ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master’s Thesis

Tenzin Nyandak

Assessment and Rehabilitation
of the Bôco Bridge

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DECLARATION

Name: Tenzin Nyandak
Email: tenzinny@buffalo.edu

Title of the Msc Dissertation: Assessment and Rehabilitation of the Bridge

Supervisor(s): José Sena-Cruz
Year: 2015

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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University: Universidade do Minho

Date: July 13, 2015

Signature: ___________________________
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To my mother Sonam Sither la and my late father Phuntsok Sither la

for giving me this precious life.
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ABSTRACT

The Bôco Bridge is the oldest reinforced concrete bridge in use in Portugal. It is located along the road EM595-1, connecting the regions of Amares and Vieira do Minho, in the district of Braga in Northern Portugal. It was designed by the architect Sebastião Lopes and built between the year 1909 and 1910 using the Hennebique system. The bridge is 33 m long and 4.55 m wide. A major rehabilitation and strengthening work was carried out on the bridge in 1960s. Previous studies in the ambit of a dissertation included historical study, geometrical and damage surveys, physical, chemical and mechanical characterization of concrete and steel.

This dissertation is a continuation of the previous studies of the bridge. Damage survey was performed on the bridge to study its current state. It is followed by a non-destructive evaluation of steel reinforcement bars using ground-penetrating radar. The results obtained from this assessment are used to verify the reliability of the existing construction drawings. An in-situ dynamic characterization of the bridge was performed by using ambient vibration test. In order to analyze the load carrying capacity of the bridge, an in-situ load test was performed using a patented system for measurement of vertical deflection of bridge decks. The results obtained from the load test and dynamic assessment were used to calibrate and verify the numerical FEM model. Finally, a proposal was prepared for the rehabilitation of the Bôco Bridge to guarantee its safeguard in future by upgrading its load carrying capacity. The retrofitted structure was verified by performing a safety analysis on critical members using European regulation, i.e. EN 1992-1-1:2004.
RESUMO

Atualmente, a ponte Bôco é a mais antiga ponte em betão armado em uso em Portugal. Localizada na estrada municipal EM595-1, a ponte Bôco liga os municípios de Amares e Vieira do Minho, no distrito de Braga, no norte de Portugal. A ponte foi projetada pelo arquiteto Sebastião Lopes e construída entre os anos 1909 e 1910 utilizando o sistema Hennebique. A ponte tem 33 m de comprimento e 4.55 m de largura. O principal trabalho de reabilitação e reforço da ponte ocorreu em 1960. Os estudos anteriores no âmbito de outra dissertação incluíram estudo histórico, levantamento geométrico e danos, caracterização física, química e mecânica do betão e aço.

A presente dissertação constitui uma continuidade dos estudos anteriores encetados. Assim, realizou-se o levantamento exaustivo dos danos para o estado atual da ponte. Seguidamente efetuou-se a avaliação das armaduras existentes com recursos a ensaios NDT. Os resultados obtidos a partir destes ensaios foram posteriormente usados na avaliação da fiabilidade das peças desenhadas existentes. Foi realizada a caracterização dinâmica in-situ da ponte com recurso a ensaios de vibração ambiental. A fim de analisar a capacidade de carga da ponte, foi realizado um ensaio de carga in situ, utilizando-se para tal um sistema recentemente patenteado para a medição de deslocamentos verticais em tabuleiros de pontes. Os resultados obtidos no ensaio de carga quase estático e na caracterização dinâmica foram usados para calibrar e avaliar a precisão o modelo numérico desenvolvido, baseado no método dos elementos finitos. Finalmente, foi preparado uma proposta de reabilitação da ponte do Bôco de modo a preservá-la, assegurando em simultâneo a sua utilização, através da atualização de sua capacidade de carga. A verificação da segurança estrutural de membros críticos realizou-se com recurso à regulamentação europeia, EN 1992-1-1:2004.
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Chapter 1: Introduction

1.1 Historical Reinforced Concrete Structures

The reinforced concrete is one of the most widely used building materials in the world and its application is found in every type of infrastructure. The reinforced concrete presents very remarkable advantages when compared to traditional building materials such as timber, adobe, bricks and stone. The primary characteristics of concrete that makes this material so widespread are strength, availability, fireproof, constructability, water-tightness, and ease of construction. These qualities of reinforced concrete were main reason for its rapid emergence as the preferred building construction material. However, it must be reminded that reinforced concrete appeared in the construction world on in the second half of the 19th century. This makes reinforced concrete a new construction material in terms of it performance and durability. Very little was known about the durability of reinforced concrete during the 20th century, when reinforced concrete was adopted as the means to reconstruct the troubled nations after two world wars.
Therefore, durability and preservation of historical reinforced concrete structures is oftentimes an overlook field of research. The challenges inherent in the study of historical reinforced concrete structures are limited knowledge of the construction technology, materials used and theoretical background of the engineering analysis (Armande, 2013). In addition to this, at the turn of the 20th century, major builders used constantly evolving semi-empirical method of design (Armande et. al, 2012). This makes the study of historical reinforced concrete even more challenging. On the other hand, very few historical reinforced concrete structures have survived up to present day due to its inability to conform to present use and serious durability issues faced due to its inherent qualities and lack of maintenance.

The historical reinforced concrete structures represent a very significant chapter of human knowledge in regards to history of building construction. It is an extremely important heritage to mankind because of its cultural, historical, architectural, and technical significance. Therefore, preservation of such structures will benefit our future generations as well. The purpose of preserving such structures can be explained most clearly with following paragraph from philosopher R. G. Collingwood:

"What is history for? My answer is that history is 'for' human self-knowledge. Knowing yourself means knowing what you can do; and since nobody knows what he can do until he tries, the only clue to what man can do is what man has done. The value of history, then, is that it teaches us what man has done and thus what man is." (Collingwood, 1946)

The Bôco Bridge, being the oldest reinforced concrete bridge in use in Portugal makes it an extremely important heritage that must be preserved at all cost. The failure to do so will result in the same fate that the previous oldest bridge suffered. Luiz Bandeira Bridge in the district of Oliveira de Frades was submerged by the construction of hydroelectric dam (Cunha et al. 2008). This is a loss of an extremely important heritage and this loss is irreversible. Therefore, Bôco Bridge must be preserved by maintaining its continued use and following a comprehensive maintenance plan. Although a rehabilitation of the bridge is recommended to guarantee its continued use in future, the original characteristics of the bridge must be preserved to maintain its authenticity and heritage value. Another advantage of preserving Bôco Bridge is that it is a sustainable solution and will result in the least impact on the environment when compared to its replacement with a construction of a new bridge. Most importantly, its preservation will safeguard this extremely important national treasure.
1.2 Scope and Objectives of the Thesis

Currently, the Bôco Bridge is considered to be the oldest reinforced concrete bridge in use in Portugal. It is located along the road EM595-1 (GPS coordinates: 41°39′12.52″N, 8°14′55.91″W), connecting the regions of Amares and Vieira do Minho, in the district of Braga in Northern Portugal. It was designed by the architect Sebastião Lopes and built between the year 1909 and 1910 using the Hennebique system (Gallotti 1908) as shown in Figure 1.1. The bridge is 33 m long and 4.55 m wide. A major rehabilitation and strengthening work was carried out on the bridge in the year 1962. The previous studies included historical study, geometrical survey, physical and chemical characterization of the original and the newer concrete.

![Bôco Bridge](image)

**Figure 1.1 – Current state of the Bôco Bridge**

The work to be accomplished with this thesis aims to fulfil the following general objectives:

- To complete the existing assessment of the bridge by performing non-destructive evaluation of the Bôco Bridge. Ground-penetrating radar will be used to gain better understanding of the existing condition and layout of the reinforcement bars.
• To perform damage survey and damage mapping of the bridge to analyze the current state of the bridge and causes of the major damage types.
• To carry out in-situ dynamic assessment of the bridge using ambient vibration test. The frequencies, mode shapes and damping coefficients will be determined from the experimental dynamic assessment.
• To perform in-situ load test of the bridge using the new system to measure vertical displacement of the deck when the bridge is loaded with a moving truck.
• To perform structural analysis of the bridge using SAP2000 software. The numerical model will be calibrated using the results obtained from the in-situ load test.
• To propose a repair and strengthening solutions to upgrade the load carrying capacity of the bridge. The rehabilitated structural members will be verified for the safety analysis using European regulation, i.e. EN 1992-1-1:2004.

Finally, it should be stressed that at present the bridge is being classified as a national monument. The Bôco Bridge is a very important historical monument and must be preserved at all cost. It contains a wealth of knowledge about the early concrete construction technology and deterioration process of historical concrete. It is a very unique reinforced concrete structure built by one of the pioneer of the reinforced concrete that has undergone historical rehabilitation work. This combination makes this structure a very important document of historic reinforced concrete technology.

1.3 Outline of the Thesis

The thesis is divided into eight chapters. The thesis starts with the first chapter on introduction, which presents the importance of studying and preserving historical reinforced concrete bridges and a brief description of the Bôco Bridge.

The second chapter describes the previous studies that included historical study, geometrical survey, physical and chemical characterization of concrete and steel used in the Bôco Bridge.

To study the current state of the damage of the bridge, a brief damage survey was presented in chapter three. It is complimented by a damage mapping.
To obtain better understanding of the reinforcement steel used for the original construction and rehabilitation project, non-destructive evaluation of steel reinforcement bars using ground-penetrating radar. The results obtained from this assessment were used to verify the accuracy of the reinforcement layout presented in the existing construction drawings. The GPR study was complemented with a brief analysis of cracks using microscopic images. The results obtained from the GPR scans and crack interpretation are presented in chapter four.

The fifth chapter describes in-situ dynamic assessment of the bridge that was performed by using ambient vibration test. The natural mode shapes, corresponding frequencies, and damping coefficients were obtained from the in-situ dynamic assessment.

Furthermore, in order to analyze the load carrying capacity of the bridge and the vertical deflection of the bridge deck, an in-situ load test was performed using a patented system for measurement of vertical deflection of bridge decks. The description of the vertical deflection measuring system and the results are provided in chapter six.

The results obtained from the load test were used to calibrate and verify the numerical model. The chapter seven describes the numerical model and the calibration of the model. The main objective of the numerical model was to find the level of stress in the structural members and to find the critical members.

Chapter eight proposes a practical and well-reasoned repair and strengthening solutions to guarantee the safeguarding of this extremely important national heritage. A step-by-step guide of the entire process of rehabilitation is elaborated as well. The retrofitted structure will be verified by performing a safety analysis on critical members.

The final chapter provides the main conclusion of the thesis, followed by a brief description of the future possibilities for the continuation of the future research of the Bôco Bridge.
Chapter 2: Preliminary Studies of the Bôco Bridge

Before the structural analysis and dynamic characterization of the bridge was performed, detailed preliminary studies were carried out to obtain better understanding of the geometrical layout, material characterization and mechanical properties of the Bôco Bridge. The historical studies of the bridge showed the layout of the steel at the time of original construction as well as the rehabilitation project of the year 1962. It also showed how the structure was retrofitted by means of cross-section enlargement. The geometrical layout of the bridge is rather simple. The complexities in this structure lied in the fact that the bridge was rehabilitated in the year 1962 using different concrete and reinforcement layout. It was found that the concrete used in the rehabilitation project was of inferior quality when compared to the original concrete. This information was crucial in later stages of the analysis of the bridge because the low-quality concrete used in the rehabilitation project gave rise to numerous durability issues that affects the bridge today. The mechanical test of the steel was not performed on this bridge. The results obtained from the test carried out on the steel specimen from the nearby bridge built in same period by the same contractor using the patented Hennibique system was used.
2.1 Historical Surveys

The Bôco Bridge was built between the years 1909 and 1910 by the Moreira de Sá & Malavez Company, who was the official dealer in Portugal of the patented Hennebique system. The bridge was designed by architect Sebastião Lopes. The structural design (including reinforcement design) was carried out by bureau d'études Hennebique in Paris and Moreira de Sá & Malavez in Lisbon. After the Luiz Bandeira Bridge in the district of Oliveira de Frades was submerged by the construction of hydroelectric dam (Cunha et al. 2008), the Boco Bridge became the oldest reinforced concrete bridge in use in Portugal today. This makes the Bôco Bridge an extremely important national heritage that needs to be preserved at all cost. Figure 2.1 shows the bridge shortly after its completion in the year 1910.

![Bôco Bridge in the year 1910](image)

Figure 2.1 – Bôco Bridge in the year 1910 (Le Béton Armé, 1910)

After the construction, the bridge was subjected to full scale in situ loading test. The test included uniformly distributed load using sand bags, deflection measurement under the live load of horse-drawn carriages, and vibration test using group of 50 men (Sena Cruz et al. 2013). On June 20th 1950, the bridge was the target of a major survey that involved several engineers from different institutions. They concluded that the bridge was in poor condition and required interventions. The main damages found on the bridge were spalling of concrete and corrosion of the reinforcement bars in the main arches and the deck of the bridge. This
survey resulted in authorities issuing a limit of single traffic with the maximum load of each vehicle set to 5 tons.

From the year 1961 to 1962, a rehabilitation project was undertaken by the Ministry of Public Works and carried out by a Porto based company Alberto Sousa Beetle & Gautier. The project was completed on March 16th, 1962 with a total cost of Escudos 1099. The strengthening and rehabilitation works were mainly carried out by cross-section enlargement of the structural members such as arches, columns, longitudinal girders and transversal girders. Additional reinforcement bars were added along with stirrups to the enlarged sections. The main aim was to retain the original architectural geometry as much as possible. The concrete used for the rehabilitation work were of inferior quality when compared to the original concrete used during the initial construction. This low quality concrete gave rise to the majority of the damages found in the current state of the bridge. This will be discussed more in detail in the following sections.

2.2 Geometrical Description

The Bôco Bridge is 33 m long in span and has a width of 4.55 m. The deck of the bridge, is with variable thickness, is supported on two longitudinal girders, one intermediate girder and transversal girders. The deck is composed of central carriageway of 3.05 m and two pedestrian sidewalks of 0.75 m on two sides. On the other hand, the longitudinal girders are supported on two arches using regularly spaced columns. The columns are rested on two main lateral longitudinal girders. The abutments of the bridge are made of masonry foundation made of granite stones, which are supported on hard rock. The geometrical survey of the bridge was carried out by J. C. Araújo (2011) and the results are shown in Figure 2.2. The geometrical survey was performed using laser distance meter, tape measure, camera and ruler.

The rehabilitation work carried out by the Ministry of Public Works included a geometrical survey of the bridge, which includes the original cross-sections and the enlarged cross-sections resulted from the rehabilitation works. The drawings are presented in the Figures 2.3 and 2.4. It must be noted that the construction drawings of the rehabilitated bridge is not accurate and cannot be used for future analysis without further investigation. This was discovered when non-destructive results obtained from using Ground Penetrating Radar (GPR) was used to study the layout of the reinforcement bars.
Figure 2.2 – The current geometry of the bridge (Araújo, 2011)
Figure 2.3 – Historical drawings from the Ministry of Public Works showing the original (top or left) and the rehabilitated (bottom or right) state of the reinforcements: (a) plan view of the deck of the bridge; (b) sections v-v’ and t-t’ showing the arches, columns and deck. (Ministry of Public Works, 1962).
Figure 2.4 – Historical drawings from the Ministry of Public Works showing the original (top or left) and the rehabilitated (bottom or right) state of the reinforcements: (a) section e-f showing transversal girders; (b) sections a-b and c-d showing the arches, columns and longitudinal girder (Ministry of Public Works, 1962).
The detailed cross-sections of the structural members before and after the rehabilitation are shown in Figure 2.5. Here, it must be noted that the concrete cover used in the rehabilitated cross-section is very inadequate. This resulted in severe consequences.

Moreover, the information provided here are based on the data obtained from the inspection report of the year 1950, rehabilitation project of 1960s, and initial geometrical survey of the bridge. Therefore, to verify the results pertaining to the geometry of the reinforcement bars, non-destructive tests were carried out.

Figure 2.5 – Cross-sections of current and original structural members: (a) arches at the mid-span; (b) arches at the abutments; (c) columns; (d) intermediate longitudinal girders; (e) lateral longitudinal girders; (f) transversal beam. (Araújo, 2011)
2.3 Material Characterization

2.3.1 Concrete

To analyze the concrete mix design, chemical composition, and physical properties of the concrete and steel, 12 cylindrical core samples and steel samples were taken from the bridge. Figure 2.6 shows the locations of 12 cylindrical concrete cores. The mix design and chemical composition for the concrete was analyzed only for the original concrete. It was found that the average binder to aggregate ratio was 1:6.3 in weight. Scanning electron microscopy (SEM) was used to find the chemical composition of the concrete and the result for the sample 1 is shown in the Figure 2.7. The binder was Portland cement and contains approximately 60% CaO, 36% SiO₂, and 3% Al₂O₃. The aggregate used was approximately 84% SiO₂, 8% Al₂O₃, and 5% K₂O. The granulometric grading of the sand showed 83% of insoluble fraction, with 80% of sand particle diameter greater than 1.4 mm.

Figure 2.6 – Locations of 12 cylindrical cores taken from the bridge: (a) plan; (b) downstream elevation
To analyze the physical and mechanical properties of the concrete, carbonation depth was determined in accordance with the Portuguese national standard LNEC E391:1993 and the modulus of elasticity in compression was determined using LNEC E397:1993. The analysis made a striking discovery, which sheds light on the main cause of the severe damages found in the current state of the bridge. The new concrete used during the rehabilitation work carried out in the year 1962 has a significantly lower value of density and higher porosity when compared to the original concrete. The average porosity index for the original concrete was 3.2%, whereas the new concrete has an average porosity index of 9.8%. On the other hand, the average specific mass of the original concrete was found to be 2418 kg/m$^3$, whereas the new concrete has an average specific mass of 2144 kg/m$^3$. This is clearly visible in the Figure 2.8 (a) and (b), showing the original and the new concrete respectively. Upon visual inspection of the bridge, the poor quality of the new concrete is very apparent and can be seen very clearly where the concrete spalling is observed.
It is very apparent that the porosity and the density have high correlation. This shows that the concrete used in the rehabilitation project of 1962 was of extremely poor quality and workmanship.

![Figure 2.9 – Carbonation front at the intersection of the old and the new concrete](image)

Furthermore, the carbonation depth of different structural members was analyzed for the old as well as the new concrete. It was found that the new concrete was completely carbonized. In the old concrete, there was no carbonation observed. This is shown in the Figure 2.9. This observation can be explained by two factors: (i) poor quality of the new concrete and (ii) the removal of outer layer of the old concrete during the rehabilitation work in the year 1962. The carbonation of the new concrete is the main cause of the corrosion of the reinforcement bars in the new concrete.

The mechanical test were performed to obtain the modulus of elasticity and the compressive strength of the old and new concrete using standards LNEC E397:1993 and NP EN 12390-3:2003 respectively. Table 2.1 shows the summarized results, showing the modulus of elasticity ($E_c$) and the in-situ compressive strength of normalized cylindrical cores ($f_{is,cyl}$) of 30 cm height and 15 cm diameter. The important conclusion that can be made from the mechanical test is very high compressive strength of the old concrete, with an average normalized compressive strength of 52.1 MPa. In comparison, the new concrete has significantly lower average normalized compressive strength of 21.1 MPa. The normalized compressive strength takes into account the shape factor correction, which depends on the height to width ratio. The average value for the modulus of elasticity of the old concrete was 41.3 MPa.
The old concrete has very high strength, which can be attributed to the granite gravel of large diameter. According to the EN 206-1:2007, the compressive strength class of C35/45 was obtained for the old concrete.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Structural Element</th>
<th>Type of concrete</th>
<th>$E_c$ (MPa)</th>
<th>$f_{u,cyl}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Arch</td>
<td>New</td>
<td>-</td>
<td>14.9</td>
</tr>
<tr>
<td>C10</td>
<td>Deck</td>
<td>-</td>
<td>-</td>
<td>27.2</td>
</tr>
<tr>
<td>C3</td>
<td>Pillar</td>
<td>Old</td>
<td>54.08</td>
<td>58.8</td>
</tr>
<tr>
<td>C4</td>
<td></td>
<td></td>
<td>45.73</td>
<td>48.0</td>
</tr>
<tr>
<td>C5</td>
<td></td>
<td></td>
<td>-</td>
<td>59.2</td>
</tr>
<tr>
<td>C6.2</td>
<td>Arch</td>
<td></td>
<td>43.15</td>
<td>58.0</td>
</tr>
<tr>
<td>C9</td>
<td>Deck</td>
<td></td>
<td>36.61</td>
<td>29.9</td>
</tr>
<tr>
<td>C11</td>
<td></td>
<td></td>
<td>-</td>
<td>64.8</td>
</tr>
<tr>
<td>C12.2</td>
<td></td>
<td></td>
<td>26.86</td>
<td>46.1</td>
</tr>
</tbody>
</table>

2.3.2 Steel

Chemical test was performed on the sample of steel taken from the bridge using X-ray fluorescence spectrometer and carbon element test. The test showed mild steel with low level of carbon. The steel was then polished and analyzed in SEM as shown in Figure 2.10. The microstructure of the steel sample showed manganese sulphide inclusions, which is typical of mild steel. Table 2.2 presents the chemical composition of the steel sample. To study the layout and the geometry of the steel bars, non-destructive test was performed using GPR.

The mechanical parameters of steel used in the Bôco Bridge are not available. Therefore, results from a nearby bridge in Terras do Bouro village were used in the safety analysis and numerical simulation. This bridge was built in the year 1909 by the same contractor using the patented Hennebique system. Therefore, it can be assumed that these two bridges used same reinforcement bars with similar mechanical characteristics.
Table 2.2 – Chemical composition of the steel

<table>
<thead>
<tr>
<th>Element</th>
<th>Composition (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fe</td>
<td>43.5</td>
</tr>
<tr>
<td>Mn</td>
<td>29</td>
</tr>
<tr>
<td>Pb</td>
<td>9.5</td>
</tr>
<tr>
<td>P</td>
<td>6.7</td>
</tr>
<tr>
<td>Si</td>
<td>4.6</td>
</tr>
<tr>
<td>S</td>
<td>4.3</td>
</tr>
<tr>
<td>C</td>
<td>2.4</td>
</tr>
</tbody>
</table>

Tensile test was performed on four samples of reinforcement bars using DARTEC universal tensile testing machine at the University of Minho by M. Kord (2013). The stress-strain relationship of four samples tested is shown in the Figure 2.11. The average yield strength of four samples was found to be 361.79 MPa with a coefficient of variation (CoV) of 11.68%. The ultimate tensile strength of four specimens was 486.69 MPa, 415.53 MPa, 413.53 MPa,
and 429.16 MPa respectively. The corresponding average ultimate tensile strength of the four test specimens was 436.23 MPa with a coefficient of variation (CoV) of 7.9%.

Figure 2.11 – Stress-strain curve and the yield strength of the four test specimens. (M. Kord, 2013)
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Chapter 3: Damage Survey and Mapping

The damage survey and mapping of the Bôco Bridge is extremely crucial for the safety analysis and the proposed strengthening of the bridge. The damage assessment of the bridge provides very important information about the causes of the damages and its severity. The damage types found on the structure are mainly spalling of concrete, corrosion of steel bars, and biological growth. The primary reason for the damages found in the current state of the bridge is the poor quality of concrete and inadequate concrete cover used in the rehabilitation work carried in the year 1962. To assess the damages, visual inspection was carried out on the bridge. Although there were several different types of damages that were found in the Bôco Bridge, only the major damage types were discussed here. The results obtained from the visual and photographic inspections and corresponding damage mappings are presented here. It must be noted that for the strengthening of the Bôco Bridge, which will be discussed later in the thesis, more detailed analysis is needed. This is due to the fact that the strengthening process will involve removing the corroded steel bars and repairing it by either replacement or the adding supplementary layer.
3.1 Damage Identification

The damage identification of the bridge can be classified into two main types: spalling of concrete (mainly caused by corrosion of steel bars) and vegetation growth. There are minor damages that are not covered in these two categories, which are not significant in terms of size and severity. There is a minor salt attack on the first column at the upstream arch at the side of Vieira do Minho. Apart from this, there are some cracks on the structure. The most severe crack was found on the abutment on the side of Vieira do Minho. The vertical crack runs from the top of the abutment to the arch. This crack lies in the mortar joints of the abutment wall made of masonry. The cause of this crack may be attributed to differential soil settlement. A further study must be carried out to analyze the possible cause of this crack and to know if the crack is still active or not. In addition to this, there were two cracks on the bottom surface of each arch at the abutment. The pressure exerted by the corrosion of reinforcement steel most likely caused these cracks.

3.1.1 Spalling of Concrete and Corrosion of Steel Bars

The spalling of concrete is caused by the corrosion of the steel bars. The synergies of the two following aspects can justify the observed damage: (i) poor quality and high porosity of the new concrete and inadequate concrete cover low used during the rehabilitation work done in the year 1962; (ii) high moisture content of the area due to presence of river, coupled with the long and heavy rainy season. From the material characterization of the concrete, it was found that the new concrete is almost entirely carbonated. The original concrete does not contain any carbonation. Therefore, it is apparent here that in order to prevent further damage to the structure, the outer layer of new concrete must be removed and replaced with a superior concrete.

Figures 3.1 and 3.2 shows the photographic survey of the concrete spalling and steel corrosion. The rehabilitation work carried out in the year 1962 used a very low grade concrete, with a very high porosity, low density and low compressive strength. In addition to this, the workmanship of the construction is very low quality. This can be seen in the placement of stirrups and concrete cover used for the steel bars. The concrete cover used in the rehabilitation project carried out in the year 1962 is extremely inadequate. This can be seen in the Figure 3.1 (a) and (e), where the reinforcement steels were laid extremely close...
to the surface of the arch. Due to the poor workmanship, the concrete cover is not regular throughout the structure.

Figure 3.1 – The spalling and corrosion near the abutment on the side of Vieira do Amares: (a) arch at the abutment; (b) bracing beams between the arches; (c) top layer of the upstream arch; (d) inner surface of the downstream arch; (e) close up view of the first bracing beam.

The stirrups were not placed according to the layout specified in the construction drawings. Some of the stirrups were placed so close to the surface with concrete cover of less than 1 cm. This is one of the main causes of the corrosion of the steel bars, which consequently led to the spalling of concrete due to expansion of the bars. This is clearly visible in Figure 3.2 (a) and (b). In some areas, the longitudinal bars were corroded as seen in Figure 3.1 (a), (e), and Figure 3.2 (c), where one can see the lack of adequate concrete cover used for the longitudinal reinforcement bars as well. This is extremely risky because the corroded steel bars can greatly affect the load bearing capacity of the structure.
Figure 3.2 – The spalling and corrosion of steel near the abutment on the side of Vieira do Minho: (a) upstream arch at the abutment; (b) close up view; (c) bottom surface of the upstream arch; (d) columns on the upstream arch; (e) downstream arch at the abutment; (f) intersection of the first bracing beam and upstream arch.
3.1.2 Vegetation and Biological Growth

The vegetation and biological growth is very common in the vicinity close to the abutment on the side of Vieira do Minho. The biological film can be found on the bottom surface of the arches and on the bracing beams. The higher plants found on the abutment poses a very pressing threat to the integrity of the masonry wall of the abutment. The reason is the vegetation growth acts as a source of accumulation of moisture, which in turn degrades the structural integrity of the wall as well as the core. The presence of higher plants can be seen in Figure 3.3 (a) and (d).

![Figure 3.3 - Growth of biological film and higher plant](image)

(a)  (b)  (c)  (d)

Figure 3.3 – Growth of biological film and higher plant: (a) abutment and the bracing beam; (b) bottom surface of the arch; (c) faulty drainage system; (d) higher plant at the abutment.

The layer of biological film found on the arches and bracing beams are caused by the presence of moisture, which is resulted from the improper design of the drainage system.
This is clearly illustrated in the Figure 3.3 (c). The drainage pipe designed on the deck of the bridge was placed in such a way that the water flows on the girder beam. The water falling from the drainage splashes on the arch and causes more damages. The bridge is located in a very rainy region, which explains the severity of the damages that resulted from the faulty drainage system. The presence of the biological film accumulates moisture, which causes further growth of biofilm. Eventually, this will lead to more corrosion and spalling of concrete. This damage can accumulate in a long run and cause serious damage to the structure as well. The faulty drainage system can be fixed with very little effort and cost. The drainage pipe must be changed or elongated in such a way that the rain water is disposed without falling on the structural members.

3.2 Damage Mapping

The detailed damage analysis was performed on the current state of the bridge. As expected, significant damages of different types were found. Apart from the damages that were discussed in the previous section, several other damage types were shown here. It includes damages that were incurred during previous interventions to analyze the mechanical properties of the concrete by extracting cylindrical cores. Other damage type includes black stain that resulted from the rusting of metallic railing on the deck. This is found throughout the side of the deck as shown in Figure 3.4. Salt efflorescence is found in areas below the faulty drainage system.

Apart from the two main types of damages found on the bridge, there are several minor damages as mentioned earlier. Those damages are not discussed in detailed because those do not pose serious threat to the structure when compared to the concrete spalling, corrosion of reinforcement bars, and biological growth. Nonetheless, those damages are shown in the damage mapping hereafter. Although these minor damages are not as damaging to the structure as the corrosion of steel reinforcement bars or the high moisture level, these can be easily removed by implementing minor changes to the drainage system and applying paint on the railings. Figure 3.4 and 3.5 shows the detailed damage mapping of the bridge with the index showing different damage types.

The iron railing on the side of the
Assessment and Rehabilitation of the Bôco Bridge

Figure 3.4 – Damage mapping of the deck, the outer surface of the two arches and columns supporting the deck of the Bôco Bridge
Figure 3.5 – Damage mapping of the abutments, the arches and the bracing beams of the Bôco Bridge
Chapter 4: In-situ Non-destructive Tests

The information about the steel reinforcement bars and stirrups discussed in the previous chapters were based on the construction drawings obtained from the historical sources. Therefore, it was very crucial to verify the cross section of all the structural members using non-destructive tests. Ground-Penetrating Radar (GPR) was used to analyze the cross-sections of the structural members. It must be pointed out that due to limited resources available due to lack of funding and inadequate safety equipment, very limited amount of radargrams were obtained. Moreover, the radargrams were obtained at only two locations: bridge deck and near the abutment. The radargrams obtained are presented below with the necessary discussions about the comparison made between the data obtained from construction drawings and present NDT test. Prof. Francisco M. Fernandes of the Universidade Lusíada and Universidade do Minho carried out the original work, which formed the basis for this study. The initial observations made by Prof. Francisco Fernandes were used here to make comparison with the information obtained from the original Hennebique construction, the rehabilitation work carried out in the year 1962 and damage survey of the present state of the Bôco Bridge. In addition to the GPR studies, microscopic crack measurements were done on the two cracks found on the arch at the abutment as each side.
4.1 Ground-Penetrating Radar (GPR) Studies

4.1.1 Deck of the Bridge

The reading was made transversely to the axis of the bridge to analyze the deck of the bridge. The location of the radargram is shown in Figure 4.1. The radargram illustrated in Figure 4.2 shows the different depths of sidewalk and the main deck. The sidewalk slab is relatively thinner compared to the central deck. It can also be deduced that the sidewalk slab does not contain any steel reinforcement. The radargram also shows one interesting observation, which is the complete lack of reinforcement near the surface of the deck. When compared to the construction drawing obtained from the previous rehabilitation project carried out in 1960s, as shown in the Figure 2.3 (b), it can be seen that the sidewalk slab do not contain longitudinal reinforcement, but it contains plate stirrups throughout the deck. Furthermore, the rehabilitated deck contains 16 reinforcement bars and numerous stirrups. The possible reason for these discrepancies is the corrosion of the reinforcement bars and stirrups. When the steel bars are corroded, the GPR scans do not show noise instead of clear parabolas. The outer surface of the deck is made of cobblestones, which contains enormous amount of void space between the stone blocks. Considering the fact that the bridge is located in very rainy region, corrosion is very likely and that might have caused separation of the layers due to expansion.

![Radargram reading performed on the bridge deck](image1)

**Figure 4.1 – Radargram reading performed on the bridge deck**

![Radargram 487 having a cross-sectional view of the bridge](image2)

**Figure 4.2 – Radargram 487 having a cross-sectional view of the bridge**
Another important fact is the curvature on the opposite surface of the deck, which in reality is horizontal. Such a signal would indicate that the filling material contains characteristics that decreased the speed of propagation of the emitted wave. This normally happens when the moisture content in the material is high. Therefore, it is hypothesized that the central portion of the deck filling may have accumulated some moisture, which is very detrimental to the durability of the concrete deck.

4.1.2 Transversal Bracing Beams Between Arches

The cross-section of the transversal bracing beam obtained from the construction drawing shows no section enlargement as opposed to other structural members (see Figure 2.4 (c)). This is confirmed by the radargram readings performed on the beam. Three measurements were made: two longitudinal and one transversal as shown in the Figure 4.3. Here, filtering was applied to the radargram to remove the horizontal line.

The radargram 479 illustrated in Figure 4.4 confirms the horizontal thickness of the beam, which is 20 cm as obtained from the construction drawings. In addition to that, it also shows the existence of two layers of stirrups, which is consistent with the construction drawings.
Furthermore, the distances between the stirrups are approximately 35 cm. The separation between the two layers of stirrups is around 5 cm. The parabolas in the radargrams are wide, confirming the flat plate used for stirrups, which is widely used in Hennebique constructions.

The radargram performed on the longitudinal profile of the beam is shown in Figure 4.5. The radargram is performed on the top of the beam as shown in the Figure 4.3. Therefore, it complies with the cross-section obtained from the construction drawings that shows no stirrups on the top and bottom area of the beam. The double layer stirrups are only found on the sides of the beam. This is verified by the radargram 480 because no signal was observed. It is likely that the U-shaped plate stirrup was used here, which is very typical of Hennebique construction. The depth of the beam was found to be 30 cm, which is consistent with the cross-section obtained from the construction drawings.

There is a sharp signal variation close to the right end of the radargram at the depth of 30 cm (Figure 4.5). This can be attributed to the possible delamination of the concrete cover. On further visual inspection of the beam, such concrete delamination is very common in this bridge. This is caused by the corrosion of the reinforcement bars and subsequent expansion of the corroded steel bars.

To analyze the longitudinal reinforcement bars inside the beam, radargram 481 was performed. Due to very short length of the reading, it was not possible to perform radargram across the full depth of the beam. Therefore, only top 12 cm can be seen in the radargram 481 in Figure 4.6. Nevertheless, it shows the top longitudinal reinforcement bar very clearly. The concrete cover shown here is much thicker than the one obtained from the construction drawings.

![Figure 4.5 - Radargram 480 before and after applying the horizontal filter](image-url)
4.1.3 Column

The cross-section of the column obtained from the construction drawing is shown in Figure 2.4 (c). It can be seen that the rehabilitation project of the 1960s enlarged the section and added four extra reinforcement bars. Four radargrams were obtained on the first column on the side of Vieira do Minho, as shown in Figure 4.7. The results obtained from the longitudinal profile are shown in the Figure 4.8. It can be seen that the rehabilitated stirrups are spaced between 20 to 25 cm. The original stirrups, which are located 5 cm deeper, are spaced much further at 40 cm. One interesting observation that can be made here is that radargram 483 shows one original stirrup (left) is placed much closer to the surface. This means that at the time of rehabilitation, the original surface was not uniform.

Figure 4.6 – Radargram 481, the side of the cross beam

Figure 4.7 – Readings made on the column
The radargram 485 in Figure 4.9 (a) is made across the column horizontally. As mentioned earlier, it was only possible to obtain radargram on one part of the column. The original reinforcement bar is seen very clearly in the center of the radargram. The later addition of the longitudinal bar can be seen on the right end of the radargram. The distance between the original longitudinal bar and the later addition is around 20 – 30 cm. This is consistent with the information obtained from the historical construction drawings.

Figure 4.9 – Comparison of radargram results and historical construction drawings: (a) Radargram 485; (b) cross-section of the column according to restoration project of year 1962.
4.1.4 Arches

To verify the reinforcement bars in the arch, radargram was performed on one arch. Due to limitations in safety equipment and accessibility, only the section of arch near the abutment was analyzed. The locations of the readings were shown in the Figure 4.10.

![Figure 4.10 - Readings on the extrados and the vertical surface of the arch on downstream](image)

The results obtained from the readings taken on the extrados of the arch and very close to the abutment are illustrated in Figure 4.11. To overcome the difficulty faced in previous sections with regard to short span of readings, two radargrams were obtained in opposite direction. Here, it can be observed that near the two ends of the section, the original longitudinal reinforcement bar can be seen with clarity.

![Figure 4.11 - Radargrams 475 and 476, obtained on the extrados near the abutment](image)

Figure 4.12 shows radargrams 477 and 486, which were obtained before the first column and after the first column respectively. This was done to gain clearer understanding of the spacing between the bars. The readings taken close to the abutment did not provide clear
signal due to interference from the signals coming from the abutment. Radargram 486 clearly shows two longitudinal bars with the spacing of 25 cm and depth of about 4 cm. This result is inconsistent with the reinforcement bar layout shown in the historical construction drawings.

Figure 4.12 – Radargram 477 and 486 performed before and after the first pillar respectively

Figure 4.13 shows the radargram obtained in the longitudinal axis. It shows stirrups placed very close to the surface and at spacing of 25 cm. The numerous incoherent signals observed below these stirrups may be attributed to original stirrups or possible concrete spalling.

Figure 4.13 – Radargram 478 in the longitudinal axis

Figure 4.14 shows the cross-section of the arch according to the historical construction drawings. As mentioned earlier, the results obtained from the GPR studies shows very contradictory results when compared to the historical construction drawings.
The most apparent discrepancy between the two are spacing between the longitudinal bars as observed in radargram 486 in Figure 4.12 and in the construction drawings shown in Figure 4.14. In radargram, the spacing was found to be 25 cm but in the construction drawings, the spacing was around 10 cm.

Figure 4.15 and Figure 4.16 shows radargrams taken on the vertical surface of the arch. Radargrams shown in Figure 4.15 show readings taken radially on the vertical surface of the arch. One interesting observation that can be made here is the fact that three main rehabilitated longitudinal reinforcement bars are not spaced equally as shown in the construction drawings in Figure 4.16. The spacing between the topmost and middle bar was about 40 cm and the spacing between the bottom bar and middle bar was about 25 cm. This may be due to difficulty in putting the longitudinal bars in correct place because of the curved profile of the arch. Another important observation that can be made here is that the longitudinal bars are placed extremely close to the surface. This can explain the existence of severe damage in this vicinity in the form of rebar corrosion and concrete spalling.

Radargrams 467 and 472 show many incoherent signals that may have resulted from the existence of intricate reinforcement system at the abutment. When the reading was performed farther away from the abutment, radargram 468 shows much clearer radargram with no signal noise.
Figure 4.15 - Radargrams 467, 468 and 472 made radially on the vertical surface of the arch.

Figure 4.16 – The construction drawing showing equal spacing between the longitudinal bars (Ministry of Public Works, 1962).

Figure 4.17 shows radargrams performed along the direction of curvature on the vertical surface of the arch. The rehabilitated stirrups can be seen very clearly. The spacing between the stirrups are not uniform and varies between 20 – 35 cm. The depths of the rehabilitated stirrups are very shallow as seen in other areas. The original stirrups can be seen very clearly in radargram 470. The spacing between the original stirrups is about 20 cm and their depth varies between 12 – 15 cm.
Another observation that can be made here is that the spacing between the stirrups increases with the increase in height of the reading. This shows that the stirrups were placed perpendicular to the curvature of the arch. This is consistent with the construction drawings from the rehabilitation works carried out in the 1960s. Furthermore, the parabolas of the rehabilitated stirrups are very wide, which verifies the use of flat plates used as stirrups in the Hennebique system.

![Stirrups and Original Reinforcements](image)

**Figure 4.17 - Radargrams 469, 470 and 471 made longitudinally on the vertical side of the arch**

4.2 Crack Measurement and Interpretation

In addition to the GPR studies on the Bôco Bridge, another non-destructive test performed on the concrete structural members was carried out, mainly crack measurements. These tests aimed to gain better understanding of the current state of the bridge. As mentioned earlier, there were three major cracks found on the Bôco Bridge: one on the masonry wall of the abutment on the side of Vieira do Minho and two cracks on each arch at the abutment. Due to lack of access to the structure below the deck, it was not possible to inspect for cracks near the mid span. The vertical crack on the abutment wall existed on the mortar joint and was not accessible for the microscopic crack measurement. The two cracks on the arch at the abutment were analyzed using Veho VMS-004 400X USB Microscope and the post-processing software MicroCapture. As previously refereed, the primary objective of this study was to gain better understanding of the current state of the damage on the structure, interpret the results and to document the crack widths. The results obtained from the analysis were categorized based on the location where the cracks were found.
The cause of the two cracks can be attributed to the internal pressure exerted by the expansion of reinforcement steel bars caused by corrosion. The corrosion of steel bars is caused by carbonation of concrete, high moisture content of the area, faulty drainage system, low density and high porosity concrete. On the side of Vieira do Minho, the maximum crack width found was 4.83 mm. On the side of Vieira do Amares, the maximum crack width found was 8.27 mm and the length of the crack was comparatively longer than the previous crack. Figure 4.18 shows locations of two cracks. The results from microscopic crack measurement performed on the side of Vieira do Minho is shown in Figure 4.19. The results from microscopic crack measurement performed on the side of Vieira do Amares is shown in Figure 4.20.

Figure 4.18 – Locations of two cracks

Figure 4.19 – Cracks located at the abutment on the side of Vieira do Minho
Figure 4.20 - Cracks located at the abutment on the side of Amares
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Chapter 5: Experimental Dynamic Characterization

The experimental dynamic assessment was performed on the Bôco Bridge using the ambient vibration of the bridge. The out-only modal analysis was carried out to obtain mode shapes, frequencies, and damping coefficients. For the test, nice accelerometers were used at fourteen locations to record data in vertical and horizontal directions. Seven different test setups were used to accommodate all the locations. For the post-processing of the data, a MATLAB code developed by researcher Gonçalo Escusa was used. The method used for the modal identification of the Bôco Bridge was Enhanced Frequency Domain Decomposition (EFDD). This method was chosen due to its simplicity and user friendliness. Due to limited time, different method such as Stochastic Subspace Identification (SSI) method was not used to compare the results and obtain modal assurance criterion (MAC) values. Nevertheless, this is highly recommended in future. Several challenges were faced during the test. Some of the channels were damaged and resulted in damaged signals. The resulting mode shapes contained clear sign of damage in the structure and the frequencies were well spaced.
5.1 Ambient Vibration Test

The full-scale dynamic identification can be performed using either ambient vibration of the bridge or the forced vibration. Ambient vibration test is output only system identification technique and it uses the natural excitation of the bridge in the form of wind, vehicles or pedestrian loading as the input excitation. The advantage of ambient vibration test is that it can be performed while the bridge is in use. Forced vibration requires external vibration sources and can be costly. The disadvantage of ambient vibration testing is that the induced excitation level to the bridge is unknown. Therefore, the dynamic characteristics of the bridge, especially damping coefficient obtained from the test is not reliable (Salawu, 1995). To obtain a reliable damping coefficient, dynamic test using a forced excitation in the form of a vehicle passing over the bridge can be useful. This excitation will provide more useful information in the signal processing regarding how the excitation dampens over time.

The output only identification techniques can be classified into frequency domain methods or time domain methods and can be used in single-degree-of-freedom (SDOF) or multi-degree-of-freedom (MDOF) systems. Frequency domain methods include classical single-degree-of-freedom (SOF) method such as peak picking, or multi-degree-of-freedom (MDOF) methods such as Frequency Domain Decomposition (FDD), Enhanced Frequency Domain Decomposition (EFDD), and polimax method. Time domain methods include Random Decrement (RD), recursive techniques, maximum likelihood methods, and Stochastic Subspace Identification (SSI) methods (Pospisil, 2015).

For the modal identification of the Bôco Bridge, Enhanced Frequency Domain Decomposition (EFDD) method was used. The primary reason for choosing this method is its user-friendliness, fast processing time and simplicity when compared to methods such as Stochastic Subspace Identification method (Brincker et. al, 2000b). On the other hand, the classical peak picking method gives reliable mode shapes only when the mode shapes are well spaced and clearly distinguishable from the other modes (Brincker et. al, 2000a). It assumes that the resonant mode shape dominates the resonant frequency and there is no contribution from the other modes (Pospisil, 2015). When the mode shapes are close to one another, this method gives erroneous result. The EFDD method is basically an improved version of the FDD method, which is based on the classical peak picking method. The EFDD method and the FDD method use the same initial steps. The difference lies in estimation of resonance frequencies and damping coefficients. The FDD method estimates the resonance
frequencies with discrete frequencies whereas the EFDD estimates resonance frequencies in time domain (Pospisil, 2015).

5.2 Test Setup and Procedure

To perform the dynamic analysis of the Bôco Bridge, nine accelerometers were used in fourteen locations to record vibration in vertical as well as horizontal direction. Six smaller accelerometers (PCB 393B12) were used and three bigger accelerometers (PCB 393B31) were used as reference sensors. The sensitivity of the accelerometers was 10 V/g and it is able to measure 0.07 mg. The data acquisition system used was NI SCXI 1000 and NI SCXI 1531 modules. Wooden cubes were glued to predefined critical locations, on which the sensors were attached as shown in Figure 5.1 (a). The channels were connected to a laptop to acquire the signals as shown in Figure 5.1 (b).

Figure 5.1 – Test setup of the experimental dynamic characterization: (a) accelerometer; (b) data acquisition system; and (c) test being performed on the bridge.
To accommodate the fourteen locations, a total of seven different test setups were used. For each setup, duration of the measurement was 15 minutes for the first five setups and 10 minutes for the last two setups. Figure 5.2 shows the locations of the sensors as well two reference points (Ref 1 and Ref 2). In all the setups, a sampling frequency of 200 Hz was used, which is the maximum value recommended by the manufacturer of the accelerometer. The locations of sensors and references were chosen strategically so that the most important mode shapes can be captured. The locations chosen were directly above the columns to obtain more precise information about the entire structural system of the bridge. In all the test setups, three reference sensors were not altered.

![Figure 5.2](image-url)

**Figure 5.2** – Locations of sensors and reference points: (a) plan view; and (b) elevation view.
The corresponding channels and sensors used for the seven test setups were shown in Table 5.1. The signals obtained from the channels shown in red (bold) were damaged and therefore not used in the analysis. This was mainly caused by damaged channels using in the test setup. The channel A3, A4 and A7 were damaged as shown in Table 5.1. In some cases, poor connections between the channels and the sensor resulted in damaged signals. Vehicles passing over the channels while the test was being performed might have caused this. With these seven test setups, all sensor locations were covered in both degrees of freedom: vertical (V) and horizontal (H). The correct assignment of the sensors and channels was crucial in obtaining reliable results in post processing stage.

Table 5.1 – Channels and sensors used for the 7 setups of the ambient vibration test

<table>
<thead>
<tr>
<th>Setup</th>
<th>Channels and sensors used for the ambient vibration test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref2 H, Ref1 V, 1 V, 1 H, 2 V, 2 H, 3 V, 3 H</td>
</tr>
<tr>
<td>2</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref2 H, Ref1 V, 4 V, 4 H, 5 V, 5 H, 6 V, 6 H</td>
</tr>
<tr>
<td>3</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref2 H, Ref1 V, 7 V, 7 H, 8 V, 8 H, 9 V, 9 H</td>
</tr>
<tr>
<td>4</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref2 H, Ref1 V, 10 V, 10 H, 11 V, 11 H, 12 V, 12 H</td>
</tr>
<tr>
<td>5</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref2 H, Ref1 V, 10 V, Ref1 H, 11 V, 4 V, 12 V, 12 H</td>
</tr>
<tr>
<td>6</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref1 V, Ref1 H, 10 V, 10 H, - 11 H, 12 V, -</td>
</tr>
<tr>
<td>7</td>
<td>Channel: A0, A1, A2, A3, A4, A5, A6, A7, A8</td>
</tr>
<tr>
<td></td>
<td>Sensor: Ref2 V, Ref1 V, Ref1 H, 10 V, 10 H, - 4 H, -  -</td>
</tr>
</tbody>
</table>

(Note: signals obtained from the sensors marked in red (bold) were not used for the analysis because the signals were damaged)
5.3 Ambient Vibration Test Results

The data obtained from the ambient vibration test was post-processed using ARTeMIS (SVS, 2006) initially. But due to erroneous results obtained from ARTeMIS, the results were analyzed using a MATLAB code developed by researcher Gonçalo Escusa. The enhanced frequency domain decomposition (EFDD) method was used to process the data. The possible reason for the faulty results from post-processing of the data using ARTeMIS are damaged signals obtained from some sensors and vibrations caused by external sources such as vehicles passing over the bridge while the test was in progress. On the other hand, the transient signals due to passing of vehicles can provide the excitation to the structure, which can be used to estimate the damping coefficient of the structure.

Figures 5.3 and 5.4 shows examples of signals recorded from sensor placed at the mid-span in the vertical and horizontal direction respectively. The maximum excitation caused by the passing of vehicles is less than 8 mg.

![Figure 5.3 – Time record for the sensor Ref2 V (vertical) from the setup 1](image)
The first twelve mode shapes are shown in Figure 5.5, along with the corresponding frequencies ($f$) and their damping ratios ($\xi$). The natural frequencies are well spaced between 3.8 Hz to 66.41 Hz. The first mode with the frequency of 3.8 Hz is the horizontal and vertical mode shape. The second mode with the frequency of 4.11 Hz is first torsional mode shape. The third mode with the frequency of 6.24 Hz is the first flexural mode shape. In the Figure 5.5, it can be observed that the third mode shape contains clear sign of damage to the structure because the flexural mode shape is not symmetric as expected from the numerical model. Another reason for the lack of symmetry in the mode shapes is geometric imperfections in the bridge. Furthermore, the rehabilitation work carried out in the year 1962 by enlarging the cross-section can be one of the reasons as well. The fourth mode with the frequency of 9.88 Hz is the first flexural + torsional mode shape. Likewise, the fifth mode with the frequency of 11.55 Hz and the sixth mode with the frequency of 14.76 Hz are the second and the third flexural + torsional mode shape respectively. The seventh mode with the frequency of 18.53 Hz, the eighth mode with the frequency of 20.1 Hz and the ninth mode with the frequency of 21.91 Hz are the second, third and fourth torsional mode shapes respectively. The seventh mode shows clear sign of damage in the structure. The tenth mode and the eleventh mode shapes are localized mode shapes. Lastly, the twelfth mode shape with the frequency of 66.41 Hz is the fifth torsional mode shape.
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1\textsuperscript{st} Mode, Frequency ($f$) = 3.8 Hz, Damping ratio ($\xi$) = 5.27%

2\textsuperscript{nd} Mode, Frequency ($f$) = 4.11 Hz, Damping ratio ($\xi$) = 1.94%

3\textsuperscript{rd} Mode, Frequency ($f$) = 6.24 Hz, Damping ratio ($\xi$) = 3.00%

4\textsuperscript{th} Mode, Frequency ($f$) = 9.88 Hz, Damping ratio ($\xi$) = 1.31%

Figure 5.5 – Mode shapes from the experimental ambient vibration test
Figure 5.5 – Mode shapes from the experimental ambient vibration test (contd.)
Assessment and Rehabilitation of the Bôco Bridge

9\textsuperscript{th} Mode, Frequency ($f$) = 21.91 Hz, Damping ratio ($\xi$) = 3.11\%

10\textsuperscript{th} Mode, Frequency ($f$) = 27.22 Hz, Damping ratio ($\xi$) = 1.90\%

11\textsuperscript{th} Mode, Frequency ($f$) = 33.41 Hz, Damping ratio ($\xi$) = 1.45\%

12\textsuperscript{th} Mode, Frequency ($f$) = 66.41 Hz, Damping ratio ($\xi$) = 0.96\%

Figure 5.5 – Mode shapes from the experimental ambient vibration test (contd.)
The average value of 2.23% was obtained for the damping coefficients. The average damping coefficient obtained from the analysis was within the expected range for concrete structures. Figure 5.6 shows the comparison of frequencies with the damping coefficient. It can be concluded that the damping coefficient of the structure is nonlinear and can be represented by a polynomial with the $n^{th}$ power.

![Damping coefficients of the first 12 modes](image)

**Figure 5.6 – Damping coefficients of the first 12 modes**
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The in-situ load test was performed to obtain vertical displacement of the deck when loaded with moving truck of known weight. The resulting information obtained from this test will be used to calibrate the numerical simulation. The information obtained from the load test is very crucial in understanding the structural integrity of the bridge and its current state. To measure the vertical deflection of the deck of the bridge, a new system designed by Dr. Jose Sena-Cruz was used. The system uses LVDT sensors to record displacement of the deck with the help of a steel cable hanged from the frame. A dead weight is attached to one end of the cable and dropped in the river. The other end is attached to a spring, which is connected to LVDT sensors. The system was tested twice before and produced very reliable results. When the system was used for the load test of the Bôco Bridge, several challenges were faced and resulted in unusable data. Only on test setup produced good result in two sensors placed at the mid-span. The possible causes of the problems associated with the measuring system were that the holes drilled on the deck of the bridge was too small, the dead weights were inadequate, and the stiffness of the springs had considerable influence on the vertical movement. It is highly recommended that the load test be repeated with all the causes addressed in order to obtain better results for future use.
6.1 Patented System for Measuring Displacement in Bridges

The vertical displacement in the bridges is the most important parameter that can be used to obtain crucial information about the load carrying capacity of the structure and to calibrate the numerical modeling. Moreover, vertical displacements in the bridges provide very important information about the structural integrity of newly constructed bridges. It is also very instrumental in assessing the current state of the bridge in terms of its load carrying capacity and the current state of the damage for short and long term (ACI 2003 a).

The most difficult aspect of obtaining vertical displacement for the bridges is lack of access to the stationary point. Other challenges faced in obtaining vertical deformation of the bridges are the high cost of the systems and inadequate sampling frequency and accuracy. There are numerous techniques that are available in the market. These techniques include laser scanners, ground radar interferometry and non-contact imaging techniques such as photogrammetry. Furthermore, GPS can be used to measure deflection but the precision is very low and the system is very costly. Although there are numerous new systems of measuring vertical displacement of the bridges that are being developed currently, many factors are stopping those from being able to use for the real structure. The challenges include high cost, need for highly skilled operator, inadequate sampling rate and low precision of the readings.

To overcome these challenges, an alternative system for measuring vertical displacement of bridge is developed by Dr. Jose Sena-Cruz. This system is used for the load test of Bôco Bridge. The main advantage of the bridge is low cost and applicability in the bridges with river flowing underneath. The vertical displacement is measured directly. Furthermore, it can be operated by an unskilled labor as well. The downside of this system is that the signal obtained may contain noise due to wind or flow of river on the steel cables. This system requires a small hole to be drilled. Therefore, this system can be considered a minor destructive technique.

6.1.1 Description of the System

The system uses a very simple setup using Linear Variable Differential Transducers (LVDT) to record vertical displacement of the bridge deck. The system will require a steel cable of length equal to the distance from the top surface of the deck to the bottom of the river. Before the system is used, this distance needs to be measured to make sure that the length
of the cable is adequate. Once this system is assembled, it can be readily used on other structures.

The setup of the system is shown in Figure 6.1. It consists of following components: (1) dead weight made of concrete block; (2) steel cable of length equal to the vertical distance from the deck of the bridge to the bottom surface; (3) LVDT tab attached to the cable; (4) spring system; (5) stretcher for the purpose of pre-stressing the steel cable; and (6) steel frame that can be attached to the deck of the bridge; and, a LVDT. The steel cable is assumed to have an infinite axial stiffness and the spring to have no axial stiffness. All the vertical movement of the bridge will occur in the spring and thereby transferred to the LVDT readings. The use of LVDT allows us to obtain significantly high data acquisition rate of 100 readings per minute or higher and very acute resolution in the order of microns.

![Diagram of measuring system]

(a)  (b)  (c)

Figure 6.1 – The set-up of the measuring system: (a) schematic diagram (Sena et. al, 2015); (b) system used on the Bôco Bridge; (c) close-up view of the set-up.

6.1.2 Testing of the System in the Lab

The system was tested using two hydraulic jacks placed under the steel frame moving simultaneously as shown in Figure 6.2 (a). The result obtained from the test was compared
against the known displacement imposed by the hydraulic jacks. The comparison is shown in the Figure 6.2 (b) and the result is very satisfactory. To analyze the effect of the temperature on the performance of the system, the system was tested in the climatic chamber with the temperature varying from -15°C to +44°C. The result is shown in the Figure 6.2 (c).

6.1.3 In-situ Test of the System

The system was tested at the mid-span of the central section of the newly constructed bridge and a truck of 445 kN was driven across the bridge twice. The bridge has a total span of 390 m, broken into three spans of 120 m, 170 m and 100 m. The result obtained from the test is shown in the Figure 6.3. The system gave a very promising result even when the test was performed in a very windy day. This explains the presence of negligible noise in the readings.

Figure 6.2 – (a) setup of the system in lab; (b) results; and (c) thermal test (Sena et. al, 2015)

Figure 6.3 – In-situ test result on a new bridge over Dão River near Coimbra in Portugal (Sena et. al, 2015)
The presence of two piles that divides the bridge into three spans explains the upward displacement of the mid-span of the central section when the truck passed through the mid-span of the two sections on the side.

The system generated very good results when tested in the lab as well as in-situ testing on a newly constructed bridge. Therefore, the system was used to obtain vertical displacement of the Bôco Bridge when subjected to a static load as well as moving load. The results obtained from the test are presented in the following sections.

6.2 Load Test of the Bôco Bridge

The load test was performed on the Bôco Bridge using the aforementioned displacement measuring system. The objective of performing load test on the Bôco Bridge was to analyze the current state of the bridge in terms of its capacity and to calibrate the numerical model. The calibrated numerical model can be used to perform the safety analysis of the bridge in its current state and rehabilitated bridge. The results obtained from the load test can also be used to design the rehabilitation of the bridge if it was found that the initial cross-section of the bridge was not adequate. Since the bridge is in the process of being labeled national heritage, the installation of the measuring system must be carried out while giving utmost importance to the bridge. The system cannot cause major damage to the bridge. Therefore, the holes that are needed to be drilled on the deck of the bridge has to be very minimal.

The initial prediction before the test was performed was that the vertical deflection at the mid-span would be highest when compared to the other locations. Furthermore, it was predicted that the vertical deflection of the bridge deck at the mid-span when a truck with 10 tons of load crosses the bridge would be very minimal and less than 1 mm. This is due to the fact that the span of the bridge is significantly shorter when compared to the new bridge over Dão River near Coimbra, where the maximum deflection was 3 mm and the span was 390 m. In addition to this, the influence line of the bridge was predicted to be only in the negative direction, unlike the influence line at the mid-span of the new bridge over Dão River near Coimbra, as shown in Figure 6.3.

It was decided that six LVDT sensors would be used for the load test. The location of LVDT sensors were decided based on the preliminary results obtained from the numerical model before the calibration was performed. It was decided that the three sensors would be placed on each side of the bridge, with two LVDT sensors at the mid-span. The locations of other
four LVDT sensors were placed at the fourth vertical column from the abutment. At this location, it was found that the vertical displacement was most sensitive to the moving vehicle and the influence line confirmed this observation as well. The final locations of six LVDT sensors are shown in Figure 6.4. The four LVDTs on the side were placed 8 m away from the abutment. For the LVDT locations 2, 3, and 4, hollow cylindrical concrete tubes of approximately 100 kg were attached to the end of the cable and submerged into the river. For the LVDT locations 0, 1 and 5, the ends of the cable were directly anchored on the exposed rock as shown in Figure 6.4.

![Figure 6.4 – The set-up of the measuring system](image)

The diameter of cables used were 16 mm while the diameter of holes that were drilled on the deck of the bridge was 22 mm. It was preferred to drill holes that were much larger than 22 mm to allow for obstruction free movement of the cables during the test. Due to lack of
sufficient human resources and equipment, some of the holes were slanted and caused friction on the cables. The LVDTs used were ACT series LVDT from RDP Electronics with stroke of +/- 50 mm and linearity of 0.10%. The data acquisition system used was NI SCXI 1000 and NI SCXI 1531 modules. Figures 6.5 shows the load test being performed and Figure 6.6 shows the setup of the measuring system and the data acquisition system.

(a)                                                                 (b)

Figure 6.5 – The load test in progress: (a) moving truck; and (b) the steel cables.

(a)                                                                 (b)

Figure 6.6 – The set-up of the displacement measuring system: (a) system used on the Bôco Bridge; and (b) the data acquisition system.

The truck used for the load test had three axles with the total load of 115.6 kN. This load level was defined due to the limitations in terms of current use of the bridge as shown in
Figure 6.7. The truck weighed 53 kN and the tanker filled with water weighed 62.6 kN. The distance between the three axles were 2.3 m and 5.4 m respectively. The vertical deflection measured by the measuring system detects only the vertical displacement of the deck due to the truck loading because the self-weight of the bridge is not taken into account. Figure 6.8 shows the dimension of the truck and the resulting weight on each axle.

Figure 6.7 – Current vehicle restriction imposed on the Bôco Bridge

Figure 6.8 – The truck used for the load test: (a) distance between the wheels; (b) axle dimensions; and (c) loads assigned at each axle.

When the results from all tests performed were analyzed, several problems were noticed with the data. Out of four tests that were performed on the Bôco Bridge, only one result was usable for the further analysis. The other results provided faulty influence lines and could not be used for the present analysis. The possible cause of the faulty results were due to the dead weight being insufficient, lack of proper technicians to install the system, lack of proper tools and problems associated with the assembling of the system itself. Furthermore, the
The diameter of holes that were drilled on the deck of the bridge was not adequate and caused resistance to the movement of the cables. In addition to this, the stiffness of the springs used in the system was supposed to be close to zero, when compared with the stiffness of the cables. Unfortunately, due to some geometrical imperfections on the cables required additional force in the springs to straight them. However, the pre-stressed applied to the cable may be not sufficient to perfectly straight them.

The results obtained from the LVDT locations 3 and 4 are shown in Figures 6.8 and 6.9 respectively. The maximum vertical deflection observed in LVDT 3 and 4 were -0.4997 mm and -0.4297 mm respectively. The difference between the maximum vertical deflection registered by LVDT 3 and 4 (at about 16%) may be justified by the path used by vehicle that might be off centered and by the existing damage on the bridge. As seen in Figures 6.8 and 6.9, the vertical deflection reading in LVDT 4 has a time lag of about 3 seconds when compared to the reading observed in the LVDT 3. Another abnormality in the recorded data is that the displacement reading of the LVDT 4 did not reach the zero as it is expected. These observations might be caused by slight malfunction in the measuring system such as abrasion between the cable and the hole, or loosening of anchoring at the cable end. Nevertheless, the maximum vertical displacement recorded from this test can be used for the calibration of the numerical model. Four different tests were performed and the results obtained from the six sensors were analyzed. It was observed that the maximum vertical displacement obtained in all the tests performed gave results that are very closed to the results obtained from the LVDT 3 and LVDT 4 of this test. The influence line in Figure 6.8 shows initial period of linear increase of vertical displacement, followed by a flat plateau and then a linear decrease of vertical displacement. This behavior is consistent with the prediction made prior to the test.

![Figure 6.9 – Vertical displacement recorded at LVDT location 3](image-url)
Figure 6.10 – Vertical displacement recorded at LVDT location 4
Chapter 7: Numerical Simulations

The numerical simulation of the Bôco Bridge was performed using SAP2000 (CSI, 2011) software. The geometry of the bridge was based on the historical construction drawings and the material properties of the concrete was obtained from the material characterization performed on the concrete specimen. The analysis of the bridge assumed uncracked concrete structural members with no damage. The analysis performed was linear elastic analysis. The numerical model was then calibrated using results obtained from the in-situ load test. The calibration was carried out manually by adjusting the support conditions, connection between the elements, and simulation using different element types. The objective of the calibration was to match the vertical displacement of the deck to the result obtained from the load test. The primary objective of the numerical simulation was to determine the maximum bending moment applied to each structural member. The results obtained from the numerical simulation will be used in the safety analysis of the critical structural members.
7.1 Calibration of Numerical Modeling

The numerical simulation of the bridge was performed using SAP2000 (CSI, 2011). The analysis was carried neglecting the damage in the structure and it used linear elastic analysis. The aim of the numerical simulation was to identify the critical members and the moments subjected to it. When performing numerical simulations of concrete, it is very challenging to include the damage in the structural members. A more detailed analysis that includes cracks in the concrete cross-sections is beyond the scope of this dissertation. Therefore, a simpler approach was taken to simulate the Bôco Bridge. Moreover, reinforcement steel was neglected in the numerical modeling because it does not significantly affect the resulting bending moment applied on individual structural members. Nevertheless, the reinforcement steel was included in the safety analysis to obtain the capacity of each structural member. Two different types of numerical models were considered for the analysis: Model M1 and M2. The difference between the two models lies in the way deck of the bridge was modeled. Here, both models were presented and their results were compared against the result obtained from the in-situ tests.

The material properties used for the concrete were obtained from the material characterization performed using the concrete specimen. The class of concrete used was C35/45 with the modulus of elasticity 34 GPa. The density of concrete used was 25 kg/m$^3$. The Poisson’s ratio of concrete was 0.2 and the shear modulus of concrete used for the numerical modeling was 14.17 MPa. The cobblestone pavement of the deck in the carriageway was constructed using cubes of 10 cm, which were simulated using a dead load of 2.5 kN/m$^2$.

Figures 7.1 and 7.2 shows the two different numerical models used for the Bôco Bridge. The model M2 was a slight modification of the initial model created by Araújo (Araújo, 2013). The difference between the two models was the way the deck of the bridge was simulated. The model M1 used two thin shell elements for the carriageway and the sidewalk, which were offset vertically from the level of longitudinal beam. The model M2 used a square column of 10 cm side to connect the sidewalk shell element with the longitudinal beam. The support condition, material properties, and the cross-section geometries used for both models were same.
The calibration was performed manually by taking into consideration different support conditions, structural element types, end restrictions, and connections between structural members. The results obtained from the load test were compared with the results obtained from the numerical simulation and the difference between the two values was minimized by performing many iterations with necessary changes.

The model that produced most accurate result was the one in which the deck slab and the sidewalks were simulated with thin FE shell elements. Other structural members such as
longitudinal beams, transversal girders, columns, and arches were simulated using beam elements. The support condition used for the arches at the abutments was simulated using fixed connection. The support connection between the deck slab and abutment was simulated with free translational movement along x direction, free rotation along y direction and restrained in other degrees of freedom. Figure 7.3 shows the vertical displacement contour when the bridge is loaded with the same truck that was used in the in-situ load test. Figure 7.4 shows the vertical displacement at the mid-span of the deck.

Figure 7.3 – The vertical displacement contour of (a) model M1; and (b) model M2.

Figure 7.4 – Vertical displacement at the mid-span in the model M1 (U3 = -0.49 mm)
The vertical displacements obtained from the load test were 0.50 mm for LVDT 3 and 0.43 mm for LVDT 4. The vertical displacements obtained from the calibrated numerical model were 0.49 mm and 0.46 mm for the numerical model M1 and M2, respectively. The difference between the average values of vertical displacements obtained from the load test with the result obtained from the numerical models were 0.027 mm and 0.004 mm for the model M1 and M2 respectively. In this context, in the following sections model M1 is used due to its balance in terms of simplicity and accuracy.
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Chapter 8: Rehabilitation of the Bôco Bridge

To guarantee the safeguarding of the Bôco Bridge, it is extremely crucial that it is continued to be used as a traffic bridge connecting the Vieira do Minho and Amares. The municipal authorities of both district showed interest in upgrading the present traffic capacity of the bridge to accommodate higher traffic load. Another option at hand was to construct a new bridge. Due to limited budget, rehabilitation of the Bôco Bridge is the more desirable alternative. This will help improve the current state of the bridge and maintain it's important in years to come. The proposal to upgrade the capacity of the structure was presented here. The main idea was to remove the inferior quality concrete used during the rehabilitation project of the year 1962 and replace it with a higher quality concrete, which was used in the original construction. The step-by-step guide to the process of rehabilitating the Bôco Bridge is presented here. In the end, safety analysis of the bridge was performed using the numerical model. The critical members were identified in the numerical model along with the subjected bending moments. The applied loads were then compared against the capacity of each structural member to guarantee its safety. The suggested rehabilitation was proposed since it was expected that the desired safety levels were reached with this solution. If a structural member is not verified, then its cross-section will be modified to reach the desired capacity.
8.1 Proposed Rehabilitation of Concrete Structural Members

After having discussions with the municipal authorities of Vieira do Minho and Amares, it became clear that there are two options regarding the future fate of the Bôco Bridge. The present load carrying capacity and the traffic volume of the Bôco Bridge is inadequate. There are two engineering solutions: to rehabilitate the Bôco Bridge or to construct a new bridge. Construction of a new bridge is extremely costly when compared to the cost of rehabilitation of the Bôco Bridge to meet current requirements. More importantly, upgrading the current Bôco Bridge will guarantee it’s safeguarding in the future. If a new bridge is constructed, Bôco Bridge will surely suffer neglect and lack of maintenance. This will result in the loss of a very important heritage structure of Portugal. Since Bôco Bridge is Portugal’s oldest reinforced concrete bridge in use, it is a national treasure and must be safeguarded at any cost. A continued use of Bôco Bridge will not only stop it from being demolished, it will guarantee its continuous maintenance as well. Therefore, it was decided with the municipal authorities that a proposal to rehabilitate the Bôco Bridge is required.

The material used for the proposed rehabilitation of the Bôco Bridge is similar to the concrete and steel used in the original construction. The material characterization of the concrete cylinders for the Bôco Bridge found that the concrete used in the original construction was of very high quality, with the compressive strength class of C35/45. Therefore, for the rehabilitation of the Bôco Bridge, C35/45 concrete will be used. Regarding the steel, grade of S400-B will be used. In Portugal, the commonly used steel for the reinforced concrete is S400-B and S400-C. For the Bôco Bridge, S400-B was chosen because the higher ductility steel S400-C is not needed in the northern Portugal where seismic actions are not as common as southern Portugal. This grade of the steel has more ductility when compared to the S400-A. For the safety analysis of the critical structural members, mechanical parameters of C35/45 concrete and S400 steel are used.

8.2 Procedure for Proposed Rehabilitation of Bôco Bridge

Before rehabilitation of deteriorated concrete is carried out, it is very important to properly analyze the extent of damage, cause of the damage, current state of the damage process, location of damaged concrete, depth of the deterioration, and the state of the corrosion of the reinforcement bars. Without obtaining such crucial information, it is pointless to proceed
with the following steps. Section 3.1 and 3.2 discussed the main types of damages found in the Bôco Bridge, which can be the foundation for a detailed analysis of the damage survey necessary for the rehabilitation of the bridge. To study whether the corrosion of the reinforcement bars is active or not, non-destructive tests such as half-cell electrical potential test and measurement of chloride ion content of the surrounding concrete can be used (ACI 2003 Vol. 2). Although one might think that a visual inspection might be adequate for studying the concrete delamination and spalling, the actual area of deteriorated concrete beneath the delaminated concrete or spalling is much larger. Therefore, extra care needs to be taken when removing deteriorated concrete. To analyze the depth of concrete cover, hand-held or a rolling pachometer can be used (ACI 2003 Vol. 2). A detailed study of the damage state of the bridge needs to be documented for future reference.

The material used for the retrofitting must be a similar to the original concrete used in the construction. The concrete used in the original construction was found to be of grade C35/45, which is recommended for the retrofitting work presented here. The repair material must be self-binding, with sufficient amount of binding agent to obtain proper adhesion to the original concrete. Before the new concrete is used in retrofitting, its appearance must be checked and must closely resemble the concrete used in the bridge before this work is carried out. Since this bridge is in a process to be designated a national heritage, extra care must be taken to maintain its original appearance. The reinforcement steel to be used in the rehabilitation must be S400. A step-by-step guide to retrofit the bridge is explained below.

8.2.1 Removal of Deteriorated Concrete Layer

There are numerous methods available in market to remove deteriorated concrete from the structure, each with its own applicability and limitations. These methods fall into four main types: cutting, impacting, blasting, and hydrodemolition (ACI 2003 Vol. 1). Cutting methods can be implemented using diamond saw, wire cutting, mechanical shearing, stitch drilling, thermal cutting and high-pressure water jet. For impacting methods, available techniques are hand-held breakers, boom-mounted breakers, and scabblers (ACI 2003 Vol. 1). Blasting method uses explosives to blast boreholes to remove thicker concrete layers. Lastly, hydrodemolition uses high-pressure water jetting to remove deteriorated concrete while keeping the reinforcement bars for reuse.

For the particular case of Bôco Bridge, hydrodemolition is chosen as the means to remove deteriorated concrete. The reason for choosing this method is that it doesn’t cause damage to the remaining concrete and reinforcement steel that are not damaged can be used in
retrofitting. The difficulty of access to the bridge superstructure also plays an important role in selecting this method. Moreover, the maximum depth of concrete removal for hydrodemolition is 15 cm, which is suitable for the Bôco Bridge. Furthermore, this method is ideal for the Bôco Bridge because of the poorer grade of concrete used in 1962 rehabilitation project. The pressure of the jetting can be set in such a way that only the new concrete is removed from the structure. This is possible because of significant difference in strength and quality of concrete used in original construction and new concrete used to enlarge cross-sections in the rehabilitation project carried out in the year 1962. On the other hand, reinforcement bars that are not corroded can be reused in the retrofitting, resulting in considerable cost saving. Another method with similar characteristics is the sandblasting. The reason for not using sand blasting for Bôco Bridge is because this method uses certain chemical and it has negative environmental effects. Figure 8.1 shows two examples of use of hydrodemolition on bridges.

This method requires large quantity of potable water for the jetting, which can be obtained directly from the river under the bridge with no difficulty. The output pressure resulting from high-pressure water jetting ranges between 70 to 140 MPa and the quantity of potable water required is 75 to 150 L/min. This method leaves a rough surface of remaining concrete, which means that the concrete members will not require surface roughening or sand blasting for proper bonding with the new concrete layer.

![Figure 8.1](image)

**Figure 8.1** – Examples of hydrodemolition: (a) hydrodemolition used on a bridge (courtesy of Knights Group, Ireland), and (b) hydrodemolition used on the overhead surface of a bridge (courtesy of Tro-Chaînes, Canada).
8.2.2 Surface Preparation and Reinforcement Repair

Before the new layer of concrete is casted, the surface needs to be properly prepared. This step is very crucial in obtaining proper bonding between the old and new concrete, resulting in weak structural performance of the retrofitted structure. A temporary support structures needs to be designed and erected to guarantee adequate safety. Since hydrodemolition is used for the removal of deteriorated concrete, the resulting surface is rough and do not require further surface preparation in terms of surface roughening in most cases. If the surface is too smooth or rough, sand blasting or high-pressure water blasting might be required. The roughness of a concrete surface can be described quantitatively by using the mean roughness $R_a$, which is the average deviation of the profile from the mean line (fib, 2013). For the proper bonding with the new concrete, the roughness of the old concrete surface must be between Rough to Very Rough. A Rough surface corresponds to $R_a \geq 1.5$ mm and a Very Rough surface corresponds to $R_a \geq 3$ mm.

Proper care must be taken to make sure that the exposed reinforcement bars are treated properly before being used for the retrofitting. The steel bars need to be carefully inspected for corrosion. If corrosion is detected, it must be removed using high-pressure water blast. If the backside of the perimeter of steel bar is corroded, the surrounding concrete needs to be removed and steel needs to be cleaned on all sides.

Since removal of concrete is mostly on the surface with partial depth, the geometry and the depth of removal must be carefully planned. The geometry of the removal must be kept as simple as possible with corners of 90 degrees. Feather edges must be avoided to obtain proper bonding between the old and the new concrete. The depth of the removed concrete layer must be greater than the recommended minimum thickness of the repair concrete. Using hydrodemolition on concrete leaves slurry, which must be removed entirely before it hardens. Last step in surface preparation involves cleaning the surface of the concrete to remove any dirt, slurry, or aggregates that are not bonded to the surface.

8.2.3 Sealing of Cracks

The first step in repair and sealing of cracks is to assess the current state of the cracks and to study the cause of the cracks. This preliminary study must include a detailed analysis on knowing if the observed cracks are active or dormant using crack monitoring techniques. If the observed cracks are active, further actions must be taken to stabilize the structure. Considering the fact that the lower quality concrete that was used in the restoration project in
the year 1962 will be completely removed, the structure should not have many cracks in the original concrete due to its very low porosity and high density.

There are numerous methods to repair cracks in concrete such as epoxy injection, routing, stitching, gravity filling, grouting, drypacking, polymer impregnation, and autogenous healing (ACI 2003 Vol. 1). For this particular case, epoxy injection is recommended. The reason for choosing this technique is that it can seal crack as narrow as 0.05 mm and it is widely used in repairing cracks in bridges (ACI 2003 Vol. 1). However, it is recommended to seal only the cracks wider than 0.5 mm. This is because a new layer of concrete will be applied on the old surface. It requires highly skilled labor. The general process of this technique is cleaning the cracks, sealing the surface to prevent leakage, preparing the epoxy mix, injecting the epoxy, and finally removing the seal.

8.2.4 Replacement or Overlaying of Corroded Reinforcement Bars

Before reinforcement bars are repaired, a careful inspection of the reinforcement bars is vital. After the reinforcement bars are cleaned, the pouring of concrete must be planned as early as possible to avoid rusting of steel bars during the wait time. If the pouring of concrete takes long time, rusting of steel bars will occur. If the rust is loose, it must be cleaned again using wire brushing or sandblasting before fresh repair concrete is poured.

To repair the corroded bar after it is properly cleaned, there are two alternatives: complete replacement of damaged bar or addition of supplementary bars. The decision to replace or to add supplementary bar depends on the availability of adequate funding, length of the affected steel bar, and the depth of the corrosion. If supplementary steel bar is added on the damaged steel bar, adequate lap splice length should be provided to obtain proper overlap of original and supplementary steel bars (ACI 2003 Vol. 1). The required lap of steel bars between the old steel bars and new bars can be connected using swedged bolts, stick weld or threaded coupling (Sena-Cruz, 2015). In practice, the supplementary steel bar is placed 2 cm away from the affected bar. In this case, due to very limited concrete cover, the supplementary bar can be welded onto the original steel bar (ACI 2003 Vol. 2). It is recommended to coat the reinforcement bars before the fresh concrete is poured. The coating must be less than 0.3 mm. Otherwise, it will result in loss of bonding with the new layer of concrete.
8.2.5 Formwork and Pouring of Concrete

Formwork for the pouring of repair concrete must be constructed in a very well thought out manner. The geometry of the retrofitted structural members must closely resemble the former geometry of the bridge before the removal of concrete. The Bôco Bridge is the oldest reinforced concrete bridge in use in Portugal. Therefore, the formwork must for the pouring must closely match the original geometry of the structural members.

The application of fresh concrete can be done using several different techniques such as dry packing, pumping, cast-in-place, shotcrete and preplaced aggregates. Considering the geometry and accessibility of the concrete members in the Bôco Bridge, cast-in-place and pumping techniques are recommended. The repair material must have low water-cement ratio to increase the strength of the concrete, and more workable concrete to allow proper casting. The reason for choosing lower water-cement ratio was to avoid shrinkage and to obtain higher strength. To obtain a higher workability concrete, which is very necessary, chemical additives can be added. The chemical additives can reduce shrinkage as well. The repair material must be compacted using rodding if cast-in-place technique is used. Cast-in-place method is suitable for vertical elements where gravity assists in compacting the fresh concrete.

Where reinforcement bars are congested, or the area of application is in overhead locations, fresh concrete may be pumped using a concrete pump. This method is more suitable for the inclined members such as arches, and bracing beams between the arches. When the pressure built up reached desired value, the valve can be closed. In addition to the grade of concrete, which was specified as C35/45, other characteristics such as maximum aggregate size and slump class are important. The maximum aggregate size in terms of diameter for the rehabilitation of Bôco Bridge must be 12.5 mm and the slump class that is suggested for this project is S4 (EN 206-1:2000).

Throughout the process of retrofitting the concrete structural members on the Bôco Bridge, quality control must be performed constantly to check if the repair work meets the standard. This can be performed by doing visual inspection, in-situ testing, or extracting core samples for testing. When the retrofitting work is completed, detailed study must be performed to verify the durability, strength and serviceability requirements. Figure 8.2 shows the step-by-step process of the proposed rehabilitation of the Bôco Bridge in the form of a flowchart.
Proposed Rehabilitation of Concrete Structural Members

Initial Studies and Preparations
- Detailed damage analysis
- Detailed mapping and documentation of deteriorated concrete and steel
- NDE tests to analyze the corrosion and concrete cover
- Check the visual appearance of the new concrete

Removal of Deteriorated Concrete Layer
- Safety verification of the bridge prior to the removal
- Set up of hydromechanical machine and water inlet/outlet
- Removal of the deteriorated concrete

Surface Preparation and Reinforcement Repair
- Verify that the surface is ‘Rough’ to ‘Very Rough’

Sealing of Cracks
- Preliminary studies to check if the cracks are active
- Active cracks?
  - Yes → Further actions to stabilize the structure
  - No

Replacement or Overlaying of Corroded Reinforcement Bars
- Detailed inspection of steel bars for corrosion
  - Severe, deep & long crack?
    - Yes → Replacement of corroded steel bars
    - No → Coating of steel bars
  - Overlying of supplementary steel bars

Formwork and Pouring of Concrete
- Installing formwork of accurate replication of original geometry
  - Cast-in-place possible?
    - Yes → Cast-in-place pouring technique
    - No → Pumping of fresh concrete

Verification of Durability, Strength and Serviceability Requirements
- Detailed inspection of the completed work
- In-situ load test

Plan and initiate a comprehensive maintenance plan

Figure 8.2 – Flowchart for the proposed rehabilitation of the Bôco Bridge
8.3 Safety Analysis of Rehabilitated Bridge

The safety analysis of the rehabilitated bridge was performed using SAP2000 numerical model and CSAnalysis (Miranda et. al, 2008). The loading of the bridge was determined using the specifications from Eurocode EN 1991-2:2003 (Eurocode, 2003). SAP2000 numerical model was used to determine the maximum bending moment and maximum compressive strength subjected to each category of structural members. CSAnalysis software was used to determine the capacity of each category of structural member in terms of its bending moment and compressive force capacity. For this purpose, the cross-section used for the analysis was the enlarged cross-section with the added reinforcement bars. The load carrying capacity obtained from this analysis was compared with the applied loads in the numerical model.

For the loading on the bridge structure, Eurocode EN 1991-2:2003 was used. The load model used for safety analysis was Load Model 1 (LM 1), which consists of a concentrated load comprising of two axles, and a uniformly distributed load. This load model was chosen because it is used for general verification. The notional lane used for the Bôco Bridge was 1 because the width of the carriageway width was 3 m. Since the Bôco Bridge is located in the rural area with significantly lower traffic volume when compared to the traffic level considered in the Eurocode EN 1991-2:2003, a reduction factor was used to reduce the load level to a reasonable value. The concentrated axle load of 50 kN/m and a uniformly distributed load of 4 kN/m were used. For the safety analysis, specifications given in Eurocode EN 1992-1-1:2004 was used (Eurocode, 2004).

8.3.1 Critical Members

The calibrated numerical model was used to find the members with critical stresses. The aforementioned loading pattern was applied to the bridge, and the resulting bending moment values were studied. The structural member that is subjected to the maximum moment was chosen and its values were recorded for the safety analysis. Figure 8.3 shows the locations of critical members. Since the concentrated load was applied in the mid-span due to its maximum applied force, the locations were symmetric. The resulting bending moment applied on the structure in the numerical model M1 is shown schematically in the Figure 8.4.
Assessment and Rehabilitation of the Bôco Bridge

Figure 8.3 – Critical members for the safety analysis

Figure 8.4 – Schematic representation of the applied bending moment in model M1
8.3.2 Safety Verification of the Retrofitted Critical Members

The safety verification was performed using the model M1 because the resulting stresses were much higher than the model M2, and consequently more conservative ones. The maximum applied moments were shown in the Table 8.1. It was observed that the rehabilitated Bôco Bridge has extremely high capacity when compared to the bending moment applied to the structure using modified Eurocode. If the unmodified loading pattern prescribed in the Eurocode EN 1991-2:2003 were used, it will result in significantly higher external moment. This gives an interesting hint that the rehabilitated Bôco Bridge can be used for higher traffic loading. This will require a more detailed and sophisticated analysis in terms of the load carrying capacity of the bridge as well as more detailed safety verification. The loading combination used for the analysis includes the permanent loads (dead load and the weight of the deck pavement), which were multiplied by a factor of 1.35 and the live loads (concentrated axle load and distributed load), which were multiplied by a factor of 1.5.

<table>
<thead>
<tr>
<th>Structural Member</th>
<th>Action (design values)</th>
<th>Resistance (Design values)</th>
<th>Safety Verification</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Arch at abutment</td>
<td>164.5 kN·m</td>
<td>368.8 kN·m</td>
<td>Verified</td>
</tr>
<tr>
<td>b) Arch at mid-span</td>
<td>167.5 kN·m</td>
<td>543.9 kN·m</td>
<td>Verified</td>
</tr>
<tr>
<td>c) Column</td>
<td>290.3 kN, 191.1 kN·m, 7.18 kN·m</td>
<td>258.5 kN·m</td>
<td>Verified *</td>
</tr>
<tr>
<td>d) Intermediate long. girder</td>
<td>97.43 kN·m</td>
<td>102.3 kN·m</td>
<td>Verified</td>
</tr>
<tr>
<td>e) Lateral long. girder</td>
<td>284.0 kN·m</td>
<td>376.1 kN·m</td>
<td>Verified *</td>
</tr>
<tr>
<td>f) Transverse girder</td>
<td>64.81 kN·m</td>
<td>90.93 kN·m</td>
<td>Verified</td>
</tr>
</tbody>
</table>

(* the original layout of the reinforcement bars were modified to increase the moment capacity)

For the safety analysis of the column, the maximum axial load (290.3 kN) was used along with two maximum principal bending moments (191.13 kN·m and 7.18 kN·m). It was found that the area of steel was not sufficient. Therefore, the original reinforcement of two steel bars of 16 mm diameters were replaced with four steel bars of 25 mm diameters. This resulted in increase of capacity of the column and the safety verification was satisfied. Similarly, the steel reinforcements in the lateral longitudinal girder were not sufficient. The
original cross-section of the girder does not contain outer reinforcement as found in other structural members. Therefore, four steel bars of 25 mm diameter were added to the outer layer of concrete to increase the moment capacity of the lateral longitudinal girder. Finally, it should be referred that for the present analysis of the selected column the second order effects were neglected.

Figure 8.5 shows the results obtained from the CSAnalysis. The reinforcement steel used for the analysis was unified and placed at the correct distance from the horizontal centerline. The area of the steel was added and combined at one location in the center. This was done to simplify the analysis because it does not affect the analysis. To replicate the rehabilitated structural members, the original steel in the analysis used yield strength of 362 MPa. The rehabilitated steel used S400-B steel. Therefore, the yield strength of the outer steel used yield strength of 400 MPa. The yield strength of the steel was divided by the safety factor of 1.15 and resulted in 347.8 MPa and 314.8 MPa for the outer and inner steel bars respectively.

\[
M_x = 368.8 \text{ kN} \cdot \text{m} \quad M_x = 543.9 \text{ kN} \cdot \text{m} \quad M_x = 258.5 \text{ kN} \cdot \text{m}
\]

(a) (b) (c)

\[
M_x = 102.3 \text{ kN} \cdot \text{m} \quad M_x = 376.1 \text{ kN} \cdot \text{m} \quad M_x = 90.93 \text{ kN} \cdot \text{m}
\]

(d) (e) (f)

Figure 8.5 – Flexural capacity of each structural members using CSAnalysis (design values).

(refer to Table 8.1 for legends)
Chapter 9: Conclusion

9.1 Main Conclusion

The present work is a continuation of assessment of the Bôco Bridge, considered to be the oldest concrete bridge in use in Portugal. The preliminary studies of the bridge included historical, geometrical survey, physical and chemical characterization of concrete and steel. The assessment was continued with the damage survey and mapping of the bridge. The main damage found on the structure was spalling of concrete, corrosion of reinforcement bars and biological growth. The causes of the damage were poor quality of concrete used for the rehabilitation work, high moisture content in the area, and faulty drainage system.

The steel reinforcements were studied using ground-penetrating radar. It was found that the layout of reinforcement bars shown in the historical construction drawings was inaccurate
and unreliable. Another important discovery made from the GPR scans was that the workmanship of rehabilitation project of the year 1962 was of very poor quality. The concrete cover was insufficient and the geometries of reinforcement bars were altered. A brief crack interpretation along with the crack measurement using microscopic imaging was presented. This can be used in future analysis to study if the cracks were active or not.

It was followed by an experimental dynamic assessment using ambient vibration tests. Twelve mode shapes, with corresponding frequencies and damping coefficients were estimated using Enhanced Frequency Domain Decomposition (EFDD). This method was chosen due to its simplicity, user friendliness and fast post-processing time. The frequencies were well spaced and ranged from 3.8 Hz to 66.41 Hz. The average damping coefficients obtained was 2.23% and a nonlinear relation was observed between frequencies and damping coefficients. The mode shapes of the bridge showed clear signs of damage and imperfections in the structure.

The in-situ load test was performed on the Bôco Bridge using a new system to measure the vertical displacement of the deck. The result showed maximum vertical displacement of two sides of the mid-span of the bridge to be 0.50 mm and 0.43 mm. The displacement value obtained from the test was consistent with the predicted behavior of the bridge. Although four different test setups were performed at six locations, only two sensors form one test setup gave accurate result. The possible cause of the faulty results were dead weight being insufficient, lack of proper technicians to install the system, lack of proper tools and problems associated with the assembling of the measuring system.

A finite element model was developed and calibrated using the results obtained from the in-situ load test. Two different numerical models were used for the analysis. The results obtained from the numerical models produced very close results when compared against the results obtained from the in-situ load test. The numerical model was considered acceptable for the safety analysis.

Finally, a proposal was prepared for the rehabilitation of the Bôco Bridge to guarantee its safeguard in future by upgrading its load carrying capacity. The retrofitted structure was verified by performing a safety analysis on critical members using European regulation, i.e. EN 1992-1-1:2004. The proposed rehabilitated structural members used the same cross-section geometry as the original layout. The outer layer of concrete was removed and replaced with C35/45 concrete. The corroded steel bars were replaced with S400 steel bars. Apart from the column and the lateral longitudinal girder, other structural members satisfied the safety analysis without any modification. The column and lateral longitudinal girder did
not satisfy the safety verification and required additional reinforcement bars. The safety analysis of the modified structural members was verified thereafter.

With regards to the objectives of the thesis set forth in the beginning of the thesis, all the objectives were generally fulfilled. There were some areas that needed some improvisations and further analysis, which are presented below.

9.2 Opportunities for Further Research

During the thesis, several areas demanded further analysis and considerations. The main part of this thesis was the in-situ tests, which is very challenging when performed within limited time and resources. One future possibility in the study of Bôco Bridge is to use other non-destructive tests such as radar detector using the principle of magnetic induction to assess the correct layout of the reinforcement bars. The rate of corrosion of the reinforcement bars and the location of corroded bars must be studied. Obtaining detailed information about the geometry and the current state of the reinforcement bars is extremely crucial for the rehabilitation of the Bôco Bridge.

Due to limited time, different method such as Stochastic Subspace Identification (SSI) method was not used to compare and verify the results. This can be carried out in future as an extension of the current study. Other modal identification methods can be used to compare the results and obtain modal assurance criterion (MAC). To determine more accurate damping coefficient, one possibility is to excite the bridge externally.

It is highly recommended that the in-situ load test be repeated with all the possible causes mentioned addressed. In order to obtain good results, proper tools and technical resources are needed. The measuring system needs to be checked on site before carrying the test. The new results can then be used to verify the numerical model. The numerical model will require further optimization by using the results obtained from the experimental dynamic assessment, which was not performed in this thesis due to the complexity of the structure and limited time.

The numerical model can be fine-tuned by considering addition of springs and improvising the connection between structural members. The additional information obtained from the complementary non-destructive evaluation tests can be used to improvise the numerical model.
Finally, the proposed rehabilitation work can be improved by specifying the materials and specifications used for the repair work. An in-depth damage study and mapping is needed for the rehabilitation work. Detailed construction drawings can be prepared to accurately illustrate where the concrete removal is required and the temporary support system needed for the repair work. The safety analysis can be repeated using the information about the location of reinforcement bars obtained from the non-destructive tests such as radar detector. Additional tests can be performed to correctly determine the layout of the reinforcement bars to be used for the safety analysis.
References


