Seismic Evaluation of The
North Bound N1-R300
Bridge Interchange

by
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Thesis presented in fulfilment of the requirements for
the degree of Master of Engineering in Structural Engineering in
the Faculty of Engineering at Stellenbosch University

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December 2015

The financial assistance of the National Research Foundation (NRF) towards this research is hereby acknowledged. Opinions expressed and conclusions arrived at, are those of the author and are not necessarily to be attributed to the NRF.
Declaration

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Signature: ..........................

Date: ..........................
Abstract

The N1 / R300 or Stellenberg interchange was designed in 1982 and construction was completed in early 1986. The bridge was designed according to the Code of procedure for the Design of Provincial Bridges and Culverts published by the Cape Province Department of Roads in June 1977. The design of the bridge over the N1 did not account for any significant form of seismic excitation but a review of current design code states that it falls within the highest seismic risk bracket in South Africa. Although this highest risk bracket still only equates to a moderate intensity earthquake, structures not designed accordingly could undergo serious damage. Current research suggests that the maximum possible peak ground acceleration expected in the region is almost double the modern design requirements. Additionally, numerous features of the bridge do not agree with many of the recommended best practices for bridges in seismic prone areas.

This study evaluates the structural response of the Stellenberg interchange bridge when subjected to moderate intensity earthquakes. This was achieved through experimental tests to determine the mode shapes and accompanying frequencies to calibrate the finite element model of the horizontal circular arch bridge. A detailed finite element model was developed using Abaqus software. Once the finite element model was calibrated, the model was subjected to various scaled earthquakes to determine whether the bridge would be able to sustain a moderate intensity earthquake.
Uittreksel

Die N1 / R300 of Stellenberg wisselaar was ontwerp in 1982 en was voltooi in Februarie 1986. Die ontwerp was gedoen volgens die 'Code of procedure for the Design of Provincial Bridges and Culverts' wat deur die Kaap Provinsie se Departement van Paaie gepubliseer is in Junie 1977. Die ontwerp van die brug wat strek bo-oor die N1 het nie voorsiening gemaak vir enige vorm van seismiese aktiwiteit nie. Die huidige ontwerp kode dui wel aan dat die brug in die hoogste seismiese risiko groep val wat in Suid-Afrika voorgeskryf word. Alhoewel hierdie hoogste risiko groep steeds net gelykstaande is aan 'n matige intensiteit aardbewing kan dit tot groot skade lei as dit nie daarvoor ontwerp word nie. Navorsing dui aan dat die maksimum moontlike grond versnellings wel in 'n orde van meer as dubbel die is wat voorgeskryf is deur die ontwerp kodes. Daarbenewens is daar van die strukturele aspekte van die brug wat nie ooreenstem met die voorgestelde beste praktyk om te volg vir brûe in seismiese gebiede nie.

Hierdie studie evalueer dus die strukturele eienskappe van die Stellenberg wisselaar brug wanneer dit onderworpe is aan 'n matige intensiteit aardbewing. Hierdie was gedoen deur fisiese toets om die modus vorme en hul frekwensies te bepaal sodat die numeriese model van die horisontale boog brug bevestig kan word. Verder was 'n gedetailleerde numeriese model ontwikkeld met die gebruik van Abaqus sagteware. Die fisiese toets resultate was gebruik om die numeriese model te verifieer. Verskeie afgeskaalde aardbewings tot en met 'n grond versnelling van 0.2g is dan toegepas op die numeriese model wat dan die struktuur se reaksies sal toets.
Acknowledgements

This paper is presented with the approval of the South African National Roads Agency SOC Limited. The contents of the paper reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the South African National Roads Agency SOC Limited.

Furthermore, I would hereby like to acknowledge the assistance and support I received from the following individuals and institutions:

- Dr Trevor Haas, my supervisor, for his guidance and effort throughout the completion of this study.

- All the staff and lecturers at the Structural Engineering Department at Stellenbosch University who helped me during the completion of this study.

- SANRAL, for their cooperation and assistance during all stages of this study, especially their assistance during the physical testing of the bridge.

- Professor Pilate Moyo and his PhD students at the University of Cape Town for their assistance during physical testing and signal processing of the test results.

- My classmates for their assistance, academic discussions and friendship throughout my time at university.

- My fiancé, Nina Wehmeyer, for her patience and support throughout the process of writing this thesis.

- My parents for their unconditional love and support throughout my life, I would not be where I am now without them.
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Nomenclature

Subscripts

$C$  Concrete
$S$  Steel
$x$  X Direction/Axis
$y$  Y Direction/Axis
$z$  Z Direction/Axis

Variables

$\nu$  Poisson’s Ratio  
$\bar{\rho}$  Composite Density  [Pa]
$\bar{E}$  Composite Young’s Modulus  [Pa]
$\rho$  Density  [kg/m$^3$]
$A$  Area  [m$^2$]
$a$  Acceleration  [m/s$^2$]
$E$  Young’s Modulus  [Pa]
$I$  Moment of Inertia  [m$^4$]
$J$  Torsional Constant  [m$^4$]
$u$  Displacement  [m]
$v$  Velocity  [m/s]
Chapter 1

Introduction

Bridges are considered some of the most important structures in modern society. It links places to one another that would have previously been inaccessible and/or substantially reduces travelling time. Bridges therefore serve as a link between communities, cities and even countries.

Seismic excitation is a major risk for all bridge structures located in earthquake prone regions. Numerous bridges have sustained serious damage and have even collapsed due to the strong forces induced by these natural hazards. Therefore, careful consideration of these loading effects during the design stage of a bridge is crucial. The collapse of an important bridge could cause severe disruptions to the surrounding communities. In certain cases, collapse of a bridge during a seismic event could cause restricted access or obstruction to major routes. This could impede emergency services from reaching areas in need of assistance after an earthquake. The collapse of a bridge could cause disruptions long after the seismic event has occurred. These disruptions could last until the bridge is repaired or reconstructed, which could take months or years to complete. This could lead to serious detrimental economic effects to the surrounding communities.

South Africa is not as prone to severe earthquakes compared to countries such as Japan, Indonesia and Turkey. In addition, the occurrence of moderate intensity earthquakes is uncommon in South Africa, with only a limited number of moderate to strong seismic events occurring in the past. Civil and structural engineers in South Africa were not required to include seismic load effects on civil infrastructure before the early 1980’s. However, current research indicates that certain regions of South Africa are prone to moderate intensity earthquakes. The first requirement for earthquake loading of infrastructure in South Africa only appeared in the loading code, SABS 0160 [1989]. Prior to 1981, engineering professionals were not required to design bridges and culverts to resist the forces induced by earthquakes (Cape Provincial Administration Department of Roads, 1977). These factors question the integrity of infrastructure constructed in seismic prone regions prior to the 1980’s, especially important bridges on major routes and highways.

The North bound N1/R300 interchange (Stellenberg interchange) was selected for this study due to:
• The bridge crosses a national highway in a congested area,

• The bridge provides a link between two national roads, N1 & N2,

• Consequences of failure would lead to significant disruptions to the greater Cape Town area,

• Initial investigations indicate that some design features do not conform to modern best practices for bridges in seismic prone areas,

• The bridge’s design did not consider seismic loading since it was designed using the Cape Provincial Administration Department of Roads’ Planning Manual Part 3: Bridge Design Manual (1977)

1.1 Bridge Information

The R300 project provides a link between two national highways (N1, N2) leading in and out of Cape Town. In terms of the greater Cape Town area, the N1 is an important link to neighbouring communities. The N1 provides a major access route between the Cape Town CBD and numerous large suburbs. These suburbs and towns include Bellville, Durbanville, Goodwood, Paarl and Stellenbosch. Figure 1.1 shows the locality map of the greater Cape Town area. It shows all the major routes and suburbs mentioned, along with the location of the Stellenberg interchange.

![Figure 1.1: Locality map of the Stellenberg interchange (AfriGIS (Pty) Ltd, 2014)](image_url)
The Stellenberg interchange connects the N1 with the R300. The R300 provides a further link between the N1 and the N2 as well as large suburbs and industrial areas such as Kuils River and Blackheath. In addition, the R300 provides access to the Cape Town International Airport when travelling from the above-mentioned regions. Serious damage or failure of the Stellenberg interchange would have a detrimental effect for people in these areas and would cause major disruptions.

In addition, the latest traffic index published by Tom Tom International B.V. (2014) concluded that Cape Town is currently the most congested city in South Africa. The study showed that the road network is currently close to its capacity and any loss of infrastructure would cause extreme congestion. The Stellenberg interchange is thus an important part of the national road infrastructure network.

The design of the Stellenberg interchange was completed in February 1982, with construction completed in early 1986. The interchange was designed using the Planning Manual Part 3: Bridge Design Manual (1977) published by the Cape Provincial Administration’s Department of Roads in June 1977 (Latimer, M. 2014, pers.comm., 11 November). The interchange is shown in Figure 1.2.

The bridge utilises a post-tensioned concrete box girder. The bridge consists of 12 spans ranging between 27.5 m and 38 m in length with a total length of 418 m. The bridge is curved in plan with an approximate radius of 245 m. The box girder has a width of 11.2 m with a total depth of 1.85m. The superstructure is supported by nine single columns and two pairs of double columns at position C5 and C9, which is shown in Figure 1.3.

Columns C5, C6 and C7 are supported on pile group foundations while the remaining columns are supported on pad footings. Columns C4 to C10 are monolithically cast into the box girder
while columns C2 and C12 support the superstructure via unidirectional plate bearings at their apexes, allowing movement in the longitudinal bridge direction. The remaining columns C3 and C11 support the superstructure via fixed plate bearings that allow only rotational freedom. Two side-by-side unidirectional plate bearings support the box girder at the abutments, allowing movement in the longitudinal direction.

1.2 Problem Statement

Priestley et al. (1996) discusses various best practice features for bridges in areas prone to seismic excitation. Inspection of the Stellenberg interchange reveals certain aspects of the bridge’s design that do not conform to these best practices for bridges susceptible to seismic loading. These features include:

- Monolithically cast columns to superstructure connections,
- In plan curvature of the deck,
- Single columns to wide superstructure connections,
- Off-perpendicular abutment connections in terms of the longitudinal axis of the bridge,
- Semi-soft foundational soil conditions.

These features indicate the bridge could potentially be susceptible to serious damage or even collapse during a moderate intensity earthquake, of which could be expected in the region. The features are reviewed and discussed in Chapter 2.

The design date of the structure is indicative of further potential problems. The current bridge design code, TMH 7 (1981) was published in 1981, one year before the design of the Stellenberg interchange was completed. If TMH 7 (1981) was used for the design of the Stellenberg interchange, consideration for seismic excitation would have been required. During an interview with one of the design members of the project team, Mr. M Latimer on 11 November 2014, he confirmed seismic loading of the structure was not considered during the design phase of the intersection. The design code used by the design team of the Stellenberg interchange, Cape Prov. Bridge Design Manual (1977), did not require any consideration for seismic excitation.

Therefore, a structural and dynamic review of the bridge using modern software could identify potential frailties of the structure. This information could be reviewed to establish whether the
bridge can withstand a moderate intensity earthquake.

1.3 Objectives

The objective of this study was to determine whether the North bound Stellenberg interchange bridge can sustain a moderate intensity earthquake. This was achieved by applying various magnitude earthquakes to a calibrated finite element (FE) model. The sectional forces obtained from the FE model were compared to the capacities of each section to evaluate the level of damage that may occur. The results from the simulations were used to evaluate the structural robustness of the bridge.

1.4 Methodology

A finite element model and calibration process along with a dynamic seismic analysis was used to evaluate the structural response of the Stellenberg interchange. The procedures followed during this study are discussed in this subsection.

Development of initial finite element model

An initial FE model was developed to obtain the dynamic characteristics of the structure. The initial FE model enabled effective planning of the physical modal testing required for the calibration of the FE model. The modal behaviour of the initial FE model could be reviewed to determine effective placement of accelerometers used to capture the structure’s vibration. Without this information, measurement devices could have potentially been placed at modal node points, delivering no functional data.

Physical modal testing

The physical modal testing was performed to obtain data required to calibrate the final FE model. This took into account the current state of the bridge, including all possible cracks and defects that accumulated over time. The mode shapes along with their corresponding frequencies were measured and were compared to the FE model to confirm the model’s mass and stiffness properties. In addition to the model verification, the damping ratio of the structure was determined from the test data. The complex interaction between all the elements that contribute to the damping of a large structure makes it impossible to calculate a theoretical damping ratio.
Detailed finite element model development

The model complexity was increased to provide an accurate representation of the actual structure, while maintaining computational efficiency. The initial FE model was modified utilising the physical testing data to develop the final calibrated FE model. This was obtained by implementing appropriate refinements and adjusting appropriate parameters of the model. The final model was used to perform all the earthquake simulations. These parameters are discussed in further detail in Chapter 4. Once the FE model was calibrated, no further structural changes were made to the model.

Earthquake Simulations

Recorded ground accelerations were applied to the abutments and each column base of the FE model. The ground accelerations were applied in various configurations, intensities and magnitudes to evaluate all possible cases of seismic excitation that the structure could experience.

Data evaluation

Once all seismic simulations were completed, sections of the bridge that indicated high forces or a possibility of failure were identified. The capacity of each of these sections was calculated and compared to the response from the FE model. Using these comparisons, an evaluation of the probability of failure of each section could be made. All of the problematic sections were evaluated separately for different loading conditions, for all earthquake simulations.

Conclusion

Once all the problematic sections were evaluated, a conclusion on the structural robustness when exposed to seismic excitation was made. The potential of serious damage or failure was evaluated for each of the different magnitude earthquakes applied to the FE model. Additionally, recommendations on further studies and research were made.
Chapter 2

Literature Review

Priestley et al. (1996) provides a list of recommended best practice structural characteristics for seismic bridge design. Numerous features of the Stellenberg interchange do not conform to these best practice recommendations. These best practice features include:

• **Straight Deck:**
  
  "Curved Decks causes complicated responses and additional torsional effects."

• **Equal Column Heights:**
  
  "Varying column heights causes focused stresses at certain columns usually causing damage to shorter or stiffer columns."

• **Foundations bedrock or firm alluvium:**
  
  "Foundations on softer soil causes magnified displacements of the structure that could results in liquefaction of the soil."

In addition, Ryall et al. (2000) states that, bridges in seismic prone areas should avoid skew abutment connections and single monolithic column to wide superstructure connections. Since the bridge forms part of an interchange, some of these structural features were unavoidable. These structural aspects could however subject the bridge to additional forces which could have been omitted during the design stage.

The mentioned features will be discussed in this literature review taking cognisance of past research. A brief history and insight into seismicity in the region is also presented.

2.1 Seismicity of the Region

This study investigates the structural response of the Stellenberg interchange when subjected to moderate earthquakes. It is useful at this stage to define a moderate earthquake and how it is used in this study. Numerous sources provide varying definitions of a moderate earthquake. This study employs the definition provided by the United States Geological Survey (USGS). The USGS defines a moderate earthquake as a seismic event with a magnitude ranging between 5.0
and 5.9 on the Richter scale with an intensity of VI - VIII on the modified Mercalli scale (MMI). These magnitudes and intensities were compared to the expected seismicity for the Western Cape.

Recurring seismic events with large magnitudes are unlikely to occur in the Cape Town region. Cape Town is situated approximately 3000 km from the edge of the African tectonic plate, as opposed to places such as Japan or San Francisco, which are situated on the edge of major tectonic plates. Furthermore, the closest major plate boundary to the Western Cape, where the African plate meets the Australian and South American Plates, are divergent plate boundaries. Divergent plate boundaries are less prone to frequent severe seismic events compared to convergent or transverse plate boundaries (Tarbuck et al., 2013). Moderate earthquakes are however likely in certain regions of the Western Cape. This is due to numerous minor faults and seismic clusters in this region.

Minor faults and seismic clusters caused the moderate earthquakes experienced in the Cape Town region. The Milnerton fault would most likely have the highest probability of effecting the Stellenberg interchange. The Milnerton fault lies approximately 10 km from the Cape Town CBD and the Stellenberg interchange as shown in Figure 2.1.

![Figure 2.1: Location of the Milnerton fault (Adapted from Hartnady (2004))](image)

The most prominent cluster in the Western Cape is the Ceres cluster. The Ceres cluster is situated at the Western end of the Kango - Baviaanskloof fault. This is an active seismic cluster experiencing earthquakes with magnitudes ranging between one and three on the Richter scale with an average six occurrences per month (Singh et al., 2009). This cluster was the epicentre of the well-known Tulbagh-Ceres earthquake that occurred in 1969.
The most destructive earthquake in South Africa’s history occurred in Tulbagh, approximately 120 km from Cape Town, on 26 September 1969. This earthquake measured an estimated 6.3 on the Richter scale. It claimed 12 lives and caused massive property and infrastructure damage. Between 1620 and 1902, there were numerous reports of large tremors in the Cape Town area. The largest of these tremors was estimated at a magnitude of 6.3 on the Richter scale which occurred in 1809. This earthquake was most likely caused by the Milnerton fault (Hartnady, 2004). The epicentre of the other reported earthquakes during the time, was stated as Cape Town, which is where the observers felt the tremors. The epicentre however could have been up to 100 km away. Since seismographs and other recording equipment were not available at the time, the epicentres are only estimates (Singh et al., 2009). Table 2.1 shows a brief summary of some significant seismic events that occurred in South Africa.

Table 2.1: Significant seismic events in South Africa (Singh et al., 2009)

<table>
<thead>
<tr>
<th>Date (Richter)</th>
<th>Location</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1809 6.3</td>
<td>Cape Town Region</td>
<td>• Collapse of farm houses</td>
</tr>
</tbody>
</table>
| 1932 6.3      | Cape St Lucia | • Serious damage to buildings  
|              |           | • Cracks in road surfaces |
| 1969 6.3      | Tulbagh   | • 12 people killed  
|              |           | • Serious damage and total collapse of some buildings  
|              |           | • Large cracks in road surfaces |
| 1976 5.2      | Welkom    | • Serious damage and total collapse of buildings |
| 1991 5.0      | Ceres     | • Damage to buildings |

Visser and Kijko (2010) estimated an earthquake magnitude of 6.0 – 6.87 on the Richter scale with a return period of 300 years would be appropriate for the Cape Town area. This worst-case scenario of a 6.87 magnitude earthquake would cause a level IX shaking intensity on the Mercalli Magnitude Intensity (MMI) scale. The MMI scale provides a representation of the ground movement at a certain point experienced by people, buildings and the environment. The greatest MMI value of an earthquake is experienced at the epicentre. This intensity reduces with distance away from the epicentre. It is estimated a level IX on the MMI scale would cause bridges not specifically designed for seismic excitation, as with the Stellenberg interchange bridge, to have a high probability of partial failure with permanent deformation or damage (GeoNet, 2012).

Kijko et al. (2003) established a probabilistic seismic hazard map in terms of peak ground accelerations (PGA) in addition to a spectral acceleration for South Africa. The maps indicate estimated peak ground acceleration with a 10% probability of exceedance with a 50-year period, which equates to a return period of 450 years. This is the same reliability index used in both the current building seismic loading code (SANS 10160-4) and the current bridge design code.
Figure 2.2 indicates that for the given return period, the Stellenberg interchange could experience a peak ground acceleration of approximately 0.2g. This is twice the magnitude specified in both SANS 10160-4 and TMH 7 codes. This is however similar to the PGA suggested in the previous seismic loading code, SABS 0160 [1989]. This design magnitude of 0.2g was reduced to 0.1g after a study by Wium [2010] concluded that the design PGA produced designs that were unrealistic and uneconomical. Figure 2.2 shows the peak ground acceleration hazard map developed by Kijko et al. [2003] superimposed with the peak ground acceleration contours from the SABS 0160:1989 for the Western Cape area. Figure 2.2 shows an accurate correlation between the two maps indicating that the PGA given in SABS 0160:1989 is accurate.

2.2 South African Design Codes

2.2.1 Current Seismic Loading Code

The current South African seismic loading code for all infrastructure excluding highway bridges and culverts, SANS 10160-4:2011, was adopted in 2009 and updated 2011. The code replaced the previous loading code, SABS 0160:1989. As mentioned, the seismic loading conditions of SABS 0160 was deemed to be too stringent resulting in uneconomic designs (Wium, 2010).

SANS 10160-4 (2011) distinguishes between two types of seismic excitation. Zone I denotes
natural seismic activity while Zone II denotes natural and mining induced seismic activity. These zones are categorised in terms of peak ground acceleration values with a 10% probability of exceedance in a 50-year period. Figure 2.3 shows the hazard map provided in SANS 10160-4 \cite{2011_sans}. 

![Figure 2.3: Seismic Hazard zone of South Africa (SANS 10160-4, 2011)](image)

The hatched areas on the map show the two different zones discussed. The hatched areas on the map are superimposed on a seismic hazard contour map published by the Council of Geoscience in 2003. The Zone I's hatching matches a 0.125g contour line of the original map and covers another 0.15g contour around the Milnerton area. SANS 10160-4 \cite{2011_sans} however requires the use of the 0.1g ground acceleration instead of 0.15g which reduces the effect of a potential earthquake, placing the infrastructure further at risk.

### 2.2.2 Current Bridge Design Code

The Technical Methods for Highways no.7 of 1981 parts One to Three, The Code of Practice for The Design of Highway Bridges and Culverts in South Africa, hereafter referred to as TMH7, is the current bridge and culvert design code used in South Africa. TMH7 has its own provisions regarding seismic loading. TMH7 also defines seismic regions on a seismic hazard contour map, which is shown in Figure 2.4. The map is defined in terms of the MMI scale, which is then classified in terms of peak ground accelerations. This conversion table is shown in Table 2.2.

Inspection of the map indicates that the Stellenberg interchange should be designed for a MMI level VII intensity earthquake. This relates to a design ground acceleration of 0.1g. The current
Table 2.2: Modified Mercalli classification (TMH 7, 1981)

<table>
<thead>
<tr>
<th>Modified Mercalli Intensity at epicentre</th>
<th>Maximum ground acceleration at epicentre</th>
</tr>
</thead>
<tbody>
<tr>
<td>ii-iii</td>
<td>0.003g</td>
</tr>
<tr>
<td>iv-v</td>
<td>0.01g</td>
</tr>
<tr>
<td>vi</td>
<td>0.03g</td>
</tr>
<tr>
<td>vii-viii</td>
<td>0.1g</td>
</tr>
<tr>
<td>ix</td>
<td>0.3g</td>
</tr>
<tr>
<td>x-xi</td>
<td>1g</td>
</tr>
</tbody>
</table>

Figure 2.4: Seismic hazard map from TMH 7 (1981)
building seismic code, SANS 10160-4, specifies this same peak ground acceleration for the region. The TMH7 additionally states:

"The map is, of necessity, approximate and can only be improved if and when new earthquake data becomes available."

This clause is relevant when examining research published by Kijko et al. (2003), discussed in Section 2.1, stating the region is susceptible to a 0.2g instead of a 0.1g earthquake.

TMH7 requires rigorous dynamic analysis using suitable computer software when designing important structures. The Stellenberg interchange bridge leads off and crosses over a national highway and is regarded as an important structure. At the time of the conceptual and final design, computer analysis software was very limited compared to modern computing capabilities. This further emphasises a need to re-evaluate the seismic and dynamic performance of the bridge.

### 2.2.3 Design Code Used for the Stellenberg Interchange

The Planning Manual Part 3: Bridge Design Manual (1977) published by the Cape Provincial Administration Department of Roads (Cape Prov. BDM) was used for the design of the Stellenberg interchange. It does not specify any requirements in terms of seismic loading conditions. The only mention of seismic excitation is as a source of movement when considering bearing connections. Table 2.3 shows a comparative summary between the different recommendations for seismic excitation with their respective recommended PGA values.

<table>
<thead>
<tr>
<th>Source</th>
<th>Seismic Consideration</th>
<th>Specified PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cape Prov. BDM (1977)</td>
<td>Not Specified</td>
<td>N/A</td>
</tr>
<tr>
<td>TMH 7 (1981)</td>
<td>Required</td>
<td>0.1g</td>
</tr>
<tr>
<td>SANS 10160-4 (2011)</td>
<td>Required</td>
<td>0.1g</td>
</tr>
<tr>
<td>SABS 0160 (1989)</td>
<td>Required</td>
<td>0.2g</td>
</tr>
<tr>
<td>Kijko et al. (2003)</td>
<td>Required</td>
<td>0.2g</td>
</tr>
</tbody>
</table>

### 2.3 Sub to Superstructure Connection

One of the most common failure modes in bridges subjected to seismic excitation is at the connection of the sub and superstructure. Various applicable features and failure modes are discussed in this section.

Columns are generally connected to the superstructure using two distinct methods. One of the methods to support the superstructure is to use a sliding-plate or pot bearing, which allows the superstructure certain degrees of freedom. The other method is to cast the column monolithically into the superstructure. Sample drawings of these connections are shown in Figure 2.5. Monolithic casting of the connection is generally less expensive in terms of materials and requires
less maintenance than bearing connections (Ryall et al., 2000). Monolithic connections provide the designer with a higher degree of redundancy since it provides fixity at the apex of the column. This reduces the effective length of the column, resulting in a higher effective resistance at mid height. This does however, result in heavier reinforcement required at the column apexes. Monolithic sub to superstructure connections are typically used for bridges with short spans or bridges utilising slender columns (Priestley et al., 1996).

![Diagram of Column to Superstructure Connections](https://scholar.sun.ac.za)

**Figure 2.5: Column to superstructure connections (Adapted from Priestley et al. (1996))**

Additional considerations are however required when designing monolithic connections. The additional fixity provided by the monolithic connections is advantageous for general bridge loading, but could be unfavourable when subjected to seismic excitation. During a seismic event, it is generally advantageous to have increased flexibility of the structure. This allows the structure to dissipate some of the energy through movement rather than resist the additional applied load. The additional rigidity of the structure does subject it to additional transferred moments and shear forces through the columns and into the superstructure. This effect is compounded when a single column connects to a wide superstructure (Ryall et al., 2000). The Stellenberg interchange utilises nine single column to wide superstructure connections.

Monolithic connections require additional attention to reinforcement detailing when it is subjected to seismic loading. The additional moments and shear forces induced by seismic excitation could potentially act in any direction, depending on the location and nature of the earthquake. These moments include the torsional behaviour of the superstructure during a seismic event. Special attention should be given to all the aforementioned forces and moments when designing the reinforcement for a monolithic column connection.

Box girder superstructures, as present in the Stellenberg interchange, typically utilise crossbeams at the column to superstructure connections. A general recommendation is to taper these crossbeams longitudinally along the section. A sudden transition with no tapering between a
general section and a crossbeam section could cause focussed stresses resulting in localised damage. Figure 2.6 shows an example of a tapered and an un-tapered box girder. The Stellenberg interchange longitudinally tapers the box girder sections at both sides of the crossbeams.

![Tapered and un-tapered cross-beams](image)

**Figure 2.6: Tapered and un-tapered cross-beams**

The concrete and steel reinforcement in the superstructure needs to transfer all of the forces and moments into the columns. Priestley et al. (1996) illustrated that single axisymmetric columns with monolithic connections are however inefficient in transferring these moments. Considering transverse loading of the bridge, the superstructure would resist movement through its transverse stiffness and its own in-plan torsional stiffness. For long span bridges, the transverse superstructure stiffness tends to become small in comparison to the rest of the structure. If the transverse stiffness of the superstructure is small, the columns need to provide the lateral resistance. This results in additional transverse column stiffness requirements. The longitudinal stiffness of a box girder is generally greater than the transverse stiffness, allowing the box girder to resist the majority of the lateral load instead of transferring the load into the columns. As less load is transferred to the column, lower column resistance is required in that specific direction. Axisymmetric sections provide the same resistance in any direction; this usually results in a section with excessive resistance in a particular direction. To improve the efficiency of the columns, rectangular pier supports are generally utilised. Piers supports however, are not always aesthetically pleasing. Another configuration is to utilise two axisymmetric columns side-by-side acting as a frame. The latter being the case with the double column arrangement of the Stellenberg interchange. As the Stellenberg interchange only utilises two of these side-by-side columns acting as a tall frame, the transverse behaviour of the Stellenberg interchange could be a potential cause for concern.

Considering the transverse behaviour of the bridge, the natural horizontal sway mode shapes of the deck could be excited during a seismic event. The first, lower frequency sway mode typically displaces the entire deck in one-half sine wave as depicted in Figure 2.7 (a). In this case, if the columns are sufficiently flexible, they should move freely with the deck. A higher frequency mode typically displaces the deck as a full sine wave, seen in Figure 2.7 (b). If this second mode shape is excessively excited, it could potentially induce large transferred torsional moments and shear forces from the deck into the connection. This could lead to unseating of the superstructure or spalling of the concrete in columns at the connections (Rai, 2008).

To disassociate the superstructure’s modes from the columns, omnidirectional-bearing connec-
Figure 2.7: Horizontal deck mode shapes

tions are typically utilised. These connections allow the columns to act semi-independently beneath the superstructure, allowing movement between the soffit and column apexes. In addition, these bearings cause no additional moments on the superstructure and reduces the chance of irreparable damage during seismic excitation. Bearing connections are however susceptible to unseating of the superstructure. The semi-independent behaviour occurs as the bearings are not completely free. They are within a pot or plate assembly with certain allowable horizontal force capacities. Exceedance of these horizontal forces leads to failure of the bearing and unseating of the bearings, causing severe damage. When bearings are used in support connections, it is important to consider the global behaviour of the structural system. A structural layout in terms of the structure’s load-displacement requirements should be carefully considered.

To account for thermal, vibrational and other horizontal forces in the longitudinal direction, some freedom of movement should be provided in the structural system. The superstructure needs to expand or contract from a fixed point in the structural system, with the displacement dissipated in pre-determined points of freedom. In the case of short span bridges, the fixed point is usually at one of the abutments. The opposite end abutment is typically free to move in the longitudinal direction by means of an expansion joint or bearing configuration. This type of layout is shown in Figure 2.8.

Figure 2.8: Fixed point at abutment (Hirt and Lebet, 2013)

Long span bridges with slender columns tend to utilise a different structural layout. The point of fixity in the system would generally be located near the middle of the bridge, i.e. at one of the internal columns. This is achieved by restraining the sub to superstructure connection
against movement at those columns. The flexibility of the slender internal columns allows
the superstructure to displace as the loading requires. Both abutment connections are then
free to move in the longitudinal direction at both sides by fitting expansion joints or bearing
configurations. If there are shorter, stiffer columns closer to the abutments, they are usually
fitted with bearing connections to facilitate the movement (Hirt and Lebet, 2013). A graphical
representation of this structural layout is shown in Figure 2.9.

![Figure 2.9: Fixed point at interior columns (Hirt and Lebet, 2013)](image)

This second structural layout was utilised for the Stellenberg interchange. The first two columns
from both sides are connected to the superstructure via bearings and the rest of the internal
columns are cast monolithically. While this is an effective design in terms of general traffic and
ambient bridge loading, it could be unfavourable during seismic excitation if it was not designed
accordingly. These movement effects are further emphasised and complicated when the deck is
curved.

## 2.4 Curved Deck

The curved nature of the bridge deck subjects the bridge to complex loading. The complexities
increase the vulnerability of the super- and sub-structural elements (Scawthorn and Chen, 2002).
These loading effects generally result in increased moments and torsional loads to the super and
substructure. The method of load transfer between the sub and superstructure is of great
importance for curved bridges.

One of the effects that are complicated in curved a deck is thermal expansion and other loads
resulting in longitudinal displacement of the superstructure. Straight bridges accommodate
these longitudinal loads and displacements by using expansion joints and bearing supports.
For curved bridges, longitudinal loading causes displacement in the longitudinal and transverse
directions. This effect is depicted in Figure 2.10. In the figure, the expansion is accommodated
by angled, unidirectional longitudinal bearings.

The additional imposed loadings that cause complicated effects are generally the secondary hori-
zontal loads. These forces act in a tangential direction to the curve, include braking and skidding
forces. Braking loads are very apparent in curved bridges as all vehicles reduce their speed when
entering a curve. This is emphasised at interchanges, such as the Stellenberg interchange, where vehicles are travelling at freeway speeds before entering a bend. These loadings are applied to all types of bridges but the difference is however in the direction of the load application with respect to the longitudinal axis of the bridge. For a straight bridge, it is resisted by the longitudinal stiffness of the bridge. For curved decks however, they are applied in a tangential direction to the curve of the bridge, which causes additional transverse moments and torsional effects on the super and substructure. Centrifugal forces are also present in a curved bridge deck but always act in a transverse direction to the curve.

All the aforementioned forces and considerations should of course have been included in the design of the Stellenberg interchange. Although it is highly unlikely that these load combinations would act simultaneously, a seismic event could accentuate some of the aforementioned transverse forces past the design capacities of the bridge, resulting in damage to the structure.

### 2.5 Columns

Ryall et al. (2000) defines a bridge column as a section that is designed to resist bi-axial bending moments. This generally follows that circular, square, octagonal or similar in-plan symmetrical cross-sections are employed. This produces a more efficient column in terms of bending resistance in both directions. This differs from pier sections that behave as reinforced concrete walls providing more moment resistance its transverse direction. The curved nature of the Stellenberg interchange imposes bi-directional moments on the columns through the fixed beam to column
connections. The bi-axial bending and the aesthetics of the columns were most likely the primary reason for the choice of column section utilised for the Stellenberg interchange. Two of the columns, C5 and C9, are however double columns, acting as very tall frame supports. These frames resist the additional transverse moments and forces induced by the curved deck.

Clarification on a number of terms used further in this section would be useful at this point. Firstly, when discussing the aspect ratio of a column, it refers to the height over depth ratio of the column, i.e. tall slender columns would have a high aspect ratio and short 'stocky' columns would have a lower aspect ratio. Secondly, the plastic zone or plastic hinge refers to a section of the column that would exceed its elastic capacity and continue into the plastic range of loading. This results in localised damage at the plastic hinge section, typically at either end of the column. This localised damage is partially acceptable as column are designed to sustain local damage and therefore not lead to complete collapse of the structure. The hinge acts as a damper to allow some rotation from the rest of the structure, instead of suffering further damage in those sections. The plastic zone is usually chosen and designed at points that could be repaired after the seismic incident, typically at the base of a column. The concept is to limit catastrophic damage of the entire bridge but rather restrict the damage to known, localised areas. Figure 2.11 shows a plastic hinge that formed in a column after the Niigata-Ken Cheutsu earthquake which occurred in Japan in 2004.

![Figure 2.11: Example of a plastic hinge (Bardet, 2004)](image)

Effective plastic hinges could prevent total collapse of the structure. Numerous other column failure modes pose a large probability of collapse. Priestley et al. (1996) discusses various failure modes for columns and sub-structure elements under seismic loading. The first problem presented, is flexural strength and ductility features of columns. Flexural strength is generally a
concern when engineers applied insufficient static lateral forces during the analysis of a structure to simulate seismic loads. This could be a result of a design code specifying an inadequate peak ground acceleration. In the current case, of the Stellenberg interchange, the factor is 10% of the design gravity load (TMH 7, 1981). In some cases in severe earthquake prone regions, it was noted that for the elastic response of the structure the load factor might exceed up to 100% of the gravity load. Although the Cape Town area is not susceptible to these severe earthquakes, in the 1970’s engineers in California used a mere 6% of the gravitational load for some structures situated in severe prone seismic areas (Priestley et al., 1996). This highlights the engineering community’s underestimation of seismic loads and shear loading in particular. Premature ending in lap length and connection reinforcement was also identified to result in partial or complete column failure at above or below mid height of the columns.

The next important failure mode addressed by Priestley et al. (1996) is shear failure of columns. Although it is predominantly a problem for short columns, the brittle nature of shear failure in concrete causes it to be a serious concern for all columns. Older bridges in California were found to have insufficient shear reinforcement to resist the shear force induced by seismic excitation. This led to the suspicion that design for shear forces was regarded as insignificant at the time. This suspicion was emphasised as engineers did not value the principles of interaction between flexural and shear capacity. Therefore engineers did not adhere to the design philosophy described in Priestley et al. (1996) for good column performance under seismic loading. Priestley et al. (1996) recommended that shear strength should at least equal or exceed the flexural capacity of a column. Figure 2.12 provides an indication of the severity of shear failure showing the failure of a column during the Niigata-Ken Cheutsu Earthquake.

![Figure 2.12: Example of a shear failure in a column (Bardet, 2004)](image)

Figure 2.12 shows the column failing in shear, close to the monolithic connection with the
superstructure. The severe spalling of the concrete resulted in the column losing significant bending capacity. Although it might still be supporting the superstructure axially, a slight eccentricity of the superstructure could introduce moments that could lead to collapse of the column. This lack of moment resistance could lead to complete failure, if the column undergoes further loading due to aftershocks or unfavourable ambient loading.

Shear failure is generally a result of shear detailing deficiencies of the columns, especially with respect to lateral and confinement reinforcement. Insufficient detailing reduces the ductility of the section, which results in a brittle failure. Pritchard (1997) reported, the most common reason for older bridges failing condition assessments is due to inadequate shear detailing. The codified design requirements at the time for shear reinforcement were based on an empirical approach. The codes produced adequate designs when the entire procedure specified by the code is followed. The empirical methods however produced inadequate designs when only certain selections from the empirical methods were chosen. This led to numerous misinterpretations of shear link spacing and anchorage of shear reinforcement. These inadequate designs generally resulted in large shear link spacing when compared to modern equivalent designs. This was largely due to a misinterpretation or misunderstanding of the interaction between main tension reinforcement and shear reinforcement (Pritchard, 1997). The importance of comprehensive shear design is highlighted by the failure mode of the Kobe Expressway Bridge in Japan. During the Hyogo-Ken Nanbu earthquake in 1995, a 600 m portion of the bridge collapsed due to inadequate shear detailing resulting in an inductile column. A photograph of the failed section is shown in Figure 2.13.

![Image of Kobe Expressway Bridge failure](image)

Figure 2.13: Shear failure of the Kobe expressway (Priestley et al., 1996)

Increased ductility and the possible incorporation of plastic hinge designs could have prevented this complete failure. The shear behaviour within a ductile zone however should be carefully considered. The shear detailing of a plastic hinge is critical as the formation of a plastic hinge in a column reduces the shear capacity of the section in the plastic region. This effect is shown in Figure 2.14 where a column is at the verge of complete collapse after the San Fernando earthquake in 1971. This type of failure could lead to collapse of a bridge if not properly compensated for in the design.

The shear capacity of shorter columns is greater than slender columns with the same cross...
section. The torsional and bending capacity of shorter columns however have no appreciable reduction in capacity than with slender columns (Prakash et al., 2010). The reduction in aspect ratio of the column however, reduces the size or height of the possible plastic zone at the top and bottom of the columns. This should be accounted for in the amount and placement of shear reinforcement in the plastic zone.

Another consideration in terms of shear capacity is torsional stress in the columns. Torsion is effectively a radial shear stress within the column which adds to the shear stress already present in the column. Torsional loads are especially important with respect to the Stellenberg interchange due to the aforementioned secondary torsional effects induced by the curved nature of the bridge. Prakash et al. (2010) observed that columns with a higher aspect ratio experience core degradation at mid height of the column under torsional loading. Columns with a lower aspect ratio conversely show a uniform distribution of stress, resulting in no localised damage at mid height of the column. This could potentially be a problem for the Stellenberg interchange as the amount of steel reinforcement reduces along the height of the column while increasing close to the apex. This leads to a reduced column capacity at mid height, which could lead to core damage.

The slenderness and variance in height of the columns supporting the Stellenberg interchange should produce additional torsion to the columns. This might not have been accounted for in the initial design (Prakash et al., 2010). These torsional effects are only observed when subjected to dynamic analysis, typically not available during the 1970’s and 1980’s when the Stellenberg interchange was designed. It is possible that this additional torsional stress induced by the variance in column height and the curvature of the bridge deck, could cause damage to the columns due to insufficient reinforcement detailing.

Prakash et al. (2010) observed that columns with a higher aspect ratio are more flexible and attract less load than shorter and stiffer columns. This is a favourable feature of the Stellenberg interchange. It leads to the assumption that it was the reason for placing bearing plate supports
at the apexes of the first two interior columns. The first two interior columns from both sides (C2, C3, C11, C12) have the same cross-sectional properties as the taller columns at mid span of the bridge, most likely for aesthetic purposes. This would result in stiffer and more rigid columns in comparison and should attract higher stresses causing localised failure at the connections. As discussed in Section 2.3 the addition of a bearing pad would reduce the fixity which in turn redistributes some of the imposed moments and torsional forces exerted on the bridge. With this in mind however, the third column from either end (C4, C10) are monolithically cast and are not significantly taller than the second columns (C3, C11). This would result in these columns being relatively stiffer than the other interior columns, with the additional fixity at their apexes. The concern with these columns is emphasised as they are situated next to the columns with bearing connections. The movement of the superstructure, which is increased by the bearing supports, would have to be resisted by these column resulting in focussed stresses. The sudden level of fixity has to be resisted by these columns. During seismic excitation displacements could increase beyond design limits which could lead to columns damage.
Chapter 3

Physical Testing

The physical testing of the bridge was conducted to calibrate the finite element model. Ambient vibrational testing was conducted using accelerometers placed along the centreline of the bridge. The processing of the accelerometer measurements yielded the structure’s natural frequencies along with their corresponding mode shapes. All aspects regarding the physical testing conducted for this study is discussed in this chapter.

3.1 Test Plan and Outcomes

The physical testing of the Stellenberg interchange provided the results required to calibrate the FE model. This verification was performed by comparing the mode shapes and corresponding natural frequencies of the structure. The natural frequency of a structure is a function of the mass, stiffness, damping and boundary interaction of a structure (Craig and Kurdila, 2006). The calculation of the mass can be accurately achieved by specifying accurate geometry and material properties. This leaves the stiffness, damping and boundary interactions as unknowns. Due to the complex interactions between the soil and the structure, the boundary interactions were approximated using methods discussed in Chapter 4. The damping of the structure was measured during the testing. Thus, once the FE mode shapes and frequencies matched the actual structures behaviour the stiffness of the FE model is calibrated.

The height and accessibility of the bridge necessitated that the accelerometers be mounted on the road surface to perform the physical testing. This, accompanied with the road width and vehicle speed, necessitated some form of closure of the bridge to ensure the safety of the testing equipment and personnel. One closed lane provided sufficient space to setup all of the required equipment. In order for the bridge to vibrate, some form of excitation is required. Typically, it is less complicated to determine the required dynamic behaviour of the bridge by using an inertial shaker. The shaker can be used to excite certain modes and then record the structure’s response. This however requires that the structure should only be excited by the inertial shaker and no other external factors, such as in this case, moving traffic. As the bridge leads of a national highway, the South African National Roads Agency (SANRAL), which is the governing
transport authority, could not allow a total closure of the bridge. This would lead to serious congestion of the N1 even during non-peak hours, which was considered. SANRAL did however permit the closure of the left hand side lane and shoulder of the bridge. A photograph of the lane closure on the approach of the bridge is shown in Figure 3.1. The accommodation of traffic plan supplied by the routine road maintenance coordinator, Mr Robin Davidse from MD Civils, is attached in Appendix A.

![Figure 3.1: Lane closure on approach](image)

The traffic present during the testing would neutralise the effectiveness of an inertial shaker. Instead of using an inertial shaker, the bridge was excited by ambient vibration from the wind and small vehicles. This form of testing allowed the right hand side lane to be open for traffic while allowing sufficient space for the testing equipment. Time steps during the passing of heavy vehicles over the bridge were recorded. These time steps were later discarded from the results as the large vehicles’ mass, causes a mass damping effect, which influences the dynamic behaviour of the structure.

All tests were conducted within a 7-hour period granted by SANRAL. The tests were conducted on a Sunday due to reduced traffic volume during weekends. This ensured that there would be no congestion on the N1 because of the lane closure. The allotted time had to include the closing and reopening of the lane.

Because of the allotted time, only six of the twelve spans were tested at third intervals along each span. This configuration allowed the maximum number of tests to be performed. The six spans were sufficient to calibrate the finite element model. More spans could have been tested if the resolution or measuring point spacing were increased. This however requires more interpolation between measuring points at the signal processing stage, resulting in averaged and less accurate data. Testing half the spans at third intervals along each span was selected as an optimal solution.
3.1.1 Accelerometer Placement and Orientation

The accelerometers were placed at third intervals along the centre line of each span of the bridge. Due to the strict time restraint, tests could only be conducted along a single line and not as a grillage over the entire deck. A grillage accelerometer placement is required to record torsional deck modes. Although the curved structure would certainly have numerous torsional modes, the centreline placement of the accelerometers would exclude these modes. This was intentionally done as torsional modes identified by only a single line of accelerometers could appear to be vertical or sway modes during the signal processing stage. The beam elements utilised in the initial and final models also eliminates torsional deck modes. The elimination of the torsional modes in both cases enabled easier identification of the mode shapes between the physical tests and FE models.

The accelerometers were built in an assembly consisting of three accelerometers in three orthogonal directions. This allowed the assembly to record accelerations in three directions. The three accelerometers were mounted on an aluminium plate that was levelled using a bubble level fitted to each assembly. The assemblies were placed at the measuring points (MP) to capture the vertical and two orthogonal accelerations. One of the four assemblies only consisted of two accelerometers with a vertically and horizontally orientated accelerometer. The accelerometer measuring points 1 through 19 are shown in Figure 3.2 with a photograph of test points 17 through 19 seen in Figure 3.3.

![Figure 3.2: Accelerometer Layout](image)

3.1.2 Equipment Used

A brief summary of the equipment used to perform the physical test is discussed in this section.

**Accelerometers**

Eleven Honeywell Q-Flex QA–700 force balance accelerometers were used and built into four separate assemblies as discussed in subsection 3.1.1. These accelerometers have a sensitivity of
6V/g and a resolution of <1µg, which is suitable to capture the required vibrations (National Instruments, 2009). Force balance accelerometers consist of a balanced mass that lies between electronically equivalent springs. Movement of the accelerometer moves the suspended weight and applies a force to the springs. These force and mass parameters is then converted to accelerations using Newton’s second law of motion, which is the product of force and mass. Figure 3.4 shows a photograph of an accelerometer assembly.

Force balance accelerometers were beneficial for two reasons in the testing of the Stellenberg interchange. Firstly, they are able to record very low frequency vibrations, which is necessary to capture the first mode shapes of the bridge. Secondly, the force balance accelerometers are current driven, instead of voltage driven. Current driven accelerometers produce less noise and signal loss in cables, enabling the use of longer cables. The use of the longer cables made it possible to perform all seven test setups with only one fixed reference point, significantly reducing the set up time between tests. The long cables were connected to a conversion unit, converting...
the current into voltage. To minimise and standardise the losses from the conversion unit to the data acquisition system (DAQ), short, standard length cables were used to connect the signal conversion box to the DAQ.

Data Acquisition System

Specialised data acquisition hardware designed for vibrational test, developed by National Instruments was used for capturing the data. The accelerometers connect to NI PXI-4472B device-cards mounted on a NI PXI-1045 chassis, both developed by National Instruments. These cards have a voltage range of -5V to +5V. This system allows the user to capture all channels simultaneously with 24-bit, high-resolution analogue-to-digital converters. The system has anti-aliasing filters to prevent aliasing and noise from effecting the measurement quality. Figure 3.5 shows a photograph of the DAQ system setup in the testing van used on the day of testing.

![Figure 3.5: Mobile test unit](image)

The DAQ system connects to a PCI card on a standard desktop workstation via a fibre cable. LabVIEW Real-Time module software was used to capture the data to the workstation and to perform preliminary checks on the results.

Surveying equipment

The initial distances between points were measured using a surveying wheel and a 100m measuring tape. Upon placement of the accelerometers, a Leica Disto D210 laser distance-measuring tool was used to set out the measuring points. All MPs were placed along the centre line of the bridge. Transverse distances were measured from the base of the balustrade at a perpendicular angle. Longitudinal distances were measured from the previous point, with the first point at the known start of the bridge.
3.2 Test Set-ups

In order to test all six spans with the available equipment, seven test set-ups were developed. Accelerometer assemblies 1, 3 and 4 consisted of three accelerometers while assembly 2 consisted of only two accelerometers. Accelerometer assembly 4 was placed at the fixed reference point and remained in that position for the entire duration of the tests. The reference point is located at measuring point 9 which is shown in Figure 3.2.

All test set-ups followed a similar two-staged pattern. Setup 1 had accelerometer assemblies one to three located at MP1 to MP3 and assembly 4 at the reference point (MP9). Assembly 2 would then record in the transverse and vertical directions, with the other assemblies recording in all three directions. Setup 2 would have assemblies one and three placed at MP4 and MP5. Assembly 2 remained at MP2 but was rotated to capture vibration in the longitudinal and vertical direction. The second vertical measurement at MP2 was used to compare the measurements between the two setups. This pattern was repeated for all the test setups. Figure 3.6 shows a graphical representation of the first two, and part of the third test setup.

3.2.1 Test Details

All seven test set-ups were tested using the same test parameters. The sampling rate was fixed at 2048 Hz for each test. The minimum recording frequency of the device cards were used instead of the recommended minimum of twice the structure’s natural frequency (National Instruments, 2009).

Even though the sampling rate was very high, the nature of ambient vibrational testing requires long time series recordings. This is due to the source of excitation during ambient tests. These sources are either short as in the case of vehicles passing by or have a very moderate intensity in the case of light winds. To increase the signal resolution and to obtain reliable results, the recording-time of the test series had to be increased. All seven test setups were tested for a minimum of 20 minutes as recommended by National Instruments (2009).

The re-sampling rate explained in the following section was set at 1024 Hz with the recommended overlap of 66.67%. This overlap is the percentage of overlap between the re-sampled datasets.
A low-pass filter also explained in the following section was used to narrow the results down to a band of interest between 0 Hz and 13 Hz.

### 3.3 Signal Processing

Obtaining the raw vibrational data is the first step in obtaining the dynamic properties of a structure. The signal processing of the acquired vibrational data produces the required modal results from the recorded vibrational data. To understand some of the aspects involved with the signal processing it is important to review some analysis and signal processing theory.

Generally, there are two types of modal analysis, Experimental Modal Analysis (EMA) and Operational Modal Analysis (OMA). EMA was the first method developed. The concept of EMA is to excite a structure, where after the structural response is recorded. This requires a measurable input force to excite the structure. Typically, an impact hammer or an inertial shaker is used. When a measured input force is used, it is referred to as Forced Vibrational Testing (FVT). This type of testing requires all input forces to be measured. This is very difficult when attempting to perform tests on large structures subjected to significant ambient loads. These ambient forces are very difficult to measure as input parameters.

To bypass these difficulties, Operational Modal Analysis was developed. This type of analysis does not require the structure to be excited by a measured force but rather by ambient and operational forces. This is called Ambient Vibrational Testing (AVT). AVT does not completely remove the need for measured input forces. The input forces are partially assumed using certain parameters, such a Gaussian white noise for this specific case. In addition to these assumptions, the data recorded at the reference point is used as input values for the roving accelerometers. As the reference point is at a stationary position during all the tests it can be used as a benchmark or input parameter. As mentioned in preceding sections an OMA was performed to obtain the dynamic properties of the Stellenberg interchange.

To understand some of the transformations performed on the raw data it is important to distinguish between time and frequency domain. The data captured by the accelerometers is in time-domain. This is the acceleration captured in \( m/s^2 \) over time measured in seconds. With tests running over 20 minutes and logging data points every millisecond, data sets become extremely large. This typically results in data that is difficult to interpret.

The data is then transformed to frequency domain to allow for easier interpretation of data sets. This is typically achieved using Fast Fourier Transforms (FFT). The FFT decomposes a complex combination of frequency components in to their various basic components. The basic components are represented as peaks in a frequency domain diagram. The frequency domain enables a viewer to identify estimates of the peak frequencies in the vibrational data. Figure 3.7 shows a graphical representation of a basic sample of the differences between time and frequency domain. Figure 3.7 also indicates how a FFT decomposes the component frequencies as peaks.

For large or complex structures, the data seen in frequency domain directly from a FFT is usually
too condensed to identify the peaks. This is because the signal is too complex and consists of too many basic component frequencies. The FFT data is therefore further transformed and filtered to produce a concise curve, called a Frequency Response Function (FRF). A FRF is fitted to the data and clearly indicates the natural frequencies present in the vibrational data. There are numerous methods to filter the data in order to obtain a distinct FRF. Firstly, the data is de-trended to remove any trends induced by noise in the accelerometer time-history signals. This ensures the data oscillates about the same axis throughout the time series. Secondly, the data is then re-sampled to bring all the records to a common sampling rate and duration. This reduces spectral leakage between data sets. Finally, the data can be passed through a low-pass signal filter. This is done in frequency domain. The low-pass signal filter removes the data points that are not of any relevance to the specific case, e.g. high (+30 Hz) frequencies when examining large civil structures.

Windowing is another method used in order to obtain a distinct FRF. This method reduces spectral leakage that could occur when some of the initial assumptions used during the FFT are not met or, the total set of cycles does not meet the predefined integral number of cycles assumed by the FFT. When various test set-ups are measured to capture ambient vibrations on large structures, it becomes impossible to match this number of cycles. Some spectral leakage would take place. This leakage distorts the amplitude data between set recording points that could result in inaccurate mode shape composition. Using windows over the data minimises this effect. Past tests using different windows over various data sets have found that a Hanning window is most suitable for vibrational test on large structures. Details for the Hanning window including its definitions can be found in a manual published by National Instruments (National Instruments, 2009). A Hanning window was used on the vibrational data for the Stellenberg interchange to produce a distinct FRF and to obtain the modal parameters.

To obtain the values of the modal data required, they have to be selected from the data. This can be done in either frequency or time domain. To ensure accurate results, it is generally
recommended to gain a set of results from both frequency and time domain and compare them with each other. In both domains numerous methods have been developed, each with their own advantages and disadvantages for their specific purpose. For this study, Enhanced Frequency Domain Decomposition (EFDD) and Peak Picking (PP) were used in the frequency domain and Stochastic Subspace Identification (SSI) was used in time domain. All the methods have the ability to produce natural frequencies, mode shapes and structural damping ratios.

Peak picking was the first modal identification process developed. It produces reliable results if the mode shapes are not closely spaced and the structure has low damping. Peak picking enables the user to scan through the processed data in frequency domain and pick out certain frequencies that indicate peaks throughout all the test data. When modes are closely spaced, these peaks start to merge and picking between them could become subjective. Signal processing software was developed to partially remove this subjectivity by performing automated peak picking procedures.

EFDD is a further development on the original FDD (Frequency Domain Decomposition). This method estimates a spectral matrix, which performs a single value decomposition of the entire spectral matrix at each frequency and then reconstructs single value decompositions. This is then used to identify the natural frequencies. This method was refined to EFDD to produce structural damping. EFDD is an improvement on the peak picking method as it can still provide the required modal properties, while accurately identifying closely spaced modal frequencies. The EFDD provides a more automated process that is easier to perform and less subjective than the peak picking method.

Stochastic subspace identification is an advanced OMA technique that derives an idealised model for the dynamic behaviour of the structure. Brincker and Andersen [2006] detailed the mathematical methods and framework for the SSI method. Due to the level of detail required it is not explained in this dissertation. There are three different algorithms under SSI namely, Unweighted Principle Component (UPC), Principal Component (PC) and Canonical Variate Analysis (CVA). Each of these was developed for different applications with UPC being an ideal algorithm for obtaining mode shapes from acceleration data.

For this study EFDD and SSI-UPC was primarily used to obtain the results with the selected data being confirmed with the peak picking method.

All signal processing results for this study were obtained using a commercial OMA suite called ARTeMIS Modal Pro 2015. The vertical modes were checked using ME’scope Modal software. ARTeMIS Modal Pro has all the methods discussed in this section programmed into the software and provides a user-friendly interface that graphically produces the measured mode shapes.
3.4 Results

The results obtained from the OMA for the five most prominent modes are presented and discussed in this section. These results are based on the two OMA algorithms used namely; EFDD and SSI-UPC. Table 3.1 shows the summary of the test results.

<table>
<thead>
<tr>
<th>Mode</th>
<th>EFDD Frequency (Hz)</th>
<th>SSI Frequency (Hz)</th>
<th>MAC (%)</th>
<th>Damping Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.855</td>
<td>2.824</td>
<td>97.9</td>
<td>0.811</td>
</tr>
<tr>
<td>2</td>
<td>3.041</td>
<td>3.046</td>
<td>94.3</td>
<td>0.743</td>
</tr>
<tr>
<td>3</td>
<td>3.383</td>
<td>3.451</td>
<td>90.5</td>
<td>0.537</td>
</tr>
<tr>
<td>4</td>
<td>3.368</td>
<td>3.871</td>
<td>95</td>
<td>0.935</td>
</tr>
<tr>
<td>5</td>
<td>9.99</td>
<td>10.03</td>
<td>87</td>
<td>0.648</td>
</tr>
</tbody>
</table>

The MAC or Modal Assurance Criterion value listed in Table 3.1 is a widely used method of checking the validity of recorded mode shapes. It relates different modes that were measured using different methods and provides a check to determine whether the frequencies and the actual mode shapes match. It produces an easy to understand value of whether the test results are dependable. A value close or equal to one shows perfect or good correlation and that the modes only differ by a scale factor. A value close or equal to zero shows that the modes are orthogonal to one another. MAC values higher than 0.8 is widely considered as an acceptable modal match with MAC values lower than 0.4 considered as a very poor modal match. As seen in Table 3.1, all MAC values for the tests performed on the Stellenberg interchange are above 0.87, signifying that the modes show good correlation and thus reliable.

Figure 3.8 shows the spectral decomposition of the results obtained from the three recorded directions. The matching peaks on the data shows the natural frequencies and is highlighted with vertical indication lines.

A stabilisation check can be performed on the results obtained from the SSI method. This ensures the method produces reliable results and checks whether the frequencies correspond to natural mode shapes. This was conducted for all tests. As an example, the stabilisation diagram of the third test setup is shown in Figure 3.9. The straight vertical line throughout the data set at the identified frequencies indicates a good representation of the modes. A jagged line would indicate poor representation of modes.

Figure 3.10 shows a summary of the stabilisation diagram with all tests superimposed. The diamond markers highlight each frequency from each test set.

The mode shapes corresponding to the five most prominent frequencies are shown in Figure 3.12. Only half of the spans were measured at intervals of a third of the length of each span along each span, as is visible in the mode shape plots. The mode shapes show realistic values with prominent node points at column apexes as expected. The mode shapes were similar to those predicted with the initial model.
Chapter 3. Physical Testing

Figure 3.8: Spectral values from physical testing

Figure 3.9: SSI-UPC Stabilisation Diagram for Test Setup 3
Chapter 3. Physical Testing

Figure 3.10: SSI-UPC stabilisation diagram for all test setups combined

The MAC diagram shown in Figure 3.11 graphically indicates the MAC values for the combined test series. The diagram shows good correlation across the diagonal of the matrix, with minor off diagonal values present. This indicates good correlation between recorded modes.

Figure 3.11: MAC diagram for the physical testing
Section 3. Physical Testing

Mode 1

2.824 Hz

Mode 2

3.045 Hz

Mode 3

3.451 Hz

Mode 4

3.871 Hz

Mode 5

10.03 Hz

Figure 3.12: Physical testing results
Chapter 4

Finite Element Modelling

The results and conclusions of this study were obtained through the utilisation of a finite element (FE) model. The physical testing results enabled the calibration of the FE model to ensure it produces accurate results. The final model was developed using SIMULA’s Abaqus FE analysis software. Abaqus is a general finite element analysis program that is used by various engineering disciplines. Abaqus has the functionality to obtain all the required forces, displacements and stresses required to evaluate the structure. An important feature of these types of dynamic analysis is the functionality to use recorded base acceleration data as input variables. This was used to simulate seismic events using data recorded during specific earthquakes. The FE model was periodically refined from the simplest beam model to the final calibrated model. An initial model was developed using Prokon and Abaqus.

4.1 Initial Model and Comparison

The initial model was developed prior to physical testing to determine appropriate placement of the accelerometers. During the initial modelling an identical model was developed using two software packages. Prokon frame-analysis software was used for the first model and then replicated in Abaqus. Prokon is very restricted in terms of the requirements for this study. It does however, provide a simple and easy to modify input user interface. This interface simplified the initial complications in model development. The additional functionality and depth of modelling available in Abaqus increased the complexity of the modelling procedure. This generally results in errors in the model that are difficult to identify and correct. The Prokon model was used as a reference in the development of the initial Abaqus model to ensure there are no obvious errors in the model.

4.1.1 Element Selection and Material Properties

The initial model was developed using linear three-dimensional beam elements with 6 degrees of freedom per node. Each node has three translational degrees of freedom in each axis direction.
as well as three rotational degrees of freedom (DOF) about each axis. Figure 4.1 depicts these 12 element degrees of freedom.

![Three-dimensional beam element](image)

**Figure 4.1: Three-dimensional beam element**

Beam elements provided sufficient force and displacement information while being very computationally efficient. The use of beam elements resulted in significantly higher computational efficiency compared to solid three-dimensional elements. Beam elements would however not be able to simulate torsional deck mode shapes. This is acceptable as the relative stiffness between slender columns and superstructure suggest the primary mode shapes would not include torsional modes. Quadratically interpolated shear-flexible (Timoshenko) B32 beam elements using full integration were used for all beam elements in all Abaqus models (SIMULA, 2013).

Although specific material stresses could not be obtained from the beam elements, the use of solid three-dimensional elements to model the global structure would be computationally inefficient. Appropriate composite section and material properties were developed to overcome this problem.

The cross-sectional properties of the beam elements were calculated using ProSec, a sub module of the Prokon package. ProSec calculates the required cross-sectional properties of a complicated geometrical shape. The geometry was drawn using a suitable CAD software and imported into the ProSec. All the required cross-sectional properties for the beam elements are shown in Table 4.1. For the initial models, the double columns were modelled as an effective single beam element. This was later refined to two separate columns in the final model.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Box Girder</td>
<td>6.177</td>
<td>2.477</td>
<td>49.640</td>
</tr>
<tr>
<td>General Column</td>
<td>1.401</td>
<td>0.157</td>
<td>0.157</td>
</tr>
<tr>
<td>Large Column (C6, C7, C8)</td>
<td>2.118</td>
<td>0.358</td>
<td>0.358</td>
</tr>
<tr>
<td>Effective Double Column (C5, C9)</td>
<td>2.804</td>
<td>0.313</td>
<td>5.650</td>
</tr>
</tbody>
</table>

Table 4.1: Cross-sectional properties

The material properties for the initial model was used as specified in SANS 10100-1 (2000). The code specifies a density of 2300 kg/m$^3$ for normal weight aggregate concrete. The elasticity moduli for 35 MPa concrete is 29.5 GPa and 31 GPa for 40 MPa concrete. All columns were constructed using 40 MPa concrete while the box girder section was constructed using 35 MPa concrete.
4.1.2 Boundary Conditions

Accurately assigning boundary conditions to a model is an important aspect in the modelling process. Due to the nature and variability of founding conditions, accurate modelling of the boundary conditions can be difficult and computationally inefficient. This is due to unknown soil conditions coupled with the complicated interaction between a structure’s foundations and the soil. This complexity makes it difficult to model these conditions without developing a three-dimensional soil model, which would be computationally inefficient. Suitable assumptions and modelling techniques are therefore required to achieve sufficient accuracy while maintaining computational efficiency.

The Stellenberg interchange implements two types of foundations. The interior columns C5 to C7 are supported by pile group foundations, while the remainder of the columns C2 to C4 and C8 to C12 are supported on pad footings. The column positions are shown in Figure 4.2.

![Figure 4.2: Plan layout of the bridge](image)

The boundary condition between the structure and the soil is not the only interaction that requires specific evaluation. The interaction between the sub and superstructure is an important feature in the dynamic behaviour of the structure. The Stellenberg interchange implements various types of connections between the sub and superstructure. Both abutments (C1, C13) and first interior columns from both sides (C2, C12) are connected to the superstructure through unidirectional plate bearings. The abutments are connected by two side-by-side unidirectional plate bearings. This restricts rotation about the vertical axis at the abutments but allows longitudinal movement. The second interior columns from both sides (C3, C11) are connected with fixed plate bearings. The fixed bearings only allow rotational movement about the vertical axis. All further columns (C4-C10) are cast monolithically to the superstructure.

Table 4.2 presents a summary of the boundary and interaction conditions used for the initial model compared to the actual bridge conditions. The fixed support condition restrains all degrees of freedom against any movement at that node. The pinned condition transfers all translational degrees of freedom from one node to the other but does not transfer the rotational degrees of freedom. This simulates a pinned connection, which was used to model the bearing connections. The full transfer condition transfers all the degrees of freedom from one node to the other. This
provides a full link between nodes and was used for the monolithically cast connections.

Table 4.2: Initial model boundary conditions

<table>
<thead>
<tr>
<th>Support</th>
<th>Actual Structure to Soil Interaction</th>
<th>FE Structure to Soil Interaction</th>
<th>Actual Column to Deck Interaction</th>
<th>FE Column to Deck Interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Side-by-side bearings</td>
<td>Pinned</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>C2</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Bearing</td>
<td>Pinned</td>
</tr>
<tr>
<td>C3</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Bearing</td>
<td>Pinned</td>
</tr>
<tr>
<td>C4</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C5</td>
<td>Pile Group</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C6</td>
<td>Pile Group</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C7</td>
<td>Pile Group</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C8</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C9</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C10</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Monolithic</td>
<td>Full Transfer</td>
</tr>
<tr>
<td>C11</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Bearing</td>
<td>Pinned</td>
</tr>
<tr>
<td>C12</td>
<td>Pad Footing</td>
<td>Fixed</td>
<td>Bearing</td>
<td>Pinned</td>
</tr>
<tr>
<td>C13</td>
<td>Side-by-side bearings</td>
<td>Pinned</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

4.1.3 Initial Model Results

The initial model provided the first insight into the dynamic behaviour of the structure. A modal analysis was performed with the mode shapes and corresponding frequencies extracted from both models. The results obtained from the Prokon model were compared to the Abaqus model. The results of the first two sway and the first two vertical mode shapes are shown in Figure 4.3.

The results indicate good correlation between the two models, with slight variations of approximately 3% – 4% due to mesh sizing and minor refinements in the Abaqus model. This correlation does not signify accuracy with regards the actual structure but does show the initial Abaqus model was correctly modelled.

4.2 Finite Element Model Tuning

Adjusting the appropriate parameters and refining the FE is required to obtain an accurate representation of the actual structure. This section discusses the various model refinements and adjustments made to the initial model to develop the FE final model.
<table>
<thead>
<tr>
<th></th>
<th>Prokon Model</th>
<th>Abaqus Model</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1\textsuperscript{st} Sway Mode</strong></td>
<td><img src="image1" alt="Prokon Model" /></td>
<td><img src="image2" alt="Abaqus Model" /></td>
</tr>
<tr>
<td></td>
<td>0.67Hz</td>
<td>0.63Hz</td>
</tr>
<tr>
<td><strong>2\textsuperscript{nd} Sway Mode</strong></td>
<td><img src="image3" alt="Prokon Model" /></td>
<td><img src="image4" alt="Abaqus Model" /></td>
</tr>
<tr>
<td></td>
<td>0.81Hz</td>
<td>0.80Hz</td>
</tr>
<tr>
<td><strong>1\textsuperscript{st} Vertical Mode</strong></td>
<td><img src="image5" alt="Prokon Model" /></td>
<td><img src="image6" alt="Abaqus Model" /></td>
</tr>
<tr>
<td></td>
<td>2.41Hz</td>
<td>2.50Hz</td>
</tr>
<tr>
<td><strong>2\textsuperscript{nd} Vertical Mode</strong></td>
<td><img src="image7" alt="Prokon Model" /></td>
<td><img src="image8" alt="Abaqus Model" /></td>
</tr>
<tr>
<td></td>
<td>2.96Hz</td>
<td>3.06Hz</td>
</tr>
</tbody>
</table>

Figure 4.3: Initial model comparison
4.2.1 Composite Material Properties and Section Definitions

To model the columns using three-dimensional solid elements would be computationally inefficient. The columns were therefore modelled using B32 beam elements. These elements are three-dimensional, quadratically interpolated element with six DOFs per node. To employ the accurate use of these elements, the reinforced concrete columns required equivalent composite material properties. This was achieved using the section transformation theory discussed by Hibbeler (2014). The theory relates the different areas of the materials present in the section. It uses this relationship to calculate a composite modulus of elasticity and a composite material density. The theory is best understood when examining the equations shown in Equation 4.1 and 4.2 for the modulus of elasticity and densities respectively.

\[
\bar{E} = \frac{E_s A_s + E_c A_c}{A_s + A_c}
\]  

(4.1)

\[
\bar{\rho} = \frac{\rho_s A_s + \rho_c A_c}{A_s + A_c}
\]  

(4.2)

where:

- \(A_s\) : Area of Steel
- \(A_c\) : Area of Concrete
- \(E_s\) : Elastic Modulus of Steel
- \(E_c\) : Elastic Modulus of Concrete
- \(\rho_s\) : Density of Steel
- \(\rho_c\) : Density of Concrete

Since the mass and stiffness are key parameters when considering the modal behaviour of the structure it becomes very important that the equivalent properties be correctly determined. When examining the reinforcement drawings, it is noted that the amount of steel in the sections could have a significant effect on the composite section properties. The reinforcement drawings could unfortunately not be made publically available as per the contractual agreement with SANRAL. The method used by Hibbeler (2014) further incorporates effective moments of inertia of the cross-section to further refine the section behaviour. This was not included as the reinforcement layouts found in the box girder section were too complex. For the column sections, symmetry would cause the additional refinements in the method to have a lesser effect than it would have with asymmetrical sections.

The base material properties that were used for the calculations of the composite sections are summarised in Table 4.3. The values used are recommended in SANS 10100-1 (2000). Unfortunately, core samples of the concrete sections could not be taken and tested to determine the actual material properties. The agreement with SANRAL prohibited any destructive testing measures.

The total area of steel in the column sections was calculated from the total steel mass obtained from the as-built bending schedules. An effective volume per meter, which is equal to the effective area in the section, was calculated. This area was subtracted from the total section area calculated as mentioned in Section 4.1. This resulted in the total concrete area for the columns. As the box girder section has numerous bending schedules that include different applicable
section stages, the same method could not be followed for the composite deck material.

Table 4.3: Material properties for composite section calculations

<table>
<thead>
<tr>
<th>Density [kg/m$^3$]</th>
<th>Poisson’s Ratio</th>
<th>Modulus of Elasticity [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>35 MPa Concrete</td>
<td>2300</td>
<td>0.2</td>
</tr>
<tr>
<td>40 MPa Concrete</td>
<td>2300</td>
<td>0.2</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>7850</td>
<td>0.3</td>
</tr>
<tr>
<td>Post Tensioning Steel</td>
<td>7850</td>
<td>0.3</td>
</tr>
</tbody>
</table>

A typical box girder section was selected to calculate the composite deck material properties. The longitudinal bars were counted and double-checked against the bending schedule for that specific segment. For the transverse reinforcement, an equivalent steel area per meter length was calculated using bar size and spacing. The post tensioning steel was included using the same method as the longitudinal steel. The strand area and material properties were obtained from the Freyssinet prestressing brochures (Freyssinet, 2010).

As the reinforcement in the balustrades is significantly less than the rest of the box girder section, the reinforcement thereof was disregarded. The balustrades were however included in the density calculations as it added dead weight to the structure. This was achieved by increasing the material density of the deck, but disregarding the balustrades from the effective stiffness calculations. All of the initial and composite material properties that were used for the final model are summarised in Table 4.4.

Table 4.4: Composite section material properties

<table>
<thead>
<tr>
<th>Bridge Section</th>
<th>Area $[m^2]$</th>
<th>Modulus of Elasticity [GPa]</th>
<th>Density $[kg/m^2]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel $A_s$</td>
<td>Concrete $A_c$</td>
<td>Total $A_t$</td>
<td>Initial $E_c$</td>
</tr>
<tr>
<td>Deck</td>
<td>0.084</td>
<td>6.756</td>
<td>6.840</td>
</tr>
<tr>
<td>C2</td>
<td>0.033</td>
<td>1.368</td>
<td>1.401</td>
</tr>
<tr>
<td>C3</td>
<td>0.030</td>
<td>1.371</td>
<td>1.401</td>
</tr>
<tr>
<td>C4</td>
<td>0.059</td>
<td>1.342</td>
<td>1.402</td>
</tr>
<tr>
<td>C5</td>
<td>0.043</td>
<td>1.359</td>
<td>1.401</td>
</tr>
<tr>
<td>C6</td>
<td>0.032</td>
<td>2.087</td>
<td>2.118</td>
</tr>
<tr>
<td>C7</td>
<td>0.033</td>
<td>2.086</td>
<td>2.118</td>
</tr>
<tr>
<td>C8</td>
<td>0.032</td>
<td>2.086</td>
<td>2.118</td>
</tr>
<tr>
<td>C9</td>
<td>0.036</td>
<td>1.366</td>
<td>1.401</td>
</tr>
<tr>
<td>C10</td>
<td>0.045</td>
<td>1.357</td>
<td>1.401</td>
</tr>
<tr>
<td>C11</td>
<td>0.028</td>
<td>1.373</td>
<td>1.401</td>
</tr>
<tr>
<td>C12</td>
<td>0.024</td>
<td>1.377</td>
<td>1.401</td>
</tr>
</tbody>
</table>

In addition to the composite materials, further revision of the beam sections was implemented to the deck section. Although the cross-sectional properties calculated in Section 4.1 accurately represent the area and moment of inertia of the cross-section, it does not model the nature of
the connection between the sub and superstructure. Figure 4.4 shows the different connections implemented in the two models along with an actual cross-section of the box girder for reference.

Figure 4.4: Sub to superstructure connection modelling

A standard connection between beam elements in Abaqus connects beam elements at their node points, located at the centroid of the section. In this case, the apex of the column connects directly to the centroid of the deck section, shown in Figure 4.4b. This is incorrect for two reasons. Firstly, when specifying a generalised section using only cross-sectional properties for a beam element, the centroid and shear centre are, as per the element definition, at the middle of the section. An additional offset has to be added to the section definition to define the location of the centroid. For a non-symmetric section the shear centre and the centroid are not coincident points, this similarly needs to be offset during the section definitions. The Abaqus CAE has the offset functionality programmed into the interface and can be easily adjusted. This offset is indicated in Figure 4.4c and was implemented in the final model.

The box girder section of the bridge is symmetrical about its vertical axis and thus the centroid and the shear centre only required a vertical offset. The ProSec software used to calculate the other cross-sectional properties was utilised to calculate the locations of both these points. The centroid was shifted down by 1.231 m and the shear centre was shifted down by 0.589 m.

The second error resulting from the standard element connection is the location of the connection itself. The top of the column does not connect to the deck section at its centroid but rather at the soffit of the box girder, especially when bearing connections are present. To account for this additional offset, couplings were used to connect the column apexes to the deck section. Using couplings instead of rigid links provided the additional functionality of enabling transfer of only certain degrees of freedom from one element to the other. This was required to ensure effective
4.2.2 Additions to Model

Numerous additions and refinements were made to the model to add levels of detail. Some were removed during the process of model calibration, as their effects were not significant and only added computational inefficiency to the model. Computational efficiency is very important once earthquake simulations are introduced to the model. The data sets used as input variables are very large, which could result in extremely lengthy simulations. Comparative data on the percentage change that each modification had was omitted as the each percentage value would be different for each mode, resulting in large comparative tables. As the final configuration is a combination of numerous modifications, combined comparisons would further increase these comparison tables. Each modification is presented through stating their level of significance it had on the modal results.

Modelling of Double Columns

As stated in Section 4.1, the two sets of double columns supporting the bridge were modelled as one element, with effective cross-sectional properties for the combined columns. This was done to simplify the model geometry during the development of the initial model. This simplification was refined for the final model with the two columns modelled separately. For the double columns, a rigid link was used between the two individual column apexes, where after it was coupled to the deck section. All degrees of freedom of the midway point of the rigid link were transferred to the deck element. This enables the columns to move freely along their length and then transfer their respective displacements to the deck element and vice versa. This refinement had a significant effect on the model and was retained in the final model.

Effective Piles and Footings

To determine the effect the founding conditions had on the dynamic behaviour of the model, some foundational features were added. Pad footings were modelled using shell elements and were supported by an effective elastic soil. Abaqus has an elastic foundation module that represents an elastic soil acting on a predefined surface. The module applies a soil stiffness to a selected surface of the footing. The only variable this module requires is the soil’s subgrade of modulus reaction. The module then effectively simulates the soil as a set of linear springs. The subgrade of modulus reaction was determined using borehole data accompanied by standard penetration test data available on the as-built construction drawings. Boreholes were taken at each column footing resulting in data available for all foundations.

Since all of the columns are not supported by pad footings, the effect of the pile groups was also evaluated. To accurately model a pile group is computationally expensive and is usually
simplified when considering a global model (Kerciku et al., 2008). This is due to the complex interaction between the soil and the piles, and the different soil layers that the piles pass through. This usually requires a three-dimensional solid element model of the soil and the pile group. A general simplification in the preliminary design stage of bridges is to disregard the pile groups and model the base connections as fully fixed boundary condition. This does not have a significant effect when designing the sub and super structure as this approach yields accurate foundation reactions. This would however not be sufficient for effective pile designs (Tomlinson and Boorman, 2001).

Another method widely used method to simplify pile behaviour is the point of fixity method. This method extends the columns by a certain length and uses a fixed connection at the extended base of the piles. This effectively simulates the stiffness provided by a pile or pile group. During the early design stage, when geotechnical information is usually not available, this point of fixity is assumed as a fixed factor of the column height or estimated pile depth. When more information becomes available, this factor can be refined using certain methods by incorporating the soil parameters and geometrical properties of the structure.

Modelling all the pile groups and the founding soil was found to be highly computationally inefficient, therefore the effective point of fixity method was used for this study. As an initial value, all columns founded on piles were extended one third of the pile height and fixed at their base. These values ranged between 8.5 m and 10.5 m. The refinement of these lengths was implemented using the methods described by Wilson and Hilts (1967). Wilson and Hilts (1967) incorporated the soil stiffness and cross-sectional properties of the piles. The effective lengths could then be deducted from a series of graphs. The use of this method reduced the effective lengths to between 6.5 m and 9 m. These changes did not have a significant effect on the response of the structure.

These foundation additions to the model marginally lowered the effective stiffness of the model which led to all natural frequencies being reduced. During the calibration process, it was observed that the model provided slightly better results without the foundation elements. These additions were removed to increase the computational efficiency and accuracy of the model. This negligible effect confirms the statement by Tomlinson and Boorman (2001) that it would in most cases have an insignificant effect on the behaviour of the global structure.

**Bearing Assumptions**

Accurate modelling of the bearings was also an area of concern during the model development. The true behaviour of a bearing is difficult to simulate in a global model when beam elements are used. The behaviour is therefore not entirely free, nor is it entirely restrained. The friction in the bearing has a significant effect on the dynamic behaviour of the structure. This friction could be modelled as effective springs with stopping limits as utilised by Magalhães et al. (2008). Determining the stiffness of the effective spring is however very difficult without removing the bearing and conducting actual testing on it. Magalhães et al. (2008) achieved this stiffness by incrementally increasing the stiffness of the springs until the FE model’s mode shapes matched
the physical test modes. This method was viable for their specific case as the bridge they investigated only consisted of three piers founded on soil with only unidirectional bearings at the abutments. The Stellenberg interchange has eleven columns with varying heights, varying foundation conditions and six supports connected to the superstructure using bearing connections, all with different configurations. With this approach, it would be impossible to determine the actual correct stiffness of the springs due to all the variables.

During the calibration of the model, the level of restraint for the bearing connections in certain degrees of freedom was varied, until a realistic and optimal solution was obtained. The transfer of translational and rotational DOF’s between the deck and column elements could be varied. The connection at the top of column bearings of columns C2, C3, C11 and C12 were varied between transferring all degrees of freedom, pinned (releasing all rotational degrees of freedom) and only restraining the vertical degree of freedom. Some of the bearings provide freedom in both lateral and rotational directions. The first and last variations produced unreliable results, as the first effectively removes the bearing all together while the last provides too much freedom of movement for the superstructure. The best-fit solution was to model the bearing as pinned connections for all column bearings.

The bearing configuration at the abutments consists of two side-by-side unidirectional plate bearings allowing freedom of movement in the longitudinal direction of the bridge. The modelling of these bearings was varied between fully fixed, pinned and freeing all but the vertical and transverse directions. Although some of the variations produced acceptable results, the best-fit solution was to fully fix the connection and release the beam element’s second rotational degree of freedom, effectively allowing free rotation about the transverse axis of the bridge. Selective release of specific degrees of freedom for beam elements is not implemented in Abaqus CAE interface and had to be hard-coded into the model’s input files. This selective release was implemented using the *RELEASE functionality. The *RELEASE function requires the user to specify the element number, which side of the element is applicable and then the DOF number which should be released. With reference to Figure 4.1, DOF 5 at abutment C1 and DOF 11 at abutment C13 was released with all other DOFs restrained against translation and rotation. The release of these DOF’s simulated the in-plan horizontal rotational fixity that the side-by-side bearings provide, but still allows accurate vertical displacement and vertical rotation of the superstructure.

Table 4.5 provides a summary of all the additional features considered for the model and the features that were retained in the final model.

### 4.2.3 Mesh Refinement

A sensitivity analysis was performed on the final model to determine an appropriate mesh size for the elements. Starting with a very coarse mesh, the mesh size was incrementally decreased by a factor of two until no significant differences were observed. Mesh optimisation was an important step in ensuring computational efficiency while maintaining an accurate representation of the structure. Table 4.6 shows the results of the mesh optimisation and indicates after the third
Table 4.5: Summary of additional model features

<table>
<thead>
<tr>
<th>Model feature</th>
<th>Implementation in initial model</th>
<th>Effect on dynamic behaviour</th>
<th>Retained in final model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modelling of double columns</td>
<td>Partial</td>
<td>Significant</td>
<td>Retained</td>
</tr>
<tr>
<td>Effective pile foundations</td>
<td>None</td>
<td>Insignificant</td>
<td>Omitted</td>
</tr>
<tr>
<td>Modelling of pad footings</td>
<td>None</td>
<td>Insignificant</td>
<td>Omitted</td>
</tr>
<tr>
<td>Bearing conditions</td>
<td>Partial</td>
<td>Moderate</td>
<td>Retained</td>
</tr>
<tr>
<td>Composite material properties</td>
<td>None</td>
<td>Significant</td>
<td>Retained</td>
</tr>
</tbody>
</table>

refinement step the average change in results to be less than 1%. This resulted in a mesh size of approximately 3.6 m per element for the columns and 2.7 m per element for the deck sections. These mesh sizes were used for all earthquake simulations and modal calibrations.

Table 4.6: Mesh refinement results

<table>
<thead>
<tr>
<th>Refinement Step</th>
<th>Approx. Mesh size [m]</th>
<th>Average Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Deck</td>
<td>Columns</td>
</tr>
<tr>
<td>1</td>
<td>14.4</td>
<td>10.8</td>
</tr>
<tr>
<td>2</td>
<td>7.2</td>
<td>5.4</td>
</tr>
<tr>
<td>3</td>
<td>3.6</td>
<td>2.7</td>
</tr>
<tr>
<td>4</td>
<td>1.8</td>
<td>1.35</td>
</tr>
<tr>
<td>5</td>
<td>0.9</td>
<td>0.675</td>
</tr>
</tbody>
</table>

4.3 Model Calibration

The model tuning and calibration was performed in independent steps to determine the effect each change had on the modal behaviour of the model. The model calibration was performed by comparing the physical and FE mode shapes and corresponding frequencies. The chosen comparative modes were the five most prominent modes from the physical tests. These prominent natural frequencies are indicated on the spectral decomposition diagram in Chapter 3. Similar mode shapes were selected from the FE model output by means of their frequencies. All mode shapes within a reasonable frequency range were then examined to determine whether they match the actual mode shapes from the physical tests. Figure 4.5 shows the selected mode shapes with their corresponding frequencies of the physical tests and finite element model.

The experimental mode shapes are plotted as they were recorded at third spans, without any modal smoothing and interpolation. The finite element mode shapes are plotted over all element node points resulting in the smooth mode shapes seen in the comparison. All compared mode shapes were below the 5 % error as recommended by Magalhães et al. (2008) for dependable model calibration using modal comparison. The largest error was observed at 3.3 % for the fifth mode. Table 4.7 shows a summary of the comparative data. The FE model was compared to the SSI-UPC recorded mode shapes as stabilisation checks could be performed on the recorded data.
## Chapter 4. Finite Element Modelling

### Physical Mode Shapes (SSI-UPC)

<table>
<thead>
<tr>
<th>Mode</th>
<th>2.824 Hz</th>
<th>2.784 Hz</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Mode</th>
<th>3.046 Hz</th>
<th>3.049 Hz</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Mode</th>
<th>3.451 Hz</th>
<th>3.401 Hz</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Mode</th>
<th>3.871 Hz</th>
<th>3.822 Hz</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Mode</th>
<th>10.03 Hz</th>
<th>9.703 Hz</th>
</tr>
</thead>
</table>

*Figure 4.5: Modal comparisons*
Table 4.7: Summary of modal comparison

<table>
<thead>
<tr>
<th>Mode</th>
<th>Physical Modes [Hz]</th>
<th>Num. Modes [Hz]</th>
<th>Error [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EFDD</td>
<td>SSI-UPC</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2.855</td>
<td>2.824</td>
<td>2.784</td>
</tr>
<tr>
<td>2</td>
<td>3.041</td>
<td>3.046</td>
<td>3.049</td>
</tr>
<tr>
<td>3</td>
<td>3.383</td>
<td>3.451</td>
<td>3.401</td>
</tr>
<tr>
<td>4</td>
<td>3.368</td>
<td>3.871</td>
<td>3.822</td>
</tr>
<tr>
<td>5</td>
<td>9.990</td>
<td>10.03</td>
<td>9.703</td>
</tr>
</tbody>
</table>

Figure 4.6 shows a comparison between a model representation of the bridge and the un-rendered FE beam model used for the all the earthquake simulations. The green arrows in Figure 4.6(a) indicate the principle directions that the earthquake accelerations were applied in. These directions are explained in Chapter 5.

Figure 4.6: Load combination failure examples
Chapter 5

Earthquake Simulations

The calibrated FE model was then used to simulate seismic loading on the structure. This was achieved by applying recorded ground acceleration time-histories to the base of the columns to simulate various seismic events. This chapter discusses the various aspects applicable to the earthquake simulations. These aspects include the choice of earthquake data series, data handling, applied orientations and applied magnitudes.

5.1 Earthquake Selection

A selection of appropriate earthquake time-histories was required to produce a series of realistic and reliable simulations. There is no comprehensive earthquake data available of the required magnitudes and intensities for the Cape Town region. This necessitated the use of suitable datasets from different locations. The amount of recorded seismic data available today is extensive due to the development and implementation of modern recording instruments. It is thus important to carefully consider the selection of appropriate data.

The standard procedure when preforming a time-history based seismic analysis is to ideally utilize seven recorded ground motions with a minimum of three for a specific PGA value. The selection of appropriate ground motions are based on the expected PGA, magnitude, distance and location of the epicentre, source mechanism and site soil conditions. The time-histories are then converted to spectral accelerations and then checked with a design response spectra. If unavailable this spectra should be developed for various soil conditions to evaluate the effect of potential amplification of motion due to the soil. As most of the selection criteria was unknown in this case and there is even a great discrepancy over the appropriate PGA applicable to the site, incorporating all the unknowns would result in a large number of simulations and results. To reduce the amount of simulations and results, some deviations from the typical approach were made. To cover a broad spectrum of PGA values, two motions from one seismic event were taken for each PGA value. For each PGA a maximum and a minimum response was selected to establish a probability band for the response of the structure.
This study used the recorded data from the Chi-Chi earthquake, which occurred in central Taiwan in 1999. The available comprehensive PGA and intensity ranges of the Chi-Chi earthquake governed the basis of this selection. Although this earthquake did not physically occur in the Cape Town region, the response of the structure would still highlight any structural weaknesses. The selected datasets conform to the expected seismic activity range for the Cape Town area.

The numerous stations recording the Chi-Chi earthquake were widely spaced across the region. This resulted in complete displacement, velocity and acceleration datasets for the specific earthquake. Since the stations were scattered across Taiwan, the earthquake provided seismic intensities of varying magnitudes. This eliminated the need to scale certain sets of earthquake data.

Three datasets from the Chi-Chi earthquake represented the three cases considered for this study. The acceleration time histories were individually applied to the bridge in both directions orthogonal to the vertical plane. These three acceleration cases are:

- A low magnitude earthquake with a PGA of approximately 0.05g which would have a higher probability of occurrence.
- An earthquake representing the recommendation prescribed by the TMH 7 (The current bridge and culvert design code) with a PGA of approximately 0.1g.
- An earthquake with a similar PGA of approximately 0.2g which is expected to occur in the region based on the recommendation of Kijko et al. (2003) and the Council of Geoscience.

Simulations were also conducted for a PGA of 0.15. This is omitted from the discussion as it presented similar trends only resulting in excessive result presentations. For all three cases, the recorded acceleration and displacement data was required in both the North-South and East-West directions.

Furthermore, due to the varied locations of the measuring stations, there were numerous stations recording the same magnitude, but at different intensities. This is due to their location and distance from the epicentre. These varied intensities from numerous recording stations, which record the same PGA, were utilised to produce a probability band of the structural response. The variance in intensity is due to the position and soil conditions at the different stations. Figure 5.1 shows two acceleration time-histories of a 0.1g magnitude earthquake from two different measuring stations.

The maximum PGA peaks is approximately 0.1g. It is difficult to establish which of these would result in greater displacements and greater damage to a structure. The major difference between the two is the displacements produced by the acceleration time-histories. Figure 5.2 shows the displacement time-histories recorded at the two recording stations.

The acceleration recorded at station TCU104 clearly yielded a much larger displacement of 520 mm compared to the 79 mm from station CHY065. The displacement time-histories clearly indicate that even though the maximum PGAs were similar, the displacement histories are very different. These differences in intensities result in very different structural responses. The use
Figure 5.1: Acceleration time-histories from stations TCU104 and CHY065 for 0.1g Chi-Chi Earthquake

Figure 5.2: Displacement time-histories from stations TCU104 and CHY065 for 0.1g Chi-Chi Earthquake
of variable structural responses from the various recorded data sets enabled the establishment of a probability band between maximum and minimum intensity earthquakes.

Each earthquake simulation consisted of eight sets of data. This included acceleration and displacement data for a minimum and maximum intensity earthquake for each PGA simulation in both North and West directional components. This amounted to 24 sets of recorded data for all the simulations. The Pacific Earthquake Engineering Research Centre’s (PEER) Ground Motion Database provided all the required data.

As might be noted, the consideration of vertical accelerations was omitted for this study. Vertical accelerations should only be taken into account when considering peak ground accelerations exceeding 0.6g (Button et al., 2002). This study only evaluated earthquakes up to 0.2g and therefore vertical accelerations were omitted.

5.2 Data Handling and Condensation

Each of the 24 datasets selected consisted of a very large number of data points. The selected stations recorded the data at sampling rates between 200 Hz and 250 Hz, for 90s to 150s. This equates to between 18 000 and 37 500 data points per set. To use the raw data obtained from the PEER database would result lengthy simulations. The sampling rates of the recordings however are very high, resulting in oversampled data. Oversampled data can be significantly reduced using certain filtering techniques.

Omitting is the simplest of these filtering techniques. This is simply the selection of every $n^{th}$ data entry while omitting the rest from the data. For many oversampled datasets, this method is easy to implement and effective, especially for direct use of the data, i.e. it does not require further integration or processing. This is however not the case for a seismic analysis as the input values for the numerical models are accelerations. The double integration of the acceleration in terms of time produces displacement which is shown in Equation 5.1. This is the same integration used by the Abaqus software to calculate the displacements from the acceleration input data.

\begin{equation}
    u = \int v dt = \int \int a dt \tag{5.1}
\end{equation}

where:

$u$ : Displacement  $v$ : Velocity  $a$ : Acceleration

Using data omission, the error in the double integration becomes very large, due to the omitted data that results in incorrect integration constants. The further integration of these incorrect constants results in an even larger error for the integration of the next time step. Although this error is present in any incremented data integration, skipping data points accentuates it. This continual error in integration could lead to very large discrepancies in results. An example of this integration error is shown in Figure 5.3, where there is a difference of 365 mm at the completion of the time series. Figure 5.3 clearly indicates how the integration error increases...
with time. Each increment uses an incorrect initial value from the previous step and produces a further incorrect integration constant for the next. An effective loss in information is clearly present when integrating datasets with omitted values. Although it is possible to remove the trend resulting from the time series integration, this requires an additional processing step.

![Displacement responses for condensed and omitted data](image)

Figure 5.3: Displacement responses for condensed and omitted data

A more effective method to reduce the number of data points without such a significant loss in information is to use the condensation technique. Instead of omitting data points from the set, this method replaces an \( n \) amount of data points with an average value of that range. This is done for every \( n \)th data point. This \( n \) amount of points is called the condensation factor and is the factor whereby the dataset is reduced. The condensation method simply utilises an average calculation, seen in Equation (5.2), on every \( n \) amount of data points.

\[
\bar{a}_i = \frac{a_{i-n} + \ldots + a_i + \ldots + a_{i+n}}{n}
\]  

(5.2)

Although the averaging does slightly reduce the maximum peaks of the input data, the reduction is constant throughout the series. Omission on the other hand, could possibly remove an entire peak of the acceleration data. The loss of data when using omission is therefore erratic throughout the series causing the large integration errors. The main advantage of condensation is the preservation of information between points during integration. This is clearly seen in Figure 5.3 also showing the integration error for a 0.1 PGA Chi-Chi earthquake.

The condensation factor for the condensed data in Figure 5.3 was 10, reducing the data by a factor of 10. For the omission data in Figure 5.3, every fifth data point was selected, effectively reducing the number of data points by a factor of five. Figure 5.3 shows the integrated displacement from the condensed data virtually matching the actual recorded displacements, while the omitted data resulted in a very large integration error.
All datasets obtained from the PEER database were condensed to reduce the number of input values for the simulations. This significantly increased the computational efficiency of the earthquake simulations.

### 5.3 Earthquake Orientation

The earthquakes applied to the structure can be oriented in endless arrangements and configurations. To limit the number of simulations required, a selection of certain orientations and configurations were investigated. Various sensitivity analyses were performed to establish appropriate orientations. The aim was to obtain a worst-case orientation and execute all simulations in this orientation. Generally, one motion is selected and applied in a partial combination, for example 100%N + 30%E. As the North and East time-histories were available, they were applied in full to the structure as they were recorded.

A base orientation was selected which acts as the reference orientation for the sensitivity analyses. The base North direction aligned with the transverse direction of the bridge at midpoint. Figure 5.4 shows the base configuration on the plan view of the bridge. All variations considered or other configurations used this orientation as the reference orientation.

![Figure 5.4: Base configuration for earthquakes](image)

The first step was to evaluate if any significant differences occurred when switching the North and West components of the earthquakes, effectively rotating the orientation by 90°. Although there were slight differences in the results, they were not consistent throughout all simulations. Some of the columns experienced higher forces and moments while other yielded lower values, indicating that the 90° rotation is not necessarily a better or worse case for all the columns. The orientation was then rotated by 30° and 45°. As an example, The 30° rotation is also indicated on the orientation in Figure 5.4. For both these orientations, the base orientation with components perpendicular to the bridge at midpoint indicated greater force and moment results for almost all the column bases. The base configurations resulted in the worst-case orientation. All simulations were performed with the North and East components in the base configuration.
Varying the orientations changed the directional components of the column base forces. Thus, comparing magnitude values would be a more sensible evaluation of the forces and moments obtained from the simulations. The magnitudes were calculated using Equation 5.3.

\[
F_R = \sqrt{F_x^2 + F_y^2} \quad M_R = \sqrt{M_x^2 + M_y^2}
\]

where:
- \(F_x\): Force in x-Direction
- \(M_x\): Moment about x-Axis
- \(F_y\): Force in y-Direction
- \(M_y\): Moment about y-Axis
- \(F_R\): Resultant Force
- \(M_R\): Resultant Moment

The resultant values were calculated and used for all simulations and were used in all comparative evaluations.

### 5.4 Simulation Verification

To ensure the simulations produced accurate results, a result validation was required. To validate the simulations, the base displacement values from the simulations were compared to the actual recorded displacement for each data set. This was achieved by isolating the column base displacements in the directions that the accelerations were applied. These results were superimposed on the same graph to evaluate their correlation.

Figure 5.5 shows an example of this check for the minimum intensity 0.1g earthquake simulation. Abaqus employs the same double integration as discussed in Equation 5.1 to determine the displacements. This double integration was also used to check displacements from simulations that did not correlate and to double-check the condensed data sets.
The simulations of all 24 datasets were checked in this manner. The simulations produced all the results required to evaluate the structural response of the Stellenberg interchange. Table 5.1 shows a summary of the recording stations from which the data was obtained to perform the earthquake simulations. This data is freely available from the PEER ground motion database. All maximum and minimum responses as discussed and seen in figures in Chapter 6 were obtained using the data from the recording stations shown in Table 5.1.

Table 5.1: Summary of Chi-Chi earthquake data recording stations

<table>
<thead>
<tr>
<th>PGA</th>
<th>Intensity</th>
<th>Recording Station</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>Maximum</td>
<td>CHY 060</td>
</tr>
<tr>
<td></td>
<td>Minimum</td>
<td>TTN 044</td>
</tr>
<tr>
<td>0.1</td>
<td>Maximum</td>
<td>TCU 104</td>
</tr>
<tr>
<td></td>
<td>Minimum</td>
<td>CHY 065</td>
</tr>
<tr>
<td>0.15</td>
<td>Maximum</td>
<td>TCU 109</td>
</tr>
<tr>
<td></td>
<td>Minimum</td>
<td>CHY 015</td>
</tr>
<tr>
<td>0.2</td>
<td>Maximum</td>
<td>TCU 082</td>
</tr>
<tr>
<td></td>
<td>Minimum</td>
<td>ILA 067</td>
</tr>
</tbody>
</table>
Chapter 6

Results and Discussions

This chapter presents the significant results from the earthquake simulations with detailed discussions thereof. Various failure modes of the bridge were inspected to evaluate the possibility of damage to the structural elements of the bridge. All column bases were evaluated in terms of their shear and moment capacities for various applicable earthquakes. The bearing connections were also evaluated in terms of its horizontal force capacity to determine the possibility of unseating of the superstructure. These aspects were assessed for varying earthquakes with peak ground accelerations between 0.05g and 0.2g as discussed in Chapter 5. Due to the large amount of data available from the analysis, the detailed discussions in this chapter will only be in terms of the worst-case columns for each failure criteria. The remainder of the columns are discussed as a summary, with their detailed time-history responses to the design earthquake of 0.1g available in Appendix B. The worst-case columns were chosen as a combination of the forces experienced by the columns and its capacity to resist those forces.

6.1 Column Capacity Calculations

The load bearing capacities were required to assess the structural performance of each element. The steel reinforcement data required for the calculations was available on the as built drawings and bending schedules provided by SANRAL. The calculation of each structural element’s capacity is discussed in this section.

6.1.1 Shear Capacity

The actual base shear capacities of the columns were calculated according to the current concrete design code, SANS 10100-1 (2000). The calculated horizontal force capacities of the bearing connections were supplied on the as-built drawings. A summary of the base shear capacity of each column and the horizontal force capacity of the bearings is summarised in Table 6.1.

The shear capacities of the columns are very similar even though the tensile reinforcement varies throughout all the columns. This is due to two reasons. Firstly, the shear reinforcement in the
Table 6.1: Summary of the calculated column capacities

<table>
<thead>
<tr>
<th>Column</th>
<th>Bearing Shear [kN]</th>
<th>Base Shear [kN]</th>
<th>Base Moment [kN·m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2</td>
<td>920</td>
<td>2 437</td>
<td>13 380</td>
</tr>
<tr>
<td>C3</td>
<td>860</td>
<td>2 437</td>
<td>12 840</td>
</tr>
<tr>
<td>C4</td>
<td>N/A</td>
<td>2 437</td>
<td>12 764</td>
</tr>
<tr>
<td>C5.1</td>
<td>N/A</td>
<td>2 398</td>
<td>11 312</td>
</tr>
<tr>
<td>C5.2</td>
<td>N/A</td>
<td>2 398</td>
<td>11 312</td>
</tr>
<tr>
<td>C6</td>
<td>N/A</td>
<td>2 998</td>
<td>10 778</td>
</tr>
<tr>
<td>C7</td>
<td>N/A</td>
<td>2 998</td>
<td>10 938</td>
</tr>
<tr>
<td>C8</td>
<td>N/A</td>
<td>2 998</td>
<td>10 927</td>
</tr>
<tr>
<td>C9.1</td>
<td>N/A</td>
<td>2 437</td>
<td>8 275</td>
</tr>
<tr>
<td>C9.2</td>
<td>N/A</td>
<td>2 437</td>
<td>8 275</td>
</tr>
<tr>
<td>C10</td>
<td>N/A</td>
<td>2 398</td>
<td>12 769</td>
</tr>
<tr>
<td>C11</td>
<td>760</td>
<td>2 437</td>
<td>11 161</td>
</tr>
<tr>
<td>C12</td>
<td>780</td>
<td>2 370</td>
<td>8 560</td>
</tr>
</tbody>
</table>

columns with the same cross-section is equal. Secondly, the (SANS 10100-1, [2000]) specifies a maximum limit to the tensile reinforcement’s contribution to the shear capacity of a section. This limit was reached for all columns except columns C5, C9, C10 and C12 as the tensile reinforcement in these columns was too light to reach this limit. The large cross-section of the columns combined with the shear reinforcement contributes to the majority of the columns’ shear capacity. This is clear when examining the shear capacities of the three larger columns (C6, C7, C8). The larger columns have an equal amount of shear reinforcement, with less tensile reinforcement than the other columns. The base shear capacity of these columns are however 561 kN greater than the smaller columns, thereby indicating the large influence of the cross-section.

6.1.2 Bending Moment Capacity

The bending moment capacity of each column base was calculated using the general column design module (GenCol) which is a sub-module of Prokon. This module allows the user to provide any generalized cross-section geometry and reinforcement layout, which it uses to determine interaction diagrams for the particular section. The GenCol module also uses SANS 10100-1 (2000) as reference to perform the calculations.

The bending moment capacity is calculated as a function of the axial force. The appropriate amount of tensile reinforcement required in a column is generally determined by developing an interaction diagram. The required percentage steel is obtained when using this interaction diagram in conjunction with the applied bending moment and axial force. The GenCol module produces an interaction diagram for the specified section geometry provided. The axial force is required to determine the bending capacity of a section using an interaction diagram.

The design axial forces required were not specified on the as-built drawings of the columns and
thus needed to be calculated. The only design axial forces available were supplied as pile cap forces for the C5, C6 and C7 column bases. These axial forces were used in conjunction with the dead load axial forces obtained from the FE model. Using the pile cap and FE model axial forces, a load factor was determined and factored into the dead load axial forces of all the unknown columns (C2-C4, C8-C12). This produced approximate design axial forces required to obtain the bending moment capacity from the interaction diagram. An interaction diagram was developed for each column as the tensile reinforcement is different for each column. The bending moment capacities of the column bases are shown in Table 6.1

6.2 Peak Data Selection

To produce sensible histogram plots for all the columns, the time-histories of the base shears and bending moments needed to be reduced to a single value. Using the maximum values experienced at the column bases would not be an accurate representation of the response since most of the time-histories include significant outliers. These outliers might not occur for a different earthquake of similar magnitude. The average of the forces and moments would also not be a very indicative value, as it would result in an incorrect assessment of the response compared to the actual high forces experienced by the columns. The solution therefore was to calculate an effective peak average. This was achieved using a peak-picking algorithm implemented into a Matlab code with certain set boundaries for each case. These boundaries were the minimum range between peaks and a minimum threshold value that peaks have to exceed to be selected. The boundaries required some adjustments as the magnitudes of the responses were not of the same order throughout all the time-histories for all the columns. As an example, Figure 6.1 shows the aforementioned conditions, with a plot of the peak values that were selected by the algorithm. The data presented in Figure 6.1 is the base shear time-history response of column C7 during the 0.1g Chi-Chi Earthquake.

6.3 Column Base Response

The column base responses are presented and discussed in this section. The column bases were assessed on their performance in terms of shear force and bending moment responses. Time-history responses of the worst-case columns are discussed for each case with a summary of all other columns presented in a histogram format. This was done to reduce the amount of excessive graphs presented in this document. The time-history responses of all other column bases is presented in Appendix [1] for the design earthquake loading of 0.1g. The value of 0.1g was selected since it conforms to the specified PGA in TMH 7.

The structural response is discussed separately regarding each loading condition for each evaluated PGA. A time-history response of the worst-case column for each loading condition is presented for each PGA ranging from 0.05g to 0.2g. Each time-history response is discussed, firstly in terms of the maximum intensity earthquake and then in terms of the minimum intensity earthquake for each PGA. A damage probability band between the maximum and minimum
values is established for each case with applicable conclusions drawn and discussed for each case. This is followed by a histogram that shows a response summary of all the columns and a discussion of all the columns for the specific PGA. Significant structural behaviour is then discussed in each section where applicable.

6.3.1 Base Shear Response

The base shear time-histories are presented for the current codified design earthquake PGA of 0.1g and for the predicted PGA suggested by Kijko et al. (2003) of 0.2g. Inspection of the various time-histories indicated that the shear capacities of the column bases are not a serious cause for concern for moderate intensity earthquakes. Since it is not a cause for concern, the earthquake simulation with the smallest PGA of 0.05g will not be discussed in terms of base shear. The selected earthquake loadings indicate the current state of the base shear capacity when subject to the current design requirements and worst-case expected loadings for the region.

The worst-case column was selected through inspection of the time-histories in combination with the column capacities. The worst-case column in terms of base shear was column C10 for all earthquakes considered. Column C10 has the second lowest shear capacity at 2 398 kN and experiences a greater shear force during the majority of the simulations.

0.1g

Figure 6.2 shows the time-history response of column C10 for a representative PGA of 0.1g which coincides with the codified PGA requirements.
Figure 6.2: Base shear force responses for column C10 during 0.1g Chi-Chi earthquake

Figure 6.2 indicates that for the current design earthquake magnitude of 0.1g, the column base-shear capacity is not a cause for concern. The maximum peak shear force experienced by the column base only reaches 56% of the column capacity during the maximum intensity 0.1g earthquake. These peaks, occurring at 62 s and 72 s, could even be regarded as outliers. Even when considering these possible outliers, the shear capacity is sufficient to resist the loading. Thus, for the worst-case column, during the highest intensity 0.1g earthquake, the bridge should have a low probability of experiencing damage to the column base due to shear.

Since the maximum intensity 0.1g magnitude earthquake was not problematic for the worst-case column, a discussion of the response to the minimum intensity earthquake as well as a summary of all columns for the 0.1g case is not presented. The summary histogram of the base shear response of all the columns is available in Appendix B.

**0.2g**

The worst-case shear force would naturally be experienced during the simulation with the largest PGA of 0.2g. Figure 6.3 shows the time-history responses of column C10 in terms of base shear forces during the 0.2g simulation.

The base shear force for the maximum intensity earthquake with a magnitude of 0.2g PGA exceeds, or is very close to, the shear capacity of the column base. This occurs approximately six times between 76 s and 90 s, which indicates that in the worst-case, column C10 could potentially undergo damage due to shear capacity exceedance. These values are however, potential outlier values and might not necessarily occur during a different earthquake with a similar magnitude and intensity. A more reasonable comparison is the average peak value of the response. The
average peak value of 1 806 kN obtained for the maximum intensity 0.2g PGA earthquake is 75% of column C10’s 2 388 kN capacity. Although this peak average value is less than the shear capacity, combined effects from potential bending moment capacity exceedance could lead to a reduction in shear capacity, resulting in extensive damage to the section. This progressive failure could only be determined by utilising a three-dimensional FE model using solid elements with non-linear material parameters. This type of modelling however falls outside of the scope for this study since this study was an initial investigation to determine the possibility of damage or failure.

The minimum intensity 0.2g earthquake does not indicate any cause for concern.

Figure 6.4 shows the summary of the time-history responses of all the columns in a histogram format. Figure 6.4 shows each column’s capacity together with the average peak value of the 0.2g PGA maximum and minimum intensity earthquakes. This format is followed for all histogram summary plots presented in this chapter.

It is clear from the summary histogram plot which columns experience the largest shear forces. Columns C4, C8 and C10 experience significantly greater shear forces than the other column bases. Columns C4 and C10 experience approximately the same shear forces at 1 829 kN and 1 806 kN respectively. Column C10 was chosen as the worst-case due to the lower shear capacity of the column. Columns C4 and C10 are the first monolithically cast columns adjacent to the columns supporting the superstructure via bearings. The high forces in these columns are expected, as the additional fixity would result in larger stresses. The increased level of movement provided by the bearings must be resisted by the columns with fixed apex connections. This sudden change in fixity focuses additional stresses to these columns resulting in increased stresses to the columns adjacent to the bearing supporting columns.
Column C8 experiences the third largest shear force during the simulations with a peak average force of 1,501 kN. The high forces in the column are due to the relative height and stiffness of column C8 compared to those adjacent to it. Figure 6.5 shows column C8 along with its adjacent columns.

Column C8 with a height of 26.7 m is located between C7, which is 2.9 m taller than C8 and the double pairing of the C9 columns. The double column combination provides a high stiffness to the structure. Columns C7 and C6 are the tallest columns at 29.6 m and 28.7 m respectively. These tall columns provide a higher level of flexibility to that portion of the structure. The shorter height of column C8 results in the column being stiffer relative to C7 and C6. With respect to Figure 6.5, the high flexibility on left hand side of column C8 and the high stiffness on the right hand side requires column C8 to resist some of the displacements resulting from
the increased flexibility. This theory is corroborated by the fact the C5 double column pairing experiences higher stresses than the C9 pairing, indicating that column C8 does in fact resists some of the forces.

In conclusion, there is a low possibility that base shear force would be the major cause of damage to the structure during a seismic event. In the worst-case, base shear forces might be a secondary cause of damage, depending on the level of damage caused by other forces induced at the base. This secondary action however could potentially be the final addition to cause collapse of the structure during a seismic event and should not be disregarded.

### 6.3.2 Base Moment Response

The base moment response is presented for all earthquake simulations from 0.05g to 0.2g. The 0.05g magnitude earthquake was included in the results as the base moment response exceeded the column capacities at lower PGAs. The 0.05g magnitude earthquake has a higher probability of occurrence than the design earthquake magnitude of 0.1g. It is thus important to assess the structure for an earthquake with a lower PGA than the prescribed codified PGA.

The worst-case column throughout all the simulations in terms of base moment was column C8. This is due to the combination of high base moment experienced by the column and the lower moment bearing capacity of column C8.

**0.05g**

Figure 6.6 shows the base moment responses of column C8 during the 0.05g magnitude earthquake simulation.

Figure 6.6 shows that the response does exceed the moment capacity of the section. This capacity exceedance is however not severe, only exceeding the capacity three times during a 6 s period at 78 s into the maximum intensity earthquake simulation. In addition, the maximum moment only exceeds the column’s 10.93 MN·m capacity by 5.2%. Once again, these maxima could be seen as outliers, further reducing the occurrence probability of such high forces during a 0.05g PGA earthquake. This indicates a very low probability of damage to the worst-case column for the greatest intensity 0.05g PGA earthquake.

The maximum value, from the minimum intensity earthquake only reaches 62% of the column’s capacity. Indicating there is no cause for concern for a minimum intensity 0.05g magnitude earthquake. The same conclusions for the minimum and maximum intensities are observed for the remainder of the columns. The summary plot for all the columns for the 0.05g earthquake simulation is therefore omitted from the document but is available in Appendix B.
Figure 6.6: Base moment responses for column C8 during 0.05g Chi-Chi earthquake

0.1g

Figure 6.7 shows the bending moment time-history responses at the base of column C8 for the prescribed codified magnitude PGA of 0.1g.

At the design load, the base moment at column C8 indicates some cause for concern. The maximum intensity loading shows numerous exceedances of the 10.93 MN·m capacity of the column. The period between 58 s and 76 s of the maximum intensity earthquake results in a crucial segment of the simulation. During the 18 s second period, the average peaks of the base moments is 12.81 MN·m. This is 17% greater than the column’s capacity. Additionally, this period of high intensity follows after a small number of load exceedances that could potentially weaken the section, causing further damage. Even without considering the early load exceedances occurring before 58 s, there is a high probability of damage if the bridge is subjected to the maximum intensity 0.1g PGA earthquake.

The minimum intensity simulation indicates a much lower probability of damage to the column. The response indicates only one peak reaching the capacity of the column at 51 s. Considering a combination of the maximum and minimum intensities, a probability band can be established. This band lies between the minimum and maximum intensity responses and denotes a range between the two extremes. For the 0.1g base moment case, the capacity is in the top half of that range. This indicates that there is some cause for concern and the section has a moderate probability of undergoing damage during an average intensity earthquake with a 0.1g PGA.

Figure 6.8 shows the summary of the base moment response for all the columns during the 0.1g PGA simulations.
Figure 6.7: Base moment responses for column C8 during 0.1g Chi-Chi earthquake

Figure 6.8: Summary of base moments for all columns during Chi-Chi 0.1g
As with the shear force response, the columns experiencing the largest moments are columns C4, C8 and C10. This further corroborates the effect of the relative height, flexibility and freedom of movement of the columns adjacent to column C8, as explained in Section 6.3.1.

It is very clear when examining the summary plot in Figure 6.8 that C8 is the worst-case column in terms of bending moments. It has a lower capacity compared to the other columns while experiencing similar large magnitude moments. The maximum and minimum average peak values for column C8 clearly indicate the aforementioned probability band. The capacity of column C8 is approximately midway between the maximum and minimum values. The column’s capacity is thus in the middle of the probability band signifying a moderate probability of damage.

Column C8 is the only column that experiences average peak loads that exceed its capacity. All of the other columns would have a low probability of undergoing significant damage during a 0.1g PGA seismic event.

In conclusion of the 0.1g case, column C8 has a moderate probability of undergoing damage due to base moment exceedance. The damage should however be localised to column C8. As discussed in Section 6.3.1, the shear force experienced during the 0.1g PGA earthquakes does not pose a significant threat to the structure. Thus, the only cause for concern during a prescribed design code loading of 0.1g PGA would be localised damage to column C8’s base due to bending moment capacity exceedance.

0.2g

Figure 6.9 shows the bending moment time-history responses at the base of column C8 for the 0.2g PGA earthquake.

As expected, the 0.2g earthquake yielded greater bending moment responses than the lower PGA simulations. The maximum intensity 0.2g simulation exceeds the capacity of column C8 by 62% for approximately 30 s during the latter stage of the simulation. This would cause severe damage to the column, with a high possibility of total collapse of the column. This was expected as the maximum intensity 0.1g simulation already suggested a moderate probability of damage. The important observation however is the minimum intensity behaviour.

The main deduction that should be emphasised from Figure 6.9 is the response of the minimum intensity earthquake. The minimum response exceeds column C8’s capacity approximately eight times between 51 s and 74 s. Once again, the magnitudes of these exceedances are very high and might be outliers. The important fact however is the peak average of the minimum intensity earthquake, which equates to 87.3% of the column’s capacity. This places the columns capacity in the lower part of the probability band resulting in a high probability of failure during any intensity 0.2g PGA seismic event. Additionally, certain other columns also indicate a cause for concern.
Figure 6.9: Base moment responses for column C8 during 0.2g Chi-Chi earthquake

Figure 6.10 shows the summary of the base moment response for all the columns for the 0.2g PGA simulations.

As with all of the other simulations, columns C4, C8 and C10 indicated the highest moment response. For the 0.2g case, seven of the thirteen columns exceeded their capacity during the maximum intensity simulations. Some of these columns, such as the C5 double column pairing, are very high in the probability band, resulting in a lower probability of damage. This however is only when considering the columns in isolation. When considering the global behaviour of the structure, this could be considerably different. As column C8 has a high probability of undergoing serious damage, it could lose most of its load carrying capacity during a 0.2g PGA seismic event. The columns adjacent to column C8 would have to resist the load that column C8 supports, placing them under much greater stress. As the two double column pairings are already within the failure probability band, additional load further increases their probability of damage. A large part of the transverse stiffness of the structure is provided by the double column pairings. Failure of a set of double columns during a seismic event could potentially lead to total collapse of the structure.

Another important factor to consider is load combination at the column base during a seismic event. The damage inflicted through the bending moments could reduce the columns capacity to resist shear forces. With reference to the shear forces during the 0.02g earthquake, any reduction in shear capacity could shift the columns capacity into the shear-failure probability band. This could lead to potential combined shear and bending moment failure. Figure 2.14 and Figure 2.13 is shown again as Figure 6.11 as a reminder of the potential after effects of these combined failure modes experienced by bridges in the past.

Although both cases shown in Figure 6.11 were due to much larger earthquakes than what is
Chapter 6. Results and Discussions

Figure 6.10: Summary of base moments for all columns during Chi-Chi 0.2g

(a) Shear failure in plastic zone

(b) Failure of Kobe expressway

Figure 6.11: Load combination failure examples
expected in the Cape Town region, the failure modes are the important factor. Figure 6.11a indicates shear failure in the plastic zone caused by moment exceedance, the same load combination as mentioned above. Although Figure 6.11a shows the failure at the apex of the column, similar failure could be experienced at the base of the column. Figure 6.11b shows column base failure of the Kobe expressway. Both these failure modes are possible for the Stellenberg interchange during a 0.2g PGA seismic event.

6.3.3 Bearing Response

Damage to the bearings supporting the superstructure at columns C2, C3, C11 and C12 are another aspect of the bridge that could result in serious damage. Bearing failure occurs when the horizontal force capacity of the bearing is exceeded. This could lead to unseating of the superstructure causing serious damage, or even collapse of a box girder section.

Columns C3 and C11 support the superstructure via a fixed plate bearing, only allowing in-plane rotational freedom. Thus, the root magnitude of the shear force as explained in Chapter 5 was used to assess the bearing response, which signifies the maximum horizontal force in any direction. Columns C2 and C12 support the superstructure via unidirectional plate bearings, allowing freedom of movement in the longitudinal direction of the bridge. For the unidirectional bearings, the absolute value of the shear force in the transverse direction was used. The bearing situated at the apex of column C11 was the worst-case in terms of shear response throughout all of the simulations.

Figure 6.12 shows the shear force responses of column C11 during the prescribed codified 0.1g PGA magnitude earthquake.

![Figure 6.12: Bearing shear force responses for column C11 during 0.1g Chi-Chi earthquake](image-url)
Figure 6.12 indicates that the horizontal force response for the worst-case bearing only attains 38% to 40% of the bearing’s 760 kN capacity. From Figure 6.4, the base shear summary plots, it is noted that the bases of the bearing connected columns (C2,C3,C11,C12) experience a relatively low shear force response compared to the other columns. Therefore, it was expected that the bearings would not experience large horizontal forces.

As the worst-case column does not indicate any cause for concern, the summary plot of the 0.1g simulation is omitted, but it is available in Appendix A. Figure 6.13 shows the shear response of column C11 during a 0.2g earthquake simulation.

As with the 0.1g simulation, the shear response in the worst-case bearing during the 0.2g simulation does not pose any cause for concern. The maximum intensity 0.2g simulation only attains 74% to 76% of the bearing’s capacity. This indicates that the bearing has a very low probability of failure due to the horizontal forces applied to the bearing. The other bearings follow the same trend as the bearing on column C11.

Figure 6.14 shows the summary plot of the shear forces in all of the bearings during the 0.2g simulation. Figure 6.14 leads to the same conclusion for the other bearings as for the C11 bearing. None of the bearings indicate any serious cause for concern. All of the capacities are significantly lower than the failure probability band. As the bearings only experience horizontal and vertical forces and no bending or torsional moments, load combinations are not applicable. Additionally, the bearings are manufactured to high tolerances resulting in their capacities being considerably more accurate than the concrete sections. These features further corroborate the fact that the bearings have a very low probability of failure due to the applied horizontal forces for any expected magnitude earthquake.
Figure 6.14: Summary of bearing shear force for all columns during Chi-Chi 0.2g
Chapter 7

Conclusion and Recommendations

7.1 Conclusion

This dissertation evaluated the seismic response of the Northbound N1-R300 bridge interchange. Earthquake simulations were performed by utilising a calibrated FE model. This FE model was calibrated using physical modal test results.

The results from the simulations indicate that for a current prescribed codified design earthquake with a peak ground acceleration of 0.1g, the bridge would be exposed to a moderate risk of damage at the column bases. This was concluded from the design bending moment for the worst-case column, calculated according to SANS 10100-1 (2000), being exceeded on several occasions by the maximum intensity of the 0.1g earthquake. This damage would however be localised to the worst-case column (C8) as the other columns indicated a low, to very low probability of undergoing damage. The shear capacities of the columns, also calculated with SANS 10100-1 (2000), were not exceeded even during the maximum intensity of the 0.1g earthquake simulation. Shear resistance could however be reduced by bending moment capacity exceedance, increasing the probability of damage to the column bases. The cyclic nature of the load application during earthquake accentuates the combined failure modes.

Current research suggests that an earthquake with a PGA of up to 0.2g could be expected in the region of the Stellenberg interchange. This PGA value of 0.2g correlates with the design PGA in the previous design code, SABS 0160 (1989). The bending moment capacity of the worst-case column (C8) was exceeded by up to 62% of the columns capacity during a third of the 90 s maximum intensity simulation, indicating a high probability of damage and potential failure.

A small number of exceedances were experienced by the columns during the minimum intensity 0.2g simulation. This places the columns capacity within the probable failure range between the maximum and minimum intensity 0.2g earthquake simulations. Furthermore, columns C4, C5-1, C5-2, C8, C9-1, C9-2 and C10 were also in the failure probability range. The two pairs of double columns at C5 and C9 provide a large part of the transverse stiffness of the structure.
Therefore, failure of either one of the double column pairings could lead to total collapse of the bridge. This risk could be increased if the damage to the column bases significantly reduces the shear capacity of those columns.

The shear response of the column bases did not indicate a high risk of damage even when subjected to the maximum intensity of the 0.2g earthquake. A reduction in capacity due to damage caused by the moment exceedance could however increase this probability of damage. This could lead to combined loading conditions that could potentially lead to severe damage and even collapse of the structure.

The investigation also focussed on the capacity of the bearing assemblies used throughout the structure. None of the bearings indicated serious cause for concern as the maximum intensity of the 0.2g earthquake only attained 75% of the bearings capacity. No load combination failure was applicable to the bearings as none of the bearings transferred moments between the sub and superstructure.

The full extent of the aforementioned damage conditions would however require further investigation by incorporating a three-dimensional solid element model utilizing non-linear material parameters.

In conclusion, for the worst-case conditions exposed to the various peak ground accelerations applied to the calibrated FE model, the following results were found:

0.05g
For an earthquake with a lower return period than the current suggest design loading with a PGA of 0.05g, the bridge should not experience any significant damage or failure.

0.1g
For the 0.1g loading suggested by current seismic and bridge design codes, the bridge would be exposed to a low probability of damage. This damage should however be localised to only certain sections of the bridge.

0.2g
In terms of current research on seismicity in the area, suggesting an applicable PGA of 0.2g, the bridge would be exposed to a moderate to high probability of damage.

### 7.2 Recommendation for Further Studies

Possible further investigations resulting from this dissertation are discussed in this section.

#### 7.2.1 Progressive Failure

An investigation into the progressive failure of the bridge should be considered to determine the specific level of damage the structure would experience when subjected to seismic loading. Progressive failure could be investigated for two different aspects. Firstly, in terms of a localised
damage zone with respect to the loss of resistive capacity due to certain damage conditions and cyclic loading. This can only be achieved by incorporating solid three-dimensional elements and a non-linear material model. Such a model could be used to determine extent of the damage caused by the loading and failure combinations discussed in this study.

The second aspect that could be considered is with respect to the global performance of the structure. Investigation on the remaining structure’s response when a column or other support loses its flexural rigidity due to damage or unseating should be considered. The structural robustness of the remaining structure could then be investigated to evaluate the probability of collapse of the structure.

7.2.2 Evaluation of Further Stellenberg Interchange Bridges

As the conclusion of this study found that the Northbound N1 - R300 Bridge does indicate a potential cause for concern, it would be useful to also evaluate the other bridges that form part of the Stellenberg interchange. The same procedure could be followed as used in this study. Consideration should also be given to evaluating the entire interchange taking cognisance of non-linear material properties.

7.2.3 Compare and Evaluate Retrofitting Solutions

The possible risk of damage to the bridge indicates a need for solutions to the problem. A feasibility study could be conducted to determine the best retrofitting, or other solutions to increase the column capacities. Typical column retrofitting is done by jacketing of the columns. This is generally done with reinforced concrete, steel, pre-assembled fibreglass panels or even on site carbon fibre weaving to the columns. A study could determine a suitable cost-effective solution.

7.2.4 Evaluation of N2-R300 Interchange Bridges

The Southern end of the R300 connects to the N2 with a similar interchange and was constructed with similar design features to the Stellenberg interchange. The same structural layouts in terms of the bearing supports and double columns are utilised on the main bridge. The main difference is that the columns on the N2-R300 are shorter, and thus less slender. This could increase the shear forces in the columns as the columns have less flexibility to dissipate the energy. The N2-R300 interchange could have similar problems to what is mentioned in this study. This study could again be used as a guideline and incorporate the same methodologies to investigate the problem.
7.2.5 Collaboration with SANRAL

During meetings and correspondence with SANRAL representatives, it was clear that there are certain bridges that they would like to evaluate not only for seismic loading, but also for general traffic and ambient loading. Some concerns arose over certain bridges that are simply very old and some that shows signs of damage or unhealthy concrete. It is rather difficult to assess the condition of concrete and usually leads to some form of destructive testing. Some of the older bridges that form part of the network have been modified or enlarged to facilitate more traffic than the original design intended. These bridges could duly benefit from a similar evaluation. The methods used in this study could be used to do damage assessment on other bridges in the SANRAL network.

7.2.6 Dynamic Monitoring

Numerous studies have been conducted in other countries and universities to develop long-term dynamic monitoring systems for bridges. Most of these systems use modal analyses as their basis, as utilised in this study. Damage parameters can be incorporated into the model to allow real-time assessment of the condition of the structure. Similar studies could be undertaken in South Africa to develop a monitoring network to assess bridge conditions on a real-time basis.
References


Wilson, SD and Hilts, DE (1967). “How to determine lateral load capacity of piles”. In: *Wood Preserving*.

Appendix A

Physical Testing Documents
Based on Detail 13.66.1 SARTSM Vol 2 Chapter 13
MAINTENANCE IN THE ROAD RESERVE (LANE CLOSURE, SLOW LANE)

**DATE:**
**TIME:**
**LOCATION:**
**SECTION:**
**KM:**
**WEATHER:**
**ACTIVITY:**
**NOTES/COMPLIANCE/NON-COMPLIANCE:**

**CONTRACT DETAILS:** XROSS-007-0011/1 (TOLL) AND R300-010-0021/1 (NON-TOLL) RRM ON N 1 Sections 1-2 FROM OLD OAK INTERCHANGE TO SANDHILLS, N 2 Sections 1-2 FROM SWARTKLIP INTERCHANGE TO BOTRIVER (TOLL) AND R300 FROM SWARTKLIP INTERCHANGE TO STELLENBERG INTERCHANGE (NON TOLL)

**TA4A**

**DAILY ROAD SIGN REPORT**

**CONTRACTOR:**
**DIRECTION:**
**BASED ON TABLE 13.4 PAGE 13.5.1**
**ACTIVITY:**
**NOTES/COMPLIANCE/NON-COMPLIANCE:**

**Figure A1: Traffic Accommodation Plan**

Michael Solms
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Appendix B

Columns Base Responses

Figure B1: Base Shear Force for Column C2 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Base Shear for C3 during Chi-Chi 0.1g

![Graph](image)

**Figure B2:** Base Shear Force for Column C3 During 0.1g Chi-Chi Earthquake

Base Shear for C4 during Chi-Chi 0.1g

![Graph](image)

**Figure B3:** Base Shear Force for Column C4 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Base Shear for C5-1 during Chi-Chi 0.1g

Figure B4: Base Shear Force for Column C5-1 During 0.1g Chi-Chi Earthquake

Base Shear for C5-2 during Chi-Chi 0.1g

Figure B5: Base Shear Force for Column C5-2 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Figure B6: Base Shear Force for Column C6 During 0.1g Chi-Chi Earthquake

Figure B7: Base Shear Force for Column C7 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Base Shear for C8 during Chi-Chi 0.1g

Figure B8: Base Shear Force for Column C8 During 0.1g Chi-Chi Earthquake

Base Shear for C9-1 during Chi-Chi 0.1g

Figure B9: Base Shear Force for Column C9-1 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Figure B10: Base Shear Force for Column C9-2 During 0.1g Chi-Chi Earthquake

Figure B11: Base Shear Force for Column C10 During 0.1g Chi-Chi Earthquake
Figure B12: Base Shear Force for Column C12 During 0.1g Chi-Chi Earthquake

Figure B13: Summary of Base Shear Force for All Columns during Chi-Chi 0.1g
Appendix B. Columns Base Responses

Figure B14: Summary of Bearing Shear Force for All Columns during Chi-Chi 0.1g

Figure B15: Summary of Base Moments for All Columns during Chi-Chi 0.05g
Appendix B. Columns Base Responses

Base Moment for C2 during Chi-Chi 0.1

Figure B16: Base Moment for Column C2 During 0.1g Chi-Chi Earthquake

Base Moment for C3 during Chi-Chi 0.1

Figure B17: Base Moment for Column C3 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Figure B18: Base Moment for Column C4 During 0.1g Chi-Chi Earthquake

Figure B19: Base Moment for Column C5-1 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Figure B20: Base Moment for Column C5-2 During 0.1g Chi-Chi Earthquake

Figure B21: Base Moment for Column C6 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Base Moment for C7 during Chi-Chi 0.1

![Graph](image1)

Figure B22: Base Moment for Column C7 During 0.1g Chi-Chi Earthquake

Base Moment for C9-1 during Chi-Chi 0.1

![Graph](image2)

Figure B23: Base Moment for Column C9-1 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Base Moment for C9-2 during Chi-Chi 0.1

Figure B24: Base Moment for Column C9-2 During 0.1g Chi-Chi Earthquake

Base Moment for C10 during Chi-Chi 0.1

Figure B25: Base Moment for Column C10 During 0.1g Chi-Chi Earthquake
Appendix B. Columns Base Responses

Figure B26: Base Moment for Column C11 During 0.1g Chi-Chi Earthquake

Figure B27: Base Moment for Column C12 During 0.1g Chi-Chi Earthquake