STRESS ANALYSIS OF BOLTED COLD FORMED STEEL STRIPS BY FEM

by

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February,2002

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Ms.Sc.THESIS EXAMINATION RESULT FORM

We certify that we have read the thesis, entitled " **STRESS ANALYSIS OF BOLTED COLD FORMED STEEL STRIPS BY FEM** " completed by **S.Özmen ERUSLU** under supervision of **Assoc.Prof.ÇİÇEK KARAOĞLU** and that in our opinion it is fully adaquate ,in scope and in quality,as a thesis for the degree of Master of Science

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ABSTRACT

In this work a finite element model with three dimensional solid elements is established with Ansys 5.4 (CAE) program for the structural performance of bolted connections and bearing failure of high yield strength and low ductility cold-formed steel bolted connections under shear. In our work we used two different material namely G300 which has low strength , high ductility and G550 which has high strength, low ductility. We used M12 bolt and two different washer size as smaller and larger during our analysis. And also we did our analysis for two different friction coefficient. Non linear material, geometrical and contact analysis is carried out .The failure mode of investigation is the bearing failure of cold formed steel strip around the bolt holes. The predicted bearing resistance and the strain and stress values of the bolted connections compare well with referance test data. It is found that contact stiffness and frictional coefficient between element interfaces , clamping force developed in bolt shanks and stress-strain curves are important parametres for accurate prediction of the load extension curves of the bolt connections. Comparison with the results of the finite element modelling shows that the existing design rules in both *BS5950: Part 5*[2] and *Eurocode 3: Part 1.3*[3] give unconservative bearing resistances when thick steel are used.

ÖZET

Bu çalışmada yüksek mukavemetli ve düşük rijitliğe sahip civatalı bağlantıların çeki yükü altındaki yapısal davranışını tespit etmek için Ansys 5.4 programı ile üç boyutlu bir sonlu eleman modeli oluşturulmuştur. Çalışmada düşük mukavemetli, yüksek sünekliğe sahip G300 çeliği ile yüksek mukavemetli ve düşük rijitliğe sahip G550 çeliği kullanılmıştır. Analiz boyunca büyük ve küçük olmak üzere iki tip pul ile M12 civata kullanılmıştır. Ayrıca iki farklı sürtünme katsayısı için analiz yapılmıştır. Yapılan çalışmada civata deliği çevresindeki malzeme davranışı önem kazanmaktadır. Birim uzama ve gerilme değerleri referans data ile karşılaştırılmıştır. Civata bağlantılarının yük-uzama eğrilerinin doğru tespiti için temas rijitliğinin ,sürtünme katsayısının, gerilme-birim uzama eğrilerinin ve bağlama kuvvetinin önemli parametreler olduğu ortaya çıkmıştır. BS5950 ve Eurocode3 dizayn kurallarının sonlu eleman çözümleri ile karşılaştırıldığında kalın çelik şeritlerde doğru değerler vermediği ortaya çıkmıştır.

NOMENCLATURE

EPTO : total strain distribution

EPEL: Elastic strain distribution

EPPL : Plastic Strain distribution

Fou =Pull-out capacity of sheet steel

Pbs : Bearing resistance of *BS5950 design standart*

Py : Yield strength of material

S: Distance between left edge of sheet steel and hole left part.

U : Total displacement

W : The strain in the y direction

V: The strain in the z direction

γ_{M2} = Safety factor

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CHAPTER ONE
INTRODUCTION

Cold formed steel products are used extensively in building industry. Many different shapes and sizes of thin steel sections are used as basic building elements. These sheet steels must meet the requirements in the applicable national design standarts. When we investigate design standarts we found evaluation of designs by the time and results as recommendations.

For example according to Australian /New Zealand standart for cold formed steel structures allows for the use of thin (t = 0.9mm) high strength 550Mpa . But due to the low ductility exhibited by sheet steels that are cold reduced to thickness engineers are required by design standarts to use a yield stress and ultimate strength reduced to 75% of the minumum specified values. With the same reason The American Iron and Steel Institute limits the use of thin high strength steels. The ductility criterian specified in these standarts are based on an investigation of sheet steels by Dhalla nad Winter which did not include the thin higher strength G550 sheet steels that are available today

Rogers and Hancock working for the design of cold-formed steel structures concluded that the ability of G550 sheet steels to undergo deformation is dependent on the direction of load within the material, where transverse specimens exhibit the least amount of overall, local and uniform elongation and also concluded that the low ductility measured in coupon tests did not influence the net section fracture mode of failure, although a modification to the bearing coefficient provisions for thin G550 and G300 sheet steels is required to account for the reduced bearing resistance of *the connected materials. Like this John wiley and Sons worked for cold-formed steel design in 1991. . (Rogers & Hancock, 1998, p 798)*

As a result design expressions according to design standarts are semiemprical expressions formulated according to test data of specific ranges of material properties and geometrical dimensions.

These semiemprical expressions are applicable for cold-formed steel strips with high ductility and design strengths between 280 N/mm^2 and 350 $N/\ mm^2$ they may not be suitable for high strength cold formed steel strips of low ductility. With the advancement of steel technology, cold formed steel strips with design yield strength up to 550 $N/\ mm^2$ are now available for building applications. Although the increment in strength is highly desirable the ductility of high stregth cold formed steel strips is significantly reduced. This may be affect adversely the structural performance of the high strength cold formed steel strips, particularly at connections where local stresses and strains are very high leading to premature failure. A close examination on the strength and the deformation characteristics of the bolted connections between cold formed steel strips is desirable in order to provide efficient and safe design recommendations for high strength cold formed steel structures.(Chung & Ip , 2000,p 1272)

In our work we aim to investigate the structural performance of bolted connections of cold-formed steel structures and search to add the new design recommadations with the light of the previous researchs

CHAPTER TWO
AN OVERVIEW TO COLD-FORMED STEEL STRUCTURES

Cold-formed steel offers versatility in building because of its lightweight and ease of handling and use. Cold-formed steel represents over 45 percent of the steel construction market, and this share is increasing.

2.1 What is Cold-Formed Steel?

In building construction, there are primarily two types of structural steel: hot-rolled steel shapes and cold-formed steel shapes. The hot-rolled steel shapes are formed at elevated temperatures while the cold-formed steel shapes are formed at room temperature. Cold-formed steel structural members are shapes commonly manufactured from steel plate, sheet or strip material. The manufacturing process involves forming the material by either press-braking or cold roll-forming to achieve the desired shape. Examples of the cold-formed steel are corrugated steel roof and floor decks, steel wall panels, storage racks and steel wall studs. Press-braking is often used for production of small quantity of simple shapes. Cold roll-forming is the most widely used method for production of roof, floor and wall panels. It is also used for the production of structural components such as Cees, Zees, and hat sections. Sections can usually be made from sheet up to 60 inches (1.5m) wide and from coils more than 3,000 feet (1,000m) long.

2.2 Effect of Cold Work

When steel is formed by press-braking or cold rolled-forming, there is a change in the mechanical properties of the material by virtue of the cold working of the metal. When a steel section is cold-formed from flat sheet or strip, the yield strength, and to a lesser extent the ultimate strength, are increased as a result of this cold working, particularly in the bends of the section.

Since cold work produced during forming increases the strength of the steel, it will permit the designer to treat the formed steel as a stronger material than the original unformed steel. Often, it is possible to take a weighted average of the post-forming strength of the cross section and use this higher figure as the overall strength of the steel. A shape that is cold worked to a higher degree will have a higher yield strength, which may make it more appropriate for certain applications. Sometimes cold work is added by overbending and straightening. The end result is that a more complex shape may have higher strength than a less complex shape. Other factors that can affect cold working include roll pressure, corner radius and the properties of the steel.

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Depending on the type of cold-forming and the thickness of the steel, the amount of strength increase could reach 20% to 50%. However, the ductility of the steel is reduced as result of cold-forming process.

2.3 Properties of Cold-Formed Steel

Generally, the grades of carbon steel and high strength low alloy steel used for cold-formed steel products are characterized by two main properties: the yield point and the tensile strength. Other important properties are ductility, hardness and weldability.

The yield point of the steels commonly used for cold-forming ranges from 33 to 55 ksi (230 to 380 MPa), and may be higher. Tensile strength and ductility are important because of the way they relate to formability, and because of the local deformation demands of bolted and other types of connection. In members that include bolted connection because of special design, may be subject to high stress concentrations, the tensile strength often must be taken into account. The ratio of tensile strength to yield strength for cold-formed steels commonly ranges from 1.2 to 1.8. However, steels with a lower ratio can be used for specific applications.

2.4 Advantages of Cold-Formed Steel

Cold-formed steel offers many advantages, including: ease of prefabrication and mass production, uniformity of quality, low weight, economy of transportation and handling, and quick and simple erection or installation.

The obvious benefit of permanent metal decks forms over the conventional wood form system is a significantly reduced construction cost and schedule since the metal forms remain in place as opposed to the wood forms which must be stripped. In addition, permanent forms provide a safe working platform for the subsequent operations of shear stud placement and re-bar installation of Hancock

(Helen Chen American Iron and Steel Institute)

2.5 G550 and G300 sheet steels

The steels that were investigated as a part of this paper were produced using a process called cold reduction, which can be used to increase the strength and hardness as well as to provide an accurate thickness for sheet steels and other steel products. This process causes the grain structure of cold reduced steels to elongate in the rolling direction which produces a directional increase in material strength and a decrease in material ductility. Various types of heat threatment exist and are used for different steel products. Both G300 and G500 sheet steels are stress relief annealed, i.e the total dislocation density is reduced by annealing altough recrystallisation does not occur. Stress relief annealing involves heating the steel to below the recrystallition temperature, holding the steel until temperature is constant throughout its thickness, then cooling slowly. G300 sheet steels are annealed to a greater extent in comparison with G550 sheet steels. This procedure results in near isotropic material properties for the mild sheet steels (G300) although some prefered grain orientation remains. The G550 sheet steels that were used for this research must be differentiated from other sheet steels whose high yield stress and ultimate strength values are obtained by means of an alloying process, i.e. high strength low alloy steels. The material property requirements for G300 or similar mild sheet steels and G550 or grade E sheet steels are specified in Australia by AS and in North America by the ASTM standarts. (Rogers & Hancock ,1999,pp 124,125)

CHAPTER THREE
DESIGN CRITERIA

3.1 Cold-formed steel bolted conection design provisions

The nominal cross section capacity of member which is not subject to shear and fails by material yielding of the gross cross section is formulated for all design standarts except AISI standarts as follows

3.1.1 $N_t = A_g f_y$

where A_g *is the area of the cross section* f_y *is the yield stress.*

The nominal cross section capacity of member which is not subject to shear and fails by rupture of the net cross section away from connections is represented for all of the design standarts except for AISI standarts as follows

3.1.2 $N_t = A_n f_u$

where A_n *is the area of the net cross section and* f_u *is the ultimate strength.*

For the design of thin G550 sheet steels 0.75 f_y *and 0.75* f_u *must be used in Eqs.*

The Australian, Usa and European design standarts all have seperate requirements for net cross section capacity at connections where washers are provided under both the bolt head and nut. The present form of the design equation for the AS/NZS 4600 and AISI design standarts is as follows

Where r is the ratio of the force transmitted by the bolts ,d is the diameter of the bolts and s is the spacing of the bolts perpendicular to the line of the force or for single bolts the width of the sheet. The design formulation for the eurocode design standart is similar to that however d is defined nominal diameter of the bolt hole. The CSA-S136 design standart does not contain a stress reduction factor based on the number and position of bolts in the cross section.

The design bearing capacity Per bolt for connections regardless of the design standart used it is

3.1.3 $V_b = Ctdf_u$

where d is the nominal diameter of the bolt, t is thickness of the sheet steel and C is a bearing coefficient. The Australian /New Zealand and USA design standarts require that C=3 for single lap shear connections,for Eurocode design standart C=2.5. In the Canadian design standart C represents the stability of the hole edge based on the ratio of the bolt diameter to sheet thickness.

End pull-out capacity of a bolted connection is dependent on the length of two parallel lines which extend from the bolt hole in the direction of the applied force The nominal end pull-out capacity Per bolt is given for Australian/New Zealand and USA design standarts

3.1.4 $V_f = tef_u$

where e is the distance measured parallel to the direction of the applied force from the center of the standart hole to the nearest edge of an adjacent hole or to the end of the connected part. The end-pull out capacity determined using the Eurocode design standart is formulated in similar fashion , however, the nominal capacity is reduced by a factor

3.1.5 $V_f = tef_u/1.2$ *(3.4)*

The nominal end-pull out capacity Per bolt of a connection designed using the CSA-S136 design standart is determined as previously described for the net cross section tension capacity.

3.1.6 $V_f = A_n f_u$

where A_n *net cross sectional area used for each bolt. (Rogers & Hancock, 1999,pp125,126,127)*

According to the design recommedations thre rules for geometry is determine in figure 3.1

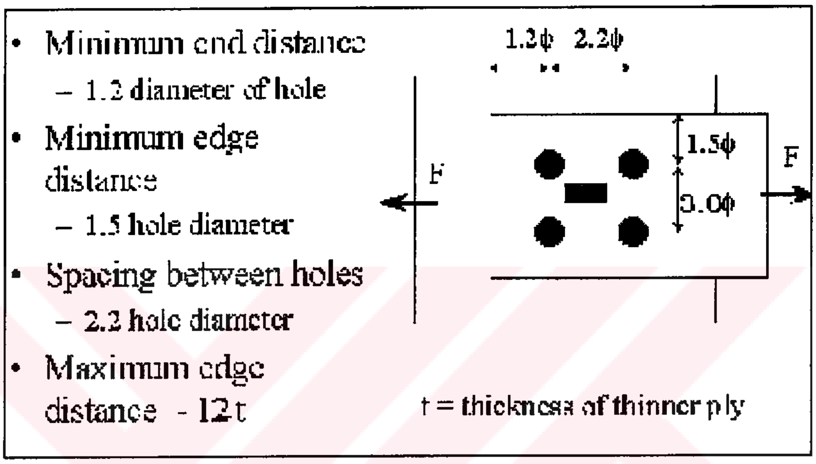

Fig 3.1 Rules For Geometry

3.2 Load Carrying Capacity of G550 and G300 Sheet Steels Under Static Shear

Cold formed structural members may be joined with bolted connections, which are designed with the aid of applicable national design standards. The ultimate load carrying capacity of a connection will be governed by one of many failure modes including; bearing, end pull-out, net section fracture, bolt shear, block shear rupture. It was concluded that there are a number of problems with the existing load capacity formulations contained in the current cold formed steel design standards, based on observations made during the testing of thin bolted connection specimens. A modification to the bearing coefficient provisions for thin G550 and G300 sheet steels is necessary to account for the reduced bearing resistance of the connected *materials. This reduction in bearing resistance is related more to the steel thickness than to the steel grade. Furthermore, a detailed analysis of the procedure used to identify the cause of failure in bolted connections is needed to ensure that accurate failure mode assessments are made, and ultimately to ensure that accurate design equations are formulated. Misidentification of failure modes and the misuse of data can lead to serious errors in the accuracy and applicability of design equations.*

(Roger & Hancock, 1998 research report)

CHAPTER 4
FINITE ELEMENT METHOD AND FORMULATION

4.1 History of the Finite Element Method and Ansys

The finite element method is a numerical procedure that can be applied to obtain solutions to variety of problems in engineering. Steady, transient, linear or nonlinear problems in stress analysis, heat transfer, fluid flow, and electromagnetisim problems may be analyzed with finite element methods. The origin of the modern finite element method may be traced back to the early 1900s, when some investigators approximated and modeled discrete equivalent elastic bars. However Courant (1943) has been credited with being the first person to develop the finite element method his book was published in the early 1940s. Courant used piecewise polynomial interpolation over triangle subregions to investigate torsion problems

The next signifacant step in the utilization of finite element methods was taken by Boeing in the 1950s when Boeing followed by others used triangular stress elements to model airplane wings. Yet it was not until 1960 that Clough made the term "finite element popular ". During 1960s, investigators began to apply the finite element method to the other areas of engineering, such as heat transfer and seepage flow problems . Zienkiewitz and Cheung (1967) wrote the first book entirely devoted to the finite element method in 1967. In 1971, Ansys was released for the first time.

ANSYS is a comprehensive general purpose finite element computer program that contains over 100,000 lines of code. ANSYS is a capable of performing static, *dynamic, heat transfer, fluid flow and electromagnetism analyses. ANSYS has been leading FEA for well over 20 years. The current version of ANSYS has a completely new look, with multible windows incorporating Graphical User Interface (GUI), pull down menus, dialog boxes, and a tool bar. Today you will find ANSYS in use in many engineering fields, including aerospace, automotive, electronics and nuclear. In order to use ANSYS or any other "FEA computer program intelligently it is imperative that one first fully understands the underlying basic concepts and limitations of the finite element methods."*

*ANSYS is a very powerful and impressive engineering tool that may be used to solve a variety of problems . However, a user without a basic understanding of the finite element methods will find himself or herself in the same predicament as a computer technician with access to many imperessive instruments and tools who can not fix a computer becuse he or she doesn't understand the inner workings of the computer. (*Moaveni Saeed , Finite Element Analysis with Ansys *)*

4.2 Goals of Analysis

The goals of analysis in the design cycle coincide with the goals of all CAD/CAM software. Three goals of analysis in the design cycle are:

4.2.1 Reduce the Cost to Introduce a New Product

From the perspective of an engineering manager, analysis may seem a frivolous expense that hasn't yet been necessary. But, the potential for analysis to reduce the number of prototype presents great cost and timing reduction opportunities for most design cycles. Additionally, analysis can decrease the time and expense of testing and validating these prototypes. Of course testing should be used to validate finite element techniques. By decreasing the testing and prototyping time, one can arrive at the production date quicker. Analysis can reduce the number of components in a system by assessing combined parts and optimizing linkages. It can result in lighter parts and less expensive materials. Additionally, the potential to reduce the factor of safety may reduce the part cost. Factors of safety provide insurance for two conditions:

- Unknown operating conditions
- Inadequate design verification

The portion of the factor of safety resulting from inadequate design verification can be reduced inversely to the confidence gained by performing the analysis. However, factors due to unknown operating conditions will not change.

4.2.2 Produce a Superior Product

A primary use of analysis throughout history has been to improve a product's performance. This is accomplished, for example, by reducing weight or increasing durability. Furthermore, decreased failures may reduce warranty claims, causing greater customer satisfaction and possibly lower warranty costs.

4.3 Things all Engineers Should Know Before Using FEA

4.3.1 FEA is Approximate.

The first issue to understand in Finite Element Analysis is that it is fundamentally an approximation. The underlying mathematical model may be an approximation of the real physical system (for example, the Euler-Bernoulli beam ignoring shear deformation). The finite element itself approximates what happens in its interior with interpolation formulas. The interior of a 2-D or 3-D finite element has been mapped to the interior of an element with a perfect shape, so a severely distorted element can not deform in a manner that has an accurate match to the real physical response. Badly shaped (by distortion, warping or extreme aspect ratio) elements can give less accurate results. Elements approximate the local shape of the real body. Numerical analysis difficulties such as ill-conditioned matrices may reduce the accuracy of calculated results. A linear analysis is an approximation of the real behavior. The loading of the model is an approximation of what happens in the real world. The boundary conditions approximate how the structure is supported by the outside world. The material properties assumed are approximate. Flaws are not represented unless the analyst incorporates a model of a flaw. The overall dimensions of the model approximate real structures that are manufactured within a tolerance. Many details are idealized, simplified, or ignored. Element results may be reported at integration points or nodes, not continuously evaluated with the interpolation functions over the whole element interior. Stress and strain results are based on the derivatives of the displacement solution, amplifying the errors.

The result of an analysis contains the accumulated errors due to all of the contributing approximations. Good analysis and interpretation of results requires knowing what is an acceptable approximation, development of a complete list of what should be evaluated, appreciation of the need for margin of safety, and comprehension of what remains unknown after an analysis.

Engineers in every industry are integrating finite element analysis (FEA) into the design cycle to ensure that their products are safe, cost-effective, and fast to market. But, analysis is not as simple as putting a CAD model into any FEA package. There are more software options today than ever before. For many years, engineers were limited to using linear static stress analysis. More recently, finite element packages have been extended to include nonlinear static stress, dynamic stress (vibration), fluid flow, heat transfer, electrostatic, and FEA- based stress and motion analysis capabilities. These capabilities are frequently combined to perform analyses that consider multiple physical phenomena, and are tightly integrated within a CAD interface. A finite element model is a discrete representation of the continuous, physical part being analyzed. This representation is created using nodes, which are connected together to form elements. The nodes are the discrete points on the physical part where the analysis will predict the response of the part due to applied loading. This response is defined in terms of nodal degrees of freedom (DOF). For stress analysis, up to six degrees of freedom are possible at each node (three components of translation and three components of rotation), depending on the element type selected (e.g., beam, plate, 2D, and 3D elements).

4.4 Design Criteria

In any analysis, an engineer first needs to determine the significant physical phenomena and environmental conditions to which the part will be exposed, and also the desired design objective. For example, one of the most common concerns for engineers involves maximizing the part's durability. The first step in an analysis is to determine whether the design will be subject to static or dynamic conditions. In its real-world application, is the part fixed in space, subject to vibration, or does it move relative to other parts in the assembly? What happens when you run the entire product through its motion cycle? For years, engineers faced with expensive computing resources have simplified the problem by using static FEA software to calculate stresses at a single instant in time. This method works only if the design does not experience impact, motion, or changes in applied loads over time.

4.5 Multiphysics:

In addition to considering a part's ability to withstand mechanical stresses, FEA software often enables engineers to predict other real-world stresses, such as: the effects of extreme temperatures or temperature change (heat transfer analysis), the flow of fluids (fluid flow analysis), or voltage distributions over the surface or throughout the volume of an object (electrostatic analysis).Often these effects work in unison, so it is important that the FEA program can consider their effects on one another. For instance, a computer chip may be heating up over time, cooled down by airflow from a fan, vibrated against other parts, and electrically charged. A typical approach might be to isolate and calculate each variable, then feed the results into the FEA program one at a time. Yet each variable could also affect all the others, so either a coupled analysis or tools for relating results is often necessary.

4.6 3-D Elasticity Theory For Solid Elements

$$\sigma = \{\sigma\} = \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix}, or \left[\sigma_{ij}\right] \qquad \textbf{Strains :}\ \varepsilon = \{\varepsilon\} = \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \end{Bmatrix}, \qquad \text{or } \left[\varepsilon_{ij}\right]$$

4.6.1 Stress-Strain Relations

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & \nu & 0 & 0 & 0 \\ \nu & 1-\nu & \nu & 0 & 0 & 0 \\ \nu & \nu & 1-\nu & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1-2\nu}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1-2\nu}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix}$$

or $\sigma = E\varepsilon$

4.6.2 Displacement: $u = \begin{Bmatrix} u(x,y,z) \\ v(x,y,z) \\ w(x,y,z) \end{Bmatrix} = \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix}$

4.6.3 Strain-Displacement Relation

$$\varepsilon_x = \frac{\partial u}{\partial x}, \varepsilon_y = \frac{\partial v}{\partial y}, \varepsilon_z = \frac{\partial w}{\partial z} \qquad \gamma_{xy} = \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y} \quad , \quad \gamma_{yz} = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}$$

$$\gamma_{xz} = \frac{\partial u}{\partial z} + \frac{\partial w}{\partial x}$$

or $\varepsilon_{ij} = \frac{1}{2}\left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i}\right)$ (i,j=1,2,3)

4.6.4 Equlibrium Iterations

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} + f_x = 0, \qquad \frac{\partial \tau_{yx}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} + f_y = 0$$

$$\frac{\partial \tau_{zx}}{\partial x} + \frac{\partial \tau_{zy}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + fz = 0$$

4.6.5 Finite Element Formulation

$$u = \sum_{i=1}^{N} N_i u_i \quad , \; v = \sum_{i=1}^{N} N_i v_i \qquad w = \sum_{i=1}^{N} N_i w_i$$

In Matrix Form $\begin{Bmatrix} u \\ v \\ w \end{Bmatrix} = \begin{bmatrix} N_1 & 0 & 0 & N_2 & 0 & 0 & \cdots \\ 0 & N_1 & 0 & 0 & N_2 & 0 & \cdots \\ 0 & 0 & N_1 & 0 & 0 & N_2 & \cdots \end{bmatrix}_{(3\times 3N)} \begin{Bmatrix} u_1 \\ v_1 \\ w_1 \\ u_2 \\ v_2 \\ w_2 \\ . \\ . \\ . \end{Bmatrix}_{(3N\times 1)}$

Or $u = N\,\mathrm{d}$

Using relations between strain displacement relation and displacement field we can derive the strain vector.

$$\varepsilon = \mathbf{B}\ \mathbf{d}$$

$$(6\times 1)(6\times 3N)\times(3N\times 1)$$

4.6.6 Stiffness Matrix

$$k = \int_v B^T E\ \mathrm{B}\ \mathrm{dv}$$

$$(3\times N) \quad (3\mathrm{N}\times 6)\times(6\times 6)\times(6\times 3N)$$

Numerical quadratures are often needed to evaluate the above integration. Rigid-body motions for 3-D bodies (6 components): 3 translations, 3 rotations.
These rigid-body motions (singularity of the system of equations) must be removed from the FEA model to ensure the quality of the analysis.

CHAPTER 5
MODELLING AND MESHING

5.1 Numerical Investigation

With the advent of computer hardware and software, numerical simulation became popular. In the field of structural engineering, results from finite element simulation may provide detailed information on the stress and strain distributions in structures. Such information is not easily available from experiments. And therefore numerical investigation may be used to provide supplementary data for improved understanding. Furthermore parametric studies on the finite element models may be performed to improve the efficiency of structural design. The pioneering work in three dimensional modeling of bolted connection is attributed to Krishnamurthy. Then recently Gebbeken N and Bursi S. modelled beam to column connections with bolted extended end plates also Chung KF, Ip KH. Used solid elements with contact elements and they carry out non linear material, geometrical and contact analysis to investigate the structural performance of bolted connections.

5.2 Finite element Modelling

In the present study the ANSYS 5.4 finite element package is used to predict the performance of bolted connections between cold formed steel strips under shear. Three dimensional eight node iso parametric solid elements SOLID 45 are employed to model all the components namely the cold formed steel strips , the bolt and also the washers in order to capture yielding propagation throughout the material thickness. Such elements are especially suitable for the plasticity type problem since they allow discontinuous strain fields in simulating shear bands. The element is defined by eight nodes having three degrees of freedom at each node : Translations in the nodal x,y,z directions. The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities. It is defined by orthotropic material properties. Orthotropic material directions corrrespond to the element coordinate directions.

5.2.1 Meshing

Production of a good quality mesh is a major topic. The mesh should be fine enough for good detail where information is needed, but not too fine beceuse the analysis will require considerable time and space in the computer. A mesh should have well-shaped elements i.e : only mild distortion and moderate aspect ratios. This can require considerable user intervention, despite FEA software promotional claims of automatic good meshing. The user should put considerable effort into the generation of well-shaped meshes. This will include setting element densities, gradients in element size, concatenation of lines or areas to permit mapped meshing, playing with automatic meshing controls, and re-meshing individual areas and volumes until the result looks right.

Most finite elements are stiffer than the real structure. For these elements, a coarse mesh generally results in a structure that underpredicts deflection, and overpredicts buckling load and vibration frequency. A coarse mesh is less sensitive to and "hides" stress concentrations. A fine mesh generally gives an answer closer to the exact solution. It is also results in larger models, more data storage, and longer model solution and display times.

A fundamental idea of using the finite element procedure is that the body is sub-divided up into small discrete regions known as finite elements. These elements defined by nodes and interpolation functions. Equations are written for each element and these elements are assembled into a global matrix. Loads and constraints are applied and the solution is then determined.

The question is: How small do I need to make the elements before I can trust the solution? In general there are no real answers on this. It will be necessary to conduct convergence tests. By this we mean that you begin with a mesh discretization and then observe and record the solution. Now repeat the problem with a finer mesh (i.e. more elements) and then compare the results with the previous test. If the results are nearly similar, then the first mesh is probably good enough for that particular geometry, loading and constraints. If the results differ by a large amount however, it will be necessary to try a finer mesh yet.

Finer meshes come with a cost however: more calculational time and large memory requirements (both disk and RAM)! It is desired to find the minimum number of elements that give you a converged solution. In a solid mechanics problem, this would be done by creating several models with different mesh sizes and comparing the resulting deflections and stresses.

In our study we try both coarse mesh and finer mesh respectively and find better result near the finer mesh. Usually we take smart size of program for meshing the sheet steels. We both try solid modelling and two dimensional shell modelling we have got a trouble especially in shell modelling for arriving the converged solution. Before we mesh the model we define the material properties of components namely cold-formed strips, washer and bolt which is integtarated each other.

5.2.2 Contact Element

In the study contact interfaces between the cold formed steel strips , the bolt and the washer are modelled by contact elements CONTAC 49 .

In order to simplify the model , only half of the specimen is modelled. We took symmetry along the longitudinal axis of the test specimen. Bottom cold formed steel plate is assumed to be rijid and fixed in space; it is modelled by a single element with all degrees of freedom restrained. The bolt assumed to be threadless and forms an integral component with the washer. The bolt - washer component is assumed to be linear elastic throughout the analysis. The symetry part of the specimens are restrained from the symetry axis against the motion The true stress-strain curves established from reference coupon tests are adopted for cold formed steel strips in order to establish both yielding and strength degradation during deformation. Nonlinear material, geometrical and contact analysis is then carried out. The finite element modeling may provide detailed information on the yield zones of the cold formed steel strips also the resistance contributions of the bearing and the frictional actions respectively. Furthermore the effect of clamping forces in bolt shanks will also be examined.

We take a coupon test as a reference and take material properties of cold-formed steels from true stress-strain curves which is based on coupon test there. You can see this coupon tests'data in table 5.1 and stress-strain curve in figure 5.1 below.

Table 5.1 Measured Material properties of cold-formed steel strips in coupon tests

Test specimen	*Thickness*	*Elastic Modulus*	*Yield Strength*	*Tensile Strength*
	Nominal (mm)	(kN/mm^2)	(N/mm^2)	$(N/mm^2$
G550-1	*1.6*	*222.2*	*616.7*	*625.1*
G550-2	*1.6*	*220.0*	*622.1*	*648.4*
G550-3	*1.6*	*212.9*	*623.2*	*641.6*
G300-1	*1.5*	*203.3*	*320.1*	*440.1*
G300-2	*1.5*	*206.7*	*321.2*	*470.4*
G300-3	*1.5*	*202.7*	*320.3*	*450.3*

(Chung & Ip, 2000,1273)

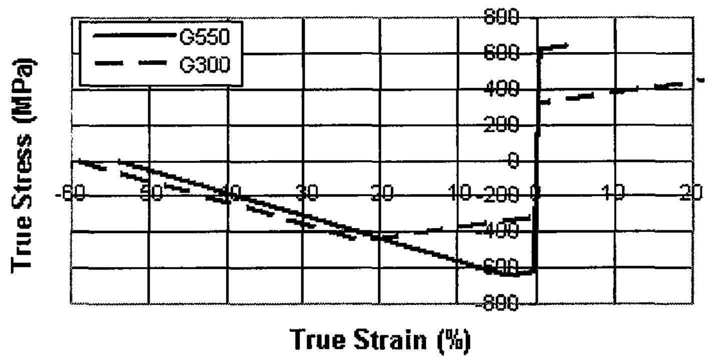

Figure 5.1 True-Stress-Strain curve for G550 and G300 sheet steels
(Chung & Ip,1999)

Table 5.2 Cold-formed Steels Properties For Finite Elment Analysis

Components **CFS strips**	**Elastic modulus** (kN/mm^2)	**Yield Strength** (N/mm^2)	**Tensile strength** (N/mm^2)
G550 (t=1.6mm)	220	620	638.5
G300 (t=1.5mm)	205	320	454
Bolt- washer	205.0	-	-

In our study we take material properties in table 5.2 . We use two different material see in table namely G300 and G550 we take lap shear test results as a reference and use two different type washer

Washer1 =External diameter 25.7mm, Internal diameter 13mm Thicknes: 2.3mm

Washer2 = External diameter : 32.1mm, Internal diameter : 14.6mm Thickness: 3mm

You see the model of our work below in figure 5.2

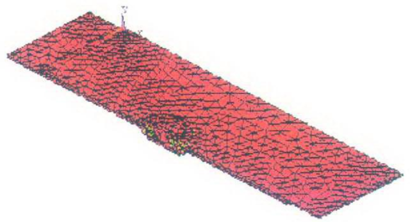

Figure 5.2 Finite Element Modelling of our Specimen

5.2.3 Contact stiffness, frictional coefficient and clamping force inthe bolt

Inorder to model the contact condition between the cold formed steel strips and the bolt-washer component, the generation of contact elements according to the penalty technique. Each contact element is defined by a node (contact node) on one surface and four other nodes on another surface (target surface) and thus each contact element is tetrahedral in shape. The interfaces overpenetrating is prevented through the devolopment of contact forces. These forces act on contact nodes both normal and tangential. The compatibility control is governed by contact stiffness which should be large enough to maintain accuracy but small enough to convergence. The nodes of cold formed steel strips around bolt holes are taken as contact nodes while facets of shank are selected as target surfaces. Contact elements were generated between each contact node and all target faces. In a similar manner contact conditions for cold formed steel strips to the washer are established.

The value of contact stiffness is taken as 2000 $N/\ mm$. The elastic coulomb friction coefficient μ is taken to be 0.2 at first then is taken 0.4 for all contact interfaces. Both the values of contact stiffness and the friction coefficient are assumed to remain constant throughout the analysis. We take clamping force as 8000 N throughout the analysis then take 10000 N for comparision. We predicted about eight type model

"During first iteration of the analysis, the washer and the cold formed strips will push against each other by inducing tensile stresses in the bolt shank while compressive stresses in areas beneath the washer are established." (Chung & Ip, 2000, 1278)

CHAPTER 6

DEFINITION OF PROBLEM AND SOLUTION PROCEDURE

The finite element model incorporates material, geometrical and contact non-linearity and therefore nonlinear analysis is required. In the present investigation, the solution procedure requires the full load to be applied in a series of small increments so that the solutions may follow the load-extension curves of reference lap shear test (table 6.1) closely. A value of %5 is recommended as the maximum plastic strain increment after each incremental load. In order to simulate the lap shear test results, a series of displacement increments is applied to the end of the cold formed steel strips up to a total extension of 3mm. The size of each increment is automatically adjusted by the program .

At each sub-step the solution is obtained through a number of equlibrium iterations. Which is accomplished by the Full Newton Raphson procedure. The nodal displacements, the out of balance forces, and the tangent stiffness matrix of the structure are updated after each equlibrium iteration. A force based convergence criterion is adopted which requires the square root of the sum of squares(SRSS) of the load imbalance to be less than 1% of the SRSS of the applied loads in an equlibrium iteration.. As a boundary conditions I restrained specimen from left of the lower part of lap shear connection.

Table 6.1 Lap Shear Test Results

Test	Cold formed steel strips		Washers	Applied Load (kN) under extension at		
	Grade	Nominal Thickness (mm)		1 mm	2 mm	3 mm
A11A-1	G550	1.6	A	19.85	26.44	28.96
A11A-2				18.81	25.20	28.82
A11A-3				21.77	26.78	28.92
		(Avaraged value)		20.14	26.14	28.90
A11B-1	G550	1.6	B	16.66	23.31	27.18
A11B-2				17.99	24.84	29.11
A11B-3				17.79	24.65	29.77
		(Avaraged value)		17.48	24.27	28.69
A21A-4	G300	1.5	A	14.73	18.44	17.91
A21A-5				16.13	18.47	18.63
A21A-6				13.61	17.26	19.04
		(Avaraged value)		14.83	18.06	18.53
A21B-4	G300	1.5	B	13.66	17.66	20.79

(Chung & Ip, 2000,1274)

6.1 Basic datas for G550 and G300 test specimens after lap shear test

6.1.1 G550 test specimens

The maximum load carrying capacities of test specimens A1AA are found to be very close to those test specimens A11BB at respective extensions despite the use of larger washers in test specimens A11BB. As the same torque is used for bolt installation in both sets of test specimens, the compressive normal stresses and also the frictional stresses between the washers and the cold formed strips will vary inversely with the size of washers. The two effects of reduced frictional stress and increased contact area tend to cancel each other and the load carrying capacities of both sets of test specimens are very close. It is further considered that the resistance contribution of bearing action between the bolt shanksand the cold formed steel *strips is much more important than the contribution of frictional action between the washers and the cold-formed steel strips. The end sections of test specimens A11A and A11B were found to curl up after peak load carrying capacities are reached after the extension exceeded 3mm.*

6.1.2 G300 test specimens

The maximum load carrying capacities of test specimens A21B are found to be approximately 10% higher than those in test specimens A21A due to the presence of the larger washers. This suggest that the resistance contribution of frictional action between the washers and cold-formed steel strips may be significant for low strength cold-formed steel strips. Furthermore curling of the ends of the specimens was observed at an extension of about 2mm before the peak load carrying capacities were reached. Both the coupon tests and the lap shear tests provide a basic data in the forms of stress- starin curves and load extension curves for the calibration of finite element modelling. (Chung &I p,2000, pp1273-1275)

6.2 Stress Distribution

The Von mises stress distribution around bolt holes of G550 test specimens in the cold formed steel strips at two different extensions are presented in figure 6.1 and 6.2. It is shown that for those elements around bolt hole in direct contact with the bolt shank, it is revealed stress concentration due to direct end bearing. As the extension increases yielding occurs in these elements and the yield zone increases in size rapidly as shown in figure6.2, figure 6.10 for G300 and G550 specimens respectively. For G300 specimens yielding is only confined to a small area of material when compared with the G550 test specimens . The other figures figure6.3-6.8 show the large strain variations in the vicinity of these elements in the x,y,xy directions at two different extension.

When we used 1.5mm sheet steel with μ=0.2 and small bolt we found stress distribution and strains below

Fig 6.1 Stress distribution of bolted connection G300 with extension of 0.5 mm

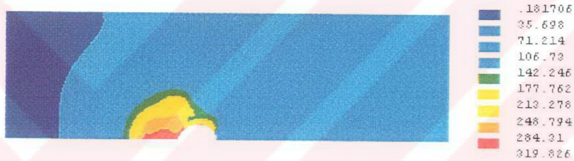

Fig 6.2 Stress distribution of bolted connection G300 with extension of 3 mm

Fig 6.3 G300 Specimen's X direction strain at 0.5 mm extension

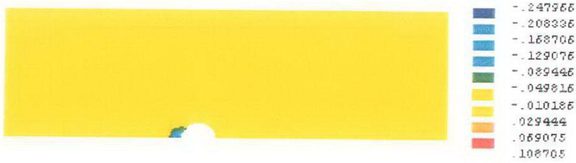

Fig 6.4 G300 Specimen's X direction strain at 3 mm extension

Fig 6.5 G300 Specimen's Y direction strain at 0.5 mm extension

Fig 6.6 G300 Specimen's Y direction strain at 3 mm extension

Fig 6.7 G300 Specimen's XY direction shear strain at 0.5 mm extension

Fig 6.8 G300 Specimen's XY direction shear strain at 3 mm extension

It is seen that the strain increasing at x direction more than y direction and at xy direction the strain increasing is not as much as x and y

When we used 1.6mm sheet steel with μ =0.2 and small bolt we found stress distribution and strains below in figure 6.9- figure 6.16

Fig 6.9 Stress distribution of bolted connection G550 with extension of 0.5 mm

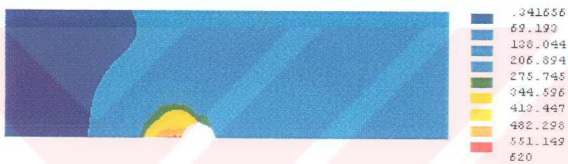

Fig 6.10 Stress distribution of bolted connection G550 with extension of 3 mm

Fig 6.11 G550 Specimen's X direction strain at 0.5 mm extension

Fig 6.12 G550 Specimen's X direction strain at 3 mm extension

Fig 6.13 G550 Specimen's Y direction strain at 0.5 mm extension

Fig 6.14 G550 Specimen's Y direction strain at 3 mm extension

Fig 6.15 G550 Specimen's XY direction shear strain at 0.5 mm extension

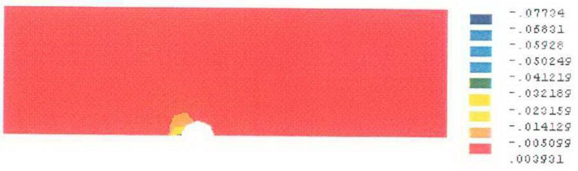

Fig 6.16 G550 Specimen's XY direction shear strain at 3 mm extension

It is seen that especially left part of hole the strain rising is much more than the other part and we may say the failure will start from there

Also we may say the distance of hole to the left edge of the sheet steel is more important than the other side for the single bolt lap shear connections.

6.3 Bearing and Frictional Actions

Near the bearing resistance the frictional resistance may be developed along the contact interfaces. The frictional forces are dependent on the clamping forces in the bolts, the frictional coefficient between the contact interfaces and size of the washers.

When we compare the stress and strain distribution of G300 test specimen with different friciton coefficient at 3 mm extension. We see the results in figure6.17-6.18

Fig 6.17 Stress distribution of bolted connection G300 with large washer at extension of 3 mm for $\mu = 0.2$

Fig 6.18 Stress distribution of bolted connection G300 with large washer at extension of 3 mm for $\mu = 0.4$

It is shown that the increasing friction coefficient does not effect the von mises stress distribution and we may say we don't see clearly effect of friction coefficient at vonmises stress distribution at 3mm extension . It is also same for small washer G300 specimens. When we investigate G550 specimens we see small differences with different friction coefficient value in figure 6.19-6.20

Fig 6.19 Stress distribution of bolted connection G550 with small washer at extension of 3 mm for $\mu = 0.2$

Fig 6.20 Stress distribution of bolted connection G550 with small washer at extension of 3 mm for $\mu = 0.4$

On the other hand the strain distribution of components for G550 test specimens in figure 6.21-6.22 doesn`t change at all.

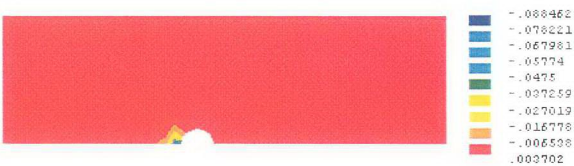

Fig 6.21 G550 Specimen's X direction strain at 3 mm extension with $\mu = 0.2$

Fig 6.22 G550 Specimen's X direction strain at 3 mm extension with $\mu = 0.4$

We observed that the vonmises stress and strain distribution is not change when we change the friction coefficient. We don't say exact effect of friction coefficient from this but we may say its effect is not much more for load carrying capacity. We work to see the nearly values of effects at the end with load extension curves and with failure modes of specimens.

It should be note that washer size is important parameter for friction resistance capacities of test specimens we search washer effects in figure6.23-6.26

Fig 6.23 Stress distribution of bolted connection G300 with large washer at extension of 1.5 mm for $\mu = 0.2$

Fig 6.24 Stress distribution of bolted connection G300 with small washer at extension of 1.5 mm for $\mu = 0.2$

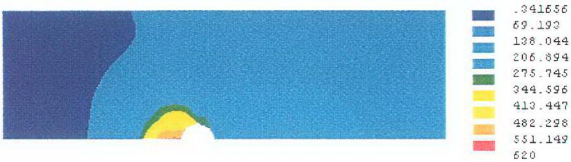

Fig 6.25 Stress distribution of bolted connection G550 with small washer at extension of 3 mm for $\mu = 0.2$

Fig 6.26 Stress distribution of bolted connection G550 with larger washer at extension of 3 mm for $\mu = 0.2$

We see that with a larger washer yield place of plate is increasing at same extension. And we may say that for frictional forces it is important parameter and its' effect is more than friction coefficient.We may see this effect at larger extension and failure positions.

Then we search the effects of clamping force with increase the clamping force 8000N to 10000N in figures 6.27,6.28

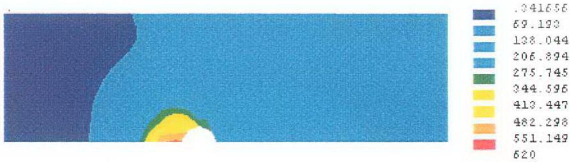

Fig 6.27 Stress distribution of bolted connection G550 with larger washer at extension of 3 mm for $\mu = 0.2$ with 8000 N clamping force

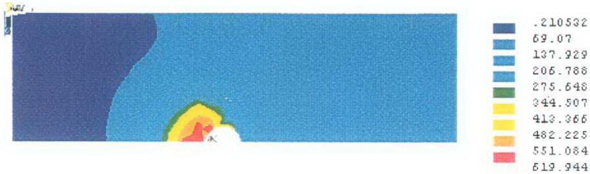

Fig 6.28 Stress distribution of bolted connection G550 with larger washer at extension of 3 mm for $\mu = 0.2$ with clamping force 10000 N

It is seen that clamping force is important parameter for friction resistance of specimens and with rising clamping force load carrying capacity of the specimens increase.

6.4 Elastic-Plastic Behaviour of Sheet Steels

We are investigate elastic – plastic behaviour of steels in this part of thesis.First we deal with graphical projection of elastic-plastic strain distribution for G300 sheet steels.Figure6.29-6.46 G300 show the sheet steels behaviour with large washer for two different friction coefficient.

EPTO show total strain distribution, EPEL show elastic strain distribution, EPPL show plastic strain distribution at various edirections x, y and xy respectively

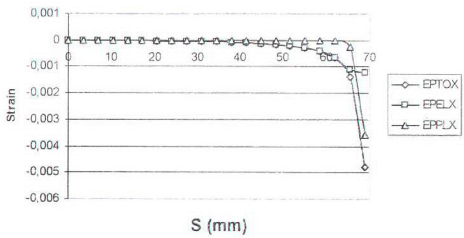

Fig 6.29 Strain distribution of G300 sheets steels for 0.5mm extension at x direction ($\mu = 0.4$)

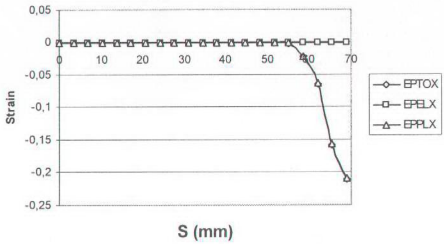

Fig 6.30 Strain distribution of G300 sheets steels for 1.5 mm extension at x direction ($\mu = 0.4$)

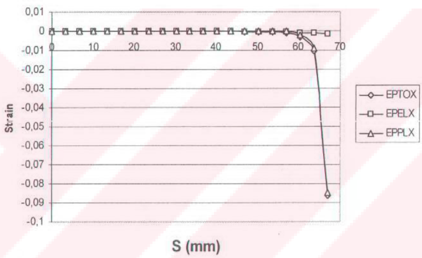

Fig 6.31 Strain distribution of G300 sheets steels for 3 mm extension at x direction ($\mu = 0.4$)

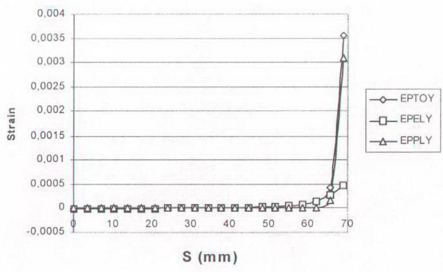

Fig 6.32 Strain distribution of G300 sheets steels for 0.5 mm extension at y direction ($\mu = 0.4$)

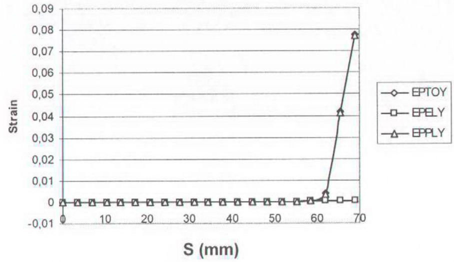

Fig 6.33 Strain distribution of G300 sheets steels for 1.5 mm extension at y direction ($\mu = 0.4$)

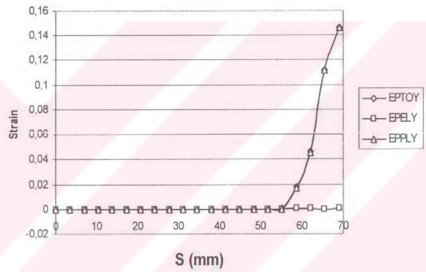

Fig 6.34 Strain distribution of G300 sheets steels for 3 mm extension at y direction ($\mu = 0.4$)

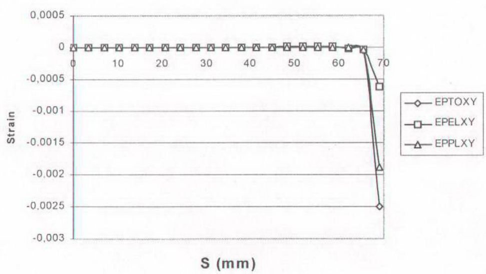

Fig 6.35 Strain distribution of G300 sheets steels for 0.5 mm extension at xy direction ($\mu = 0.4$)

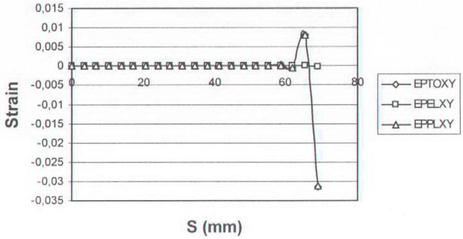

Fig 6.36 Strain distribution of G300 sheets steels for 1.5 mm extension at xy direction ($\mu = 0.4$ **)**

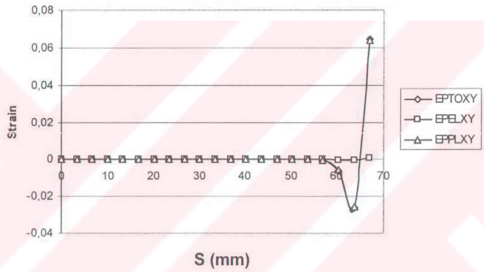

Fig 6.37 Strain distribution of G300 sheets steels for 3 mm extension at xy direction ($\mu = 0.4$ **)**

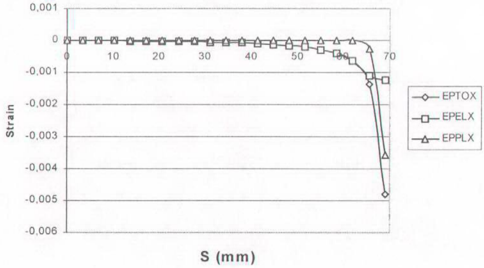

Fig 6.38 Strain distribution of G300 sheets steels for 0.5 mm extension at x direction ($\mu = 0.2$ **)**

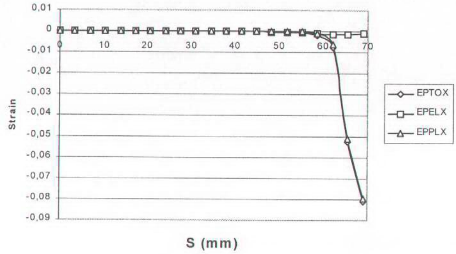

Fig 6.39 Strain distribution of G300 sheets steels for 1.5 mm extension at x direction ($\mu = 0.2$)

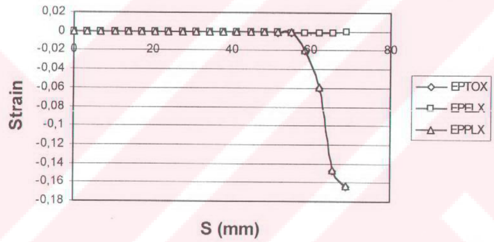

Fig 6.40 Strain distribution of G300 sheets steels for 3 mm extension at x direction ($\mu = 0.2$)

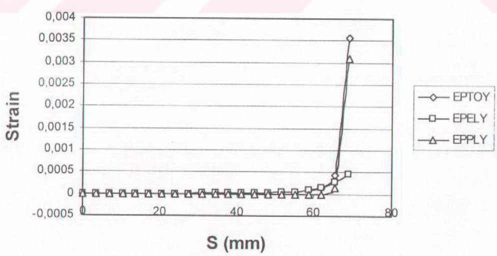

Fig 6.41 Strain distribution of G300 sheets steels for 0.5 mm extension at y direction ($\mu = 0.2$)

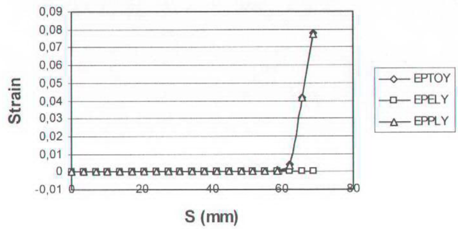

Fig 6.42 Strain distribution of G300 sheets steels for 1.5 mm extension at y direction ($\mu = 0.2$)

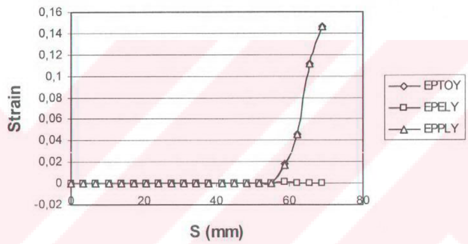

Fig 6.43 Strain distribution of G300 sheets steels for 3mm extension at y direction ($\mu = 0.2$)

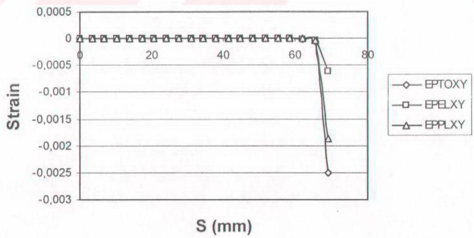

Fig 6.44 Strain distribution of G300 sheets steels for 0.5mm extension at xy direction ($\mu = 0.2$)

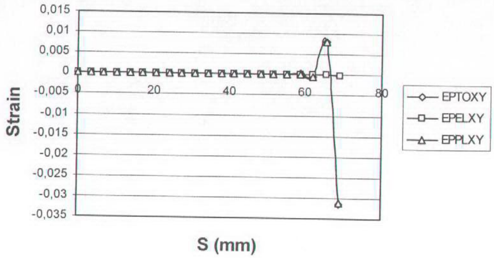

Fig 6.45 Strain distribution of G300 sheets steels for 1.5mm extension at xy direction ($\mu = 0.2$)

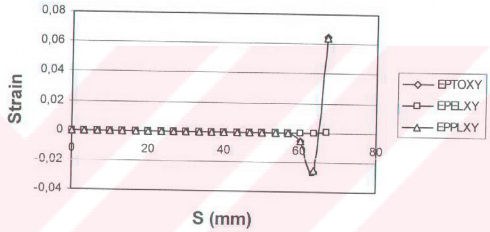

Fig 6.46 Strain distribution of G300 sheets steels for 3mm extension at xy direction ($\mu = 0.2$)

We search the effect of washer size at 3mm extension for G300 specimen below

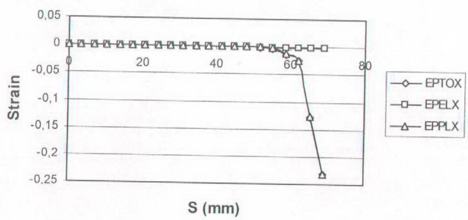

Fig 6.47 Strain distribution of G300 sheets steels for 3 mm extension at x direction ($\mu = 0.2$) for small washer

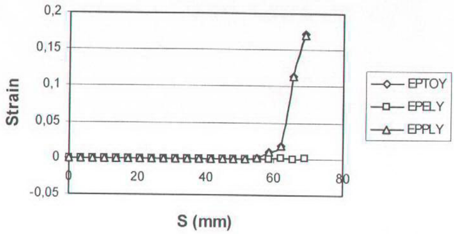

Fig 6.48 Strain distribution of G300 sheets steels for 3mm extension at y direction ($\mu = 0.2$) for small washer

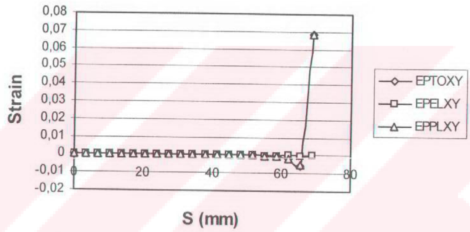

Fig 6.49 Strain distribution of G300 sheets steels for 3mm extension at xy direction ($\mu = 0.2$)

For G550 sheet steels the graphic demonstration is for two different friction coefficient with small washer size is shown below in figure 6.50 – 6.67

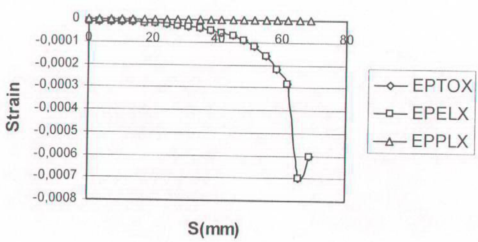

Fig 6.50 Strain distribution of G550 sheets steels for 0.5 mm extension at x direction (μ=0.2)

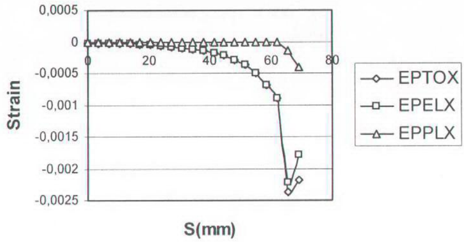

Fig 6.51 Strain distribution of G550 sheets steels for 1.5 mm extension at x direction (μ=0.2)

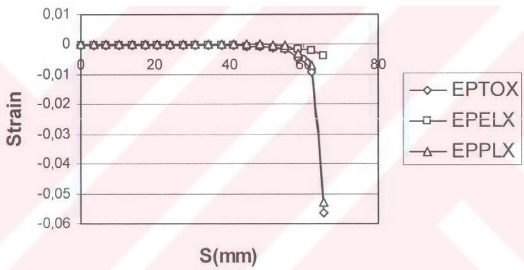

Fig 6.52 Strain distribution of G550 sheets steels for 3 mm extension at x direction (μ=0.2)

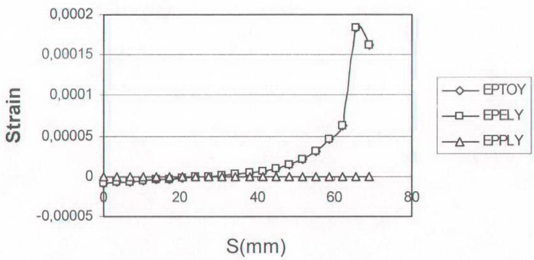

Fig 6.53 Strain distribution of G550 sheets steels for 0.5 mm extension at y direction (μ=0.2)

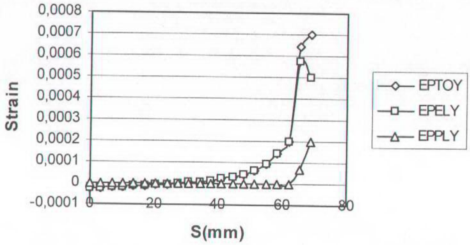

Fig 6.54 Strain distribution of G550 sheets steels for 1.5 mm extension at y direction (μ=0.2)

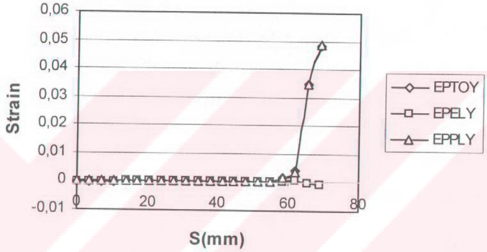

Fig 6.55 Strain distribution of G550 sheets steels for 3 mm extension at y direction (μ=0.2)

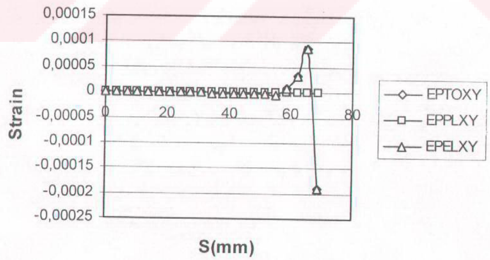

Fig 6.56 Strain distribution of G550 sheets steels for 0.5 mm extension at xy direction (μ=0.2)

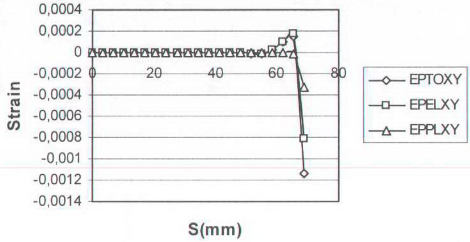

Fig 6.57 Strain distribution of G550 sheets steels for 1.5 mm extension at xy direction (μ=0.2)

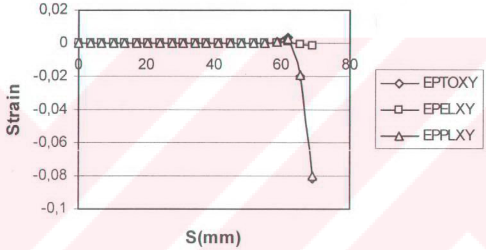

Fig 6.58 Strain distribution of G550 sheets steels for 3 mm extension at xy direction (μ=0.2)

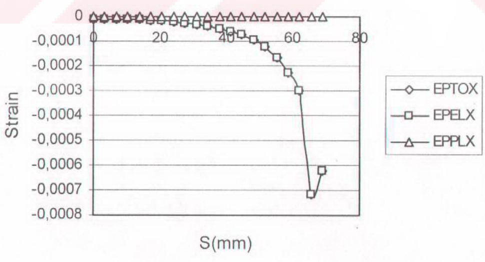

Fig 6.59 Strain distribution of G550 sheets steels for 0.5 mm extension at x direction ($\mu = 0.4$)

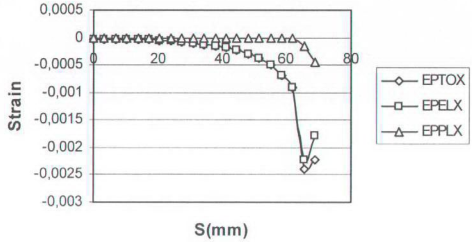

Fig 6.60 Strain distribution of G550 sheets steels for 1.5 mm extension at x direction ($\mu = 0.4$)

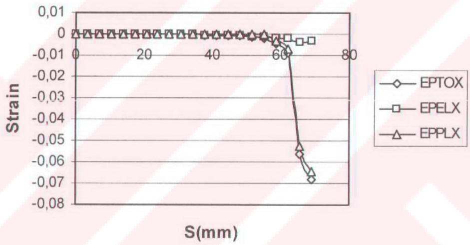

Fig 6.61 Strain distribution of G550 sheets steels for 3 mm extension at x direction ($\mu = 0.4$)

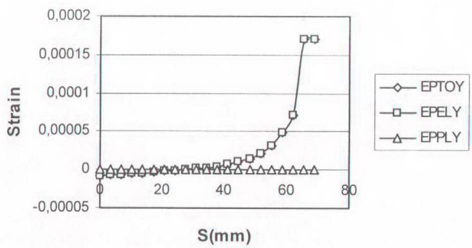

Fig 6.62 Strain distribution of G550 sheets steels for 0.5 mm extension at y direction ($\mu = 0.4$)

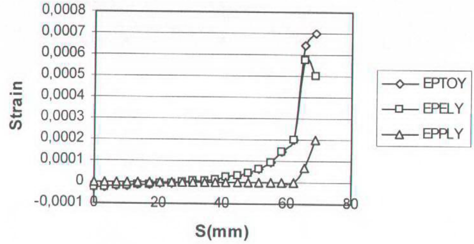

Fig 6.63 Strain distribution of G550 sheets steels for 1.5 mm extension at y direction ($\mu = 0.4$)

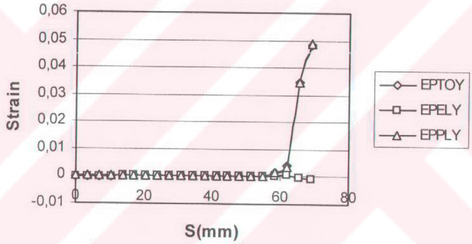

Fig 6.64 Strain distribution of G550 sheets steels for 3 mm extension at y direction ($\mu = 0.4$)

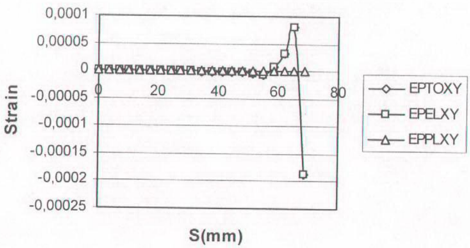

Fig 6.65 Strain distribution of G550 sheets steels for 0.5 mm extension at xy direction ($\mu = 0.4$)

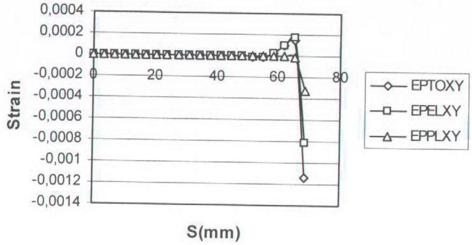

Fig 6.66 Strain distribution of G550 sheets steels for 1.5 mm extension at xy direction ($\mu = 0.4$)

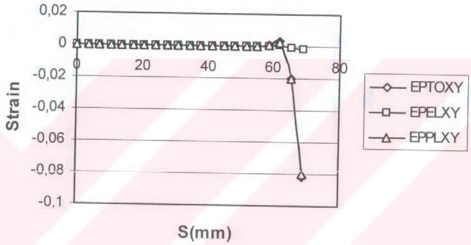

Fig 6.67 Strain distribution of G550 sheets steels for 3mm extension at xy direction ($\mu = 0.4$)

We want to compare small washer with larger washer for $\mu = 0.2$ the graphics are done below

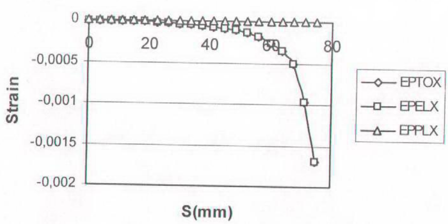

Fig 6.68 Strain distribution of G550 sheets steels for 0.5 mm extension at x direction (μ=0.2) with larger washer

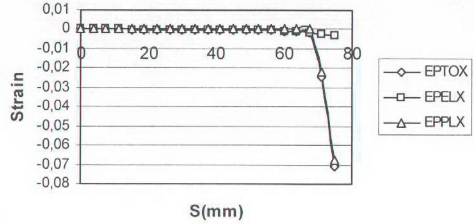

Fig 6.69 Strain distribution of G550 sheets steels for 1.5 mm extension at x direction (μ=0.2) with larger washer

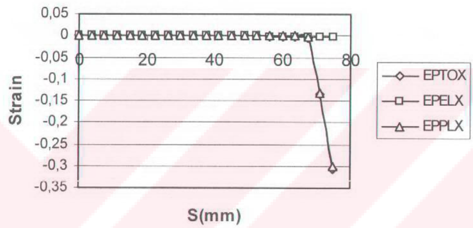

Fig 6.70 Strain distribution of G550 sheets steels for 3 mm extension at x direction (μ=0.2) with larger washer

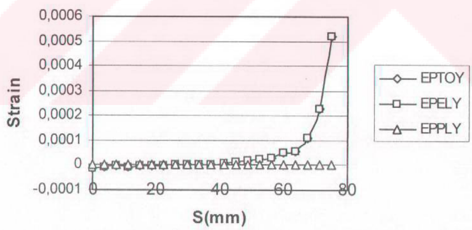

Fig 6.71 Strain distribution of G550 sheets steels for 0.5 mm extension at y direction (μ=0.2) with larger washer

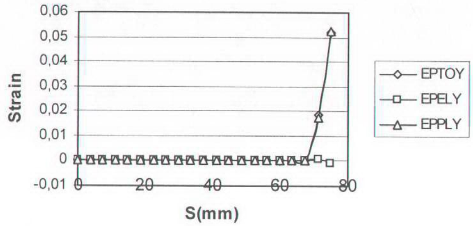

Fig 6.72 Strain distribution of G550 sheets steels for 1.5 mm extension at y direction (μ=0.2) with larger washer

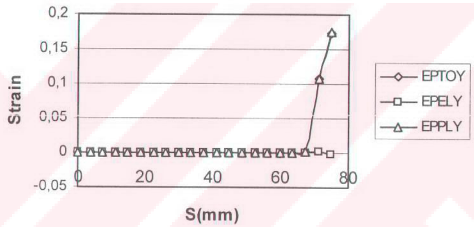

Fig 6.73 Strain distribution of G550 sheets steels for 3 mm extension at y direction (μ=0.2) with larger washer

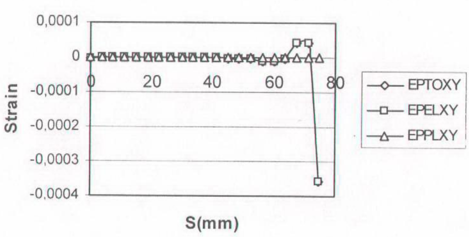

Fig 6.74 Strain distribution of G550 sheets steels for 0.5 mm extension at xy direction (μ=0.2) with larger washer

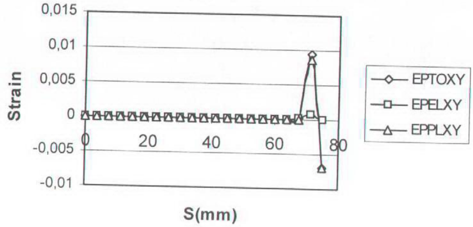

Fig 6.75 Strain distribution of G550 sheets steels for 1.5 mm extension at xy direction (μ=0.2) with larger washer

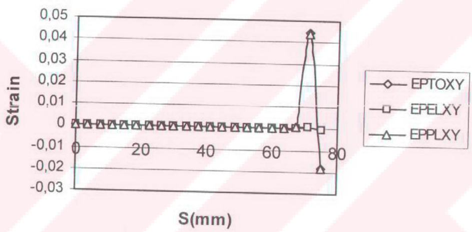

Fig 6.76 Strain distribution of G550 sheets steels for 3 mm extension at xy direction (μ=0.2) with larger washer

For G300 and G550 specimens we see that the strain increment in the x direction is regular and rising as negative curve around the hole we may say from this the failure will start from the left part of hole. And also x direction strain increment is more than y and xy direction. We observe that the increase in the y direction is positive we may say the specimen forced at positive y direction with the extension. The xy direction increasing is irregular and not important for G300 specimens. The strain increment for G300 specimen is more than G550 specimen in the 3mm extension. So we may say it depends the difference of strength values. On the other hand we concluded that when the friction coefficient increase the strain distribution is not changed at all with these datas too for G300 specimens. But we don't say same thing for G550 specimens with larger friction coefficient the strain of specimen at x direction increase but not change at y and xy direction. Also we observe the plastic behaviour of specimens plastic strain increament is not change except around the hole so we just see plastic behaviour around the hole.

We examine the behaviour of G300 and G550 specimen with the washer size again. And we observe with small washer the strain of x direction larger than large washer one for G300 test specimen. The strain of y and xy direction is not effected from the washer size. For G550 specimens it is not same with G300 specimen. The strain is increase at x.xy and y direction for G550 test specimen with larger washer.

When we use G300 double hole lap shear connections for $\mu = 0.2$ with small and larger washer we see them respectively below in figure 6.74-6.85

Fig 6.77 G300 sheet steel strain distibution for 0.5mm extension at x direction

Fig 6.78 G300 sheet steel strain distibution for 3 mm extension at x direction

Fig 6.79 G300 sheet steel stress distibution for 0.5 mm extension

Fig 6.80 G300 sheet steel stress distibution for 3 mm extension

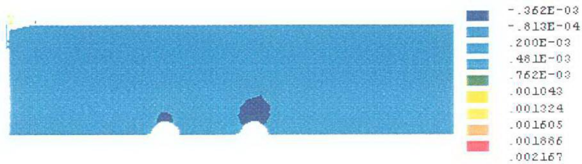

Fig 6.81 G300 sheet steel strain distibution for 0.5 mm extension at y direction

Fig 6.82 G300 sheet steel stress distibution for 3 mm extension at y direction

G300 specimens with larger washer in figures 6.80-6.85

Fig 6.83 G300 sheet steel strain distibution at x direction for 0.5 mm extension

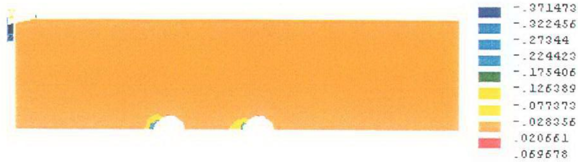

Fig 6.84 G300 sheet steel strain distibution at x direction for 3 mm extension

Fig 6.85 G300 sheet steel strain distibution at y direction for 0.5 mm extension

Fig 6.86 G300 sheet steel strain distribution at y direction for 3mm extension

Fig 6.87 G300 sheet steel stress distribution for 0.5 mm extension

Fig 6.88 G300 sheet steel stress distribution for 3 mm extension

We see same effect with single lap shear connections except the failure mode in this study clearly and we may say near the bearing failure net section failure may become with increasing extensions. And yielding area of the plates are larger with the extension according to single lap connections.

When we use G550 double hole lap shear connections for $\mu = 0.2$ with small and larger washer we see strain and stress distribution respectively below in figure 6.86-6.97

Fig 6.89 G550 sheet steel strain distibution for 0.5mm extension at x direction

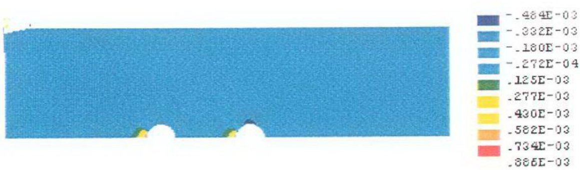

Fig 6.90 G550 sheet steel strain distibution for 3 mm extension at x direction

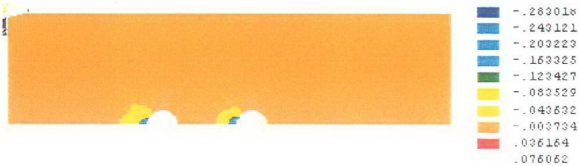

Fig 6.91 G550 sheet steel strain distibution for 0.5 mm extension at y direction

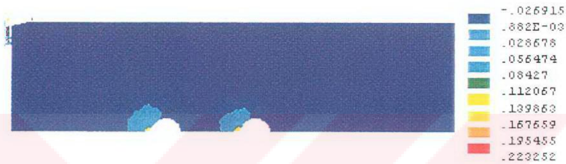

Fig 6.92 G550 sheet steel strain distibution for 3 mm extension at y direction

Fig 6.93 G550 sheet steel stress distribution at 0.5mm extension

Fig 6.94 G550 sheet steel stress distribution at 3 mm extension

With larger washer

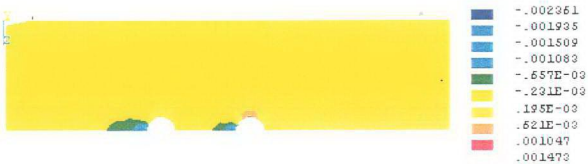

Fig 6.95 G550 sheet steel strain distribution at x direction for 0.5mm extension

Fig 6.96 G550 sheet steel strain distribution at x direction for 3mm extension

Fig 6.97 G550 sheet steel strain distribution at y direction for 0.5mm extension

Fig 6.98 G550 sheet steel strain distribution at y direction for 3 mm extension

Fig 6.99 G550 sheet steel stress distribution at 0.5mm extension

Fig 6.100 G550 sheet steel stress distribution at 3 mm extension

We find that with larger washer the strain in the x direction is increased. But in the y direction it is decreased. And von mises stress are decreased too. It is show that with larger washer load carrying capacity is increased.

6.5 Load Carrying Capacity of Specimens

In this section we aim to compare the strain results and findings with strain and stress distributions with load-extension curves of specimens as a result the load carrying capacity of specimens.

Load extension Curves of G300 and g550 sheet steels .

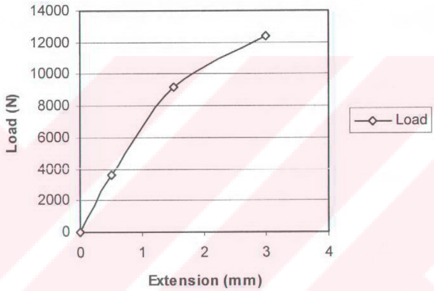

Fig.6.101 G300 Sheet steel Load extension curve for small washer and $\mu = 0.2$

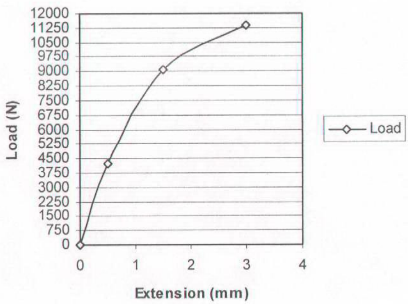

Fig.6.102 G300 Sheet steel Load extension curve for small washer and $\mu = 0.4$

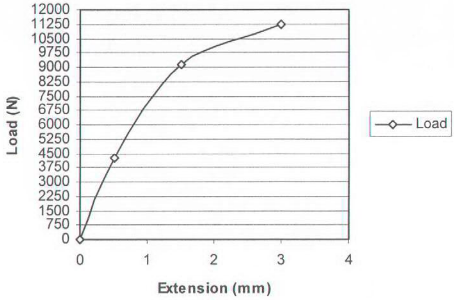

Fig.6.103 G300 Sheet steel Load extension curve fo large washer and $\mu = 0.4$

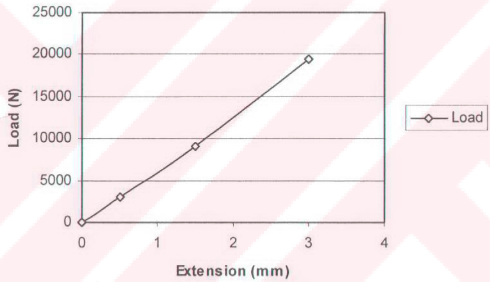

Fig.6.104 G550 Sheet steel Load extension curve for small washer and $\mu = 0.2$

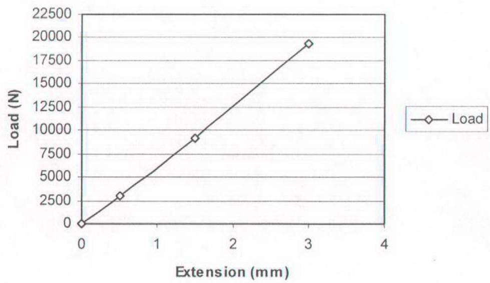

Fig.6.105 G550 Sheet steel Load extension curve for small washer and $\mu = 0.4$

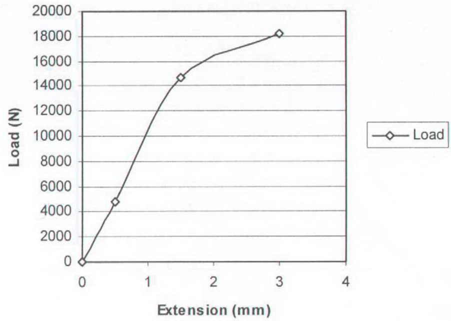

Fig.6.106 G550 Sheet steel Load extension curve for large washer and $\mu = 0.4$

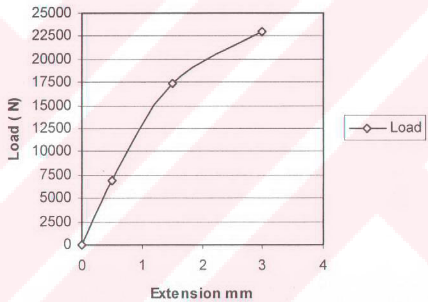

Fig.6.107 G300 Load extension curve of double bolt sheet steel for small washer and $\mu = 0.2$

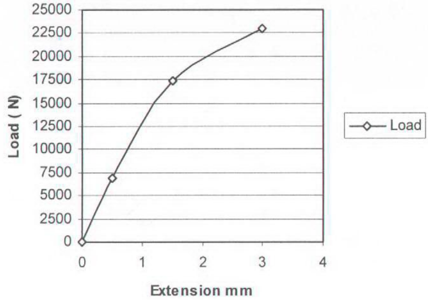

Fig.6.108 G300 Load extension curve of double bolt sheet steel for larger washer and $\mu = 0.2$

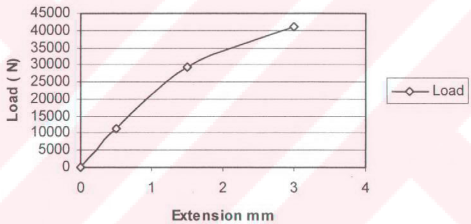

Fig.6.109 G550 Load extension curve of double bolt sheet steel for small washer and $\mu = 0.2$

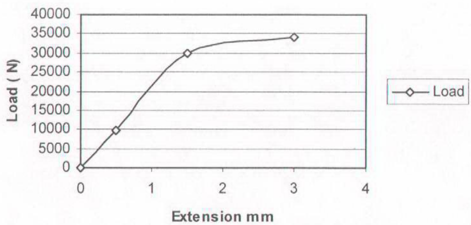

Fig.6.110 G550 Load extension curve of double bolt sheet steel for large washer and $\mu = 0.4$

It is seen that with the increasing of friction coefficient and washer size the G300 specimens` load carrying capacity is increase

For G550 specimens it is not change with larger friction coefficient but the larger washer effect its capacity a bit. So we may say the load carrying capacity of G300 specimens effect from friction more than G550 specimens for single bolt lap shear connections.

Fordouble bolt lap shear connections it is not same with single one. It is seen that G550 specimen is effect from friction resistance more for double bolt lap shear connection and load carrying capacity is increasing with large washer.

For understanding the failure modes of our specimens we investigate the behaviour of specimens at ultimate extensions we tested specimens with same extension

Fig 6.111 G300 specimen's failure mode for $\mu = 0.2$ **with large washer**

Fig 6.112 G300 specimen's failure mode for $\mu = 0.2$ **with large washer**

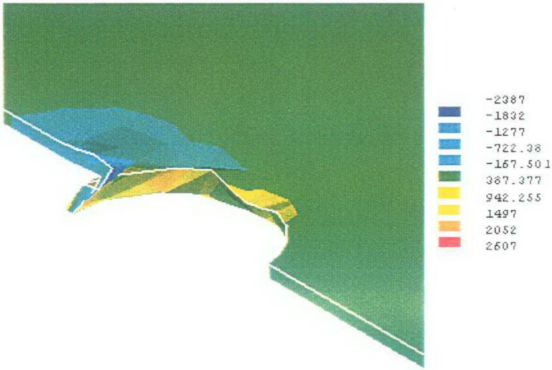

Fig 6.113 G300 specimen's failure mode for $\mu = 0.2$ with small washer

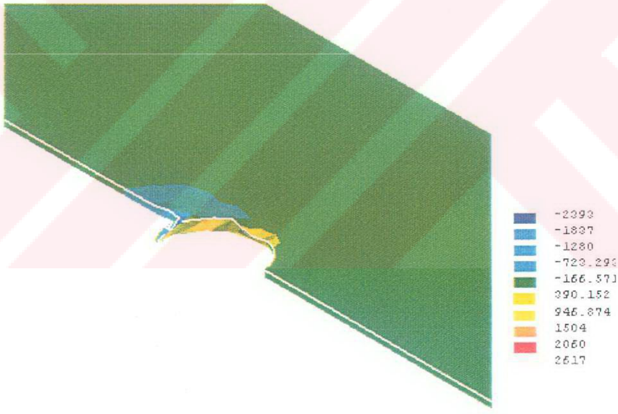

Fig 6.114 G300 specimen's failure mode for $\mu = 0.4$ with small washer

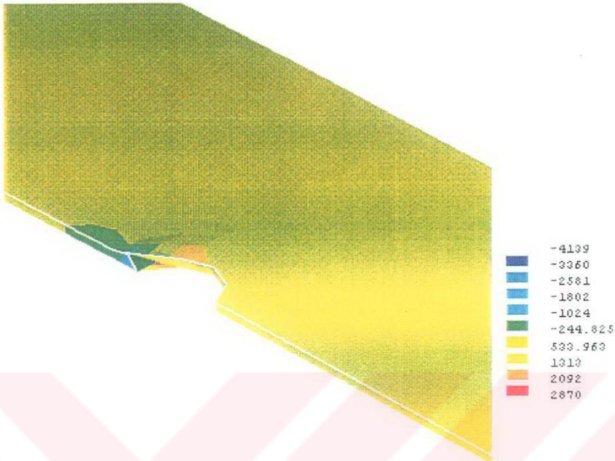

Fig 6.115 G550 specimen's failure mode for $\mu = 0.2$ with small washer

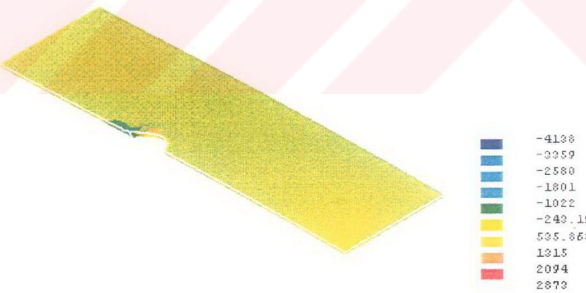

Fig 6.116 G550 specimen's failure mode for $\mu = 0.4$ with small washer

The previous researchs said that the friction resistance accounts for the %18 and %22 of the load carrying capacities for G550 and G300 test specimens ın our work when μ is increased to 0.4 the load carrying capacity of connections will increase about %15 according to our research. As a result the load carrying capacity is not sensitive to the value of friction coefficient. Bearing resistance of cold-formed steel remains reasonably constant we see this from both form semiemprical formulations

$\sigma_b = \frac{p}{t.d}$ P is maximum load at 3mm extension t = Strip thickness

d= bolt diameter

and also we see this from von mises stress distribution in figures 17,18,19,20
Bearing resistance is only change with the value of changable extension conditions. Bearing resistance is more important for the G550 test specimens beceuse where local streses and strains are ver high leading to premature failure . Friction actions between the washers and the cold-formed steels strips are more important for low strength cold-formed steel strips beceuse when we use larger washer both G300 and G550 test specimens we see that larger washer doesn't effect the load carrying capacity of G550 specimen but it is not same for G300 specimen its load carrying capacity is change approximately %10 see in figures 89,90,91,93
The frictional forces are dependent on the clamping forces in the bolts , the frictional coefficient between contact interfaces , and also the size of the washers.With the increasing clamping force, friction coefficient and the size of the washer the friction resistance and load carrying capacity is increase.this is about %20-%30 according to the previous researchs.

Contact stiffness is a important parameter for the structural performance of sheet steels it is assigned after comparing the measured and predicted load extension curves of sheet steels. It is found that according to lap shear tests, and findings of finite element analysis. Finite element analysis is conservative at various extension conditions. According to our research we see that the von mises stress distribution around bolt hole for G300 specimen confined to small area but it is not for where G550 the stress distribution and also strain around bolt hole

Also we found that the stress strain distribution is a impotant parameter for accurate tests espically for Finite element analysis. According the references for the accurate understanding the behaviour of load carrying capacity of specimens especially for G550 test specimens strength degradation and bilinear stress-strain curve is important parameter.

6.6 Possible Failure Modes Test Specimens

Winter categorized the failure of bolted connections into four seperate modes as end-pullout, bearing, net section fracture and bolt shear. Today it was possible to observe three remaining failure modes end-pullout or pull-through ,bearing and net section failures. End pullout failure occured for single bolt connections where the distance from the center of the bolt hole to the end of the specimen was less than three times the diameters of the bolt. With the increasing extension we see that some piling of the sheet steel occured infront of the sheet steel with two longitudinal tears extending from the piled material to the end of the specimen. During high wind events such as storms and cyclones, these localized failures then lead to severe damage to buildings and their contents. In recent times, the use of thin steel battens, purlins and girts has increased considerably, which has made the pull--out failures more critical in the design of steel cladding systems. In our work the failure mode if we increase the extension large enough we see the pull-out failure problem too see in figure 6.108 -113

Connections that failed by bearing exhibited an initial pull-out tear in the direction of load with piling of the sheet steel in front of the bolt , similar to that observed for end-pullout failure and in some instances there were additional diagonal tears at the edge of the piled material nearest the end of the test specimen

Anumber of bolted specimens in our work (two bolted connection) exhibited large amounts of bearing distortion prior to failure through the net section . The net section failure was identified in these cases by slight necking of the specimen followed by the fracture of the material at the at the center to the end of the test specimen.

The Australian [6], American [7] and the European provisions [8] include design formulae for the pull--out capacity, Fou , of bolted connections in tension as shown by Equations (1a) and (1b).

Australian and American Fou =0.85 t d fu (1a)
European Fou =0.65 t d fy (1b)
where t = thickness of member,
d = nominal diameter of hole
fu = ultimate tensile strength of steel and
fy = yield stress of steel.
Now we will search our specimen's pull-out capacity at ultimate extension 3mm according to Australlian and European formulaes for G300 specimen
According to Australlian and American Fou = 0.85 1.5 12 450 = 6885 N
According to European Fou =0.65 1.5 12 300 =3510 N
But in our work we see that Altough we pass the limits of these pull-out capacity we don't observe the pull out
Australlian and European formulaes for G550 specimen
According to Australlian and American Fou = 0.85 1.6 12 638.4 = 10418.688 N
According to European Fou = 0.65 1.6 12 638.4 =7967.232 N
It is found that this design provisions can not be used to accurately predict the failure mode of bolted connections . It is necessary to incorporate a variable bearing resistance equation that is dependent on the thickness of the connected material similar to that found in the CSA-S136 design standart

When we investigate the bearing resistance according to BS5950 and Eurocode design standarts for G300 and G550 test specimens we found we find the design rules unconservative again.

The ultimate load carrying capacity of a connection will be governed by one of many failure modes including; bearing, end pull-out, net section fracture, bolt shear, block shear rupture, etc.. It was concluded that there are a number of problems with the existing load capacity formulations contained in the current cold formed steel design standards, based on observations made during the testing of thin bolted connection specimens. A modification to the bearing coefficient provisions for thin G550 and G300 sheet steels is necessary to account for the reduced bearing resistance of the connected materials. This reduction in bearing resistance is related more to the steel thickness than to the steel grade (G550 vs. G300) and a proposed gradated bearing coefficient method is presented in research report. A revision of the net section fracture design method is also required. Furthermore, a detailed analysis of the procedure used to identify the cause of failure in bolted connections is needed to ensure that accurate failure mode assessments are made, and ultimately to ensure that accurate design equations are formulated. Misidentification of failure modes and the misuse of data can lead to serious errors in the accuracy and applicability of design equations. It is necessary to incorporate a variable bearing resistance equation which is dependent on the thickness of the connected material, similar to that found in the Canadian CSA-S136 Design Standard. Calculation of the ultimate tensile strength of a bolted connection using the net cross-sectional area and the ultimate material strength, without a stress reduction factor, is accurate and reliable.

As a result existing design rules are not conservative the extensive investigation which is made by Chung is take as a reference in this section below

After calibrating against test data, an extensive parametric study using the finite element model was performed to a range of steel thicknesses, bolt diameters, and also steel grades. The predicted bearing resistances of the bolted connections are presented in Table 2.

In order to compare with the design rules given in both BS5950: Part 5 and Eurocode 3: Part 1.3, the design bearing resistances were evaluated as follows:

- *BS5950: Part 5:1998: clause 8.2.5.2*

$$P_{bs} = (1.65 + 0.45t)\, d\, t\, p_y$$

- *ENV 1993-1-3: 1996 - EC3: Part 1.1 Table 4*

$$F_{b,Rd} = 2.5\, d\, t\, f_u / \gamma_{M2}$$

The design bearing resistances were evaluated using the tensile strength rather than the yield strength of the steel materials in BS5950 while the partial safety factor γ_{M2} is set to unity. The design bearing resistances are also presented in Table 2 for easy comparison with those predicted by the finite element model

Table 6. 2 Bearing resistances of bolted CFS strips at 3mm extension

	Material specification				P_{bf}			***Model factor***	
	t (mm)	*d* (mm)	p_y (N/mm^2)	U_s (N/mm^2)	*BS5950* (kN)	*EC3* (kN)	*FEA* (kN)	*FEA* / *BS5950*	*FEA* / *EC3*
B111	*1.2*	*12*	*280*	*390*	*12.30*	*14.04*	*17.30*	*1.41*	*1.23*
B112	*1.2*	*12*	*350*	*450*	*14.19*	*16.20*	*19.48*	*1.37*	*1.20*
B121	*1.2*	*16*	*280*	*390*	*16.40*	*18.72*	*19.82*	*1.21*	*1.06*
B122	*1.2*	*16*	*350*	*450*	*18.92*	*21.60*	*22.12*	*1.17*	*1.02*
B211	*1.6*	*12*	*280*	*390*	*17.75*	*18.72*	*22.78*	*1.28*	*1.22*
B212	*1.6*	*12*	*350*	*450*	*20.48*	*21.60*	*25.62*	*1.25*	*1.19*
B213	*1.6*	*12*	*450*	*550*	*25.03*	*26.40*	*29.06*	*1.16*	*1.10*

B214	*1.6*	*12*	*550*	*600*	*27.30*	*28.80*	*29.52*	*1.08*	*1.03*
B221	*1.6*	*16*	*280*	*390*	*23.66*	*24.96*	*25.84*	*1.09*	*1.04*
B222	*1.6*	*16*	*350*	*450*	*27.30*	*28.80*	*29.18*	*1.07*	*1.01*
B223	*1.6*	*16*	*450*	*550*	*33.37*	*35.20*	*33.28*	*1.00*	*0.95*
B224	*1.6*	*16*	*550*	*600*	*36.40*	*38.40*	*34.52*	*0.95*	*0.90*
B312	*2*	*12*	*350*	*450*	*27.54*	*27.00*	*31.8*	*1.15*	*1.18*
B313	*2*	*12*	*450*	*550*	*33.66*	*33.00*	*35.94*	*1.07*	*1.09*
B314	*2*	*12*	*550*	*600*	*36.72*	*36.00*	*36.62*	*1.00*	*1.02*
B322	*2*	*16*	*350*	*450*	*36.72*	*36.00*	*35.96*	*0.98*	*1.00*
B323	*2*	*16*	*450*	*550*	*44.88*	*44.00*	*40.98*	*0.91*	*0.93*
B324	*2*	*16*	*550*	*600*	*48.96*	*48.00*	*42.54*	*0.87*	*0.89*
B412	*2.5*	*12*	*350*	*450*	*37.46*	*33.75*	*39.34*	*1.05*	*1.17*
B413	*2.5*	*12*	*450*	*550*	*45.79*	*41.25*	*44.06*	*0.96*	*1.07*
B414	*2.5*	*12*	*550*	*600*	*49.95*	*45.00*	*45.7*	*0.91*	*1.02*
B422	*2.5*	*16*	*350*	*450*	*49.95*	*45.00*	*44.08*	*0.88*	*0.98*
B423	*2.5*	*16*	*450*	*550*	*61.05*	*55.00*	*49.98*	*0.82*	*0.91*
B424	*2.5*	*16*	*550*	*600*	*66.60*	*60.00*	*52.08*	*0.78*	*0.87*

(Chung & Ip,1999)

CHAPTER 7
CONCLUSIONS

A finite element modelling on bolted connections between cold-formed steel strips was carried out with non-linear material, geometrical and contact analysis. Based on reference results of coupon tests twelve reference lap shear tests with two steel grades, one bolt diameter and two washer size were carried out to calibrate the finite element models. It is found that for extensions up to 3mm the load-extension curves of the bolted connections compare well with test data in terms of both the initial and the final slopes and the maximum load carrying capacities . Factors that influenced successful convergence of the analytical models were identified during the initial phase. These factors included the mesh density and distribution, contact constraints, material behavior, fastener installation strategy, and convergence tolerances. Fastener installation is a key factor that influences joint load-elongation behavior through induced cold work and clamping effects. A means for varying the convergence tolerance during an analysis was also developed and implemented.

With the help of the finite element modelling, the paterns of yielding and the strain distribution around the connections are established in detail. It is important to incorporate reduced strength at large strains for accurate prediction of the load carrying capacities of bolted connections. Furthermore it is also found that the frictional resistance contributes typically %20 of load carrying capacities of bolted connections. This depends on the clamping forces in bolts, the frictional coefficient between contact interfaces and also the sizes of the washers.

Consequently the finite element model is an effective tool to investigate the structural performance of bolted connections between cold-formed steel strips under static shear loading. The failure mode under the present investigation is the bearing failure of cold-formed steel strips around the bolt holes while the other modes failure may be investigated. A recommended finite element modelling procedure is also presented. It is shown that for cold-formed steel strips with high strength and low ductility ,the resistance of the bolted connections is typically %15 lower than those steels with bi-linear stress- strain curves,i.e. without strength degradation.

It is shown that for steels with design strengths equal to *280* N/mm^2 and *350* N/mm^2, the existing design rules give conservative bearing resistance of bolted connections with thin cold-formed steel strips, $t < 2.0$ mm, and small bolt diameters, i.e. $d < 16$ mm, where t and d are the thickness of cold-formed steel strips and the bolt diameter respectively. However, for bolted connections with thick steel strips and large bolt diameters, the existing design rules give unconservative bearing resistances, especially, for high strength low ductility steels, i.e. steels with design strengths equal to *450* N/mm^2 and *550* N/mm^2. Based on the results of the parametric study, strength over-estimation up to *30%* is found.

It is recommended that the existing design rules for the bearing resistance should not be used for cold-formed steel strips of high strength and low ductility. Design rules for cold-formed steel with high strength and low ductility should be developed to ensure safe application of such materials.

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