries, the use of damping can be seen in the three predefined zones discussed in chapter 3.1 (corresponding to table 3.4).

The material properties of the backfill in these three zones have the exact same properties as the regular backfill, but with only differating the damping. At the end boundaries it can also be seen a vertical interface placed on both sides of the model. This was made in order to enable the «free-field» option in PLAXIS, which allows for shear waves to travel continous out at the boundaries. More information regarding this is provided inchapter 3.3, Appendix A.

After overfilling the culvert the ballast together with the track is built simultaneously in one single phase. The track section is 59.6 meter long and the traveling distance for the trains loads 56 meters (the point loads seen on top of the rails). This traveling distance was considered sufficient in order to ensure that the end boundaries didn't affect the displacements at the culvert during the at the «train passage».

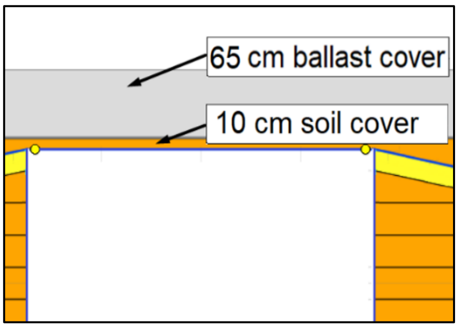


Figure 3.3 – Soil cover conditions based of the minimum requirement set for rigid culverts (75 cm)

According to the norwegian railway normal «teknisk regel-verk», the minimum soil cover (including ballast) over a rigid foundation must range in between 0.7-0.8 meters seen from the top of the rails [9]. After consulting with Bane NOR, it has been suggested to stick with 750 mm as a minimum based of the presumption that this is a modern track built according to the design basis of the InterCity lines [23]. It was therefore decided to use a ballast thickness of 65 centimeter and a soil cover (i.e with 20/120 mm crushed rock) of 10 centimer for this culvert, see figure 3.3.

3.3 Two profile types of flexible steel-soil composite culverts

The vertical stiffness seen at the surface over flexible steel-soil composite bridges have very high dependency on factor such as geometry, steel thickness, backfill materials, compaction loads and span lenght. The basis for selecting the two cases was therefore to investigate «a soft case» with long-span pipe arch culvert and «a stiffer case» with a short-span circular culvert.

These two cases was predefined based of intuision and engineering judgement where referance projects and the formulations in the SDM-manual [13] was the background in creating these cases.

3.3.1 Pipe arch with MP 200x55x7

The first case representing flexible steel is a large span «pipe-arch». This perdefined case is a culvert with a cross-section made of three radiuses and was given the properties of a MP 200x55 mm corrugation and 7.0 mm steel thickness. The steel thickness was chosen based of two referance projects, Sjånes and Holme railway culvert where a steel thickness of 6.0 mm was used.

These two norwegian railway culverts had a span lenght between 4-5.8 meter and was built with a MP 200x55 corrugation. Having as shallow as 0.93-1.1 meter soil cover, they were though to be representative for the pipe-arch of this section. See chapter 2.1 and 2.2 in Appendix B for more on these culverts.

The predefined pipe-arch spans 7.0 meters long, is 5.24 meters high and is given three radiuses. The radiuses was selected in order to make this culvert particulary wide (i.e «soft») with respect to the track stiffness seen from the surface, see figure 3.4.

According to the SDM [13], there is a certain ratio between these three radiuses that must be furfilled in order for a culvert to pass the minimum criteria of a «pipe arch», which is

- $\frac{R_t}{R_c} = \frac{1.8}{1.4} = 1.28 \leq 5.0$
- $\frac{R_b}{R_c} = \frac{7.0}{1.4} = 5 \leq 10$

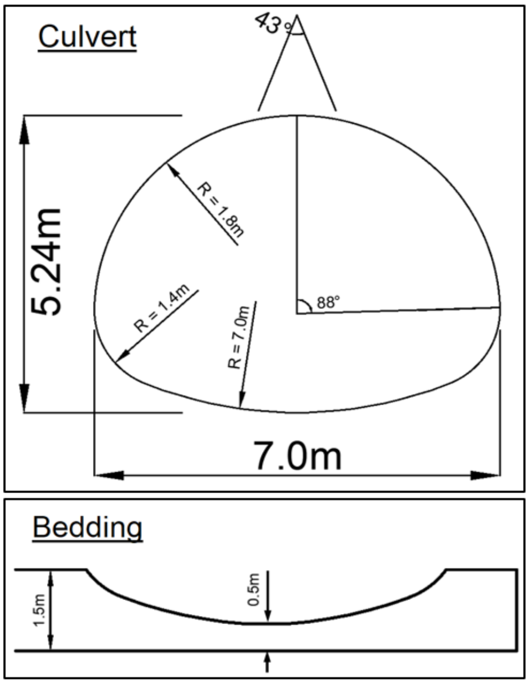


Figure 3.4 – Geometric properties of the predefined pipe arch selected for this specific case

The uncompacted bedding of this culvert is built as a 0.5 meter soil body over the bedrock, gradually in-creasing in thickness towards the sides of the culvert and reaching a total thickness of 1.5 meter on the sides, see figure 3.4. More on practices for preparing beddings can be found in [13,33], also see chapter 4.2.5 in Appendix A.

The Swedish Design Method (SDM) provides cross-section data for several corrugation types, and with the principles outlined in chapter 2.1.3 the cross-sectional data of a MP 200x55 corrugation can be used from SDM direcly when determining the properties of the culverts elastic plate elements, see table 3.6.

Table 3.7 – Material properties for the steel plates in PLAXIS

t [mm]	EA [kN/m]	EI [kNm^2/w]	w [kN/m/m]
7	14.73*10^5	673.68	0.5495

A parameter that describes the soil-culvert interaction of any flexible SSCB, is the flexibility number (see chapter 1.3.3). This quantity describes the overall stiffness of the culverts interaction with the soil, and is used in SDM for calculating bending moments for design [13]. This parameter can also to some extent be considered a quantity that describes the track stiffness over a flexible culvert. According to SDM, the soil stiffness used in their formulation of the flexibility number is the characteristic stiffness of the backfill, i.e the average stiffness surrounding the culvert.

Since the culvert presented in this thesis is not created for design, material factors are avoided. The stiffness used as the referance stiffness when calculating the flexibility number is the stiffness of the material closest to the culvert (i.e sand) with 30 MPa, which is not that far away from the stiffness that was calculated with SDMs method B for estimating the stiffness of the Enköping sand (27.5 MPa), see 6.3.1 in Appendix A.

The intent of the flexibility number in this context is foremost to validate and compare the properties of this culvert to the circular culvert of the next section. By using the cross-sectional data from table 3.6, then equation 1.2 (in chapter 1.3.3) can be used directly. The «n-factor» for describing the culverts ability to distribute earth pressure is also calculated, leading to

- $\lambda = \frac{E_S * D^3}{EI} = \frac{(30*10^3)(7.0^3)}{(673.68)} = 15273$ (criteria set by SDM: $100 \leq \lambda \leq 50000$)
- $n = \frac{EI}{E_S * D^3} = \frac{(673.68)}{(30*10^3)(7.0^3)} = 6.54*10^{-5}$ (< 0.1 corresponding to K ≈ 1.0, see chapter 1.3.3)

What these quantities shows, is that the culvert falls into SDMs category of «flexible» which also can be seen in Vaslestads formulation of earth pressure distribution with the n-factor, showing that this culvert will distributed the vertical overburden with a redistribution coefficient of about 1.0 since the n-factor is lower than 0.1 [13,18], see chapter 1.3.3. More background information is also given in chapter 4.2.1 in Appendix A.

After the properties of this culvert has been validated the procedure previously described proceeds. A foundation is prepared in three steps which involves constructing the layers, compacting them with a static line-load, then unloading. It was decided to increase the general foundation thickness of 3.0 meter to 4.0 meter in this case to avoid to much influence from the bedrock to the system due to the culvert especially long span. When the foundation has been prepared, the uncompacted bedding together with the culvert is built simultaneously in one single phase.

After creating the foundation, the backfill layers are built alternately 0.3 meters thick, compacted with a light compaction force in a section about 1.5 meter out from the culvert (i.e 25 kN/m), and medium compaction force (i.e 50 kN/m) further out about 4.0 meter. When soil covering this culvert it is ensured that only light compaction (i.e 25 kN/m) is used above the structure, while medium compaction force as usual on the side, see figure 3.5 for the finished model.

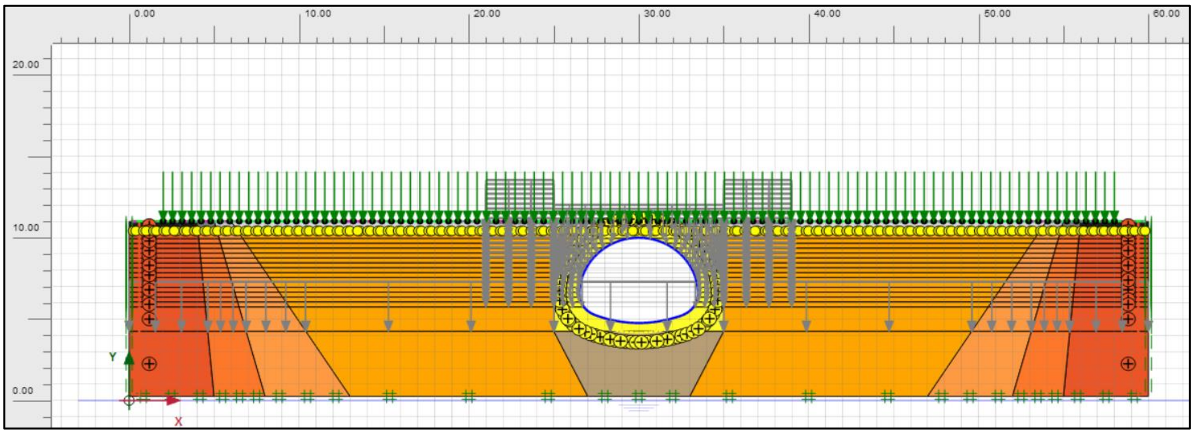


Figure 3.5 – The finished model for the first steel case showing the 7.0 meter span «pipe-arch» with 1.0 meter of soil cover

What this figure shows is that the model is 60 meter long, has 59.6 meter of track and 56 meter of traveling distance for the train loads. The end boundaries are damped in three individual zones (corresponding to table 3.4). The material properties of each zone is exactly the same as the regular backfill.

In total three materials is involved in the backfill, that is sand used as bedding and the closest meter around the culvert circumferential cross-section, 20/120 mm crushed rock in the outher zones and a foundation given twice the unloading stiffness of the crushed rock (corresponding to table 3.3).

For the minimal case, a shallow soil cover of 1.0 meter was chosen according to the limitations set in the SDM and Bane NORs regulations [13,9]. For the minimal case, the soil cover was built with 50 centimeter sand and 50 centimeter ballast, see figure 3.6A).

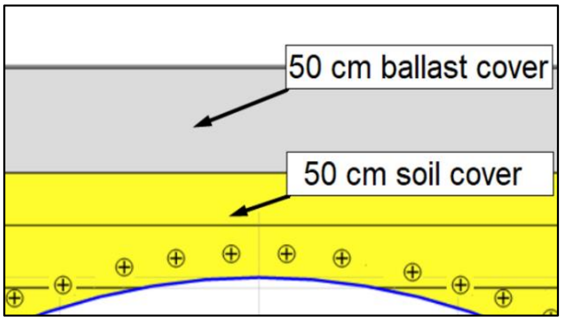


Figure 3.6 – Soil cover conditions based of Bane NORs minimum requirement set for flexible culverts (0.75 meter)

3.3.2 Circular culvert with MP 200x55x4

The second case involving flexible steel, is a smaller circular pipe spaning 3.5 meter. This case was predefine in order to study how a «stiffer» geometry and smaller span of flexible culvert types would affect the overall track stiffness.

A corrugation of the type MP 200x55 was chosen with 4 mm steel thickness. This was based of similar projects such as Solheim culvert at Ofotbanen with 3.5 mm steel thickness and the Märsta culvert from Sweden which had 5.25 mm of steel thickness [19].

The preselected culvert for this case is a completely circular pipe with a constant radius of 1.75 meter. The bedding of this culvert is similar to the previous case, 0.5 meter thick at the middle of the culvert, but was reduced to 1.25 meter on the sides due to its shorter span lenght, see figure 3.7

The stiffness of the soil-culvert interaction are investigated with the flexibility numbers, where the sands referance stiffness of 30 MPa are also used as the characteristic stiffness of the backfill (compared to the previous case). Together with the cross-sectional data for the culvert, the following are obtained

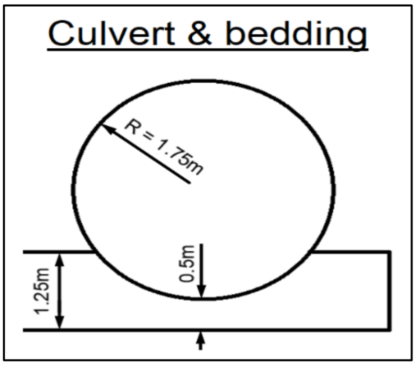


Figure 3.7 – Geometric properties for the predefined circular culvert for case no.2

- $\lambda = \frac{E_s * D^3}{EI} = \frac{(30*10^3)(3.5^3)}{(380.31)} = 3382$ (SDM: $100 \leq \lambda \leq 50000$)
- $n = \frac{EI}{E_s * D^3} = \frac{(380.31)}{(30*10^3)(3.5^3)} = 2.95*10^{-4}$ (< 0.1 corresponding to K ≈ 1.0, see chapter 1.3.3)

What these values show, is that the criteria in SDM is furfilled and its reasonable to assume that the overburden earth pressure will redistribute with factor of 1.0, but these quantities also shows that this culverts overall interaction with the soil in fact is stiffer than the pipe arch despite being built of more slender steel plates seen from the lower flexibility number (λ) or higher n-factor.

Since this culvert was also made of the corrugation MP 200x55, cross-section data could be used directly from the SDM manual when determining the material properties for PLAXIS, see table 3.7

Table 3.8 - Material properties for the steel plates of the culvert in PLAXIS

t [mm]	EA [kN/m]	EI [kNm^2/w]	w [kN/m/m]
4	9.933*10^5	380.31	0.314

After the conditions for this culvert were set, it is constructed exactly as the pipe arch but only decreasing the foundation thickness back to the general thickness of 3.0 meter instead of 4.0 meter, see figure 3.8.

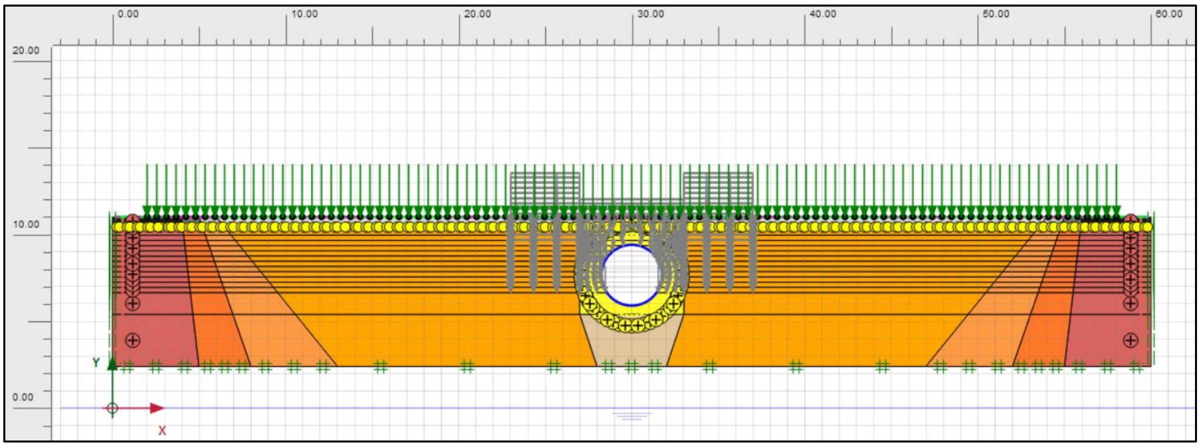


Figure 3.8 – The finished model for the second steel case, showing the 3.5 meter diameter pipe with 1.0 meter of soil cover

Similar to the previous cases, after preparing the foundation the culvert is built simultaneously at its bedding in one phase, followed by backfilling. Each layer of the backfill is 30 centimeter thick and «light compaction» (i.e 25 kN/m of force) is used in a section 1.5 meter around the culvert, and «medium compaction» (i.e 50 kN/m of force) further out in a section of 4.0 meter. The model is finished by constructing the ballast and railway track.

Similar to all other cases, this model is 60 meters long with 59.6 meter of track, and has a traveling distance of 56 meter for the train loads. The end boundaries are damped, gradually going from 400% damping to zero damping in the regular backfill.

The materials involved in this case is exactly the same as for the pipe arch where the sand is used as the bedding and around the culverts entire circumferential cross-section in about 1.0 meters width, and the 20/120 mm crushed rock further out (i.e the «outer zones»). The foundation is given twice the unloading stiffness of the backfill while all other properties remains the same, see table 3.3 in chapter 3.1.

The soil cover conditions for this case is identical to the conditions for the pipe arch. For the minimal case (1.0 meter soil cover) a 50 centimeter thick ballast bed is built on top of 50 centimer sand layer, see figure 3.9.

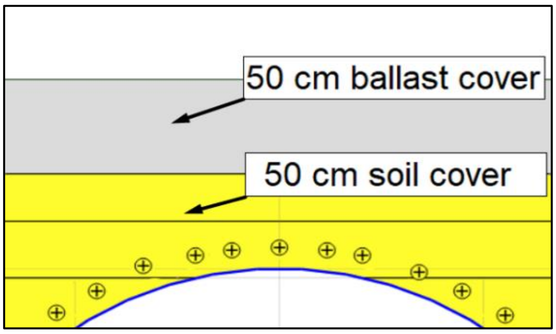


Figure 3.9 - Soil cover conditions based of Bane NORs minimum requirement set for flexible culverts (1.0 meter)

4 Results, discussions and comparisons

In this chapter results from the four specific models of chapter 3 are discussed and compared to field measurements. The overall aim with these models has been to catch the general stiffness pattern seen at the rail over these culverts but also to quantify them.

Due to time constraint was it decided to stick with eight train passages for plastifying each model. As some of these cases will show, this resulted in the presence of the convergence problem discussed in chapter 2.3.4. It was thought that the small occourance of this problem after running eight simulations was a better option than damping the entire model.

It was also decided to limit the use of the passenger train to only two of the four cases. This was decided from since its primarily the quasi-static aspect of the stiffness problem that are studied and not dynamics and veichle behaviour. The «freight train» in this context will exaggarate the displacements more than the passenger train. All these simulations are done by using a medium mesh.

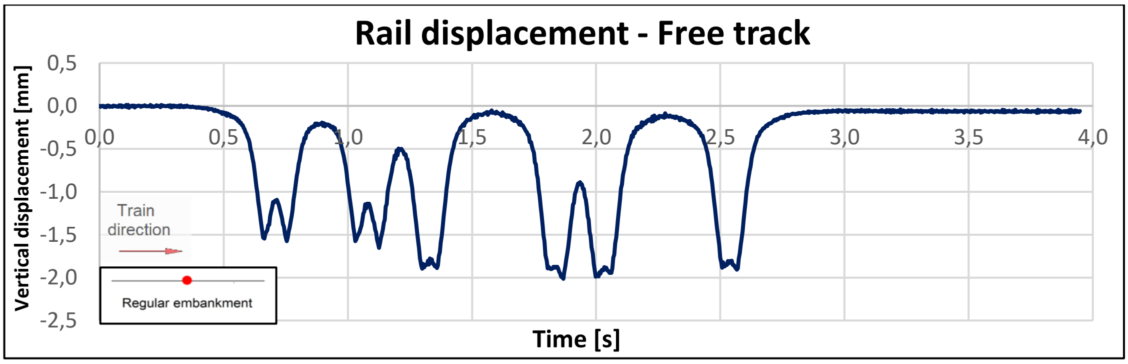
4.1 Stiffness variations over rigid concrete culverts and slab behaviour

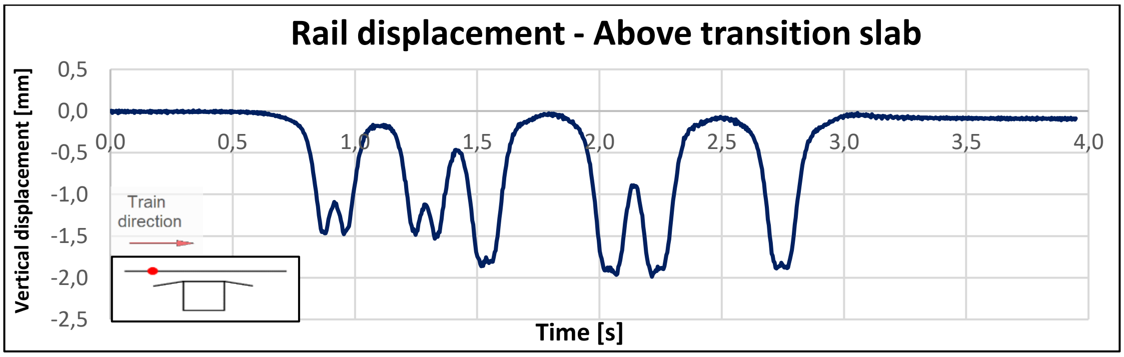
In this chapter aspects of track stiffness and slab behaviour are investigated for the two predefined cases of chapter 3.2. Aspect of backfilling for rigid culverts are not studied in this section, but are followed to the point according to the procedure described in chapter 2.4.1.

4.1.1 Concrete culvert with 2.5 meter transition slab

After the foundation has been prepared, the culvert is built with the properties defined in chapter 3.2 followed by constructing the backfill and the railway track. The model is then plastified by running eight train simulations. It has in general been found that the problem with persistent plastification is worse for cases with rigid concrete culverts oppose to the flexible steel ones, and in this case despite unhooking the «tension cut-of» option in PLAXIS the problem still shows its presence.

After constructing the culvert and reaching almost full elastic state in the model, vertical displacement results from the passage of a freight train is presented in figure 4.1. The figure presents the vertical displacement of the rail at the free track, the rail above the transition slab and the rail above the culvert.





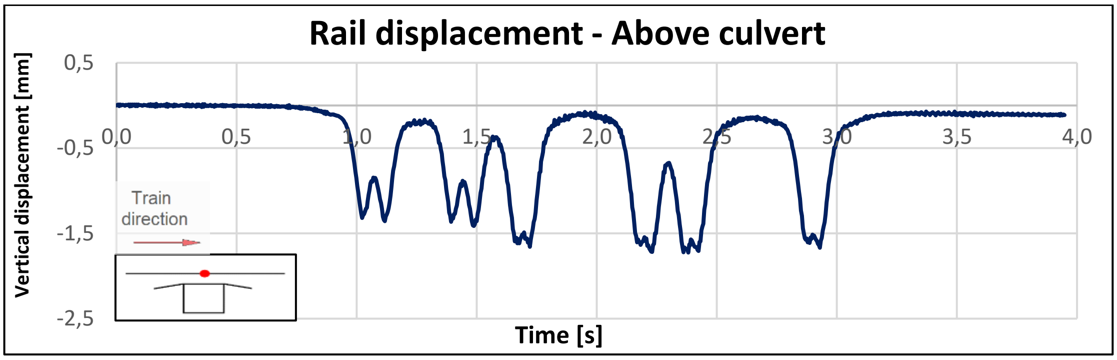


Figure 4.1 - Vertical displacements during the simulation of a passing «freight train» traveling at 28 m/s (100.8 km/h) showing results from the rail in the free track, the rail above the transition slab and the rail above the culvert

What these plots shows is the passage of the predefined freight train from chapter 2.3.1. It can be seen in the first plot (i.e the free track) that the first bogie hits the node at about 0.7 seconds where the peak represents the first axle of that freight train passing that point. This patterns is followed continously for all 12 axle of the «train», where it can be seen that the displacements caused by the wagons is higher than from the locomotive. This relates to their difference in axle loads, see chapter 2.3.1

The problem with convergence is also present in these plots from that the rail never returns to zero after the train loads has passed. It is also seen that the problem gradually worsens as one moves closer to the culvert, from having an offseth of about 0.05 mm in the free track to about 0.1 mm over the culvert. The problem present here is still thought of as not being critical for the purpose of this study.

From comparing the displacements at these three locations it can be seen from the maximum peak in at above the end of the transition slab but decreases to about 1.75 mm over the concrete culvert.

In order to better understand how the displacements propagates over the slab, a displacement pattern is deriven from investigating the maximum peak displacements seen at the rail from using the train wagons as referance. The displacement pattern does not represent the track stiffness since it is influenced by several axles, but gives an idea of how the characteristic stiffness patterns propragates over this culvert as a train is passing it, see figure 4.2.

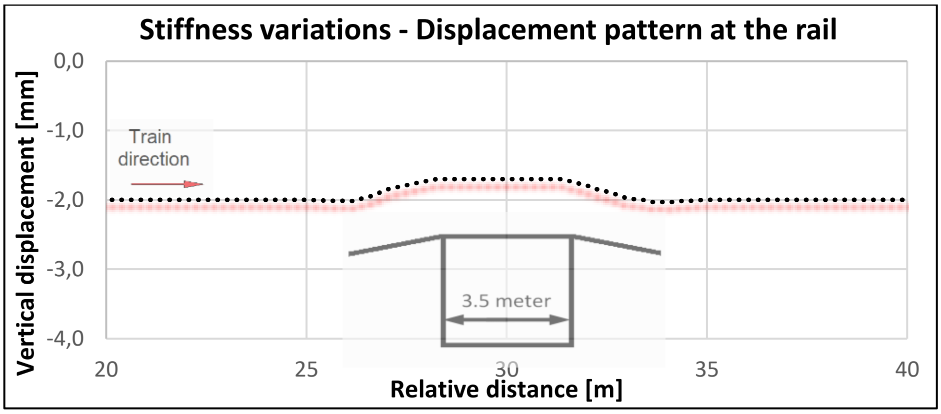


Figure 4.2 - Displacement pattern over a culvert with a short 2.5 meter transition slab

What this displacement pattern suggests, is that the stiffness is nearly unchanged as the «train» reaches the end of the slab. The stiffness then undergoes a gradual change until the culvert is reached. This change however, seems be largest at the middle section of the slab during the trains entry, suggesting that a certain contact lenght is required before the slab is able to provide the necressary support for the train over to the bridge.

When the train is passing the culvert and reaches its exit, a slightly different pattern is seen. The displacement now seem to propagate longer out on the slab compared to at the entry, suggesting that train direction influences the behaviour of the transition slab.

4.1.2 Concrete culvert with 4.0 meter transition slab

For the second concrete case, the exact same procedure for construction and backfilling used here compared to the previous case, but only increasing the lenght of the slab. Similar to the previous case, after the culvert has been built with the railway track, the model is plastified by running eight train passages. Oppose to the previous case, will both the results from a freight train and a passenger train be presented.

In figure 4.3, the displacements during the passage of a «freight train» is presented, showing the displacements of the rail at the free track, above the transition slab and culvert.

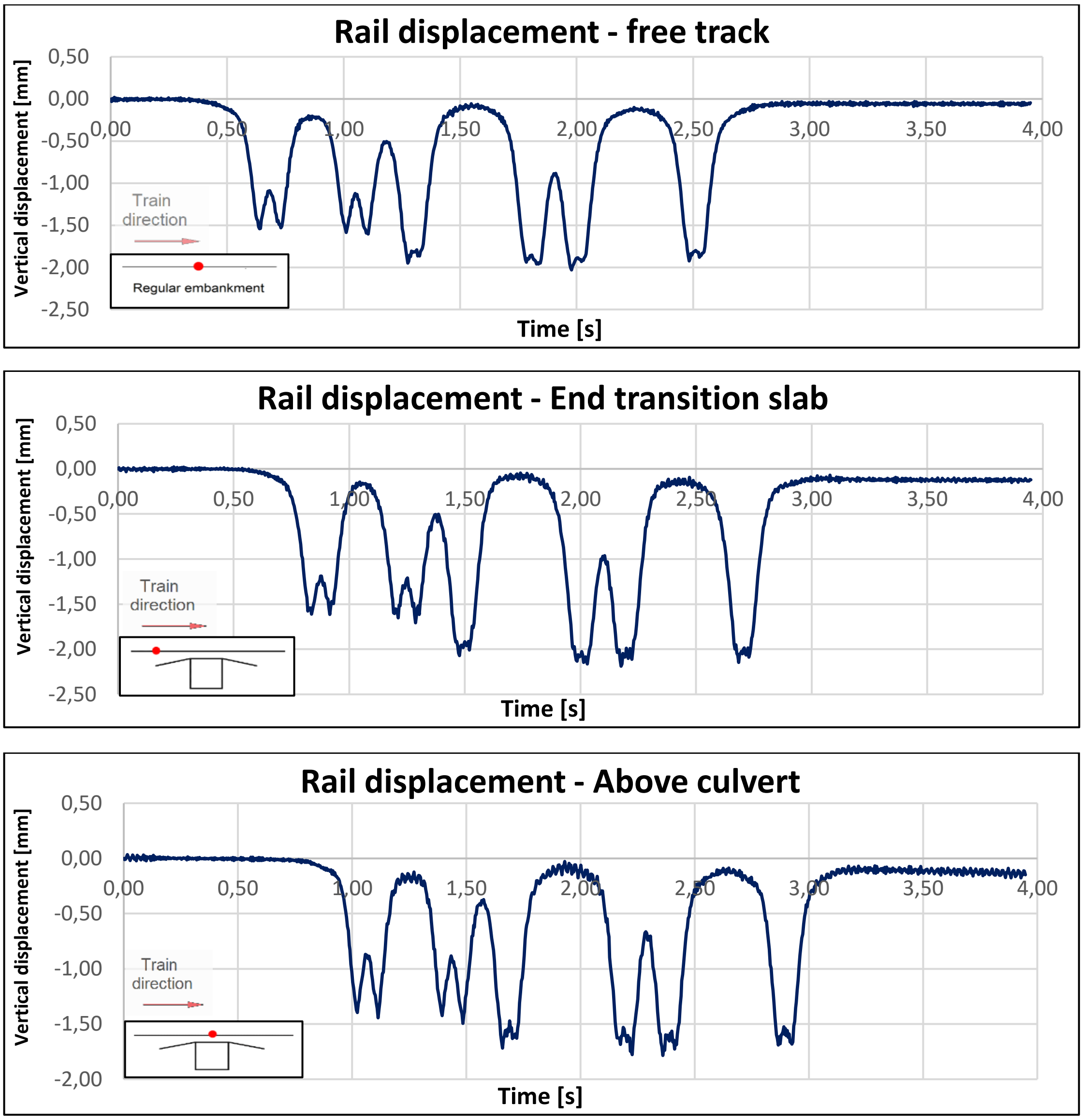


Figure 4.3 - Vertical displacement during the simulation of a freight train traveling at 28 m/s (100.8 km/h) showing results from the rail at the free track, the rail above slab and the rail above the culvert

Similar to the previous case, the problem with convergence is still present as the rail doesn't seem to displace back to zero after the passenge. The problem has its maximum presence over the culvert, with a magnitude of about 0.1 mm.

Oppose to the previous case with the shorter slab, it can now be seen that the maximum displacements increases from about 1.95 mm at the free track to about 2.15 mm at the end of the slab, to then decrease to about 1.7 mm above the culvert. This behaviour of the models suggests that increasing the slabs lenght leads to a bigger «dip» compared to the shorter slab. It is not fully understood if this is related to the model or actually is a realistic occourance, and will be adressed further in chapter 4.1.3.

In figure 4.4, the displacements from train loads presenting a «passenger train» is shown, which includes displacements at the same locations as was selected for the freight train.

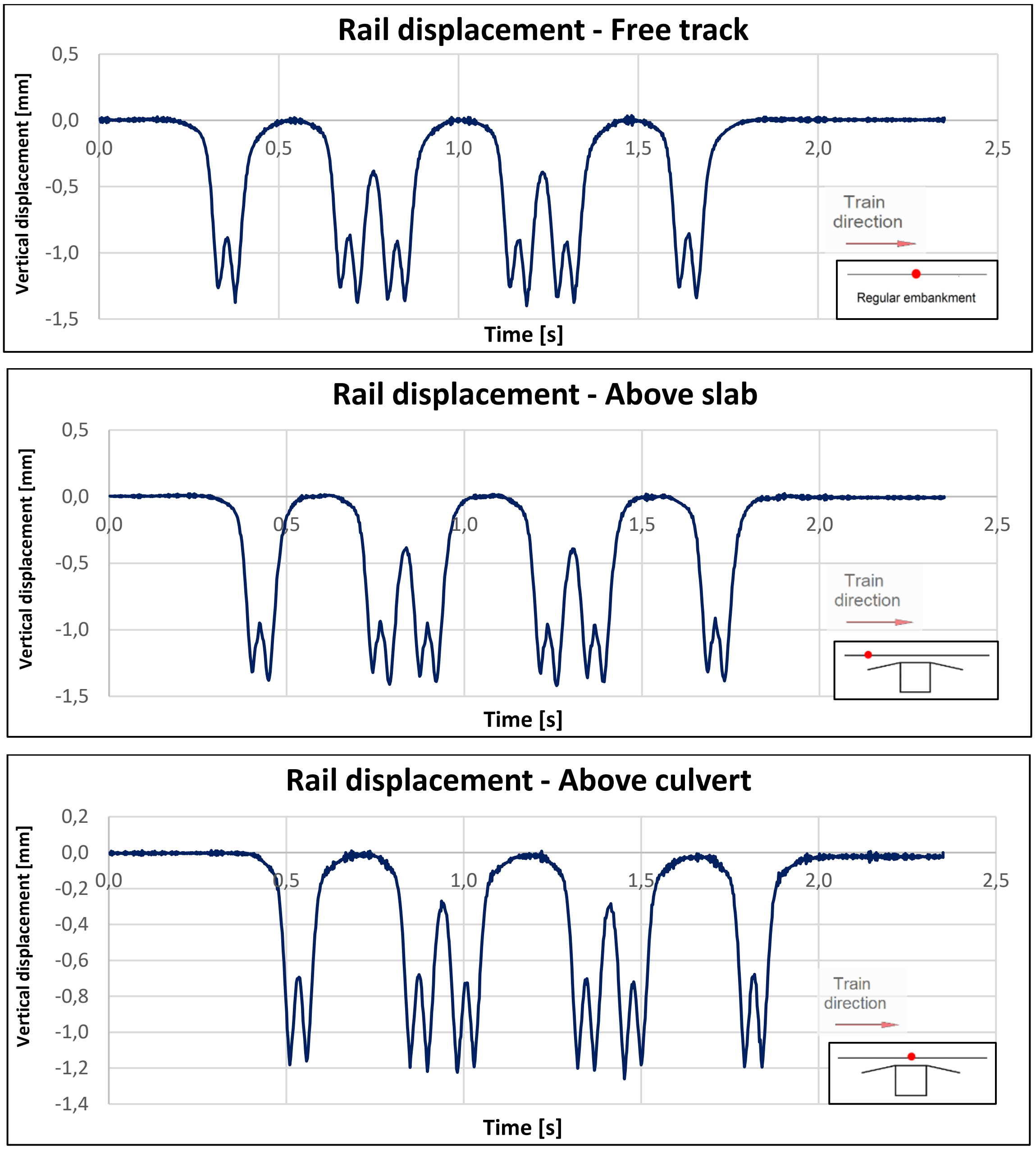


Figure 4.4 - Vertical displacement during the simulation of a passenger train traveling at 56 m/s (201.6 km/h) showing results from the rail at the free track, the rail above culvert and the displacementss over the culvert

These results shows that in the free track, the vertical rail displacements is about 1.35 mm which is increased to about 1.4 mm above the slab, to then decrease down to about 1.2 mm over the culvert. This suggests that similar a tendency as was seen with «the freight train» is present, but to a much smaller degree. When the «passenger train» reaches the slab, the displacements increases slightly which creates a «dip».

In order to better understand how the stiffness changes over the slab, a displacement pattern is deriven from investigating the maximum displacements seen at the rail from using the train wagons as a referance. It should be highlighted again that this displacement pattern does not represent track stiffness, but more how the peak displacements along the rail changes as these train loads are passing the culvert, see figure 4.5.

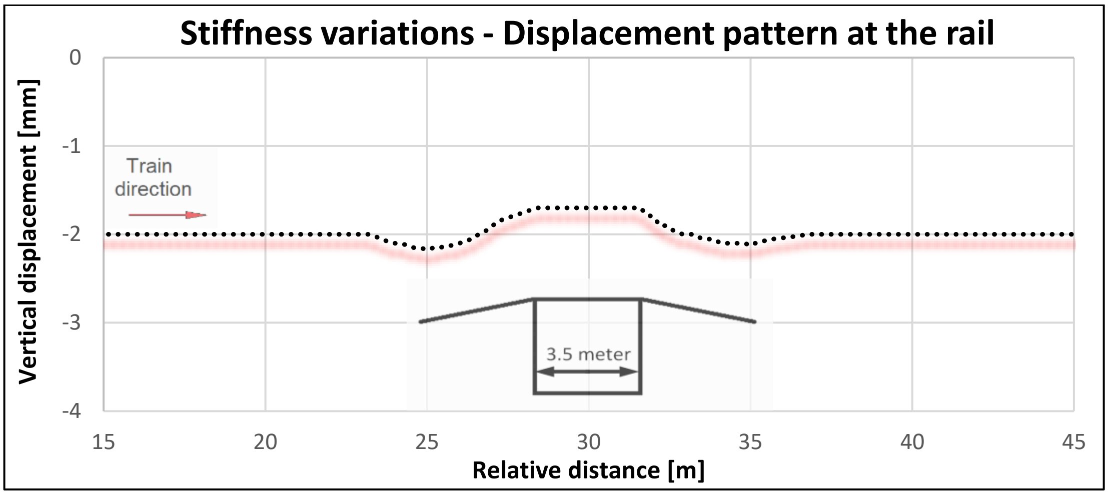


Figure 4.5 - Displacement pattern over the culvert when using the train wagon as a reference

What this displacement pattern shows is that the stiffness at the track decreases as the «train» reaches the end of the slab. The stiffness then undergoes a gradual increase untill the top point of the slab is reached. When the the train loads leaves the culvert a similar occourance is present, the track displaces slightly more at the end of the slab compared to the free track, suggesting that a decrease in track stiffness is present.

4.1.3 Slab behaviour

As has been demonstrated with the previous FEM simulations for these two cases, when increasing the lenght of the slab from 2.5 to 4.0 meters the model then suggests that the stiffness in fact decreases at the entry of the slab. This behaviour is not fully understood as increasing the slab lenght is expected to cause a increase in track stiffness due to increasing the contact lenght at the end to the soil beneath.

The thought is that this can either relate to a real occourance in the complex interaction between moving train loads, backfill and transition slab or the model conditions themselves. It adviced to threat this behaviour as a suggestion from the model.

When investigating the displacement of the slab furher, it can be seen that the longer transition slab actually displaces more than the shorter one in during the train passage, see figure 4.6.

Despite the convergence problem being present in both simulations, presuming that this model shows an representative behaviour, from a practical stand point this can only be explained from the end of the longer transition slab taking either more than 50% of the over-burden pressure or the the influ-ence of the axle spacing being present for the longer slab.

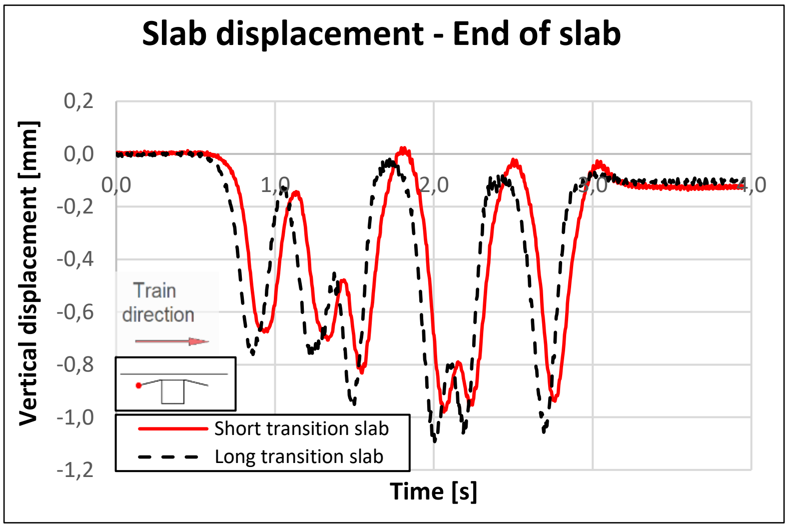


Figure 4.6 – Displacement of the transition slab as the «freight train» passes the transition zone at the entry of the culvert

To better understand this behaviour, figure 4.7 and 4.8 shows results from both simulations when the train has reached the end of the transition slab, showing B) effective normal stresses and C) shear stresses under the slab. These results are deriven from a complete train simulation where each step was saved and were taken when the train was at the locations as shown in figures (i.e fig. 4.7A and 4.8A)

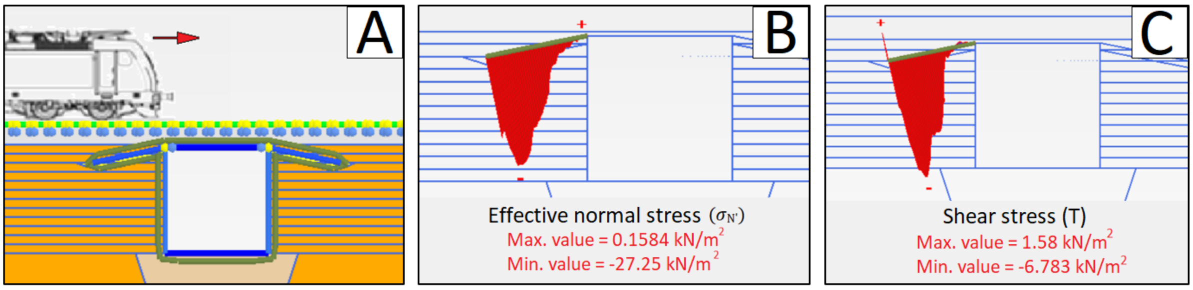


Figure 4.7 – Slab behaviour of the 2.5 meter case, showing A) train location, B) effective normal stress and C) shear stress

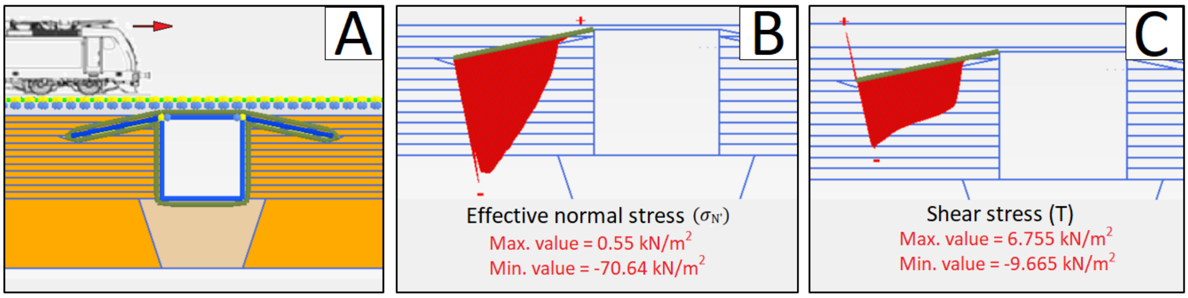


Figure 4.8 - Slab behaviour of the 2.5 meter case, showing A) train location, B) effective normal stress and C) shear stress

What these results suggests is that the stress concentration bellow the slab is over twice as high under the longer 4.0 meter slab compared to the shorter 2.5 meter slab. It is not fully understood if this in fact relates to the model or is a real occourance, but from inspecting the shear stresses it can also be seen that the longer slab has a more evenly distribution but also a higher magnitude than the short slab case.

This suggests that the longer slab is in full contact with the soil oppose to the shorter slab, which might be related to the axle spacing of the train where the influence from both axles will be more present for a longer slab compared to a shorter one, which would've explained the additional displacements seen in figure 4.6.

When replacing the freight train with the passenger train which has lower axle loads, it is seen that the «dip» is almost completely gone. From comparing the displacement patterns of the 2.5 meter with the 4.0 meter long slab, it can also be seen that despite this «dip» seen for the passenger train, the stiffnes transition for the shorter slab is still more sudden, whereas the longer transition slab leads to the a more gradual transition. This suggests that it would be more favourable to use a longer transition slab for a passenger train where there almost is no «dip» present but the transition more gradual.

As a comparison. ProRail in Netherland requires a minimum 4.0 meter long slab for all stiff bridges, a requirment that is regulated as a function of speed. The lenght of the slab is to be increased to 6 meter as the speed reaches 160-200 km/h, and minimum 9 meter above 200 km/h [38].

4.1.4 Comparison to field cases

It has been a challenge to find cases with concrete culverts under normal circumstances (i.e normal foundation and track conditions) where rail displacements has been measured above the slab and culvert. The measurements presented here are done only to provide some sort of basis for understanding what magnitudes one should expect for displacements and stiffness over a stiff culvert.

The first and most important comparison is with the measurements from the culvert at Gouda Goverwelle, Netherland. This case was very special, the 2x2 square culvert was founded on piles down to a stiff pleistoscene sand layer while the free track was founded on a sand fill directly on soft clay. The clay was about 4 meter thick and settled autonoumsly for about 1.0 mm pr. month, see figure 4.9.

The full-scale field test included a comprehensive field survey of the site after a request from Pro Rail, which at the time wanted to assess the extermely high maintenance frequency of this transition zone [3,4,8].

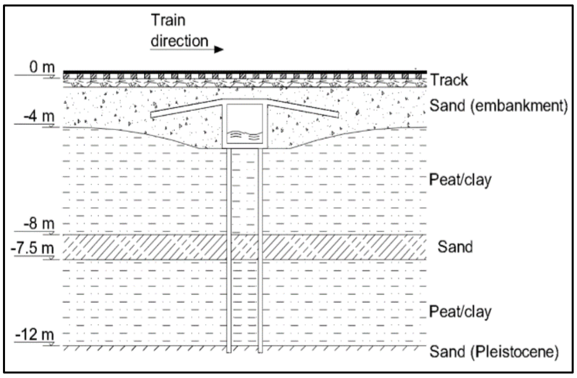


Figure 4.9 – Intreprented soil profil for the «Gouda Goverwelle culvert», showing that the railway track was founded on soft clay but culvert on piles(Modified after Coelho. B, 2011)

The culvert was primarily built as a guideway for water while the tack was a part of a four track line with trains traveling in specific directions, see figure 4.10. The operating speed of the line was 140 km/h with mixed traffic. More background information on this culvert can be found in [8] or chapter 1.1 in Appendix C.

The field survey at this culvert was divided into two parts, a static monitor programme and two individual dynamic tests. The programme was carried out in order to get at better understanding between the correlation between maintenance and deterioration of the transition zone. More information on the monitor programme can be found in chapter 3, Appendix C.

Figure 4.10 - The «Gouda Goverwelle» culvert during the passage of a passenger train (Modified after Hölscher. P, Meijers. P, 2009)

What will be presented here is the measurements performed in the first dynamic test, about 2 months after the previous tamping cycle which according to Coelho [3], was performed before the autonomous settlement had reached and affected the measured sleeper at the free track.

The measurements were carried out by mounting geophones on the wodden sleepers at the free track and above the culvert. The results from the geophones were intreprented by Bruno Coelho in 2011 [3], and presented in figure 4.11.

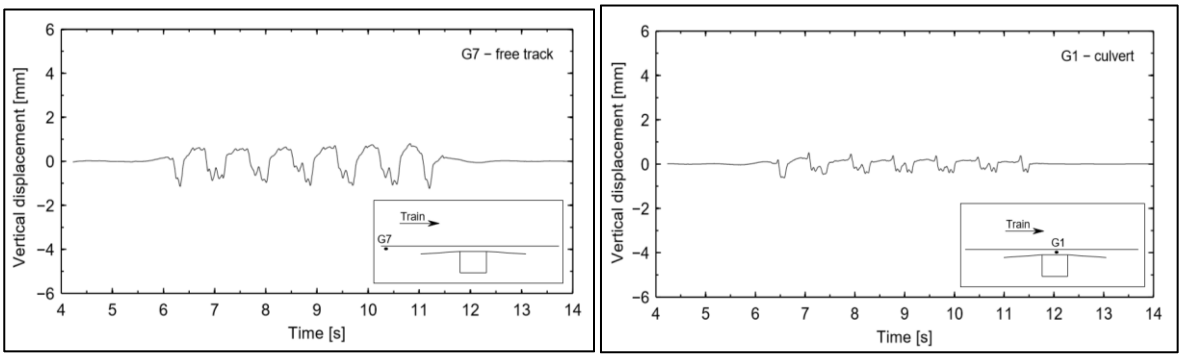


Figure 4.11 – Measurements at the transition zone of Gouda Goverwelle, showing results from the free track and above culvert (Modified after Coelho. B, 2011)

The displacement peaks found from these measurements suggested that the track settles about 1.5 mm in the free track, while 0.7 mm above the culvert. Coelho also suggested that the apparanet stiffness above the culvert was about 1.5 times higher than at the free track. More on these measurements can be found in [3,8], but also see chapter 5.1.1 in appendix C.

Similar magnitudes was reported in a paper from Davis and Li in 2005 [6], where the transition zones of four stiff railway bridges were studied. Background information on these bridges are somewhat limited, but all these referance cases reported a track modulus about the double of the modulus in the free track for all four cases, see figure 4.12.

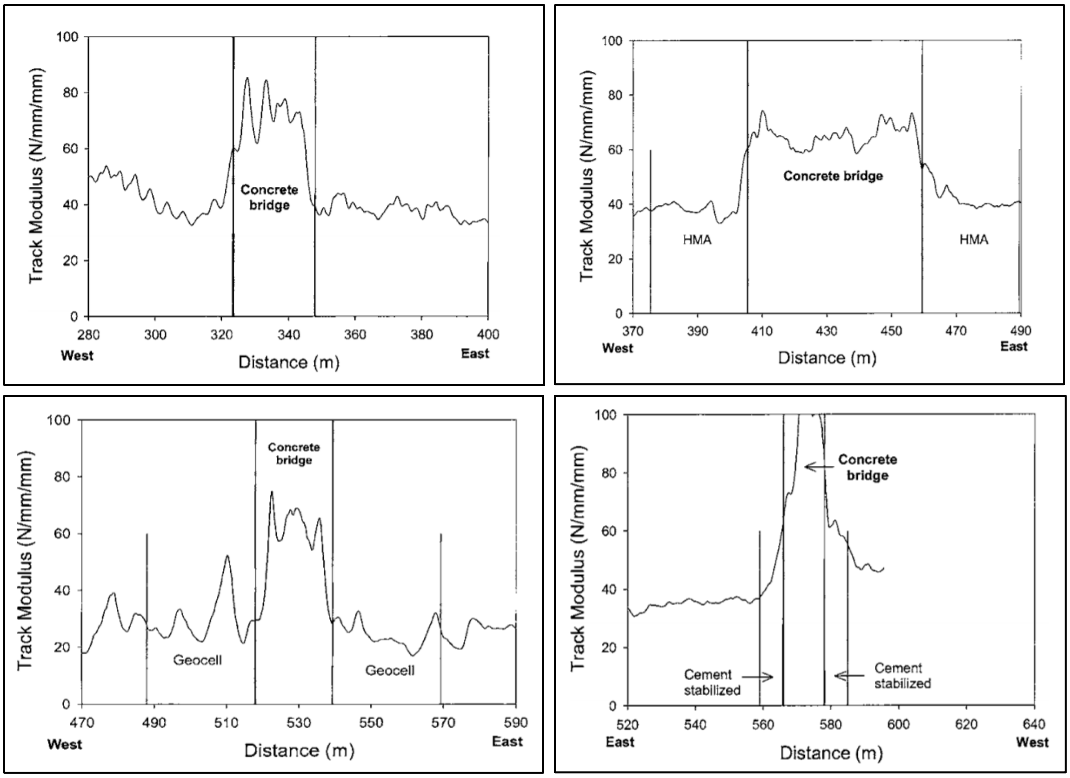


Figure 4.12 – Track modulus measured at four concrete bridges from the US (After Li. D and Davis. D, 2005)

The information these bridges provide despite the background for then being limited, is that a track modulus seen on a typical stiff concrete bridge is about twice compared to a free track. These measurements also correlate well to what purposed by Coelho at Gouda Goverwelle.

When thinking in terms of a small concrete culvert founded on the same ground conditions as the free track (expect for the foundation), the difference in track stiffness is expected to be lower than these referance cases. In the two simulations with the freight train was it seen that the maximum displacements at the free track was about 1.9-2.0 mm, and decreased to about 1.7 mm above the culvert which is though of as being reasonable.

There is no basis for specifying exactly how stiff a railway track really is over a concrete culvert without having field measurements, but this is thought to be on such detailed level that it is not considered important for this project, and the prerequisite of maintaining a track stiffness that is higher over the culvert compared to the free track is thought of as being within reasonable limits.

4.2 Stiffness variations over flexible steel-soil composite culverts

In this section aspects of track stiffness and interaction forces between flexible steel-soil composite culverts are investigated for the two predefined cases of chapter 3.3. After these culvert has been constructed and the effect of hardening comes into play (after some train simulations), the elastic behaviour of the model are then thought of as being representative for a long-term field condition.

It was decided to stick with the minimum requirement of 1.0 meter soil cover for both cases in order to investigate «a worst case scenario». It was also decided to use the load configuration of the passenger train only for the pipe arch since that case was expected to be worse than the circular case in terms of track stiffness.

4.2.1 Pipe arch

This culvert is built from using the predefined backfill materials of chapter 3.1, the structural culvert properties of chapter 3.3.1, and by following the backfill methodology of chapter 2.4.1 to the point.

Constructing this culvert involves first preparing a foundation, then building the culvert on a uncompacted bedding followed by stepwise backfilling. Each backfill layer is placed alternately in 0.3 meter thick layers and compacted with light and medium line loads. When the backfill reached the crown level and the maximum displacement was present, the maximum deformation of this culverts crown was 35.3 mm, see figure 4.13.

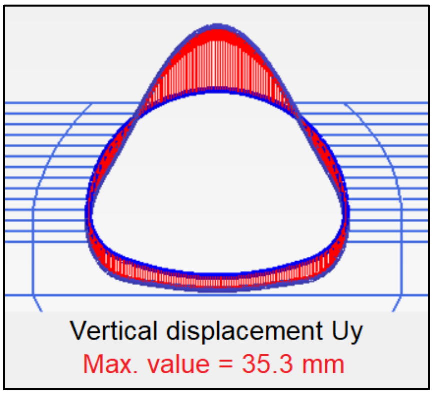


Figure 4.13 – Maximum displacement seen at the top point (crown) of the culvert

According to SDM [13], the crown during backfilling are not allowed to displace more than 2% of the culverts total span lenght, a criteria which this culvert furfills. It can also be seen that this culvert deforms significantly less than the Enköping example from chapter 2.4.3, despite having a larger span and using heavier compaction forces. This is related to this culverts steel thickness of 7.0 mm (oppose to 2.95 mm for Enköping), which makes this culvert much stiffer despite spaning longer.

After the backfill is completed and the railway track been built, the stiffness of the sand used in the inner zone are increased to a unloading stiffness of 600 MPa with a cohesion of 25 kPa. This was a necessity in order to catch any realistic track stiffness during the simulation, see chapter 2.5 and 3.1.

As was discussed in chapter 2.4.4, when this model plastifies and hardens from the passing train loads, the bending moments are expected to change insignificant compared to the change in axial forces. This is a behaviour that agrees well with what has been reported from several field measurements [18,21], and as shown in figure 4.14, the change in bending moments after 8 train passages is very small.

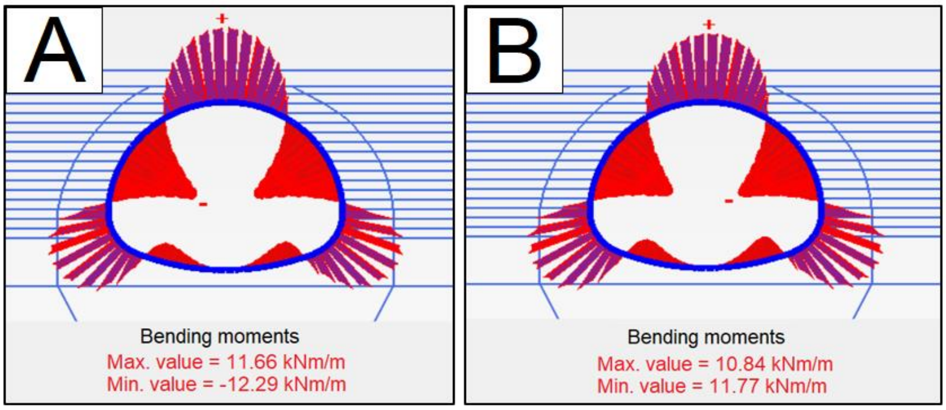


Figure 4.14 – Bending moments after A) construction finish and B) after eight train passages (i.e when the models elastic stage has been reached), showing that the change is insignificant

In figure 4.15 a similar comparison is shown for the internal thrust forces, which shows that the thrust forces increases as the model hardens and reaches its «long-term conditions» (i.e elastic state).

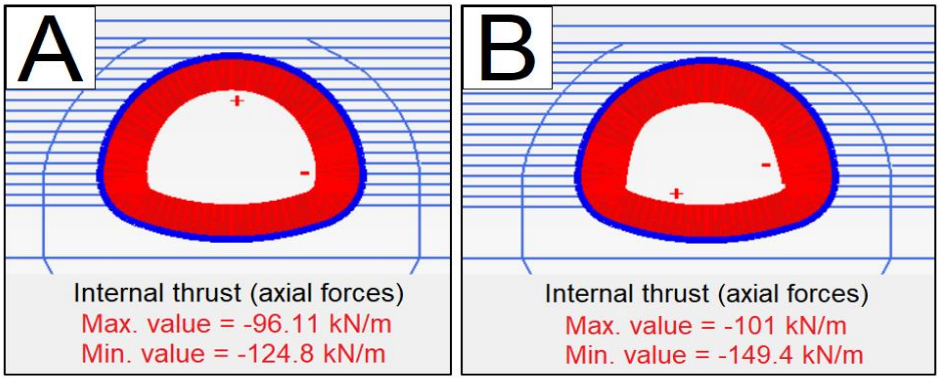


Figure 4.15 – Internal thrust forces after A) construction finish and B) after eight train passages, showing that the structure takes more in ring compression as the model reaches its «long-term» condition

These results verify that the thrust forces increases as the model hardens despite changing the material properties of the sand after compaction. This demonstrates that these structures ends up with taking more in ring compression as the model reaches its «long-term» state, a behaviour which agrees well with reported field behaviour that suggests that flexible culverts normally takes more of the over-burden pressure in ring compression with time [16,18,20,21].

After constructing these culverts and reaching the models «elastic state», a simulation of a freigh train is performed by using the specifications described in chapter 2.3.2. In figure 4.16 the vertical displacements of three preselected locations from the model are presented, that is at the rail in the free track, at the rail above the culvert and at the crown of the culvert.

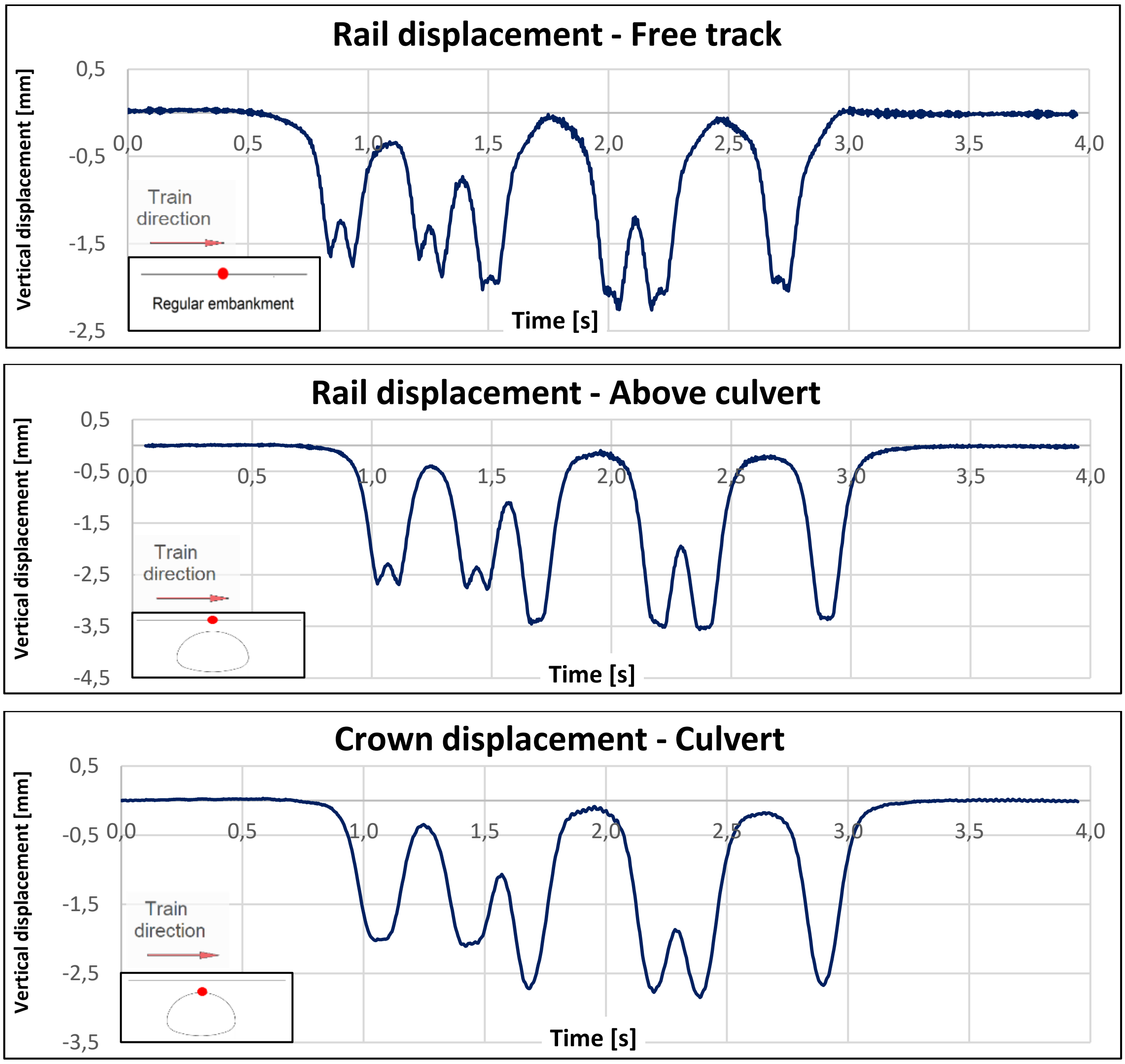


Figure 4.16 – Vertical displacement during the passage of a «freight train» traveling at 28 m/s (100.8 km/h) showing displacements from the rail at the free track, the rail above the culvert and the crown of the culvert

Oppose to the cases with concrete the convergence problem is now not present any longer (discussed in chapter 2.3.4) despite not using damping. This suggests that the convergence problem also relates to the interaction with the culvert, not only the soil model.

These results shows that in the free track, the vertical rail displacement seen from the front bogie of the locomotive is about 1.75 mm, but is about 2.2 mm under the wagons. As been seen from all the other simulations, the largest displacements occours between the two wagons due to their short axle spacing and heavier axle loads. When these train loads reaches the culvert, the maximum displacement of the crown is about 2.7 mm (fig. C), causing an increase in rail displacement from 2.2 mm to about 3.5 mm over the culvert which demonstrates that a stiffness reduction at the track is present.

In figure 4.17 results from a similar train simulation is presented, but now replacing the configurations of the freight train with the configurations of the passenger train of chapter 2.3.1.

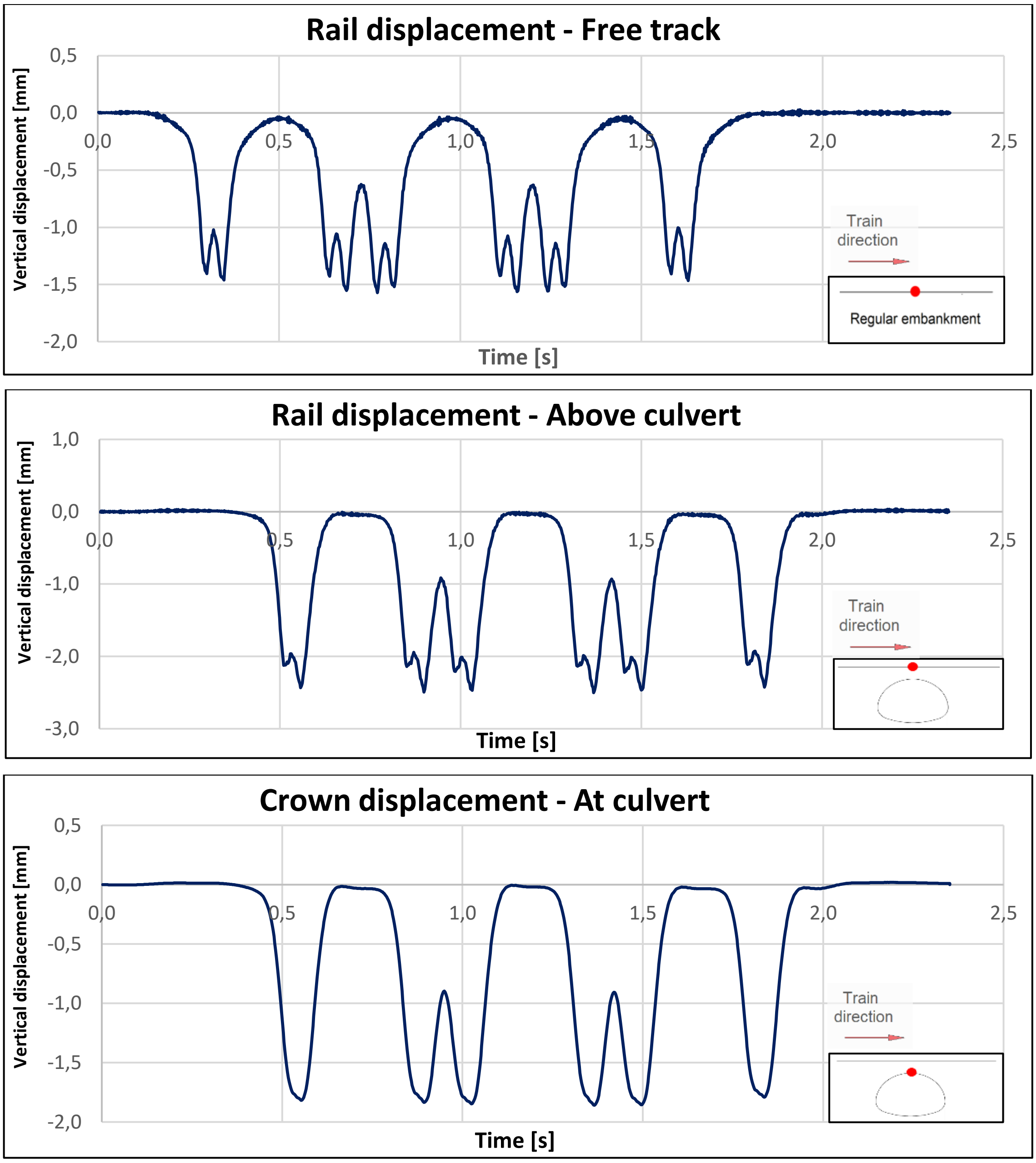


Figure 4.17 - Vertical displacements during the simulation of a passenger train traveling in 56 m/s, showing displacement results from the rail at the free track, the rail above the culvert and the crown at the culvert

Similar to what was seen in the previous simulation, the rail displacement is highest above the culvert due to the large displacement of the crown. At the free track, the maximum displacement is about 1.6 mm, and increases to about 2.45 mm above the culvert. The displacement of the crown is about 1.8 mm.

From comparing the results from these two simulations it is found that the stiffness variations between free track and above culvert are nearly proportional in both cases. The maximum peak displacement at the rail increases with about 37% for the freight train and about 34.7% for the passenger train.

From inspecting the maximum peak displacements at every point along the rail and using the displacements at the wagons of the freight train as a referance, it was possible to create a displacement pattern over the transition zone of this culvert, see figure 4.18.

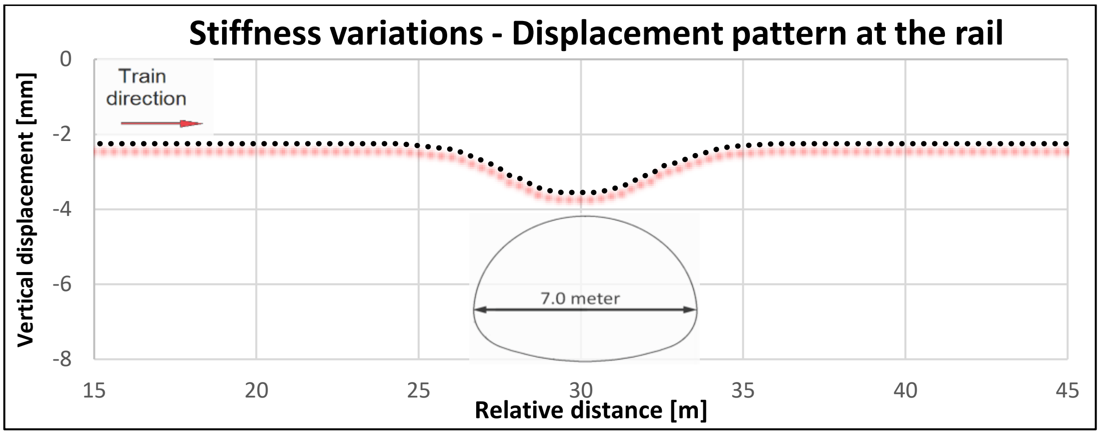


Figure 4.18 – Displacement pattern over the culvert during a train passage, using the train wagon as reference

What this displacement pattern shows is how the maximum peak displacement seen from the wagons propagates along the rail, which indirectly relates to track stiffness. What this displacement pattern suggests is that the track stiffness starts to decrease at a relative distance of 25 m, about 1.5 meter away from the culvert which gradually decreases towards the crown of the culvert where the maximum rail displacement is present (i.e lowest track stiffness).

This behaviour suggests that a «transition zone» for a flexible steel-soil composite culvert is present already before the axles of a train reaches the culvert where the track will undergo a gradual decrease in stiffness until the axles has reached the crown, where it is lowest. Seen from the diagramme, the pattern suggests that the stiffness variations over this culvert is nearly symmetrical.

4.2.2 Circular pipe

For this specific case the exact procedure as used for the pipe arch is followed. This means, first will there be built a foundation, then a bedding built with the culvert which is followed by backfilling and compaction alternately in 0.3 meter thick layers, using both light and medium comapction loads.

When the backfill reaces the crown of this culvert however, an interesting observation is taken from that it is the lower section that displaces the most and not the crown, see figure 4.19. It has not been found any field cases of flexible culverts with circular profiles to confirm this behaviour, but regardless of the additional deformations seen at the bottom it passes SDMs criteria of not deforming more than 2% of its span [13].

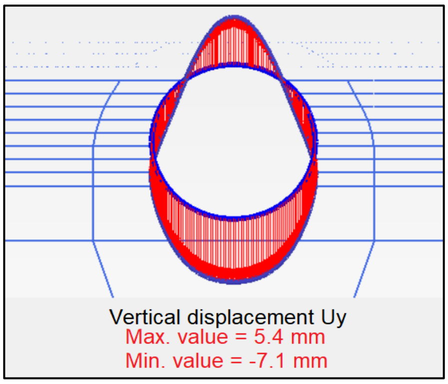


Figure 4.19 - Maximum displacements seen at the top and bottom point of the culvert

Similar to the previous case, the bending moments after construction and eight train passages are compared (i.e «short-term» and «long-term» conditions), see figure 4.20.

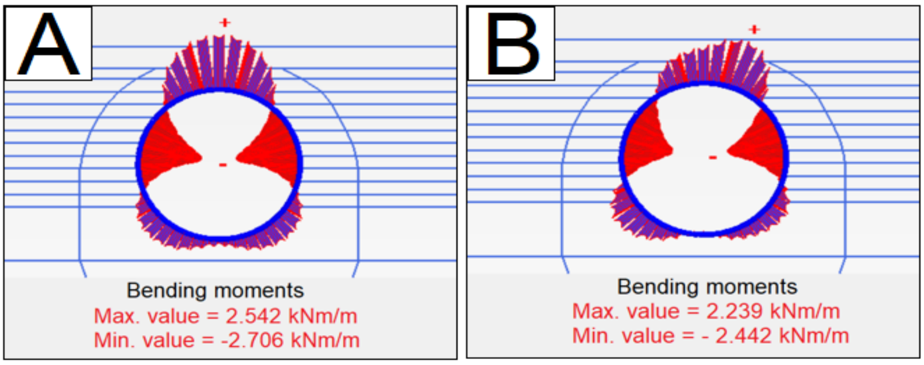


Figure 4.20 - Bending moments after A) construction finish and B) after eight train passages

As was expected beforehand, the magnitudes of these bending moments changes insignicant when the model hardens. In this case oppose to the pipe arch however, it can now be seen that the sign of the bending moments at the crown has slightly shifted towards right, suggesting that the «long-term» condition were affected by the trains loads traveling direction. According to several authors, it is well known and accepted that service loads may change the sign of the bending moments in cases where the soil cover is especially low [13,16-18,20].

Similar to the previous case, the axial forces before and after hardening of the model are compared, and shows the exact same tendency as previously seen for the pipe arch. The axial forces increases as the model hardens (i.e the culvert takes more in ring compression), see figure 4.21.

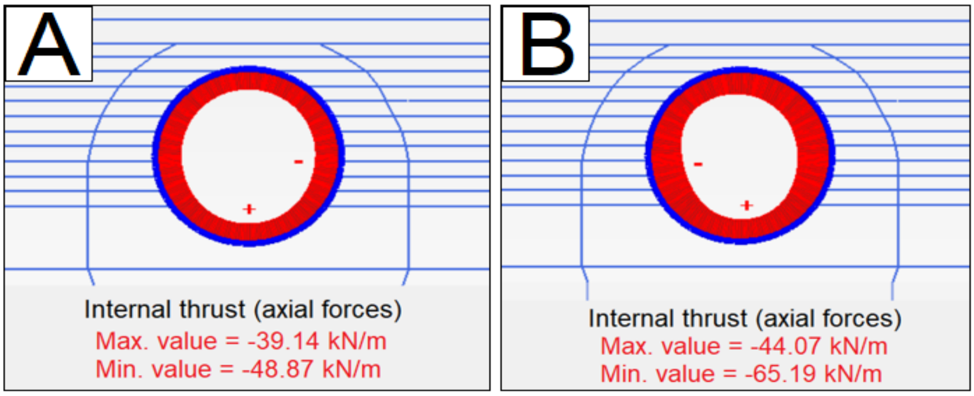
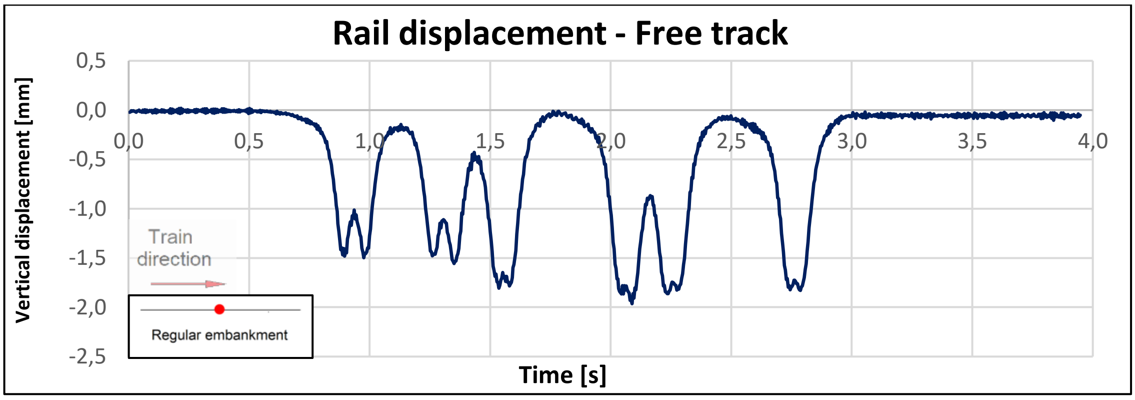
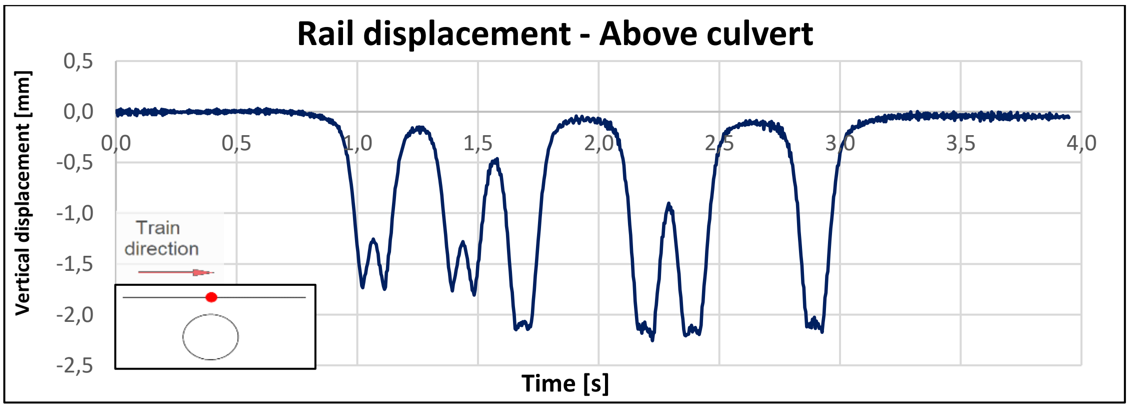


Figure 4.21 - Internal thrust forces after A) construction finish and B) eight train passages, showing that the structure takes more in ring compression as the model hardens

For this case, only simulations using the «freight train» were performed. After constructing the culvert and plastifying the model with eight train passages, the model was considered to behave close to elastic. In figure 4.22 the vertical displacements during a «train passage» is shown for the rail at the free track, the rail above the culvert and the culverts crown.





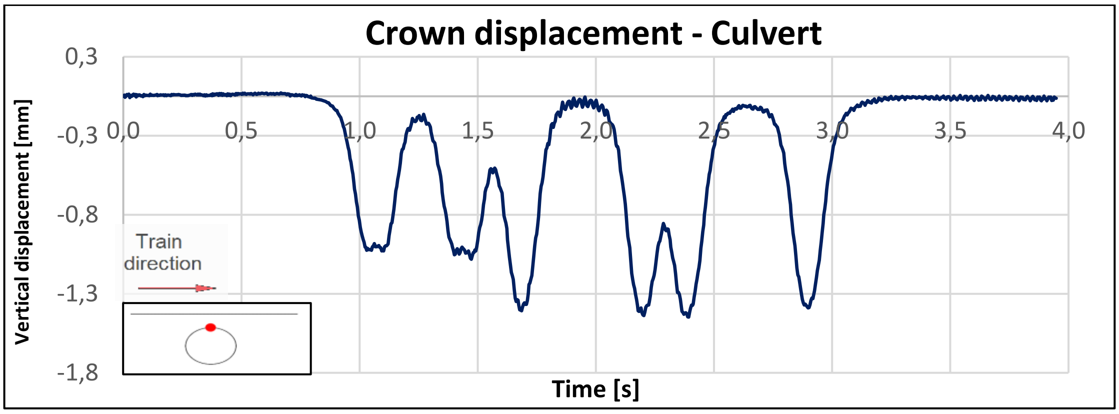


Figure 4.22 - Vertical displacement during the simulation of a freight train traveling at 28 m/s (100.8 km/h) showing results from A) rail in the free track, B) rail above culvert and C) crown displacement at culvert

Oppose to the previous case it can now be seen that the model hasn't convergated completely as the tendency of persistent plastification (from chapter 2.3.4) is still present at the free track. The rail doesn't displace back to zero after the last train load has passed the measuring point within an order of 0.05 mm.

From these plots it can be seen that the rail displaces about 1.5 mm when the front axle of the locomotive hits the measuring point in the free track and increases to about 1.9 mm when the wheels of the wagons are passing. Compared to the previous case, the displacements here at the free track is slightly lower which is related to the difference in this models total height.

Above the culvert only a slight increase in peak displacement is seen at the rail, going from 1.9 mm at the free track to about 2.2 mm above the culvert. It is also seen that the crown displaces for about 1.4 mm. Comparing this to the pipe arch case, it was seen that 2.8 mm crown displacement correspondend to about 1.3 mm increase in rail displacement. For this case however, the crown displacement is 1.4 mm (half) but only causes an additional increase in rail displacement of 0.3 mm (a fourth of the pipe arch).

This suggests that the displacements of the rail is not porportional to the displacements of the culverts crown, where double the displacement of this case (2.4 mm), led to four times higher displacement seen at the rail in the previous case. This might related to shape, but suggests that the track stiffness is nonlinear in relation to deformations of flexible culverts.

In figure 4.23 the displacement pattern over this culvert are presented, showing the maximum displacements along the rail from using the train wagons as a referance.

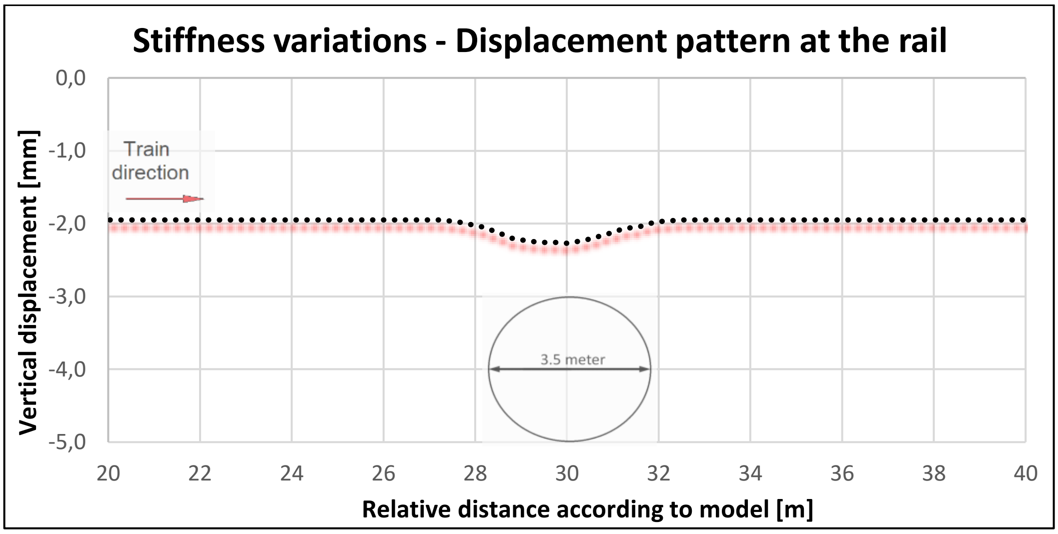


Figure 4.23 - Displacement pattern over the culvert during a train passage, using the train wagon as reference

What this plot shows is that the maximum peak (i.e the section with lowest stiffness) is also present above the culverts crown but oppose to the previous case, it doesn’t distribute as symmetrical as was seen for the pipe arch. This might relate to the change in sign seen at the bending moments in figure 4.20 which were related to the trains loads traveling direction.

The influence traffic direction has on the behaviour of flexible SSCB has been under research in Poland for several years, and according to Machelski, only when a veichle has driven back and forth over a flexible culvert can a load sequence be considered completed and the expected elastic behaviour seen at the top arch of these structures be maintained. This behaviour has been confirmed by several full-scale quasi-static field tests in Poland, both under railway traffic and road traffic [14,15,39].

Despite this being the case, the overall trend of this transition zone shows great similarity to the pipe arch that the stiffness gradually decreases untill the train reaches the crown of the culvert.

4.2.3 Comparison to three field cases

In this section three field cases are selected as a comparison and validation of the two FEM simulations involving flexible steel. It has been a challenge to find field cases where either rail displacements or veichle behaviour has been measured over these structure. Nevertheless, there are several cases where crown displacements has been measured during train passages, which gives ground for comparisment.

The first preselected case is the «Rörbru i Märsta» culvert built at the Arlanda express line, Sweden. The Arlanda express line was designed for train speeds up to 170 km/h, and is operated by mixed traffic. The culvert was a part of a series of tests performed by Kungliga Tekniske Högskolan (KTH) concerning reasearch on steel-soil composite bridges for highspeed railways, see figure 4.24.

Figure 4.24 – The «Rörbru i Märsta» culvert during a X-52 train passage at 170 km/h (Modified after Andersson. Et. al, 2012)

According to Andersson and Karoumi [19], the corrugation of the steel at this culvert was 150x50 mm with the thickness of 5.5 mm. The culvert was built as an closed vertical ellipse, spanning about 3.75 meter long and was 4.1 meter high. The culvert was built under 1.7 meter soil cover. In 2010, crown displacement measurements with strain gauges showed a vertical deflection of 0.5 mm during the passage of a X-52 commuter train. These types of trains were traveling at 170 km/h and had axle loads of about 18 tonn [19].

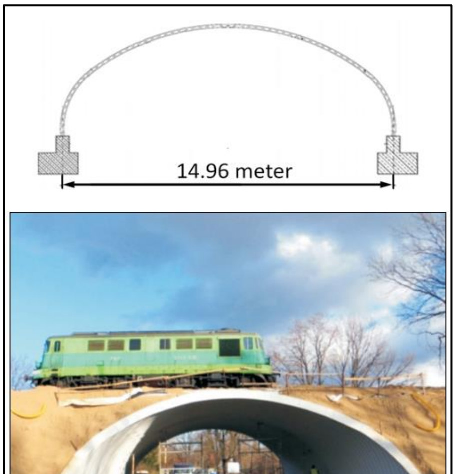


Figure 4.25 – Position of the locomotive during maximum measured displacement of the crown (Modified after Machelski. C, 2014)

The second referance case is a superspann half arch from the railway line Świdnica, Poland. The culvert was a part of a instrumented full-scale quasi-static test, reported by Machelski in 2014 with the purpose of evaluating the structures response when a veichle is driving back and forth in a quasi-static way [14].

The superspan half arch was founded on concrete foundations, spaning 14.96 meter long while being 5.25 meter high. The soil cover seen from the railway track was about 1.6 meter, primarily built of sand and a ballast bed, see figure 4.25.

The culvert was instrumented with steel plates of the corrugation type «SuperCor» (380x140mm) having 7.0 mm thick steel. The quasi-static tests were carried out using a single ST43 locomotive, driving back and forth over the culvert at 5-20 km/h. The locomotive was instrumented with tripple bogies (i.e six axles in total), having an axle load of about 19.8 tonn. Maximum displacement of the crown during this test was measured to be in an order of 2.8 mm [14].

The third referance case is the large span pipe arches «Torppari» from Finland, built as underpassings at the Hanko-Karjaa railway line in 2014 [40]. The field instrumentation was carried out under the support from Tampere University of Technology with the purpose of analyzing the effect of a railway structures presence on a flexible culvert in order to gain knowledge about its techical performance under the presence of train loads.

The largest pipe arch of these underpassings had a horisontal span of about 7.78 meter and was about 6.87 meter high. This culvert was built with a corrugation type MP 200x55. The steel thickness for this culvert was not reported. According to design drawings, the height of cover seen from the bottom of the sleepers was 1.4 m, see figure 4.26.

There is limited available information on the passing trains during these tests, but according to Asp [40], the measurements were carried out under the presence of a freight train traveling at normal speed with axles weighing about 20 tonns. The maximum displacements measured at the crown during these passages seen for the largest culvert, was about 0.85 mm [40].

Figure 4.26 – The «Torppari» culvert underpasses at Hank-Karjaa line (Modified after Asp. O, 2014)

In table 4.1 a summary of these field tests are presented together with some technical background information. Whats important to highlight about these measurements is that they are not ment as validations due to all the unknown variables associated with them. No information on the rails and limited information on the trains are factors that for example influences the displacement of the crown.

Table 4.1 – Results and background information on the three preselected referance cases from field

Culvert	Profile	Span [m]	Soil cover [m]	Steel profile [mm]	Crown displacement [mm]
Märsta	Vertical ellipse	3.75	1.7	150 x 50 x 5.5	0.5
Świdnica	Half arch	14.96	1.6	380 x 140 x 7	2.8
Torppari	Pipe arch	7.78	1.4	200 x 55	0.85

A similar table is created for the two model cases, showing two results from the pipe arch and one result for the circular pipe, see table 4.2.

Table 4.2 – Results and background information

Culvert type	Train type	Span [m]	Soil cover [m]	Steel profile [mm]	Crown displacement [mm]
Pipe arch	Freight	7.0	1.0	200 x 55 x 7	2.85
Pipe arch	Passenger	7.0	1.0	200 x 55 x 7	1.75
Circular	Freight	3.5	1.0	200 x 55 x 4	1.4

From the field case from Poland the measured diplacement with 1.6 meter soil cover was 2.8 mm, which compared to PLAXIS may suggest that the stiffness purposed by the model is to soft. But a note should be made on the corrugation used at Świdnica was SuperCor (380x140), which oppose to MP (200x55) is known to be much stiffer in bending despite using the same steel thickness. SuperCor compared to shallower corrugations such as 150x50 is for example nine times stiffer in bending [41].

The culvert from Torppari is has somewhat closer characteristics to the pipe arch in the simulations, where only 0.85 mm crown deformation was measured during the train passage. Indeed, it might be argued for that the slightly heavier train in combination with the lower soil cover is not enough to account for additional displacements seen in PLAXIS, but this will then apply on a detail level less than a millimeter and therefore not thought of as a problem with respect to the overall aim of this project.

The Märsta culvert has a similar shape to the circular profile in PLAXIS, and was measured with 0.5 mm crown deformation in field oppose to 1.4 mm in the simulation. This comparison shows better agreement compared to the pipe arches.

The soil cover at Märsta was 1.7 meter, the axle loads about 18 tonn and the steel thickness of the culvert 5.5 mm. The circular culvert in PLAXIS had 1.0 meter of soil cover, 22.5 tonn axle loads and a steel thickness of 4.0 mm. The corrugation is smaller for the Märsta culvert but is compensated for the thicker steel, which according to SDM leads to a stiffer culvert than the circular pipe in PLAXIS [13].

With this it can be concluded that the «long-term» stiffness properties given for the sand surrounding these culverts in PLAXIS are within reasonable limits, suggested from this comparison. It might be argued for that the model is slightly softer than what one would expect in field, but this is on a detail level.

When speaking in terms of track stiffness, these models have suggested that the track stiffness seen over flexible steel culverts is related to shape, span, backfill material, soil cover but also traffic loads. The predefined pipe arch for example, was given a thicker steel than the circular case but clearly demonstrated to be softer than the smaller pipe when the train loads were passing.

It was already suggested from the flexibility numbers in chapter 3.3.1 and 3.3.2 that the smaller pipes shorter span compensated for its thinner steel, suggesting that this quantity may relate to track stiffness aswell.

5 Stiffness variations, irregularities and maintenance

It is widely accepted in railways that variations in track stiffness and irregularrities leads to increased wheel/rail interaction forces which eventually leads to a higher deterioration rate of the track [2-7, 39,42]. Dahlberg suggested that there will always be stiffness variations present on a railway track founded on ballast and that any stiffness variation induces an irregular wheel/rail contact force which over time, contributes to differential settlements and eventually can create unsupported sleepers [7].

This general way of thinking is often presumed for all types of transition zones which is the background for this chapter, to investigate and discuss if this correlation applies for railway culverts but also to provide an idea of how severe these problems are. The chapter starts with representing ROGER 1000 data collected for six preselected cases related to this project, then compares these measurements to obser-vations and presumptions often found in the litterature.

5.1 ROGER 1000 measurements

This chapter summarizes the findings of Appendix B which exclusively concerns ROGER 1000 data for six preselected culverts. The only measurement from ROGER 1000 used in this project is the vertical track deviation data after it was found that investigating maintenance frequencies held to much uncertainty and a better approach was found to be the study of several ROGER measurements.

«ROGER 1000» is the measuring car used by Bane NOR in their yearly track inspection, a quality controll which is performed twice a year on every railway line in Norway, see figure 5.1

Figure 5.1 – The measuring car «ROGER 1000» used at the entire railway network in Norway (Source: www.BaneNOR.no)

ROGER 1000 is instrumented with several lasers, displacement tranducers and a camera system in the front which regulary scans the rail and measures veichle body accelereations at given intervalls, and then ties these registrations to a certain location (i.e kilometer). The spatial car body displacement measured from the displacement tranducers are compared to a target profile, which returns the vertical track deviation based of the deviation the measurement has compared to the target profile [26].

Any deviation found by ROGER 1000 outside the allowable limit set for a particular line are typically postnoted as a «workorder» with a certain priority. If the deviation is big enough then these workorders are set to «critical», which means that maintenance will be carried out consecutively. More information on ROGER 1000 and the measuring system can be found in [26], also see chapter 1.2 in Appendix B.

These measurements are to rough to be used for assessing track stiffness but provides good information on the deterioration rate of a track section if enough measurements are collected. Whats convenient with these measurements is that they are unfiltered (i.e not intreprented by someone), which was not found to be the case when investigating tamping registrations, see chapter 2.2 in Appendix B for an example.

5.1.1 Location of the six preselected culverts

In total three culverts made of rigid concrete and three culverts made of flexible steel were preselected. These culverts are located around the middle and southern part of the country, see figure 5.2.

The prerequisite for selecting these culverts was that the track as a minimum was founded on ballast over them and that cases with axuiliary rails, sharp curves or anything that could affect the track stiffness was avoided, see chapter 1.1.1 in Appendix B.

When chosing these culverts it was in some cases a challenge to find both design drawings and locations furfilling the criterias set for the conditions surrounding these culverts. From that rigid concrete culverts are not expected to behave that different from each other compared to flexible steel culverts, only ROGER 1000 photos was used when evaluating the concrete cases due to limited design drawings available. For the preselected steel culverts however, only cases with design drawing available were considered.

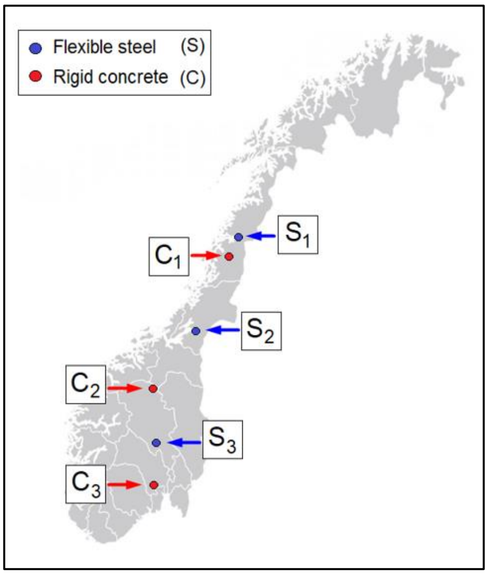


Figure 5.2 – Location of the six preselected culverts

Exact locations of the six preselected culverts are given with coordinates in table 5.1.

Table 5.1 – Location of the six preselected culverts with coordinates

Name	Symbol	Latitude [°N]	Longitude [°E]
Sjånes culvert	S_1	66.27127342° N	13.98277598° E
Holme culvert	S_2	63.80950167° N	11.45683034° E
Viker culvert	S_3	61.33532932° N	10.27328908° E
Svenningdal culvert	C_1	65.44200662° N	13.39377466° E
Vinstraveien culvert	C_2	62.51435352° N	9.60784839° E
Jarenhaugen culvert	C_3	60.00650904° N	10.02493990° E

Local conditions of the six preselected culverts were assessed with ROGER 1000 photos taken by the veichle during its regular 0.5 meter scan. In figure 5.3, ROGER 1000 photos for the three preselected steel culverts Sjånes, Holme and Viker is presented, showing that ROGER 1000 photos gives an good overview on the local conditions sorrounding these culverts.

Figure 5.3 – ROGER 1000 photos at the site conditions at S_1) Sjånes, S_2) Holme and S_3) Viker culvert

From the photos it is seen that Sjånes and Holme is built in relative slack curve while Viker is built in a completely straight section. This are variables one should have in mind when intreprenting the ROGER 1000 results as it was found that curves may cause nonsymmetrical deviations. All three steel culverts had the cross-section of pipe arches, spaning between 4-6 meter with a soil cover between 1-1.4 meter. All culverts was instrumented with MP 200x55 corrugations and built with 6.0 mm thick steel plates.

Oppose to the flexible cases the three cases with concrete was selected purely based of ROGER 1000 photos (since no design drawings were available). It was ensured that the criterias set for the local conditions were met and that all of them was instrumented with transition slabs based of their year of construction, see chapter 3 in Appendix B. The three preseleceted concrete cases are shown in figure 5.4.

Figure 5.4 - ROGER 1000 photos showing site conditions at C_1) Svenningdal, C_2) Vinstradalsveien and C_3) Jarenhaugen culvert

5.1.2 Evaluating the results

For all six cases an intervall sat 500 meter before and after each culvert was predefined in order to distinguish and locate deviations associated with the culvert and the free track. In figure 5.5 a typical ROGER measurement from Viker steel culvert is presented, showing the culverts location in the middle of the plot (at km. 217.357). Scaled up version of this plots can be found in chapter 2.3, Appendix B.

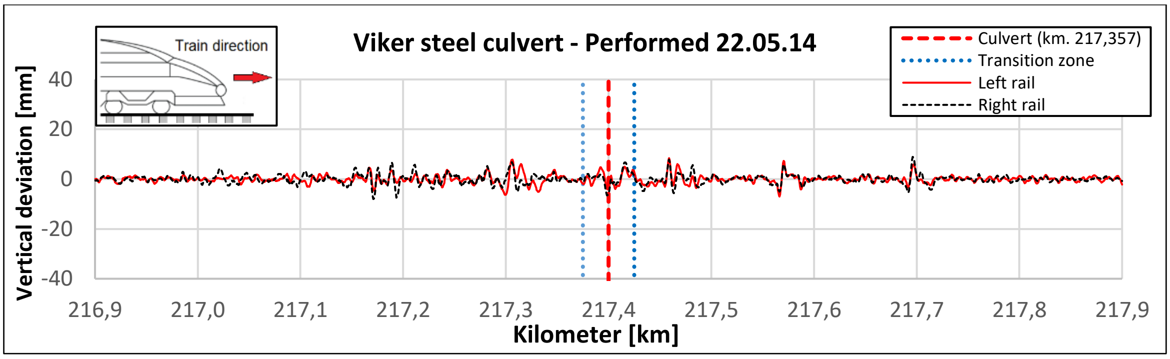


Figure 5.5 – ROGER 1000 measurement from Viker culvert, performed 22.05.14

The figure shows both the culvert located at km. 217.357 and a dashed line labeled «transition zone» 25 meter before and after the culvert. The «transition zone» was predefined based of suggestions from Bane NOR that ROGER 1000 data actually may deviate in its kilometer data, up to 20-30 meter due to micro slips and creep forces between the veichles wheel/rail contact patch. It was therefore thought that any peaks within this 25 meter intervall «may» be associated with the culvert.

The measurement presented above shows that there is a small peak in deviation seen at the culvert, but nothing that stands out compared to the deviations seen at the free track. When the measurements look like this it is thought that there is no ground to state that the peaks seen above the culvert is associated with a variation in track stiffness since similar occourances is seen at the free track.

The thought here is that if a peak can be located consistently at a specific kilometer with enough measurements, one could make some kind of correlation to the track stiffness over a culvert. For all the six cases however, the tendency shown in the figure above was the most present and no specific correlation between vertical deviation seen from ROGER 1000 and these culverts could be found.

Only two cases showed some deviations that one with confidence could tie to the culverts. That was Holme steel culvert and Jarenhaugen concrete culvert, see figure 5.6 and 5.7.

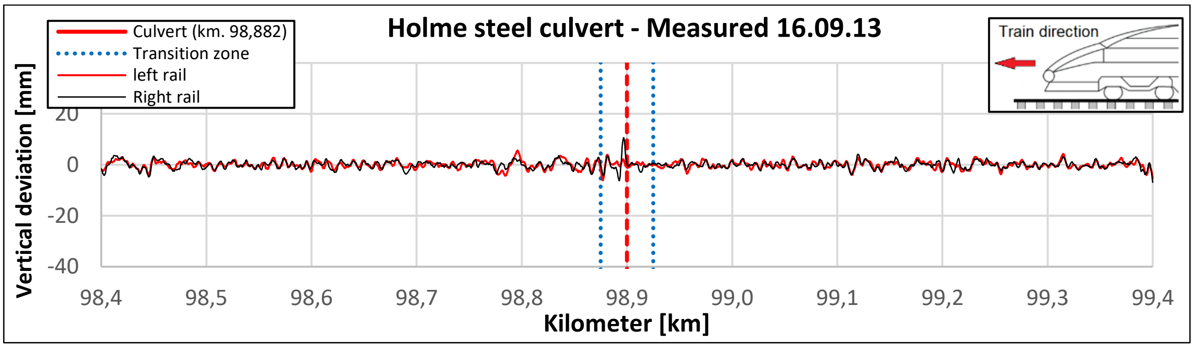


Figure 5.6 – ROGER 1000 measurement at Holme culvert 16.09.13, showing deviations at the culvert

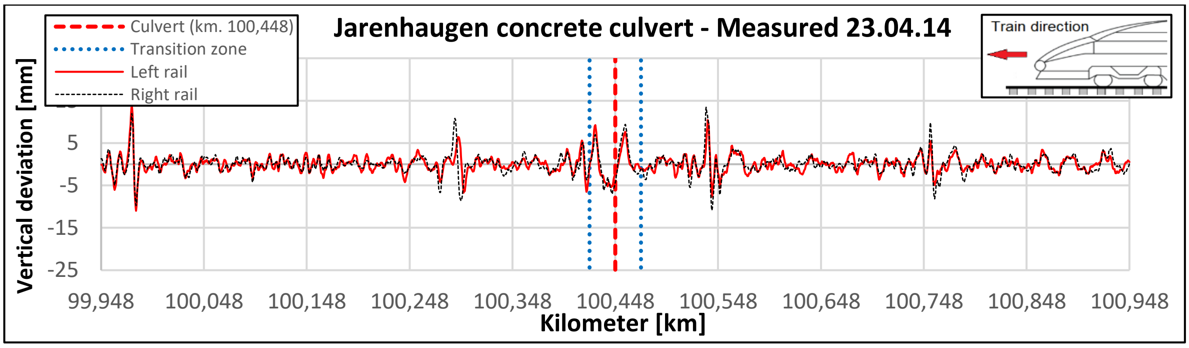


Figure 5.7 - ROGER 1000 measurement at Holme culvert 16.09.13, showing deviations at the culvert

In these two measurements there are peaks shown above at location of the culvert, suggesting that the track deviaties more here compared to the «normal track». But these deviations however, is for example larger at the free track close to Jarenhaugen culvert compared to the peaks seen at the culvert, suggesting that these deviations are not posing any additional problems compared to the deviations at the free track.

At Holme it can be seen a more obvious peak compared to the free track where the deviation has an order of 10 mm above the culvert oppose to about 5 mm at the free track. An interesting observation here is that it was only the right rail that showed deviations which suggests that curves may also influence deviations under the presence of a culvert.

When these deviations becomes as anecdotal as they did for these two culverts it is difficult to know if these deviations are tied to either the local conditions or the stiffness variations over these culverts. The deviations seen in figure 5.6 and 5.7 has rarely been observed in the other measurements, and the general trend has been more like the tendency shown in figure 5.5, were no clear deviation at the culvert is seen.

An interesting observation was seen for the second concrete case (Vinstradalsveien), where the deviation peaks that stood out only was observed at the free track and not over the culvert. This tendency was observed for all the ROGER 1000 measurements involving Vinstradalsveien, which is quite the uposite of what one would normally expect where there is certainly a stiffness variation present, see figure 5.8.

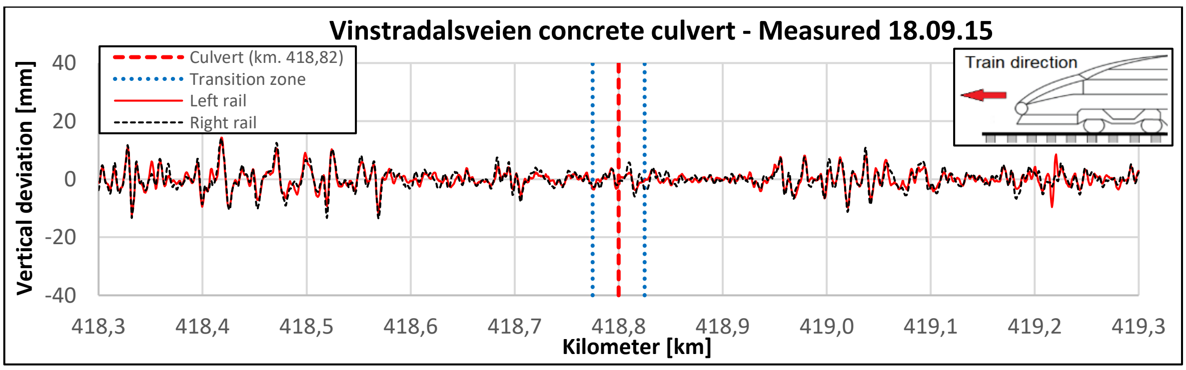


Figure 5.8 - ROGER 1000 measurement at Vinstradalsveien culvert 18.09.15, showing only deviations at the free track

From these measurements the overall conclusion is that despite the stiffness variations presence over these six culverts, the deviations measured with from ROGER 1000 over them were not any different than the deviations measured at the free track. This suggests that there is no direct correlation between additional deterioration of a track and the stiffness variations caused by culverts. One have to bear in mind that maintenance procedures tied to differential settlements such tamping and levelling is based out of these measurements, which therefore also correlates to maintenance.

5.2 Stiffness variations and deterioration of transition zones

The general formulation of the stiffness transition problem says little to nothing about how much a stiffness variation actually affects a railway track. It has been suggested by several authors that a stiffness variation tends to induce a neverending deterioration loop between increased wheel/rail interaction forces and differential settlements which only worsens untill maintenance is performed [2,7].

However, it has also been suggested that there will always be stiffness variations present on a railway track founded on ballast [7] which raises the question of how big of a stiffness variation is necessary in order to induce a deterioration loop that deviates from a normal track and what conditions must be present in a transition zone for such deterioration loop to act continously.

A theoretical coupled 2D model was created at the Royal Institute of Technology (KTH) for investigating the dynamic response of a veichle in a train-track-bridge interaction. According to a study performed by Andersson and Arvidsson with this model, is the dynamic response of a veichle more influenced by track irregularities than stiffness variations, and it is rather the irregularities as a result from the stiffness variations that leads to unfortunate veichle dynamics (i.e increased axle loads) [43].

The finite element model was primarily created for investigating ballastless tracks and should in this context only be viewed upon as something theorteical. The model was a 2D Euler-Bernullis beam founded on a simply supported span/several continous spans («slabs») and included a veichle modelled with a linearized Hertizian spring contact to the rail, see figure 5.9

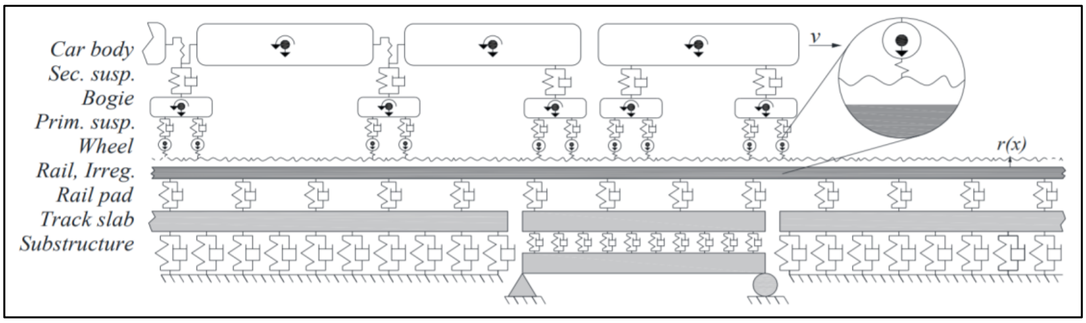


Figure 5.9 – 2D train-track-bridge coupled model (After Arvidsson and Andreasson, 2017)

In the analyzis of a transition zone was the «free track» given a bed modulus of 100 MN/m^3 («a normal track») while the bridge was set to «infinite stiff». Two comparisons were made, the abrupt change in substructure stiffness on a perfectly smooth rail profile (i.e no irregularities) and the abrupt change in substructure stiffness on a irregular profile as a result from the stiffness variation.

Both cases created an oscillation in the wheel-rail contact force, but for the perfectly smooth profile only with an amplitude of about 5 kN but 10 kN for the irregular profile. This led to the suggestion that the variation in wheel-rail contact force from only the abrupt change in substructure stiffness is low compared to the variation induced from a random track irregularity [43].

According to ERRI, experience numbers has suggested that a geometry defect of 10 mm normally leads to an increase in about 30-40% of a load if the traveling speed is 140 km/h. This agrees well with the reported measurement from Coelho [3], where under the presence of hanging sleepers in the transition zone was the axle loads of a passing train increased with about 25%, see chapter 5.1.2 in Appendix C.

According to another study reported by Dahlberg and Lundqvist in 2004 [42], could a 1.0 mm gap under a sleeper (i.e «a hanging sleeper») lead up to 70% increase in wheel contact forces while the sleeper adjecent to a hanging sleeper could displace up to 40% more.

From these magnitudes is it suggested that it is rather the irregularities as a result from the stiffness variation that creates the oscillations of a veichle (i.e increases the wheel/rail interaction) instead of the stiffness variation alone. As was suggested by Dahlberg will a ballasted railway track always have stiffness variations, and a question raised from this is how big of an irregularitiy is then required for this degradation loop to be present and is the stiffness variations over a culvert enough for it to be initiated.

According to a study on four railway bridges reported by Li and Davis in 2005 [6], was the deterioartion rate at the transition zones of these bridges always higher compared to the normal track. This was despite all of these cases having different solutions in their transition zone, see figure 5.10.

According to Li and Davis, the differential settlements were related to the abrubt change in track stiffness between these bridges and their free track, causing an increase in dynamic veichle/track interactions and leading to a neverending degradation loop [6].

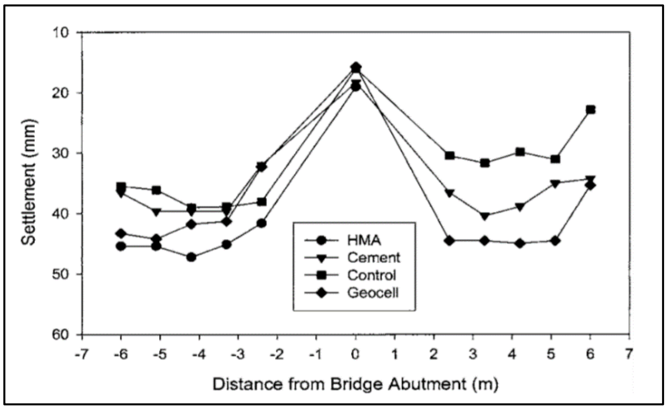


Figure 5.10 – Differential settlement development and deterioration at four individual bridges from the US (After Li &

The problem with associating the deterioration tendencies at these transition zones to track stiffness, is that little background information is provided at the bridges local conditions which makes it very difficult to know if this degradation pattern in fact relates to stiffness variations instead of local condition.

From the case study at Gouda Govervelle in Netherland for example, a very similar degradation pattern is present but the source of the problem was not related to track stiffness, see figure 5.11.

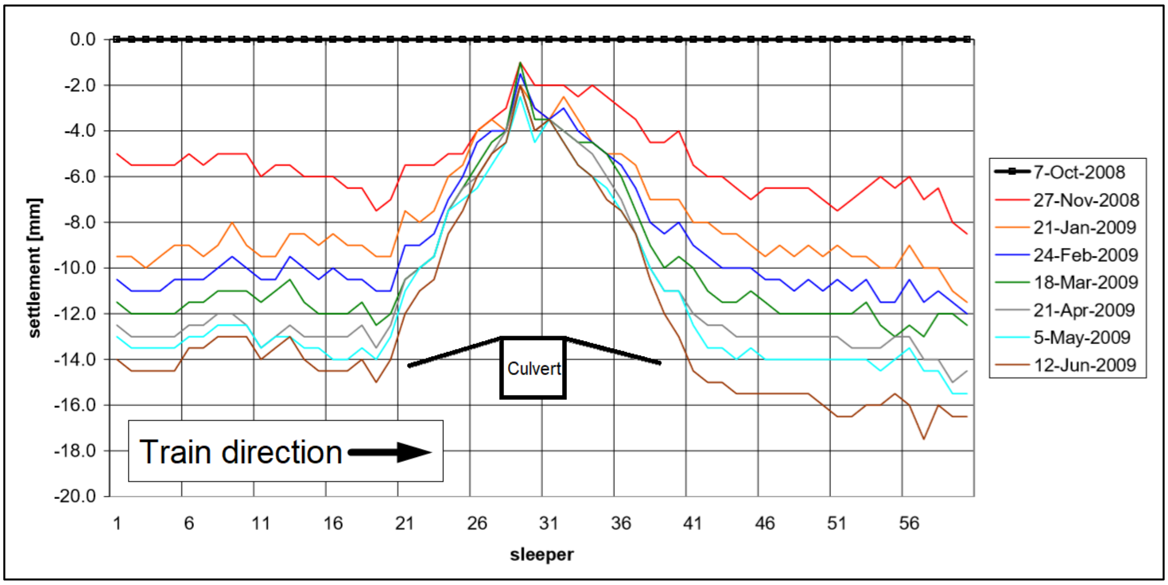


Figure 5.11 – Nine months of monitoring at Gouda Goverwelle railway culvert, showing that the deterioration rate always was higher at the end of the transition zone compared to the free track (Modified after Hölscher. P, Meijer. P, 2009)

In this field survey was the settlement along the rail over the entire culvert monitored for 9 months straight after a tamping cycle, and it was found that the transition slab on average settled about 2.5 times faster than the free embankment which settled autonomously for about 1.0 mm pr. month [8].

It was concluded here that the higher deterioration rate always present over the slab was related to the backfill material bellow the transition slab and not stiffness variations of the track and veichle dynamics. The sandy backfill under the slab was able to flow away from the transition slab with time which decreased its contact lenght to such extent that it caused it to rotate tremendously, consistently creating a «dip» [3,8].

From this field survey it was concluded that the deteriorations seen here was a result from the conditions under the transition slab, not over it. This in combination with autonomous settlements of the clay caused this section of the track to be particulary weak, eventually leading to an increase in the dynamic veichle/-track interaction forces which contributed further to the deterioration loop. Additional information on these static measurements of the slab can be found in [3,8], but also see chapter 2 and 4 in Appendix C.

5.3 Discussion

A problem in Norway when assessing transition zones has been that especially older railway bridges where the track is often fixed directly to the bridges and no transition slab has been used tends to cover up for all the less extreme cases.

Finite element calculations with correlation to field measurements has suggested that the difference in track stiffness between a normal track and a railway culvert causes an irregularity in the order of 0.2-0.5 mm. Only the extreme case with the large span flexible pipe arch caused an irregularity with an order over 1.0 mm, but is considered an extreme case.

As was discussed in chapter 5.2 and assuming that one is threating this problem according to «normal coditions» (i.e a design speed < 200 km/h and norwegian backfill conditions) it is hard to believe that the increase in wheel contact forces induced by the stiffness variations over these culverts alone is enough to start a continous deterioration loop. An important point to highlight here is that when we're talking about deterioration in this project, were talking about deterioration related to the foundation under the sleepers (i.e differential settlements), and not specific track components.

Another question that follows this is what circumstances must be present in order for the so called «deterioration loop» to be present. It is found that in a typical norwegian backfill to a bridge, only high quality crushed rock is normally used, a material which according to plate load tests is up to twice as stiff as sands [27]. As a comparison, from backcalculating the shear wave velocities measured at Gouda was the elastic modulus estimated to only be in an order of 95-115 MPa, see chapter 2.1 in Appendix C.

From presuming that a normal backfill in Norway will be both stiffer and better suited to prevent any flow, it can be stated that the occourances at Goude unlikely would occour for any stiff culvert in Norway. But what the Gouda case showed is that what really governs the long-term behaviour of a transition slab is the conditions under it, not over it. If the materials used under the slab is of high quality, well compacted and has no ability to flow, there is no grounds to suggest that a transition slab will rotate continously over time and that the transition zone continously deteriorates as a result.

A similar parallel can be drawn to the four bridges from the US. Little to no background information was provided in these cases and it is therefore no basis to suggest that the deterioration at these cases applies for a small culvert built under normal conditions.

The discussion of stiffness variations and maintenance problems has not been found in the litterature for flexible culverts, but from ROGER 1000 measurements is it suggested that the problem for flexible culverts spaning between 4-6 meter with minimum soil cover will happend to about the same degree as for a rigid concrete culvert, nearly neglectible.

What these ROGER 1000 measurements has shown is that there is no direct correlation between vertical track deviations measured by the car and the stiffness variations seen over these culverts. This suggests that the stiffness variations as a result from these culverts in general don't pose any extensive maintenance problems compared to a normal track founded on ballast.

6 Conlusions, recommendations and further work

6.1 Summary and conclusions

Transition zones at norwegian railway bridges has over several decades caused maintenance problems for Bane NOR. These problems typically arise from the accumulation of differential settlements and geometry defects at the railway track. The problem has been tremendously improved by introducing a short, 2.5-3.2 meter long transition slab at the end of all new railway bridges.

Bane NOR has reported that the transition slab undoubtedly enhances the long-term track quality at a transition zone compared to not using it. The common explaination for why it works is that it covers up the closest meter to a bridge, an area which is well known to be proun to differential settlements due to the res-trictions associated with backfilling and compaction. The slab is also thought to provide some kind of gradual stiffness transition from a free track over to a bridge, but no one is able to quantify how.

Confusions arise when the transition slab is compared to international solutions which has been observed to reach up to 20-30 meter in lenght at some high-speed lines. Contributing to this confusion is the aspect of flexible-steel-soil composite bridges were no requirement is set. In this project, it has in general been concluded that the stiffness problem should be treated separately between stiff and flexible culverts because of the different variables between them.

In this project a methodology was created in PLAXIS 2D for modelling the entire transition problem from skratch by preparing a foundation, building a culvert, building a railway track, and model moving train loads passing over these culverts.

Four models were created with this methodology in the study on variables such as changing the lenght of the transition slab or the span and profile shapes of flexible culverts. This was done in order to investigate how these variables affects stiffness variations at the track. These models suggests that the stiffness variations caused by these culverts under normal conditions, leads to a irregularity in the order of 0.2-0.5 mm when comparing the deflections seen at the free track to the deflections over these culvert. Only the extreme case with the pipe arch purposed an irregularity over 1.0 mm.

It has been suggested in the litterature that any stiffness variation induces an irregular wheel/contact force, which over time contributes to differential settlements and potentially can create hanging sleepers. This «deterioration loop» is thought to be initiated by increased wheel/contact forces as a result from a irregularity, speeding up the differential settlements which increases the wheel/contact forces further.

But it has also been suggested to that there will always be stiffness variations present on a ballasted railway track, and a question raised from this is then how big of an irregularity is required in a transition zone in order to initiate this so called deterioration loop that deviates from a normal track.

For determining how the stiffness variations at culverts actually deteriorates a railway track an assess-ment were made from ROGER 1000 data for six preselected culverts, three made out of flexible steel and three out of rigid concrete. The general conclusion is that there could not be found any trend in extensive track deviations at these culverts seen from the ROGER 1000 data compared to their adjecent normal track.

It has generally been found that the problem at especially older railway bridges tends to take all the att-ention away from the more modern bridges. It has been suggested in this project that older bridges were the railway track are fixed directly to the bridge are not representative for the stiffness transition problem because alot of the maintenance problems are actually associated with the track condition themselves

A parallell can be drawn between the older norwegian bridges and international referance cases often found in the litterature, which «proves» that transition zones always deteriorates faster than a normal track. Seen from the comprehensive Gouda Goverwelle case study, the deterioration pattern looked very similar to the patterns reported from the four bridges in the US, but the root cause at Gouda was

associated with conditions under the slab, not veichle dynamics over it. This suggests that there is no basis for associating the problems from the four bridges in the US directly to track stiffness.
When comparing the irregularities deriven from the FE analyses to the suggestions from ERRI, if any increase in wheel contact force it is expected to be low. If the backfill of a culvert is built of high quality crushed rock and there is normal track conditions over it, there is no proof that the transition zones at culvert underpassings deteriorates any faster than a normal track, suggested from the ROGER 1000 data.

6.2 Recommendations and further work for Bane NOR

It is purposed to threat the stiffness problem between rigid concrete and flexible steel-soil composite culverts separately as these structures have different variables associated to them for which, especially high speed lines makes a difference.

6.2.1 Validation

It would've been great to validate the irregularities purposed by the FE simulations in this project to field measurements. The simple way of doing so could be to compare the displacement over a culvert to the adjecent free track at any site in order to account for local conditions.

According to Wang and Markine [5] is a simple, yet proven method for doing so the use Digital Image Correlation (DIC), which is a technique that uses a high speed camera to measure the displacements of the track at certain points during train passages. Such measurements are relative simple, and can for example be peformed by measuring two points, one at the free track and over a culvert which will be very valuable for validation but also provides an idea of what magnitudes the track stiffness over these culverts really are.

6.2.2 Rigid concrete culverts and transition slabs

The explaination suggested from this thesis for why the relative short transition slab works so well compared to not using it is because it ensures that the track quality sustains at the transition zone over time by covering the most critical section of a bridge transition where the potential for differential settlements is highest. Based of the experiences made from Netherland is it suggested that it is rather the backfill under the slab that determines the long-term conditions of a transition zone with a slab where no proof is found that a longer slab makes a difference.

The small irregularity caused by these culvert will make a bigger difference for veichle dynamics at higher speeds, and in terms of providing a gradual stiffness transition to these culverts is it suggested to reconsider the short 2.5 meter slab. Despite the FE simulations suggesting that the longer slab caused a «dip», it also suggested that it provides a more gradual transition to the bridge. For the passenger train with 18 tonn axleloads, the «dip» in the FE analyses were almost neglectible compared to the dip from the freigh train, suggesting that a longer slab would've been even more favourable for a passenger train.

From this project the transition slab is considered something that should be used as a minimum solution for all railway bridges, but for preserving train dynamics at higher speeds is it recommended to use a longer slab. ProRail in Netherland regulates the slab lenght as a function of speed, where the minimum requirement starts with 4.0 meter for all speeds, but must minimum be 6.0 meter if the speed is between 160-200 km/h and over 9 meter if the speed is over 200 km/h.

6.2.3 Flexible steel-soil composite culverts

Steel-soil composite bridges present on the norwegian railways typically spans about 4-7 meter, and it is suggested based of investigating ROGER 1000 data that flexible culverts within this span range similar to concrete culverts, does not pose any general maintenance problems that deviates from a free track.

There are certain variables that should be considered when assessing the track stiffness over these structures, that is the influence of profile shape, height of cover, corrugation type and steel thickness (i.e stiffness) and the backfill.

For these structures there is only a fixed requirement set for the minimum height of cover, neither considering span lenght, train speed or profile type. The FE simulations suggests that the large span pipe arch led to four times of an bigger irregularity compared to the smaller circular culvert. As has been suggested by ROGER 1000 data, flexible culverts in general does not pose any clear maintenance issues, but the irregularity caused by this culvert can have an impact on veichle dynamics, an aspect which should be considered.

6.3 Further research

In terms of modelling there is three aspects which should be looked into further if a similar approach to what has been presented in this thesis are being used, that is the problem related to the stress dependent stiffness formulation, the convergence problem seen in all the dynamic analyses with Hardening Soil and dynamics related to the track system created in this model.

Despite this project purposing that the stiffness variations over these culvert in general was found to not have any specific correlation to increased geometry deviations, it is suggested that in relation to high speed railways there should be defined a «critical track stiffness» over culverts.

6.3.1 Stiffness problem with the formulation in Hardening Soil

PLAXIS has proven to be well suited for assessing the quasi-static aspect of track stiffness due to its ability to model complex soil behaviour but also performing dynamic analyses. Whats really convenient with using Hardening soil models oppose to more elastic models is its ability to remember previous loads which is crucial if modelling flexible steel culverts from scratch.

Since the topic of this thesis is so dependent on accuracy by the millimeter, the stress dependent formulation of stiffness in Hardening Soil led to several problems over and under these culverts. The problem was improved when reducing the power modulus, but was still present for cases with flexible culverts.

The solutions presented in this thesis was more like a workaround than a solution to the problem, and it is in general recommended to investigate this problem further if similar dynamic problems are to be modelled with Hardening Soil. A good substitute for Hardening Soil could for example be to use Hardening Soil Small where the stiffness is also controlled by the shear modulus, but is more demanding nummerical.

6.3.2 Convergence problem with dynamics in Hardening Soil

In all these simulations there has frequently been occouring a persistent plastification at any point in these models as the loads are passing. It was first thought that these plastifications were associated with PLAXIS not being able to tackle tension forces, but unhooking the «tension cut-of» option only seem to speed up the convergence to a certain point, where little change is seen further.

As discussed in chapter 2.3.4 damping seem to get rid of the problem, but only when using high rayleigh damping which ruins the dynamic aspect of the model. Before the methodology purposed in this thesis should be applied with confidence, it is recommended to find the real cause of this convergence problem.

6.3.3 Dynamic analyses for railway tracks in PLAXIS

It has been suggested from this project that the static aspect of this railway track in PLAXIS shows good agreement with Zimmermanns classic theory for beam on elastic foundation. But if this track are to be used for any dynamic analyses there are certain things that should be looked more into.

As was revealed by the Fast Fourier Transformations (FFT) modelling the railpad as two vertical plate elements creates an unrealistic frequency domain at the track. When doing quasi-static considerations (i.e investigating deformations, deflections) the influence of this frequency domain is nearly neglectible compared to other frequencies found in the model.

Looking from a dynamic perspective however, it is recommended that the frequency domain found at the pad-pad passing frequency is taken into consideration. If the purposed model here are to be validated in terms of dynamic behaviour, a good approach could for example be to compare the frequencies at the track system in PLAXIS with a receptance test, where typical frequency domains of a normal track is compared for validating the frequency domains in the FEM from PLAXIS.

6.3.4 Defining a critical track stiffness over railway culverts

This thesis has suggested that the small stiffness variations posed by railway culverts in general does not cause any additional deterioration of the track compared to a adjecent free track. But speaking in terms of irregularities and veichle dynamics it would've been interesting to define a «critical track stiffness variation» or an «critical irregularity» over a culvert.

Considering normal conditions for a rigid concrete culvert the irregularity can either be controlled by the materials under the transition slab, the height of cover or the transition slabs lenght. The three suggestions are based of the thought that

- A softer material under a transition slab will exaggarate the «dip» problem at the end of the transition slab (suggested by FE simulations)
- A lower height of cover increases the stiffness over culvert, which increases the irregularity between the free track and a stiff culvert further
- A longer transition slab makes the transition over from the free embankment to the bridge more gradual

As was seen from the FE simulations the stiffness variation posed by the pipe arch led to an irregularity that was four times higher than the circular pipe. Considering train dynamics there would be interesting to define a critical irregularity for these structures aswell. Before doing so there are certain variables one should consider

- The ratio between span lenght and soil cover
- Culvert properties such as corrugation, steel thickness or shape
- Backfill materials and compaction procedures (suggested by FE simulations)

Seen from the flexibility number in chapter 3, the flexibility number of the pipe arch was much higher than for the circular type, indicating beforehand that the pipe arch would be much softer than the pipe.

The flexibility number does not however account for height of cover or type of loads, and as a result a theoretical suggestion for assessing the stiffness of soil-steel structures under railways by taking into consideration these aspects was purposed by Machelski in 2015 [44]. His work was not included in this project.

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