



Faculty of Engineering

**Bolted Connection Design Recommendation of *Nyato* Hardwood for
Retrofitting the Unreinforced Masonry Buildings**

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Bolted Connection Design Recommendation of *Nyato* Hardwood for
Retrofitting the Unreinforced Masonry Buildings

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A thesis submitted

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DECLARATION

I declare that the work in this thesis was carried out in accordance with the regulations of Universiti Malaysia Sarawak. Except where due acknowledgements have been made, the work is that of the author alone. The thesis has not been accepted for any degree and is not concurrently submitted in candidature of any other degree.



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ABSTRACT

Referring to the design code of the Malaysian Timber Standard of MS544: Part 5: 2001, the design of timber bolted connections only considers the ductile failure mode. This is opposed to the agreed principle set by the international timber engineering community of timber design standards. The principle states that the design code should be able to identify both ductile and brittle failures in order to determine the capacity of bolted connection strength. Therefore, there is a need to validate the existing Malaysian Timber Code in order to produce an optimised retrofit design of the wall-diaphragm connections for an unreinforced masonry building in Malaysia, especially the bolted connection on the floor joists or roof rafters. The occurrence of local earthquakes in Malaysia has increased in recent years, thus it is essential to provide sufficient lateral resistance to the unreinforced masonry building due to the absence of connections between the masonry brick walls and timber floor / roof diaphragms in the building as-built constructions. There are many unreinforced masonry buildings in Malaysia, of which, if subjected to earthquake, can cause major devastation and fatality, as most of these buildings are situated in main towns and business district. Because of the lack of bolted connection experimental data on wood species that commonly used to construct roof or floor diaphragms of unreinforced masonry buildings, a series of bolted connection test was conducted. By using the results obtained from this study, the effectiveness of the current timber code was validated and the accuracy of the other design equations such as the European Yield Model (EYM) and the Row Shear Model (RSM) was evaluated. From this experimental study, a calibration factor specifically for *Nyatoh* hardwood was identified to optimize the bolted connection design using the RSM for the brittle failure mode. To optimize the bolted timber connection design using the EYM for the ductile failure mode, a specific embedding strength value for *Nyatoh* hardwood was also provided. It was found that the design capacity given by

the MS544-5 was too conservative when compared to the EYM and RSM for ductile and brittle failure modes, respectively. From the analyses of results, the 5th percentile bolted connection strength prediction of MS544-5 against the 5th percentile experimental results yielded a percentage accuracy of 27% - 39% for the connections that fail in the ductile mode. A percentage accuracy of 75% - 107% was determined using EYM for the ductile failure mode. In the brittle failure mode, the RSM yielded a percentage accuracy of 50% - 82%, while the MS544-5 generated 32% - 63%.

Keywords: Retrofit design, bolted connection, European Yield Model (EYM), Row Shear Model (RSM), Malaysian Standard (MS544)

***Cadangan Rekabentuk Sambungan Bolt ke Kayu Nyatoh untuk Mengukuhkan
Bangunan Batu yang Tidak Diperkukuh***

ABSTRAK

Merujuk kepada kod rekabentuk Piawaian Kayu Malaysia MS544: Bahagian 5, rekabentuk sambungan bolt kayu hanya mengambil kira mod kegagalan mulur. Ini bertentangan dengan prinsip yang dipersetujui dan ditetapkan oleh komuniti kejuruteraan kayu antarabangsa bagi piawaian rekabentuk kayu. Prinsipnya menyatakan bahawa kod rekabentuk harus dapat mengenal pasti kedua-dua kegagalan mulur dan rapuh untuk menentukan kapasiti kekuatan sambungan bolt. Oleh itu, terdapat keperluan untuk mengesahkan Kod Kayu Malaysia sedia ada bagi menghasilkan rekabentuk pengubahsuaian yang optimum bagi sambungan dinding-diafragma untuk bangunan batu tidak bertetulang di Malaysia, terutamanya sambungan bolt pada gelegar lantai atau kasau bumbung. Kejadian gempa bumi di Malaysia telah meningkat dalam beberapa tahun kebelakangan ini, maka adalah penting untuk menyediakan rintangan sisi yang mencukupi kepada bangunan batu yang tidak bertetulang kerana ketiadaan sambungan antara dinding batu bata dan diafragma lantai kayu / bumbung dalam binaan asal bangunan. Terdapat banyak bangunan batu yang tidak bertetulang di Malaysia, yang mana jika terkena gempa bumi, boleh menyebabkan kemusnahan besar dan kematian, kerana kebanyakan bangunan ini terletak di bandar utama dan daerah perniagaan. Oleh kerana kekurangan data eksperimen sambungan bolt pada spesies kayu yang biasa digunakan untuk membina diafragma bumbung atau lantai bangunan batu tidak bertetulang, satu siri ujian sambungan bolt telah dijalankan. Dengan menggunakan keputusan yang diperolehi daripada kajian ini, keberkesanan kod kayu semasa telah disahkan dan ketepatan persamaan rekabentuk lain seperti European Yield Model (EYM) dan Row Shear Model (RSM) dinilai. Daripada kajian eksperimen ini, faktor

penentuan khusus untuk kayu keras Nyatoh telah dikenalpasti untuk mengoptimalkan rekabentuk sambungan bolt menggunakan RSM untuk mod kegagalan rapuh. Untuk mengoptimalkan rekabentuk sambungan bolt kayu menggunakan EYM untuk mod kegagalan mulur, nilai kekuatan benam khusus untuk kayu keras Nyatoh turut disediakan. Didapati bahawa kapasiti rekabentuk yang diberikan oleh MS544-5 adalah terlalu konservatif jika dibandingkan dengan EYM dan RSM untuk mod kegagalan mulur dan rapuh. Daripada analisis keputusan, ramalan kekuatan sambungan bolt persentil ke-5 MS544-5 terhadap keputusan eksperimen persentil ke-5 menghasilkan ketepatan peratusan 27% - 39% untuk sambungan yang gagal dalam mod mulur. Peratusan ketepatan 75% - 107% ditentukan menggunakan EYM untuk mod kegagalan mulur. Dalam mod kegagalan rapuh, RSM menghasilkan ketepatan peratusan 50% - 82%, manakala MS544-5 menjana 32% - 63%.

Kata kunci: *Rekabentuk retrofit, sambungan bolt, European Yield Model (EYM), Row Shear Model (RSM) dan Malaysia Standard (MS544)*

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CHAPTER 1

INTRODUCTION

1.1 Background

Unreinforced masonry (URM) buildings can be found throughout the world. It is often defined as historic buildings in the general public's eye because this building system was constructed prior to the late 1800s. They were so popular and widely used before the reinforced concrete and steel structures building systems taking place in the late 1800s or early 1900s. Due to the unique nature of the masonry buildings, many relevant authorities are keen to preserve it due to its aesthetic look, for examples the famous A Famosa remnants in Malacca and The Old Courthouse in Kuching, Sarawak. From 1992 to 1993, the Heritage Trust of Malaysia, in collaborations with the National Museum and Local Government Ministry and Faculty of Built Environment, Universiti Teknologi Malaysia, conducted a study on the heritage buildings and have estimated that there are around 39,000 of historic buildings are worth of preservations and protections (Idid, 1995).

Barrantes (2012) stated that in hundreds of years leading up to the modern days, URM buildings were constructed as it possessed the nature of high durability, fire resistance and isolation properties. This further explains the reason why most URM buildings still stand majestically like URM buildings in Kuching, Sarawak, albeit it has undergone through many harsh weather conditions that mother nature has to offer. This URM building will continue to stand tall, provided they are not subjected to lateral or horizontal external forces. According to the Federal Emergency Management Agency (FEMA, 2009), the URM building is typically comprised of masonry brick walls with no steel reinforcement bars to act as the vertical structural component, and roof / floor diaphragms that made from wood

to serve the building as the horizontal structural element. Figure 1.1 shows the example of unreinforced masonry buildings that can be found in the Kuching City Centre. Typically, in their as built, the buildings provide no connections between the vertical and horizontal structural components, whereas the roof / floor diaphragms are found to be seated on the building perimeter walls. This significantly causes the buildings to be very vulnerable in resisting the lateral or horizontal external forces, which is predominantly induced by earthquakes. Nowadays, some of the buildings found to have wall-diaphragm connections that are found to be part of the strengthening efforts prior to lessons learned from the major earthquake events. According to Abdul Karim (2012), the major parts of the wall-diaphragm connections are divided into: (i) wall anchorages, either a through-bolt rod connected with an external wall plate, or a dowel rod drilled into the wall; and (ii) diaphragm connections, bolted connections to the timber floor joist or to the timber roof rafters.



Figure 1.1: The Old Courthouse in Kuching

The terminology of the URM buildings (FEMA, 2009) is mainly referring to the lack of extra reinforcement systems that are commonly used in modern construction practice like the reinforced concrete design. Pineda et al. (2011) stated that an ancient URM building is likely to suffer structural failures, which could result in global or local collapse mechanisms

if these buildings are situated in a seismically active area. New Zealand Society for Earthquake Engineering (NZSEE, 2006) mentioned about the susceptibility of the construction of URM buildings to earthquake shaking due to its enormous mass, lacking in integrity between elements and cannot be easily deformed. The URM building weakest point is often referred to the inadequate restraint at the building storey-heights, which is the absence of wall-diaphragm connections. Major failures involving the inadequacy of the connections are the tearing out failure of timber floor or roof joists, the out-of-plane failure of masonry brick walls, and the global collapse of the floor diaphragms (Bausabah and Bruneau, 1992). Figure 1.2 shows the example of typical failure of roof joists off their wall supports in Santa Cruz, whereas Figure 1.3 shows the example of out-of-plane failure of URM walls. The tear-out failure of the timber joists directly signals a warning that the bolted connection design provides insufficient resistance, which is the main concern of the present study.



Figure 1.2: Typical Failure of Slipped Roof Joists Off Their Wall Supports (Bruneau, 1994)



Figure 1.3: Out-Of-Plane Failure of URM Top-Storey Wall at San Francisco, Loma Prieta Earthquake (Bruneau, 1994)

Russell (2010) stated that, the masonry walls that are loaded in perpendicular direction to the wall surface is termed as out-of-plane walls. In the case of out-of-plane failures, the masonry walls can collapse either by falling outward or inward from the building perimeter. Both walls collapse can significantly claim the lives of the building occupants and even the nearby pedestrians. The greater the height of the wall, the greater the flexural stress (Bruneau, 1994). As shown in Figure 1.4, the masonry walls act as a tall cantilever wall if there are no connections between the brick walls and the timber floor diaphragms when subjected to lateral loadings (Abdul Karim, 2012). From this observation, a retrofitting of the buildings by implementing the wall-diaphragm connections are very crucial to ensure the wall to be restrained at every building floor level in order to reduce the flexural stress to the walls. Thus, this present study was primarily initiated to provide an optimised design that

focuses on the diaphragm connections, which is the bolted joint to the timber diaphragm structural members such as roof rafters and floor joists.

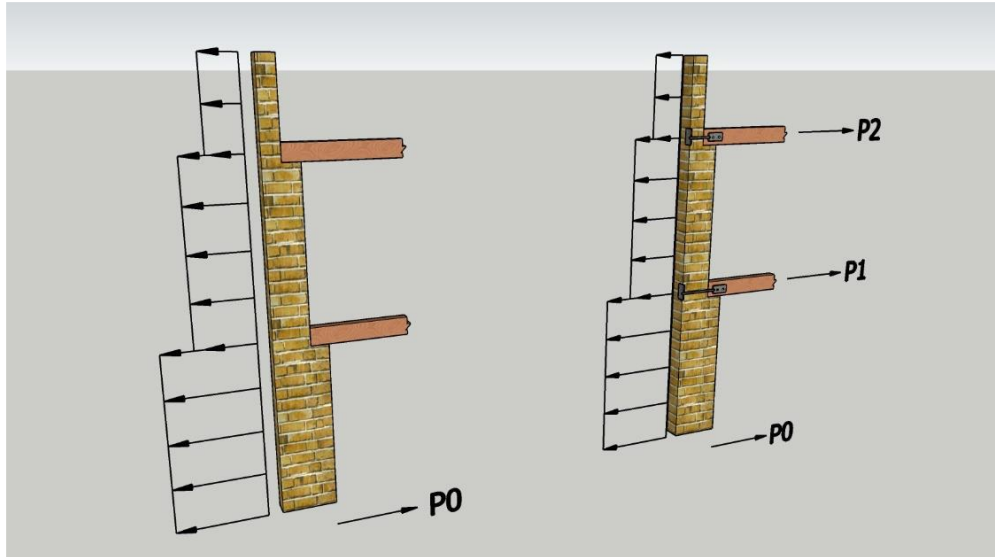


Figure 1.4: Behaviour of URM Walls Without Connections (left) and With Connections (right) (Abdul Karim, 2012)

In Malaysia, to design the timber bolted connection part, the MS544-5 (2001) can be used as a reference by the designers. The timber standard is developed based on the ductile failure mode when estimating the bolted connection strength. The brittle mode of failure is only considered with the use of k_{17} factor less than 1 for bolted connection that comprise of multiple bolts not less than 4 bolts per connection end as tabulated in Table 15 of MS544-5 (2001). Many published works (Abdul Karim et al., 2012; Abdul Karim et al., 2013; Abdul Karim et al., 2018; Abdul Karim et al., 2021; Abdul Karim et al., 2022) have identified that the geometrical parameters of the end distance, e , and spacing between bolts, s_b , govern the failure mode of the bolted connections as shown in Figure 1.5. From the studies by Abdul Karim et al. (2018), it was found out that the design strength provided by the existing Malaysian timber code was far too conservative when compared to the actual connection strength gathered from the laboratory data. The effectiveness of the design

strength provided by the Malaysian timber standard was evaluated on the local hardwood of *Meraka*. The study disclosed a comparison between the MS544-5 code and the European Yield Model (EYM) versus the experimental results in order to assess their strength prediction accuracy. The EYM provides a better strength prediction with an average of 81% accuracy, while the MS544-5 only shows 38% efficacy. Thus, the study recommends that the design of bolted connection using EYM is more suitable for the retrofitting work on the URM buildings. Another study on brittle failure by Abdul Karim et al. (2021), also on *Meraka* hardwood, shows the same flaws on MS544-5, where the average accuracy of 79% was obtained via Row Shear Model (RSM) equation against 41% of MS544 standard. Thus, this present study was initiated to investigate the effectiveness of existing design equations on the *Nyato* hardwood due to the unavailable experimental data on the bolted connection of the selected local hardwood.



Figure 1.5: Illustration of Timber Bolted Connection Configuration Indicating The End Distance (e), and Bolt Spacing (s_b)

1.2 Issues with Unreinforced Masonry (URM) Buildings

An unreinforced masonry building is without a doubt recognized as the type of construction that is most vulnerable to earthquake strikes (Bruneau, 1994). It is termed ‘unreinforced’ due to the bricks, or the walls, do not have any support or anything to hold the building structure firmer. If subjected to lateral force such as the earthquake load, the URM buildings are prone to disconnect from other building components and collapse, which

then potentially harming the nearby pedestrians in the process. Poor connections between masonry walls and the wood diaphragms are also one of the main causes of walls overturning and collapsed of floors during an earthquake (Moreira et al., 2014). Other characteristics of URM buildings defined by other sources are given below.

- i. URM common types of construction were solid cavity walls (NZSEE, 2017). Industrial buildings and buildings on the outskirts of town mainly used solid walls besides partition walls and walls, either not visible or in the lower storeys. Cavity wall meanwhile, were used as control moisture ingress in a building. New Zealand Society for Earthquake Engineering, NZSEE (2017), stated that URM buildings have a seismic capacity that is difficult to quantify and may result in margins against a collapse that are small for the following reasons:
 - a. URM buildings may have limitations of non-linear deformation capability which are directly depending on their configurations, material characteristics, vertical stress, and potential failure modes.
 - b. Relying on friction and overburden pressure from load carried and wall weights.
 - c. Varieties of material properties.
 - d. Degradation of strength and stiffness with every additional cycle of greater displacements of inelastic response to shaking.
- ii. A report by Senaldi et al. (2012) portrayed the URM buildings as a rigid structure with a low capacity to flex when subjected to high accelerations of lateral force causes by an earthquake. These buildings were originally designed only to resist

gravity and wind loads. It was also reported that most of this building consists of one, two or three storey brick buildings that were built for commercial purposes. Stone also accounts as major materials used to construct URM buildings besides brick and is called stone masonry buildings. In such building, there are other materials being incorporated as essential building components, such as timber, lime mortar and metal. However, such materials are subjected to deterioration due to aging. Most of the time, these structural elements are poorly interconnected in every other element of the buildings and may easily detach.

- iii. Russell and Ingham (2010) reported that most of the New Zealand's URM buildings were built between the 1870s and 1940s, which support the claims that URM buildings were mostly built prior to mid-20th centuries.
- iv. Russell (2010) described URM buildings are characterized based on the number of floors and typically reach three storeys high in New Zealand. In general, URM buildings have regular plan shapes, and the external walls may act as the bracing to the lateral force applied other than earthquakes.
- v. According to the Federal Emergency Management Agency, FEMA (2009), an URM building is a masonry building that does not have a grid of steel reinforcing bars embedded within the walls and only constructed using only masonry materials. In the United States, this building is normally constructed using brick walls and wood-frame for floors and roof. The exterior of the building shows that the windows have short span, and the walls are thick. The masonry walls of the building are known as bearing walls since it supports loading from the floor and/or roof frames. Bearing walls also have great performance in resisting gravity

and provides stability for the building. However, it has consistently performed very poorly when subjected to earthquake. The most common failure in most URM buildings is an outward collapse of the exterior walls caused by the loss of lateral supports due to separation of the walls from the floor / roof diaphragms. Figure 1.6 shows the detail of the structural elements in URM buildings.

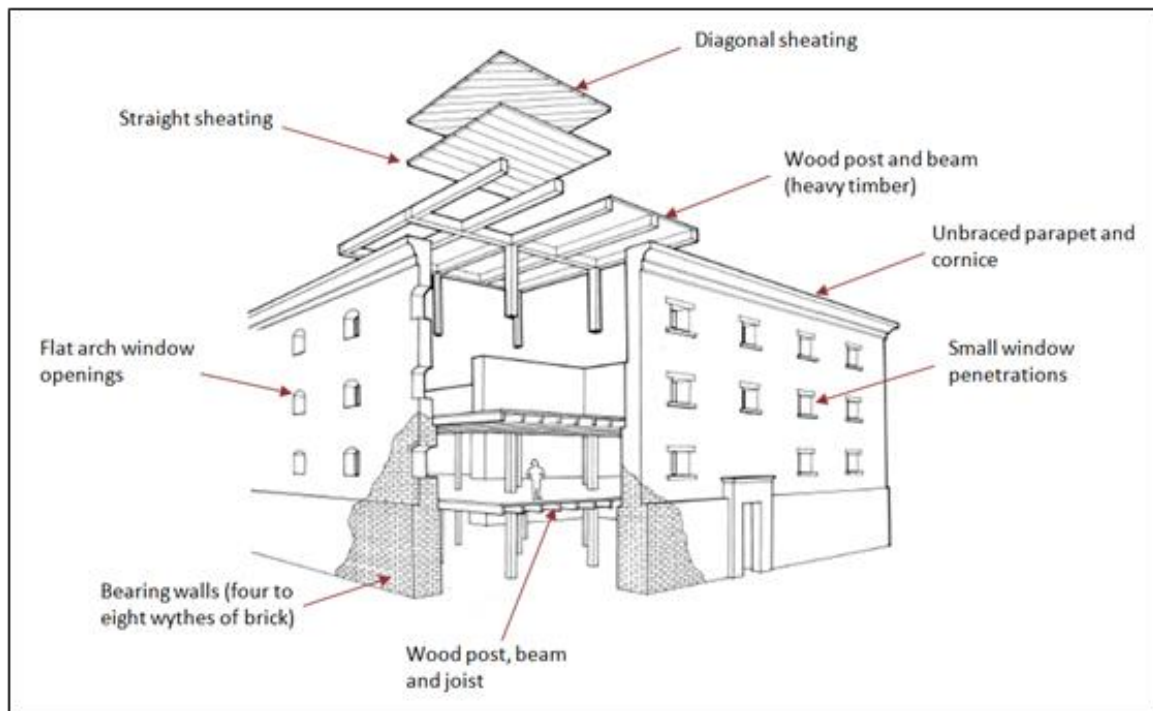


Figure 1.6: Unreinforced Masonry Buildings (FEMA, 2015)

- vi. Page (1996) stated that traditional masonry structures relied on their massiveness of construction to achieve stability and were evenly proportioned to avoid forming tensile stress. URM buildings have very poor seismic performance, as they are very heavy with its materials having brittleness in nature and low tensile strength. Having brittleness properties coupled with low tensile strength leads to having no to little ductility. In terms of seismic design, the freestanding elements must have a support to resist it from failing laterally.

- vii. Boussabah and Bruneau (1992) described URM buildings as a non-ductile type of constructions, most vulnerable to earthquakes, putting these old buildings to be at greater seismic risk compared to modern day buildings.

1.3 Problem Statement

Due to the fact that the load-bearing masonry walls of URM buildings do not have a lateral restrained mechanism, this makes them very vulnerable to total collapse when subjected to lateral forces induced by earthquakes (NZSEE, 1995; FEMA, 2006). The buildings become unsafe for both occupants and pedestrians as well as the surrounding public properties. Retrofit measures are obviously required by means of introducing the wall-diaphragm connections to restrain the masonry walls to the timber diaphragms at each floor level of the building (NZSEE, 1995; FEMA, 2006).

With reference to MS544-5 (2001), the timber code is not considering the brittle failure mode of bolted connections. Previous studies (Abdul Karim et al., 2018; Abdul Karim et al., 2021; Abdul Karim et al., 2022) have proven that the timber standard is too conservative in predicting the design strength when compared with other design equations such as the EYM and the RSM. In regard to this, further assessments on the timber standard for other local hardwoods are of vital importance.

Due to the lack of data for validating the design on bolted connections of local hardwoods that commonly used in the construction of timber diaphragm of URM buildings, it motivates this present experimental study to be initiated for the *Nyatoh* hardwood. The data obtained from this present study not only can be utilised to assess the accuracy of the existing design equation for the *Nyatoh* hardwood, but also can significantly support the development of a hardwood database. Consequently, the database can be adopted for the

design strength optimisation purposes, which can benefit the wall-diaphragm design for retrofitting works of the URM buildings. Forest Department Sarawak (FDS, 1999) stated that the *Nyatoh* hardwood is suitable to be used as structural components such as floor joists and roof rafters, thus, a reliable wood selection was attained in this present study.

1.4 Objectives of Study

The following objectives are outlined as follows in order to ensure that the main purpose of this present experimental work can be achieved.

- i. To verify the genus of the selected local hardwood specimen.
- ii. To evaluate the bolted connection strength and the timber properties of the verified local hardwood specimen via embedding strength test and moisture content and density test.
- iii. To recommend the suitable design equations of the timber bolted connections for the verified local hardwood specimen.

1.5 Scope of Study

The main purpose of this study was to assess the strength of the bolted connection on *Nyatoh* hardwood so that an optimised retrofit design of URM buildings can be provided. The present study was initiated, as there are very limited experimental data bolted connections to validate the existing design equations on local hardwood. This study was also conducted to investigate the possible failure modes of bolted connections on *Nyatoh* hardwood. Because of the Malaysian Timber Standard considers only ductile failure, its effectiveness in designing a bolted-connection joint needs to be verified. The need for verification is due to many published works have reported brittle failures in timber bolted joints. Thus, the implementation of timber bolted connection tests and timber material tests

in this present study are very crucial, so that the laboratory data can be utilised to produce an optimised design strength of bolted connections on local hardwoods for URM building retrofits. Hypothetically, the predicted strength of the timber bolted connection of the *Nyato* hardwood can be optimised by utilizing the essential design parameters such as the embedding strength for EYM and the calibration factor for RSM from both laboratory works of the bolted connection tests and the determination of the timber properties, respectively.

It should be noted that the wall-diaphragm connections are divided into two parts, i.e., wall anchorages and diaphragm connections (bolted connections to the timber floor joist or roof rafters). The reader shall be noted that this study only focuses on the timber bolted connection, by assuming that the wall anchor does not fail before the timber bolted connection. The study is also concentrating on a single row of timber bolted connections with up to maximum 3 numbers of bolted connections. The timber bolted connection test was conducted by applying the tensile strength in the form of double shear timber bolted connection with varying bolted connection configuration. There were eighteen (18) different types of bolted configurations being tested. All the above-mentioned tests were carried to fulfil the above objectives are the timber bolted connection strength test, embedding strength test and moisture content and density test.

1.6 Significance of Study

The significance of this study was not only the author seeks to validate the existing design equations using a local hardwood from another strength class, but also a verification of the hardwood wood genus was also conducted in this study. This shall eliminate the elements of doubt on the hardwood specimen used for the experimental investigations. Thus, a reliable strength result of bolted connections on the verified hardwood specimen can be

obtained. Besides that, the data obtained from this study can be utilised to support the development of a database that can significantly assist the engineers in timber bolted connection design.

As mentioned in Section 1.5, the wall-diaphragm connections are divided into two parts, the wall anchorages, and the diaphragm connections. Because of the focus of this present study was on the latter part, the reader shall take note that the bolted joint governs the resistance of the wall-diaphragm connection by assuming the masonry brick wall and timber diaphragm to be intact. Thus, the main focus of this present research work was to study the strength and the possible failure mode of the *Nyatoh* hardwood of bolted connections. From thorough analyses on the test results obtained not only an optimised design can be produced, but also a detailing on the geometrical configurations of the bolted connection can be acquired. By understanding the geometrical configuration effects on both the strength and failure modes of bolted connections, the designer can ensure that the most economical timber bolted connections can be designed.

1.7 Thesis Outline

This thesis comprises of five chapters, whereas each chapter includes subtopics that provide the details with regards to support the completion of this present study.

Chapter 1 provides the background of research to highlight the motivation to initiate the present study. A brief on the history and as-built characteristics of URM buildings was outlined. The main aim, hypothesis, significance, limitations, and objectives of this research study were specified in the scope of the present study of Chapter 1.

Chapter 2 reviews the relevant literatures incorporating the as-built characteristics of URM buildings, followed by the statistics of the buildings in Malaysia to show their occurrences in most business district areas. The records of past seismic events are included along with the performance of URM buildings in past earthquakes to show the drawbacks of the URM building systems, especially on the wall-diaphragm connection related failures. Problem statements found in published literatures were included to identify the research gaps to indicate the importance of this present work.

Chapter 3 presents the details of methodology in order to ensure the achievement of objectives. The fact that the present study was conducted with a verification on the hardwood up to the genus level, this chapter also provides the identification work details, whereas the anatomy descriptions of the hardwood end-grain section using a microscope are summarised to get the reader to be familiar with the important terms of the wood anatomy. The experimental setup and procedures of the bolted connection tests on *Nyatoh* hardwood with eighteen groups of bolted configurations are described. To determine the important parameters for the bolted connection strength predictions using European Yield Model (EYM) and Row Shear Model (RSM), the details of the embedding strength test, and the moisture content and density test on the hardwood specimen are included.

Chapter 4 reports the results of analyses and discusses the experimental results obtained from the bolted connection tests, the embedding strength tests, and the moisture content and density tests conducted. The validation of the effectiveness of the MS544-5, EYM and RSM in predicting the bolted connection strength was discussed in detail. These include the determination of parameters of the embedding strength value and the calibration

factor for optimising the bolted connection strength using the EYM and the RSM, respectively.

Chapter 5 concludes the research findings and highlights some useful recommendations for future related works.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

This chapter describes the flaws of unreinforced masonry (URM) buildings gathered from the published literatures to introduce the as-built characteristics of the building. The highlighted flaws in relations to the timber diaphragm connections, that motivates this research to explore the possibility of retrofitting the URM buildings using fewer sets of existing design equations. The statistics of URM in Malaysia and records of past seismic event that occurred either outside or inside Malaysia are highlighted here. The prior earthquake records and the performance of the URM buildings in the previous seismic events are discussed to point out the main weakness of the URM building systems. Several research studies related to wall-diaphragm connections, specifically on the timber bolted connection part, are summarised to determine the current research findings on local hardwoods. The details of existing equations to design the timber bolted connections are outlined to understand the basis of the theory used in the strength predictions of the bolted joints.

2.2 Unreinforced Masonry Buildings in Malaysia

Malaysia contains thousands of historic buildings that were once built during the invasions by the Portuguese, Dutch, Japanese and Englishmen spanning hundreds of years from the fallen of the Malaccan kingdom. A study conducted in 1992 and 1993 by the Heritage Trust of Malaysia in conjunction with the National Museum, the Ministry of Housing and Local Government, and Faculty of Built Environment of Universiti Teknologi Malaysia (UTM), reveals that there are nearly 39,000 historic buildings that were built between 1800 and 1948 (Idid, 1995). It also stated that buildings built during these periods

are classified as pre-war buildings. Table 2.1 below shows the statistical figure of historical buildings that can be found in Malaysia.

Some of the historical buildings in Malaysia are still preserved by means of using the buildings instead of abandoning it to age with time. These buildings are used for other purposes than its intended use, such as turning it into a museum, commercial buildings, mosque, churches and other important public service buildings. Many of the unreinforced masonry buildings in this country were built during the invasion of Europeans and British around late the 17th to the middle 20th centuries. Rocchi (2015) stated that during these eras, the construction of URM buildings is predominantly used high quality hardwoods, such as *Belian, Meraka, Cengal* and *Selangan Batu*.

In Sarawak, this URM building can be easily found in the central business district of Kuching. The buildings can be seen in the Kuching Waterfront area, as shown in Figure 2.1, where they are mostly double storey shophouses. The top storey is commonly used as residential, or storage and the bottom storey is typically operated as shops. From the author's observations, the building characteristics were found to be similar as described in the previous section. The walls are made from masonry bricks and the floor diaphragms are made from a timber material. From Figure 2.2, it can be seen that a wall anchor is visible on the gable wall of the roof. No wall anchorages were found at the first-floor level.

Table 2.1: The Distribution of ‘Pre-War Buildings in Malaysia (Idid, 1995)

States	Number of Historical Buildings	Percentage of Historical Buildings
Penang	5,057	24.33%
Perak	3,351	16.12%
Johor	2,323	11.18%
Malacca	2,177	10.47%
Kuala Lumpur	1,763	8.48%
Kedah	1,282	6.17%
Selangor	1,166	5.61%
Sarawak	1,010	4.86%
Negeri Sembilan	999	4.81%
Pahang	831	4.00%
Terengganu	420	2.02%
Kelantan	373	1.79%
Perlis	25	0.12%
Sabah	10	0.05%
Total	20,787	100.00%

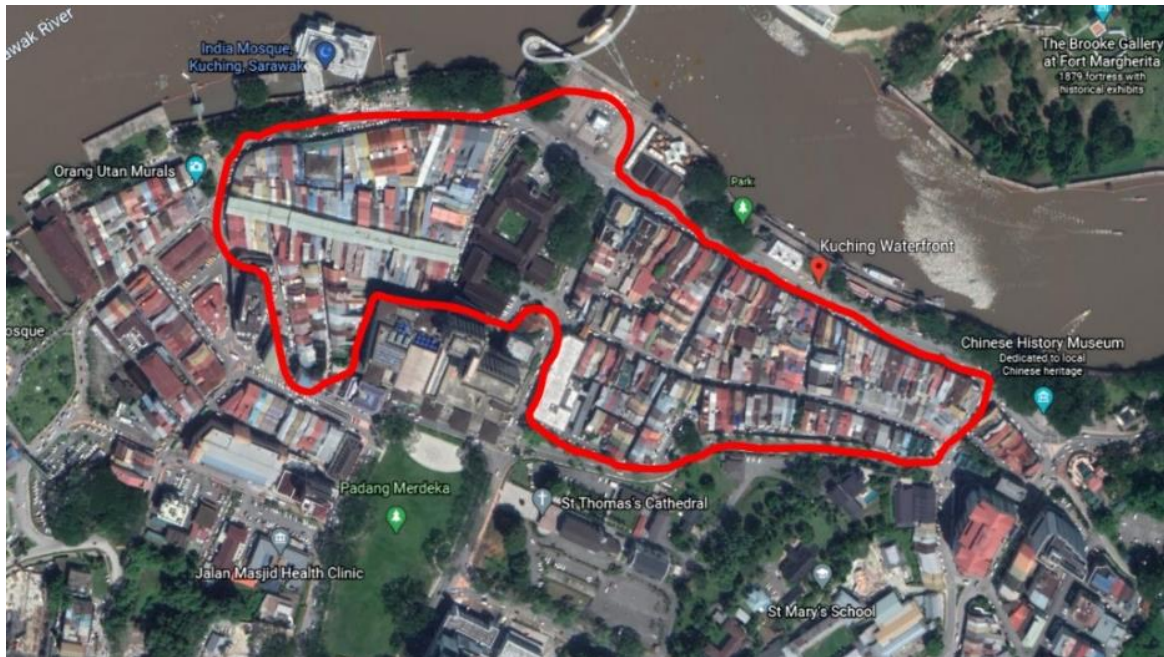


Figure 2.1: Location of URM Buildings in Kuching Waterfront, Sarawak, Malaysia



Figure 2.2: One of the URM Buildings at Kuching Waterfront With Wall Anchorages

2.3 Records of Past Seismic Events Around the World and in Malaysia

Ever since such phenomena hits Sichuan, China in 2008, Port au Prince, Haiti, in 2010 and Christchurch, New Zealand, in 2010 and 2011, these highlights the importance of the mitigation effort to preserve many unreinforced masonry buildings (Frankie T. et al, 2013). Russell and Ingham (2010) stated that the seismic performance of URM buildings depends on its general size and shape, as a small, low-rise building responds in a different way when subjected to seismic forces than a long, row-type, multi storey building. Poor construction quality, workmanship and lack of proper maintenance are the factors that can affect the performance of URM buildings as indicated by Bilgin and Korini (2012).

The records of past earthquakes that give major impact on the structural performance of unreinforced masonry buildings, taken from FEMA (2009), are tabulated in Table 2.2.

Table 2.2: Records of Past Major Earthquakes Giving Impacts to URM Buildings Structural Performance (FEMA, 2009)

Year	Location (Magnitude)
1886	Charleston Earthquake, South Carolina (M 7.7)
1925	Santa Barbara Earthquake, Southern Carolina (M 6.2)
1933	Long Beach Earthquake, Southern Carolina (M 6.3)
1983	Coalinga Earthquake, Central California (M 6.2)
1983	Borah Peak Earthquake, Idaho (M 7.3)
1989	Loma Prieta Earthquake, Northern Carolina (M 7.1)
2001	Nisqually Earthquake, Puget Sound Region, Washington (M 6.8)
2003	San Simeon Earthquake, Central California (M 6.5)

Even though Malaysia is rather fortunate to not experience earthquakes compared to their neighbouring countries such as Indonesia, there is concrete historical evidence that

Malaysia, especially the west side of Malaysia, may not be forever immune from this seismic risk. Marto et al. (2013). It was concluded that the initiation of a local origin earthquake within Peninsular Malaysia is a sign of reactivation of inactive ancient faults caused by the reformation of the Sunda-land as shown in Figure 2.3.

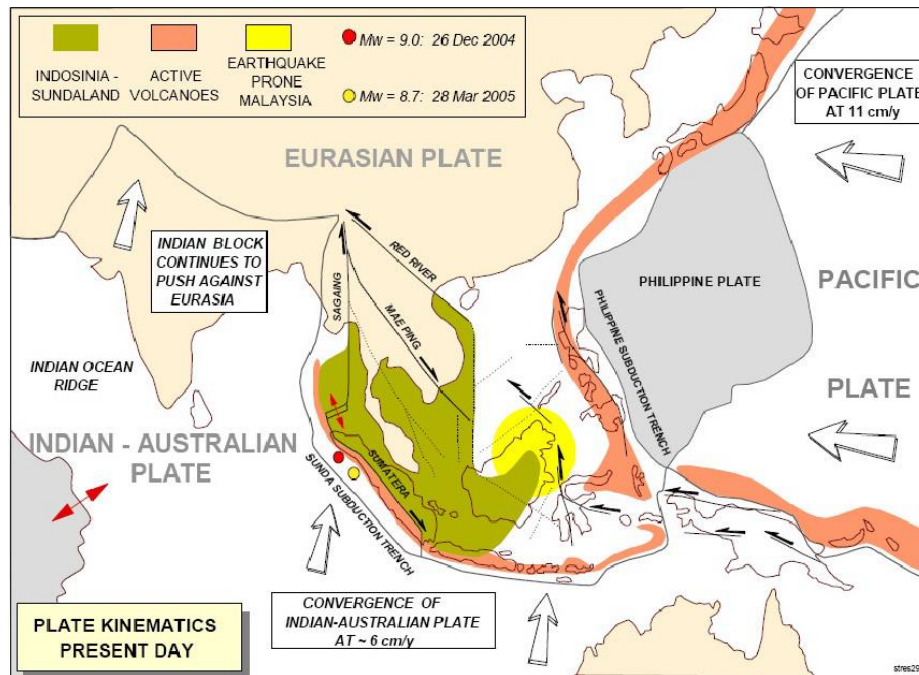


Figure 2.3: Earthquake-Prone Region of Malaysia (Marto et al, 2013)

2.4 Performance of Unreinforced Masonry Buildings in Past Earthquakes

Bruneau (1994) identified several failure modes of unreinforced masonry buildings in the 1989 Loma Prieta earthquake, which can be related to the absence of positive anchorage between URM walls and timber diaphragms, that in turn leads no lateral support (Abdul Karim, 2012). Such lack of lateral support leads to wall act as cantilever over the total building heights. Because of this structural behaviour, an excess of flexural stresses is significant at the wall base. The greater the heights, the greater the risk of building collapsing due to bending. The timber diaphragm structural members, either roof rafters or floor joists,

that are simply socketed into the URM walls without anchor are very vulnerable because the lateral force caused by the earthquakes to the URM walls is not transferred to the timber diaphragms. This renders the timber diaphragm to no functions as structural support or bracing and more of a complimentary aesthetic feature.

Unreinforced masonry buildings are commonly considered as the most seismically prone-to-collapse structure. Hess (2008) reported that those poor performances of the URM buildings to remain in one-piece post-earthquake were due to inherent brittleness, lack of tensile strength and ductility. A post-earthquake that has done the significant damage to URM building shows that it has numerous modes of failure. However, an out-of-plane wall failure was mostly reported in many collapsed of the URM buildings such as those reported by Bruneau (1994). Most of the out-of-plane wall failure was due to the poor connections between structural members, such as masonry walls to timber diaphragms, which is also reported by Moreira et al. (2014).

Published research works and post-earthquake reports (Ingham et al., 2011; Tomaževič et al., 1996; Magenes et al., 2012; FEMA 547, 2006; Dizhur et al., 2018) have been highlighting the importance of connections between the masonry walls and timber diaphragms of the URM buildings. They stated that the use of effective wall-to-diaphragm connections have shown to be significant in improving the seismic response of URM buildings. Having no lateral bracing like the wall-diaphragm leads to onset of local wall failures as observed in numerous past earthquake incidents (Dizhur et al., 2010; D'Ayala et al., 2009; Dizhur et al., 2018). Due to the lack of anchorage system, major out-of-plane wall collapse can be observed. By having the lateral restraint system in the form of wall-diaphragm connections, this can significantly reduce the effective vertical height of the

wall for adding more bending resistance when the wall is required to respond in an out-of-plane behavior.

2.5 Previous Research on Bolted Connections

A study to develop a structurally sound wall-to-floor connections for unreinforced masonry buildings using a wood diaphragm was carried out by Moreira et al. (2012). Such idea was studied to engineer a proper wall-to-floor connection, which consists of timber floor joist bolted with a tie rod and anchored to a masonry wall using an anchor plate. Elements manipulated for this study were thickness of the wall. During the experiment, several cyclic pull-out tests were performed. From this study, the behaviour of the reinforced connections was then characterized. The experiment shows various types of failures at the connection point from the yield of the steel components to masonry cone breakout due to the resultant behaviour. For all tests on the 400mm walls, masonry cone breakout as shown in Figure 2.4 was coupled with the failure of bolted connections. As for 600mm thick masonry walls, the results demonstrate the bolted connection failure and yielding of steel rod. Moreira et al. (2012) concluded that the combination of the wood joists behaviour, the bolted connection and the steel angle contribute to the high ductility of the strengthening. This research, however, neglects the deeper analysis on the influence of wood properties bolted connection failure. Furthermore, the use of anchor plate in the modern construction industry is not viable, since the connection to walls these days are using anchor bolts. Thus, the application of timber bolted connection requires more study.

A study by Solarino et al. (2019) found out that the existing URM building's horizontal diaphragm action on floors or roofs is usually very poor due to the high flexibility of the diaphragm compared to the lateral resistance of masonry walls, which is often rigid.

To increase the in-plane stiffness, reinforced concrete or precast slabs with perimeter bond-beam were used in place of the timber floors or roofs similar to the modern masonry constructions. This is by means of providing the connection between RC slab and the wall using RC bond-beams as shown in Figure 2.5 so that the structural system performs as a monolithic unit in the event of earthquake strike. However, such option is considered as the last resort, as increase the mass may cause unfavourable high seismic inertia forces, which can cause out-of-plane mode and generates torsional effects.

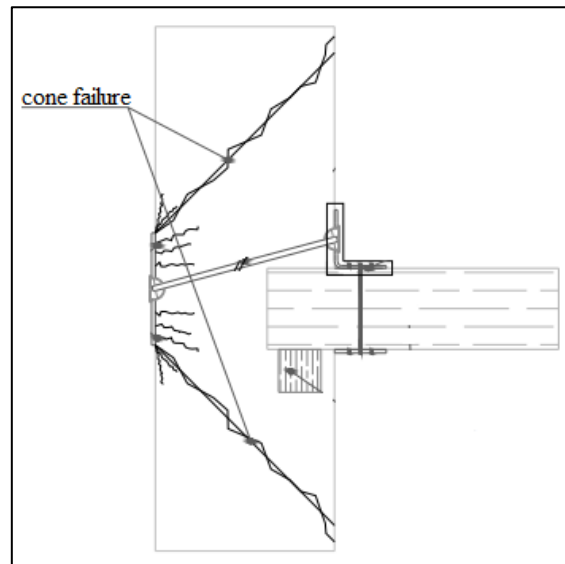


Figure 2.4: Masonry Wall Forming Pull-Out Cone Failure (Moreira et al., 2012)

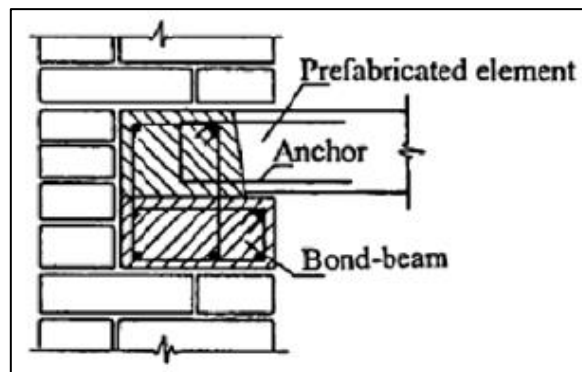


Figure 2.5: Illustration of the RC Bond-Beam (Solarino et al., 2019)

A study by Abdul Karim (2012) found that the guidelines given by NZSEE (1995) and FEMA (2006) regarding connection only relate to the possible failures of the masonry wall anchorages, and no procedures were mentioned to determine the strength values related to failure of the timber bolted connections. Abdul Karim (2012) stated that the failure of timber bolted connection design is important to be investigated because the minimum strength value that will govern the wall-diaphragm connection capacity can be acquired from the diaphragm connections. The study investigates the structural performance of the bolted connections on indigenous hardwoods of Rimu and Matai that are commonly used in the construction of roof / floor diaphragms of the URM buildings in New Zealand. The researcher also assessed the wood density and embedding strength. The study recommends the correlation between the wood shear strength parallel to the timber grain and the wood density for the strength estimation of bolted connections using the Row Shear Model (RSM). Also, for predicting the bolted strength using the European Yield Model (EYM), the study suggests the use of the experimental data from the wood embedding strength test conducted. The study has highlighted that both guidelines published by NZSEE (1995), and FEMA (2006) mentioned that wall-diaphragm connections in URM buildings consist of two major components. The first component is the wall anchor. The wall anchor may either be in the form of through-bolt with external bearing plate or the bolt is drilled into the masonry walls. The second component is the diaphragm connections, which are typically bolted joints on the roof rafters / floor joists of the diaphragm structural members.

Taking the same approach, Abdul Karim et al. (2018) conducted a study on local Malaysia hardwoods name *Meraka*. The name *Meraka* refers to one of their common trade names in Sarawak, which are also known as *Red Balau* and *Alan Batu* in West Malaysia and Brunei, respectively, by the Malaysian Timber Council, MTC (2013). The *Meraka*

hardwood was chosen due to their strength is grouped as SG3 by Malaysia Standard (MS544: Part 2: 2001), which is suitable to be used as structural members of joists and rafters by the Malaysian Timber Industry Board, MTIB (n.d.). Through this research, it was found out that the MS544-5 (2001) was too conservative in predicting the bolted connection strength of the local hardwood compared to EYM approach. Abdul Karim et al. (2021) published another study that focuses on the same *Meraka* hardwood, only this time the study focuses on the evaluation of the effectiveness of MS544-5 (2001) against the RSM. The study found out that the MS544-5 (2001) was too conservative compared to the RSM. The study concluded by stating that the use of RSM equations is a preferred option over the design code of MS544-5 for the brittle failure mode of local hardwoods. However, the research on the bolted connections of other hardwoods were found to be very limited. The validation on other strength group of wood is very essential as the previous study conducted only investigating on SG3.

2.6 Gaps of Previous Research on Bolted Connections

There are few research gaps can be identified in the previous research works that have been conducted on the bolted connections as given below.

- i. There are about 98 local hardwoods listed in MS544-2 (2001), which can be potentially used as structural members for the construction of timber roof and floor diaphragms of the Malaysia URM buildings. Only one hardwood has been studied and verified with the use of existing design equations, which is the *Meraka* hardwood. Thus, this present study extends the investigation on the local hardwoods to the *Nyatoh* hardwood.

- ii. There has been no attempt in any previous works to verify the genus of the hardwood specimens. Performing the genus identification on the hardwood specimens is very important not only to eliminate the elements of doubt on the hardwood specimens to be tested, but also to ensure the testing is being conducted on the actual intended local hardwood specimens.
- iii. The bolted connection experimental data on local hardwoods was found to be very limited. Due to this shortcoming, an effectiveness validation on the MS544-5 (2001) and other design equations in predicting the design strength cannot be done. It was also determined that the embedding strength of wood, f_h , is unavailable in the existing MS544-2 (2001). Thus, this present study provides the wood embedding strength, so that the effectiveness of EYM in predicting the bolted connection capacity for local hardwoods can be assessed.

2.7 Existing Equations for the Bolted Connection Design

The following section describes the details of the bolted connection design equations that involved in the validation of the prediction effectiveness in this present study.

2.7.1 Malaysian Timber Standard

The primary Malaysian Standard related to timber joints can be found in the document entitled ‘Code of Practice for Structural Use of Timber: Part 5: Timber Joints’ (MS544-5, 2001). Due to the purpose of evaluating the effectiveness of the existing design code in predicting the bolted connection design for the local hardwood, the details of the Clause 11 that specifically related to the bolted joints is further discussed.

$$F_{adm} = k_1 k_2 k_{16} k_{17} F \quad \text{Equation 2.1}$$

where,

F_{adm}	=	permissible load for laterally loaded fastener.
k_1	=	duration factor of the load given in Table 2.2.
k_2	=	1.0 for dry timber or 0.7 for wet timber.
k_{16}	=	1.25 for bolts that transfer the load through metal side plates of adequate strength and the bolts are a close fit to the holes in these plates provided in which; $b/d > 5$ for loads acting parallel to grain or $b/d > 10$ for loads acting perpendicular to the grain or 1.0 otherwise.
k_{17}	=	factor for multiple bolted joint bolted given in Table 2.3.
F	=	basic working load as derived in Section 11.2.2 of MS544-5

The load duration factor k_1 allows the capacity of the timber bolted connection to increase based on the duration of the load acting on the connection. From Table 2.3, the shorter the period of time, the higher the k_1 value is allowed to increase. However, for the connection strength comparison between the test results of this present study and the strength prediction calculated using MS544-2, the factor k_1 can be ignored. This is because all timber bolted connection specimens were tested with a loading rate of 1 mm per minute, which is considered a short-term load duration. In view of the notion to retrofit a URM building in Malaysia using wall-diaphragm connections, a similar loading duration is also applied to the connections because they are expected to withstand a short-term load duration during service, which is caused by the seismic loads.

From Clause 11.2.2 of MS544-5 (2001), the basic working load for bolted joints parallel to the timber grain is determined based on the type of joints, whereas for this present study, a three-member system or a double-shear connection was tested. From Clause 11.2.1 of MS544-5 (2001), a dry basic load, in kN, per fastener per shear plane can be obtained by

identifying the effective timber thickness, b , the bolt diameter, d , and the joint group, J . The classifications of joint groups can be determined in Clause 3 of the standard.

In MS544-5 (2001), the k_{17} is the only factor considers the brittle failure mode for timber bolted connections. From Table 2.4, it can be seen that any timber connections with multiple bolts more than four, the code considers the connections to fail in brittle by providing the factor k_{17} less than one.

Table 2.3: Modification Factor k_1 for Loading Duration of Different Fasteners (Table 4 of MS544-5, 2001)

Fastener	Loading Duration		
	Long term	Medium term	Short and very short term
Bolts, coach screw, split-rings, and shear-plates	1.0	1.25	1.5

Table 2.4: Values of Factor k_{17} for Use in the Design of Multiple Connector Joints of Bolts (Table 15 of MS544-5, 2001)

Type of joint	Value of k_{17}				
	$n_a \leq 4$	$n_a = 5$	$n_a = 10$	$n_a = 15$	$n_a \geq 16$
Dry timber	1.00	0.95	0.80	0.65	0.62
Wet timber (no transverse restraint*)	1.00	0.95	0.80	0.55	0.50
Wet timber (transverse restraint*)	0.50	0.50	0.50	0.50	0.50
NOTES: 1) n_a = total number of fasteners in each row per interface * the term ‘transverse restraint’ refers to the possibility of restraint to timber shrinkage due to the joint detail.					

2.7.2 European Yield Model

The European Yield Model (EYM), which is associated with Johansen's theory, considers ductile failure modes of bolted connections. Blass (2003) stated that the theory assumes that the timbers and dowels behave as an ideal plastic behaviour. The former plastic behaviour explains that the timber is resisting the applied embedding stresses caused by the bearing action as the wood is in contact with the fastener. The latter plastic behaviour describes that the fastener is in bending action when the wood embedding stresses exceeding the fastener bending yield strength.

In the calculation of the capacity of bolted connections, this theory considers several failure modes depending on the joint system, either a single-shear connection or a double-shear connection. The single-shear joint type, for an example of a steel-wood (SW) connection, consists of six possible modes of failures. The possible failure modes of the single-shear joint are as follows: (i) the first three modes show the bearing failure of the steel plate or the wood or the combination of both; and (ii) the last three modes show the bending failure of the fastener (see Table 2.5). The double-shear joint type, for an example of a steel-wood-steel (SWS) connection, consists of four possible modes of failures. The possible failure modes of the double-shear joint are as follows: (i) the first two modes show the bearing failure of the steel plate or the wood; and (ii) the last two modes show the bending failure of the fastener (see Table 2.6).

In this present study, the efficacy of the EYM in estimating the bolted connection in local hardwood was also assessed. This was done to compare with the MS544-5 (2001), so that suitable design equations can be recommended.

Table 2.5: Possible Failure Modes for Single-Shear Joint as per EYM

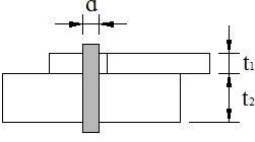
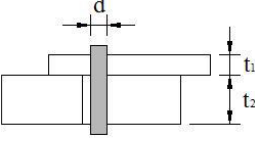
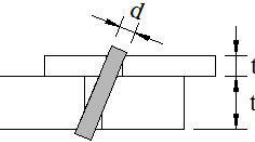
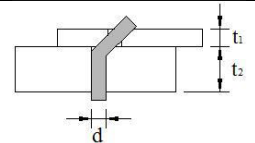
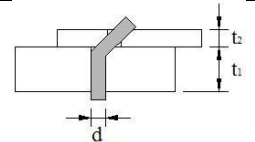
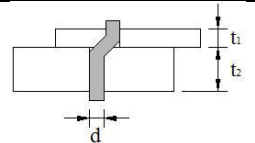
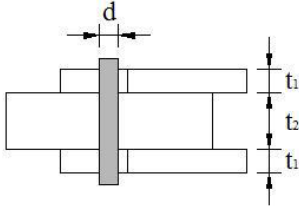
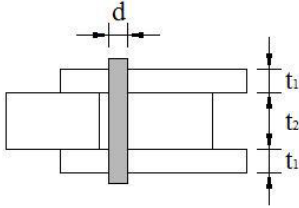
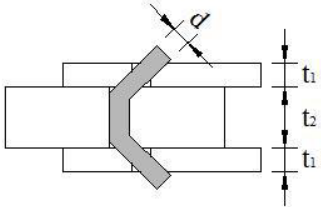
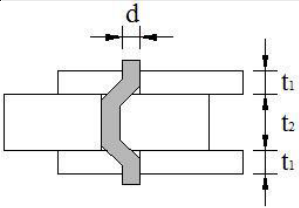
Failure mode	Resistance, R per fastener per shear plane
	$R = f_{h,1} t_1 d$ Equation 2.2
	$R = f_{h,1} t_2 d \beta$ Equation 2.3
	$R = \frac{f_{h,1} t_1 d}{1+\beta} \left\{ \sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 + \beta^3 \left(\frac{t_2}{t_1} \right)^2} - \beta \left(1 + \frac{t_2}{t_1} \right) \right\}$ Equation 2.4
	$R = \frac{f_{h,1} t_1 d}{1+2\beta} \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_y}{f_{h,1} t_1 d}} - \beta \right]$ Equation 2.5
	$R = \frac{f_{h,1} t_2 d}{1+2\beta} \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_y}{f_{h,1} t_2^2 d}} - \beta \right]$ Equation 2.6
	$R = \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2M_y f_{h,1} d}$ Equation 2.7
Notes: 1. β is the ratio of the embedding strength, $\beta = f_{h,2}/f_{h,1}$. 2. $f_{h,1}$ is the embedding strength corresponding to t_1, in MPa. 3. $f_{h,2}$ is the embedding strength corresponding to t_2 in MPa. 4. t_1 and t_2 is the timber thickness or fastener penetration of member 1 and 2, in mm. 5. d is the fastener diameter, in mm. 6. M_y is the fastener yield moment, in Nmm. $M_y = (1/6)f_y d^3$. 7. f_y is the fastener yield strength, in MPa.	

Table 2.6: Possible Failure Modes for Double-Shear Joint as per EYM

Failure mode	Resistance, R per fastener per shear plane
	$R = f_{h,1} t_1 d$ Equation 2.8
	$R = 0.5 f_{h,1} t_2 d \beta$ Equation 2.9
	$R = \frac{f_{h,1} t_1 d}{2 + \beta} \left\{ \sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta)M_y}{f_{h,1} t_1^2 d}} - \beta \right\}$ Equation 2.10
	$R = \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_y f_{h,1} d}$ Equation 2.11
Notes: 1. β is the ratio of the embedding strength, $\beta = f_{h,2}/f_{h,1}$. 2. $f_{h,1}$ is the embedding strength corresponding to t_1, in MPa. 3. $f_{h,2}$ is the embedding strength corresponding to t_2 in MPa. 4. t_1 and t_2 is the timber thickness or fastener penetration of member 1 and 2, in mm. 5. d is the fastener diameter, in mm. 6. M_y is the fastener yield moment, in Nmm, $M_y = (1/6)f_y d^3$. 7. f_y is the fastener yield strength, in MPa.	

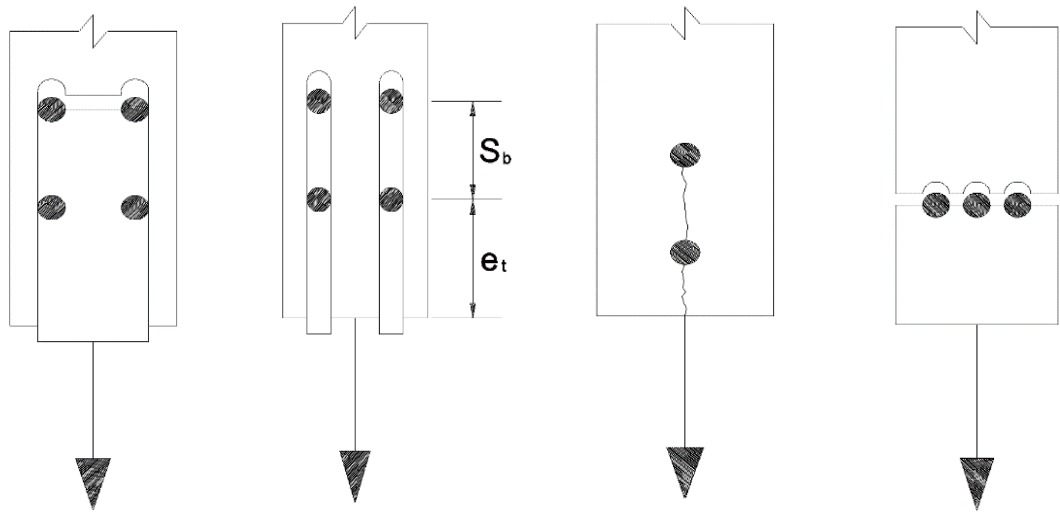
2.7.3 Row Shear Model

Quenneville (2009) stated that there are several types of brittle failure can be observed in the test specimens of bolted connections caused by the applied tension load parallel to the timber grain, which are group tear-out, row shear, splitting and net tension (see Figure 2.6). Due to this present study investigates a single row of bolted connections, only the row shear model (RSM) of failure mode was considered as the critical failure that governs the connection capacity in brittle mode. The RSM equation to estimate the design capacity of a group dowel fastener per shear plane is given in Equation 2.12.

$$R_{r\ rs} = RS_{i\ min}\ n_r \quad \text{Equation 2.12}$$

where,

- $R_{r\ rs}$ = design capacity of a group dowel fastener per shear plane.
- n_r = number of rows in the joint as per load component.
- $RS_{i\ min}$ = minimum ($RS_1, RS_2, \dots, RS_{nr}$).
- RS_i = shear capacity along two shear plane fastener row “i” in N
- = $\frac{2\ f_v\ K_{ls}\ t\ n_{fi}\ a_{cri}}{CF}$
- f_v = shear strength parallel to grain ($17.8G^{1.24}$ of Wood Handbook, 1987).
- G = 5th percentile relative density of timber in the oven dry condition.
- K_{ls} = factor for member loaded surfaces (0.65 for side member, 1 for internal member).
- t = member thickness, in mm.
- n_{fi} = number of fasteners in a row “i”.
- a_{cri} = minimum of e_t and s_b for row “i”, in mm
- CF = calibration factor



(a) group tear-out

(b) row shear

(c) splitting

(d) net tension

Figure 2.6: Brittle Failure Modes of Bolted Connections Loaded Parallel to the Timber Grain

2.8 Issues on the Existing Design Equations

By referring to the MS544-5 (2001), it can be seen that the design method was fundamentally developed based on the ductile failure mode. The brittle mode is only considered by the use of the k_{17} factor for connections that consist of multiple bolts. This is not entirely correct because, in many published literatures such as Quenneville (2009), Quenneville (2006a), Quenneville (2006b), Mohammad and Quenneville (2001), Quenneville and Mohammad (2000) and Abdul Karim (2021) the brittle failure mode of bolted connections can be influenced by the geometrical parameters of bolted connection configurations. The connection configuration parameters such as end distance, e_t , and spacing between bolts, s_b , not only can govern the bolted connection failure mode, but also can significantly affect the connection ultimate strength. That is why this present study was conducted to investigate the efficacy of the RSM in predicting the bolted connection strength in the selected local hardwood. Referring to the EYM, for estimating the capacity of the

bolted connection that fail in the ductile mode, the most important parameter of the embedding strength of wood cannot be found in the MS544-2 (2001). Thus, this present study was performed to include the embedding strength test on the local hardwood specimen, so that, the embedding strength value obtained can be used to estimate the connection capacity using the EYM. Following, a comparison between the EYM predictions and the actual laboratory results of bolted connection tests can be established and the EYM effectiveness can be validated.

CHAPTER 3

METHODOLOGY

3.1 Overview

This section provides an insight on the characteristics of *Nyatoh* hardwood as specified in MS544-2 (2011) and its potential uses. This chapter further outlined the wood identification works, which is an important step to be carried out in order to ensure that the specimens studied were truly that of *Nyatoh*. The important terms in the hardwood anatomy are also provided in this chapter in order to help the reader to understand the identifications of hardwood genus when the results of microscopic end-grain section of the specimens are presented in this chapter. This chapter also provides the methods of specimen preparations and the tests conducted, which are timber bolted connection tests, embedding strength tests, moisture content and density test.

3.2 Characteristics of *Nyatoh* Hardwood

According to the Malaysian Standard (MS544: Part 2: 2011), *Nyatoh* is classified as light hardwoods and SG5. Table 3.1 shows further characteristics of *Nyatoh* hardwood. Despite it being a light hardwood, it can still be used for the applications of structural components of beams, joists and rafters as stated by the Forest Department Sarawak (1999), which also listed *Nyatoh* as one of the common wood species found in Sarawak. Hence, *Nyatoh* can be the typical wood species that are used for the construction of structural members of timber diaphragms of the URM buildings.

Table 3.1: Grading of *Nyatoh* (MS544: Part 2: 2001)

Class	Standard Name	Other common name (s)	Species yielding the timber	Density at 19% moisture content (kg/m ³)	Specific gravity	Remarks
Light Hardwood	<i>Nyatoh</i>	<i>Nyatoh</i> (Sab.), <i>Jangkar</i> , <i>Nyatoh</i> (Sar.)	Most species of <i>Sapotaceae</i>	750	0.59	Mainly pale red-brown of Bitis
NOTE: Specific gravity (based on oven-dried weight and volume at test)						

3.3 Hardwood Identification

Prior to preparing the specimens for the purpose of bolted connection testing, the wood provided were first needs to undergo an identification. Figure 3.1 shows the process flow of performing the wood identification. A total of 50 planks with a cross section of 100mm × 50mm × 3660mm length was supplied. From each of the planks having 12ft length or 3660mm length, about 60mm of the planks part were extracted from either end of the plank. It is then further cut out in the shape of a cube of 10mm size as shown in Figure 3.2.

After all the 50 cube shape specimens extracted, the cross-section surface of each specimen was then sliced using a microtome. A microtome is a mechanical device used typically to cut a biological specimen into a very thin slice or segments for further observation using a microscope. However, for the purpose of this study, the microtome served as a tool to obtain a clean cut of the specimen's end-grain or cross section. This was done to ensure the wood anatomy can be clearly observed from the end-grain surface of the wood under the microscope. The clean-cut end-grain surface was directly observed and

analysed under a microscope with 20 times magnifications. The observed microscopic end-grain image of the specimens was captured for comparison with the microscopic image database gathered from the Forest Department Sarawak (FDS) library to find the similarities between the same family of the hardwood specimens. Table 3.2 shows the descriptions of the hardwood anatomy to help the reader understand the anatomy of *Nyato* hardwood later.

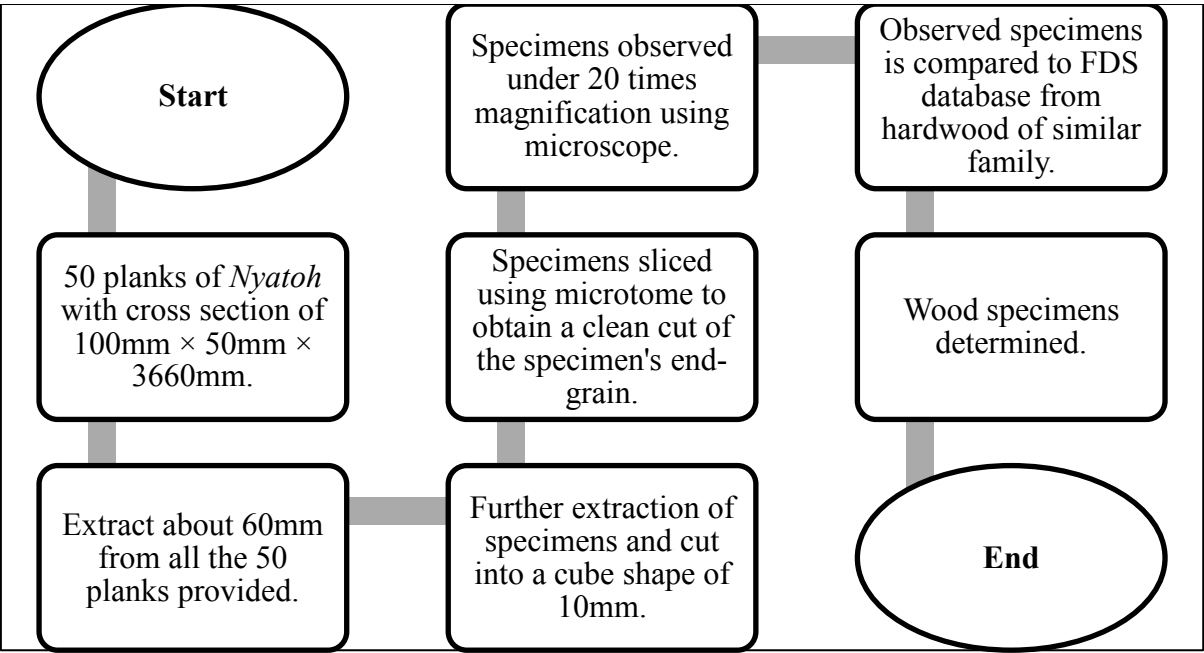


Figure 3.1: Process Flow of Wood Genus Identification



Figure 3.2: Extracted Specimen for Wood Genus Identification

Table 3.2: Hardwood Anatomy Descriptions (Meier, n.d)

Anatomy	Descriptions
Vessel elements	Holes in the wood viewed from end-grain. Often refer as pores.
Pore size (µm)	Small (< 50); Medium (50-100); Large (100-200); Very large (>200).
Pore frequency (vessels/mm ²)	Very few (< 5); Few (5-20); Moderately numerous (20-40); Numerous (40-100); Very numerous (> 100).
Pore arrangement	Solitary pores – single openings. Multiple pores – two or more adjacent pores that clearly shared the middle wall in between them. Radial multiples – typically vertically arranged pore multiples.
Pore contents	Deposits (coloured gums, resins, or others) in the wood cells as sapwood became hardwood. Tyloses (singular, tylosis) – bubble-like structures that grow into open pores, at times completely sealed off the pores of heartwood. Relative terms (sparse, common, and abundant) are used to describe frequencies and presences.
Wood parenchyma	A living tissue in the sapwood that function as storage cells, commonly referred as axial or longitudinal parenchyma that is oriented along the stem length of the tree. Apotracheal parenchyma – pores isolated from parenchyma. - Diffuse parenchyma – single but scattered parenchyma. - Diffuse-in-aggregates – joined parenchyma that forms a thin tangential (horizontal) line but noticeable. Paratracheal parenchyma – parenchyma associated with pores. - Vasicentric parenchyma – ring or circular formed parenchyma that borders the pore. - Aliform parenchyma – closely associated to vasicentric with two main variants given below: - Winged – parenchyma with wing-shaped either at one or both ends of the pores.

Table 3.2 continued

	b. Lozenge – diamond or elongated oval formed parenchyma that surrounds the pore. iii. Confluent parenchyma – an extensive aliform parenchyma that is in contact with the parenchyma from adjacent pores. iv. Banded parenchyma – tangential (horizontal) parenchyma bands with variations such as: a. Marginal parenchyma – parenchyma bands that stretches along the borderline of the growth ring. b. Reticulate parenchyma – both parenchyma (tangential/horizontal) and rays (radial/vertical) exists in thin and closely bands of space, which mimics the appearance of grid-like or pattern-like. c. Scalariform parenchyma – wide rays and short-arched parenchyma combined between the rays, creating a profile of rugs on a ladder.
Rays	Cells run in radial (vertical) direction when viewed from the endgrain, which forms a more-or-less lines that spaced evenly across the wood sample.
Ray spacing (rays/mm)	Wide (< 5); Normal (< 6-9); Fairly close (< 10-13); Close (< 14-20); Extremely close (> 21).

Figure 3.3 shows the endgrain of microscopic sections of *Nyatoh* hardwood, observed under a 20 times magnification. By observation, the size of the pores can be said to be small to medium. Frequency of the pores typically very few to few, and moderately numerous. The identification of the size and frequency of the pores is by utilising both vertical and horizontal 1 mm scale and cross-checked with the anatomy descriptions given in Table 3.2. Most pores alignment is in radial multiples as well as solitary pores present. No deposits present, but tyloses can be seen. Both parenchyma and rays are abundant and thin, which is found to be in closely spaced reticulate bands. These anatomy findings are then cross checked with some of the microscopic sections available from Wood Library, Development and Innovation Division, Forest Department Sarawak. From the microscopic

analysis made earlier, they are further compared to Figure 3.4 (a) - (d). From the findings made, the *Nyatoh* hardwood specimen used for this study were identified and confirmed as the *Palaquim spp.* genus or *Payena spp.*, that belongs to *Sapotaceae* family. The *spp.* is an acronym for *Several Species*. The identification was only made up to genus level, as the species level could not be ascertained by only conducting the above-mentioned method above. As such, objective number 1 has been achieved. The microscopic sections of the rest of the planks can be referred to Appendix A.

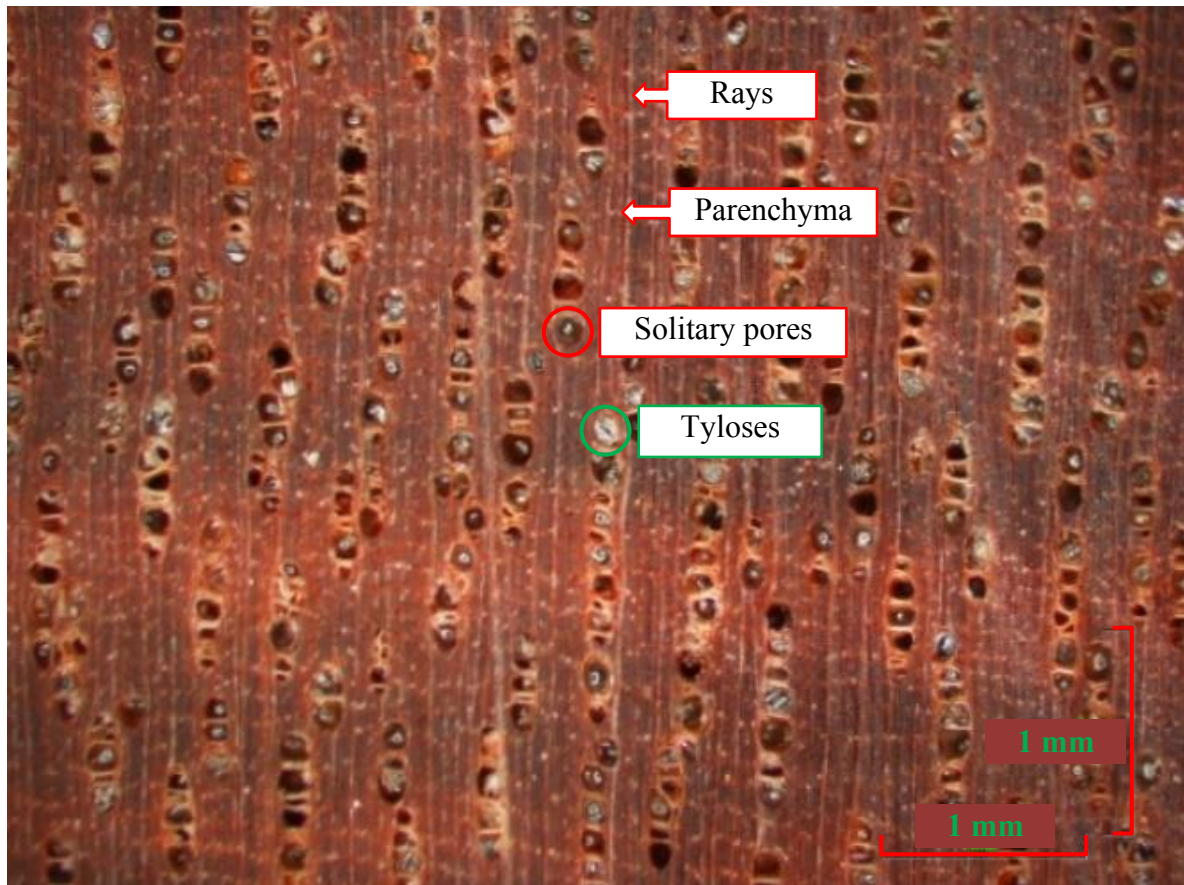


Figure 3.3: End-Grain of *Nyatoh* Specimen under 20 Times Magnification of Microscope

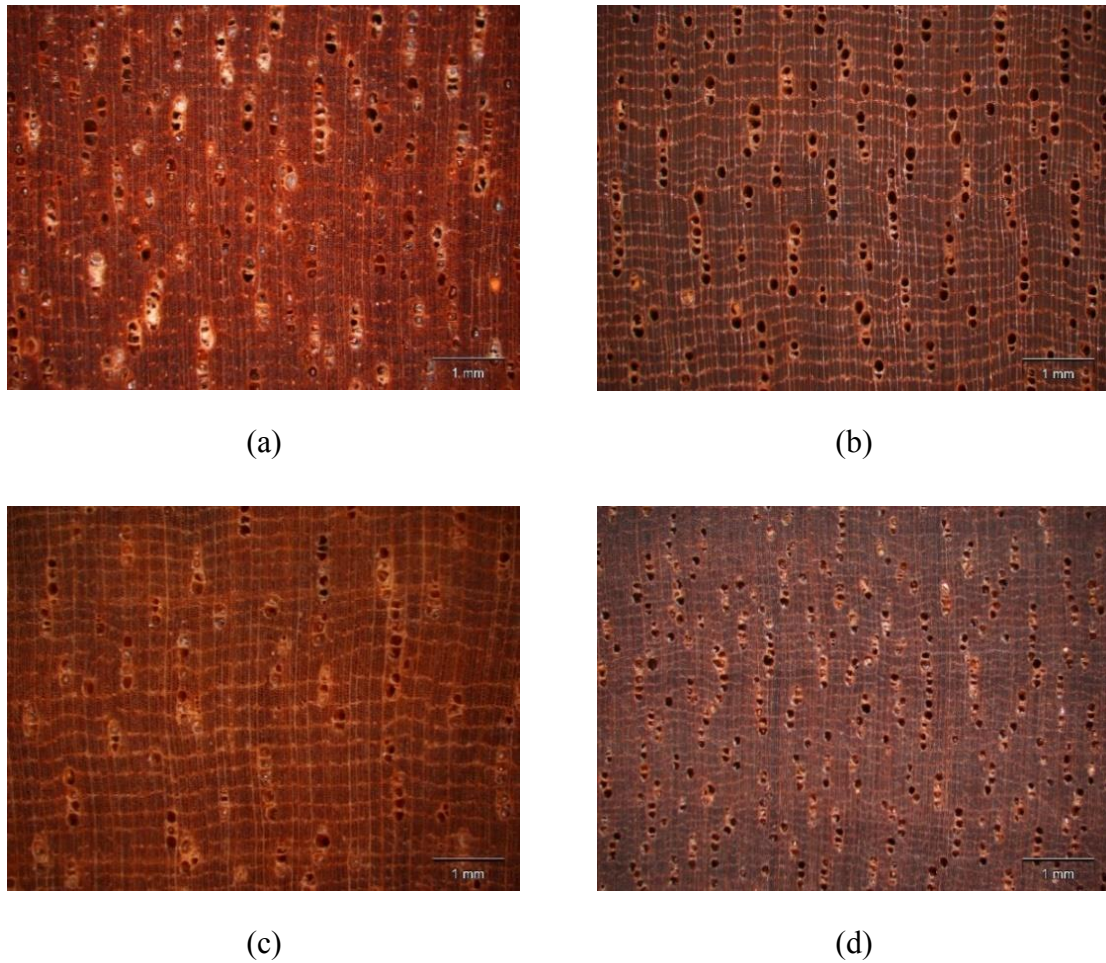


Figure 3.4: Microscopic section of 20 times magnification (a) *Palaquim beccarianum*; (b) *Palaquim ridleyi*; (c) *Palaquim rivulare*; and (d) *Payena endertii* (Source: Wood Library, Development and Innovation Division, Forest Department Sarawak)

3.4 Laboratory Testing

This section provides the details the main experimental works, i.e., bolted connection strength test and the test to determine the timber properties, which are embedding strength test and moisture content and density test. The experimental data from the timber properties determination test is of vital importance to verify the equations of predicting the strength of a bolted connection of local hardwood, specifically *Nyato* from European Yield Model (EYM), Row Shear Model (RSM) and Malaysian Standard (MS544) equations. However,

these laboratory tests only came after the bolted connection strength test and this section is needed to achieve the second and third objective.

3.4.1 Bolted Connection Strength Testing

To ensure that the data collected is in line with previous studies made in previous local hardwood timber, the author used the same approach as conducted by Abdul Karim (2018), Abdul Karim (2021) and Abdul Karim (2022) on timber bolted connection studies on local hardwood. This arrangement is to ensure consistency of the data obtained, hence, standardising the method of testing.

The cross-section size of timber used in the bolted connection test is 50 mm width $\times$ 100 mm height. The following dimension was chosen because the author is of the opinion that such size is one of the most standard sizes used as structural components. There were 18 types of bolted configurations, each were having varying end distance (e), spacing between bolts (s_b) and different number of bolts (n_f) as shown in Table 3.3. The connections with two-bolts and three-bolts were varied in terms of spacing-between-bolts. Each of the bolted connection configuration groups has a total of ten replicates, meaning the number of test specimens for this study were at least 180 specimens. The specimens were kept at room temperature for a minimum time frame of four weeks before the bolted connection strength test was conducted. The step was done to ensure that the specimen's moisture content less than or equal to 19% during the bolted connection testing, so that the timber specimens were in a dry condition as per requirement of MS544-2 (2001). There was no timber moisture meter available to directly measure the moisture content of the timber specimens after the air-dried period. However, the moisture content of all timber specimens was determined after the completion of the bolted connection tests.

Table 3.3: Details of Connection Configurations

Group	e (mm)	s_b (mm)	n_f (nos)
1	150	-	1
2	125	-	1
3	100	-	1
4	75	-	1
5	50	-	1
6	150	100	2
7	125	100	2
8	100	100	2
9	75	100	2
10	50	100	2
11	150	50	2
12	125	50	2
13	100	50	2
14	75	50	2
15	50	50	2
16	100	50	3
17	75	50	3
18	50	50	3

The testing of bolted connection strength was all in a single row arrangement, thus, a row shear failure mode is the critical failure of the connections depending on the end distance (e) and spacing between bolts (s_b). Figures 3.5, 3.6 and 3.7 each shows the example

of 1-bolted, 2-bolted and 3-bolted connections, respectively. To ensure an independent bolted connection at each specimen end during testing, a minimum distance of 400 mm was applied, which is 30 times of the bolt diameter used as per requirement of the ISO/DIS 10894-2.

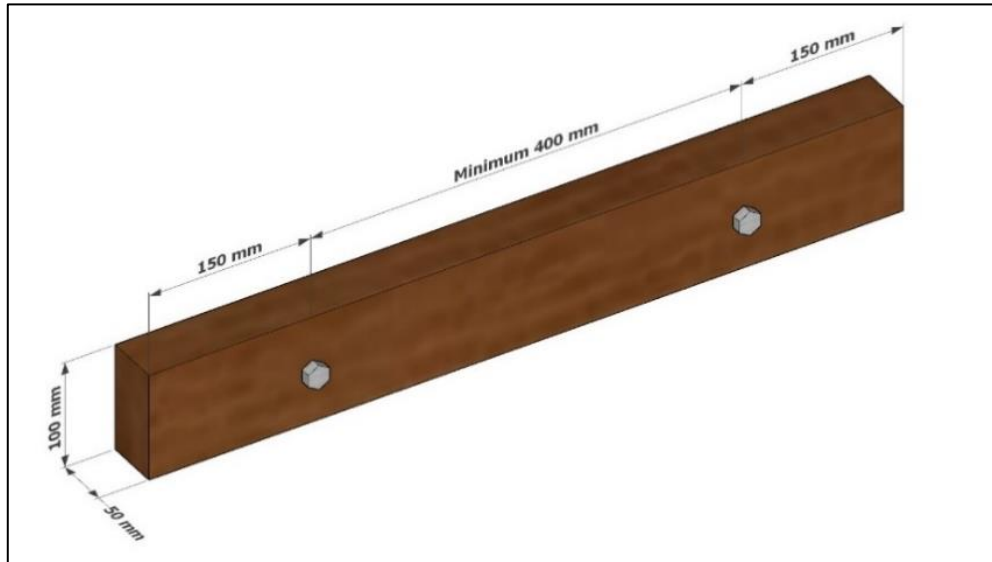


Figure 3.5: Illustration of Group 1: 1-Bolt Connection

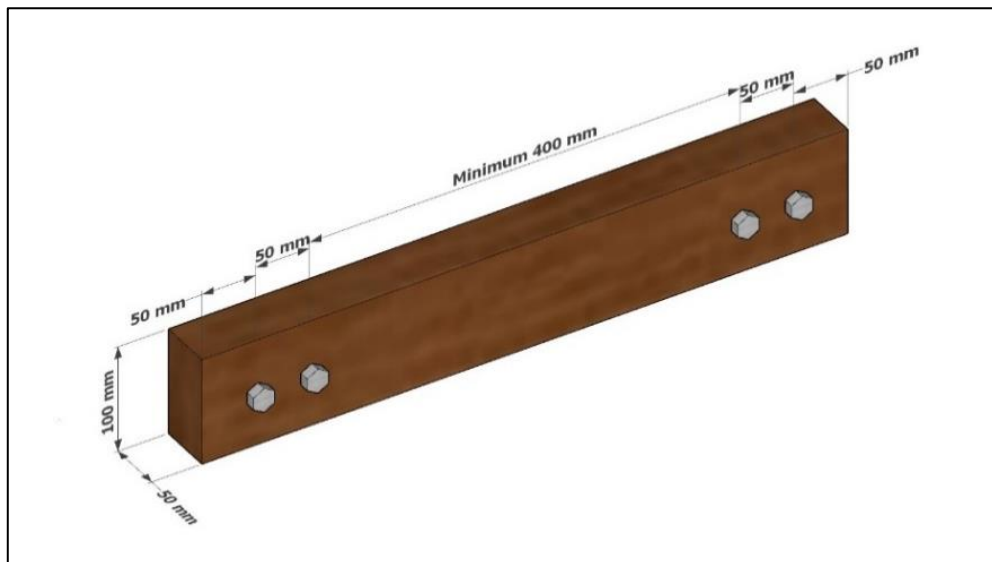


Figure 3.6: Illustration Group 15: 2-Bolt Connection

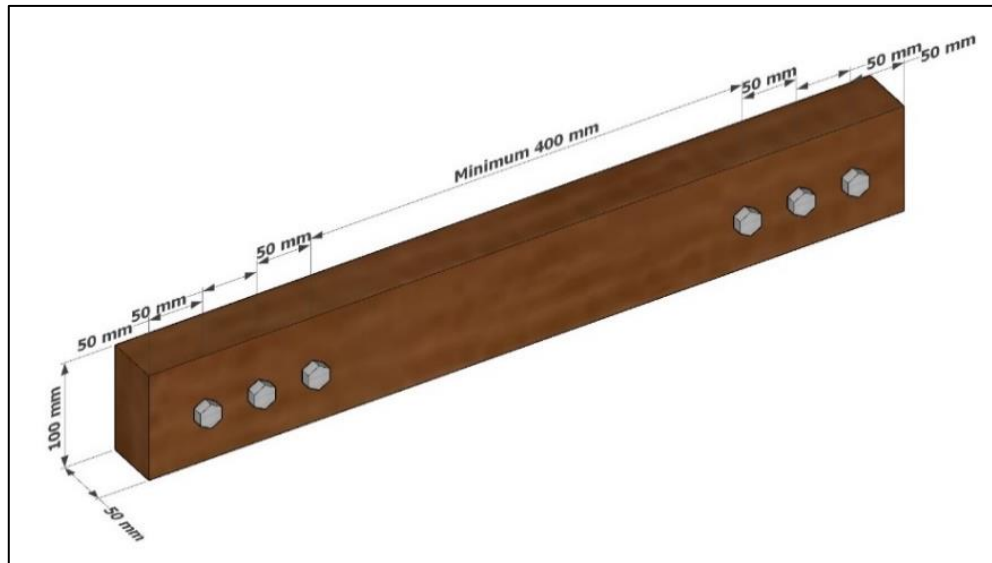


Figure 3.7: Illustration Group 18: 3-Bolt Connection

Table 3.4 shows the list of testing equipment to be fitted onto the 300kN SHIMADZU Universal Testing Machine. The hardwood specimens were sandwiched by two mild steel plates at both ends bolt into place. The other end of the plates will be connected to the testing machine by using steel rods. The specimens will be loaded with a constant tensile load until the specimen fails or breaks apart. The rate of displacement exerts on the specimen is to be set at 1mm per minute. The rate of displacement applied was in accordance with recommendations by ISO 6891 (International Organization for Standardization, 1983). Figure 3.8 shows the set-up of the bolted connection test.

Table 3.4: Details of Test Rig and Bolt Used

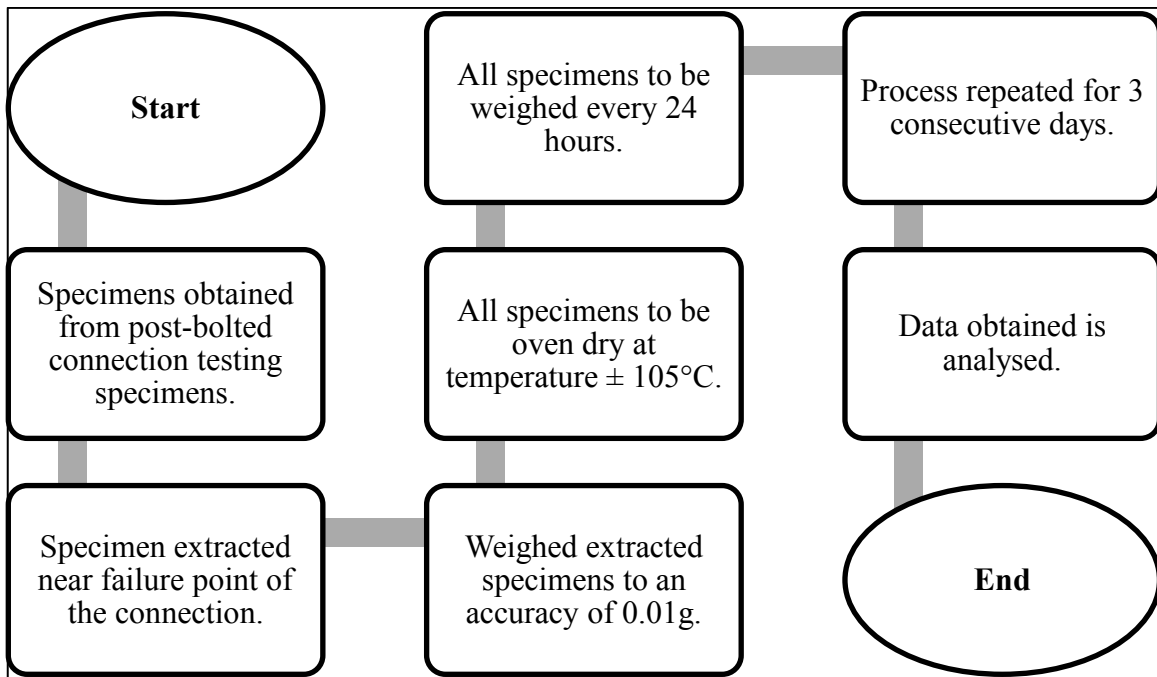
Equipment	Dimension
Stainless steel testing rig	69.7mm diameter head with 35mm diameter hole
Stainless steel rod	35mm diameter with 100mm length
Mild Steel Plate	520mm length x 100mm x 15mm thickness
Bolt	Grade 4.6, 13mm (diameter)



Figure 3.8: Test Set-Up of Timber Bolted Connection Testing

3.4.2 Moisture Content and Density Test

Figure 3.9 shows the process flow of the moisture content and density test. The specimens for the determination of moisture content and density were extracted from the tested specimen of the bolted connection strength test. It is very essential to determine the moisture content of the specimen because the connection strength measured can be affected by the dry or wet condition of the timber test piece. The size of the extracted specimens is in cube shape of 50 mm as shown in Figure 3.10.



The procedure used to determine the moisture content is based on the standard outlined in AS/NZ 1080.1 by the Australian/New Zealand Standard (1997). The initial mass of all specimens was weighed to an accuracy of 0.01 g before oven drying to a temperature $\pm 105^{\circ}\text{C}$. The specimens were left for 24 hours in the oven for the drying process as shown in Figure 3.11. After 24 hours, the specimens were weighed immediately after taking out of the oven and their mass recorded. The dimension of the specimens was also measured to check for any shrinkage in dimension. It should be noted that the number of specimens may not necessarily be the same as the number of specimens in bolted connection tests, as there were repeating tests conducted for some specimens of the timber bolted connection in order to replace the strength results that were affected by the moisture content and other influence factors such as slope of grain and occurrence of knot.

From the moisture content and density test, the initial density and the oven dry density of the wood test pieces can be calculated. The difference between the initial density and oven dry density was used as the baseline to determine the percentage of moisture content loss during the oven drying process. This analysis helped to determine the condition of the specimens during the time they underwent the bolted connection strength test. The 5th percentile of the density was then used in Equation 2.12 to validate the accuracy of the RSM equation. The 5th percentile value of density was applied to the RSM equation to enable a comparison with the characteristic strength provided by the MS544-5 (2001). The characteristic strength of timber bolted connections given by the standard is calculated based on the lower 5th percentile value, which is at least 95% of the connection test specimens should have a capacity that is higher than the 5th percentile value. The percentages of the moisture content of the specimens shall be less than or equal to 19% for it to be considered in dry state as per MS544-2 (2001). As per mentioned in previous Section 3.4.1, this moisture

content test was essential to be performed for determining the moisture content of all timber bolted connection specimens due to the unavailability of the timber moisture meter for the quick measurement of moisture content on the specimens before the bolted connection testing.



Figure 3.11: Specimens Put Into Oven for Drying Process

3.4.3 Embedding Strength Test

Similar determination of moisture content and density, the embedding strength test was performed after the bolted connection strength testing. Figure 3.12 shows the process flow of the moisture content and density test. The specimens for embedding strength were extracted from the tested specimens of bolted connection tests unless for specimens that have been split in full length were omitted. The specimens were cut into dimensions of $100\text{mm} \times 50\text{mm} \times 100\text{mm}$. The bolt hole was half of the 14mm diameter. It should be noted that the number of embedding strength test specimens may not necessarily be the same as the number

of bolted connection test specimens due to the specimens that have split in full length can reduce the total number of specimens. The total number of specimens can also be increased because of the repeated bolted connection tests conducted to replace the specimens that were affected by the strength influence factors mentioned in Section 3.4.2. A steel bar was then placed on top of the half-cut bolt hole, then pressure was applied on top of the steel bar, which then applied the forces on top of the drilled surfaces at a steady rate of 1mm per minute. All the specimens have undergone the bolted connection test. The raw data were obtained and analyzed by taking the maximum strength of every specimen tested by using Equation 3.1. The result is tabulated in Table 4.1 in the next chapter. This testing is based on the International Standard for Timber Structure (ISO/DIS 10894-2). Figure 3.13 shows the illustration of the embedding strength test. Figure 3.14 shows the state of the steel rod was fully embedded into the wood block, which is indicated that the test can be stopped because the measurement of the embedding strength was determined by dividing the maximum force with the area of contact between the bolt diameter and the width of the wood block. The following Equation 3.1 was used to determine the embedding strength of *Nyatoh* hardwood.

$$f_h = \frac{F_{max}}{d * t} \quad \text{Equation 3.1}$$

Where:

- f_h = wood embedding strength (MPa)
- f_{max} = maximum compression load applied (MPa)
- d = diameter of bolt (mm)
- t = thickness of the specimens (mm)

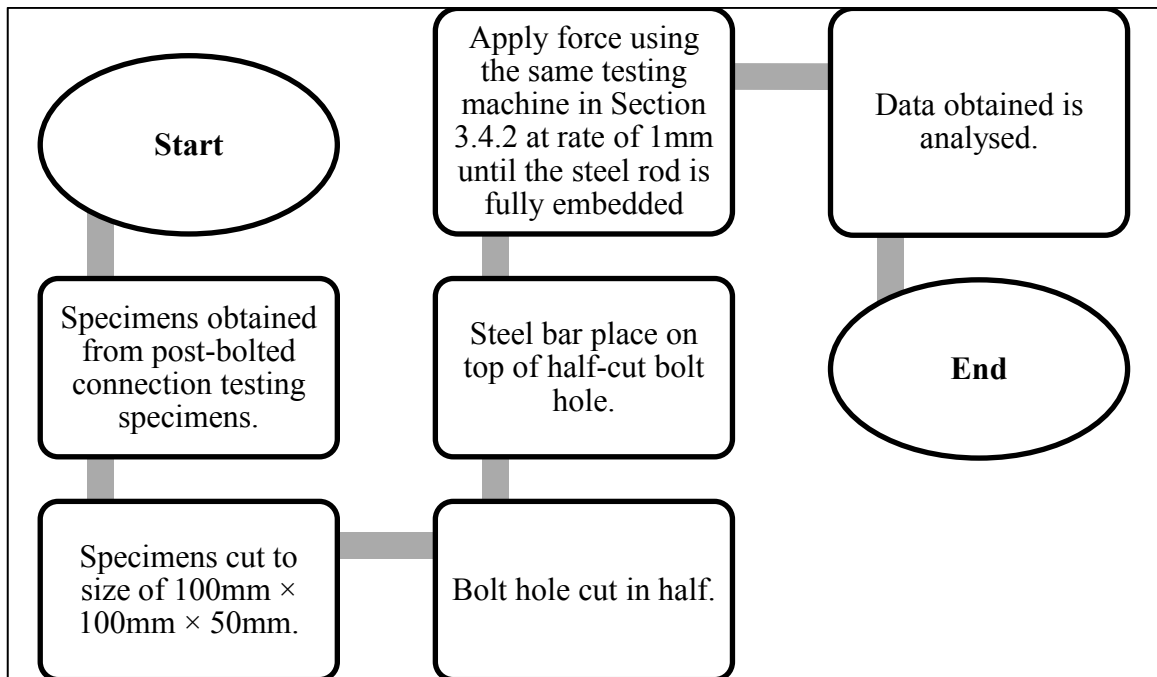


Figure 3.12: Process Flow for the Embedding Strength Test

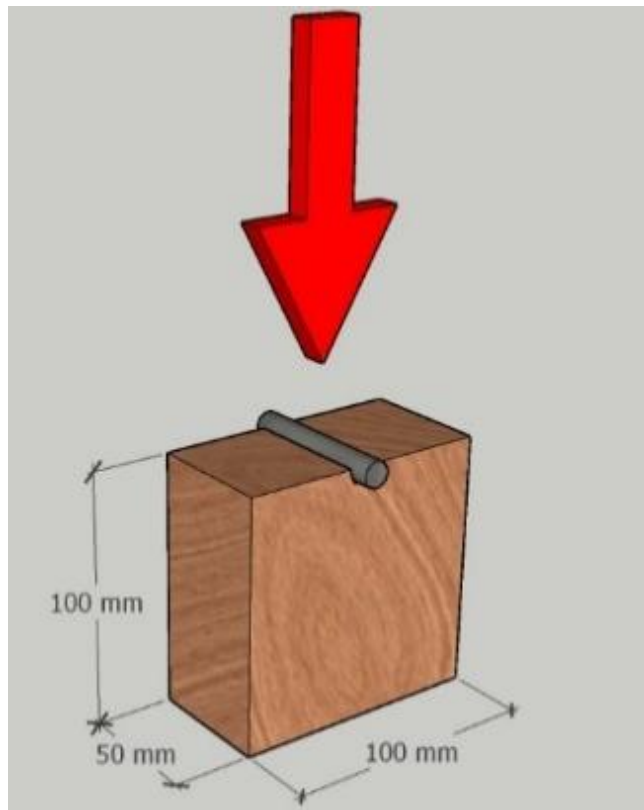


Figure 3.13: Illustration of Embedding Strength Test

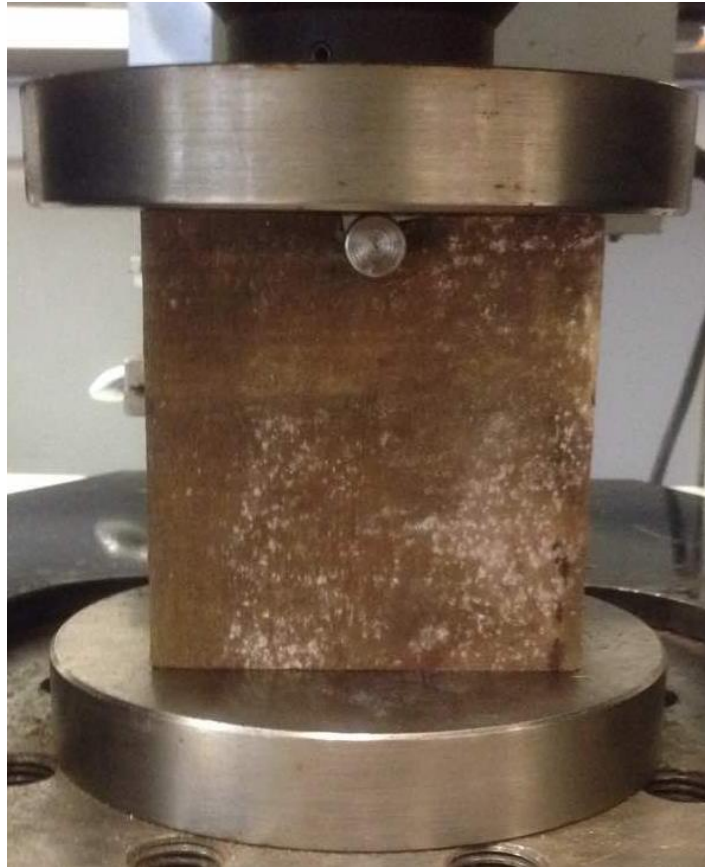


Figure 3.14: Full Embedded of the Stainless Steel Rod Into the Wood Block

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Overview

This chapter discusses the results obtained in the tests conducted, which are the determination of embedding strength, moisture content, density, and bolted connection strength test. Unlike the sequencing of testing as explained in previous chapter, this chapter discusses the results of embedding strength and moisture content and density test first, as these parameters are required to be used in the design equations of European Yield Model (EYM), Row Shear Model (RSM) and Malaysia Standard (MS544) for the readers to understand the follow of the equations utilizations. The equations from the EYM, RSM and MS544 were utilized to calculate and predict the bolted connection strength based on the given parameter obtained during the laboratory testing. Comparisons of findings between EYM, RSM and MS544 were made according to the types of failure observed and the value obtained from the testing machine. This chapter also discusses the calibration factor and the ultimate strength of connections.

4.2 Embedding Strength Test Result

A total of 183 pieces of embedding strength samples was tested. All samples with 100mm × 100mm × 50mm dimensions were extracted from the specimens that have undergone bolted connection strength tests. Only bolted connection specimens that did not split at its entire length were selected for the extraction. The average and 5th percentile values were obtained using the data of the embedding strength as shown in Table 4.1. The following value was used to determine the predicted strength of bolted connection using EYM formula, from Equation 2.8 to 2.11, as the test was all in a double shear joint. Figures 4.1 (a) – (d)

show some of the typical load-displacement relationship of the wood embedding strength tests conducted. The graphs for the rest of tested samples can be found in Appendix B. The ultimate load values of all tested specimens from Group 1 to Group 18 can also be obtained in Appendix C.

Table 4.1: Results of Embedding Strength Test of *Nyatoh* Hardwood

Hardwood	Total No. of test pieces	$f_{h,avg}$ (MPa)	CoV	$f_{h,5th\%}$ (MPa)
<i>Nyatoh</i>	183	34.69	20.09	23.22
Notes: CoV = coefficient of variation $f_{h,avg}$ = average embedding strength (using $f_{h,2}$ as failure corresponds to t_2) $f_{h,5th\%}$ = 5 th percentile embedding strength				

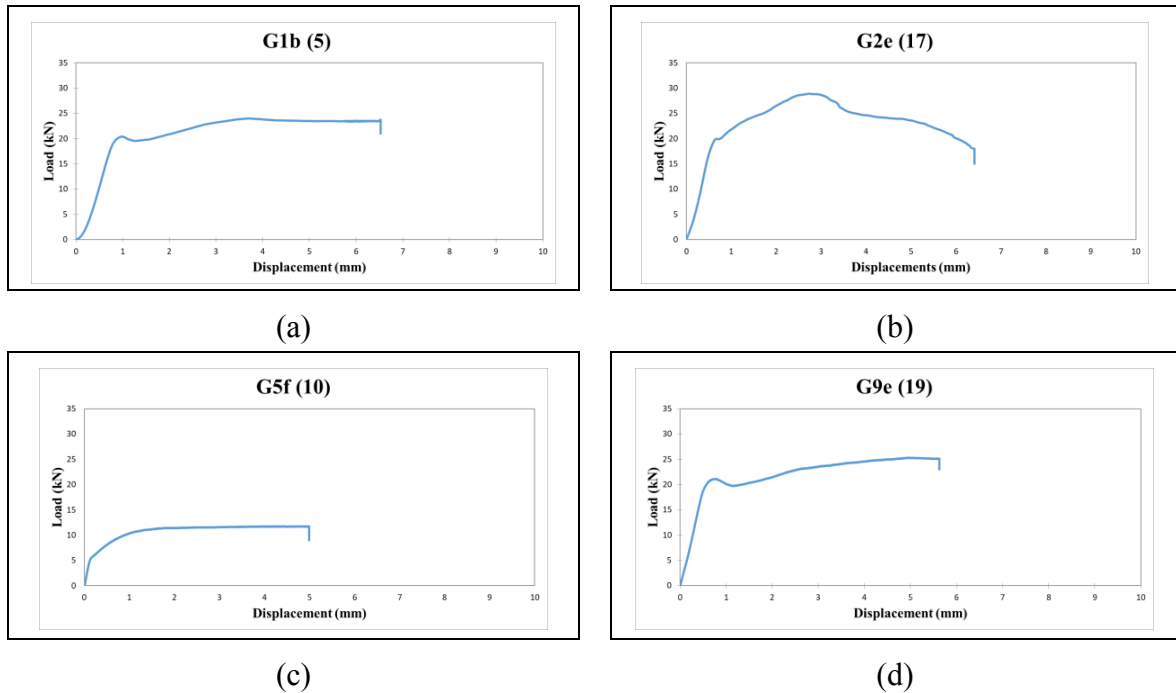


Figure 4.1: Typical Load-Displacement Relationship of Embedding Strength Test

4.3 Moisture Content and Density Results

Table 4.2 shows the result of moisture content and density for *Nyatoh* hardwood. The raw data of all the moisture content and density is shown in Appendix D. From the findings, the average moisture contents were at 16%, which is less than 19% as per the requirement of MS544-2 (2001). The 16% moisture content achieved to confirm that all wood specimens of bolted connections were tested in a dry condition, which the effect of moisture content on the connection strength can be avoided. To achieve a reliable connection test result, all timber specimens need to be in dry state. The value of 5th percentile density obtained from this specimen is used in designing bolted connection strength using RSM formula.

Table 4.2: Moisture Content and Density Results of *Nyatoh* Hardwood

Hardwood	Total No. of Specimens	ρ_{avg} (kg/m ³)	CoV	$\rho_{5th\%}$ (kg/m ³)	MC _{avg} (%)
<i>Nyatoh</i>	200	574.33	12.39	457.25	16
Notes: ρ_{avg} = average density CoV = coefficient of variation $\rho_{5th\%}$ = 5 th percentile density MC _{avg} = average moisture content					

4.4 Results for Bolted Connection Strength Test

This section discusses the findings of the bolted connection strength test, while also looking into the types of failure modes. The types of failure and the use of the calibration factor (CF) are also reported. The purpose of the determination of CF is to optimize the

equation of Row Shear Model (RSM). An in-depth discussion of the predictive strength using the formulas from EYM, RSM and MS544 against the experimental results is presented further in the later part of the discussions.

4.4.1 Nyatoh Hardwood Failure Mode

Two types of failure mode were observed during the bolted connection tests. It was found out that a bolted connection having an end spacing or a bolt spacing more than 75mm failed primarily in ductile mode, which is also known as bearing failure. Given that the failure was on specimens with 50mm wood thickness, this is in agreement with the findings reported by Abdul Karim (2018) on a local hardwood. Failure in splitting, a secondary brittle failure was also observed in many of the specimens. However, though they were present, it does not affect the resistance of the connection as the connections were primarily failed due to the wood bearing.

As shown in Figure 4.2, the load-displacement curves of Group 3 demonstrate the deformation of the bolted connections prior to its abrupt failure. Having an end spacing of 100mm, it exhibits a ductile failure mode, where one can see that the curvature elongates after reaching its ultimate tensile strength. This can also be proven from Figure 4.3, where the hole in the timber specimen elongates, which indicates the wood bearing failure. One also can clearly see that the bending of bolt in Figure 4.3. This demonstrates that the yielding of the fastener takes place before a sudden drop of the curve can be observed. Most of the abrupt failure modes of the specimens were in the form of secondary splitting failure as shown in Figure 4.3. Very few of tested specimens were found to fail in a row shear as a secondary brittle mode of failure. As for other bolt group configurations of more than 75mm, irrespective of the configurations being that of end spacing (e) or bolt spacing (s_b), they

exhibit a very similar curvature. These groups of bolted connections are Groups 1–4 (inclusive of Group 3 discussed earlier) and Groups 6–9 (see Appendix E), which they were found to fail in a similar manner as Group 3.

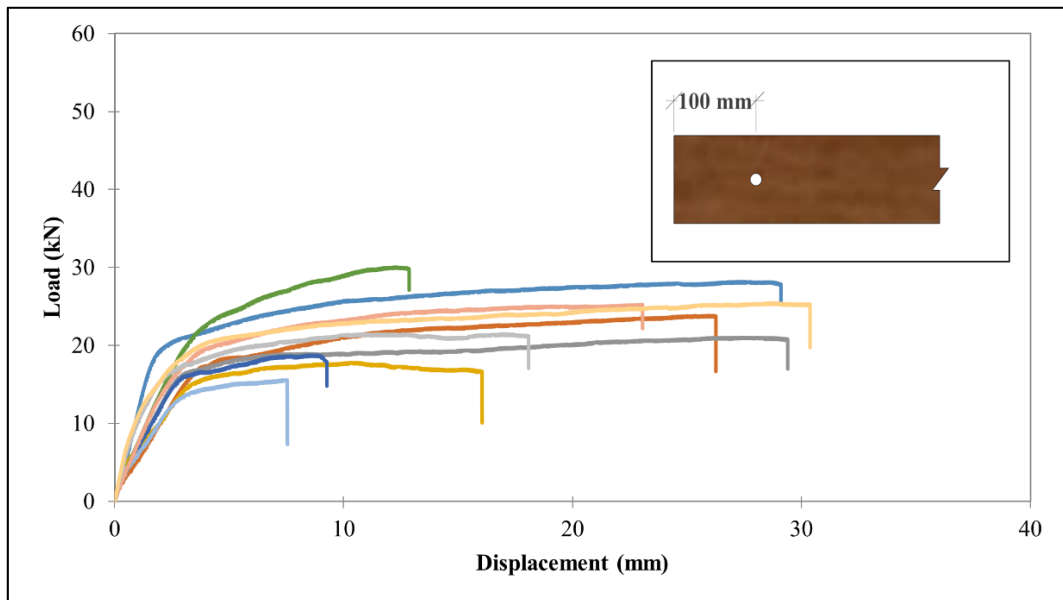


Figure 4.2: Load Versus Displacement Graph of Group 3

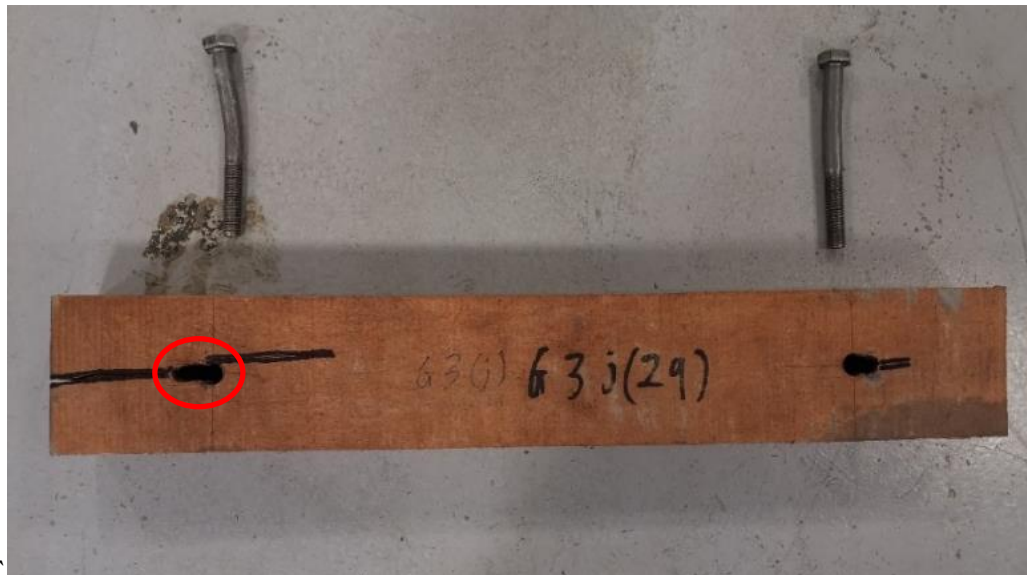


Figure 4.3: Group 3 Specimen Failed in Wood Bearing with Secondary Splitting Failure

Taking one example from the group that expected to fail in row shear, i.e., brittle mode, Figure 4.4 shows the load versus displacement graph of Group 13. Group 13 was chosen for comparison with Group 3 due to their similarity in the end distance, only differentiated by the number of bolts with bolt spacing of 50mm, which was done purposely to have higher chances of it failing in row shear. As presented in Figure 4.4, the load curvature rises till it reaches the ultimate tensile strength, only in this case, its failure shows a more rapid drop with less or no elongation of the graph curve. This is further proven when one refers to Figure 4.5, where the failure was mainly in row shear failure. All such failure modes with connection having either end distance (e) and/or bolt spacing (s_b) of 50mm. As such, it is found out that the smaller the end distance or spacing between of a bolted connection, the higher the possibility of the specimens to fail in row shear and otherwise. Other bolted configurations exhibiting similar row shear failure curvature were those in Group 5 and Groups 10 – 18, irrespective of number of bolts (see Appendix E).

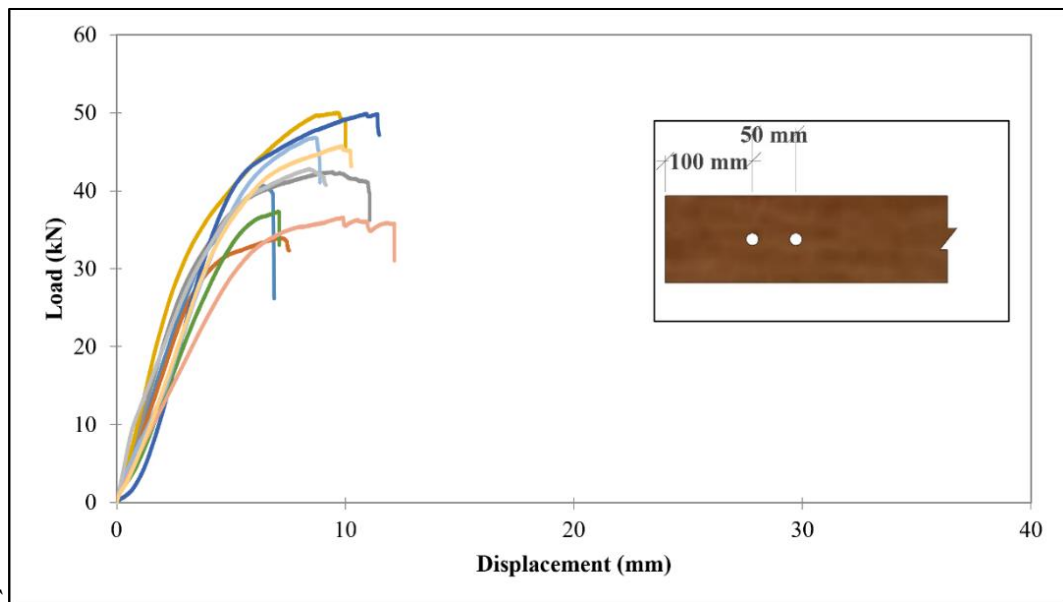


Figure 4.4: Load versus Displacement Graph of Group 13

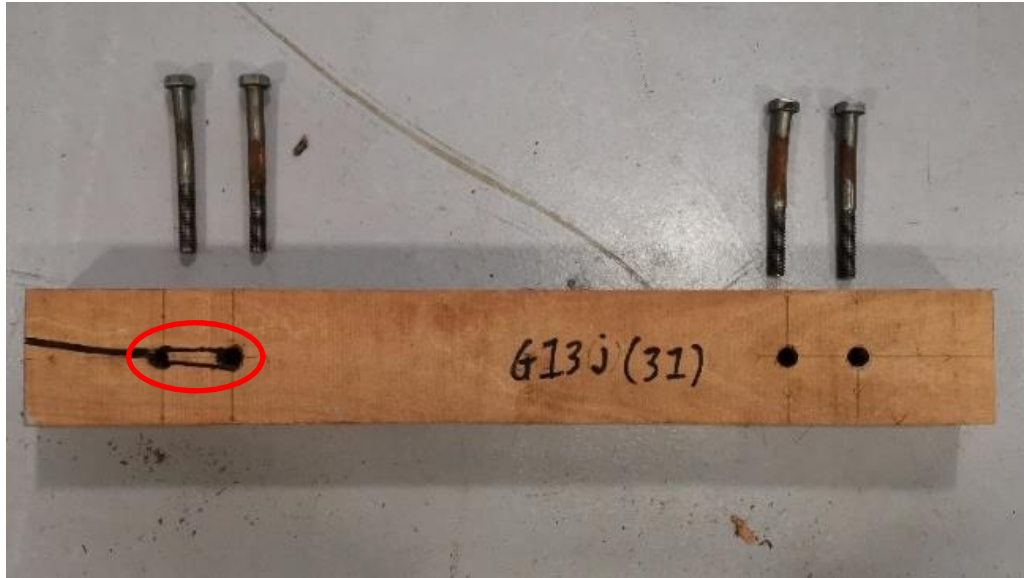


Figure 4.5: Group 13 Specimen Failed Primarily in Wood Row Shear Between the Bolt Spacing

4.4.2 Ultimate Strength of Connection

By referring to test results tabulated in Table 4.3, the ultimate strength of connections can be seen to increase significantly by the increment of bolt numbers (n_f). The increment of 5th percentile strength can be found when comparisons made between Group 1 (20 kN) and Group 6 (37 kN); Group 2 (19 kN) and Group 7 (28 kN); Group 3 (15 kN) and Group 8 (32 kN); Group 4 (16 kN) and Group 9 (36 kN); as well as Group 5 (12 kN) and Group 10 (24 kN). All connections of Groups 1-5 are in 1-bolt configuration and Groups 6-10 are in 2-bolt configuration. A similar trend can also be obtained when comparing the 2-bolt connections of Groups 13-15 with the 3-bolt connections of Groups 16-18, respectively. The increment of 5th percentile strength can be seen by the following comparisons: (i) Group 13 gives 33 kN, whereas Group 16 gives 51 kN; (ii) Group 14 gives 23 kN, whereas Group 17 gives 48 kN; and (iii) Group 15 gives 17 kN, whereas Group 18 gives 41 kN. This shows that the bolt numbers can significantly affect the ultimate strength of connections.

By considering a geometrical parameter effect of the end distance (e), specifically, one can see that the 5th percentile strength of bolted connections shown in Column 7 decreases as the end distance decreases. Group 1 with 150 mm end distance gives the highest strength of 20 kN when compared to the following: (i) Group 2 with 125 mm end distance gives 19 kN; (ii) Group 3 with 100 mm end distance gives 15 kN; (iii) Group 4 with 75 mm end distance gives 16 kN; and (iv) Group 5 with 50 mm end distance gives 12 kN. The effect of the end distance on the 5th percentile strength of the connections can also be observed in 2-bolt and 3-bolt configurations. When considering the bolt spacing (s_b), the 5th percentile strength was also found to be decreased with the decrement of the bolt spacing. This can be seen when comparing the followings: (i) Group 6 (37 kN) and Group 11 (23 kN); (ii) Group 7 (28 kN) and Group 12 (21 kN); (iii) (i) Group 9 (36 kN) and Group 14 (23 kN); and (iv) Group 10 (24 kN) and Group 15 (17 kN). However, a slight strength increment was found when a comparison made between Group 8 with 100 mm bolt spacing that gives 32 kN in connection capacity and Group 13 with 50 mm bolt spacing that gives 33 kN in connection capacity. This is mainly due to the slightly higher variation of experimental results (refer to the COV value) 13.56% obtained in Group 8, whereas 13.07% attained in Group 13. In general, the strength of bolted connections can be considerably influenced by both geometrical parameters of end distance and bolt spacing.

From the discussion made, the bolted connection ultimate strength can greatly be influenced not only by the number of bolts, but also by the configuration details of both the end distance and the bolt spacing. These findings are in line with the previous studies conducted by Quenneville and Mohamad (2000), Mohammad and Quenneville (2001), Abdul Karim et al. (2018), Abdul Karim et al. (2021), and Abdul Karim et al. (2022). The strength values of all timber bolted connections tested can be found in Appendix F.

Table 4.3: Test Results and Prediction Values by EYM, RSM and MS

Group	e (mm)	s _b (mm)	n _f	Experimental values			European Yield Model (EYM)					Row Shear Model (RSM)	MS544-5 (MS)	Failure Mode
				R _{avg} (kN)	CoV (%)	R _{5th%} (kN)	R _{D1} (kN)	R _{D2} (kN)	R _{D3} (kN)	R _{D4} (kN)	R _{D,min} (kN)	R _{r,rs} (kN)	Q _{n,min} (kN)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	150	-	1	24	8.63	20	499	15	46	20	15	34	5.48	B
2	125	-	1	24	12.45	19	499	15	46	20	15	28	5.48	B
3	100	-	1	23	20.25	15	499	15	46	20	15	23	5.48	B
4	75	-	1	22	16.01	16	499	15	46	20	15	17	5.48	B
5	50	-	1	19	21.44	12	499	15	46	20	15	11	5.48	RS
6	150	100	2	47	12.72	37	998	30	93	41	30	45	10.96	B
7	125	100	2	40	17.64	28	998	30	93	41	30	45	10.96	B
8	100	100	2	41	13.56	32	998	30	93	41	30	45	10.96	B
9	75	100	2	43	10.04	36	998	30	93	41	30	34	10.96	B
10	50	100	2	37	20.81	24	998	30	93	41	30.19	22.63	10.96	RS
11	150	50	2	36	22.08	23	998	30	93	41	30.19	22.63	10.96	RS
12	125	50	2	37	26.34	21	998	30	93	41	30.19	22.63	10.96	RS
13	100	50	2	43	13.07	33	998	30	93	41	30.19	22.63	10.96	RS
14	75	50	2	38	24.13	23	998	30	93	41	30.19	22.63	10.96	RS
15	50	50	2	32	27.34	17	998	30	93	41	30.19	22.63	10.96	RS
16	100	50	3	67	13.94	51	1498	45	139	61	45.28	33.94	16.44	RS
17	75	50	3	62	13.74	48	1498	45	139	61	45.28	33.94	16.44	RS
18	50	50	3	59	18.92	41	1498	45	139	61	45.28	33.94	16.44	RS

4.4.3 Calibration Factor

To optimize the use of RSM (Equation 2.12) in predicting the bolted connection strength of *Nyatoh* hardwood, a calibration factor must be determined. An analysis was done from connection groups that is having an end spacing (e) or a bolt spacing (s_b) equal to 50mm, which are G5 and G10-G18. A mean oven dry density (ρ_{avg}) was applied into Equation 2.12. The calculated average predicted strength values against average experimental results with one-to-one ratio line was plotted. By using a regression method, a linear line of the plotted data was established as shown in Figure 4.6.

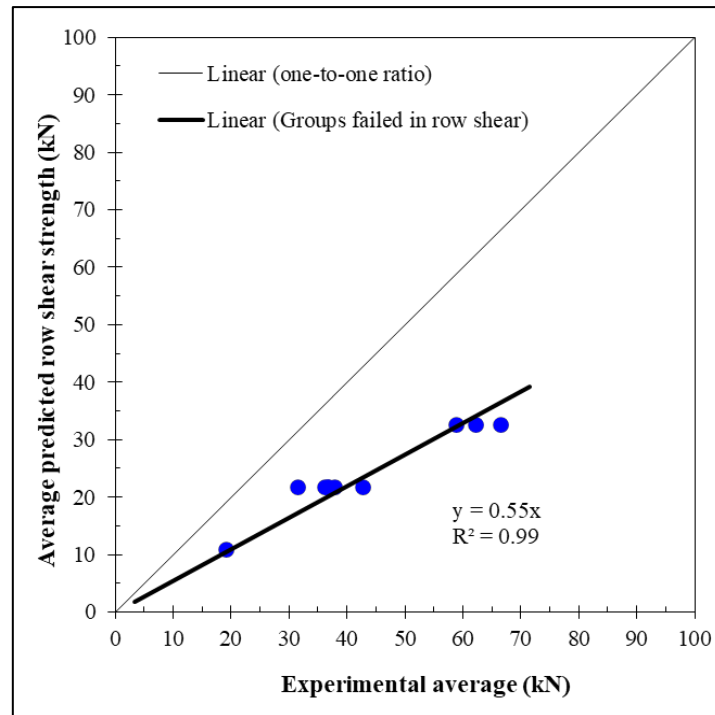


Figure 4.6: Graph of Average Predicted Row Shear Strength (kN) versus Experimental Average (kN)

The one-to-one ratio line or the 45° line was taken as a reference line for comparison with the linear regression line. A few iterations by means of trial and error were done to obtain the calibration factor (CF). This method is in line with a study by Abdul Karim et al.

(2022) on determination of CF. After a few trial-and-error series, a calibration factor of 4 was determined with a coefficient of determination (R^2) of 0.99 was obtained. In the analysis, 55% similarities between the average RSM predictions and the average experimental data are recommended. This was done to avoid overpredicting of the 5th percentile strength by the RSM equation.

4.4.4 Experimental versus Prediction Results

Table 4.3 demonstrates the calculated value of prediction results using a formula from European Yield Model (EYM) in Columns 8-12; Row Shear Model (RSM) in Column 13 and Malaysian Standard (MS) in Column 14. The prediction strength (R_{D1} - R_{D4} , in Columns 8-11) was calculated using EYM Equations 2.8-2.11 for double shear connection given in Table 2.6. $R_{D, \min}$ (in Column 12) is the lowest predicted value among the values in Columns 8-11. Column 13 provides the predicted RSM values using Equation 2.12. Average experimental values, R_{avg} , were determined in Column 5, whereas the 5th percentile strength values were calculated in Column 7, by assuming a normal distribution of experimental results obtained. Column 15 indicates the failure modes that were observed in each group, either B denotes as a bearing failure and RS denotes as a row shear failure. The calibration factor that was obtained earlier and mentioned in the previous section will be applied to the RSM equation to get the ultimate strength of the bolted connection.

To test the accuracy of the expressions from EYM, RSM and MS544, a comparison was made by tabulating the 5th percentile prediction values (Columns 3-5) against 5th percentile experimental values (Column 2) given in Table 4.4. Table 4.4 shows the effectiveness ratios of the EYM, RSM and MS predictions over the experimental results in Columns 6-8. Column 9 classifies the failure mode observed on the specimens after testing.

Table 4.4: Ratios of EYM, RSM and MS Predictions Over Test Results

Group	$R_{5th\%}$ (Exp.) (kN)	$R_{D,min}$ (EYM) (kN)	$R_{r,rs}$ (RSM) (kN)	$Q_{n,min}$ (MS) (kN)	$\frac{R_{D,min}}{R_{5th\%}}$	$\frac{R_{r,rs}}{R_{5th\%}}$	$\frac{Q_{n,min}}{R_{5th\%}}$	Failure Mode
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	20	15	34	5.48	0.75	1.26	0.27	B
2	19	15	28	5.48	0.78	1.09	0.28	B
3	15	15	23	5.48	1.00	1.12	0.36	B
4	16	15	17	5.48	0.93	0.78	0.34	B
5	12	15.09	11.31	5.48	1.22	0.68	0.44	RS
6	37	30	45	10.96	0.82	0.92	0.30	B
7	28	30	45	10.96	1.07	1.20	0.39	B
8	32	30	45	10.96	0.96	1.08	0.35	B
9	36	30	34	10.96	0.85	0.71	0.31	B
10	24	30.19	22.63	10.96	1.25	0.70	0.45	RS
11	23	30.19	22.63	10.96	1.31	0.74	0.48	RS
12	21	30.19	22.63	10.96	1.45	0.82	0.53	RS
13	33	30.19	22.63	10.96	0.90	0.51	0.33	RS
14	23	30.19	22.63	10.96	1.32	0.74	0.48	RS
15	17	30.19	22.63	10.96	1.74	0.98	0.63	RS
16	51	45.28	33.94	16.44	0.88	0.50	0.32	RS
17	48	45.28	33.94	16.44	0.94	0.53	0.34	RS
18	41	45.28	33.94	16.44	1.12	0.63	0.41	RS

For a better comparison, a graphical effectiveness ratio plotting was made as shown in Figures 4.7 and 4.8 for ductile failure mode and brittle failure mode, respectively. Any predictions that fall below the 45° line were considered safe, whereas predictions above the

line were considered unsafe. In other words, the experimental results obtained were utilized as the basis to validate the predictions given by the existing equations for the design of timber bolted connection strength. However, it should be noted that the closer the predictions to the 45° line, the better the effectiveness of the design equations.

In Figures 4.7 and 4.8, all 5th percentile strength predictions given by the MS544-5 (2001) were found to be far below the 45° line or too conservative when compared with the 5th percentile experimental values. The use of this standard can cause an overdesign of bolted connections for the purpose of retrofitting the wall-diaphragm connections in the unreinforced masonry buildings. From Column 8 of Table 4.4, the percentage of effectiveness provides by the MS544-5 (2001) was between 27% to 39% for the ductile failure mode, and between 32% to 63% for the brittle failure mode. For ductile failure mode, EYM was found to provide better 5th percentile strength predictions compared to the experimental 5th percentile strength. From Column 6 of Table 4.4, the percentage of accuracy was between 75% and 107%. A 7% of over-predicted was observed in Group 7 when using the EYM, however, this can be avoided by a reduction of the embedding strength (f_h) of *Nyato* hardwood. It should be noted that these 8 groups of Groups 1-4 and Groups 6-9 were chosen for this comparison as the failure mode observed in those groups was bearing failure, which the EYM equation is for the design of timber bolted connection considering the failure in bearing. For brittle failure mode, the percentage of effectiveness between 50% to 98% was found to be provided by the RSM equation by comparing the 5th percentile strength of RSM and the experimental 5th percentile strength given in Column 7 of Table 4.4. This comparison covers Group 5 and Group 10-18, ten groups in total, where these groups of timber bolted connection fail primarily in the row shear failure mode.

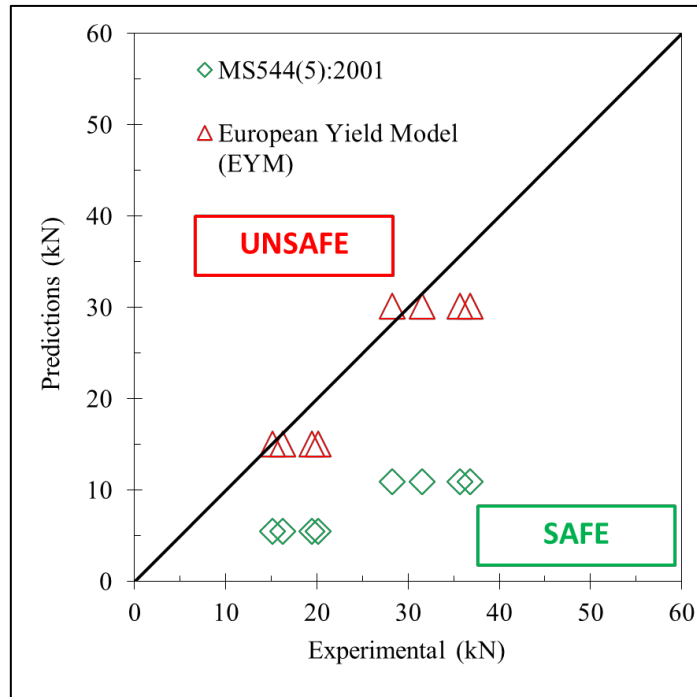


Figure 4.7: Effectiveness Comparison Between Using MS544-5 (2001) and European Yield Model (EYM) for Ductile Failure Mode

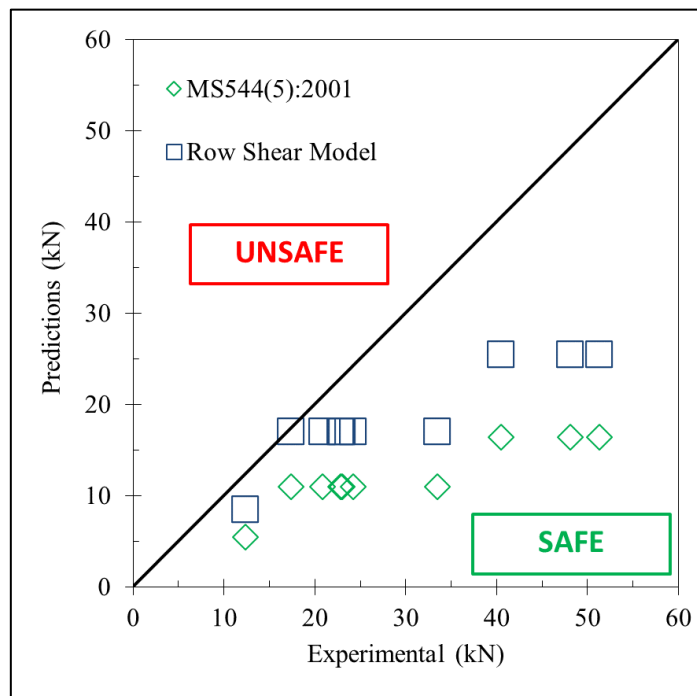


Figure 4.8: Effectiveness Comparison Between Using MS544-5 (2001) and Row Shear Model (RSM) for Row Shear Failure Mode

From the comparison made above, the purpose of evaluating the effectiveness of EYM, RSM and MS544-5 equations in predicting the strength of bolted connections on *Nyato* hardwood was achieved. As discussed earlier, the use of EYM is recommended, followed by RSM. However, the use of each design equations shall be preferable to be used on the connection modes that may yield certain types of failure. From the findings of previous studies (Abdul Karim et al., 2018; Abdul Karim et al., 2021; Abdul Karim et al., 2022), any end distance (e) or bolt spacing (s_b) of more than or equal to 75 mm, the bolted connections are likely to fail in bearing, whereas anything lower than or equal to 50mm tends to fail row shear. As such EYM shall be used to design a bolted connection with a configuration that expected to behave in a bearing failure mode, and RSM shall be used for a bolted connection with a configuration that predicted to act in a row shear failure mode. It should be noted that this finding is limited for a timber member with 50 mm thickness.

4.5 Summary

The findings of the present study can be summarised as follows: (i) Through the observations from the microscopic cross sections and the database obtained from the wood library of the Forest Department Sarawak, the timber specimen used in the current study was *Nyato* hardwood; (ii) Basic properties of *Nyato* hardwood, such as embedding strength and density, were determined because these properties are important to be used in the European Yield Model and Row Shear Model for predicting the bolted connection strength, respectively; (iii) The assessments of the strength and the possible failure modes of a bolted connection on *Nyato* hardwood were successfully determined. The strength of connections was significantly affected by both the bolt numbers and the geometrical parameters such as end distance and bolt spacing, whereas the failure modes of the connections were influenced

by the connection geometrical configuration details. The end distance or bolt spacing of less than or equal to 50 mm governs the brittle failure mode, whereas more than or equal to 75 mm governs the ductile failure mode; (iv) The Row Shear Model was optimised to predict the strength of the bolted connections on *Nyato* hardwood, and a calibration factor was determined; and (v) The prediction by MS544-5 was found to be too conservative in both ductile and brittle failure modes when compared to the actual results, whereas the European Yield Model and the Row Shear Model give a better prediction value for the bolted connections that fail in bearing (ductile) and row shear (brittle), respectively.

Based on the findings in the previous paragraph, the use of the European Yield Model and the Row Shear Model is recommended when designing the bolted connections for the purpose of retrofitting the wall-diaphragm connections in the unreinforced masonry buildings. Because both the European Yield Model and the Row Shear Model were optimized in this present study by providing the embedding strength and the calibration factor specifically for the *Nyato* hardwood, a conservative design of bolted connections can be omitted. This is because the conservative design can cause unnecessary high bolt numbers or an oversize bolt diameter to be used. Thus, an overdesign of bolted connections for retrofitting the wall-diaphragm connections of the unreinforced masonry buildings in Malaysia can be avoided. It should be noted that the calibration factor determination is specifically to be used for the *Nyato* hardwood and for the 50 mm thick timber members.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This chapter concludes the findings of the present study on assessing the effectiveness of the existing design equations of MS544, EYM and RSM in predicting the bolted connection strength on *Nyato* hardwood to produce an optimized bolted connection design for the purpose of URM building retrofitting works.

Through wood identification, the studied hardwood has been determined to be that of *Nyato* that belongs to the genus of *Palaquim spp.* or *Payena spp.* This identification eliminates any elements of doubt to the authenticity of hardwood being studied and this conclude objective (i) outlined in Section 1.4.

All bolted connection strengths consisted of eighteen groups that vary in bolt numbers, end distances and spacing between bolts were determined and the 5th percentile strength was calculated. It was observed that not only the bolt numbers give significant influence on the connection strength, but also the bolted connection configuration details of the end distance and the bolt spacing. These geometrical configuration details affect the way the connection to fail. It was found out that any connection with end spacing (e) or bolt spacing (s_b) more than or equal to 75 mm fails in bearing (ductile) while anything less than or equal to 50 mm resulted in row shear failure (brittle). The density test values were used for the equation of the Row Shear Model. A total of 200 specimens, irrespective of accepted or rejected specimens from the bolted connection tests, were tested and analyzed. The average dry density is 574.33 kg/m³, while the 5th percentile dry density is 457.25 kg/m³.

The moisture content of all specimens was determined to be 16% on average, thus, all timber samples are tested in dry condition. The embedding strength obtained was used to calculate bolted connection strength on *Nyato* hardwood using the European Yield Model equation. For embedding strength, a total of 183 specimens were tested, irrespective of rejected or repeated specimens from the bolted connection tests, the average embedding strength is 34.69 MPa while the 5th percentile value is 23.22 MPa. These findings all together with the results attained from the bolted connection tests, the moisture content and density tests conclude objective (ii) in Section 1.4.

By utilizing the bolted connection results for groups that failed in row shear, a calibration factor of four (4) was obtained. Using the density and the embedding strength results, the predictions of bolted connection strength values were determined by using the Row Shear Model and European Yield Model equations, respectively. From the validation of the effectiveness of the existing design equations including the MS544-5 (2001), the European Yield Model and the Row Shear model give better bolted connection strength predictions than Malaysia Standard when compared to the actual test results. Therefore, the use of the European Yield Model and the Row Shear Model equations is recommended in the design of bolted connection of *Nyato* hardwood, which these findings conclude objective (iii) of Section 1.4.

5.2 Recommendations for Future Works

To ensure the present study can be extensively used for further research on Malaysia local hardwoods, the listed recommendations can be considered for future studies:

- i. To conduct a double rows connection type for determining the possibility of the group tear out and its strength.

- ii. To perform investigations on other species of local hardwoods to further enhance the bolted connection data for validating the existing design equations.

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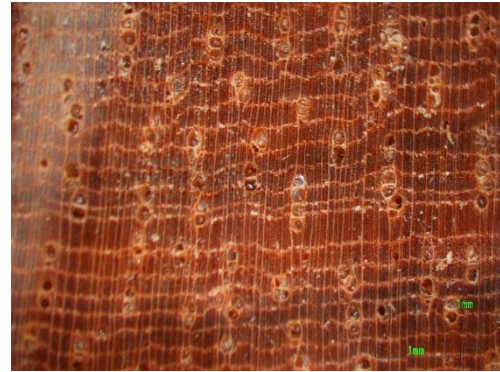
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APPENDICES

Appendix A: Microscopic Endgrain Sections of *Nyatoh* Hardwood Specimens.



Plank 1



Plank 2



Plank 3



Plank 4



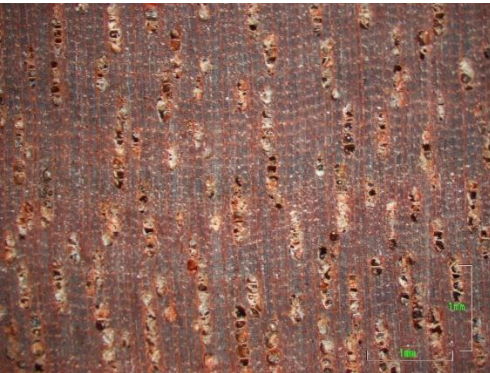
Plank 5



Plank 6



Plank 7



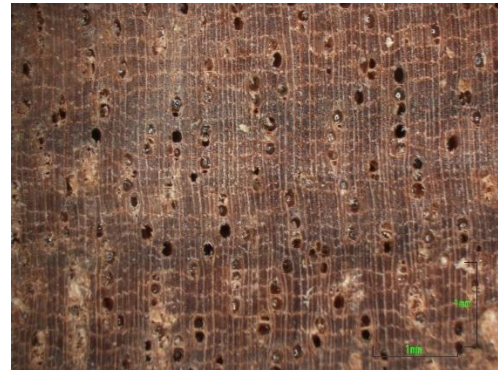
Plank 8



Plank 9



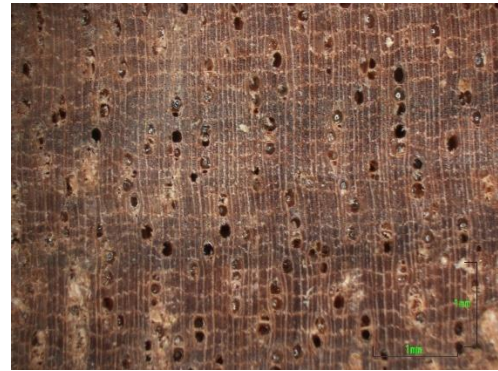
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Plank 11

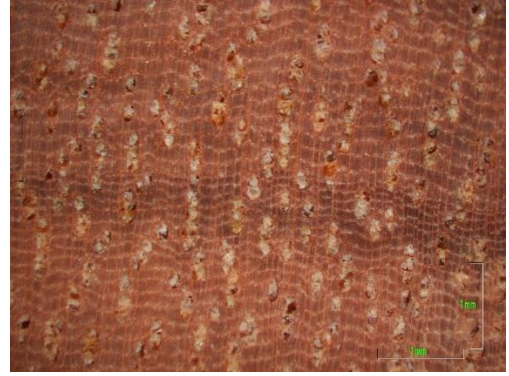


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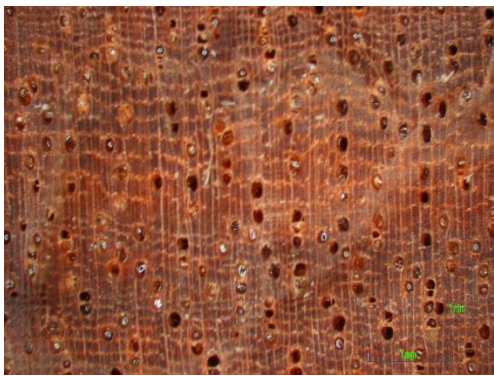




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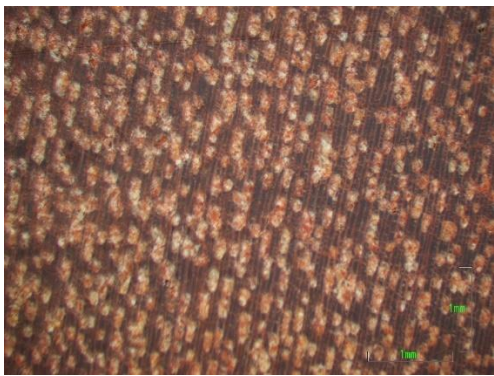
Plank 14



Plank 15



Plank 16



Plank 17



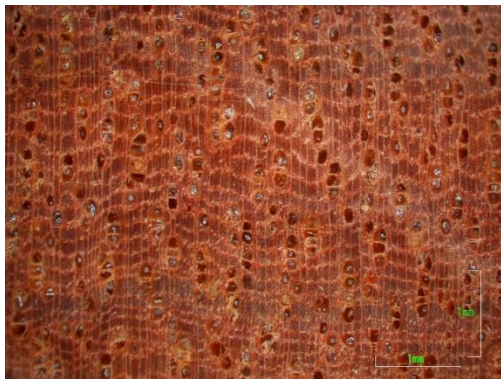
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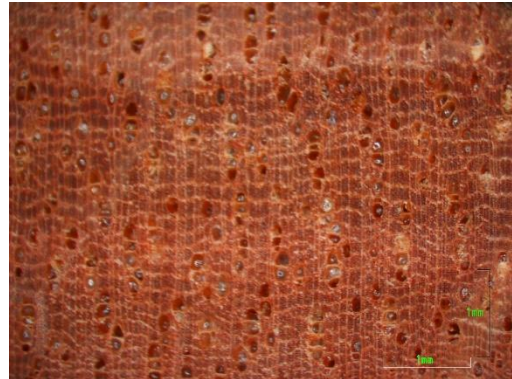
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Plank 20



Plank 21



Plank 22



Plank 23



Plank 24



Plank 25



Plank 26



Plank 27



Plank 28



Plank 29



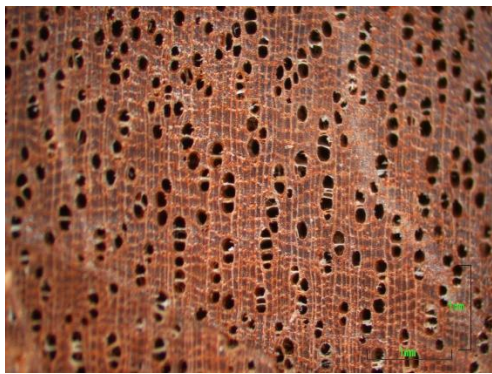
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Plank 31



Plank 32



Plank 33



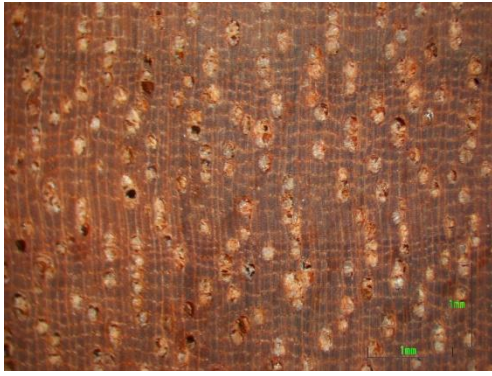
Plank 34



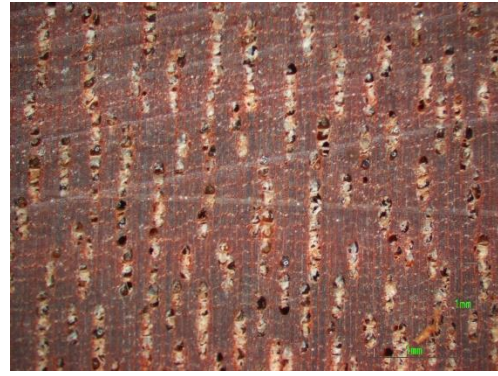
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Plank 36



Plank 37



Plank 38



Plank 39



Plank 40



Plank 41



Plank 42



Plank 43



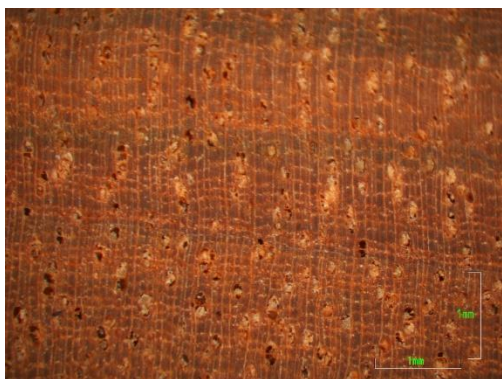
Plank 44



Plank 45



Plank 46



Plank 47



Plank 48



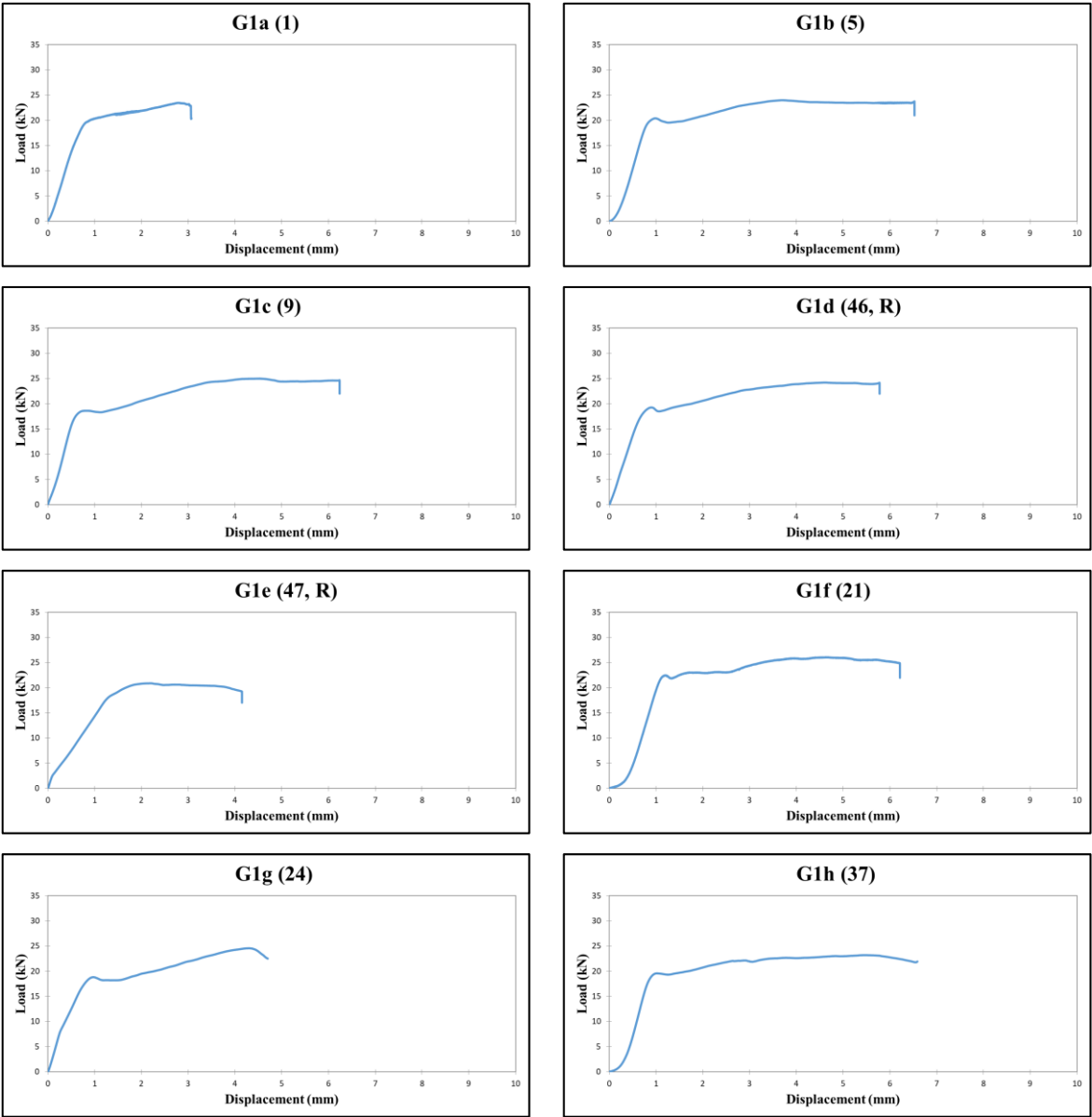
Plank 49

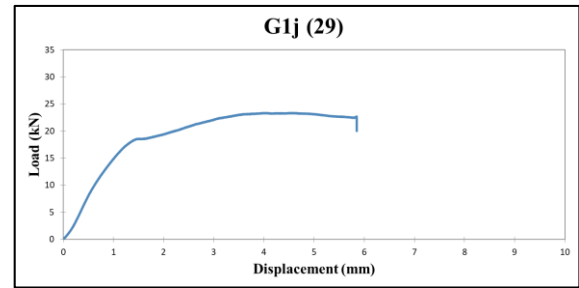
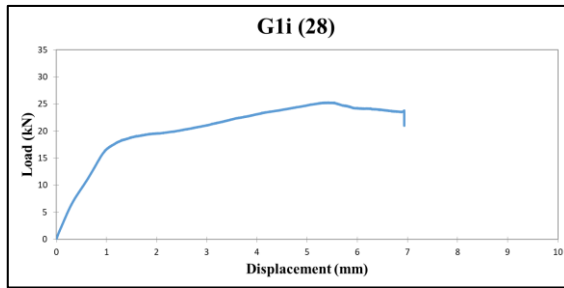


Plank 50

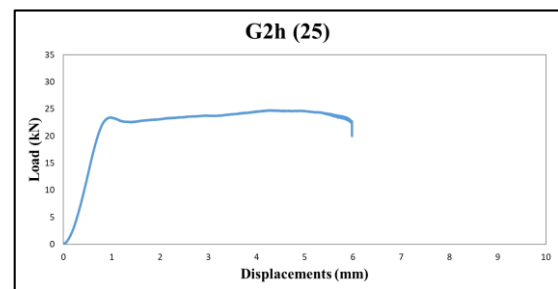
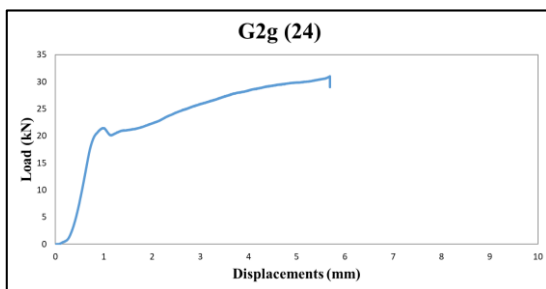
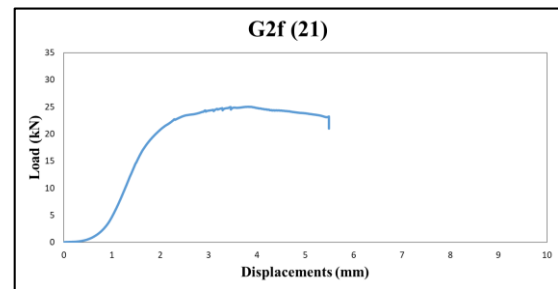
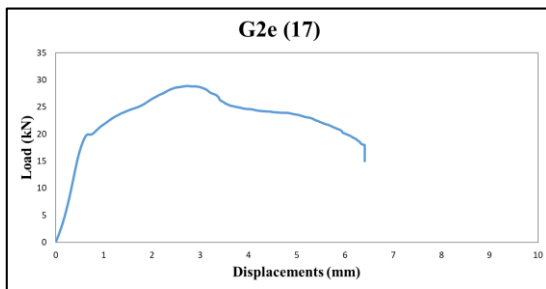
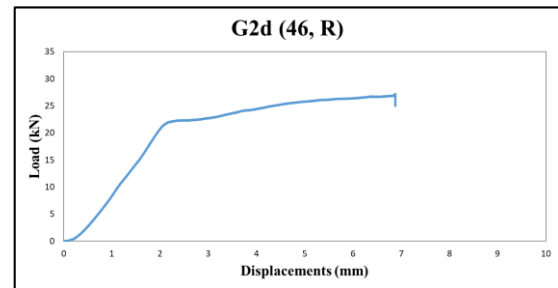
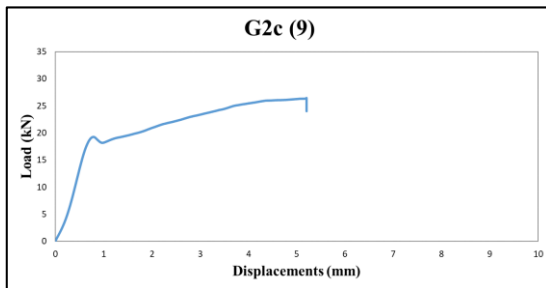
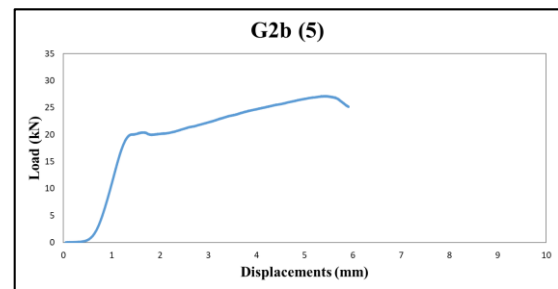
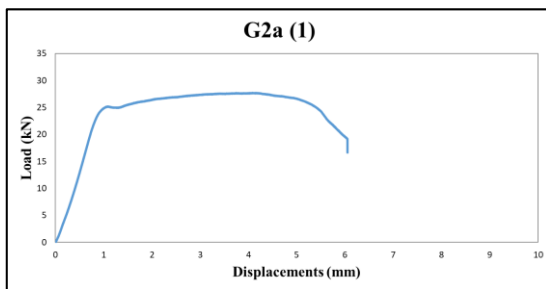
Appendix B: Load Versus Displacement Graphs of Embedding Strength Tests for *Nyatoh* Hardwood Specimens.

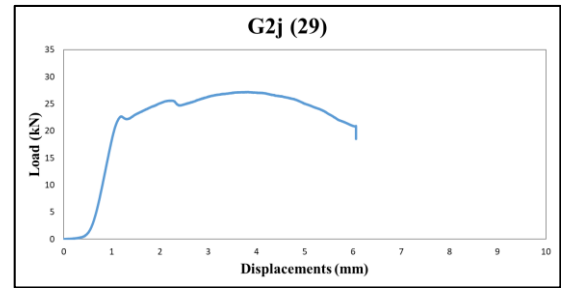
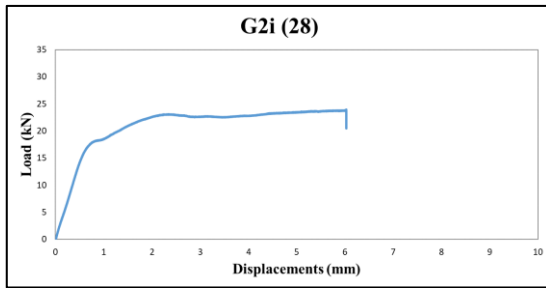
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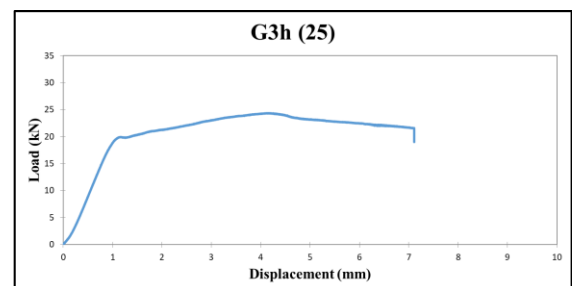
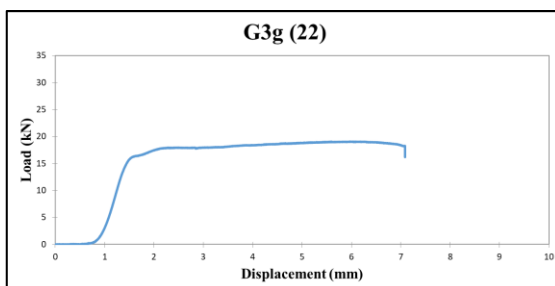
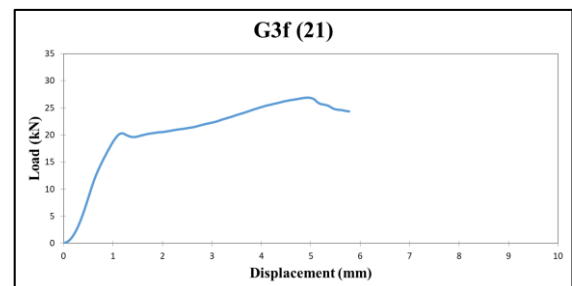
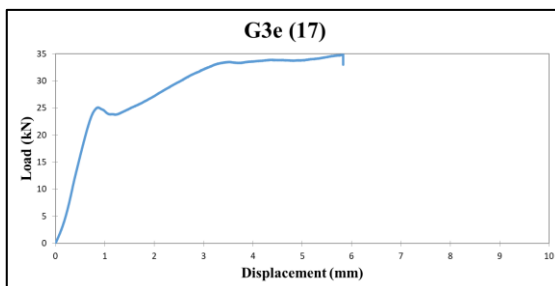
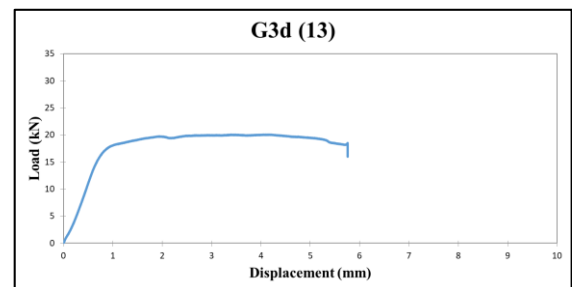
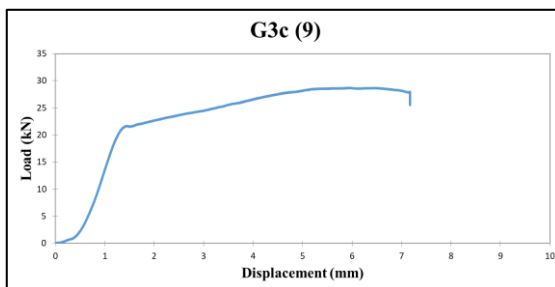
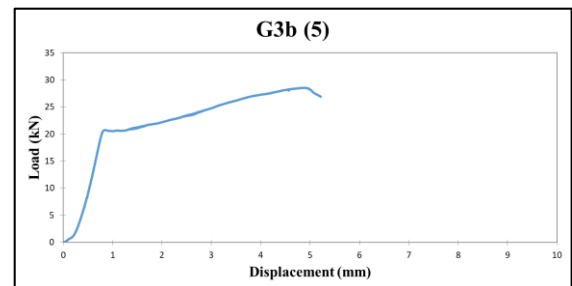
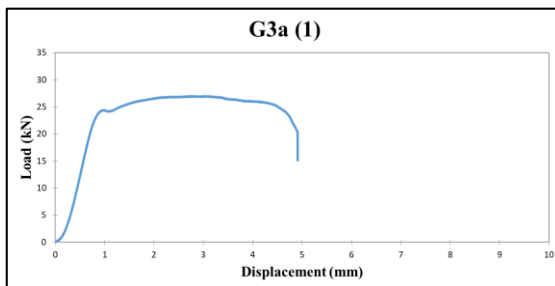


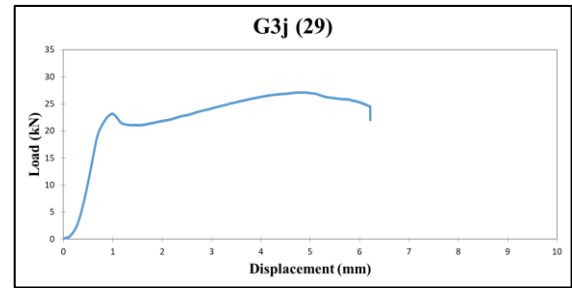
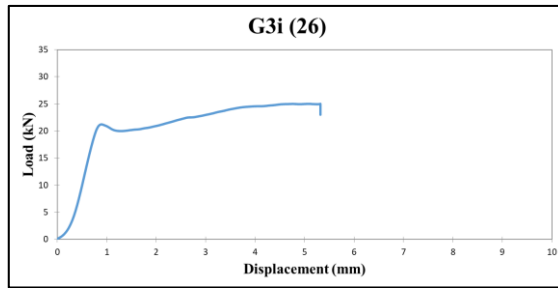
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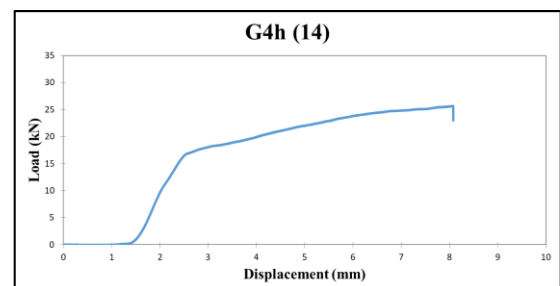
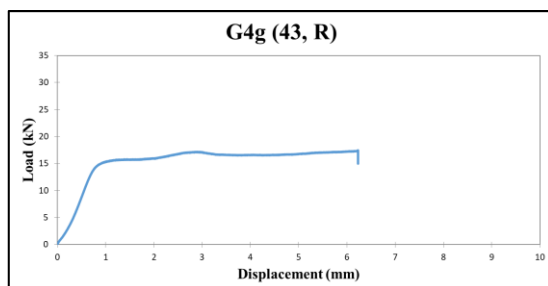
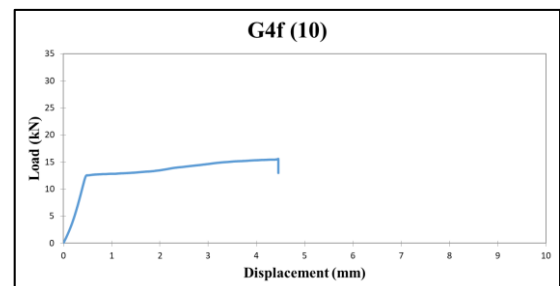
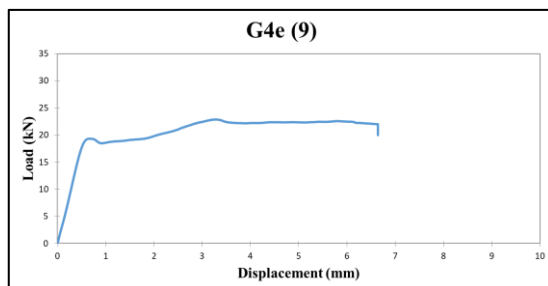
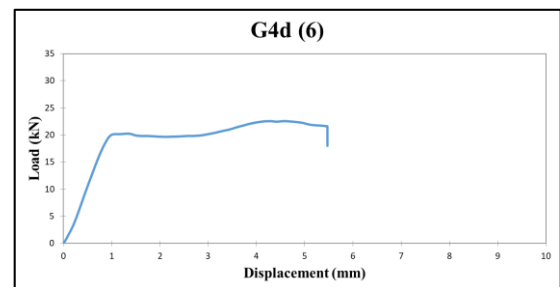
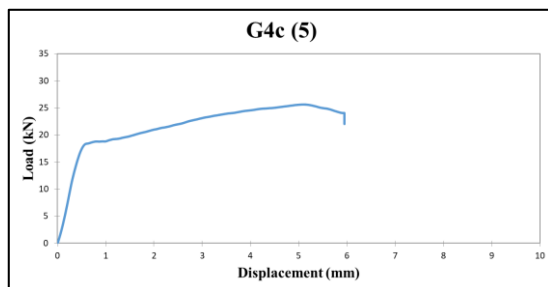
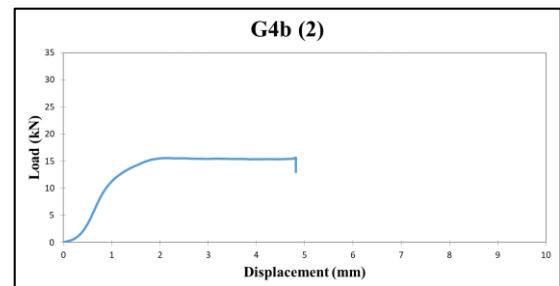
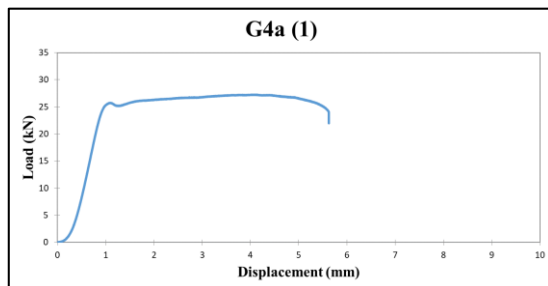


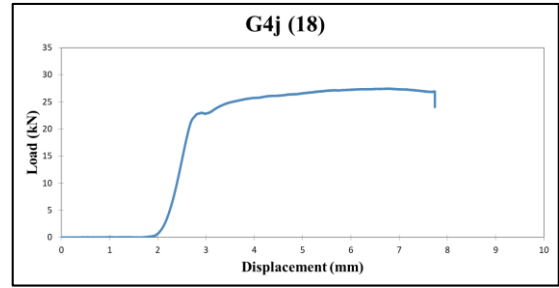
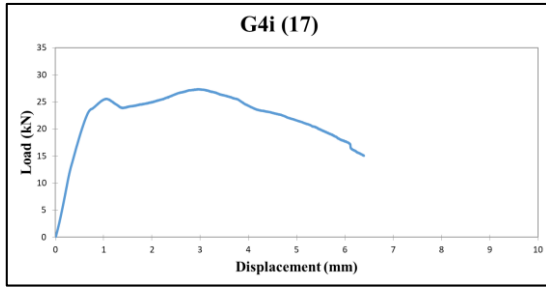
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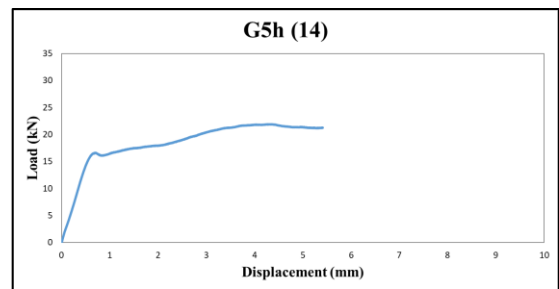
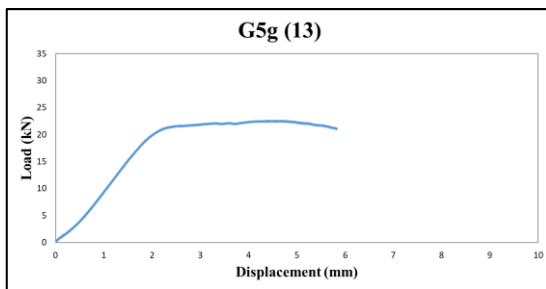
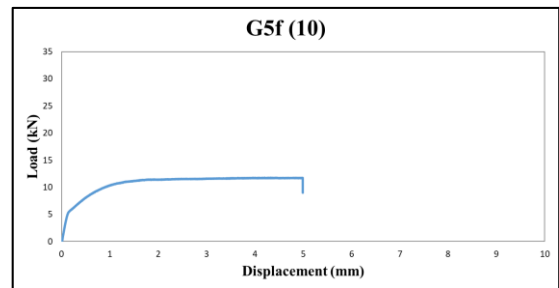
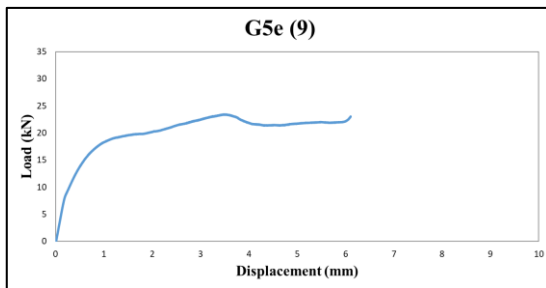
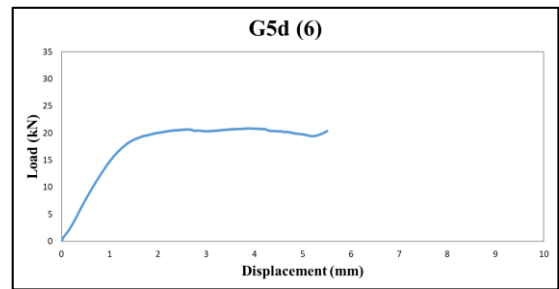
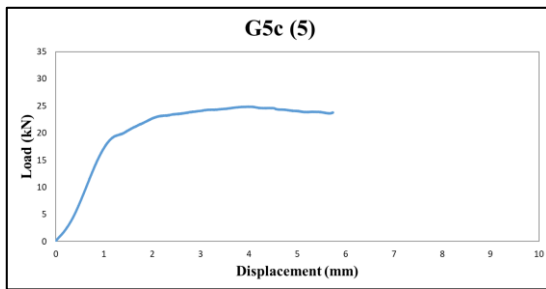
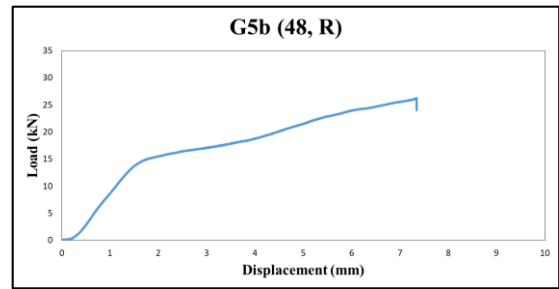
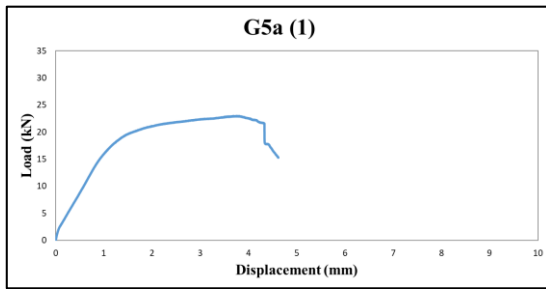


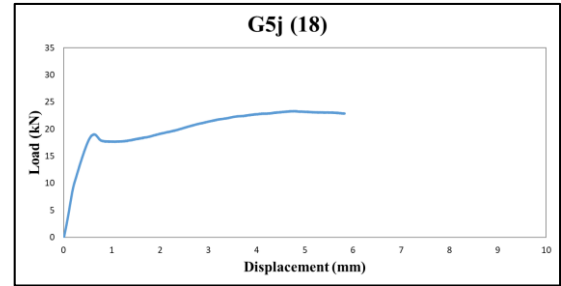
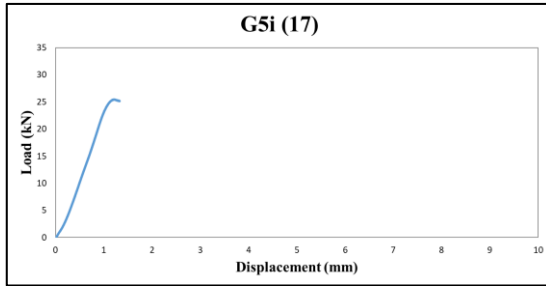
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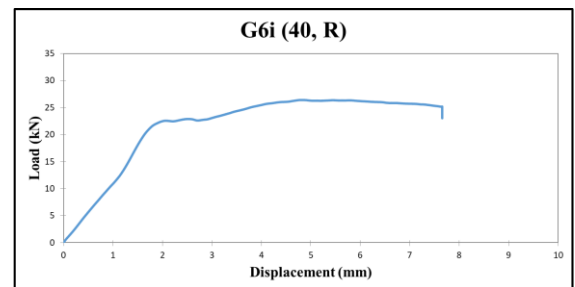
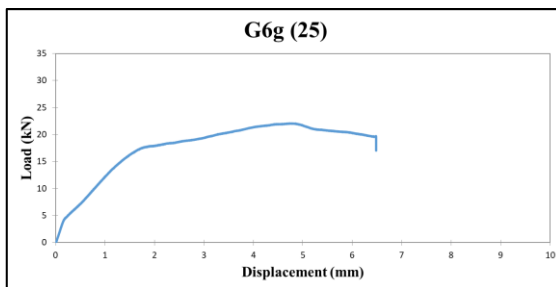
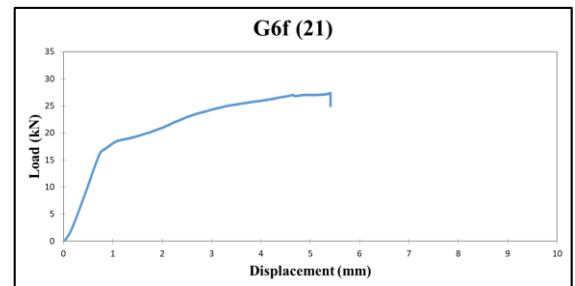
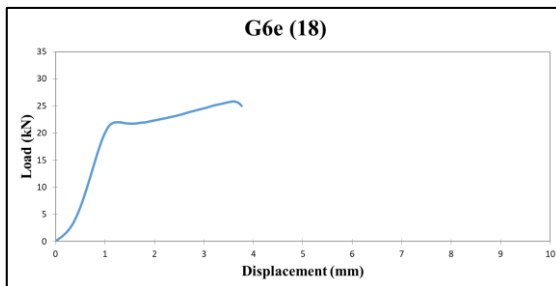
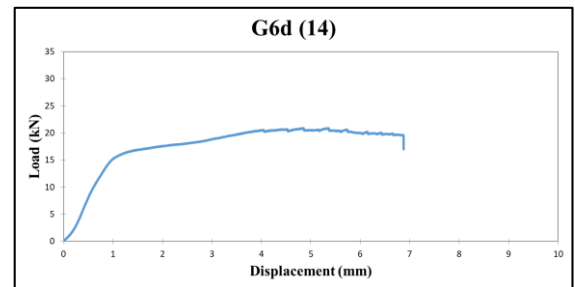
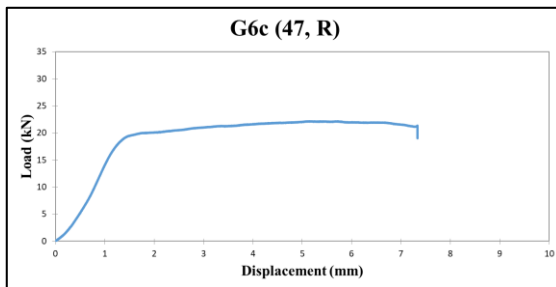
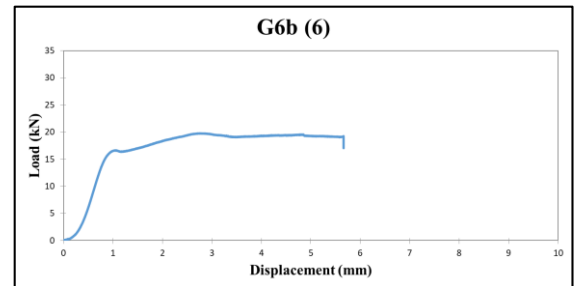
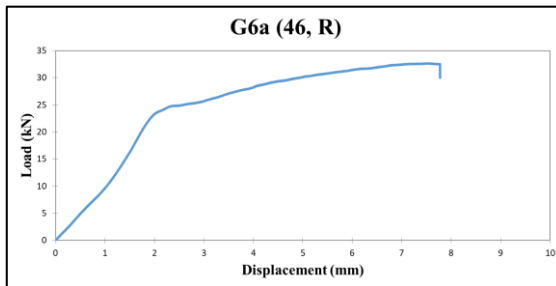


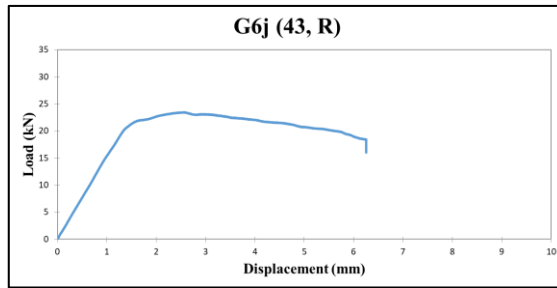
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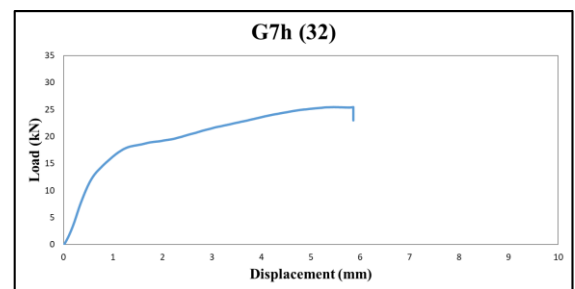
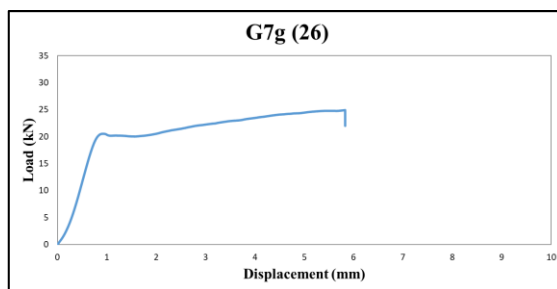
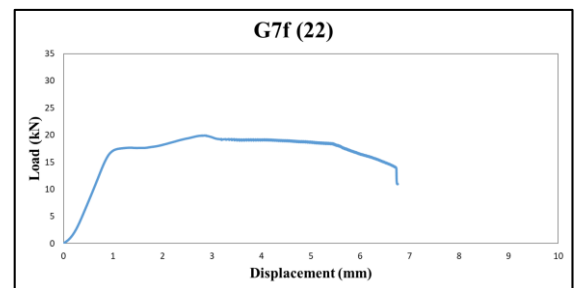
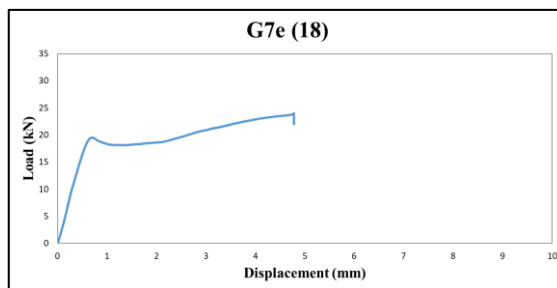
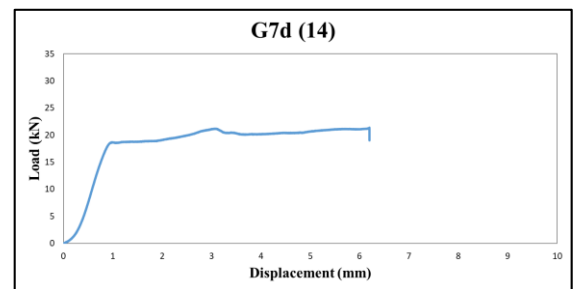
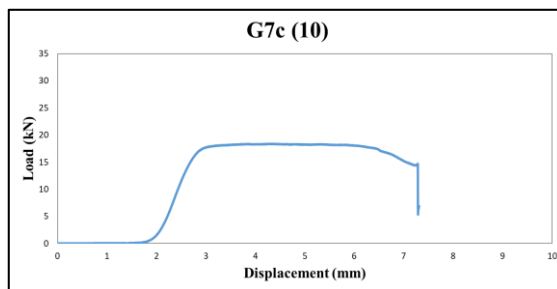
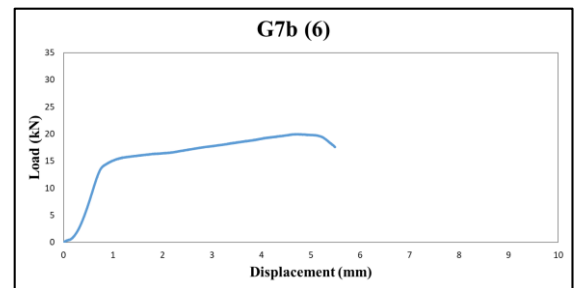
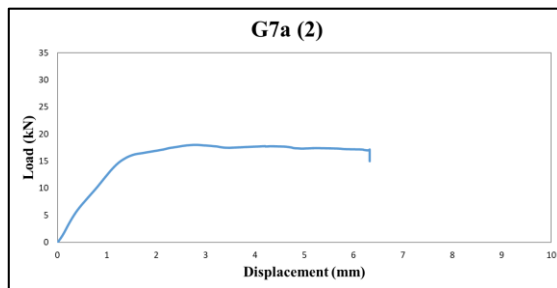


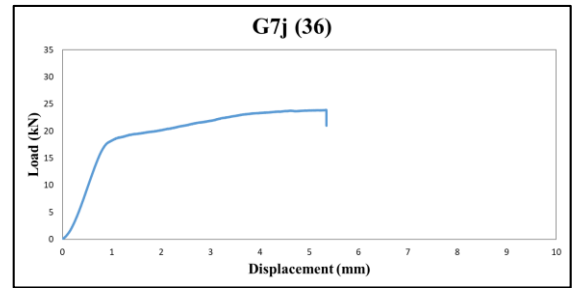
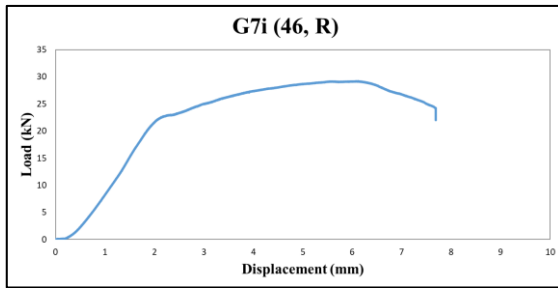
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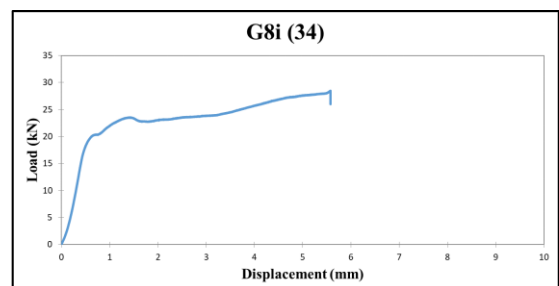
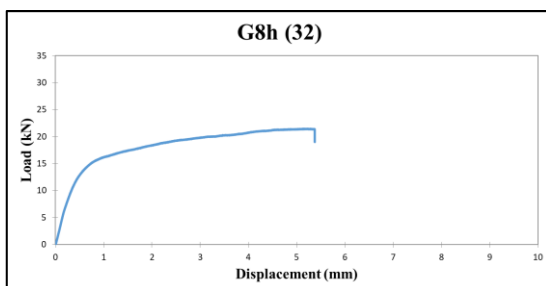
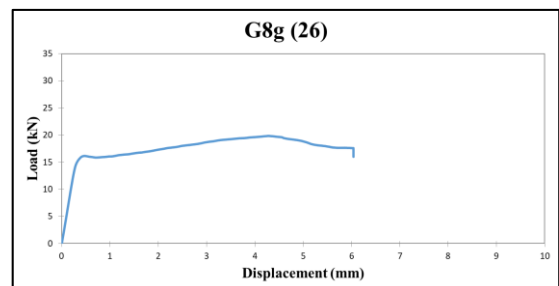
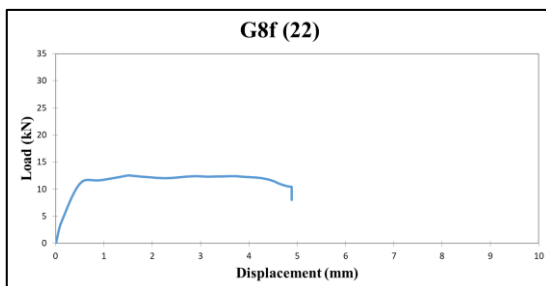
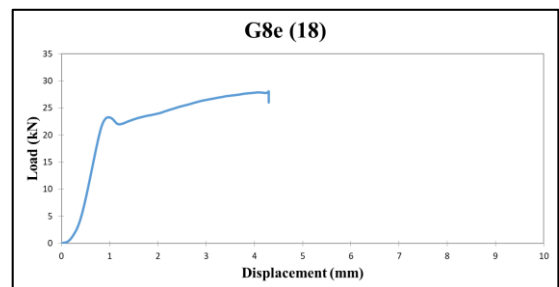
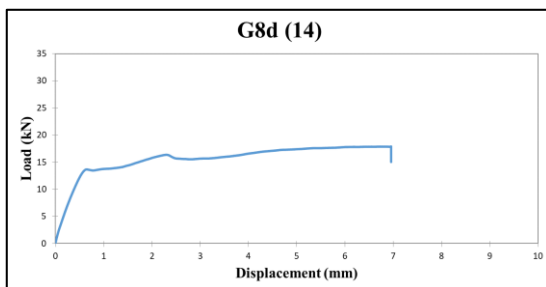
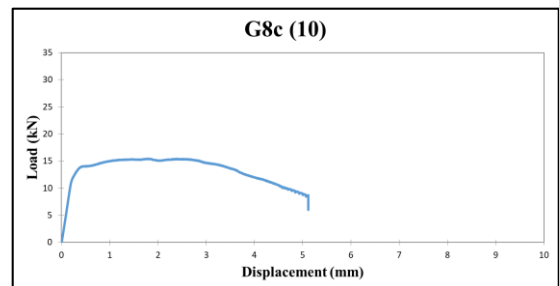
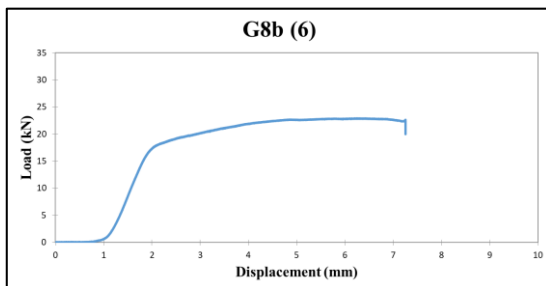


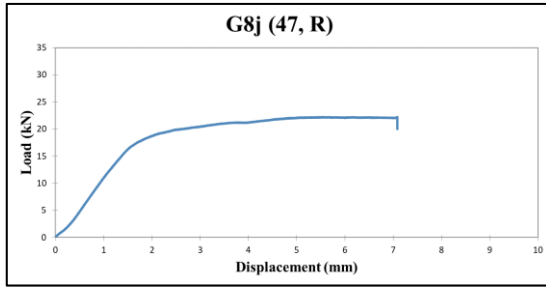
Group 7



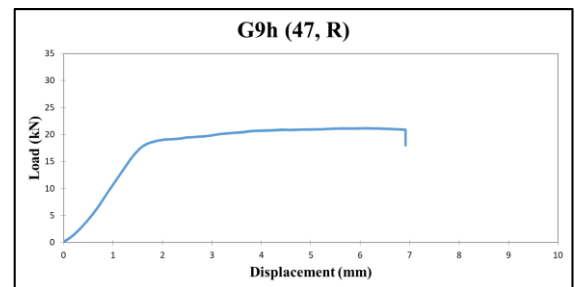
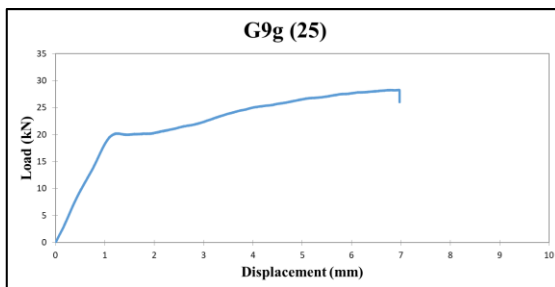
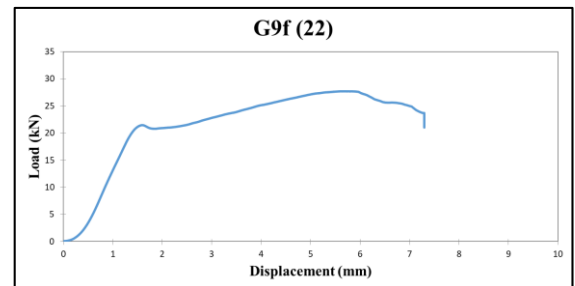
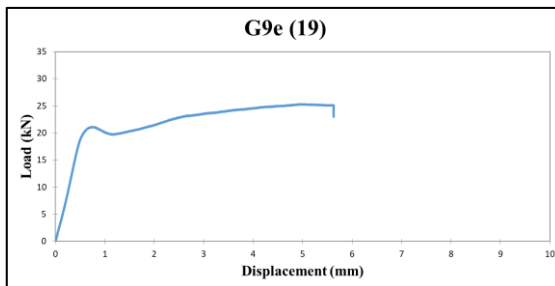
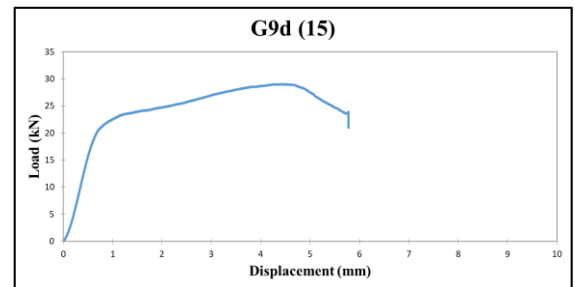
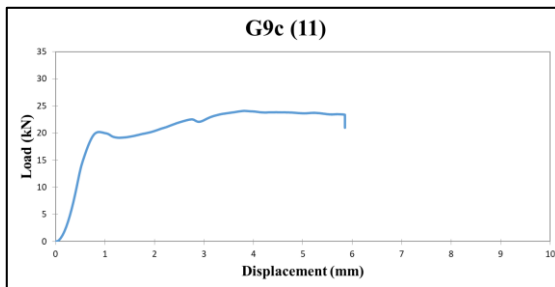
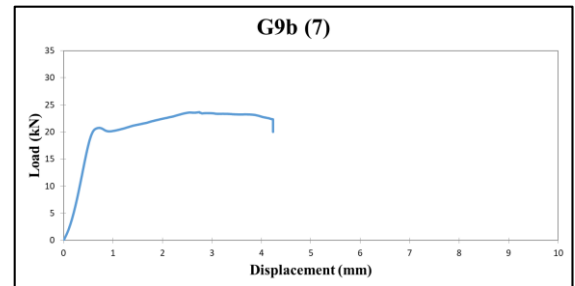
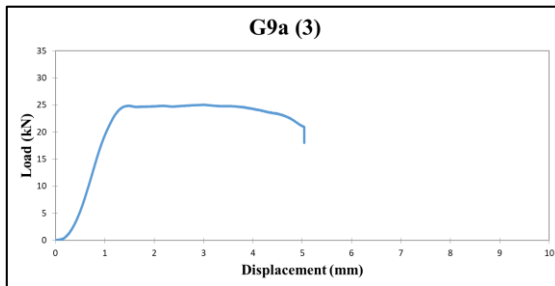


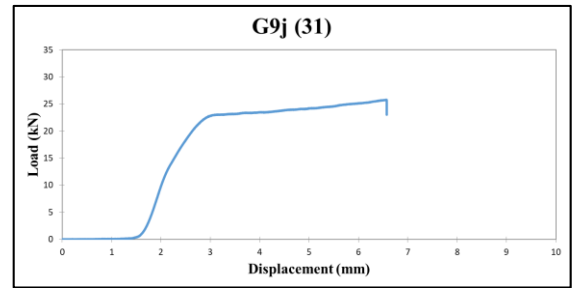
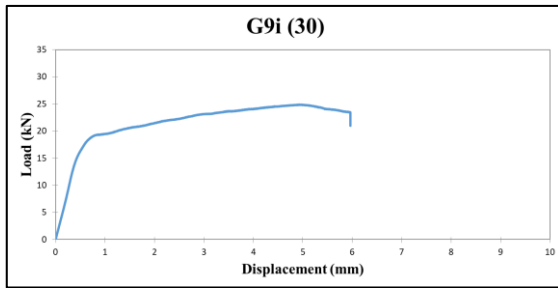
Group 8



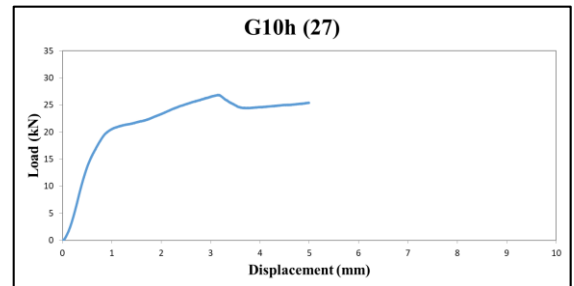
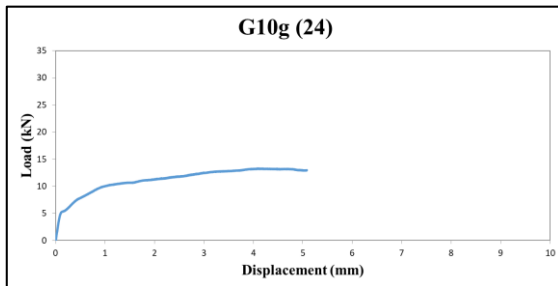
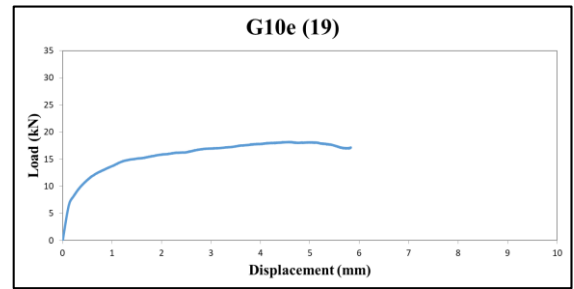
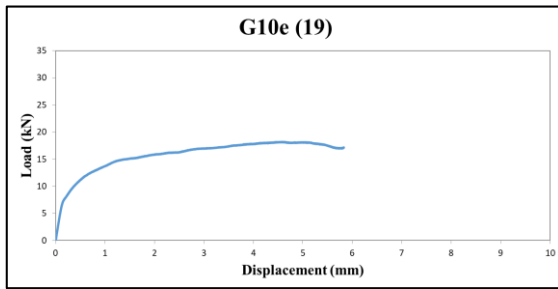
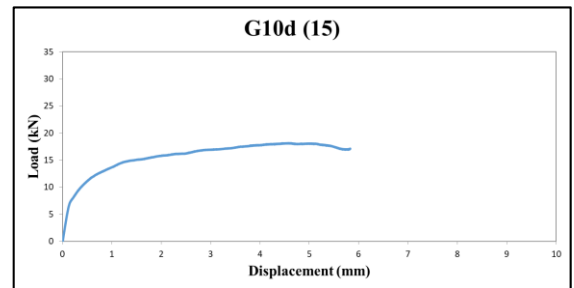
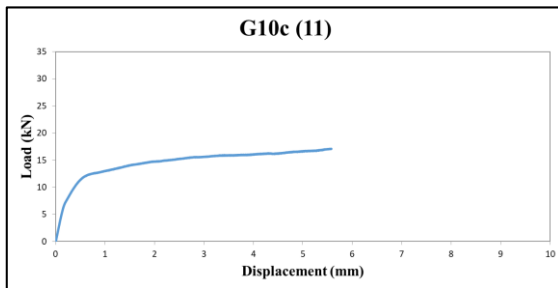
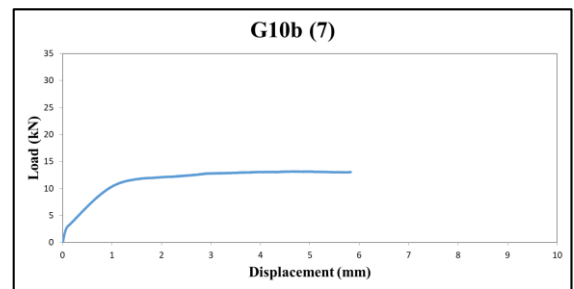
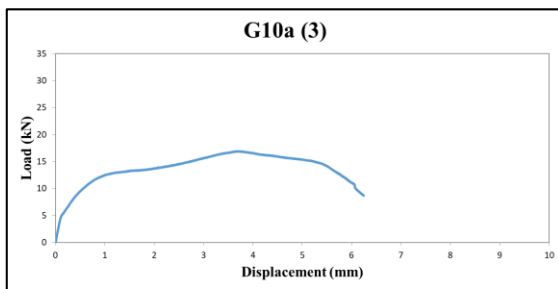


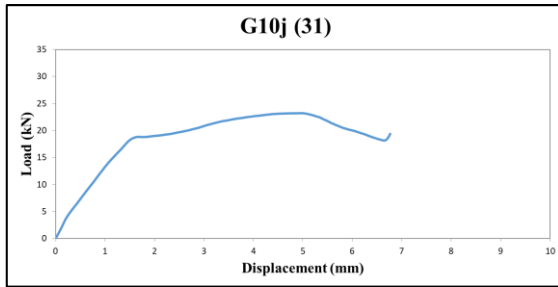
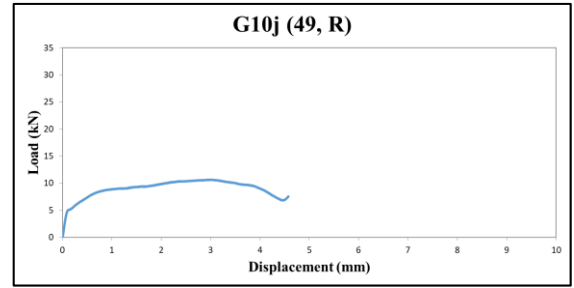
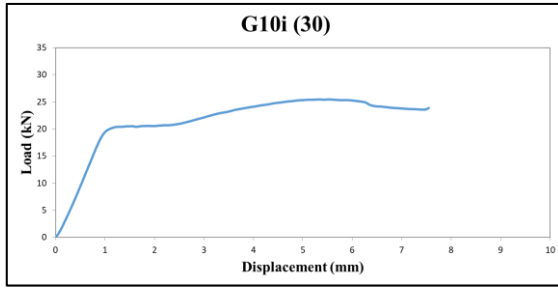
Group 9



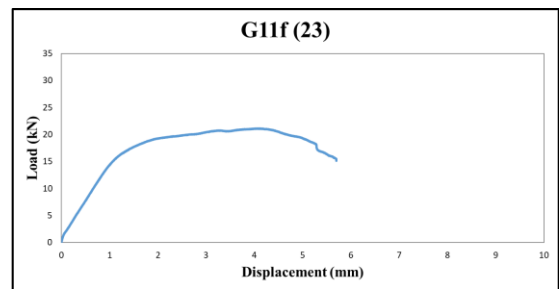
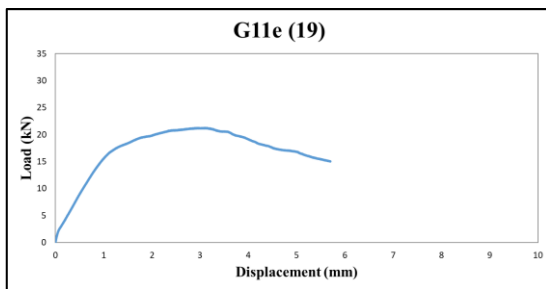
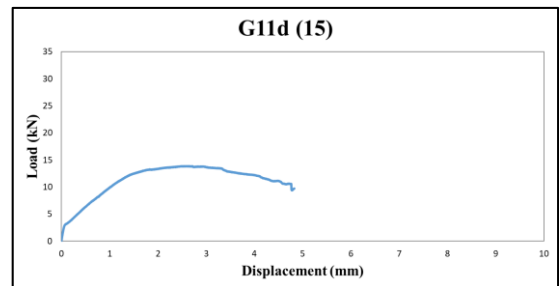
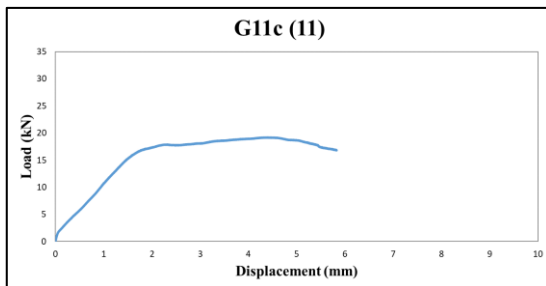
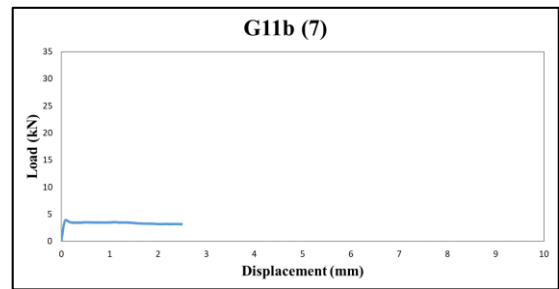
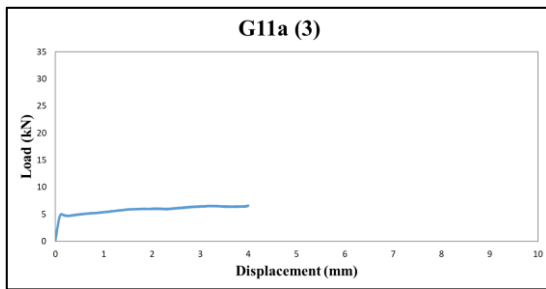


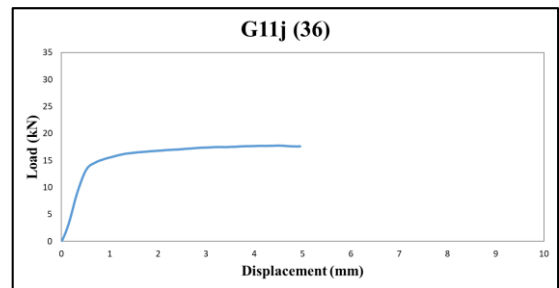
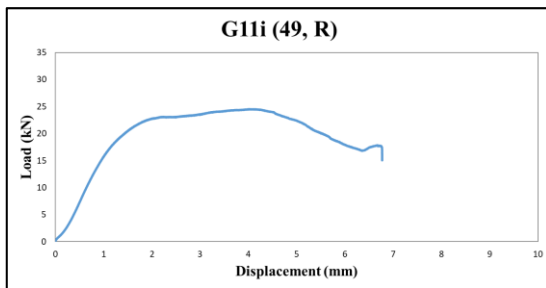
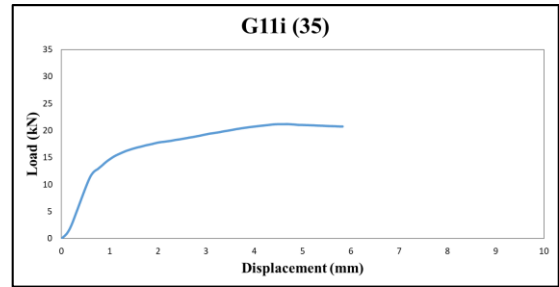
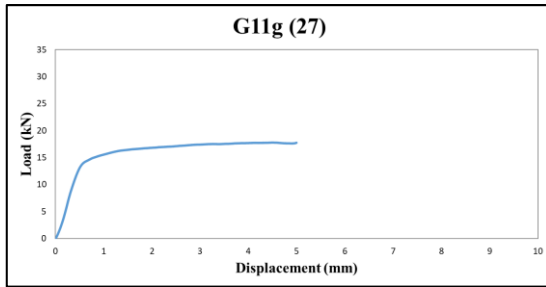
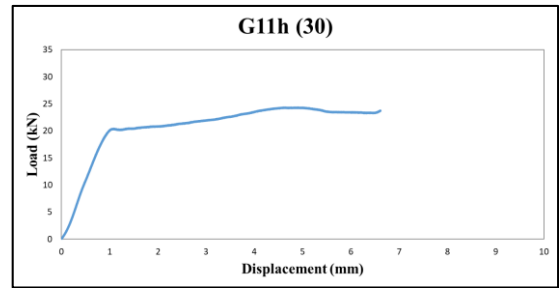
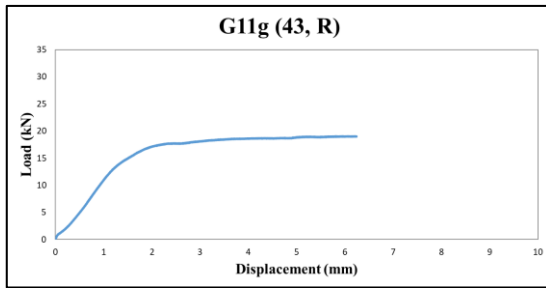
Group 10



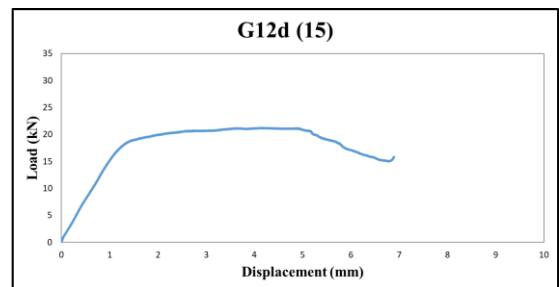
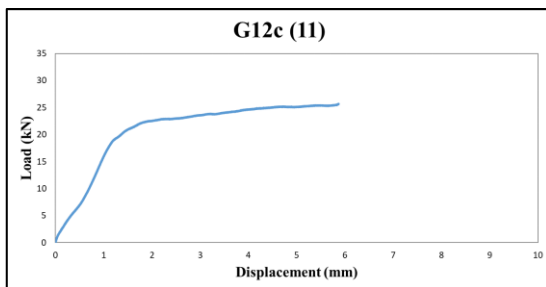
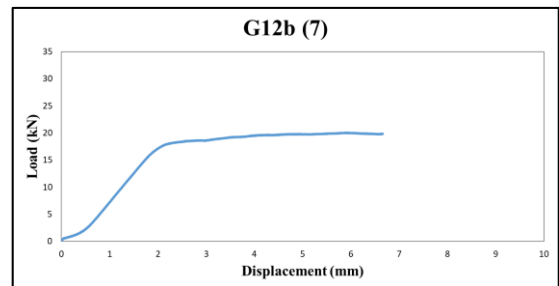
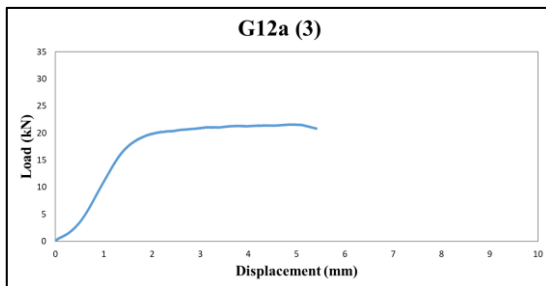


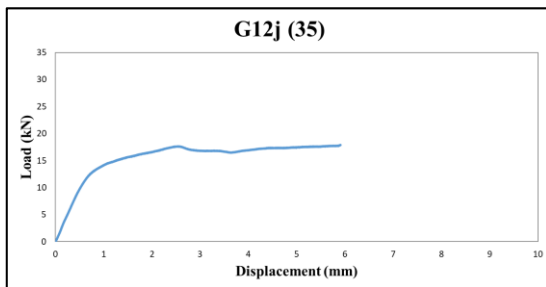
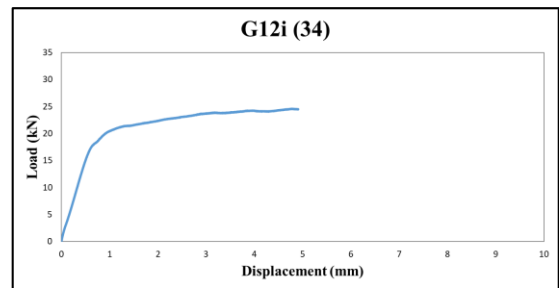
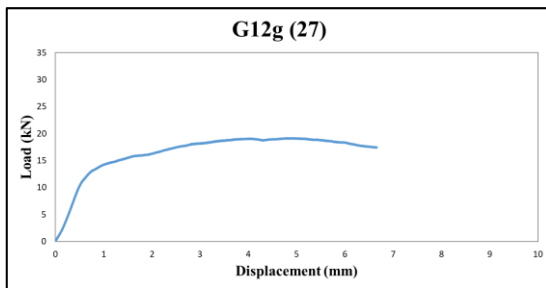
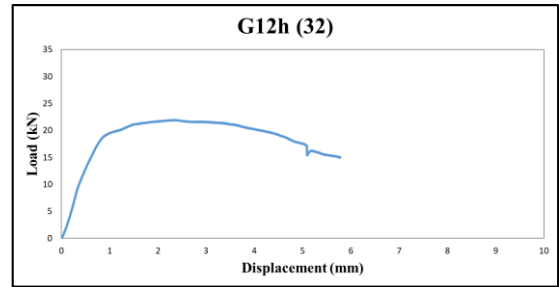
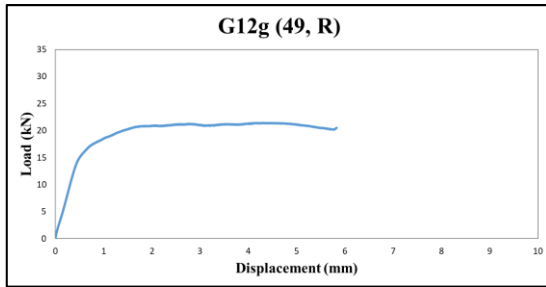
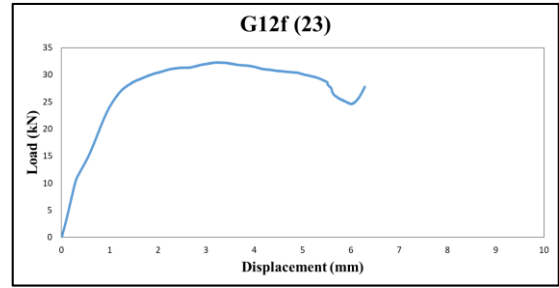
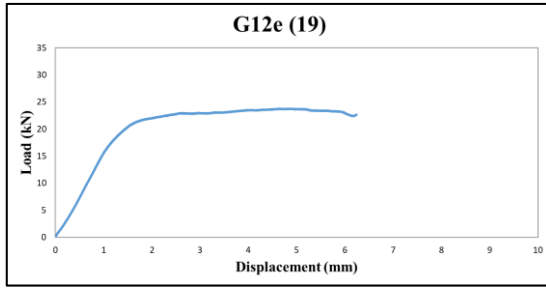
Group 11



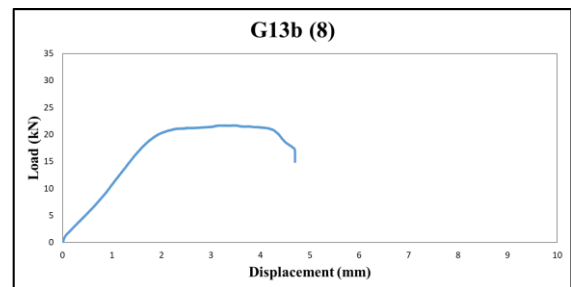
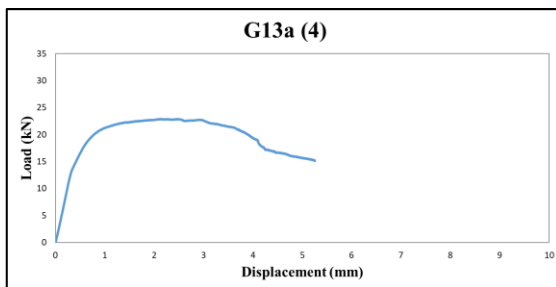


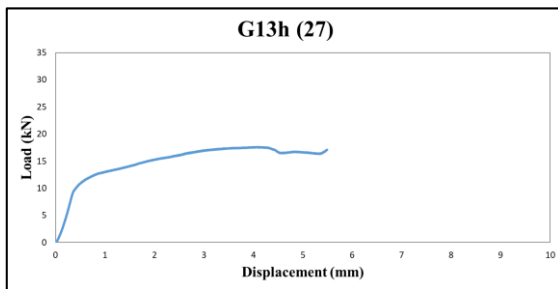
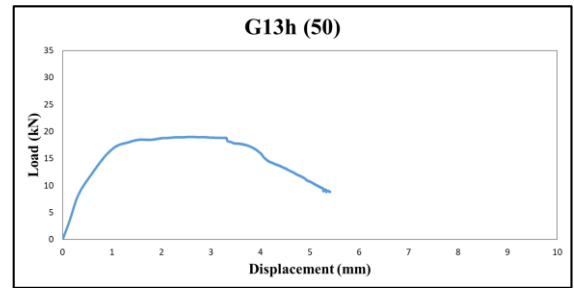
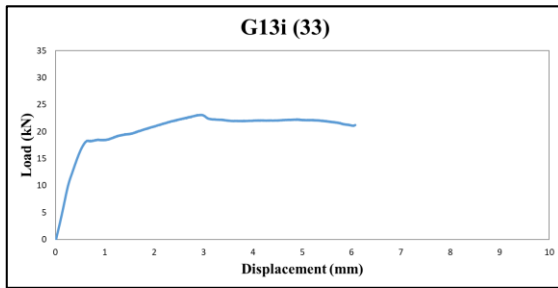
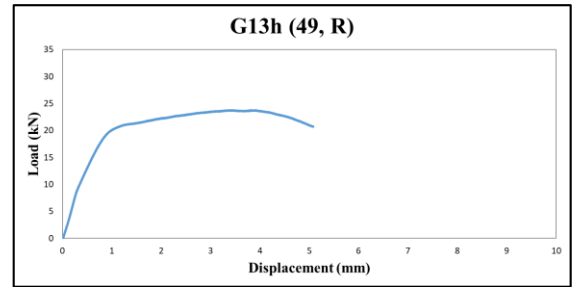
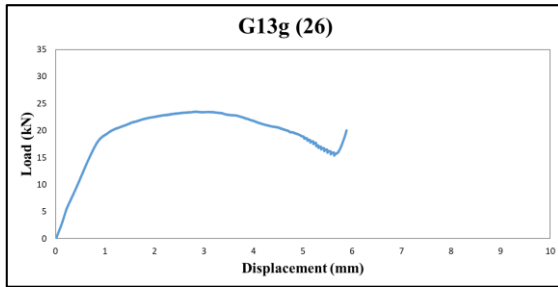
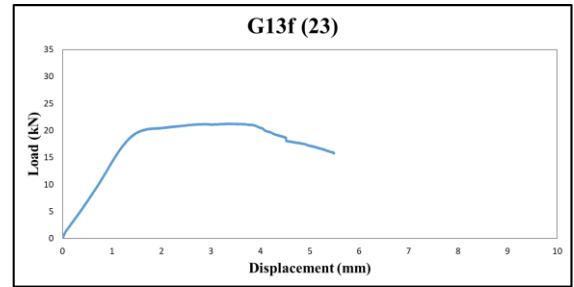
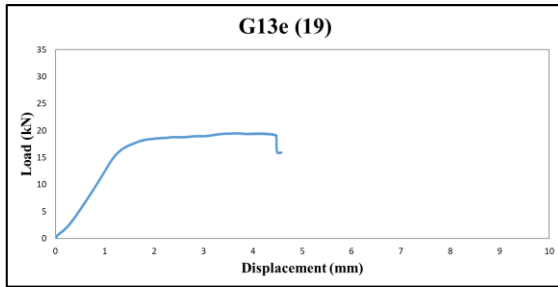
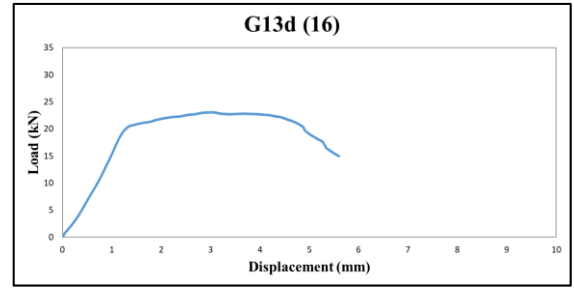
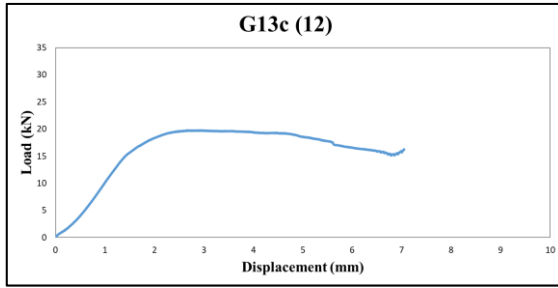
Group 12



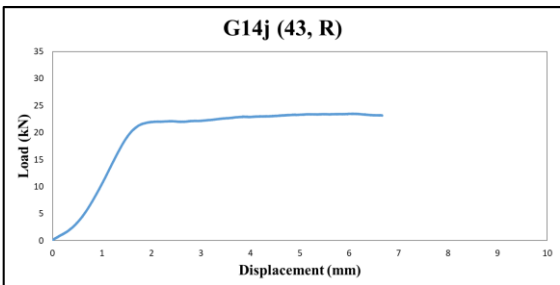
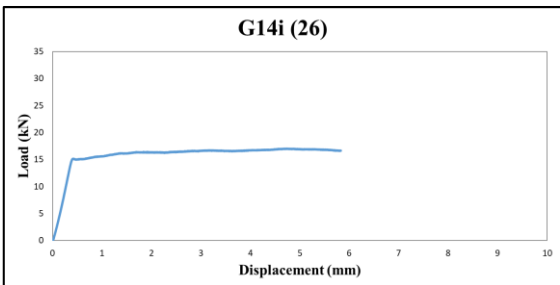
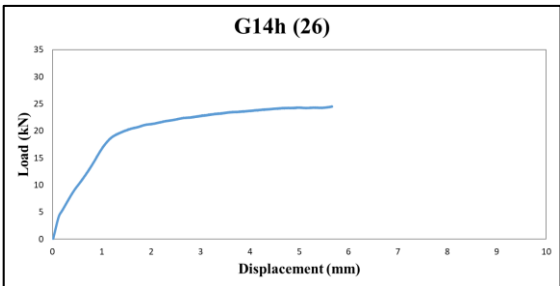
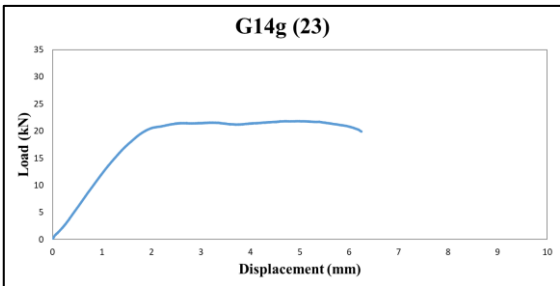
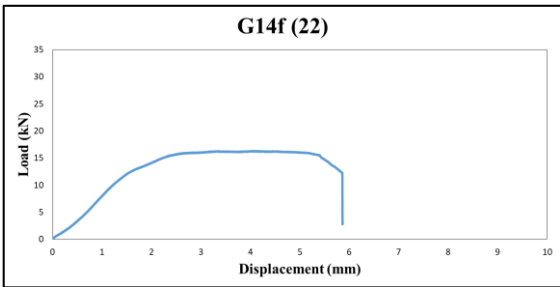
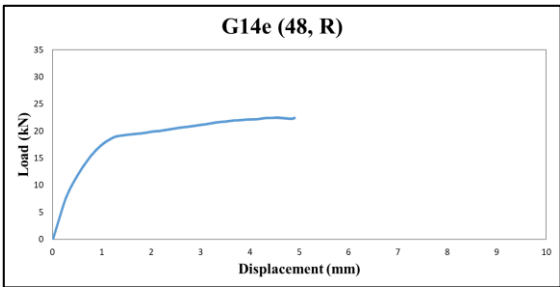
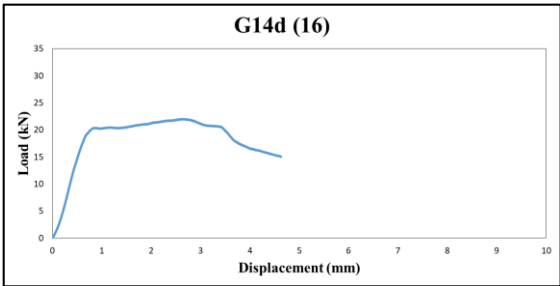
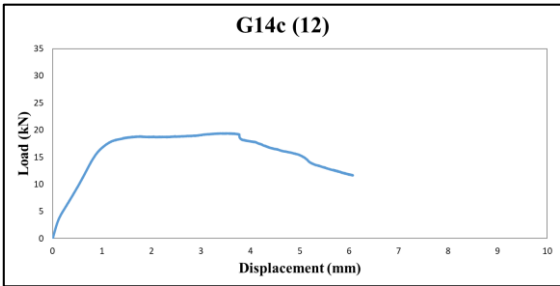
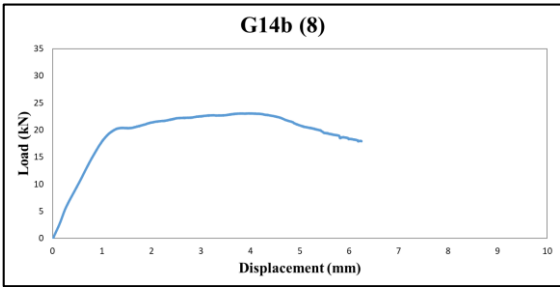
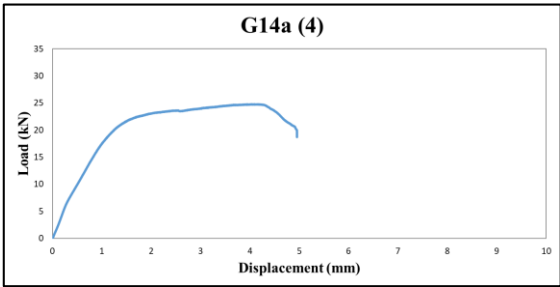


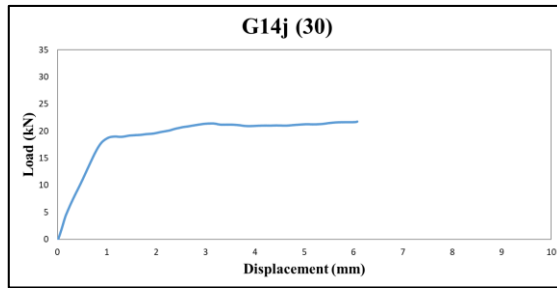
Group 13



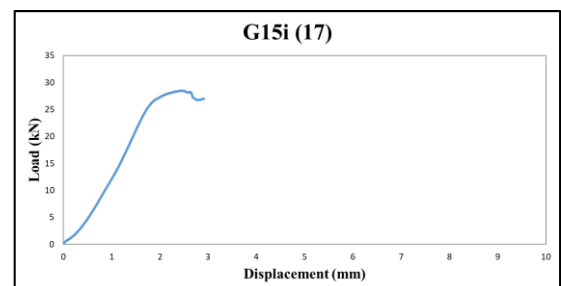
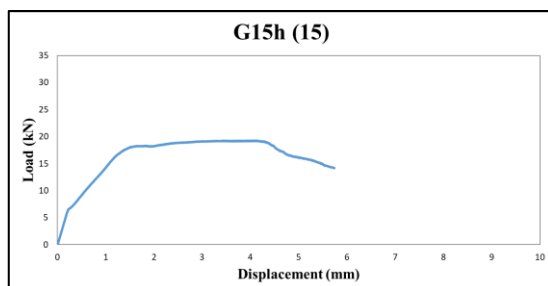
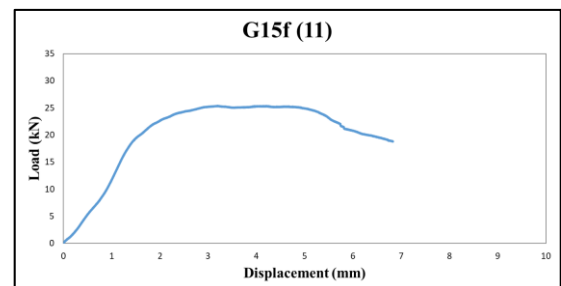
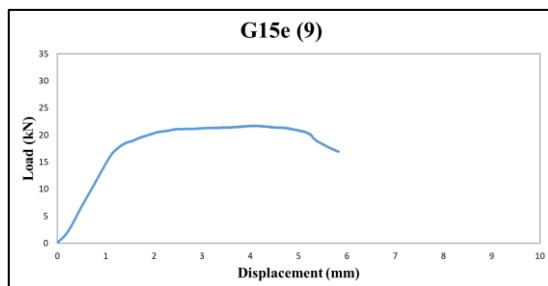
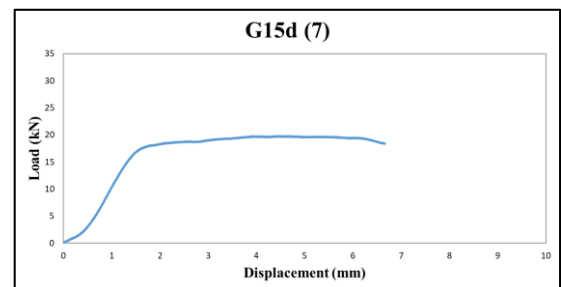
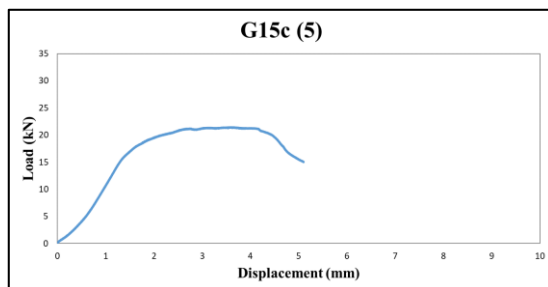
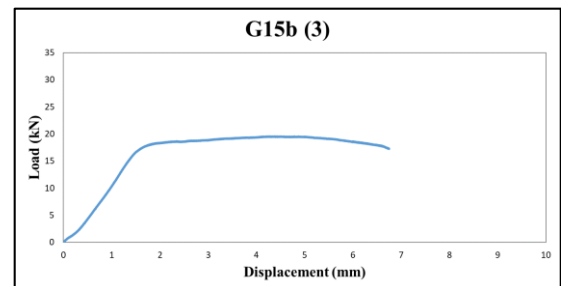
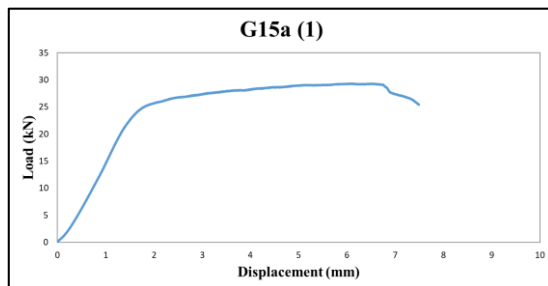


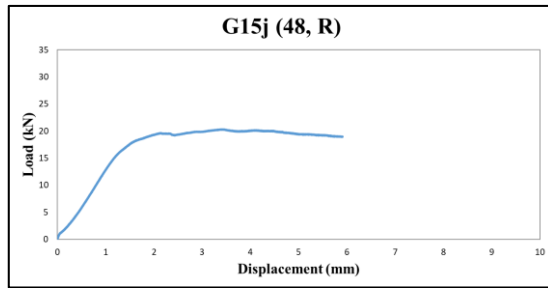
Group 14



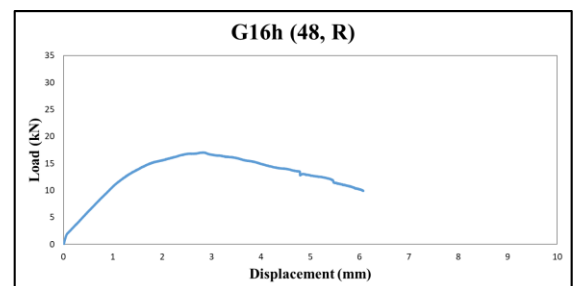
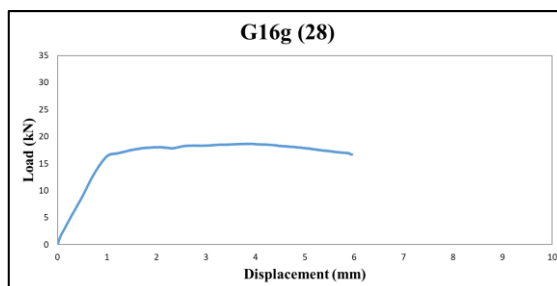
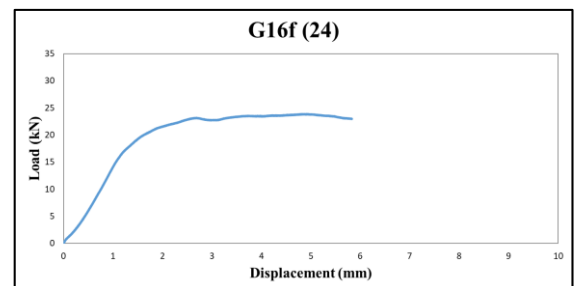
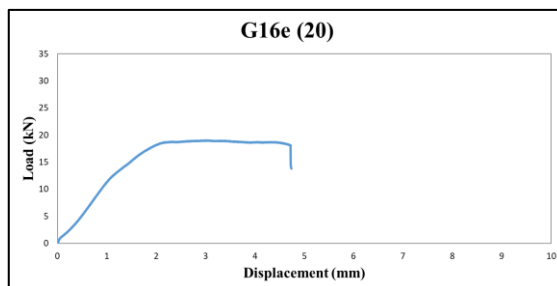
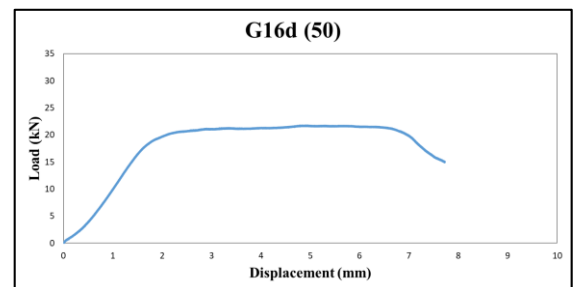
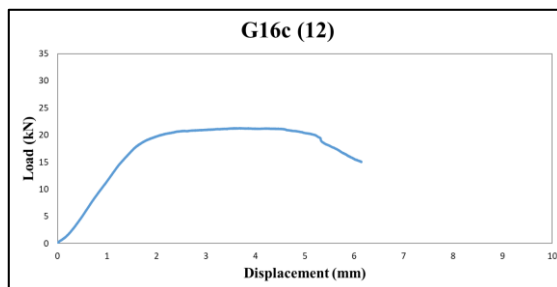
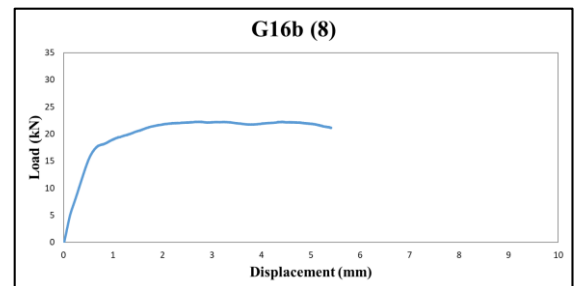
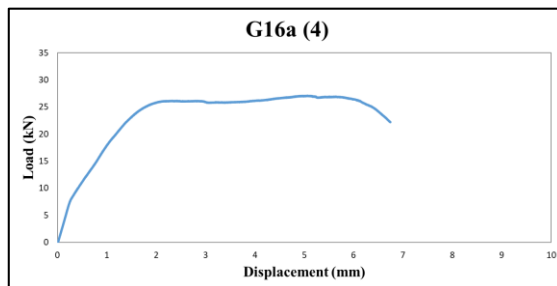


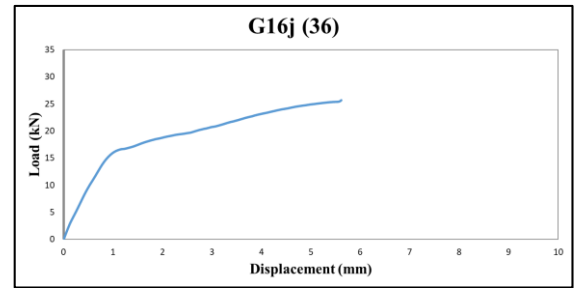
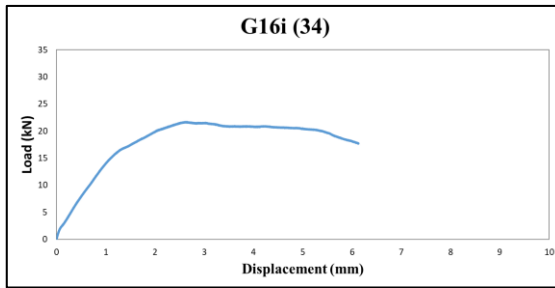
Group 15



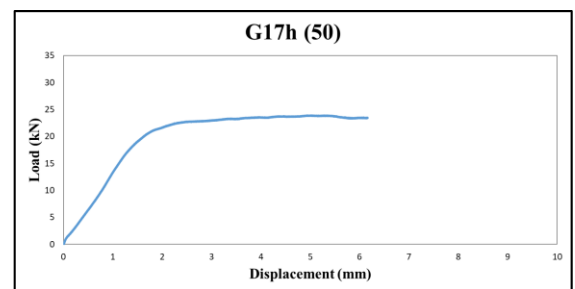
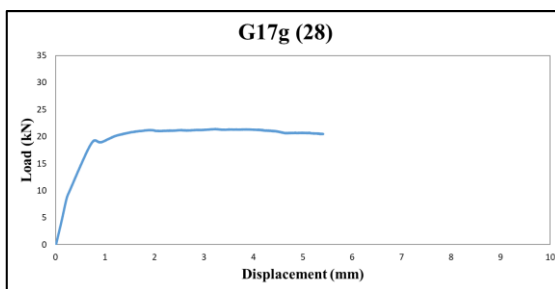
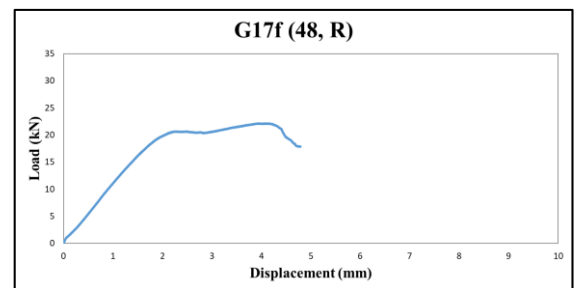
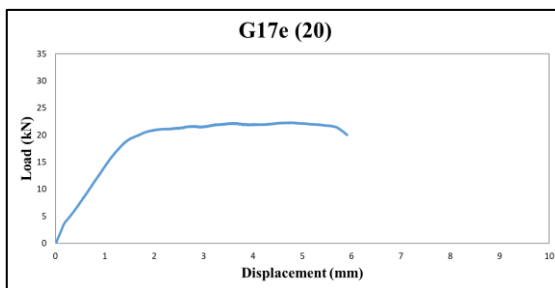
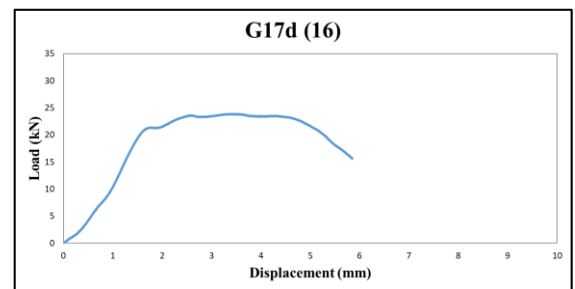
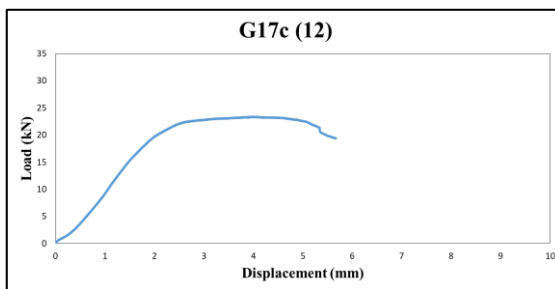
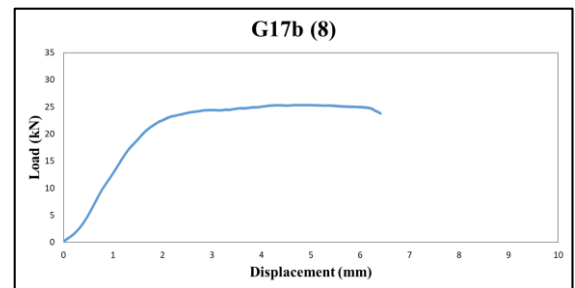
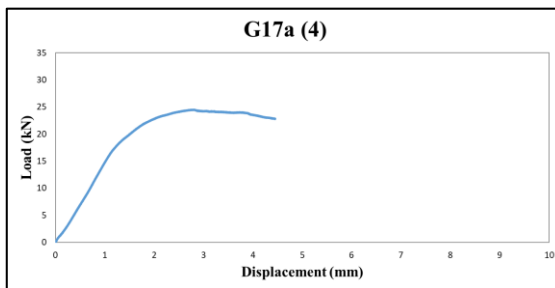


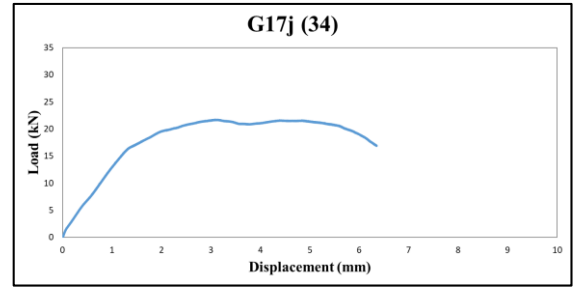
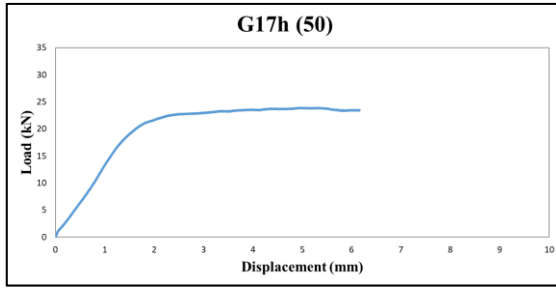
Group 16



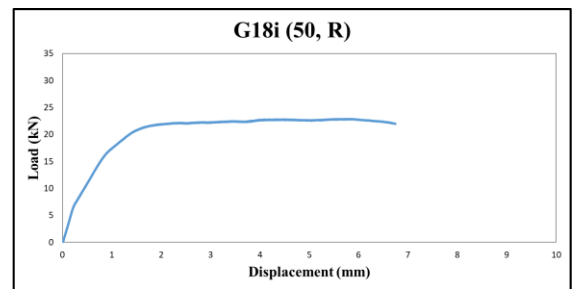
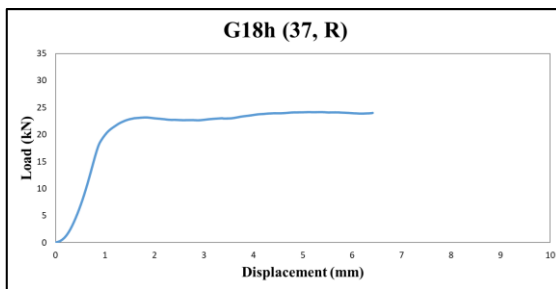
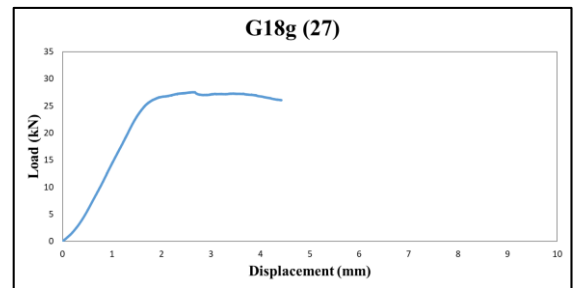
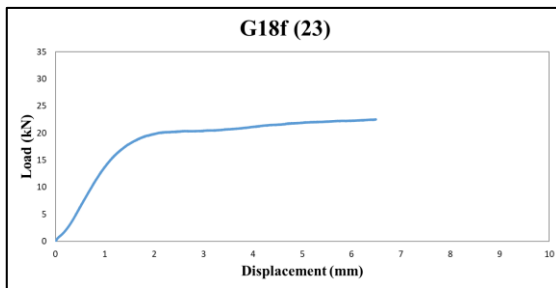
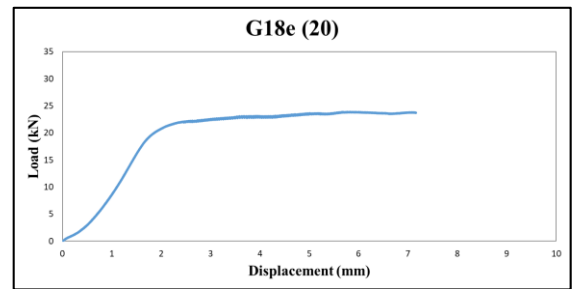
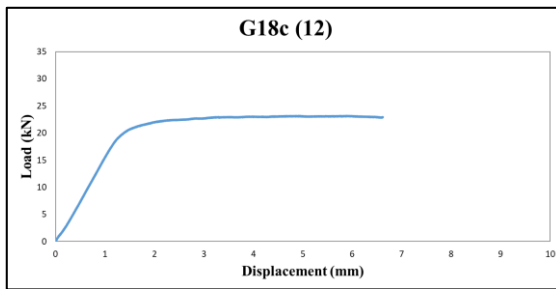
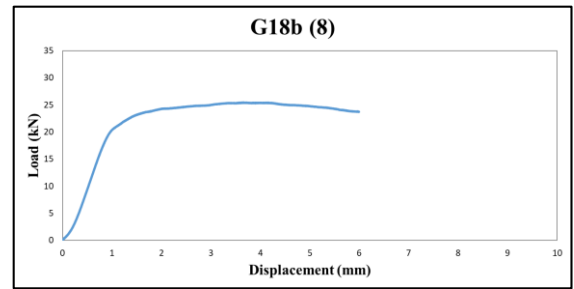
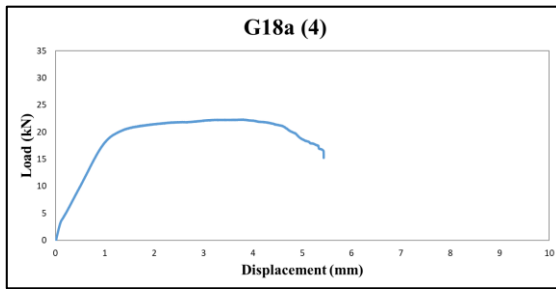


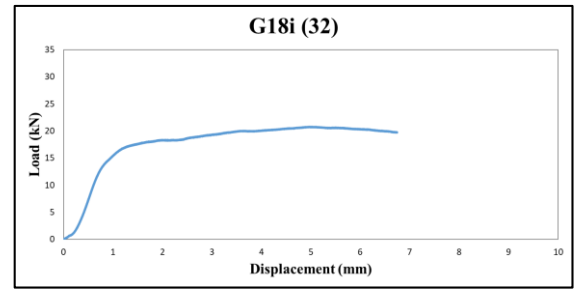
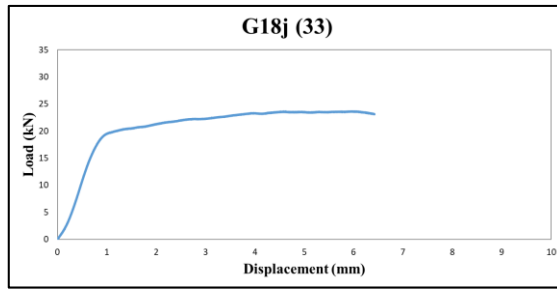
Group 17





Group 18





Appendix C: Ultimate Load Values of Embedding Strength Tests *Nyatoh* Hardwood Specimens.

Group 1

Specimens	Plank	F _{max} (kN)	d (mm)	t (mm)	f _h (N/mm ²)
a	1	23.48	13	50	36.12
b	5	24.01	13	50	36.94
c	9	24.98	13	50	38.43
d	46 (R)	24.21	13	50	37.25
e	47 (R)	20.85	13	50	32.08
f	21	26.09	13	50	40.14
g	24	24.55	13	50	37.77
h	37	23.22	13	50	35.72
i	28	25.30	13	50	38.92
j	37	23.33	13	50	35.89

Group 2

Specimens	Plank	F _{max} (kN)	d (mm)	t (mm)	f _h (N/mm ²)
a	1	27.69	13	50	42.60
b	5	27.18	13	50	41.82
c	9	26.44	13	50	40.68
d	46 (R)	27.20	13	50	41.85
e	17	28.88	13	50	44.43
f	21	25.03	13	50	38.51
g	24	31.02	13	50	47.72
h	25	24.78	13	50	38.12
i	27	23.96	13	50	36.86
j	29	27.19	13	50	41.83

Group 3

Specimens	Plank	F _{max} (kN)	d (mm)	t (mm)	f _h (N/mm ²)
a	1	27.01	13	50	41.55
b	5	28.52	13	50	43.88
c	9	28.68	13	50	44.12
d	13	20.08	13	50	30.89
e	17	35.09	13	50	53.98
f	21	26.87	13	50	41.34
g	22	19.09	13	50	29.37
h	25	24.34	13	50	37.45
i	26	25.02	13	50	38.49
j	29	27.12	13	50	41.72

Group 4

Specimens	Plank	$F_{\max}$ (kN)	d (mm)	t (mm)	f_h (N/mm ²)
a	1	27.28	13	50	41.97
b	2	15.66	13	50	24.09
c	5	25.60	13	50	39.38
d	6	22.57	13	50	34.72
e	9	22.90	13	50	35.23
f	10	15.62	13	50	24.03
g	43 (R)	17.48	13	50	26.89
h	14	25.70	13	50	39.54
i	17	27.30	13	50	42.00
j	18	27.43	13	50	42.20

Group 5

Specimens	Plank	$F_{\max}$ (kN)	d (mm)	t (mm)	f_h (N/mm ²)
a	9	22.97	13	50	35.34
b	48 (R)	26.20	13	50	40.31
c	5	24.90	13	50	38.31
d	6	20.88	13	50	32.12
e	9	23.44	13	50	36.06
f	10	11.78	13	50	18.12
g	13	22.51	13	50	34.63
h	14	21.90	13	50	33.69
i	17	25.49	13	50	39.22
j	18	23.34	13	50	35.91

Group 6

Specimens	Plank	$F_{\max}$ (kN)	d (mm)	t (mm)	f_h (N/mm ²)
a	46 (R)	32.76	13	50	50.40
b	6	19.73	13	50	30.35
c	47 (R)	22.13	13	50	34.05
d	14	20.92	13	50	32.18
e	18	25.85	13	50	39.77
f	21	27.45	13	50	42.23
g	25	22.03	13	50	33.89
i	40 (R)	26.37	13	50	40.57
j	43 (R)	23.45	13	50	36.08

Group 7

Specimens	Plank	$F_{\max}$ (kN)	d (mm)	t (mm)	f_h (N/mm ²)
a	2	18.01	13	50	27.71
b	6	19.98	13	50	30.74
c	10	18.39	13	50	28.29
d	14	21.36	13	50	32.86
e	18	24.01	13	50	36.94
f	22	19.91	13	50	30.63
g	26	24.93	13	50	38.35
h	32	25.47	13	50	39.18
i	46 (R)	29.19	13	50	44.91
j	36	23.93	13	50	36.82

Group 8

Specimens	Plank	$F_{\max}$ (kN)	d (mm)	t (mm)	f_h (N/mm ²)
b	6	22.92	13	50	35.26
c	10	15.44	13	50	23.75
d	14	17.86	13	50	27.48
e	18	28.07	13	50	43.18
f	22	12.54	13	50	19.29
g	26	19.86	13	50	30.55
h	32	21.45	13	50	33.00
i	34	28.48	13	50	43.82
j	47 (R)	22.22	13	50	34.18

Group 9

Specimens	Plank	$F_{\max}$ (kN)	d (mm)	t (mm)	f_h (N/mm ²)
a	3	25.07	13	50	38.57
b	7	23.64	13	50	36.37
c	11	24.13	13	50	37.12
d	15	29.02	13	50	44.65
e	19	25.33	13	50	38.97
f	22	27.68	13	50	42.58
g	25	28.28	13	50	43.51
h	47 (R)	21.23	13	50	32.66
i	30	24.92	13	50	38.34
j	31	25.76	13	50	39.63

Group 10

Specimens	Plank	F_{max} (kN)	d (mm)	t (mm)	f_h (N/mm²)
a	3	16.93	13	50	26.05
b	7	13.21	13	50	20.32
c	11	17.07	13	50	26.26
d	15	17.67	13	50	27.18
e	19	18.16	13	50	27.94
f	22	4.28	13	50	6.58
g	24	13.28	13	50	20.43
h	27	26.84	13	50	41.29
i	30	25.43	13	50	39.12
j	49 (R)	10.62	13	50	16.34
j	31	23.24	13	50	35.75

Group 11

Specimens	Plank	F_{max} (kN)	d (mm)	t (mm)	f_h (N/mm²)
a	3	6.58	13	50	10.12
b	7	3.95	13	50	6.08
c	11	19.19	13	50	29.52
d	15	13.85	13	50	21.31
e	19	21.23	13	50	32.66
f	23	21.14	13	50	32.52
g	43 (R)	19.01	13	50	29.25
h	30	24.31	13	50	37.40
i	49 (R)	24.51	13	50	37.71
j	36	25.77	13	50	39.65
g	27	17.75	13	50	27.31
i	35	21.16	13	50	32.55

Group 12

Specimens	Plank	F_{max} (kN)	d (mm)	t (mm)	f_h (N/mm²)
a	3	21.58	13	50	33.20
b	7	20.05	13	50	30.85
c	11	25.68	13	50	39.51
d	15	21.21	13	50	32.63
e	19	23.74	13	50	36.52
f	23	32.33	13	50	49.74
g	49 (R)	21.41	13	50	32.94
h	32	21.97	13	50	33.80
i	34	24.60	13	50	37.85
j	35	17.93	13	50	27.58
g	27	19.13	13	50	29.43

Group 13

Specimens	Plank	F_{max} (kN)	d (mm)	t (mm)	f_h (N/mm²)
a	4	22.92	13	50	35.26
b	8	21.69	13	50	33.37
c	12	19.76	13	50	30.40
d	16	23.13	13	50	35.58
e	19	19.54	13	50	30.06
f	23	21.28	13	50	32.74
g	26	23.51	13	50	36.17
h	49 (R)	23.76	13	50	36.55
h	50	19.05	13	50	29.31
h	27	17.63	13	50	27.12
i	33	23.13	13	50	35.58

Group 14

Specimens	Plank	F _{max} (kN)	d (mm)	t (mm)	f _h (N/mm ²)
a	4	24.77	13	50	38.11
b	8	23.08	13	50	35.51
c	12	19.37	13	50	29.80
d	16	21.98	13	50	33.82
e	48 (R)	22.55	13	50	34.69
f	22	16.32	13	50	25.11
g	23	21.88	13	50	33.66
h	26	24.55	13	50	37.77
i	26	17.01	13	50	26.17
j	43 (R)	23.50	13	50	36.15
j	30	21.74	13	50	33.45

Group 15

Specimens	Plank	F _{max} (kN)	d (mm)	t (mm)	f _h (N/mm ²)
a	1	29.29	13	50	45.06
b	3	19.51	13	50	30.02
c	5	21.42	13	50	32.95
d	7	19.80	13	50	30.46
e	9	21.76	13	50	33.48
f	11	25.35	13	50	39.00
h	15	19.28	13	50	29.66
i	17	28.53	13	50	43.89
j	48 (R)	20.34	13	50	31.29

Group 16

Specimens	Plank	F _{max} (kN)	d (mm)	t (mm)	f _h (N/mm ²)
a	4	27.09	13	50	41.68
b	8	22.27	13	50	34.26
c	12	21.26	13	50	32.71
d	50	21.75	13	50	33.46
e	20	18.98	13	50	29.20
f	24	23.83	13	50	36.66
g	28	18.67	13	50	28.72
h	48 (R)	17.04	13	50	26.22
i	34	21.70	13	50	33.38
j	36	25.64	13	50	39.45

Group 17

Specimens	Plank	F_{max} (kN)	d (mm)	t (mm)	f_h (N/mm²)
a	4	24.50	13	50	37.69
b	8	25.40	13	50	39.08
c	12	23.38	13	50	35.97
d	16	23.87	13	50	36.72
e	20	22.32	13	50	34.34
f	48 (R)	22.13	13	50	34.05
g	28	21.41	13	50	32.94
h	50	23.90	13	50	36.77
i	32	18.51	13	50	28.48
j	34	21.72	13	50	33.42

Group 18

Specimens	Plank	F_{max} (kN)	d (mm)	t (mm)	f_h (N/mm²)
a	4	22.30	13	50	34.31
b	8	25.42	13	50	39.11
c	12	23.17	13	50	35.65
e	20	23.89	13	50	36.75
f	23	22.54	13	50	34.68
g	27	27.52	13	50	42.34
h	37 (R)	24.15	13	50	37.15
g	50 (R)	22.87	13	50	35.18
j	33	23.65	13	50	36.38
i	32	20.79	13	50	31.98

Appendix D: Moisture Content and Density of *Nyatoh* Hardwood Specimens.

Moisture Content and Density of Test Specimens

$$\text{Moisture Content, } MC = \frac{\text{Initial mass (Day 0)} - \text{Oven Dry Mass (Day 3)}}{\text{Oven Dry Mass}} \times 100$$

$$\text{Density, } \rho = \frac{\text{Mass (kg)}}{\text{Volume (m}^3\text{)}}$$

Group 1

Specimens (plank number)	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	1	50.00	50.00	50.00	0.000125	88.4	77.1	76.0	76	707.20	608.00	16
b	5	50.00	50.00	50.00	0.000125	80.0	69.8	69.1	69.1	640.00	552.80	16
c	9	50.00	50.00	50.00	0.000125	85.0	73.2	72.6	72.7	680.00	581.60	17
d	46 (R)	50.00	50.00	50.00	0.000125	81.0	70.9	70.1	70.1	648.00	560.80	16
d	13	50.00	50.00	50.00	0.000125	68.9	59.2	59.6	59.6	551.20	476.80	16
e	47 (R)	50.00	50.00	50.00	0.000125	79.0	69.1	68.4	68.2	632.00	545.60	16
e	17	50.00	50.00	50.00	0.000125	90.1	77.6	76.9	76.8	720.80	614.40	17
f	21	50.00	50.00	50.00	0.000125	84.0	74.1	72.9	72.8	672.00	582.40	15
g	24	50.00	50.00	50.00	0.000125	85.0	75.1	74.2	73.8	680.00	590.40	15
h	37	50.00	50.00	50.00	0.000125	81.0	70.5	70.1	70.1	648.00	560.80	16
i	28	50.00	50.00	50.00	0.000125	87.0	76.5	75.8	75.6	696.00	604.80	15
j	37 (R)	50.00	50.00	50.00	0.000125	76.3	66.2	65.9	66	610.40	528.00	16

Group 2

Specimens (plank number)	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	1	50.00	50.00	50.00	0.000125	90.0	78.9	78.0	77.8	720.00	622.40	16
b	5	50.00	50.00	50.00	0.000125	83.0	73.1	72.0	72	664.00	576.00	15
c	9	50.00	50.00	50.00	0.000125	79.0	69.2	68.2	68.1	632.00	544.80	16
d	46 (R)	50.00	50.00	50.00	0.000125	77.0	67.8	67.3	67.3	616.00	538.40	14
d	13	50.00	50.00	50.00	0.000125	68.5	59.3	59.3	59.4	548.00	475.20	15
e	17	50.00	50.00	50.00	0.000125	102.0	91.0	89.0	88.4	816.00	707.20	15
f	21	50.00	50.00	50.00	0.000125	84.0	74.3	73.4	73.3	672.00	586.40	15
g	24	50.00	50.00	50.00	0.000125	87.0	77.6	76.1	75.9	696.00	607.20	15
h	25	50.00	50.00	50.00	0.000125	75.0	65.4	64.8	64.9	600.00	519.20	16
i	27	50.00	50.00	50.00	0.000125	85.0	75.7	74.3	74	680.00	592.00	15
j	29	50.00	50.00	50.00	0.000125	88.0	77.3	76.0	75.6	704.00	604.80	16

Group 3

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	1	50.00	50.00	50.00	0.000125	94.0	82.6	81.3	81.1	752.00	648.80	16
b	5	50.00	50.00	50.00	0.000125	85.0	74.7	73.8	73.2	680.00	585.60	16
c	9	50.00	50.00	50.00	0.000125	84.0	73.3	72.8	72.7	672.00	581.60	16
d	13	50.00	50.00	50.00	0.000125	70.0	61.1	60.7	60.8	560.00	486.40	15
e	17	50.00	50.00	50.00	0.000125	101.0	90.5	88.2	87.5	808.00	700.00	15
f	21	50.00	50.00	50.00	0.000125	84.0	75.2	73.8	73.6	672.00	588.80	14
g	22	50.00	50.00	50.00	0.000125	78.0	68.4	67.0	66.9	624.00	535.20	17
h	25	50.00	50.00	50.00	0.000125	84.0	73.9	72.6	72.3	672.00	578.40	16
i	26	50.00	50.00	50.00	0.000125	82.0	71.6	71.0	71.0	656.00	568.00	15
j	29	50.00	50.00	50.00	0.000125	94.0	82.0	81.3	81.0	752.00	648.00	16

Group 4

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	1	50.00	50.00	50.00	0.000125	94.00	82.90	81.50	81.10	752.00	648.80	16
b	2	50.00	50.00	50.00	0.000125	56.00	49.20	49.00	49	448.00	392.00	14
c	5	50.00	50.00	50.00	0.000125	83.00	73.10	72.20	71.9	664.00	575.20	15
d	6	50.00	50.00	50.00	0.000125	82.00	72.20	71.50	71.3	656.00	570.40	15
e	9	50.00	50.00	50.00	0.000125	79.00	69.50	68.50	68.4	632.00	547.20	15
f	10	50.00	50.00	50.00	0.000125	56.00	49.50	49.50	49.1	448.00	392.80	14
f	43 (R)	50.00	50.00	50.00	0.000125	61.00	54.00	53.60	53.4	488.00	427.20	14
g	13	50.00	50.00	50.00	0.000125	70.60	61.20	61.20	61.3	564.80	490.40	15
h	14	50.00	50.00	50.00	0.000125	79.00	70.60	69.20	69	632.00	552.00	14
i	17	50.00	50.00	50.00	0.000125	98.00	88.60	86.30	85.4	784.00	683.20	15
j	18	50.00	50.00	50.00	0.000125	88.00	77.80	76.70	76.2	704.00	609.60	15

Group 5

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	1	50.00	50.00	50.00	0.000125	98.30	87.00	85.30	85.00	786.40	680.00	16
b	48 (R)	50.00	50.00	50.00	0.000125	83.30	72.30	72.10	71.80	666.40	574.40	16
c	5	50.00	50.00	50.00	0.000125	90.30	79.80	78.30	78.10	722.40	624.80	16
d	6	50.00	50.00	50.00	0.000125	83.00	71.80	71.60	71.60	664.00	572.80	16
e	9	50.00	50.00	50.00	0.000125	91.60	79.50	78.90	78.70	732.80	629.60	16
f	10	50.00	50.00	50.00	0.000125	65.90	57.30	57.20	57.20	527.20	457.60	15
g	13	50.00	50.00	50.00	0.000125	71.50	61.40	61.60	61.50	572.00	492.00	16
h	14	50.00	50.00	50.00	0.000125	85.10	73.70	73.70	73.60	680.80	588.80	16
i	17	50.00	50.00	50.00	0.000125	108.80	96.20	93.80	93.50	870.40	748.00	16
j	18	50.00	50.00	50.00	0.000125	85.30	74.50	73.70	73.5	682.40	588.00	16

Group 6

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	46 (R)	50.00	50.00	50.00	0.000125	82.00	71.60	71.30	71.30	656.00	570.40	15
a	2	50.00	50.00	50.00	0.000125	67.10	57.90	58.00	58.10	536.80	464.80	15
b	6	50.00	50.00	50.00	0.000125	73.00	63.50	63.50	63.5	584.00	508.00	15
c	47 (R)	50.00	50.00	50.00	0.000125	70.00	61.50	61.10	61	560.00	488.00	15
c	10	50.00	50.00	50.00	0.000125	62.10	54.30	54.20	54.2	496.80	433.60	15
d	14	50.00	50.00	50.00	0.000125	80.00	70.10	69.50	69.4	640.00	555.20	15
e	18	50.00	50.00	50.00	0.000125	85.00	74.90	73.80	74.1	680.00	592.80	15
f	21	50.00	50.00	50.00	0.000125	84.00	74.40	73.10	73.1	672.00	584.80	15
g	25	50.00	50.00	50.00	0.000125	77.00	67.70	66.90	66.8	616.00	534.40	15
h	29	50.00	50.00	50.00	0.000125	90.00	78.60	77.80	77.7	720.00	621.60	16
i	40 (R)	50.00	50.00	50.00	0.000125	89.00	79.20	78.20	78	712.00	624.00	14
i	35	50.00	50.00	50.00	0.000125	64.10	55.50	55.20	55.4	512.80	443.20	16
j	43 (R)	50.00	50.00	50.00	0.000125	90.00	79.10	78.50	78.20	720.00	625.60	15
j	35	50.00	50.00	50.00	0.000125	74.90	64.10	64.80	65.00	599.20	520.00	15

Group 7

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	2	50.00	50.00	50.00	0.000125	66.00	57.30	57.00	57.20	528.00	457.60	15
b	6	50.00	50.00	50.00	0.000125	72.00	63.40	63.10	63.20	576.00	505.60	14
c	10	50.00	50.00	50.00	0.000125	70.00	61.30	61.20	61.20	560.00	489.60	14
d	14	50.00	50.00	50.00	0.000125	79.00	69.00	68.50	68.20	632.00	545.60	16
e	18	50.00	50.00	50.00	0.000125	86.00	74.90	74.60	74.50	688.00	596.00	15
f	22	50.00	50.00	50.00	0.000125	75.00	64.60	64.60	64.40	600.00	515.20	16
g	26	50.00	50.00	50.00	0.000125	80.00	70.00	69.40	69.70	640.00	557.60	15
h	32	50.00	50.00	50.00	0.000125	77.00	66.50	66.70	66.70	616.00	533.60	15
i	46 (R)	50.00	50.00	50.00	0.000125	84.00	73.90	73.50	73.40	672.00	587.20	14
i	33	50.00	50.00	50.00	0.000125	83.20	72.00	71.50	71.60	665.60	572.80	16
i	9	50.00	50.00	50.00	0.000125	102.90	94.10	92.00	91.00	823.20	728.00	13
j	36	50.00	50.00	50.00	0.000125	72.00	62.90	62.80	62.9	576.00	503.20	14

Group 8

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	2	50.00	50.00	50.00	0.000125	73.00	63.20	63.60	63.60	584.00	508.80	15
b	6	50.00	50.00	50.00	0.000125	74.00	64.20	64.20	64	592.00	512.00	16
c	10	50.00	50.00	50.00	0.000125	76.00	66.20	66.20	65.8	608.00	526.40	16
d	14	50.00	50.00	50.00	0.000125	77.00	67.80	67.40	67.2	616.00	537.60	15
e	18	50.00	50.00	50.00	0.000125	84.00	72.50	72.50	72	672.00	576.00	17
f	22	50.00	50.00	50.00	0.000125	70.00	60.10	60.20	59.9	560.00	479.20	17
g	26	50.00	50.00	50.00	0.000125	90.00	77.50	77.60	77.5	720.00	620.00	16
h	32	50.00	50.00	50.00	0.000125	73.00	63.00	63.20	63.1	584.00	504.80	16
i	34	50.00	50.00	50.00	0.000125	87.00	76.40	75.50	75	696.00	600.00	16
j	47 (R)	50.00	50.00	50.00	0.000125	68.00	59.20	58.60	58.50	544.00	468.00	16
j	35	50.00	50.00	50.00	0.000125	73.20	63.20	63.10	62.70	585.60	501.60	17

Group 9

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	3	50.00	50.00	50.00	0.000125	87.00	76.80	75.70	75.50	696.00	604.00	15
b	7	50.00	50.00	50.00	0.000125	75.00	65.50	65.00	64.70	600.00	517.60	16
c	11	50.00	50.00	50.00	0.000125	83.00	72.40	71.50	71.50	664.00	572.00	16
d	15	50.00	50.00	50.00	0.000125	80.00	70.00	68.90	68.80	640.00	550.40	16
e	19	50.00	50.00	50.00	0.000125	92.00	80.10	79.30	79.00	736.00	632.00	16
f	22	50.00	50.00	50.00	0.000125	85.00	75.90	74.30	74.10	680.00	592.80	15
g	25	50.00	50.00	50.00	0.000125	76.00	66.70	65.50	65.40	608.00	523.20	16
h	47 (R)	50.00	50.00	50.00	0.000125	70.00	61.90	61.30	61.40	560.00	491.20	14
i	30	50.00	50.00	50.00	0.000125	79.00	69.80	68.70	68.6	632.00	548.80	15
j	31	50.00	50.00	50.00	0.000125	82.00	72.00	71.00	70.8	656.00	566.40	16

Group 10

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	3	50.00	50.00	50.00	0.000125	95.80	84.40	83.00	82.70	766.40	661.60	16
b	7	50.00	50.00	50.00	0.000125	78.10	68.40	67.50	67.3	624.80	538.40	16
c	11	50.00	50.00	50.00	0.000125	91.40	80.00	78.80	78.3	731.20	626.40	17
d	15	50.00	50.00	50.00	0.000125	87.00	76.30	74.90	74.8	696.00	598.40	16
e	19	50.00	50.00	50.00	0.000125	100.10	87.70	86.50	86.30	800.80	690.40	16
f	22	50.00	50.00	50.00	0.000125	75.90	67.00	65.90	65.8	607.20	526.40	15
g	24	50.00	50.00	50.00	0.000125	98.40	88.80	86.60	86	787.20	688.00	14
h	28	50.00	50.00	50.00	0.000125	82.50	72.70	71.80	71.4	660.00	571.20	16
i	30	50.00	50.00	50.00	0.000125	75.10	66.30	65.30	65.1	600.80	520.80	15
j	31	50.00	50.00	50.00	0.000125	72.70	64.20	63.00	62.6	581.60	500.80	16
j (R)	49	50.00	50.00	50.00	0.000125	93.00	82.00	80.60	80.3	744.00	642.40	16

Group 11

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	3	50.00	50.00	50.00	0.000125	86.40	76.40	74.90	74.70	691.20	597.60	16
b	7	50.00	50.00	50.00	0.000125	77.90	68.20	67.30	66.60	623.20	532.80	17
c	11	50.00	50.00	50.00	0.000125	99.40	86.60	85.10	84.70	795.20	677.60	17
d	15	50.00	50.00	50.00	0.000125	84.90	74.60	73.30	73.30	679.20	586.40	16
e	19	50.00	50.00	50.00	0.000125	97.70	84.70	83.80	83.60	781.60	668.80	17
f	23	50.00	50.00	50.00	0.000125	91.70	80.80	79.30	78.90	733.60	631.20	16
g	43 (R)	50.00	50.00	50.00	0.000125	74.20	64.90	64.30	64.30	593.60	514.40	15
h	30	50.00	50.00	50.00	0.000125	77.70	67.80	67.10	66.90	621.60	535.20	16
g	27	50.00	50.00	50.00	0.000125	60.50	53.00	52.40	52.30	484.00	418.40	16
i	49 (R)	50.00	50.00	50.00	0.000125	92.30	80.90	79.60	79.30	738.40	634.40	16
i	35	50.00	50.00	50.00	0.000125	63.50	55.20	54.90	54.80	508.00	438.40	16
j	36	50.00	50.00	50.00	0.000125	78.20	67.80	67.50	67.20	625.60	537.60	16

Group 12

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	3	50.00	50.00	50.00	0.000125	79.10	68.80	68.50	68.20	632.80	545.60	16
b	7	50.00	50.00	50.00	0.000125	81.50	70.60	69.90	70.00	652.00	560.00	16
c	11	50.00	50.00	50.00	0.000125	103.10	89.40	88.40	88.10	824.80	704.80	17
d	15	50.00	50.00	50.00	0.000125	79.60	69.30	68.90	68.80	636.80	550.40	16
e	19	50.00	50.00	50.00	0.000125	95.40	82.40	81.80	81.80	763.20	654.40	17
f	23	50.00	50.00	50.00	0.000125	87.10	76.10	75.20	74.90	696.80	599.20	16
g	49 (R)	50.00	50.00	50.00	0.000125	98.30	86.50	85.10	84.80	786.40	678.40	16
g	27	50.00	50.00	50.00	0.000125	66.10	57.30	57.00	57.00	528.80	456.00	16
h	33	50.00	50.00	50.00	0.000125	84.00	72.90	72.10	72.10	672.00	576.80	17
i	34	50.00	50.00	50.00	0.000125	97.50	85.30	84.00	83.70	780.00	669.60	16
j	35	50.00	50.00	50.00	0.000125	59.30	51.40	51.00	50.90	474.40	407.20	17

Group 13

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	4	50.00	50.00	50.00	0.000125	86.80	75.80	75.40	75.00	694.40	600.00	16
b	8	50.00	50.00	50.00	0.000125	96.10	83.90	82.70	82.60	768.80	660.80	16
c	12	50.00	50.00	50.00	0.000125	74.50	64.50	64.20	64.20	596.00	513.60	16
d	16	50.00	50.00	50.00	0.000125	109.30	95.10	94.10	93.70	874.40	749.60	17
e	19	50.00	50.00	50.00	0.000125	81.50	70.50	70.10	70.10	652.00	560.80	16
f	23	50.00	50.00	50.00	0.000125	80.90	70.30	69.70	69.50	647.20	556.00	16
g	26	50.00	50.00	50.00	0.000125	102.60	88.20	87.90	87.60	820.80	700.80	17
h	49 (R)	50.00	50.00	50.00	0.000125	88.60	77.40	76.50	76.20	708.80	609.60	16
h	50 (R)	50.00	50.00	50.00	0.000125	84.30	73.10	72.70	72.50	674.40	580.00	16
h	27	50.00	50.00	50.00	0.000125	64.10	55.50	55.30	55.30	512.80	442.40	16
i	33	50.00	50.00	50.00	0.000125	84.90	73.40	72.90	72.70	679.20	581.60	17
j	31	50.00	50.00	50.00	0.000125	80.30	70.20	69.40	69.30	642.40	554.40	16

Group 14

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	4	50.00	50.00	50.00	0.000125	88.90	78.20	77.30	76.80	711.20	614.40	16
b	8	50.00	50.00	50.00	0.000125	97.70	85.30	84.30	84.10	781.60	672.80	16
c	12	50.00	50.00	50.00	0.000125	77.30	66.70	66.20	66.30	618.40	530.40	17
d	16	50.00	50.00	50.00	0.000125	97.20	83.70	83.40	83.30	777.60	666.40	17
e	48 (R)	50.00	50.00	50.00	0.000125	90.90	78.80	78.20	78.10	727.20	624.80	16
f	22	50.00	50.00	50.00	0.000125	74.10	64.50	63.90	63.80	592.80	510.40	16
g	23	50.00	50.00	50.00	0.000125	88.00	77.00	76.00	75.60	704.00	604.80	16
h	26	50.00	50.00	50.00	0.000125	95.40	83.20	82.40	82.10	763.20	656.80	16
i	27	50.00	50.00	50.00	0.000125	71.40	62.10	61.10	61.30	571.20	490.40	16
j	43 (R)	50.00	50.00	50.00	0.000125	88.20	77.10	76.50	76.30	705.60	610.40	16
j	30	50.00	50.00	50.00	0.000125	75.00	65.60	65.10	64.80	600.00	518.40	16

Group 15

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	1	50.00	50.00	50.00	0.000125	94.40	82.60	81.40	80.80	755.20	646.40	17
b	3	50.00	50.00	50.00	0.000125	75.60	65.90	65.20	65.30	604.80	522.40	16
c	5	50.00	50.00	50.00	0.000125	88.40	77.20	76.10	75.80	707.20	606.40	17
d	7	50.00	50.00	50.00	0.000125	77.20	68.30	66.80	66.40	617.60	531.20	16
e	9	50.00	50.00	50.00	0.000125	89.80	79.10	77.30	77.00	718.40	616.00	17
f	11	50.00	50.00	50.00	0.000125	90.70	78.60	77.90	77.80	725.60	622.40	17
g	13	50.00	50.00	50.00	0.000125	71.50	61.40	61.60	61.50	572.00	492.00	16
h	15	50.00	50.00	50.00	0.000125	86.00	74.50	74.30	74.10	688.00	592.80	16
i	17	50.00	50.00	50.00	0.000125	111.20	98.70	96.90	96.00	889.60	768.00	16
j	48 (R)	50.00	50.00	50.00	0.000125	90.10	78.00	77.60	77.50	720.80	620.00	16

Group 16

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	4	50.00	50.00	50.00	0.000125	89.70	78.70	77.70	77.40	717.60	619.20	16
b	8	50.00	50.00	50.00	0.000125	91.50	79.20	78.60	77.90	732.00	623.20	17
c	12	50.00	50.00	50.00	0.000125	77.70	67.10	66.90	66.60	621.60	532.80	17
d	50	50.00	50.00	50.00	0.000125	80.60	70.00	69.60	69.40	644.80	555.20	16
e	20	50.00	50.00	50.00	0.000125	77.30	67.10	66.70	66.70	618.40	533.60	16
f	24	50.00	50.00	50.00	0.000125	100.30	88.80	87.20	86.80	802.40	694.40	16
g	28	50.00	50.00	50.00	0.000125	85.70	75.80	74.80	74.50	685.60	596.00	15
h	48 (R)	50.00	50.00	50.00	0.000125	90.40	77.90	77.60	77.60	723.20	620.80	16
i	34	50.00	50.00	50.00	0.000125	92.80	80.60	79.80	79.80	742.40	638.40	16
j	36	50.00	50.00	50.00	0.000125	85.20	73.90	73.20	73.20	681.60	585.60	16

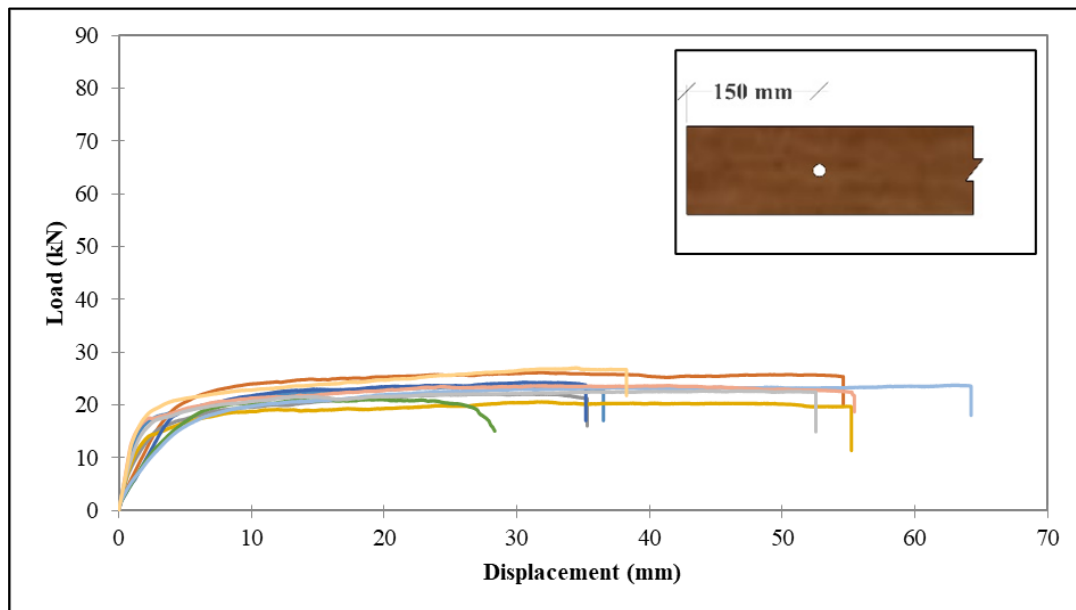
Group 17

Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	4	50.00	50.00	50.00	0.000125	85.90	75.40	74.40	74.30	687.20	594.40	16
b	8	50.00	50.00	50.00	0.000125	91.20	79.10	78.20	78.20	729.60	625.60	17
c	12	50.00	50.00	50.00	0.000125	81.80	70.70	70.50	70.20	654.40	561.60	17
d	16	50.00	50.00	50.00	0.000125	102.80	88.90	88.60	88.50	822.40	708.00	16
e	20	50.00	50.00	50.00	0.000125	78.40	68.20	68.00	67.90	627.20	543.20	15
f	48 (R)	50.00	50.00	50.00	0.000125	99.30	86.80	85.70	85.60	794.40	684.80	16
g	28	50.00	50.00	50.00	0.000125	83.10	73.10	72.40	72.30	664.80	578.40	15
h	50	50.00	50.00	50.00	0.000125	76.10	66.40	66.10	65.50	608.80	524.00	16
i	32	50.00	50.00	50.00	0.000125	74.40	64.80	64.70	64.90	595.20	519.20	15
j	34	50.00	50.00	50.00	0.000125	88.50	76.80	76.10	76.00	708.00	608.00	16

Group 18

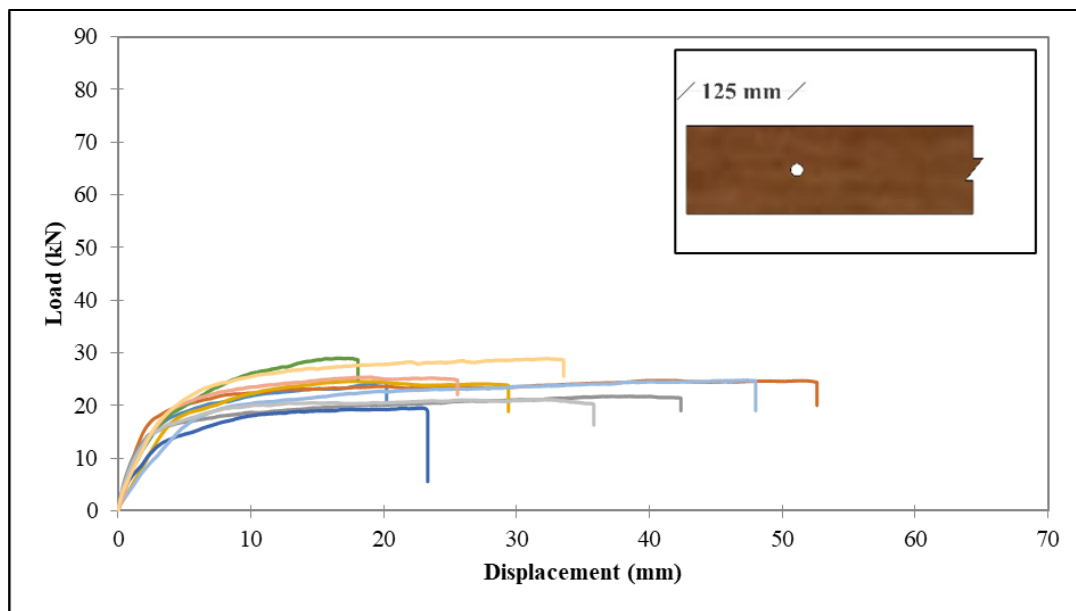
Specimens	Plank	Dimension (mm)			Volume m ³	Weight (g)				Density (kg/m ³)		Moisture Content (%)
		Width	Length	Height		Day 0	Day 1	Day 2	Day 3	Day 0	Day 3	
a	4	50.00	50.00	50.00	0.000125	85.10	74.80	73.90	73.60	680.80	588.80	16
b	8	50.00	50.00	50.00	0.000125	92.10	80.40	79.30	79.30	736.80	634.40	16
c	12	50.00	50.00	50.00	0.000125	97.30	84.20	83.70	83.70	778.40	669.60	16
d	16	50.00	50.00	50.00	0.000125	102.80	88.90	88.60	88.50	822.40	708.00	16
e	20	50.00	50.00	50.00	0.000125	81.70	70.90	70.60	70.70	653.60	565.60	16
f	23	50.00	50.00	50.00	0.000125	84.20	73.70	72.60	72.70	673.60	581.60	16
g	27	50.00	50.00	50.00	0.000125	83.50	73.90	72.80	72.80	668.00	582.40	15
h	37 (R)	50.00	50.00	50.00	0.000125	75.80	65.90	65.40	65.40	606.40	523.20	16
i	50	50.00	50.00	50.00	0.000125	83.70	72.80	72.60	72.60	669.60	580.80	15
j	39 (R)	50.00	50.00	50.00	0.000125	62.90	54.60	54.60	54.70	503.20	437.60	15
j	33	50.00	50.00	50.00	0.000125	90.40	79.30	78.40	78.60	723.20	628.80	15

Appendix E: Load Versus Displacement Graphs of Bolted Connection Tests for *Nyatoh* Hardwood Specimens.



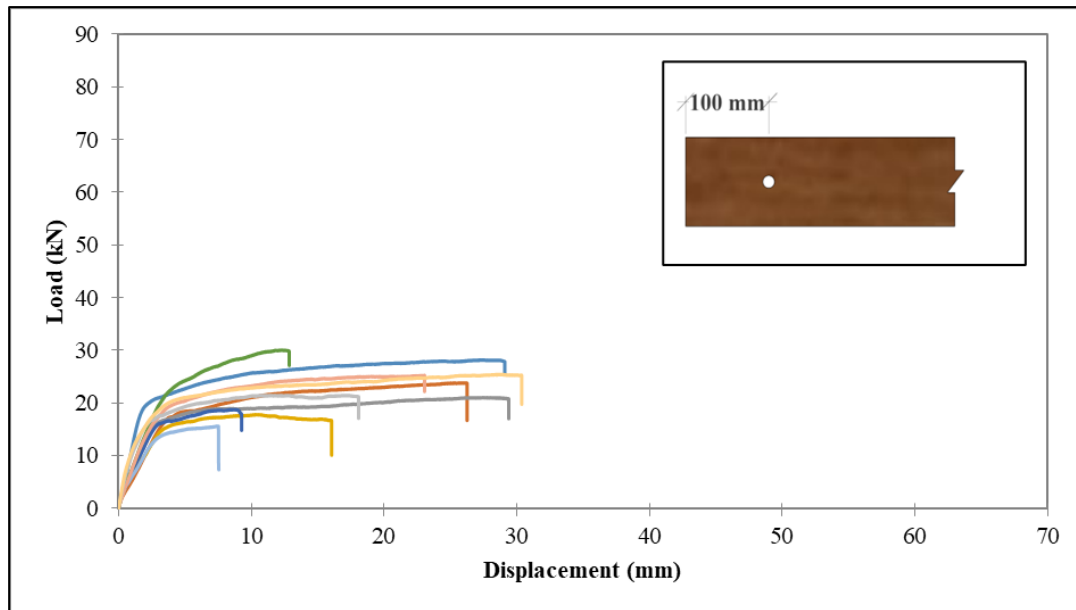
Group 1

($e = 150\text{mm}$, $n_f = 1$)



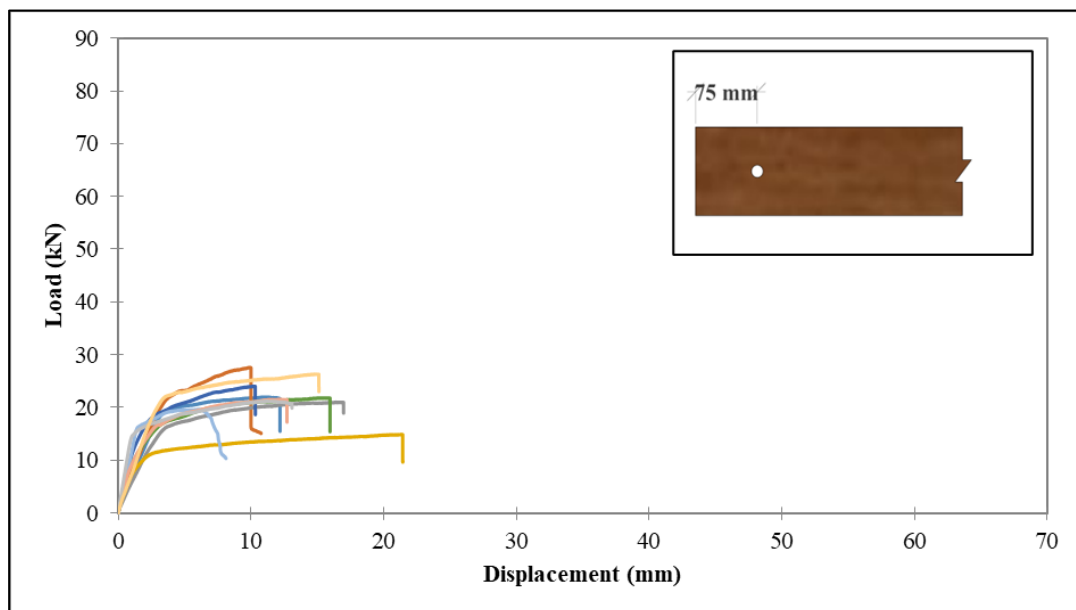
Group 2

($e = 125\text{mm}$, $n_f = 1$)



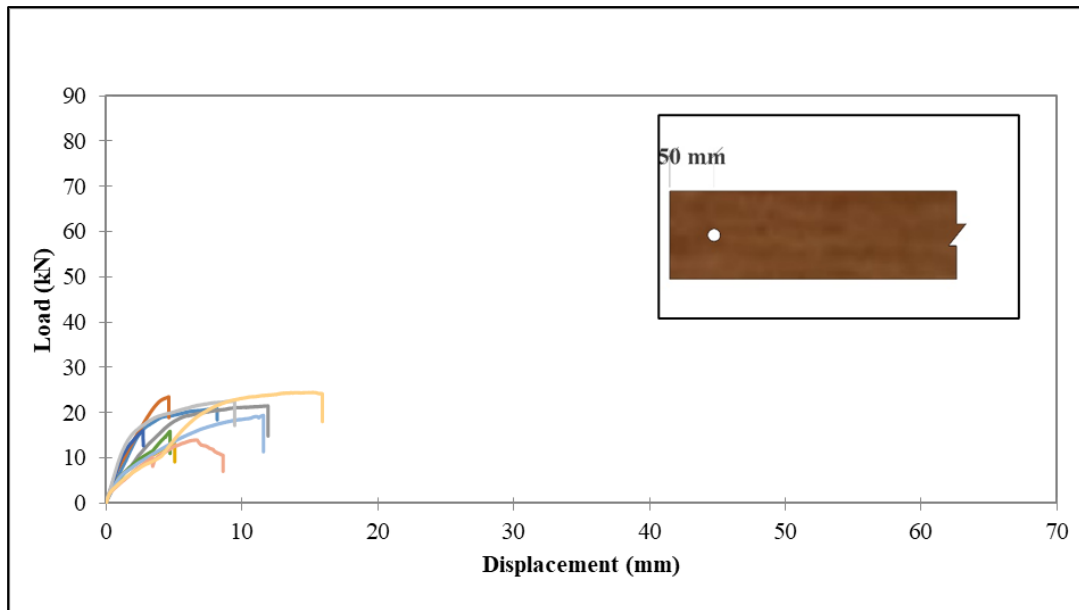
Group 3

($e = 100\text{mm}$, $n_f = 1$)

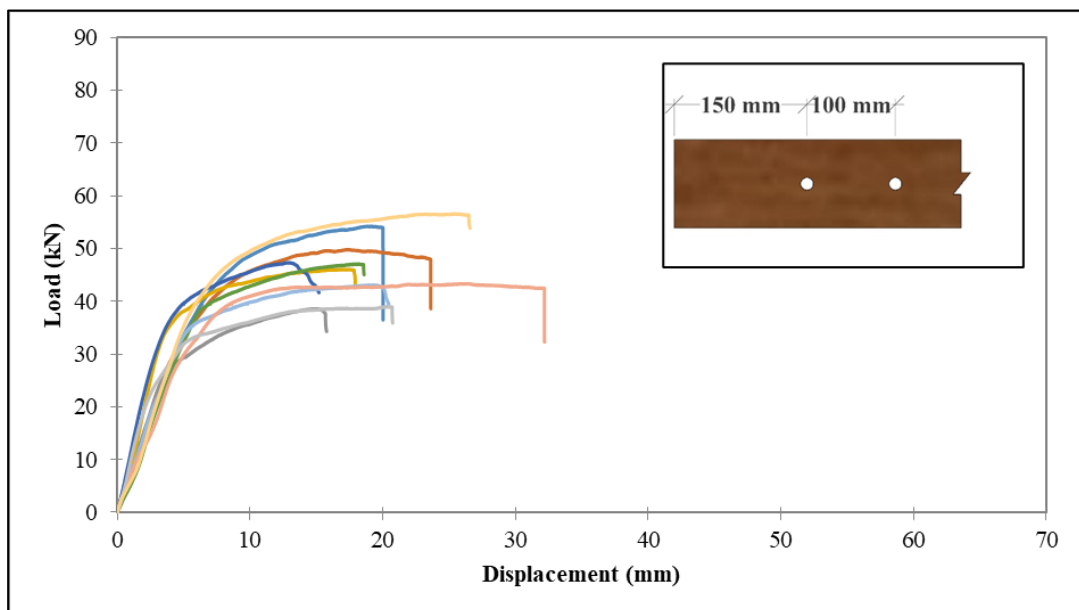


Group 4

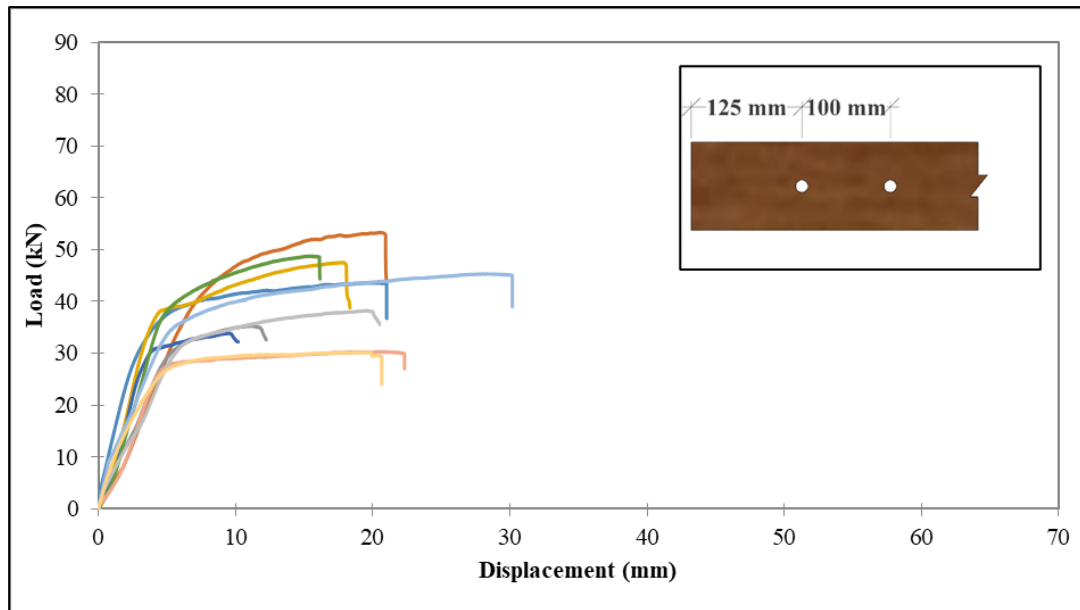
($e = 75\text{mm}$, $n_f = 1$)



Group 5
($e = 50\text{mm}$, $n_f = 1$)

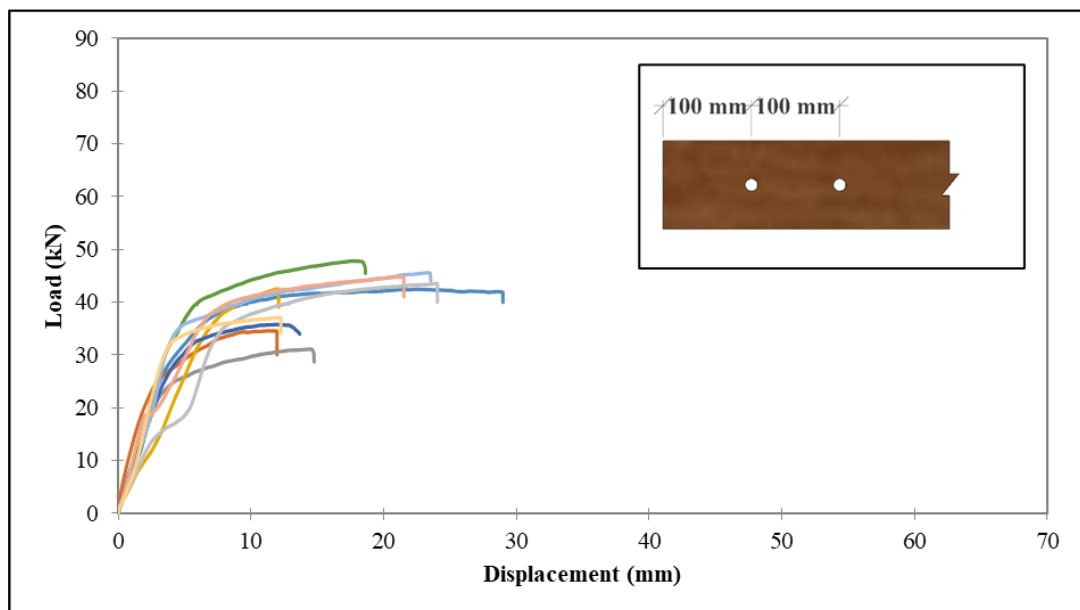


Group 6
($e = 150\text{mm}$, $s_b = 100\text{mm}$, $n_f = 2$)



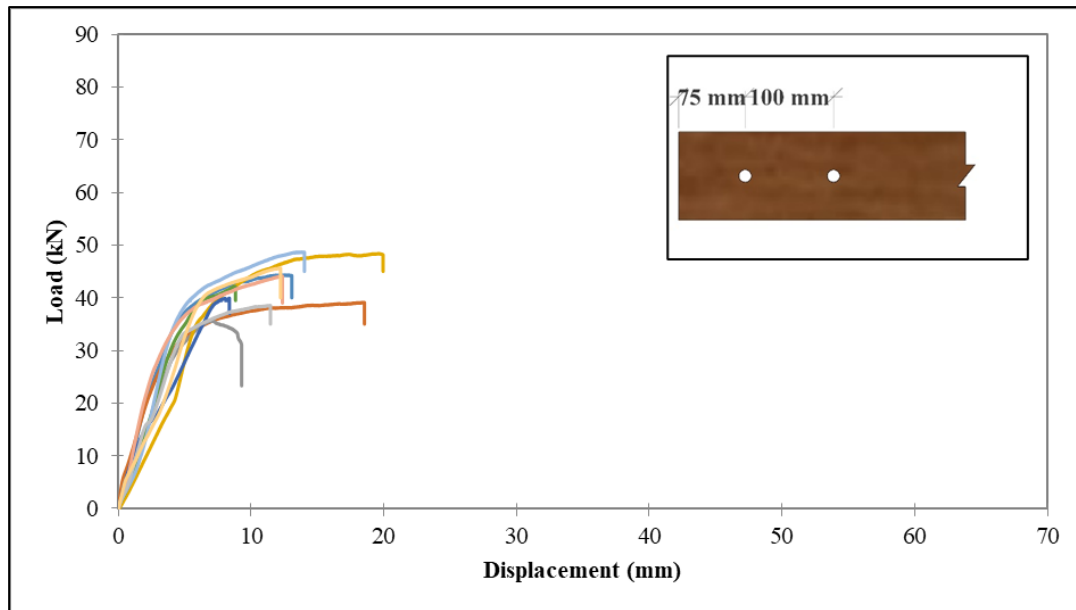
Group 7

($e = 125\text{mm}$, $s_b = 100\text{mm}$, $n_f = 2$)



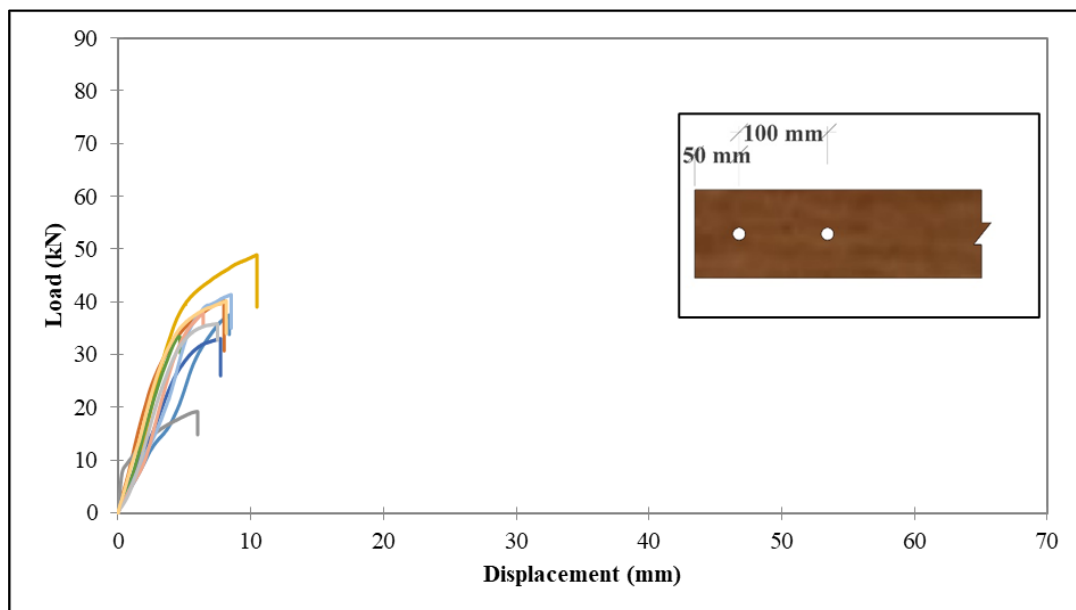
Group 8

($e = 100\text{mm}$, $s_b = 100\text{mm}$, $n_f = 2$)



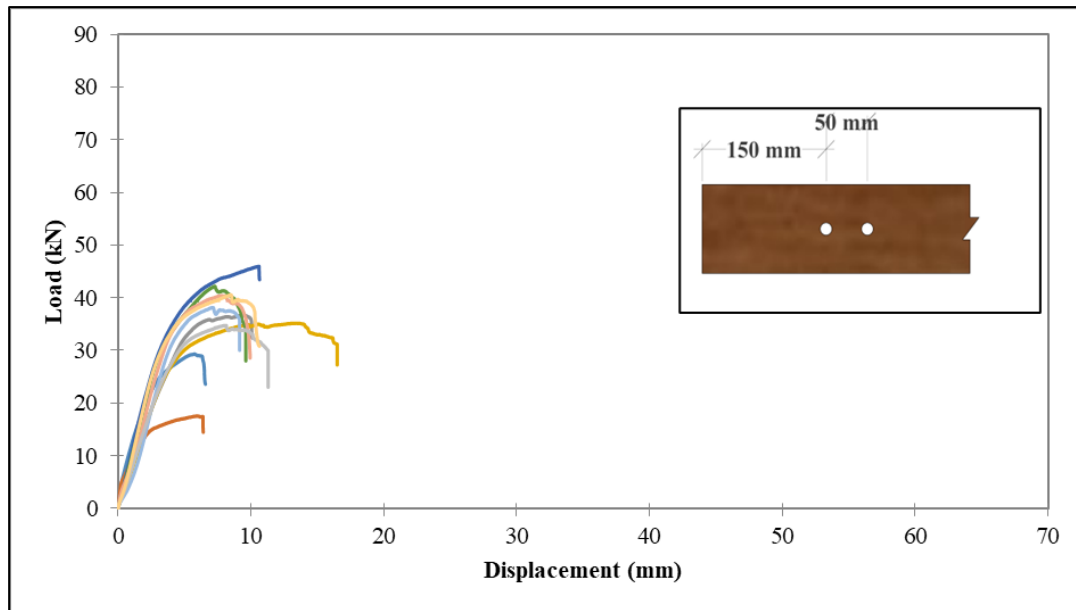
Group 9

($e = 75\text{mm}$, $s_b = 100\text{mm}$, $n_f = 2$)



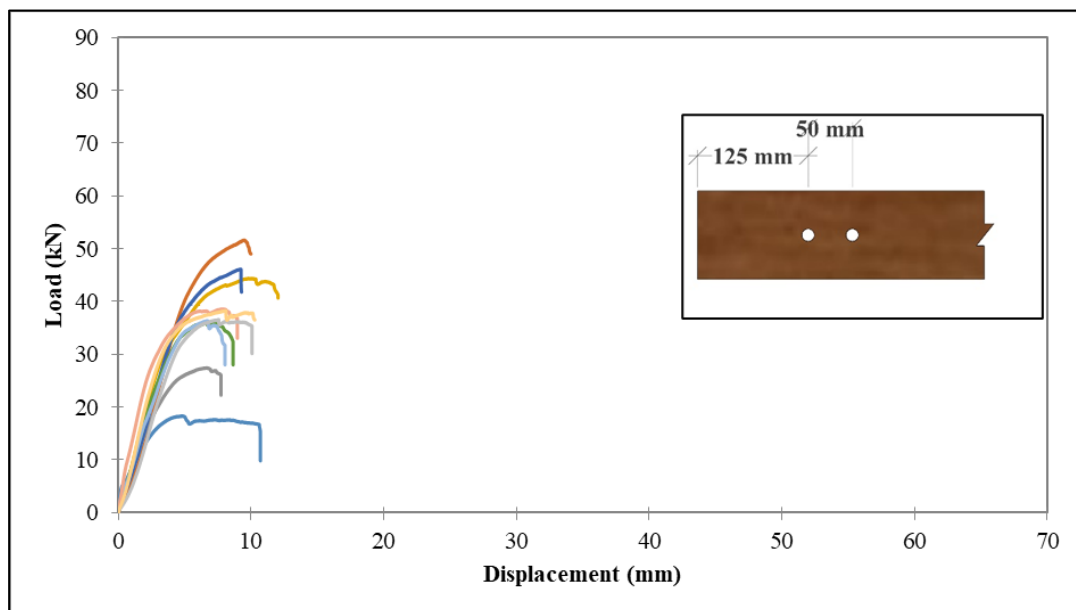
Group 10

($e = 50\text{mm}$, $s_b = 100\text{mm}$, $n_f = 2$)



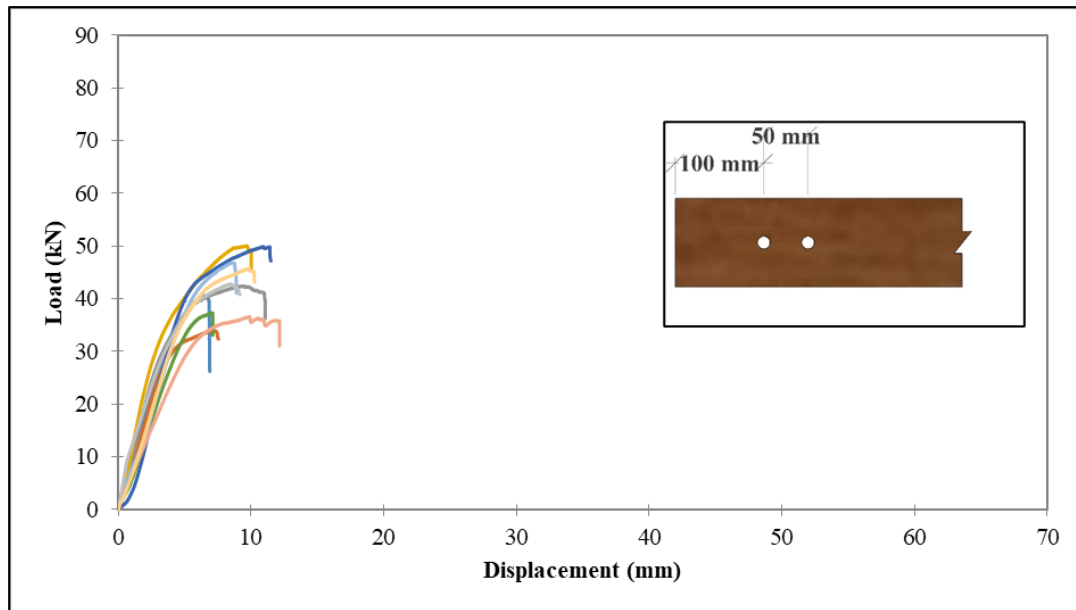
Group 11

($e = 150\text{mm}$, $s_b = 50\text{mm}$, $n_f = 2$)



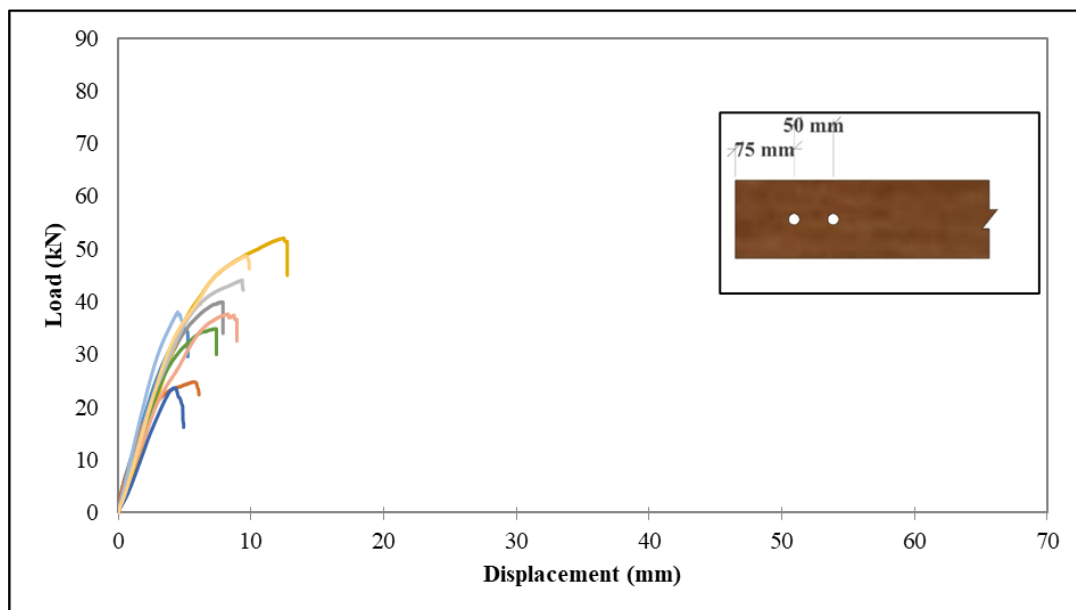
Group 12

($e = 125\text{mm}$, $s_b = 50\text{mm}$, $n_f = 2$)



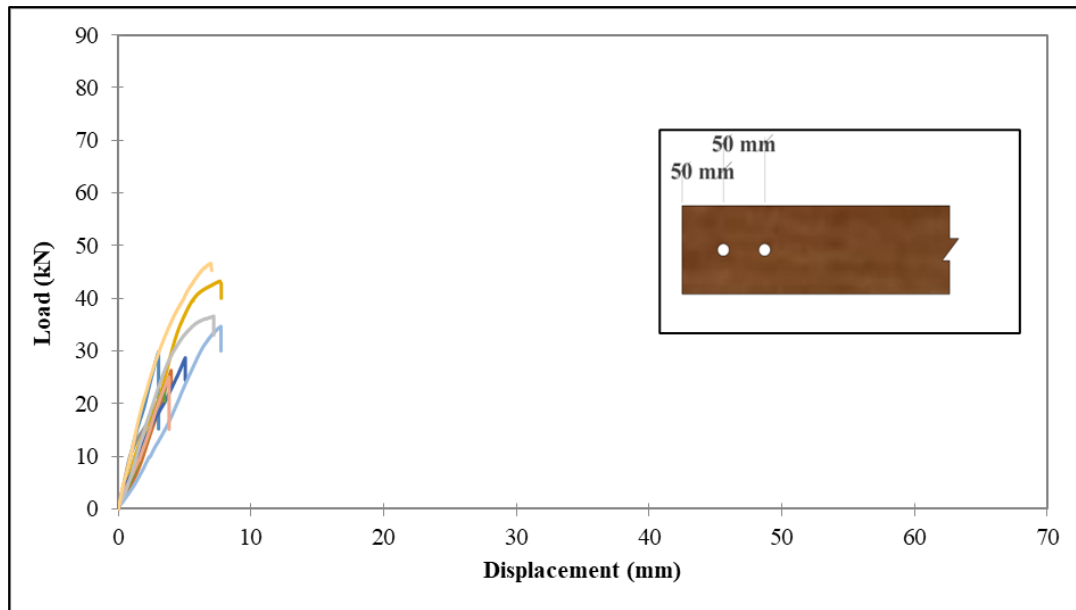
Group 13

($e = 100\text{mm}$, $s_b = 50\text{mm}$, $n_f = 2$)



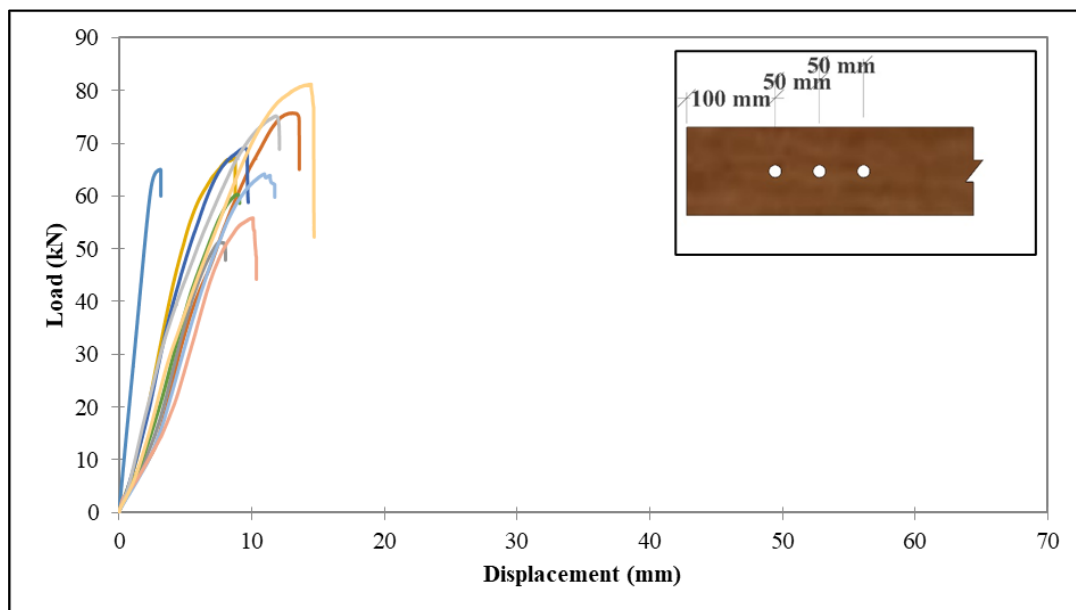
Group 14

($e = 75\text{mm}$, $s_b = 50\text{mm}$, $n_f = 2$)



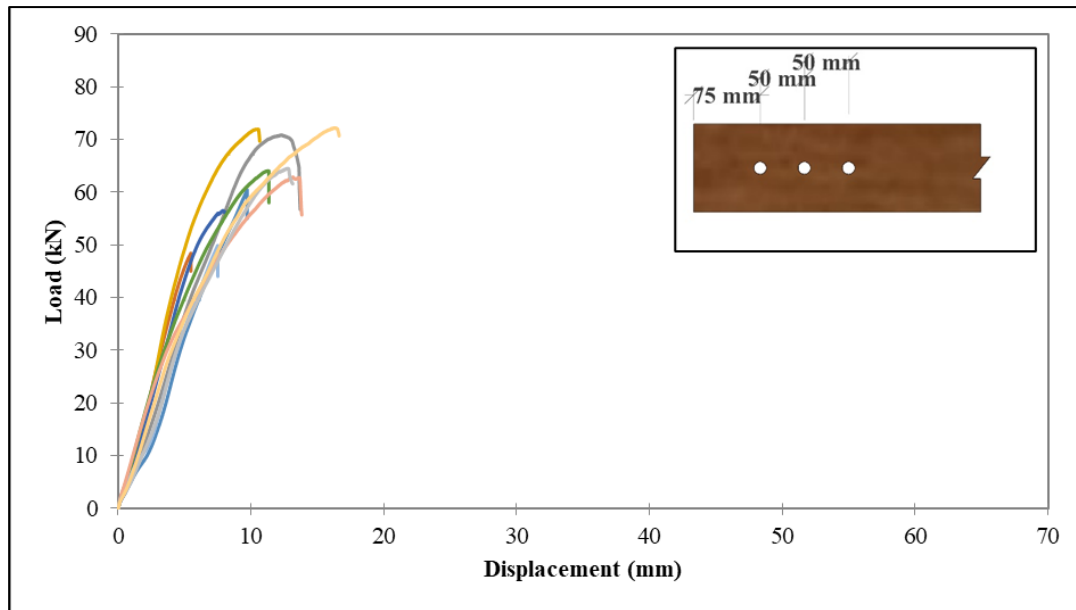
Group 15

($e = 50\text{mm}$, $s_b = 50\text{mm}$, $n_f = 2$)



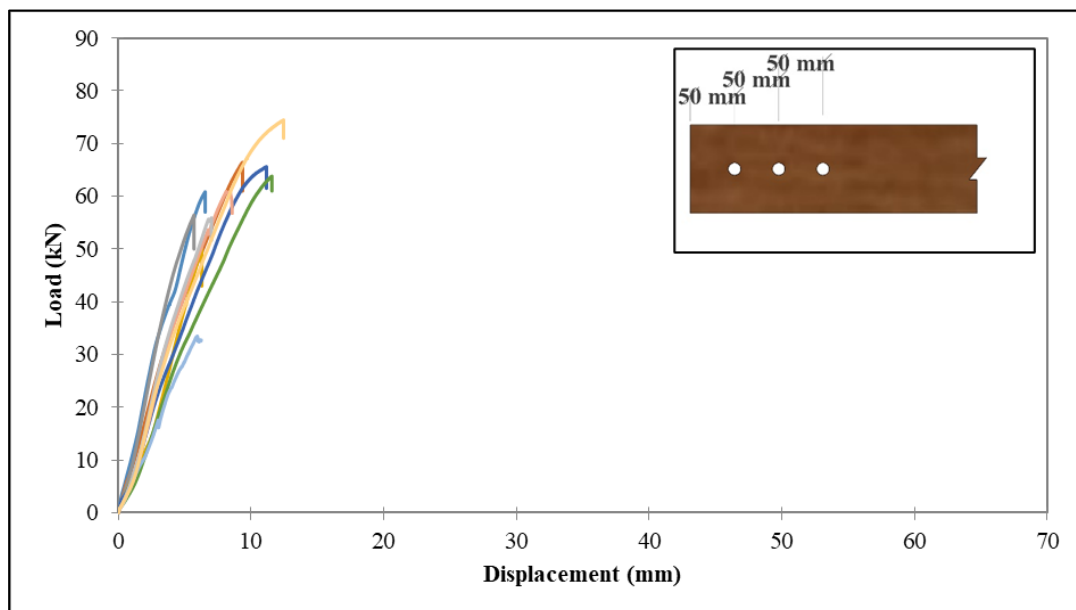
Group 16

($e = 100\text{mm}$, $s_b = 50\text{mm}$, $n_f = 3$)



Group 17

($e = 75\text{mm}$, $s_b = 50\text{mm}$, $n_f = 3$)



Group 18

($e = 50\text{mm}$, $s_b = 50\text{mm}$, $n_f = 3$)

Appendix F: Ultimate Load Values of Bolted Connection Tests for *Nyato* Hardwood Specimens.

G1		G2		G3	
Specimens	Force (kN)	Specimens	Force (kN)	Specimens	Force (kN)
a (1)	27.04	a (1)	28.93	a (1)	24.45
b (5)	22.68	b (5)	21.23	b (5)	21.47
c (9)	23.68	c (9)	25.42	c (9)	25.27
d (46, R)	23.77	d (46, R)	24.85	d (13)	15.59
e (47, R)	21.07	e (17)	28.99	e (17)	30.05
f (21)	24.29	f (21)	19.55	f (21)	18.77
g (24)	20.61	g (24)	24.64	g (22)	17.80
h (37)	22.10	h (25)	21.79	h (25)	21.05
i (28)	26.12	i (28)	24.76	i (26)	23.83
j (37, R)	23.72	j (29)	24.35	j (29)	28.19

G4		G5		G6	
Specimens	Force (kN)	Specimens	Force (kN)	Specimens	Force (kN)
a (1)	26.33	a (1)	24.50	a (46, R)	56.61
b (40, R)	21.24	b (40, R)	22.59	b (6)	38.98
c (5)	21.58	c (5)	13.98	c (47, R)	43.35
d (6)	19.69	d (6)	19.43	d (14)	43.10
e (9)	21.84	e (9)	15.88	e (18)	47.10
f (43, R)	24.05	f (43, R)	16.02	f (21)	47.33
g (43)	14.90	g (13)	13.14	g (25)	46.05
h (14)	21.03	h (14)	21.53	h (29)	38.57
i (17)	27.61	i (17)	23.52	i (40)	49.79
j (18)	22.00	j (18)	20.93	j (43)	54.23

G7		G8		G9	
Specimens	Force (kN)	Specimens	Force (kN)	Specimens	Force (kN)
a (2)	30.25	a (2)	37.11	a (3)	45.65
b (6)	38.27	b (6)	43.58	b (7)	38.55
c (10)	30.33	c (10)	44.81	c (11)	44.11
d (14)	45.34	d (14)	45.60	d (15)	48.69
e (18)	48.78	e (18)	47.84	e (19)	42.16
f (22)	33.92	f (22)	35.80	f (22)	40.79
g (26)	47.56	g (26)	42.54	g (25)	48.40
h (32)	35.34	h (32)	31.14	h (47, R)	35.63
i (46, R)	53.34	i (34)	34.60	i (30)	39.13
j (36)	43.63	j (47, R)	42.49	j (31)	44.33

G10		G11		G12	
Specimens	Force (kN)	Specimens	Force (kN)	Specimens	Force (kN)
a (3)	40.21	a (3)	40.80	a (3)	38.27
b (7)	35.87	b (7)	34.74	b (7)	30.13
c (11)	37.90	c (11)	40.42	c (11)	38.57
d (15)	41.39	d (15)	38.15	d (15)	36.37
e (19)	33.46	e (19)	42.19	e (19)	36.12
f (22)	33.05	f (23)	45.99	f (23)	46.13
g (24)	48.93	g (43, R)	35.19	g (49, R)	44.40
h (28)	19.25	h (30)	36.74	h (33)	27.46
i (30)	40.25	i (35)	17.57	i (34)	51.66
j (49, R)	37.51	j (36)	29.34	j (35)	18.36

G13		G14		G15	
Specimens	Force (kN)	Specimens	Force (kN)	Specimens	Force (kN)
a (4)	45.76	a (4)	48.71	a (1)	46.68
b (8)	42.80	b (8)	44.14	b (3)	36.62
c (12)	36.57	c (12)	37.71	c (5)	25.09
d (16)	46.81	d (16)	38.08	d (7)	34.69
e (19)	37.31	e (48, R)	34.9	e (9)	23.31
f (23)	49.90	f (22)	23.68	f (11)	28.73
g (26)	50.00	g (23)	52.10	g (40, R)	43.28
h (50)	42.42	h (26)	39.99	h (15)	20.66
i (33)	33.94	i (27)	24.84	i (17)	26.33
j (31)	40.63	j (43, R)	34.90	j (48, R)	29.85

G16		G17		G18	
Specimens	Force (kN)	Specimens	Force (kN)	Specimens	Force (kN)
a (4)	81.20	a (4)	72.22	a (4)	74.47
b (8)	75.19	b (8)	64.50	b (8)	55.91
c (12)	55.87	c (12)	62.87	c (12)	60.93
d (50)	64.20	d (16)	49.88	d (16)	33.49
e (20)	60.33	e (20)	64.07	e (20)	63.83
f (24)	69.12	f (48, R)	56.52	f (23)	65.63
g (28)	67.28	g (28)	71.95	g (27)	50.10
h (48, R)	51.31	h (50)	70.86	h (43, R)	56.44
i (34)	75.75	i (32)	48.40	i (50)	66.43
j (36)	65.07	j (34)	60.46	j (39, R)	60.85