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SIMPLIFIED DESIGN METHODS FOR PREQUALIFIED SEISMIC MOMENT CONNECTIONS USED IN A STEEL FRAMED STRUCTURE

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ABSTRACT

Extended end plate moment connections consist of a plate welded to the end of a beam that is used to splice another beam or used to bolt to the flange of a column in steel structures. These connections show different behaviors due to the influence of plate thicknesses, gauge distances, and diameters of high strength bolts. The second edition of AISC Design Guide 4 provides design procedures and examples for the evaluation of stiffness and strength of the three prequalified connections: 4E, 4ES and 8ES. The Guide recommends the use of a thick plate with small diameter bolts which leads to no prying forces. The yield line calculations of column and beam flanges can therefore subsequently be carried out. To the casual observer the selection of bolts for prequalified connections should be simple. In reality this is a difficult task. Designers must minimize time while maintaining a good selection of bolt sizes to be used with the connections. Geometrical details of the elements such as beams, columns, or end plates must be carefully selected while keeping adequate shear and moment strengths. The aim of this paper is to provide simplified design charts that offer a rapid means of selection of bolt diameter and identification of hot-rolled steel sections of the connected elements as well as selection of the geometrical features to be assigned to the connections in order to satisfy the minimum performance requirements as required by the Design Guide when subjected to wind and seismic loads. An example case study is used to demonstrate the use of the design charts.

KEYWORDS: Seismic Moment Connection; AISC Design Guide 4; High strength Bolts

1. Introduction

The AISC Design Guide 4 provides a computational procedure for designing extended end plate moment connections. Procedures presented in the guide were verified through experimental tests. Sumner and Murray [1] conducted tests including both bare steel connections in exterior sub-assemblages as well as composite connections in interior sub-assemblages. The test report shows that seismic failure was controlled by a more favourable beam flange and web buckling failure rather than the column failure. Hence, the authors recommend that connections be designed to be stronger than the beam to ensure that seismic failure would occur in the beam rather than column failure. Roeder [2] also conducted experiments to show the good seismic performance of the extended moment connections during earthquakes. The study showed analytically and experimentally that the plastic rotation capacity could be expressed as a function of beam depth. When the frame structure presents this kind of connection, structural damping increases, resulting in a corresponding change in structural motion and thus mitigates the earthquake damage. The connections were able to develop the full plastic capacity of the beam and large rotations. Shi *et al* [3] performed full-scale tests of connections under cyclic loading to investigate the effect of geometrical parameters: end-plate thickness, bolt diameter, end-plate and column stiffener. The results led to a bilinear kinematic hardening hysteretic moment-rotation model. These experimental results served as the basis for the development of nonlinear finite element models [4-6] in order to provide design formulations and to provide greater understanding of the joint responses.

In an attempt to ensure satisfactory seismic performance, this study outlines a computational procedure for designing bolted unstiffened and stiffened extended end-plate moment connections in SI units. Governing equations and failure modes are outlined in a subsequent section. It then presents the development of unified design charts for three fully restrained connections used by experienced engineers. The choice of control chart parameters is considered from an economical point of view. An example case study is presented to demonstrate the process of using the charts for finding bolt diameter for bolted-flange-plated connections with 4 bolts in a steel frame structure.

2. Governing Equations

During a seismic event, plastic effects can be generated through a plastic hinge formed in the beam. This section outlines the governing equations for the design of bolted end-plate moment connections subjected to cyclic/seismic forces. The required connection moment M_{uc} at the face of the column as stated in AISC Design Guide 4 is the sum of the moment M_{pe} and the additional moment caused by the eccentricity of the shear V_u presented at the plastic hinge L_p given by Equation (1), assuming the plastic hinge is located as shown in Figure 1 [7].

$$M_{uc} = M_{pe} + V_u L_p \tag{1}$$

As per ANSI/AISC 341-10, fully restrained moment connections designed using LRFD must be proportioned to resist 1.1 times expected flexural strength of the beam. The expected flexural strength M_{pe} is defined as $1.1R_y M_p$ where the constant R_y is the ratio of expected yield stress to the minimum specified yield stress (equal to 1.5 for F_y = 248 MPa and 1.1 for F_y = 345 MPa) and M_p is the flexural strength of the beam at the full plastic capacity, that is, $M_p = F_y Z_x$.

For stiffened and unstiffened connections, the design approach is consistent with the traditional strong-column-weak-beam approach for seismic design. The distance from column face to plastic hinge L_p is given by the following expressions

Unstiffened connections: $$L_p = \min(0.5d, 3b_f) \quad (2)$$

Stiffened connections: $$L_p = L_{st} + t_p \quad (3)$$

where d is the depth of the connecting beam, b_f is the beam flange width, L_{st} is the length of the end plate stiffener, and t_p is the thickness of the end plate.

When one of the prequalified end-plate moment connection configurations as shown in Figure 2 is selected, the required bolt size will be determined from the following equations

For 4E, 4Es: $$d_b^{required} = \sqrt{\frac{2M_{uc}}{\pi \cdot \emptyset \cdot F_t(h_0 + h_1)}} \quad (4)$$

For 8Es: $$d_b^{required} = \sqrt{\frac{2M_{uc}}{\pi \cdot \emptyset \cdot F_t(h_1 + h_2 + h_3 + h_4)}} \quad (5)$$

These equations are calculated assuming the bolts on the tension side of the beam resist all tension forces arising from the moment at the face of the column, M_{uc} and the bolts are not subjected to prying forces. The value of $\emptyset$ is described in the Guide as 0.75. F_t is the specified LRFD bolt tensile strength, h_i is the distance from bolt i to the centroid of the beam compression flange.

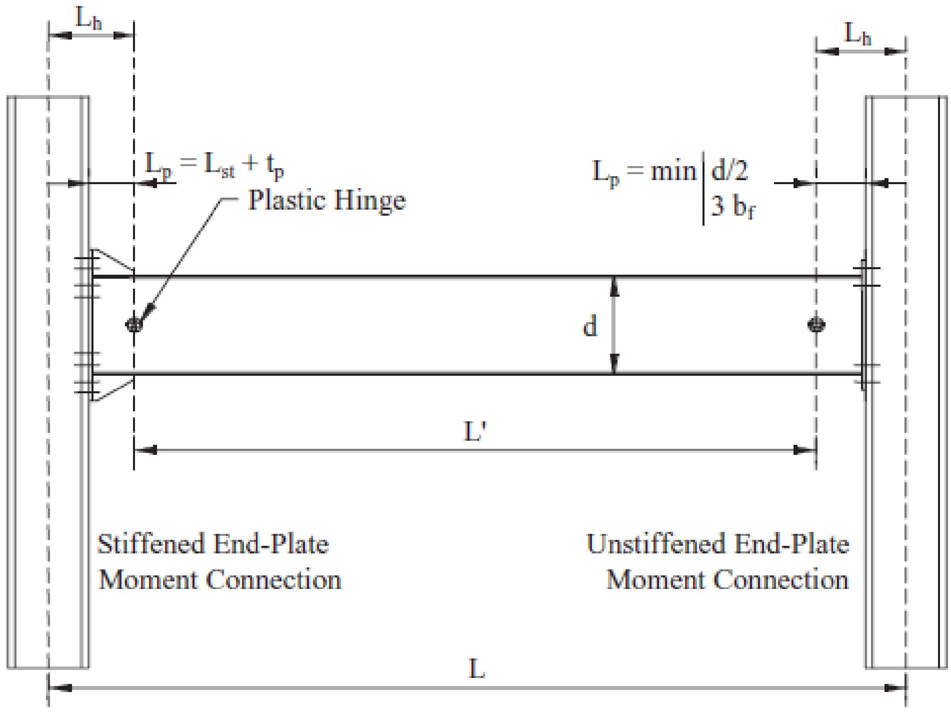


Figure 1 Plastic hinge location for stiffened and unstiffened extended end plate moment connections [7].

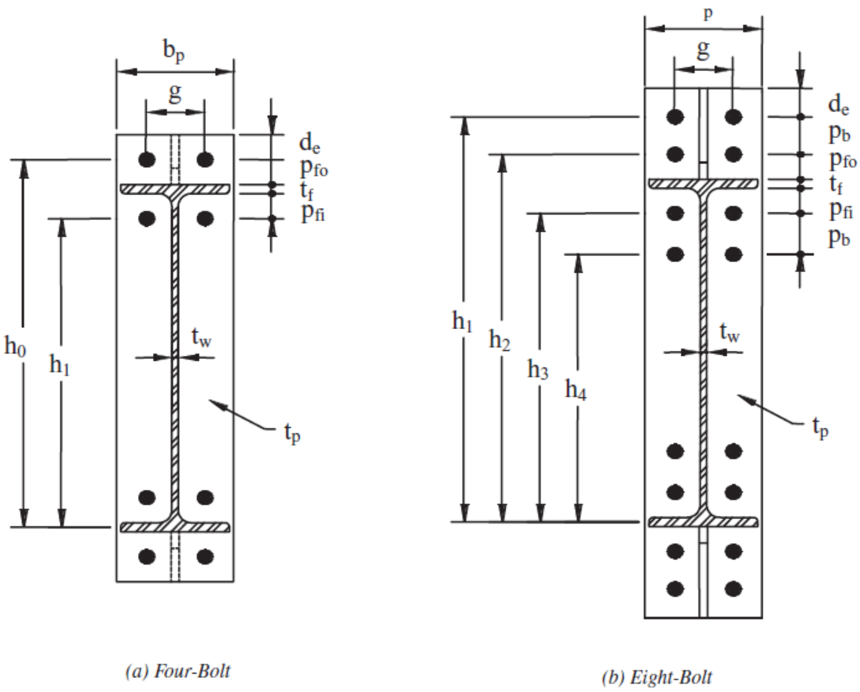


Figure 2 Four bolt and eight bolt dimension and distances used in bolt size design equations [7].

3. Development of Charts

The purpose of this study is to give practicing engineers some way of reducing the design time required for smaller projects, while still complying with the letter and the intent of the AISC Design Guide 4. The simplification of design with its attendant savings in design time result from avoiding member proportional details and bolt selections which make it necessary to consider certain complicated provisions of the Design Guide.

To develop simplified charts for bolt selections, a database of hot-rolled steel metric W-sections (ASTM A572 Gr. 50) available in Thailand along with ASTM A490 bolts is used [8]. The process begins by selecting the pre-designed w-shaped beam sections from the database. The next step is to select bolt sizes and specify the dimensions shown in Figure 2 including bolt gauge distance (g), bolt pitches (p_b and p_f) and end plate edge distance (d_e). The process continues by selecting one of the end plate connection types namely: the 4-bolt connection without plate stiffener (4E), the 4-bolt with plate stiffener (4ES) or the 8-bolt with plate stiffener (8ES). The next step is to run the calculations using a 12-step procedure for end plate and bolt design as appears in the Guide [8]. In this study, the calculations are done in excel spreadsheets [9]. The usual calculations require users to specify the dimensions that appear in Figure 2, including the bolt gauge distance, bolt pitches and end plate edge distance for each specified bolt diameter. The Guide recommends an appropriate end plate width to match the width of the beam flange (i.e., the end plate width is the beam flange width plus 1 in. or 25 mm. And the minimum required thickness can be computed using the equation provided in the Guide. In this study, the spreadsheets calculate the end plate stiffener properties as well as all height dimensions for the connections without lengthy explanations. The full moment and shear capacities will then be evaluated and checked with failure modes related to the panel zone.

Figure 3 through Figure 5 give example plots of maximum nominal factored shear and maximum nominal factored moment, checked for compliance with all bolt sizes and W-200 and W-400 steel beam sections for design convenience of 4E, 4ES and 8ES connections, respectively. The shapes are designated by the mark W200 or W400, nominal depth (mm), and nominal weight per length (kg/m). For example, aW200x21.3 is a W-shape beam that is nominally 200 mm deep and weigh 21.3 kg/m. For all other W sections, the design charts can be found in the undergraduate civil engineering senior project report [9] available at the Department of Civil and Environmental Engineering at Mahidol University.

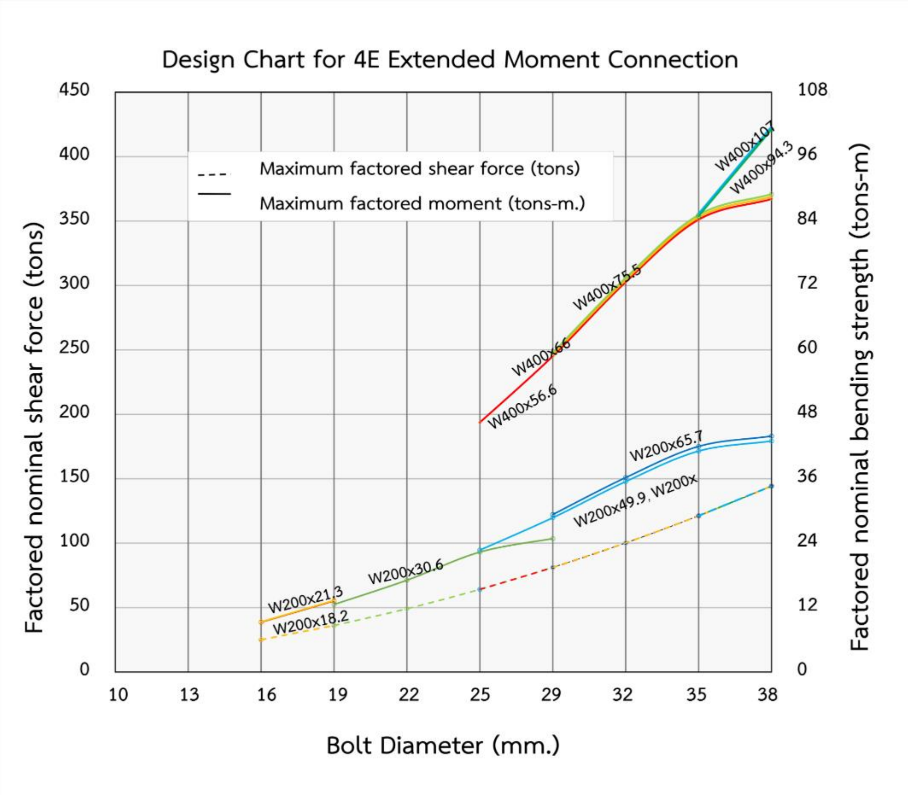


Figure 3 A simplified chart example of bolt dimensions and W-200 and W-400 beams used for 4 bolt unstiffened

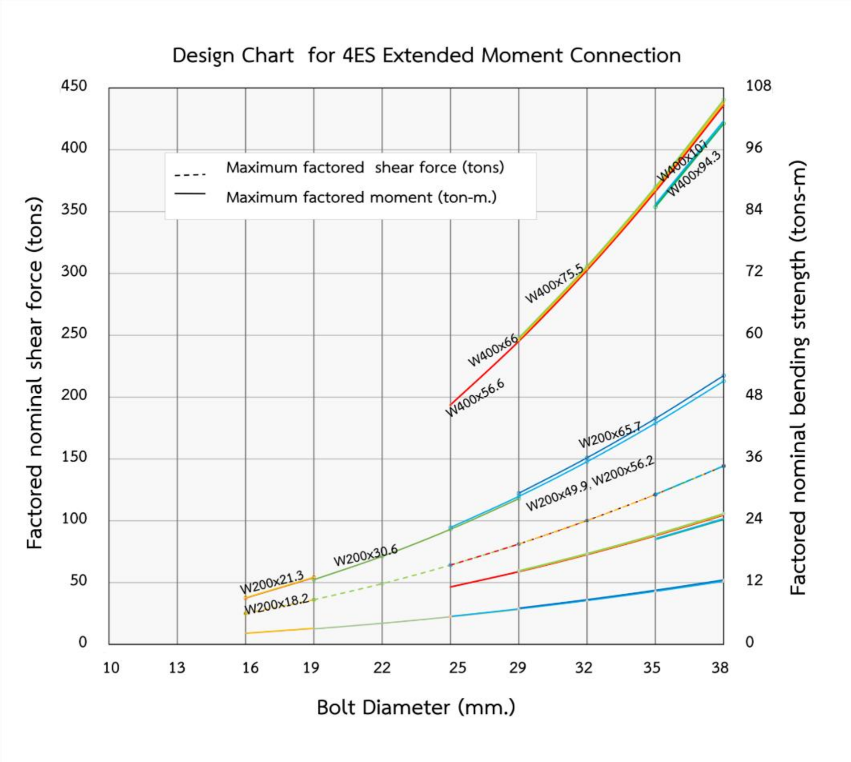


Figure 4 A simplified chart example of bolt dimensions and W-200 and W-400 beams used for 4 bolt stiffened

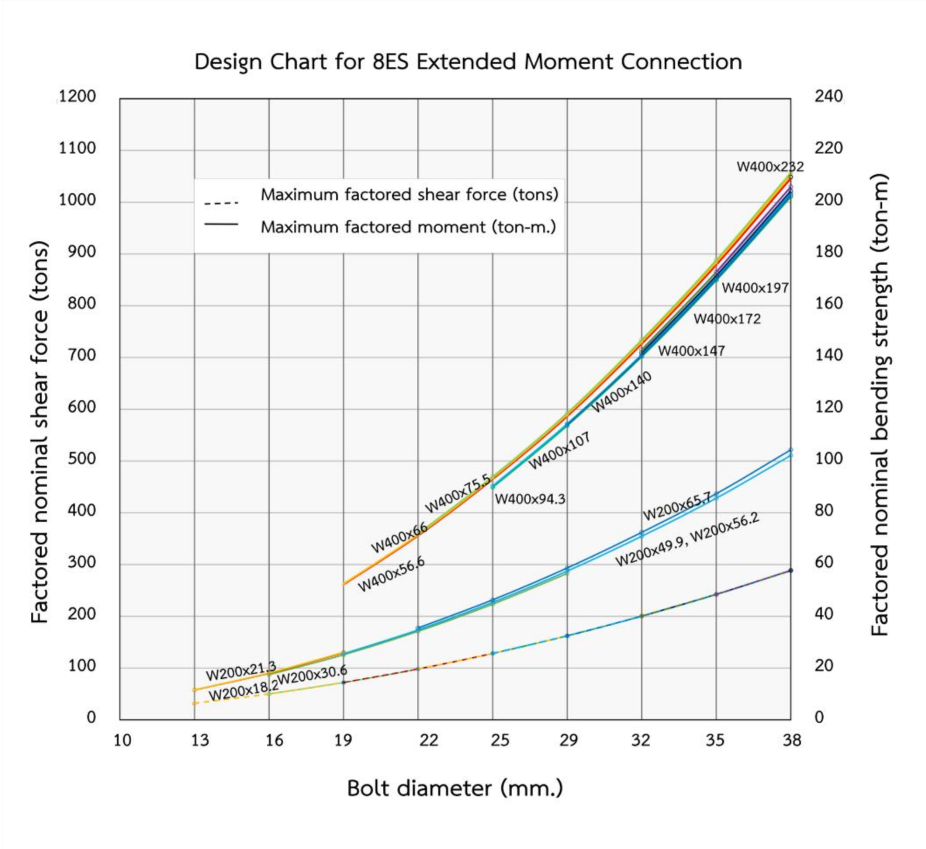


Figure 5 A simplified chart example of bolt dimensions and W-200 and W-400 beams used for 8 bolt stiffened

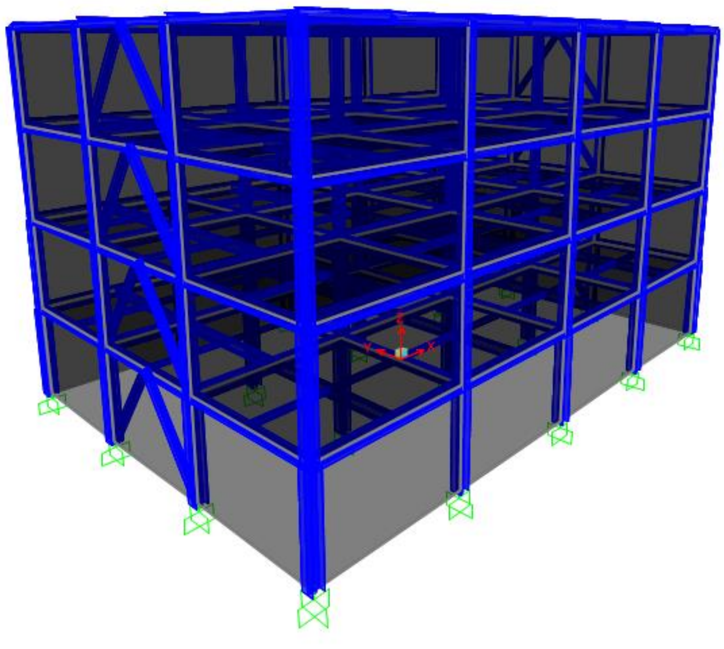


Figure 6 Typical 3D view of a 4-storey case study steel structure

In order to demonstrate the usefulness of the charts discussed above, the following section presents a case study in which a steel structure is designed using AISC LRFD.

4. Example Case Study

A 4-storey steel building was analyzed and designed in accordance with AISC LRFD specifications. Figure 6 illustrates the 3D view of the case study building. The typical floors are 3 meters high. Gravity loads include 480 kg/m^2 for the load of slab, superimposed dead load in addition to 30 kg/m^2, and 350 kg/m^2 live load. Self-weight of the structure was computed using SAP2000. Wind velocity is assumed to be 29 m/s in exposure category A. The building is assumed to be classified as risk category 1 situated on class D soil. The seismic design parameters; R = 4.5, Ω_0 = 3, and C_d = 4, are identified by assuming the overall structural system as an intermediate steel moment resisting frame (IMF). According to DPT 1301/1302-61, the building is assigned to seismic design category D, the level that requires the most rigorous design, which permits the use of IMF. The design analysis from SAP2000 shows that the perimeter IMF forces are controlled by the load combination of 1.2DL + 1.0E$_x$ + LL.

Figure 7 shows bending moments and shear forces resulting from loading analyses in SAP2000 for the case study steel building. As was the case for the beam-to-column connection at story 2, the maximum negative bending moment of 31,295 kg-m and the maximum shear of 14,804 kg were used to optimize the design of extended moment connection and select appropriate bolt sizes.

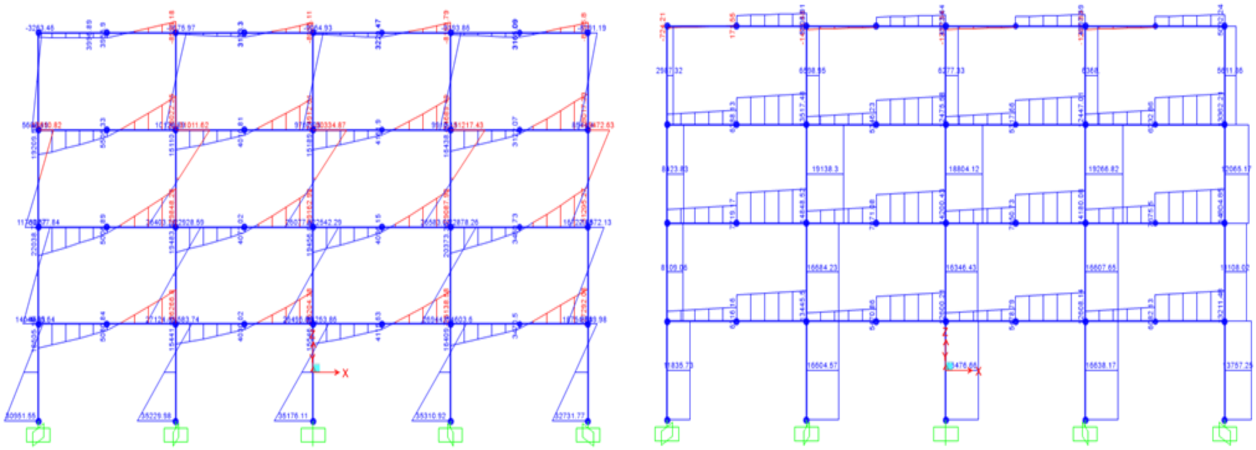


Figure 7 Bending moments and shear forces resulting from loading analyses for a case 4-storey building

According to the structural analysis, the SAP2000 selected W400 x 66 (H - 400 x 200 x 8 x 13 mm.) as the most appropriate section (hot-rolled structural steel sections available in Thailand) for maximum bending moment and shear force previously mentioned. At this stage, the proper design chart developed above will be used to select the appropriate bolt size, end-plate thickness, and geometry of connection respectively. The instructions for the use of a design chart are as follows.

1. Select the design chart of 4E configuration that illustrates the specific beam size obtained from the stage of structural design.

2. Start plotting a horizontal line using the required bending moment from the secondary y-axis (factored nominal bending strength) to the left-hand side until it converges with the solid line indicating the name of the specific beam size above. Then draw a line down to the x-axis (bolt diameter) to select the appropriate bolt diameter used for the connection. Remark: In the case that the horizontal line cannot converge with the solid line, structural designers should draw the horizontal line to the starting point of that specific solid line, then draw the line up and down to select the appropriate bolt diameter used for the connection)
3. Plot the horizontal line using the required shear force from the primary y-axis (factored nominal shear strength) to the right-hand side until it converges with the dashed line. Then draw a line down to the x-axis (bolt diameter) to select the appropriate bolt diameter used for the connection.
   Remark: In the case that the horizontal line cannot converge with the dashed line, structural designers should draw the horizontal line to the vertical line drawn in Step 2, then select the appropriate bolt diameter used for the connection

Detailed instructions for bolt selections using the simplified chart are shown in Figure 8. It should be noted that this is the minimum bolt size required by Design Guide 4. After obtaining the appropriate bolt diameter, the end-plate thickness (t_p), the end plate width (b_p), the end plate height (h_p) and geometry of the connection can be automatically selected from Tables 1 and 2. The completed information of the end plate details corresponding to all other design charts can be found in the undergraduate civil engineering senior project report [9].

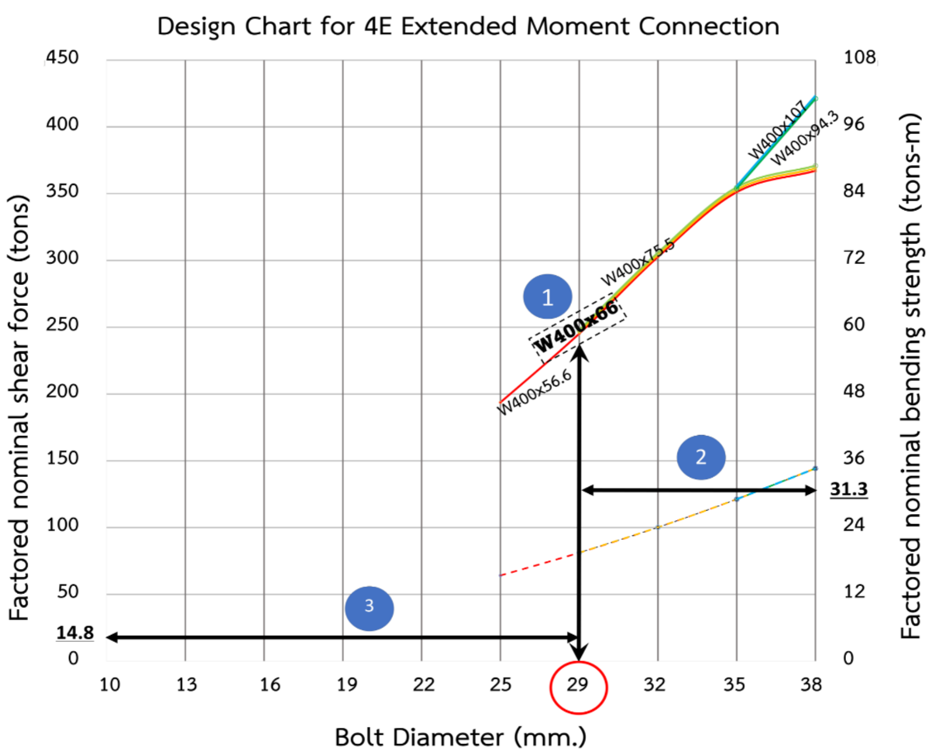
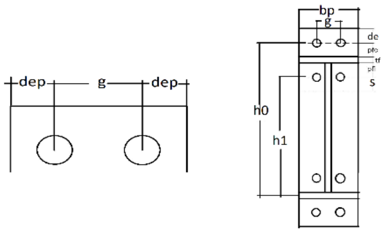


Figure 8 The utility of chart for 4 bolt unstiffened connection design for a 4-storey case study steel structure

Table 1 Factored shear and moment capacity of 4 bolt unstiffened connection with bolt diameters along with w-400 beam and end-plate thickness alternatives

Beam W -Section	**Bolt Diameter (mm)**	**13**	**16**	**19**	**22**	**25**	**29**	**32**	**35**
W400x56.6	End-Plate thickness (t_p,mm)					25	29	32	35
	Shear (kgf)					62,125	81,158	100,195	121,236
	Moment (kgf-m)					46,491	58,840	72,642	84,273
W400x66	End-Plate thickness (mm)						29	32	35
	Shear (kgf)						81,158	100,195	121,236
	Moment (kgf-m)						59,146	73,020	84,711
W400x75.5	End-Plate thickness (mm)						29	32	35
	Shear (kgf)						81,158	100,195	121,236
	Moment (kgf-m)						59,451	73,397	84,711
W400x94.3	End-Plate thickness (mm)								35
	Shear (kgf)								121,236
	Moment (kgf-m)								84,711

Table 2 Prudent choice of geometric properties of four bolt unstiffened extended end-plate connection



Beam W Section	d_b (mm.)	b_p (cm.)	g (cm.)	d_{ep} (cm.)	h_p (cm.)	p_{fo} (cm.)	p_{fi} (cm.)	d_e (cm.)	h_0 (cm.)	h_1 (cm.)	Y_p (cm.)	L_p (cm.)
W400x56.6	25	22.5	7.5	7.5	60	5	5	5	44.1	32.9	325.2	19.8
	29	22.5	12.5	5	60	5	5	5	44.1	32.9	313.8	19.8
	32	22.5	12.5	5	60	5	5	5	44.1	32.9	304.4	19.8
	35	22.5	12.5	5	65	5	5	7.5	44.1	32.9	296.5	19.8
	38	22.5	12.5	5	65	5	5	7.5	44.1	32.9	289.7	19.8
W400x66	29	22.5	12.5	5	60	5	5	5	44.4	33.0	315.2	20.0
	32	22.5	12.5	5	60	5	5	5	44.4	33.0	305.8	20.0
	35	22.5	12.5	5	65	5	5	7.5	44.4	33.0	297.8	20.0
	38	22.5	12.5	5	65	5	5	7.5	44.4	33.0	291.0	20.0

4.1 Observations

1) The bolt diameter and end-plate thickness obtained from the use of a design chart are slightly large. This is a result of the design philosophy of AISC Design Guide No. 4, intending to have a strong column, strong connection, and weak beam. Therefore, the connection is designed using the required bending moment obtained from the summation of plastic bending strength of the beam and extra moment resulting from shear force at the location of the plastic hinge. This required bending moment used to design the appropriate bolt diameter and end-plate thickness is large, preventing the most significant component, the connection, from failure first which could be a cause of a building collapse leading to loss of life and property damage.

2) The configuration of a four-bolt unstiffened end-plate connection (4E) will always be adequate to resist the maximum bending moment that occurs in connecting beams with dept h less than or equal to 600 mm. On the other hand, it will be necessary to select eight bolt stiffened configuration (8ES) as the most suitable connection as the minimum requirement for connecting beams with depth greater than or equal to 700 mm. This is due to the fact that the connection strength is entirely governed by bolt tension rupture. Therefore, the use of ASTM A490-N bolt is permitted and sufficient for the above use.

3) The limitation of these design charts is that they cannot be used for any other connecting beams with different dimensional sections for instance the built-up beams used for the construction of PEB warehouses. This stems from the fact that the end-plate moment connection is designed according to AISC Design Guide No. 4 which is the prequalified connection. Accordingly, the strength of the connection will be based on the arrangement of every component of the connection, including the geometry of connecting beams.

5. Conclusions

Extended bolted-flange-plate moment connections are attractive due to the ease of welding the end-plate to the beam in the fabrication shop. The beam can then be transported and be field bolted in situ to make beam to column joints exhibit better deformation capacity and ductility when subject to both wind and seismic loads. The design charts are developed to assist engineers select the most appropriate connections using plate geometries and bolt sizes that meet the minimum requirements of AISC Design Guide 4 for the seismic design of bolted end plate moment connections. The charts can be easily used by drawing the line of required shear across the factored nominal shear strength axis and, similarly, drawing the line of required bending moment across the factored nominal flexural strength axis.

Conflict of interest

The authors declared that this article has no conflict of interest.

Acknowledgement

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