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# A Proposed Analysis and Design of a Two-Storey Cold-Formed Steel (CFS) Residential Building using Direct Strength Method

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## Abstract:

This study focused on the analysis and design of a two-storey cold-formed steel (CFS) residential building using the Direct Strength Method. The primary objective was to address the utilization of a less common building material for construction. The study aimed to determine whether cold-formed steel could serve as a viable, sustainable option for building residential housing, given its potential advantages. Utilizing STAAD Pro Connect, a comprehensive design software, the study developed a safe and sustainable building model capable of withstanding various loadings. The software facilitated the structural analysis and design, ensuring the building met safety and durability requirements under Australian Standards. The results of the analysis demonstrated the feasibility of constructing a residential building using cold-formed steel, highlighting its robustness and longevity.

One of the key findings of the study was the significant benefits of cold-formed steel structures. These benefits included enhanced durability, making CFS an ideal material for residential buildings subject to diverse environmental factors. The structural design proposed in this study underscored the reliability and efficiency of CFS buildings. The study's significance extends beyond its immediate application, offering future researchers valuable insights into sustainable and efficient building construction methods. By illustrating that cold-formed steel can be effectively used as a construction material, the research opens the door to innovative ideas in the realm of fast and smart construction practices. This could revolutionize the approach to building accommodations and other residential structures, emphasizing sustainability, resilience, and cost-effectiveness.

*Keywords* — **Cold formed steel (CFS), STAAD Pro Connect, Australian Standards.**

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## I. INTRODUCTION

This document is a template. An electronic copy can be downloaded from the conference website. For questions on paper guidelines, please contact the conference publications committee as indicated on the conference website. Information about final paper submission is available from the conference website. As the world moves toward modernization, structures and buildings also develop. Since ancient times, when timber was the primary building material used for construction, it has evolved into reinforced concrete and steel structures like cold-formed steel (CFS). The Philippines' current partial wood ban makes it more difficult to obtain quality lumber for building homes. Cold-formed steel (CFS) is an increasingly popular alternative to reinforced concrete, which is still the material of choice in buildings. Its remarkable strength-to-weight ratio is contributing to its growing popularity.

Cold-formed steel is a lightweight construction material created by shaping thin sheets into various forms that could satisfy systematic and practical needs and support loads greater than those of thin sheets alone. CFS members are also known as "light gauge steel members". Utilizing lightweight properties and the ease of using the product can save Labor costs and produce an invaluable economy. [1]

Cold-formed steel (CFS) is often used in the construction industry worldwide. The adoption of CFS has more advantages compared to hot-rolled steel sections. There are works that examine the CFS section that have been applicable for other features like minimum local buckling, higher flexural strength, adequate seismic response, and bracing performance. Also, CFS is a hollow section that has a higher loading rate and uses infill material that brings better results in performance. The various characteristics are listed as applicable in many construction components. Lastly, it was looked at how well the CFS performed in favourable situations for finite analysis.[2]

Cold-formed steel sheets are utilized in cars, appliances, furniture, and many other common products because they have numerous exceptional qualities, such as ease of formability and a smooth, clean surface. Several benefits come with cold-formed steel, such as cost-effectiveness, favourable mechanical performance, and advantages in construction. Since cold-formed steel features are high strength, environmentally friendly, and sustainable, using them as building materials has additional benefits. CFS is a type of steel that is formed without the use of heat. It is made up of thin sheets because their thickness ranges from 0.373 mm up to 3.175 mm; therefore, it is lighter than concrete.

Despite all the advantages of CFS, it also has disadvantages: it has low fire and load resistance; there is remaining stress in the cross-section, which influences the buckling resistance of the cold-formed steel; it is prone to reverse when there is flexural compression; and lastly, it has less ability to withstand localized concentrated loads. This gives a strong and reliable connection that can be used for CFS applications like flame cutting, welding, and bolt cold bending. Additionally, it needs to be strong enough to support different kinds of loads in order to provide this CFS resistance.

Utilizing the advantages of cold-formed steel framing technology, FRAMEGO 101 CORP (FRAMEGO) is a newly founded, fully Filipino-owned building company that aims to provide an end-to-end construction solution that is economical and efficient. FRAMEGO 101 CORP is committed to giving each and every one of its clients a first-rate service experience, from design to production and construction. Their organization will soon take the lead in assisting in the reduction of the estimated 6.7 million housing units in the Philippines' backlog of housing.

In addition to producing and fabricating steel, FRAMEGO 101 CORP was founded to carry out related activities such as site preparation, building, and other structure construction, as well as enlarging, repairing, and servicing structures.

For multi-profile construction projects involving commercial, residential, and industrial structures, the ST950H is the perfect option. The large gauge capacity of 97 mils, or 2.5 mm, makes it perfect for producing long-span floor joists, wide-spanning roof trusses, and wall frames. An easy-to-use gauging system makes it possible to quickly switch between 18 and 12 gauge, or 43 and 97 mils (1.15 and 2.5 mm), in terms of measurement.

In other countries, specifically American and European countries, CFS is used as a mainframe for their structures, as stated in their structural codes, like the Australian Standard/New Zealand Standard (AS/NZS 4600) and Eurocode. Based on the AS/NZS 4600, there are different methods used in designing a CFS, and these are the Effective-Width Method (EWM), the traditional approach; the Finite Strip Method, which is commonly used in universities and official applications; and the Direct

Strength Method (DSM), which determines the member capacity.[3] DSM predicts the strength of a member based on the elastic buckling loads. In the present time, it is the design method used in AS/NZS 4600 that is developed by different researchers. These guidelines and specifications for CFS are not present in the National Structural Code of the Philippines (NSCP).

Environmental loadings in the Philippines may differ from the loadings that Western countries experience; thus, the specification of AS/NZS 4600 is unable to be promptly implemented in the NSCP. The researchers will design a two-storey structure that is acceptable on NSCP. They will adopt the Direct Strength Method of AS/NZS 4600, and they will analyze the specification to make the design method applicable to NSCP. It was discovered that the NSCP provisions' strength estimations for distortional buckling failure and torsional-flexural buckling failure did not match the outcomes of the compression testing.

## II. REVIEW OF RELATED LITERATURE

### *A. HISTORY AND DEVELOPMENT OF MATERIALS USED IN CONSTRUCTION*

Engineering had developed over the years as well as the materials that were used in construction to build structures. After the Industrial Revolution, synthetic materials such as concrete, bricks, and metals were introduced to construction. Concrete's origin as a building material goes back more than 2,000 years. Concrete is a hard, rocky-like man-made construction material comprised of aggregates, sand, water, and cement. In this sense, concrete can be viewed as one of the earliest composite materials utilized by humans. Concrete is naturally weak in tension but has great compressive strength.[4] The effect of the prestressing force on the prestressed composite beams is not as severe as that on the prestressed monolithic beams because the diagonal tensile fractures in the non-PSC are somewhat restrained by the prestressing force [5].

One of the most significant structural types in modern construction is the pre-cast concrete structure. Pre-cast concrete structures are superior over cast-in-place ones, such as less wet work on the construction site, greater component quality control, quicker construction, and favourable economic and environmental outcomes. The seismic performance of the entire structure is strongly impacted by beam-column joints [6]. The next material that is widely used in construction is steel. Iron and carbon are the two main components of steel. Steel is the most adaptable and globally applicable structural building material that is used for engineering construction. Steel significantly has more breaking resistance and is better in terms of tensile or compressive strength.[7].

### *B. COLD-FORMED STEEL AS A STRUCTURAL MAINFRAME*

Cold-formed steel is a construction material that is utilized in building processes to strengthen and speed up construction. It can also be referred to as cold-rolled steel or light-gauge steel. It does not deteriorate as quickly as timber does. In contrast to steel, it is lighter than concrete in weight.[8] The cold-formed steel structure is a modern development in construction materials that is currently being used. These materials have all been used to create purlins, and frames. The scarcity of wood and the difficulty in finding other, more affordable building materials led to the use of cold-formed steel as the structural element. This cold-formed steel frame has been employed in low- to medium-rise buildings. To ensure that the structure is systematically safe, it is necessary to assess its mechanical properties as well as its other structural qualities.[9]

One of the materials that is most frequently used in construction across a range of structures is cold-formed steel. Cold-formed steel sections are getting famously utilized because of their better structural performance and lightweight properties. Because of this, they are increasingly becoming the primary structural elements.[10] Due to cold-making, the steels that have been cold-formed show a better effect of fireproofing. The remaining strength evaluation of conventional and cold-formed steels exposed to fire may be carried out with the help of two sets of calibrated predictive models that are highly universal.[11]

In Malaysia, CFS is frequently used in place of hot-rolled steel and wood in roof truss systems for homes and small commercial structures. CFS has the highest strength-to-weight ratio and several other benefits, such as being lightweight, recyclable, and strong. As a building material, it is likewise growing in popularity far too quickly.[12] Compared to other building materials, cold-formed steel is a more sustainable choice due to its precision manufacturing method, which minimizes waste. Cold-formed steel's lightweight and handleability make it easier to work with than other materials, which speeds up construction and lowers costs.

CFS is possible to apply as single sections or built-up sections. Several studies have studied the effectiveness of the sections of CFS.[13] Cold-formed steel (CFS) is a term used to describe very thin steel that has not been reheated after being rolled to less than 1/8th of an inch and bent into a particular shape. Although it is superior due to its high strength ratio based on weight, thin sheets of steel are frequently bent into forms in cold-formed steel framing. C, U, Z, or hat channels are the most typical forms. As shown in the figures.

Fig. 1 C-Shaped

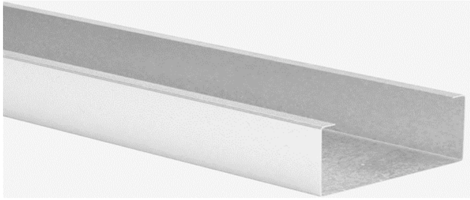


Fig. 2 U-Shaped

Fig 3. Z-Shaped Purlins

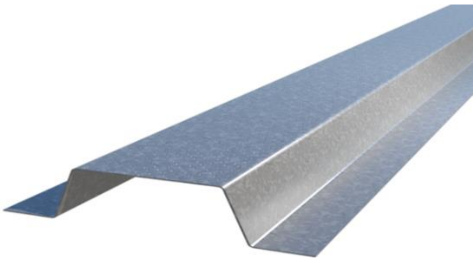


Fig 4. Hat channel

Fig 5. L-Shaped

Fig 6. rectangular hollow section

The first known use of CFS (Cold-Formed Steel) as a construction material was the Virginia Baptist Hospital. built in the year 1925 in Lynchburg, Virginia. Cold-formed steel framing became popular in the commercial industry in the 1950s and 1960s. It was utilized in recently developed systems including inside shaft walls, curtain walls, and brick veneer-adorned exterior framing.

Fig 7. The Virginia Baptist Hospital

Even though many believe cold-formed steel frames to be "new" It has been utilized in North America for almost a century as a construction product. In the 1850s, both England and the United States started using cold-formed steel members for building materials. Only a few fundamental structures were used, and the use was primarily experimental.

Fig 8. Stran-Steel house

Cold-formed steel panels with an enamel coating are used as the exterior cladding on service stations. Steel usage was still subject to post-war limitations in 1945.[14].

Fig 9. "House of Tomorrow" built by Howard Fisher's General House

Fig 10. Cold formed steel structures by Mexi Steel using EN 1993

Fig .11. Cold formed steel warehouse in USA by McElroy METAL

### *C. DESIGN METHODS USED TO ANALYZE COLD-FORMED STEEL AS MAINFRAME*

CFS or Light-Gauge steel has a high capacity for commercial buildings of 5 storey structures and below since they employ software called ETABS to cut down on construction time compared to conventional steel [15]. Due to its thinness, CFS is prone to fire, buckling, and other failures such as web crippling and torsional failure. According to numerous studies, they use the advantages of CFS to improve its components while minimizing its drawbacks [16]. Using different guidelines and specifications, there are different methods of analyzing and designing CFS.

### EFFECTIVE WIDTH METHOD

This method enables the traditional design approach to encompass more extensive limit states and can be included in the traditional effective width design specifications described in AS/NZS 4600 S100. [17] According to this method, a CFS member is treated as an equivalent plate with an established effective width based on the geometry and boundary conditions of the member. The member's capacity is then determined using standard design equations utilizing the effective width. This approach provides comparable levels of precision and dependability to the Direct Strength Method.

After comparing and analyzing the DSM and EWM, it is evident that when width-to-thickness (b/t) ratios for unstiffened deck sections exceed 40–70, DSM begins to predict lower strengths than EWM. By strengthening compression elements with stiffeners, DSM predicts higher strengths when b/t is lower than EWM [18].

### DIRECT STRENGTH METHOD

This method uses the concept of individual elements, such as studs, tracks, or connectors, rather than relying on traditional design equations. It offers a more precise forecast of the behavior of the CFS members by taking into account the geometric and material characteristics of those members. The method covers the design of structures that are susceptible to compression, bending, and shear with and without holes.[19] The Australian cold-formed steel structures standard (AS/NZS 4600) recently adopted the DSM design principles for cold-formed steel members [20].

### ELASTIC BUCKLING ANALYSIS

As it's stated, CFS are prone to buckling and this method focuses on analyzing the stability of the members under different loading conditions. Making sure the members are built to resist buckling entails calculating the critical buckling loads and comparing them with the applied loads, which included the finite strip method based on changing the entire cross-section thickness and calculation formula in this study [21].

Cross-section firmness in cold-formed steel members can be investigated with great effectiveness when the traditional finite strip method and the constrained finite strip approach are used together [22].

#### *D. EXPERIMENTAL STUDY OF THE BEHAVIOR OR PERFORMANCE OF COLD-FORMED STEEL SUBJECT TO LOADINGS*

As we all know cold-formed steel is lighter weight than other steel structures. For low- and mid-rise buildings, cold-formed steels offer a reliable, cost-efficient, and long-term solution. The conclusions from the experimental tests are between 15% and 25% less than the results from the simulated tests. Based on the findings of this analysis, it has been determined to change the CFS sections' characteristics in order to increase their bearing capacity going forward [23].

In the building sector, cold-formed steel members are frequently utilized, particularly in residential and commercial structures. An experimental and numerical analysis of the flexural behavior of cold-formed steel built-up sections is presented in this research. The experimental finding indicates that the built-up cold-formed steel beam with lip had a higher flexural capacity than the beam without lip. With the use of ABAQUS software, finite element analysis is also used to confirm the experimental results. The resulting analytical findings and the experimental results are in strong agreement [24].

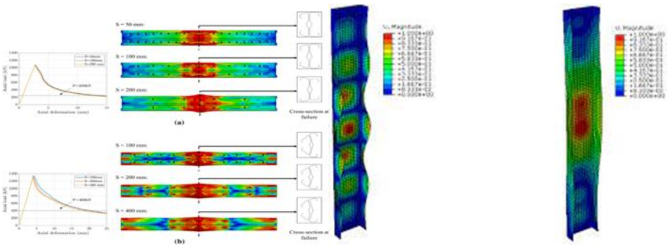


Fig 12. ABAQUS software, finite element analysis

Cold-formed structural components are becoming more and more common and important in steel construction. Its benefits in economy in transportation and handling, account for its rising popularity in building construction. The term "cold-formed steel" advert to steel products that have been shaped using cold working techniques at or near room temperature. The numerical analysis program ANSYS is used to conduct the research. The bucking behavior of CFS columns with various configurations is investigated using the ANSYS program [25].

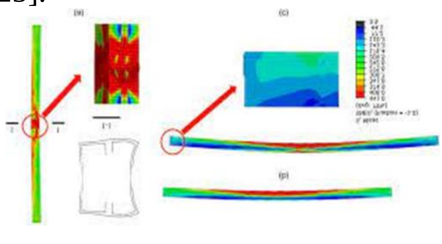


Fig 13. ANSYS program

Self-drilling screws were recently used to construct an inventive CFS T-Stub connection that exhibits great structural performance in the building sector. When the design strengths from the experiment and FEA findings were compared with those from the Australian Standard/New Zealand Standard (AS/NZS 4600 2016), Australia/New Zealand standards (AS/NZS 2018), it was discovered that the CFS T-Stub connections' final design strength had been overestimated by 16% and 14%, respectively [26].

Since a few decades ago, cold-formed steel members have become more and more common in the building business. Cold-formed steel sections were initially utilized as secondary structural components, but they are now also used as fundamental structural components. By altering the profile geometry of each conventional section, three corresponding innovative sections are created. Using the existing code processes, such as the Indian code, Australian code, Euro code, and American code (IS 801:1975, AS/NZS 4600:2005, EN 1993 1-3, AS/NZS 4600 S 100:2007), conventional sections are analytically examined. The behavior of load deflection is investigated numerically and experimentally. The traditional and novel cold-formed steel pieces are compared. The study's unacceptable conclusion is that conventional sections are stiffer and fail suddenly after achieving ultimate stress [27].

### *E. SYNTHESIS OF REVIEW OF RELATED LITERATURE*

With the advent of synthetic building elements like metals, concrete, and bricks, engineering has advanced over time. Modern construction materials were introduced such as Prestressed Concrete, Pre-casts, and Cold-formed steel (CFS) which has great seismic properties. However, the lack of CFS design standards in the NSCP 2015 is still a dilemma.

CFS sections are being used more and more in industrial building components around the world due to their high structural performance and lightweight design. It has been discovered that they exhibit sufficient seismic response, minimum local buckling, higher flexural strength, higher fire resistance, and bracing behavior. CFS is frequently used in roof truss systems for residences and small commercial buildings in Malaysia. Because of its thin surface, CFS, however, might not buckle on short, narrow sections. It is frequently utilized in cold-formed steel framing in hat, Z, U, C channel shapes, and rectangular hollow sections and among many others. These sections can be used singly or in combination, offering space at regular intervals or positioned closely. CFS sections are frequently utilized to provide a wedge or spacing element to the sheathing in studs, track sections, and hat channels. As a basic header, L-shaped CFS transfers the load to jamb studs. Shear walls and wall bracing frequently contain straps, which are thin steel sheets used for tension loads.

There are three design methods in analyzing and designing a CFS and these are Effective Width Method (EWM), Direct Strength Method (DSM), and Elastic Buckling Analysis. DSM is based on the idea of using the direct capacity of individual elements rather than relying on traditional design equations. It is the easiest or simplest method in designing a CFS and the researchers will utilize it in the continuation of their study. While EWM encompasses more extensive limit states and can be included in the traditional effective width design specifications described in AS/NZS 4600 S100. Lastly, the Elastic Buckling Analysis is a method that focuses on analyzing the stability of the members under different loading conditions. To make sure the members are built to resist buckling, it entails calculating the critical buckling loads and comparing them with the applied loads and it also uses finite strip analysis.

Experimental of cold-formed steel main framed in a low-rise structure to analyze the behavior, characteristics, and percentage of material to indicate if we can use it in design. By ANSYS and ABAQUS software we determine the flexural and buckling of frame in applying loads.

### *F. RESEARCH GAP*

Despite the increasing use of cold-formed steel in residential buildings and the availability of direct strength methods, there is a lack of comprehensive studies that investigate the specific design considerations and performance of these structures under various loading conditions.

This research gap suggests the need for further investigation into the analysis and design of a two-storey cold-formed steel residential building using the direct strength method. Specifically, there is a need to explore whether the Philippines is adopting the use of cold-formed steel in the construction industry or is ready to adopt the standard yet.

The need for it to be used as part of the framework has grown recently in the Philippines. However, there hasn't been much research done in the country to verify its true strength. National Structural Code of the Philippines 2015, the local code, requires CFS to be ductile for it to meet the design standards. Nevertheless, it was found that the country is also commercially distributing CFS that exhibits higher strength but with brittle properties. by addressing this research gap, future studies can contribute to the changes in design for cold-formed steel in the NSCP 2015.

## III. PROBLEM STATEMENT

The study aims to answer the following.

1. Can the proposed two-storey cold-formed steel design resist different types of loads like dead load, live load, seismic load, and wind load?
2. Will the use of cold-formed steel proven to be effective in a two-storey structure?
3. Is the design of cold-formed steel structure cost efficient?

## IV. OBJECTIVES OF THE STUDY

**GENERAL OBJECTIVE:**

The general objective of this study is to analyze and design a two-storey residential structure with cold-formed steel as the main framing system using NSCP 2015.

**SPECIFIC OBJECTIVES**:

- To evaluate the loads that is applied to the structure and investigate the applicability of the provisions of AS 4600 for the design of CFS in the Philippines.
- To determine the adequate structural members of the proposed 2-storey Residential Building by utilizing Cold-Formed Steel as a structural mainframe.
- To estimate the structural cost of constructing the Cold-Formed Steel (CFS) of the proposed residential structure.

## V. SCOPE AND LIMITATIONS

### *A. SCOPE OF THE STUDY*

The focus of this research is to analyze and design a two-storey residential structure using Cold Formed Steel (CFS) which can be adopted and applied in the Philippines. This study aims to design and analyze a cold-formed steel as the main frame of a structure using the AS 4600 code. The researchers will apply the standards and limits in terms of material spacing and design parameters. Rectangular hollow section and C-shape in welded connection as mainframe and steel deck as slab, which can lessen the load in the structure considering the estimation of materials.

### *B. LIMITATIONS OF THE STUDY*

The scope of the study will be constrained to providing electrical and plumbing plans. The AS/NZS 4600 design method DSM will act as a foundation for the CFS's design capacity.

## VI. METHODOLOGY

In this chapter, the researchers will discuss how they came up with the analysis and design of a proposed 2-story residential cold-formed steel building using DSM. The study will use an experimental quantitative research design and will apply AS/NZS 4600 codes and provisions, collect data, and use a formula to provide all the calculations to make the design building safe. All the procedures and methods utilized in determining the adequacy of structural members are included in this chapter. Also, the process of calculating the cost analysis is included.

***METHODOLOGICAL FRAMEWORK***

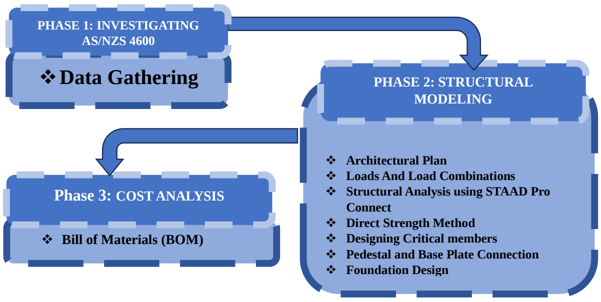



Fig 14. METHODOLOGICAL FRAMEWORK

## PHASE 1: INVESTIGATING AS/NZS 4600

### DATA GATHERING

The data required for the analysis includes Cold-formed steel (CFS) member's loading capacity, structural system, design parameters, construction, and fabrication details. The AS/NZS 4600 codes provide guidelines for designing connections between cold-formed steel members, such as welded connections. By means of gathering and interpreting the data, the researchers will use the Direct Strength Method to analyze and design a two-story CFS residential structure that meets the safety and performance requirements. Other important factors to consider in analysis and design are the analysis software and design criteria. To generate formulas that will be utilized in the CFS structure's design, the data obtained from AS/NZS 4600 will be studied, evaluated, and analyzed.

## PHASE 2: STRUCTURAL MODELING

### ARCHITECTURAL PLAN

Based on the specifications provided by the researchers, the architect will begin developing architectural plans. The floor plan, elevation plan, and perspectives can be presented via multiple software programs such as AutoCAD and will be rendered using SketchUp.

### Loads and Load Combinations

Loads and load combinations are extracted from the NSCP 2015 and ASCE 7-16.

### Load Combinations:

1.4 DEAD
1.2 DEAD + 1.6 LIVE + 0.5 ROOF LIVE
1.2 DEAD + 1.6 LIVE
1.2 DEAD + 0.5 LIVE + 1.6 ROOF LIVE
1.2 DEAD + 1.6 ROOF LIVE
1.2 DEAD + 0.5 LIVE
1.2 DEAD
1.2 DEAD + 0.5 LIVE + 0.5 ROOF LIVE
0.9 DEAD
1.2 DEAD + 0.5 LIVE + 1 SEISMIC-H (1)
1.2 DEAD + 0.5 LIVE + 1 SEISMIC-H (2)
1.2 DEAD + 0.5 LIVE + -1 SEISMIC-H (1)
1.2 DEAD + 0.5 LIVE + -1 SEISMIC-H (2)
0.9 DEAD + 1 SEISMIC-H (1)
0.9 DEAD + 1 SEISMIC-H (2)
0.9 DEAD + -1 SEISMIC-H (1)
0.9 DEAD + -1 SEISMIC-H (2)
DL+LL
WL

### STRUCTURAL ANALYSIS USING STAAD PRO CONNECT

Architectural plans are required to design the building's structural framework. Innovative analytical and design techniques utilizing AS/NZS 4600 must be applied to produce a structural design for the building that is both reliable and economical.

The structural analysis application STAAD Pro Connect will be used in designing and analyzing the structure virtually with its visualization tools, and dynamic analysis capabilities.

## DIRECT STRENGTH METHOD

Table 1. Direct Strength Method

| Criteria | Limiting variables | DSM prequalification limits |
|---|---|---|
| Stiffened element in compression [Figure 1.3(C) | $b_2/t$ | ≤500 |
| Edge stiffened element in compression [Figure 2.4.2(a)] | $b/t$ | ≤160 |
| Unstiffened element in compression [Figure 2.3.1(a)] | $d/t$ | ≤60 |
| Stiffened element in bending [Figure 2.2.3(a)] | $b/t$ | ≤200 for unstiffened web ≤260 fo bearing stiffener ≤300 for bearing and intermediate stiffener |
| Inside bend radius | $r_{min}lt$ | ≤20 |
| Simple edge stiffener overall length/overall width ratio | $\frac{(d + r_{min} + l)}{(b + 2r_{min} + 2l)}$ | ≤0.7 |
| Maximum number of intermediate stiffeners in $b_2$ | $n_f$ | 4 |
| Maximum number of intermediate stiffeners in b | $n_{fe}$ | 2 |
| Maximum number of intermediate stiffeners in web | $n_w$ | 4 |
| Yield stress used in design | $f_y$ | 655 MPa |

### Compression Members without Holes

For $\lambda_c \leq 1.5$: $N_{cc} = (0.658^{\lambda\ 2/e})\ N_y$

For $\lambda_c \leq 1.5$: $N_{cc} = (\frac{0.877}{\lambda_e^2})\ N_y$

$\lambda_c$ = non-dimensional slenderness used to determine $N_{ce}$

$= \sqrt{\frac{Ny}{Noc}}$

$N_{oc}$ = least of the elastic compression member buckling load in flexural,
torsional and flexural-torsional buckling

$= A_g f_{oc}$

$N_y$ = nominal yield capacity of the member in compression

$= A_g f_y f_{oc}$ may be determined in accordance with a rational elastic buckling analysis

### Compression Members without Holes

For $\lambda_l \leq 0.776$: $N_{cl} = N_{cc}$

For $\lambda_l > 0.776$: $N_{cl} = [1\text{-}0.15(\frac{N_{ol}}{N_{ce}})^{0.4}]\ [(\frac{N_{ol}}{N_{ce}})^{0.4} N_{ce}]$

where

= non-dimensional slenderness used to determine $N_{cl}$

$= \sqrt{\frac{N_{ce}}{N_{ol}}}$

$N_{olb}$ = elastic local buckling load

$= A_g f_{ol}$

$f_{ol}$ = elastic local buckling stress determined in accordance with a rational elastic
buckling analysis or Paragraph D1.3, Appendix D

### Beams without Holes

For $M_o < 0.56M_y$: $M_{be} = M_o$

For $2.78M_y \geq M_o \geq 0.56M_y$: $M_{be} = \frac{10}{9} M_y\ [1\text{-}\ (\frac{10M_y}{36M_o})]$

For $M_o > 2.78\ M_y$: $M_{be} = M_y$

Where

$M_o$ = Elastic lateral-torsional buckling moment determined rational elastic buckling analysis or Paragraph D2.1, Appendix D

$M_y$ = $Z_f f_y$

Where

$Z_f$ = full section modulus of the extreme fiber at first yield

Inelastic reserve for lateral-torsional buckling shall be calculated as follows:

For $M_o > 2.78M_y$:

$M_{be} = M_p - (M_p - M_y)[\frac{(\sqrt{(\frac{My}{Mo})}\ -0.23)}{0.37}] \leq Mp$

Where:

$M_o$ = elastic lateral-torsional buckling moment

$M_y$ = yield moment [see Equation 7.2.2.2(4)]

$M_p$ = plastic moment

$= S_f f_y$

$S_f$ = plastic section modulus

### Beams without Holes

For $\lambda_l \leq 0.776$:

$M_{bl} = M_{be}$

For $\lambda_l > 0.776$:

$M_{bl} = [1\text{-}0.15(\frac{M_{ol}}{M_{be}})^{0.4}][(\frac{M_{ol}}{M_{be}})^{0.4} M_{be}]$

Where:

$\lambda_l$ = non-dimensional slenderness used to determine $M_{bl}$

$= \sqrt{\frac{M_{be}}{M_{ol}}}$

$M_{ol}$ = elastic local buckling moment|

$= Z_f f_{ol}$

$f_{ol}$ = elastic local buckling

For $\lambda_l \leq 0.776$ and $M_{be} > M_y$:

## DESIGN OF MEMBERS SUBJECT TO SHEAR, AND COMBINED BENDING AND SHEAR

For $\lambda v \leq 0.815$:

$V_v = V_y$

For $0.815 < \lambda v \leq 1.227$:

$V_v = 0.815\sqrt{V_{cr}V_y}$

For $\lambda v > 1.227$:

$V_v = V_{er}$

where

$\lambda v = \sqrt{\frac{V_y}{V_{cr}}}$

$V_y$ = yield shear force of section

$= 0.6A_w f_y$

## PEDESTAL AND BASE PLATE CONNECTION

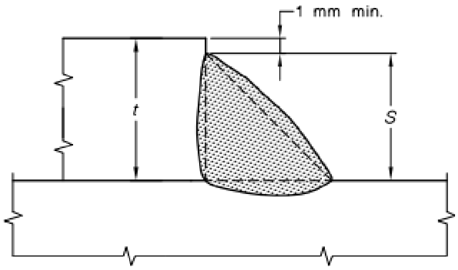
Base plate is a critical interface between the steel compression member and the concrete pedestal. It is a smooth transition of a load from your steel compression member towards the concrete pedestal by distribution of point load to compression pressure and the researchers will be going to use IDEA StatiCa 23.1 in designing the base plate and pedestal connection.

## DESIGN OF CONNECTION

Researchers will use fillet welding for the connection of the structural members.

### Fillet Welding

Under AS1554 Section 3 Figure 3.3.6, the size of fillet weld is less than the thickness of part joined minus 1mm if the thickness of members joined is greater than 6mm. Therefore, the assumed thickness of 5mm is okay.




(b) S < t – 1 mm for t > 6 mm

A fillet weld subject to a design shear force ($Vw_{req}$) shall satisfy-

$Vw_{req} = ØV_w$

### Longitudinal loading

For longitudinal loading, $ØV_w$ shall be determined as follows from the lesser of Items (a)(i) and (b)(1), or the lesser of Items (a)(ii) and (b)(ii), as applicable, as follows:

(a) (i) For $I_w / t_2 < 25$:

$Ø = 0.60$

$V_w = [\,1 - 0.01I_w / t_1]\, t_1 I_w f_{u1}$

(ii) For $I_w / t_2 < 25$:

$Ø = 0.60$

$V_w = [\,1 - 0.01I_w / t_2\,]\, t_2 I_w f_{u2}$

(b) (i) For $I_w / t_1 \geq 25$:

$Ø = 0.55$

$V_w = 0.75t_1 I_w f_{u1}$

(ii) For $I_w / t_2 \geq 25$:

$Ø = 0.55$

$V_w = 0.75t_2 I_w f_{u2}$

### Transverse Loading

For transverse loading $ØV_w$ shall be determined as follows

$Ø = 0.60$

$V_w = t_1 I_w f_{u1}$: *or*

$= t_2 I_w f_{u2}$ whichever is lesser

## FOUNDATION DESIGN

The Structure will use Isolated Square Column Footing for the foundation.

## PHASE 3: COST ANALYSIS

### Bill of Materials (BOM)

A bill of materials is a compilation of components or materials needed for construction. It includes information such as description, quantity, units, and unit cost. It is helpful for researchers to estimate materials needed for a proposed structure and make informed decisions about procurement and inventory management. In cold-formed steel members, a BOM can also optimize designs and reduce material waste. It can also help identify cost savings and areas for improvement in the construction process.

## VII. RESULTS AND DISCUSSIONS

The chapter shows the results and data collected from designing and computations of the structures, structural analyzation and evaluation were done to discover critical parts of the whole structure.

### *A. ARCHITECTURAL PLAN*

The structure has 8 units (40 sqm per unit). With a total length 20m, width of 8m, and height per floor of 3m, with a total elevation of 6m. Also, the hallway has a total length of 22.6m and a width of 1.5m. The monoslope roof is 10°.

### *B. DESIGN LOADINGS*

| Deadloads | | |
|---|---|---|
| **Component** | **Materials** | **Load(kPa)** |
| Ceiling | Gypsum board | 0.15 |
| Covering | Gypsum sheathing | 0.1 |
| Floor Fill | Light weight concrete | 0.015 |
| Frame Partition | Wood studs 50x100 plastered one side | 0.57 |
| Frame Walls | 50x150 @400mm, 15mm gypsum, Insulated, 10mm siding | 0.57 |
| Plumbing | | 0.1 |
| Truss | | 1 |
| Electrical | | 0.1 |
| Roofing | | 0.144 |
| Slab | | 2.242 |

**Slab**

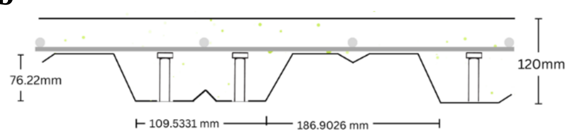



Fig 15. Slab

**Provision (SDI)**

for LWC = 110pcf = 17.25 kN/m$^3$
DLSD= 3.5 psf = 0.168 KPa½ (186.9026mm + 109.5331mm) (26 pcs) (76.2 mm) (20000mm) = 5.875x109mm2

17.28 kN/m$^3$ (5.875x10$^9$ mm$^3$) x ((1m$^3$)/(1000mm^3 )) = 101.512 kN

(101.512 KN)/(77.073x10$^6$mm$^2$) (1000mm$^2$/1m$^2$)
W= 1.317 KPa

½ (186.9026 + 109.5331) (26) (20000) = 77.073x106mm$^2$
17.28 kN/m$^3$(43.78mm) (1m/1000mm) = 0.757 KPa
WSD= 0.168 KPa

WT= 1.317 + 0.757 + 0.168 = 2.242 KPa

**Live load (KPa):**
Residential Occupancy = 1.9
Roof Live load = 0.6

**Wind load (KPa):**
Occupancy Category = Standard
Basic Wind Speed = 280 kph = 77.78
Wind Directionality Factor, KD = 0.85
Topographic Factor, Kzt = 1.0
Enclosure Classification, = Et1 closed Building
Exposure Category = C
Gust Effect Factor, G = 0.85
Internal Pressure Coefficient, GCp1 = 0.18
Terrain Exposure Constants
Zg = 274.32
Velocity Pressure Exposure Coefficient
Kh and Kz
h = 6.0 + (9.5 tan10)/2= 6.838m

Table 2. Wind loads

| z/h | $K_h/K_z$ |
|---|---|
| 0 | 0.85 |
| 4.5 | 0.85 |
| 6 | 0.90 |
| 6.838 | 0.922 |

Velocity Pressure, $q_z$ / $q_h$
$q_{z/h}$ = 0.613 ($Kz_{/h}$) ($K_{zt}$) ($K_d$) ($V^2$)
$q_z$ = 0.613 (0.85) (1.0) (0.85) (77.78)$^2$ = 2.679 KPa
$q_z$ = 0.613 (0.90) (1.0) (0.85) (77.78)$^2$ = 2.837 KPa
$q_h$ = 0.613 (0.922) (1.0) (0.85) (77.78)$^2$ = 2.906KPa

External Pressure Coefficient, $C_p$
Walls:
Wind load = $C_p$ = 0.80
Leeward = $\frac{L}{B} = \frac{9.85}{20}$ = 0.975 = $C_p$ = -0.5
Sidewall = $C_p$ = -0.70
Wind Pressure, P:
Burst Condition ($GCp_1$ = + 0.18)

Windward:

Table 3. Wind loads (Windward)

| z | $P = q\ GC_p - q_1\ (GC_{p1})$ |
|---|---|
| 4.5 | 2.679 (0.85) (0.8) - 2.906 (0.18) = + 1.299 |
| 6 | 2.837 (0?85) (0.8) - 2.906 (0.18) = + 1.406 |

Leeward:
P= 2.906(0.85) (-0.5) - 2.906(0.18) = - 1.758
Sidewall:
P= 2.906(0.85) (-0.7) - 2.906(0.18) = -2.252
Suction Condition (GCp1 = -0.18)
Leeward:

Table 4. Wind loads (Leeward)

| Z | P |
|---|---|
| 4.5 | 2.679(0.85) (0.8) - 2.906(-0.18) = +2.345 |
| 6 | 2.837(0.85) (0.8) - 2.906(-0.18) = +2.452 |

Leeward:
P=2.906(0.85) (-0.5) - 2.906 ( -0.18) = -0.712
Sidewall:
P=2.906 (0.85) (0.7) - 2.906 (-0.18) = - 1.206
Roof
Basic Wind Speed = 280kph

Table 5. Basic wind speed

| h(m) | Zone 1 | Zone 2 | Zone 3 |
|---|---|---|---|
| 6.838 | -4.0428 | -5.2146 | -9.0230 |
| 6,838 | 1.6992 | 1.6992 | 1.6992 |

## Seismic Parameters:

The proposed structure is located at Bacolor, Pampanga, and has a 52.5 km distance from the near source active fault based on PHILVOCS Faultfinder.

Table 6. Seismic Parameters

| PARAMETER | VALUE |
|---|---|
| Zone | 0.4 |
| Importance Factor (I) | 1 |
| Rw in X direction | 8 |
| Rw in Z direction | 8 |
| Soil Profile Type | 4 |
| Na | 1 |
| Nv | 1 |
| Ct | 0.0853 |

## Design of Purlins

(Assume Spacing=0.600m)
Roofing= 0.144 KPa (0.6m) =0.0864 $\frac{kN}{m}$
Roof Live Load = 0.6 KPa (0.6m) = 0.36 $\frac{kN}{m}$
Wind Pressure = 1.6992 KPa (0.6m) = 1.0195 $\frac{kN}{m}$
Estimated Weight of Purlins:
350 Cu125-68 = 0.0218 $\frac{kN}{m}$
350 Cu125-27 = 0. 0137 $\frac{kN}{m}$

## Summary of Loading

Table 7. Summary of Loading

| Section | Maximum Applied Load, W (kN/m) | Deck Weight, DW (kN/m) | Purlin Weight, PW (kN/m) | $W_D$ ($\frac{kN}{m}$) DW + PW | Total Load $W_{TS}$ (kN/m) WD + W | $M_{TS}$ (kN-m) $\frac{W_{TS}\ L^2}{8}$ |
|---|---|---|---|---|---|---|
| 1 | 1.0858 | 0.0864 | 0.0218 | 7.69 | 1.1940 | 0.9328 |
| 2 | 1.0858 | 0.864 | 0.0137 | 6.807 | 1.1859 | 0.9265 |

## Reduction Factor Data

Table 8. Reduction Factor Data

| | Thickness (mm) | $S_{eft}$ (mm³) | fyt (MPa) | $M_{nt}$(kN-m) = $S_{eft}$ fyt | $M_{TS}$ | $R_t = M_{ts}/M_{nt}$ |
|---|---|---|---|---|---|---|
| 1 | 1.7780 | 7.374 2x10³ | 248 | 1.8288 | 0.9 328 | 0.5701 |
| 2 | 1.2700 | 4.752 2x10³ | 248 | 1.1785 | 0.9 265 | 0.786 |

$\sigma_{Max}$= 0.0148
$\sigma_{Min}$= 0.0271
$R_{Tmin}$ = 0.5101 - 0.0271 = 0.4830
$R_{Tmax}$ = 0.7860 - 0.0148 = 0.7712
$M_{nt\ min}$ = 1.1785 kN-m
$M_{nt\ max}$ = 1.8288 kN-m

**Reduction Factor Relation**

$R = (\frac{R_{T\,max} - R_{T\,min}}{M_{NT\,max} - M_{NT\,min}})(M_n - M_{nt\,min}) + R_{Tmin} \leq 1.0$

$= (\frac{0.7712 - 0.4830}{1.8288 - 1.1785})\ (M_n - 1.1785) + 0.4830$

$R = 0.4432(M_n - 1.1785) + 0.4830$

**Application**

$M_n = 8.1116 \times 10^6\ (248) = 2.0128$ kN-m

$R = 0.4432(2.0128 - 1.1785) + 0.483$

$R = 0.853 < 1.0$

∴ Adequate

∴ Spacing 600mm

### *C. STRUCTURAL MODELING*

The structure was modeled using Staad Pro Connect Edition v22, a section data base from Australian cold-formed standards.

**Cold-Formed Steel Specifications**

Table 9. Cold Formed Steel Properties

| CFS PROPERTIES | |
|---|---|
| COLUMN | 200x200x9mm |
| BEAM 1 | 200x100x9mm |
| BEAM 2 | 150x100x6mm |

**Structural Modeling**

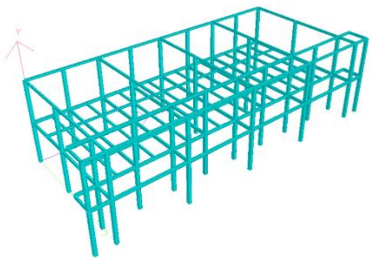


Fig 16. Structural Framing, Isometric View using STAAD Pro CONNECT

### *D. DESIGN OF CRITICAL MEMBERS*

**General**

This is the basis of design for the Structural Engineering of the proposed 2-storey Cold-Formed Steel Residential Building.

**Applied Codes, Standards, and References**

These are the latest Codes and Provisions used in designing the structure.

- National Structural Code of the Philippines 2015 Volume 1, 7th Edition
- ASCE 7-16, American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures
- Australian Standard/New Zealand Standard (AS/NZS 4600)

### *E. STRUCTURAL REPORT*

**General**

Analysis of the structure using STAAD Pro Connect was conducted, which is the basis for the actual/allowable ratio in the design.

**Base Shear Result/Earthquake Load**

**Base Shear at X- Direction**

```
 ***********************************************************
 *                                                         *
 *  X DIRECTION: Ta = 0.327 Tb = 0.385 Tuser = 0.000       *
 *  T = 0.385, LOAD FACTOR = 1.000                         *
 * UBC TYPE = 97                                           *
 * UBC FACTOR V = 0.1375 x    1227.64 =    168.80 KN       *
 *                                                         *
 ***********************************************************
```

**Base Shear at Z- Direction**

```
 ***********************************************************
 *                                                         *
 *  Z DIRECTION: Ta = 0.327 Tb = 0.350 Tuser = 0.000       *
 *  T = 0.350, LOAD FACTOR = 1.000                         *
 * UBC TYPE = 97                                           *
 * UBC FACTOR V = 0.1375 x    1227.64 =    168.80 KN       *
 *                                                         *
 ***********************************************************
```

**Utilization Ratio**

**Lowest Member Actual Ratio**

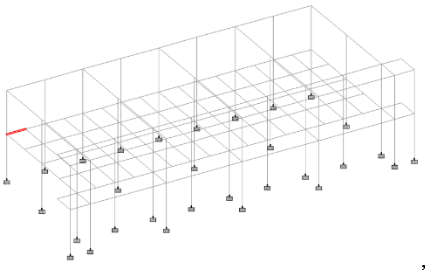


Fig 17. Lowest member ratio

Table 10. Summary of Lowest Member Ratio

| Beam | Analysis Property | Design Property | Actual Ratio | Allowable Ratio | Normalized Ratio (Actual/Allowable) | Clause | L/C | Ax m2 | Iz m4 | Iy m4 | Ix m4 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 52 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 54 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 56 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 59 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 60 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 61 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 62 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 63 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 66 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.005 | 0.000 | 0.000 | 0.000 |
| 105 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | AS-8.4.4.1 | 17 | 0.005 | 0.000 | 0.000 | 0.000 |
| 108 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 4 | 0.005 | 0.000 | 0.000 | 0.000 |
| 114 | 200X100X9.0R | 200X100X9.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 4 | 0.005 | 0.000 | 0.000 | 0.000 |
| 137 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 138 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 139 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 187 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 189 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 191 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 193 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 196 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 199 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 202 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 213 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 216 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 219 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 222 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |
| 226 | 150X100X6.0R | 150X100X6.0R | 0.174 | 1.000 | 0.174 | SLENDERNE | 5 | 0.003 | 0.000 | 0.000 | 0.000 |

## Highest Member Actual Ratio

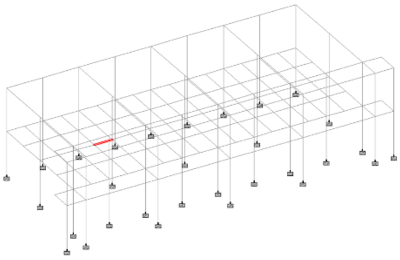


Fig 18. Highest member ratio

Table 11. Summary of Highest Member Ratio

| Beam | Analysis Property | Design Property | Actual Ratio | Allowable Ratio | Normalized Ratio (Actual/Allowable) | Clause | L/C | Ax m2 | Iz m4 | Iy m4 | Ix m4 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 239 | 150X100X6.0R | 150X100X6.0R | 0.990 | 1.000 | 0.990 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 250 | 150X100X6.0R | 150X100X6.0R | 0.979 | 1.000 | 0.979 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 241 | 150X100X6.0R | 150X100X6.0R | 0.948 | 1.000 | 0.948 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 240 | 150X100X6.0R | 150X100X6.0R | 0.943 | 1.000 | 0.943 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 244 | 150X100X6.0R | 150X100X6.0R | 0.938 | 1.000 | 0.938 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 247 | 150X100X6.0R | 150X100X6.0R | 0.933 | 1.000 | 0.933 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 242 | 150X100X6.0R | 150X100X6.0R | 0.841 | 1.000 | 0.841 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 254 | 150X100X6.0R | 150X100X6.0R | 0.827 | 1.000 | 0.827 | AS-8.4.4.1 | 8 | 0.003 | 0.000 | 0.000 | 0.000 |
| 90 | 200X100X9.0R | 200X100X9.0R | 0.696 | 1.000 | 0.696 | SLENDERNE | 4 | 0.005 | 0.000 | 0.000 | 0.000 |
| 92 | 200X100X9.0R | 200X100X9.0R | 0.696 | 1.000 | 0.696 | SLENDERNE | 4 | 0.005 | 0.000 | 0.000 | 0.000 |
| 94 | 200X100X9.0R | 200X100X9.0R | 0.696 | 1.000 | 0.696 | SLENDERNE | 4 | 0.005 | 0.000 | 0.000 | 0.000 |
| 95 | 200X100X9.0R | 200X100X9.0R | 0.696 | 1.000 | 0.696 | SLENDERNE | 4 | 0.005 | 0.000 | 0.000 | 0.000 |
| 180 | 150X100X6.0R | 150X100X6.0R | 0.695 | 1.000 | 0.695 | SLENDERNE | 17 | 0.003 | 0.000 | 0.000 | 0.000 |
| 181 | 150X100X6.0R | 150X100X6.0R | 0.695 | 1.000 | 0.695 | SLENDERNE | 16 | 0.003 | 0.000 | 0.000 | 0.000 |
| 182 | 150X100X6.0R | 150X100X6.0R | 0.695 | 1.000 | 0.695 | SLENDERNE | 16 | 0.003 | 0.000 | 0.000 | 0.000 |

## Critical Nodal Displacement Location

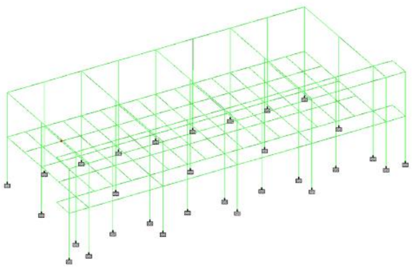


Fig 19. Critical Nodal Displacement, node 58

Table 12. Summary of Nodal Displacement Results

| | | | Horizontal | Vertical | Horizontal | Resultant | Rotational | | |
|---|---|---|---|---|---|---|---|---|---|
| | Node | L/C | X mm | Y mm | Z mm | mm | rX deg | rY deg | rZ deg |
| Max X | 113 | 15 COMB - 1.2 | 14.504 | -0.195 | 0.549 | 14.515 | 0.006 | -0.002 | -0.189 |
| Min X | 121 | 24 WL | -22.989 | 0.013 | -11.351 | 25.639 | -0.085 | 0.009 | 0.397 |
| Max Y | 58 | 24 WL | -3.959 | 2.044 | -6.809 | 8.138 | 0.006 | -0.013 | -0.022 |
| Min Y | 145 | 7 COMB - 1.2 | 0.065 | -14.868 | 0.312 | 14.871 | -0.026 | -0.001 | 0.020 |
| Max Z | 103 | 16 COMB - 1.2 | -0.036 | -0.138 | 10.913 | 10.914 | 0.114 | 0.003 | 0.001 |
| Min Z | 104 | 24 WL | -5.167 | -0.012 | -18.155 | 18.876 | -0.218 | 0.003 | 0.029 |
| Max rX | 58 | 7 COMB - 1.2 | -0.021 | -10.725 | 0.327 | 10.730 | 0.271 | 0.000 | -0.003 |
| Min rX | 128 | 7 COMB - 1.2 | -0.016 | -9.489 | 0.297 | 9.494 | -0.271 | -0.002 | 0.021 |
| Max rY | 106 | 24 WL | -7.924 | -0.145 | -12.882 | 15.125 | -0.083 | 0.085 | 0.058 |
| Min rY | 98 | 24 WL | -5.239 | -0.000 | -11.375 | 12.523 | -0.088 | -0.253 | 0.032 |
| Max rZ | 146 | 8 COMB - 1.2 | 0.046 | -8.732 | 0.338 | 8.739 | -0.028 | -0.001 | 0.460 |
| Min rZ | 155 | 8 COMB - 1.2 | -0.127 | -8.567 | 0.343 | 8.574 | -0.015 | 0.000 | -0.458 |
| Max Rst | 113 | 24 WL | -22.356 | -0.005 | -12.876 | 25.799 | -0.099 | -0.017 | 0.381 |

The maximum vertical displacement has a **2.044 mm** with **Wind Load** nodal displacement result.

## Maximum Axial Force

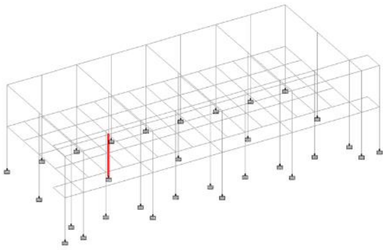


Fig 20. Column 118

Table 13. Maximum Axial Force Summary

| | Beam | L/C | Node | Fx kN | Fy kN | Fz kN | Mx kN-m | My kN-m | Mz kN-m |
|---|---|---|---|---|---|---|---|---|---|
| Max Fx | 118 | 7 COMB - 1.2 | 11 | 173.368 | 0.799 | 1.358 | 0.006 | 1.944 | -0.830 |
| Min Fx | 42 | 24 WL | 24 | -66.067 | -12.713 | 14.301 | -0.690 | -23.919 | -17.856 |
| Max Fy | 135 | 24 WL | 78 | 3.665 | 35.900 | -0.203 | -0.253 | 0.212 | 25.872 |
| Min Fy | 263 | 8 COMB - 1.2 | 87 | -0.442 | -37.213 | 0.111 | 0.321 | 0.115 | 31.438 |
| Max Fz | 23 | 24 WL | 19 | 41.189 | -9.144 | 18.100 | -0.078 | -29.303 | -15.896 |
| Min Fz | 91 | 24 WL | 27 | -49.417 | -7.950 | -16.374 | 0.021 | -27.619 | 14.750 |
| Max Mx | 18 | 24 WL | 47 | 0.188 | -4.685 | 4.908 | 5.278 | -3.859 | -3.018 |
| Min Mx | 50 | 7 COMB - 1.2 | 31 | 0.263 | 10.550 | 0.110 | -3.551 | -0.092 | 3.823 |
| Max My | 8 | 7 COMB - 1.2 | 45 | 70.251 | -0.369 | 11.278 | -0.040 | 23.378 | 0.711 |
| Min My | 23 | 24 WL | 19 | 41.189 | -9.144 | 18.100 | -0.078 | -29.303 | -15.896 |
| Max Mz | 263 | 8 COMB - 1.2 | 87 | -0.442 | -37.213 | 0.111 | 0.321 | 0.115 | 31.438 |
| Min Mz | 135 | 24 WL | 66 | 3.665 | 35.900 | -0.203 | -0.253 | -0.092 | -27.979 |

**1.2 DEAD + 1.6 LIVE + 0.5 ROOF** load combination for Maximum Axial Force. The Beam 118, node 11 which carries an Axial Force of **173.368 kN**.

## Maximum Moment Summary

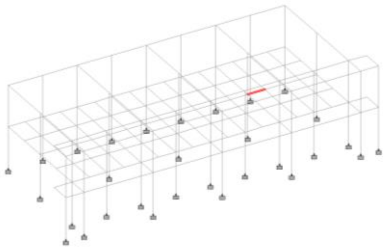


Fig 21. Beam 250

Table 14. Maximum Moment Summary

| Beam | L/C | Node | Axial Force kN | Shear-Y kN | Shear-Z kN | Torsion kN-m | Moment-Y kN-m | Moment-Z kN-m |
|---|---|---|---|---|---|---|---|---|
| 250 | 16 | 88 | 5.539 | 21.584 | 0.024 | 0.710 | -0.052 | 20.089 |
| 244 | 16 | 87 | 5.003 | 20.959 | -0.572 | 0.731 | 0.375 | 19.270 |
| 247 | 16 | 90 | 4.837 | 20.936 | -0.359 | 0.750 | 0.231 | 19.176 |
| 81 | 16 | 33 | 3.315 | 21.942 | 0.555 | 0.137 | -0.640 | 17.360 |
| 84 | 16 | 45 | 3.304 | 21.925 | -0.544 | 0.492 | 0.629 | 17.313 |
| 254 | 16 | 86 | 6.089 | 19.983 | -1.353 | 0.618 | 0.952 | 16.956 |
| 69 | 16 | 50 | 2.428 | -5.353 | -0.179 | 0.358 | -0.152 | 16.690 |
| 77 | 16 | 55 | 2.433 | -5.334 | 0.232 | 0.234 | 0.201 | 16.678 |
| 85 | 16 | 62 | 2.516 | -5.150 | 0.030 | 0.294 | 0.027 | 16.646 |
| 130 | 16 | 89 | 74.159 | 7.649 | 6.075 | -0.091 | -7.663 | 15.685 |
| 82 | 16 | 37 | 2.525 | 20.226 | 0.062 | 0.312 | -0.072 | 14.687 |
| 83 | 16 | 41 | 2.522 | 20.226 | -0.066 | 0.293 | 0.079 | 14.685 |
| 264 | 16 | 87 | -1.644 | 21.115 | 0.106 | -0.008 | -0.092 | 13.497 |

For Max Moment, The Beam 250, node 88 has a total of **20.089 kN-m** with a load combination of **1.2 DEAD + 0.5 LIVE + 1 SEISMIC**.

**Maximum Shear Summary**

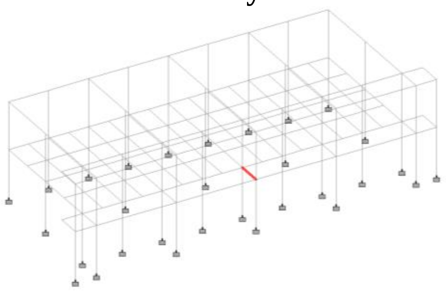


Fig 22. Beam 263

Table 15. Maximum Shear Summary

| Beam | L/C | Node | Axial Force kN | Shear-Y kN | Shear-Z kN | Torsion kN-m | Moment-Y kN-m | Moment-Z kN-m |
|---|---|---|---|---|---|---|---|---|
| 263 | 8 | 87 | 0.442 | 37.213 | -0.111 | -0.321 | -0.115 | -31.438 |
| 259 | 8 | 88 | 0.441 | 37.203 | 0.142 | -0.218 | 0.153 | -31.431 |
| 263 | 7 | 87 | 0.904 | 37.190 | -0.112 | -0.321 | -0.116 | -31.405 |
| 259 | 7 | 88 | 0.903 | 37.179 | 0.142 | -0.218 | 0.153 | -31.399 |
| 261 | 8 | 90 | 0.441 | 36.984 | 0.016 | -0.268 | 0.020 | -31.234 |
| 261 | 7 | 90 | 0.904 | 36.961 | 0.016 | -0.268 | 0.020 | -31.202 |
| 135 | 24 | 78 | 3.665 | 35.900 | -0.203 | -0.253 | 0.212 | 25.872 |
| 117 | 24 | 74 | 3.928 | 35.280 | -0.352 | -0.260 | 0.347 | 25.410 |
| 125 | 24 | 82 | 3.929 | 35.237 | 0.119 | -0.275 | -0.030 | 25.410 |
| 264 | 8 | 87 | -0.917 | 33.610 | -0.065 | -0.010 | 0.072 | 27.701 |
| 260 | 8 | 88 | -0.917 | 33.607 | 0.104 | 0.097 | -0.117 | 27.694 |
| 264 | 7 | 87 | -1.323 | 33.583 | -0.065 | -0.010 | 0.072 | 27.656 |
| 260 | 7 | 88 | -1.322 | 33.580 | 0.104 | 0.097 | -0.117 | 27.649 |
| 262 | 8 | 90 | -0.924 | 33.351 | 0.019 | 0.043 | -0.022 | 27.437 |
| 262 | 7 | 90 | -1.330 | 33.324 | 0.019 | 0.043 | -0.022 | 27.391 |
| 261 | 16 | 90 | 0.026 | 32.406 | 0.026 | -0.268 | 0.029 | -30.733 |

For Max Shear Y, The Beam 135, node 87 has a total of **35.9 kN** with load combination **1.2 DEAD + 1.6 LIVE**.

**DESIGN OF MEMBERS**

**DESIGN OF COMPRESSION MEMBERS**

Limits for design using DSM

$\frac{b}{t} \leq 160$

$\frac{200\text{mm}}{9\text{mm}} = 22.222$

∴ DSM is applicable

N= 74.99 kN

Ny = AgFy

Ag = 200mm x 200mm = 40000 $\text{mm}^2$

Fy = 248MPa

Ny = 40000$\text{mm}^2$ (248MPa) = 9.92x$10^6$ N

$N_{oe}$ = Ag foe

foe = $\frac{\pi^2 E}{(\frac{Le}{r})^2}$

E= 200x10³ MPa

Le= 0.65 L

= 0.65(3000)

= 1950 mm

$r_x = \sqrt{\frac{l_x}{A}} = \sqrt{\frac{64.5\text{x}10^6}{6600}}$

= 49.62 mm

$ry = \sqrt{\frac{l_y}{A}} = \sqrt{\frac{39.2\text{x}10^6}{6600}}$

= 77.07 mm

$f_{oe} = \frac{\pi^2(200\text{x}10^3)\frac{N}{mm^2}}{(\frac{1950\ mm}{77.07mm})^2}$ = 3083.410 MPa

$N_{oe} = A_g f_{oe}$

= 40 000 $\text{mm}^2$(3083.410 $\frac{N}{mm^2}$)

$N_{oe}$ = 123.336 x10⁶ N

$\lambda = \sqrt{\frac{N_y}{N_{oe}}}$

$\lambda = \sqrt{\frac{9.92\text{x}10^6}{123.336\text{x}10^6}}$

λ = 0.284

∴ λ < 1.5

$\therefore N_{ce} = \left[0.658^{\lambda^2}\right]Ny$

$= \left[0.658^{(0.284)^2}\right] 9.92\text{x}10^6\ N$

$N_{ce} = 0.75948\text{x}10^3 N$

$= 759.48kN$

Ø$N_{ce}$ =0.85(759.48) = **645.56 kN** > **173.543 kN**

Actual/Allowable ratio = 0.269 < 1.0

**∴ safe**

In designing the Compression Members using AS4600 the most critical member is the section with a 200mm x 200mmx9mm property, the actual ratio is less than the allowable ratio, with the result of the member being safe.

**LOCAL BUCKLING**

$N_{ol} = Agf_{ol}$

Ag = 40000$\text{mm}^2$

$f_{ol} = \left[\frac{K\pi^2 E}{12(1-V^2)}\right] \left[\frac{t}{b}\right]^2$

K = 4
E = 200X10$^6$ MPa
V = 0.3
t = 9mm
b = 200mm

$f_{ol} = \left[\frac{4\pi^2(200x10^3MPa)}{12(1-0.3^2)}\right]\left[\frac{9}{200}\right]^2$

$f_{ol}$ = 1464.172 MPa

$N_{ol}$ = Agfy

$= 400mm^2\left(1464.172\frac{N}{mm^2}\right)$

$N_{ol} = 58.567X10^6N$
$N_{ce} = 9.591x10^6N$

$\lambda_1 = \sqrt{\frac{N_{ce}}{N_{cl}}} = \sqrt{\frac{9.591x10^6}{58.567x10^6}}$

$\lambda_1 = 0.405 < 0.776$

$\therefore\ N_{cl} = N_{ce} = 95.91x10^3N$
=95.91 kN

$\emptyset N_{CL}$ =0.85(95.91) = 81.52 kN
$\emptyset N_{cl}$ =0.85(95.91) = 81.52 kN > 74.99 kN

**DESIGN OF MEMBERS SUBJECTED TO BENDING**

SECTION 150mm x 100mm x 6mm
Limits for design using DSM

$\frac{b}{t} \leq 200$

$\frac{150mm}{6mm} = 25$

∴ DSM is applicable

My =$Z_f$Fy
$Z_f$ =118x10$^3$mm$^3$
Fy =248 MPa
My =118x10$^3$[248]
My =29.264x10$^6$mm
Mo =$C_bAgr_{ol}\sqrt{f_{ol}f_{oz}}$
$M_a$ =0.4309 kN-m
$M_b$ =0.7388 kN-m
$M_c$ =0.9235 kN-m
$M_{max}$ =0.9849 kN-m

$C_b = \frac{12.5\ (0.9849)}{2.5(0.9849)+3(0.4309)+4(0.7388)+3(0.9235)}$

$C_b$ = 1.299
$A_g$ =150mm x 100mm=15000mm$^2$

$r_{ol} = \sqrt{r_{x^2} + r_{y^2} + x_o + y_o}$

$r_x = \sqrt{\frac{9.51x10^6}{2730}}$

$r_x$ = 59.02mm

$r_y = \sqrt{\frac{4.36x10^6}{2730}}$

$r_y$ = 39.96mm

$x_o = \frac{100}{2}$ =50mm

$y_o = \frac{150}{2}$ =75mm

$r_{ol} = \sqrt{59.02^2 + 39.96^2 + 50^2 + 75^2}$

$r_{ol}$ = 114.91

$f_{oy} = \frac{\pi^2E}{(\frac{Icy}{r_y})^2} = \frac{\pi^2[200x10^3]}{\left[\frac{1950}{39.96}\right]^2}$ = 828.92 MPa

**FLEXURAL - TORSIONAL BUCKLING**

SECTION 150mm x 100mm x 6mm
$A_g = 15000mm^2$
$R_{ol}$ = 114.91mm
$I_w$ = Neglect
$L_{ez}$ = 0.65(2000) = 1300mm

$f_{oz} = \frac{80x10^3(9.238x10^6)}{15000(114.91)^2}\left[1 + \frac{\pi^2(200x10^3)}{80x10^3(9.238x10^6)(1300)}\right]$

$f_{oz}$ = 3731.31 MPa
Mo =
$1136(15000)(114.91)\sqrt{828.92 \times 3731.31}$ v
$Mo = 3443.6x10^6N - m$
$Mo > 2.78\ M_y$
$\therefore M_{be} = M_y = 29.264x10^6N - mm$
$\emptyset M_{be} = 0.90(29.264\ kN - m)$
$\emptyset M_{be} = 26.338\ KN - m$
$\therefore\ \emptyset M_{bl} > Mnz(20.089\ kN - m)$
Actual/Allowable ratio = 0.763 < 1.0
∴ **safe**

In designing the Members subjected to bending using AS4600, with the most critical is the member section of 150mmx100mmx6mm, the actual ratio is less than the allowable ratio, with the result of the member being safe.

**LOCAL BUCKLING**

$M_{OL}=Z_FF_{OL}$

$f_{ol}=\left[\frac{k\pi^2 E}{12[1-x^2]}\right]\left[\frac{t}{b}\right]^2$

$=\left[\frac{4(\pi)^2(200x10^3)}{12[1-0.3^2]}\right]\left[\frac{6}{100}\right]^2$

$f_{ol}$ =2602.97 MPa

$Z_f$ =118x10$^3$mm$^3$

$M_{ol}$=118x10$^3$(2602.97)

$M_{ol}$=307.15x10$^6$N − mm

$\lambda_1 = \sqrt{\frac{M_{be}}{M_{ol}}}$

$\lambda_1 = \sqrt{\frac{29.264x10^6}{307.15x10^6}} = 0.309$

$\lambda_1 \leq 0.776$

$\therefore M_{bl} = M_{be} = 29.264$ kN − m

$\emptyset M_{bl} = 0.9(29.264$ kN − m)

$\emptyset M_{bl} = 26.338$ kN − m

**LOCAL BUCKLING**

$A_g$ = 15000mm$^2$

$r_{OL}$ = 114.91mm

$I_w$ = NEGLECT

$I_{c2}$ = 0.65 (2000) = 1300mm

E= 200x10$^3$MPa

Icy= 0.65(3000) = 1950mm

$r_y$ =39.95mm

$F_{oz} = \frac{G\,J}{Ag\ r_{ol^2}}(1 + \frac{\pi EI_W}{G\,J\,I_{ez}})$

G= 80x10$^3$MPa

$J= \frac{2b^2a^2}{\frac{b}{t_1}+\frac{a}{t_2}} \qquad = \frac{2(94)^2\,(144)^2}{\frac{94}{6}+\frac{144}{6}}$

J= 9.23x10$^6$

$f_{oz} = \frac{80x10^3(9..238x10^6)}{15000(114.91)^2}\,(1 + \frac{\pi^2(200x10^3)}{80x10^3(9.238x10^6)(1300)})$

= 3731.31MPa

$M_o$ = 1.299(15000) (114.91)$\sqrt{828.92 \text{ x } 3731.31}$

$M_o$ = 3937.72x10$^6$

Mu > 2.78Mu øM$_{BE}$ = 0.90(29.264kN-m)

ØM$_{BE}$ = 26.338kN-m

$M_{OL} = Z_p\, f_{ol}$

$F_{ol} = (\frac{k\,\pi^2\,E}{12\,(1-V^2)})(\frac{t}{b})^2$

$= (\frac{4\,(\pi)^2(200x10^3)}{12(1-0.3)^2})(\frac{6}{100})^2$

$F_{ol}$ = 2602.97 MPa

$Z_F$ = 118x10$^3$mm$^3$

$M_{OL}$ = 118x10$^3$mm$^3$(2602.97)

$M_{OL}$ = 307.15x10$^6$ N-mm

$\lambda_1 = \sqrt{\frac{M_{be}}{M_{ol}}}$

$\lambda_1 = \sqrt{\frac{29.264x10^6}{307.15x10^6}}$

$\lambda_1 = 0.309$

$\lambda_1 \leq 0.776$

∴ $M_{BL} = M_{BE}$ = 29.264 kN-m

Ø $M_{BL}$ = 0.9(29.264 kN-m)

Ø $M_{BL}$ = 26.338 kN-m

$f_{Ol} = [\frac{k\,\pi^2 E}{12(1-V^2)}](\frac{t}{b})^2$

$f_{Ol} = [\frac{4\,(\pi)^2(200x20^3)}{12(1-0.3)^2}](\frac{6}{100})^2$

$f_{Ol}$ = 2602.97 MPa

$Z_f$ = 118x10$^3$mm$^3$

$M_{OL}$ = 118x10$^3$mm$^3$

$M_{OL}$ = 118x10$^3$(2602.97)

$M_{OL}$ = 307.15x10$^6$N-mm

**DESIGN OF MEMBER SUBJECTED TO SHEAR AND COMBINED BENDING**

SECTION 150x100x6mm

Vy = 0.6 AwFy

Aw = ht

Aw = 150(6)

Aw = 900mm$^2$

Fy = 248 MPa

Vy = 0.6(900) (248)

Vy = 133.92 kN

$V_{CR} = \frac{\pi^2\,E\,Aw\,kv}{12\,(1-V^2)\,(\frac{d_1}{t})^2}$

$V_{CR} = \frac{\pi^2(200x103)(900)(5.34)}{12\,(1-0.32)\,(\frac{150}{6})^2}$

$V_{CR}$ = 1389.9874 kN

$\lambda_1 = \sqrt{\frac{V_y}{V_{CR}}}$

$\lambda_1$ = √133.92 / 1389.9874

$\lambda_1$ = 0.310 < 0.815

Vv = Vy = 133.92 kN

ØVv = 0.9(133.92)

ØVv = 120.528 kN

∴ $\emptyset V_v > V_{v_{act}}$(37.213kN)
Actual/Allowable ratio = 0.309<1.0
∴ **safe**

In designing the Members subjected to shear using AS4600 with the most critical member section of 150mmx100mmx6mm, the actual ratio is less than the allowable ratio, with the result of the member being safe.

**BASE PLATE DESIGN AND CONNECTIONS**

In designing pedestal connection IDEA StatiCa was used.

**COLUMN TO PEDESTAL CONNECTION DETAILS**

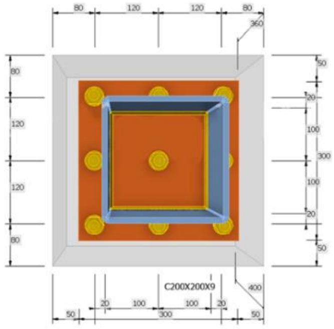



Fig 23. Column and Pedestal weld and bolts connection (top view)

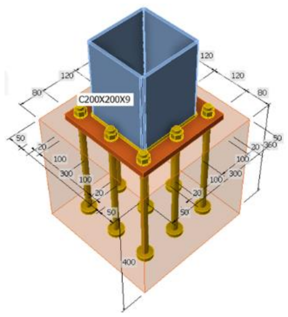
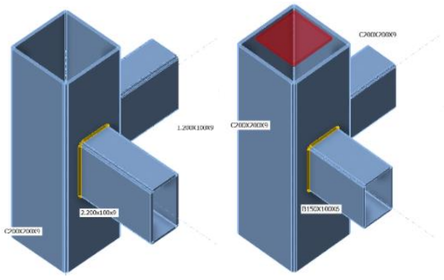
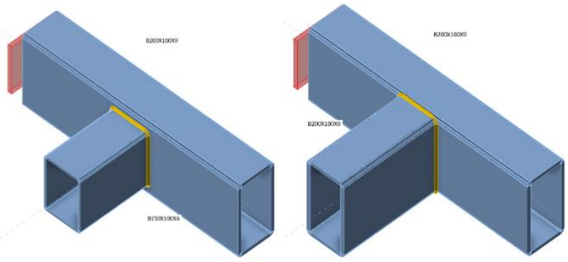



Fig 23. Column and Pedestal connection (isometric view)

**COLUMN AND BEAM CONNECTION**



**BEAM TO BEAM CONNECTION**



**3.6.8.1 Welding Connection**

t=9mm > 6mm
S < t-1
5 < 9-1
5mm < 8mm
∴**OKAY**

**COLUMN:** Fillet weld
$V_{w\ req}$ = 95.6 kN
Ø = 0.60
lw = 800 mm
t1 = 9 mm
t2 = 20 mm
fu1 =fu2 = 400 MPa
Vw = t1lwfu1 or t2lwfu2
Vw1 = 9mm(800mm)(400 MPa) = 2880kN
Vw2 = 20mm(800mm)(400 MPa) = 6400kN
∴ **Vw1 GOVERN**
ØVw = 0.60(2880 kN)
ØVw = 1728 kN
$V_{w\ req}$< ØVw
∴ **ADEQUATE**

**BEAM:**
$V_{w_{req}}$ = 21.54 kN
$l_w$ = 500 mm
t1= 6 mm
$t_2$= 9mm
$f_{u2}$ = 400 MPa
$$\frac{500}{6} = 83.333 > 25$$
Ø=0.55
$V_w = 0.75 t_2 I_w f_{u2}$
Vw = 0.75(9)(500)(400)
Vw = 1350 kN

$\emptyset Vw$ = 0.55(1350) = 742.6 kN
$\emptyset Vw > Vw_{req}$
∴**ADEQUATE**

In the design of the connection of base plate, section 5 of AS4600 fillet welding connection was used. With 5mm weld governing. The members with both column and beam resulted in being adequate.

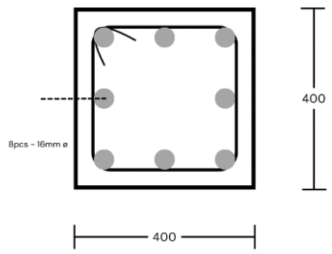
**FOUNDATION DESIGN**

DESIGN OF COLUMN (BELOW N.G.L)

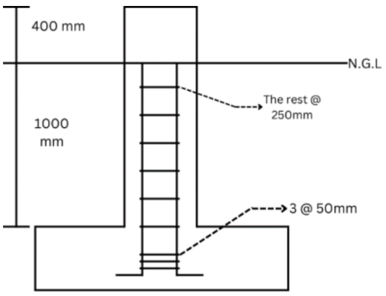
Design requirements:
f'c = 21 MPa
Fy = 345 MPa
Column size = 400mm x 400mm
16mmØ = 64π
Load Calculation
1.2DL + 1.6LL = 168 kN
Pu = 168 kN
Design Procedure:
$\emptyset Pn = 0.65(0.8)(0.85) f'c [Ag - A_{st}] + A_{st} fy$
$\emptyset Pn = 0.65(0.8)(0.85)(21)[400mm^2 - 64\pi mm^2] + [64\pi mm^2(345)]$
ØPn = 1552.62
∴**ADEQUATE**

Vertical Bars:

$$\rho g = \frac{A_{st}}{Ag} = \frac{8(64\pi\ mm^2)}{400mm \times 400mm} = 0.0101$$

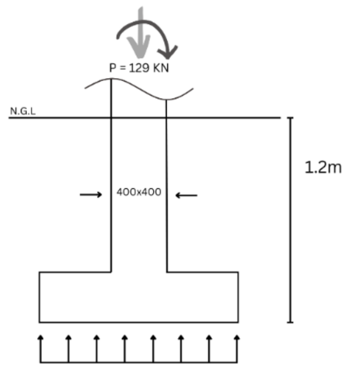



Lateral Ties:
$16 \times$ bar diameter $= 16(16) = 256mm$
$48 \times$ tie diameter $= 48(10) = 480mm$
Least diameter = 300mm =300mm
Spacing = 256 ≈ 250mm




**DESIGN OF FOUNDATION:**

COLUMN SECTION 400x 400mm
f'c =21MPa
Fy = 276 MPa
qe =90 KPa
$D_f$ =1.2m
$d_b$ =16mm Ø main bars
t = 300mm
$P = P_{DL} + P_{LL} = 129$ kN
$P_u = 1.2P_{DL} + 1.6P_{LL} = 168$ kN
$M = M_{DL} + M_{LL} = 0.619$ kN-m
λ =1.0




**Actual size of footing**

$$A_{act} = \frac{P}{qe} = \frac{129\ KN}{90Kpa} = 1.433m^2$$

$A_{act} = b^2$
$b = \sqrt{1.433m}$
$b = 1.197 \approx 1.2m$
∴ Footing Size $1.2m \times 1.2m$

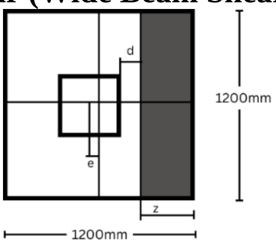
**Ultimate Soil Bearing Capacity**

$$qu = \frac{Pu}{A_{act}} = \frac{168\ kN}{1.2m \times 1.2m}$$
$$qu = 116.667\ KPa\ or\ 0.117\ MPa$$

**Eccentricity**

$$e = \frac{M}{P} = \frac{0.619}{129} = 0.005$$
$$d = 300 - 75 - 1.5(16)$$
$$d = 201\ mm$$

**One Way Shear (Wide Beam Shear)**




$V_u = q_u Z B$

$Z = 199.005mm$

$V_u = 116.667\ (0.199005)\ (17)$

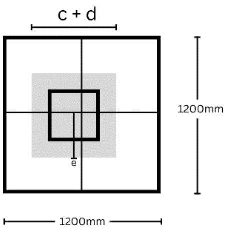
$V_u = 27.861kN$

$$\emptyset V_c = 0.75(0.17)\ (10)\sqrt{21}\,\frac{(1200)(201)}{1000}$$

$\emptyset V_c = 140.928\ kN$

$\emptyset V_c > V_u$

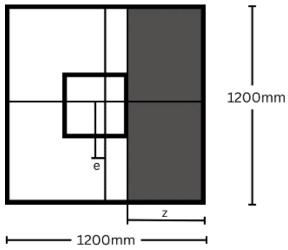
**Two Way Shear (Punching Shear)**




$$bo = 4(c + d) = 4(400 + 201) = 2404\ mm$$
$$Vu = qu[B^2 - (c + d)^2]$$
$$Vu = 116.667\ KPa[1.2m^2 - [0.601\ m]^2]$$
$$Vu = 125.860\ kN$$
$$\emptyset Vc = \emptyset 0.333\lambda\sqrt{f'c}bod$$
$$\emptyset Vc = 0.75(0.333)(1.0)\sqrt{21MPa}\,\frac{[4(601)](201)}{1000}$$
$$\emptyset Vc = 553.026\ kN$$

$\emptyset Vc > Vu$ ∴ **Adequate**

**Design of steel reinforcement (Based on Flexure)**




$$z = \frac{1.2}{2} + 0.005 - \frac{0.4}{2}$$
$$z = 0.405\ m$$
$$Mu = qu\,\frac{z^2}{2}B$$
$$Mu = 116.667KPa\left[\frac{(0.405m)^2}{2}\right][1.2m]$$
$$Mu = 11.482\ kN\text{-}m$$
$$Rn = \frac{Mu}{\emptyset bd^2}$$
$$Rn = \frac{11.482x10^6\ N-mm}{0.9(1200mm)(201mm)^2} = 0.263\ MPa$$
$$\rho_{act} = \frac{0.85f'c}{fy}\left[1 - \sqrt{1 - \frac{2Rn}{0.85f'c}}\right]$$
$$\rho_{act} = \frac{0.85(21)}{276}\left[1 - \sqrt{1 - \frac{2(0.263)}{0.85(21)}}\right]$$
$$\rho_{act} = 0.00096$$
$$\rho_{min} = \frac{1.4}{fy} = \frac{1.4}{276} = 0.00507$$

$\rho_{min}$ govern

∴ use $\rho = 0.00507$

$As = \rho bd$

$As = 0.00507\ (1200mm)\ (201mm)$

$As = 1222.884\ mm^2$

$$N = \frac{As}{Ab} = \frac{1222.884mm^2}{\frac{\pi}{4}(16)^2mm^2} = 6.08 \approx 7pcs - 16mm\emptyset$$
$$S = \frac{1200mm - 2(75mm) - 16mm}{6}$$
$$S = 172 \approx 170mm$$
$$S_{max} = 5t = 5(300mm) = 1500mm$$

∴ $S = 170mm$

∴ Provide 16mm Ø spaced @ 170mm o.c (bothways)

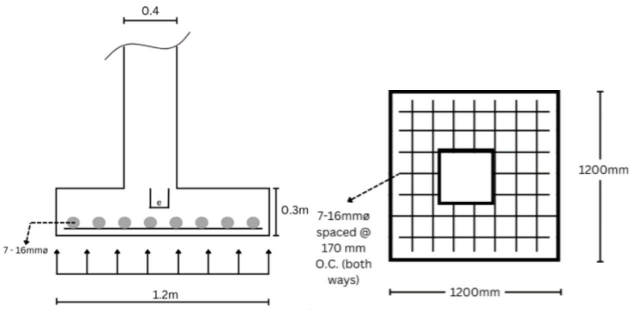


Fig 24. Foundation Specification (working drawing)

In the Foundation design, the actual size of the footing was 1.2x1.2m. Both one-way and two-way shear were adequate, with two-way shear (punching shear) governing.

### *F. COST ANALYSIS*

### BILL OF MATERIALS(BOM)

| BILL OF MATERIALS | | | | |
|---|---|---|---|---|
| **ITEM OF WORK** | **UNIT** | **QUANTITY** | **UNIT COST** | **MATERIAL COST** |
| **FOR FOOTING** | | | | |
| Sand | Cu.m | 5 | ₱ 2,000.00 | ₱ 10,000.00 |
| Gravel | Cu.m | 9 | ₱ 880.00 | ₱ 7,920.00 |
| Cement | Bag | 104 | ₱ 300.00 | ₱ 31,200.00 |
| 16 mm Diameter | kg | 300 | ₱ 60.00 | ₱ 18,000.00 |
| GI Wires | kg | 5 | ₱ 31.50 | ₱ 157.50 |
| | | | | **₱ 67,277.50** |
| **FOR COLUMN " PEDESTAL"** | | | | |
| Sand | Cu.m | 4 | ₱ 2,000.00 | ₱ 8,000.00 |
| Gravel | Cu.m | 7 | ₱ 880.00 | ₱ 6,160.00 |
| Cement | Bag | 81 | ₱ 300.00 | ₱ 24,300.00 |
| 16 mm Diameter | kg | 280 | ₱ 60.00 | ₱ 16,800.00 |
| 10 MM Diameter | kg | 140 | ₱ 50.00 | ₱ 7,000.00 |
| GI Wires | kg | 5 | ₱ 31.50 | ₱ 157.50 |
| | | | | **₱ 62,417.50** |
| **FORMWORKS** | | | | |
| PHENOLIC FLYWOOD | pcs | 9 | ₱ 880.00 | ₱ 7,920.00 |
| 2x2 Timber | pcs | 20 | ₱ 240.00 | ₱ 4,800.00 |
| WALL | kg | 2 | ₱ 100.00 | ₱ 200.00 |
| SCAFFOLDING "RENT" | kg | 10 | ₱ 25.00 | ₱ 250.00 |
| | | | | **₱ 13,170.00** |
| **PEDESTAL CONNECTION** | | | | |
| BASEPLATE | pcs | 30 | ₱ 2,090.00 | ₱ 62,700.00 |
| BOLTS | pcs | 270 | ₱ 300.00 | ₱ 81,000.00 |
| | | | | **₱ 143,700.00** |
| **SLAB** | | | | |
| STEEL DECK | pcs | 33 | ₱ 970.00 | ₱ 32,010.00 |
| SAND | cu.m | 8 | ₱ 2,000.00 | ₱ 16,000.00 |
| GRAVEL | cu.m | 16 | ₱ 880.00 | ₱ 14,080.00 |
| CEMENT | Bags | 186 | ₱ 300.00 | ₱ 55,800.00 |
| SHEAR STUDS | pcs | 3000 | ₱ 0.56 | ₱ 1,680.00 |
| | | | | **₱ 119,570.00** |
| **MEMBERS** | | | | |
| 200x200X9 | Ton | 9.308 | ₱ 39,000.00 | ₱ 363,012.00 |
| 200x100X9 | Ton | 5.555 | ₱ 35,000.00 | ₱ 194,425.00 |
| 150x100X6 | Ton | 5.127 | ₱ 33,000.00 | ₱ 169,191.00 |
| | | | | **₱ 726,628.00** |
| **TOTAL** | | | | **₱ 1,132,763.00** |

The provided Bill of Materials **(BOM)** details the quantities, unit costs, and total material costs for various construction components involved in a project. The breakdown includes items for footing, columns/pedestals, formworks, pedestal connections, slabs, and members. Each section lists specific materials such as sand, gravel, cement, steel, and plywood along with their respective units (e.g., cubic meters, bags, pieces, tons), quantities, unit costs in Philippine Pesos (₱), and the resultant material costs. For example, the footing requires 5 cubic meters of sand at ₱2,000 per cubic meter, totaling ₱10,000.

The columns/pedestals similarly require materials like sand, gravel, and different diameters of steel wires. Formworks include phenolic plywood, timber, and scaffolding rentals, whereas pedestal connections list base plates and bolts. Slabs necessitate steel decks, sand, gravel, cement, and shear studs. Finally, various members are detailed with their respective sizes and costs. The total cost for all these materials amounts to **₱1,132,763.00**. This comprehensive BOM ensures that all necessary materials are accounted for, facilitating efficient planning and budgeting for the construction project.

## VIII. SUMMARY CONCLUSIONS AND RECOMMENDATIONS

### *A. SUMMARY OF FINDINGS*

In the study conducted, the researchers used cold-formed steel as a mainframe. It uses different sizes of rectangular and square hollow sections for structural members and welding for the connection. They used the architectural plan to build the structural plan using Staad Pro Connect. For the loadings applied, they use the NSCP 2015 for the design of dead load, live load, wind load, and seismic load. After that, they started the design of purlins and trusses. the Australian Standard 4600 Direct Strength Method (AS4600 Section 7) was used for the design of structure and pedestal.

They calculated the allowable capacity of the members in compression, bending, and shear strength. It also got a less than 1.0 for the actual/allowable ratio, and it matched the result in Staad Pro Connect. After the design of the structural member, the connection follows. The calculations of the researchers support the assumed 5 mm-thick weld which was solved as adequate.

This proves the hypothesis that using Cold-formed steel as a mainframe for low-rise residential buildings can be safe structurally and the foreign code AS4600 proved to be the best possible structural code to use in designing cold-formed steel structures in Asia, specifically Philippines. Lastly, the researchers conducted a cost analysis and made a bill of materials to sum up the structural costs.

### *B. CONCLUSIONS*

The goal of the study is to analyze and design a two-storey residential structure with cold-formed steel as the main framing system by applying load using NSCP 2015 and the provisions of AS 4600, as well as the cost-effectiveness of using cold-formed steel as the main construction material.

Based on the findings, the conclusions below can be considered:

- The study proves that the design of the structural properties and connections using foreign standards with loadings from the local code NSCP 2015 is adequate because the allowable strength (compression, bending, and shear) of the structural members that are calculated using Australian Standard 4600, is greater than the actual strength of the member itself.

- Cold-formed steel, known for its lightweight yet strong properties, offers a fascinating alternative for construction materials in the Philippines. By carefully designing and analyzing structures, it has been demonstrated that the load combinations specified by the National Structural Code of the Philippines (NSCP) can be effectively used with cold-formed steel. This opens up new possibilities for more efficient and sustainable construction practices. Imagine the potential for innovative architectural designs and quicker construction times, all while maintaining structural integrity and safety.

- Based on the cost analysis conducted, the materials used in the two-storey cold-formed steel structure have a total cost of ₱1,132,763 and it is proven that this structure is cost-effective.

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