

PARAMETRIC ANALYSIS AND DESIGN OF TAPERED HAUNCHED CONNECTIONS

E. B. Machaly¹, S. S. Safar² and E. A. Ettaf³

ABSTRACT:

Tapered-haunched connections are widely used in medium-to-large span steel frames. A simple, yet accurate, method for dimensioning the different components of these connections is required. For this sake, an extensive parametric study was conducted herein using the general purpose finite element program ANSYS to assess quantitatively the different parameters that affect the strength of tapered knees. Effective geometric parameters of the connections included in the parametric study were set based on a theoretical investigation of possible modes of failure including: web buckling at corner re-entrant and at haunch-to-beam junction, lateral buckling of haunch flange, shear instability of web panel zone, web yielding at corner re-entrant and at haunch-to-beam junction, and web yielding at panel zone. The effective parameters including: width-to-thickness ratio of web and flange, haunch length-to-flange width ratio, slope of taper and stiffeners configurations were varied within practical limits and the connection moment capacity and failure mode were computed for each configuration. Finite element results were fed into a regression analysis from which design equations were established. The proposed design equations are simple to use to determine the moment capacity of such connections incorporating the integrated effect of all possible modes of failure. Recommendations for stiffeners proportioning were also set forward.

KEYWORDS: Tapered connection, Parametric analysis, Finite element analysis, Non-linear analysis, Material non-linearity, Regression analysis.

-
1. Professor of Steel Structures and Bridges, Faculty of Engineering, Cairo University.
 2. Associate Professor of Steel Structures and Bridges, Faculty of Engineering, Cairo University.
 3. Teaching Assistant, Structural Engineering Department, Faculty of Engineering, Cairo University.

1. INTRODUCTION:

Although tapered-haunched connections are widely used in medium-to-large span steel frames, little was mentioned in literature or codes of practice about their behavior and design methods. The conventional design approach [1, 2, 3, 4] was based on applying equilibrium equations of the theory of elasticity in its original format thus led to complicated set of equations that are rather cumbersome and impractical for every day design. On the other hand, they lack a direct method for determining the moment capacity of the connection [2].

The finite element analysis of a typical tapered-haunched connection proportioned by the conventional method was established by the same authors in a previous research work [5] to assess the connection behavior including: moment capacity, type and location of failure and distribution of equivalent stresses at failure. Based on stiffeners configuration, three classes of connections were designated as: un-stiffened, semi-stiffened and fully stiffened connections. It was shown that the moment capacity of fully-stiffened connections reached the beam plastic moment and failure took place by yielding of web plate at haunch tip. On the other hand, un-stiffened and semi-stiffened connections supported only a portion of the beam plastic moment and failure took place by instability of web plate at corner re-entrant or haunch tips. It was also indicated that the conventional approach overestimates the developed forces in stiffeners by discarding the web plate contribution in supporting un-balanced flange forces at haunch tips and corner re-entrant.

In this work, an extensive parametric study was conducted considering effective geometric parameters on connection performance. The analysis of the connection was conducted using the finite element program, ANSYS [6]. In light of the analysis of the connection conducted in a previous research work [5], a simple mathematical formulation of possible failure modes was established herein to determine the effective geometric parameters on the connection carrying capacity. The effect of each parameter on moment capacity

and failure mode of the connection was investigated by varying its value independently within practical limits whereas other independent parameters were kept unchanged. Results of the parametric analysis were fed into a regression analysis program to establish a set of design equations that can be used to assess the strength of tapered connections corresponding to different stiffeners configurations.

2. PARAMETRIC STUDY:

The objective of the parametric study was to identify the parameters affecting the strength of the tapered knee (see Fig. 1), and to assess quantitatively their impact on the connection moment capacity and failure modes.

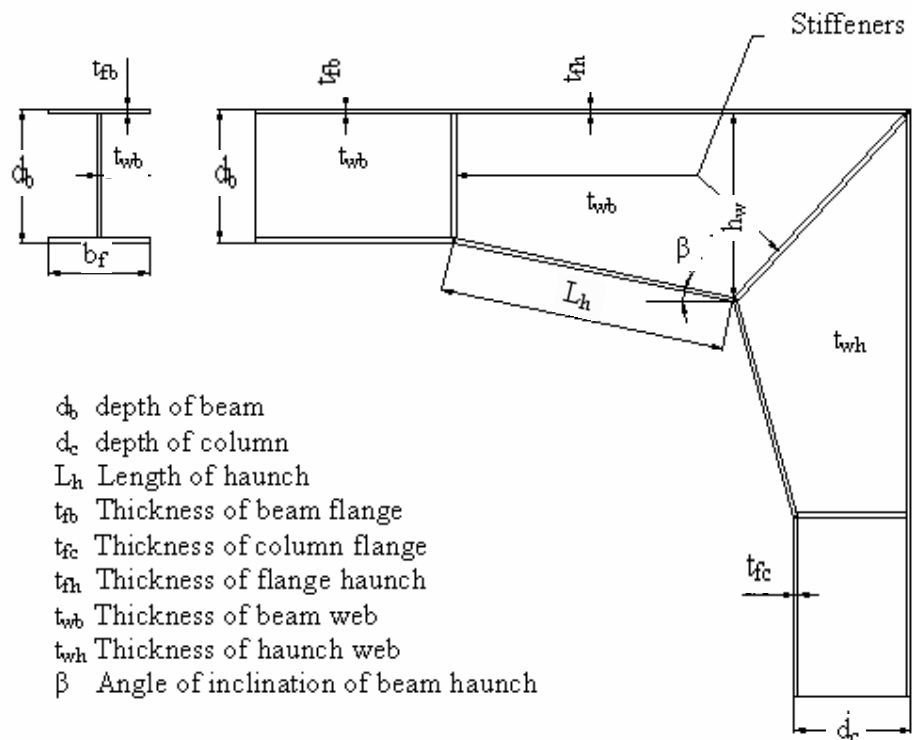


Fig. 1. Geometric configuration of a typical tapered haunched connection

2.1 Theoretical Investigation of Failure Modes:

Based on the finite element analysis of a tapered knee [5], it was concluded that failure took place by buckling or yielding of the haunch web plate due to the effect of un-balanced flange forces at haunch tips or corner re-entrant. On the other hand, failure may occur by lateral buckling of compression flange

especially for un-stiffened connections [5]. Therefore, a simple theoretical investigation was established herein to compute the connection capacity corresponding to each failure mechanism to identify the effective geometric parameters. The connection capacity was expressed by the value of shear force, Q_{lim} , at the girder inflection point assuming a linear moment diagram. The theoretical investigation was based on assuming elastic material behavior and applied only on the case of un-stiffened connections for simplicity.

2.1.1 Elastic buckling of web at corner re-entrant:

Assuming that the moment at corner re-entrant is only supported by the flanges and equating the un-balanced flange forces component to the elastic buckling load of a plate loaded by a concentrated load on its edge [7], an approximate expression of the shear force at inflection point was determined as follows:

$$Q_{lim} = \pi^2 E t_{wh}^3 / \{3(1-\nu^2) (a + L_h \cos \beta) (1-\tan \beta)\} \quad (1)$$

Where a : Distance between the inflection point and the haunch tip

E : Elastic young's modulus of the web material.

ν : Poisson's ratio.

2.1.2 Elastic buckling of web at haunch tip:

The shear force, Q_{lim} , at the inflection point can also be computed based on the elastic buckling of the haunch web plate at haunch tip utilizing the same approach outlined in Sec. 2.1.1 to give the following expression:

$$Q_{lim} = \pi^2 E t_{wh}^3 / \{3(1-\nu^2) a \tan \beta\} \quad (2)$$

2.1.3 Shear instability of panel zone:

Equating the un-balanced flange forces component in the vertical direction to the elastic buckling load of simply supported plates subjected to shear loading on the edges [7], an expression for Q_{lim} at the inflection point can be written as follows:

$$Q_{lim} = k_q \pi^2 E t_{wh}^3 / \{12 (1-\nu^2) (1-\tan \beta) (a + L_h \cos \beta)\} \quad (3)$$

Where k_q : Shear buckling coefficient.

2.1.4 Lateral buckling of haunch flange:

Equating the force developed in haunch compression flange at corner re-entrant to the out-of-plane flexural buckling load of the flange behaving as a column, an approximate expression of shear load, Q_{lim} , for such failure mode can be expressed as follows:

$$Q_{lim} = \pi^2 E b_f t_{fh} h_w \cos \beta / \{12 (kL_h / b_f)^2 (a + L_h \cos \beta)\} \quad (4)$$

Where k : Buckling length coefficient of flange acting as a column.

2.1.5 Yielding of web under the effect of concentrated loads:

In order to determine the value of Q_{lim} corresponding to local web yielding at corner re-entrant, the component of un-balanced flange forces normal to web was equated to the load causing yielding at a portion of the web depth of Ch_w [8]. The value of Q_{lim} obtained was as follows:

$$Q_{lim} = (C t_{wh}^2 \pi \sigma_y h_w) / \{2\sqrt{2} (1 - \tan \beta) (a + L_h \cos \beta)\} \quad (5)$$

Where σ_y is the material yield stress. Similarly, the value of Q_{lim} can be determined based on yielding of a portion of the web depth of Cd_b at haunch tip as follows:

$$Q_{lim} = 1/2 (C t_{wh}^2 \pi \sigma_y d_b) / \{a \tan \beta\} \quad (6)$$

2.1.6 Yielding of web in panel zone due to excessive shearing:

The shear force Q_{lim} that causes web yielding in the panel zone was computed by equating the vertical component of the un-balanced flange forces at corner re-entrant to the yield load of the web. The expression obtained for Q_{lim} was expressed as follows:

$$Q_{lim} = (\sigma_y / \sqrt{3}) h_w^2 t_{wh} / \{(a + L_h \cos \beta) (1 - \tan \beta)\} \quad (7)$$

It should be noted that the connection capacity is probably limited by a single or combination of the failure modes listed above based on the connection geometric configuration. The purpose of the theoretical investigation listed above was to indicate qualitatively the effect of connection parameters on capacity. The values of Q_{lim} computed by equations 1 to 7 are indeed approximate since the effect of

geometric imperfections and material non-linearities [7] were discarded and contribution of web in supporting the applied moment was neglected.

2.2 Parameters Selection:

Based on the above discussion, it was concluded that the parameter: h_w , d_b , L_h , b_f , t_{wh} , t_f and β included in the expressions of Q_{lim} had major effect on the connection capacity. They were expressed in a non-dimensional format in the parametric study as follows: h_w / t_w , L_h / b_f , β and $b_f / 2t_f$. Stiffeners configuration, length, L_s , and thickness, t_s , were also included to examine their effect on connection performance [5]. The selected parameters were varied to cover a significant part of practical range of dimensions that are usually encountered by designers. Hence, the obtained results will be trusted for the range of values studied (see Table 1).

Table 1 Selected Parameters in the Parametric Analysis

Parameter	Range of values in parametric analysis	
	Minimum	Maximum
h_w / t_{wh}	45	150
L_h / b_f	5	20
β	6.5°	19.5°
$b_f / 2t_f$	6	16
L_s	$d_b / 2$	d_b
t_s	$t_{sd} / 2$	$2 t_{sd}$

Where t_{sd} : Stiffener thickness determined by the conventional design method [2].

2.3 Methodology:

A typical tapered knee geometric configuration was selected as per Fisher [1]. Dimensioning of the tapered knee components was performed by the elastic approach [8] in accordance with the Egyptian Code of Practice for Steel Construction and Bridges [6] to support the straining actions induced on the

beam-to-column connection of a portal frame with 30 m span and with 8 m height when subjected to gravity and wind loads. The values of the non-dimensional parameters of such "reference connection" were: $h_w/t_{wh} = 100$, $\beta = 9.5^\circ$, $L_w/b_f = 9$ and $b_f/2t_f = 9$ [8]. Edge and diagonal stiffeners were designed to support the un-balanced flange force at points of abrupt change in flange direction and to satisfy local buckling requirements [9]. According to stiffeners used, three connection configurations were studied: Un-stiffened, semi-stiffened and fully-stiffened connections [5]. For the sake of comparison and verification, the elastic buckling load, EB , the plastic limit load, $EP(W/O)$, using elastic-plastic material model without geometric imperfections, and the limit load, EP , using elastic-plastic material model with geometric imperfections were determined for each connection configuration. Geometric imperfections applied to determine the limit load were similar to the respective first buckling mode with maximum amplitude not exceeding the limits stipulated by the ECP 2001 [9].

2.4 Finite Element Results:

In the subsequent sections, the parametric analysis results were presented. The maximum moment obtained in each case, M_{max} , was normalized by the respective plastic moment, M_p , of the section at haunch tip for comparison.

2.4.1 Un-stiffened connections:

Figure 2 shows that the connection capacity is inversely proportional to h_w/t_{wh} . Failure was governed by inelastic buckling of the web at corner re-entrant rather than by yielding. However, elastic buckling of the web controlled the connection capacity for h_w/t_{wh} values greater than 140.

Figure 3 shows that the limit load of the connection is proportional to the haunch slope, β . This was attributed to the fact that the un-stiffened connections capacity is governed by the strength of the panel zone. Therefore, as the taper slope increases, the depth of the haunch increases and hence the

straining actions induced in the panel zone decreases and the connection can sustain higher loads. However, when β exceeded 17° , buckling of web was transferred to haunch tip, and the connection limit load was reduced.

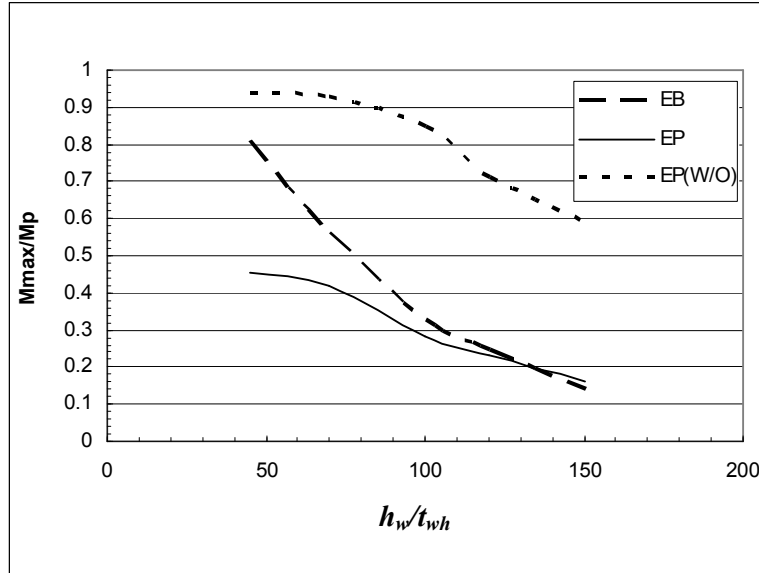


Fig. 2 Effect of h_w/t_{wh} on the un-stiffened connection moment capacity

Figure 4 shows that the connection limit load is proportional to the ratio $b_f/2t_f$ and failure was generally governed by inelastic buckling of web at corner re-entrant. When the ratio $b_f/2t_f$ increased, the torsional-restraint provided by the flanges to the web increased and hence the capacity of the web was pronounced. The same applied to the plastic limit load. However, the elastic buckling load was inversely proportional to $b_f/2t_f$ since by increasing $b_f/2t_f$ the flanges become more susceptible to local buckling. The peculiar behavior of the elastic buckling load at $b_f/2t_f=13$ was due to the transfer of the buckling mode from web local buckling at the corner re-entrant to web buckling in panel zone. Flange local buckling governed the elastic buckling load at $b_f/2t_f=16$.

2.4.2 Semi-stiffened connections:

In such connections, a diagonal stiffener was added in the panel zone to resist the unbalanced component of the flange forces at corner re-entrant and to strengthen the panel zone against the effect of large shear stresses.

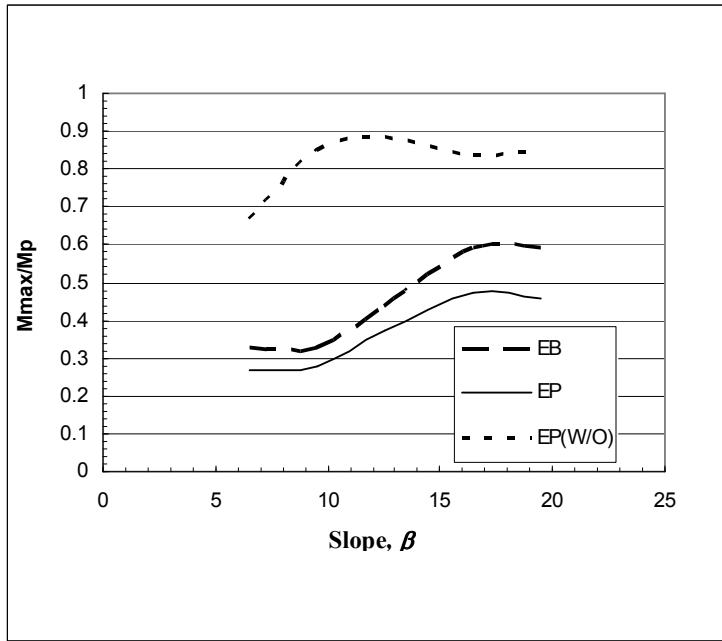


Fig. 3 Effect of β on un-stiffened connection moment capacity

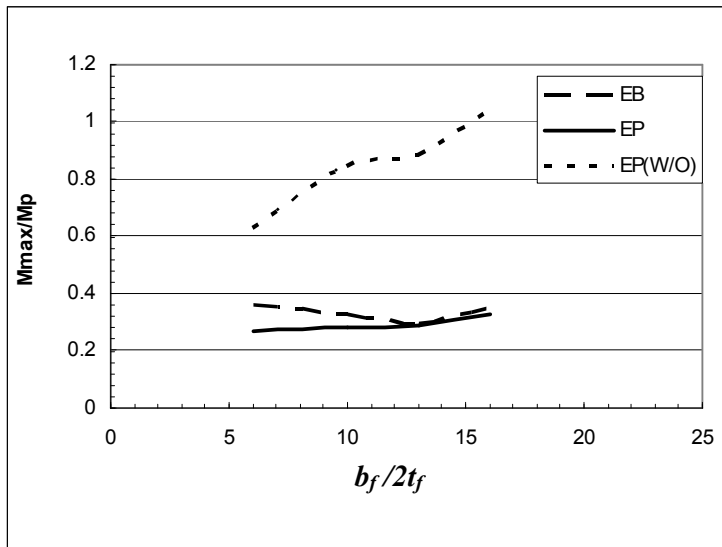


Fig. 4 Effect of $b_f/2t_f$ on un-stiffened connection moment capacity

Figure 6 shows that the connection capacity was inversely proportional to h_w/t_{wh} in all analyses cases. The connection capacity was almost doubled on average compared to the un-stiffened case. The failure mechanism was controlled by inelastic buckling only and the elastic buckling load exceeded

the plastic limit load. On the other hand, effect of geometric imperfections on strength was generally reduced.

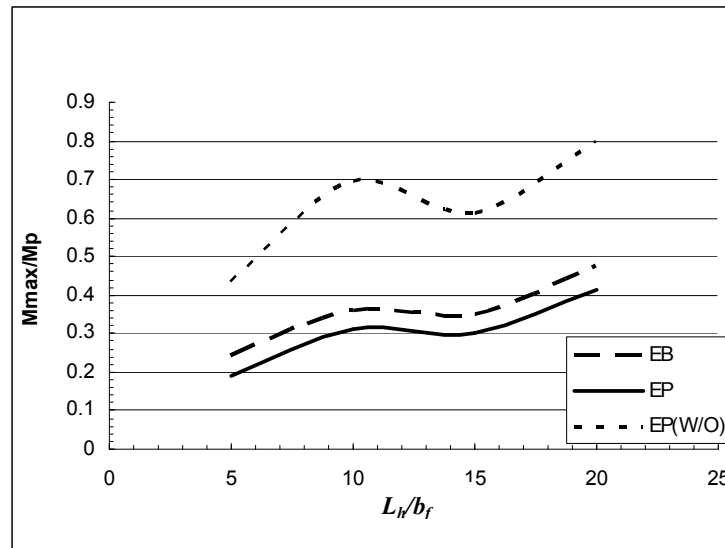


Fig. 5 Effect of L_w/b_f on un-stiffened connection moment capacity

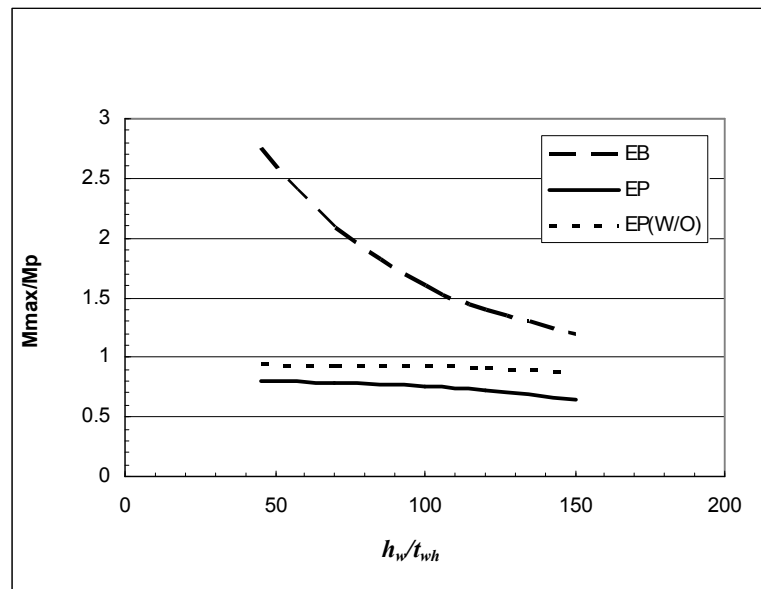


Fig. 6 Effect of h_w/t_{wh} on semi-stiffened connection moment strength

Unlike un-stiffened connections, the limit load of semi-stiffened connections was inversely proportional with β as depicted in Fig. 7. Due to the addition of the diagonal stiffener, the web buckling zone was no longer located in the web

panel zone but rather, it was shifted towards the haunch tips. Since the flange un-balanced force at haunch tip increased with β , the connection limit load was reduced by web buckling. Figure 7 indicates that failure at limit load was mainly attributed to inelastic buckling since the elastic buckling load was much greater than the plastic limit load and therefore the effect of geometric imperfections was minor.

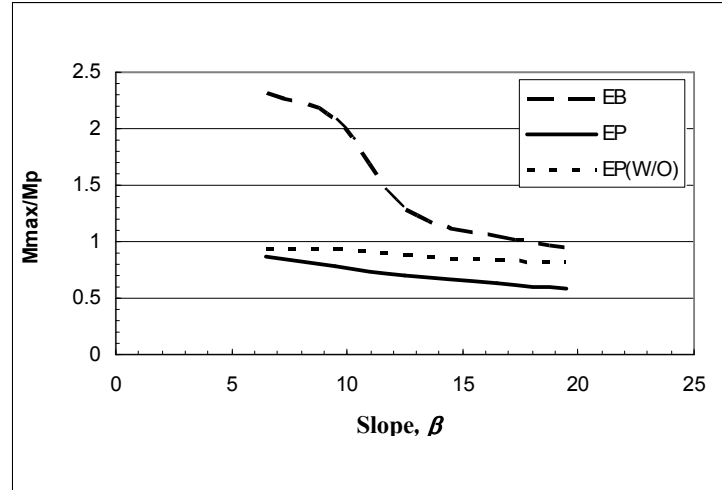


Fig. 7 Effect of β on semi-stiffened connection moment capacity

Figure 8 indicates that the effect of $b_f/2t_f$ on connection limit load was minor similar to the case of un-stiffened connection. However, the limit load in such case was almost doubled compared to the un-stiffened case. On the other hand, failure took place by inelastic buckling of the web at haunch tip for all values of $b_f/2t_f$ rather than elastic buckling for $b_f/2t_f \geq 13$ for the un-stiffened case (see Fig. 4). The connection limit load was slightly reduced by the increase of L_h/b_f as depicted in Fig. 9. This was attributed to the reduction in flange stiffness with the increase of L_h/b_f . On the other hand, the positive effect of increasing L_h/b_f on un-stiffened connection capacity due to the increase of h_w was eliminated by adding diagonal stiffeners since the limit load was governed by the inelastic buckling of web at haunch tip. For $L_h/b_f \geq 18$, failure was governed by lateral buckling of the flange. The failure of the connection was more governed by elastic buckling by the increase of L_h/b_f and thus effect of geometric imperfections on strength was also pronounced.

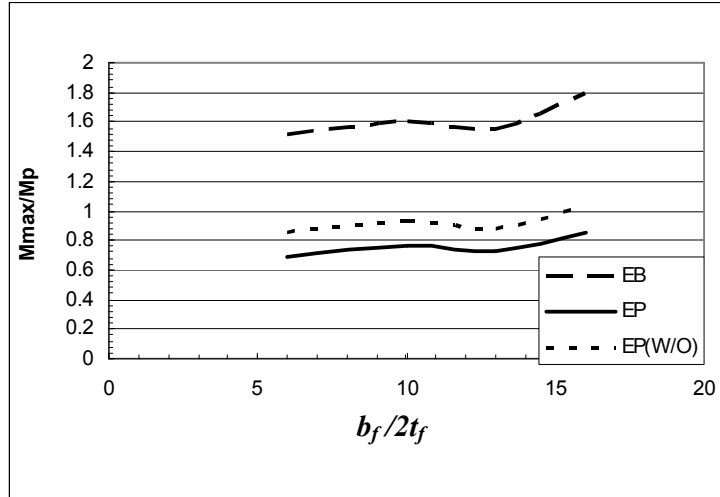


Fig. 8 Effect of $b_f/2t_f$ on semi-stiffened connection moment capacity

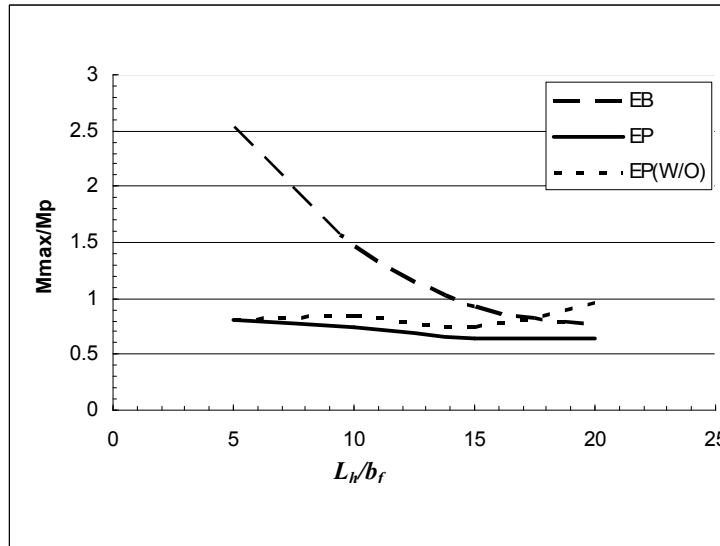


Fig. 9 Effect of L_h/b_f on semi-stiffened connection moment capacity

2.4.3 Fully-stiffened connections:

In this case, additional edge stiffeners were added at haunch tips to resist the unbalanced component of the flange forces at such points. The parametric analysis results obtained were compatible with the expected behavior, therefore the parameters h_w/t_{wh} , β , $b_f/2t_f$ and L_h/b_f had minor effect on the limit load obtained as depicted in Figs 10, 11, 12 and 13 respectively. Failure was mainly governed by yielding of the beam section at haunch tip thus the connection almost supported the full plastic moment M_p and geometric imperfections

effect on limit load was almost negligible. It should be noted herein that a slight reduction in the limit load was noticed with the increase of β due to pronounced shear force effect on yielding resulting from unbalanced flange forces at haunch tips.

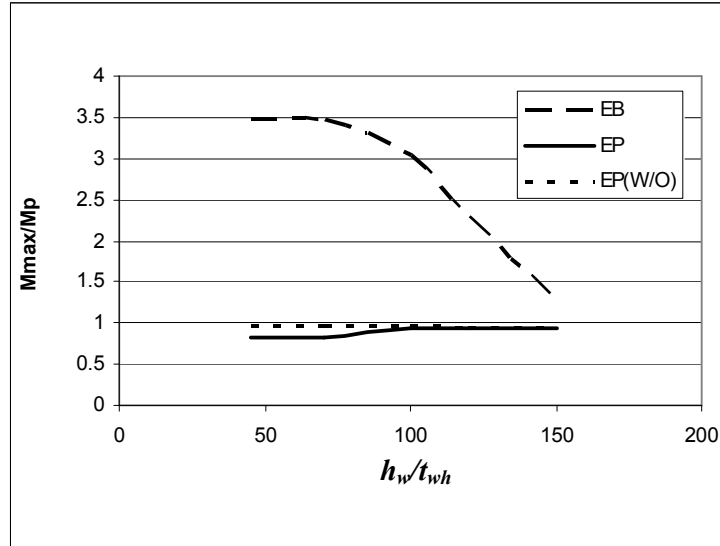


Fig. 10 Effect of h_w/t_{wh} on fully-stiffened connection moment capacity

2.5 Assessment of Stiffeners Configuration Effect on Capacity:

The parametric analysis results revealed that the addition of diagonal stiffeners almost doubled the connection capacity and altered the failure zone from corner re-entrant to haunch tips and failure was mainly governed by elastic buckling. Further addition of edge stiffeners pronounced the connection moment capacity to reach the plastic moment of the connected beam and failure was governed by yielding. In this section, the variation in connection limit load capacity due to addition of stiffeners was clarified quantitatively. The effect of adding diagonal stiffener, *Sti-Mid*, was evaluated by computing the percentage increase in moment capacity compared to an identical un-stiffened connection. On the other hand, the effect of further addition of edge stiffeners, *Sti-Mid & Edge*, was determined by computing the percentage of fully-stiffened connection moment capacity compared to an identical semi-stiffened connection. The impact of stiffeners geometric configuration such as length and thickness was also discussed.

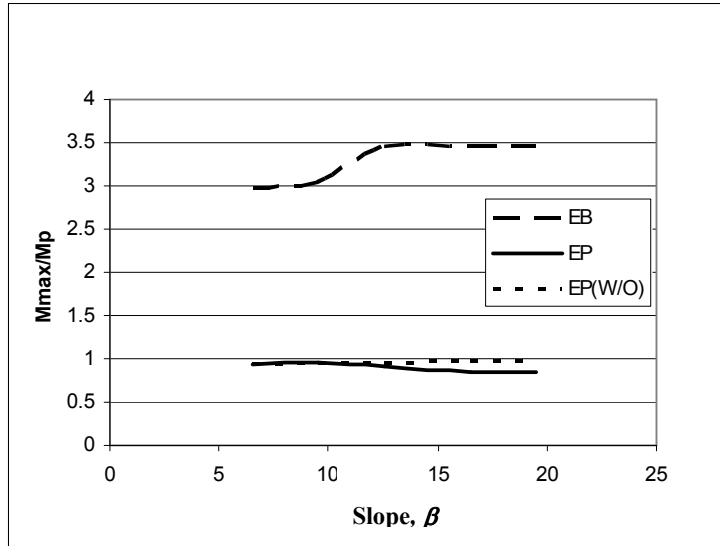


Fig. 11 Effect of β on fully-stiffened connection moment capacity

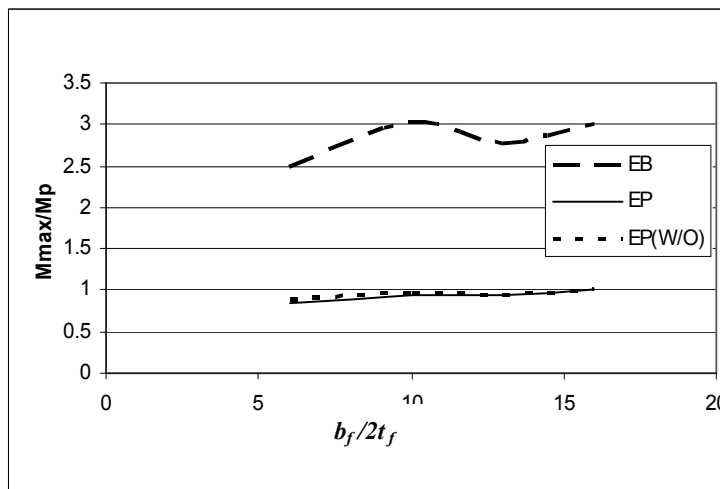


Fig. 12 Effect of $b_f/2t_f$ on fully-stiffened connection moment capacity

Figure 14 illustrate the percentage increase in moment capacity due to the addition of stiffeners corresponding to h_w/t_{wh} . Diagonal stiffener proved to be very crucial to the capacity of the connection especially in the case of large values of h_w/t_{wh} . The influence of edge stiffeners was more pronounced when steeper haunch slopes were used as depicted in Fig. 15.

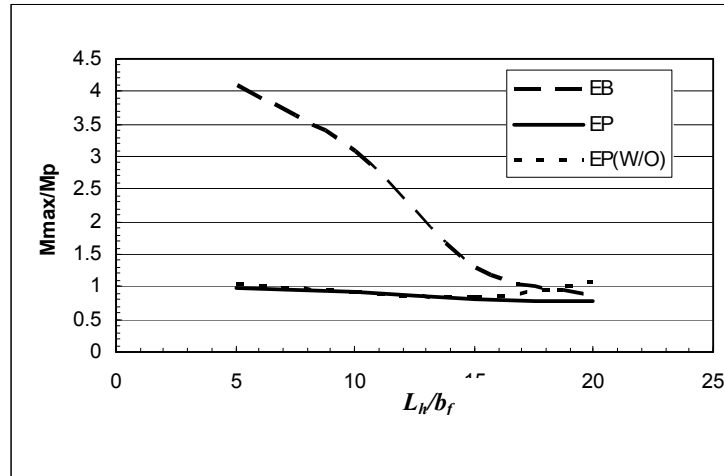


Fig. 13 Effect of L_w/b_f on fully-stiffened connection moment capacity

Effect of stiffeners length was also investigated. Diagonal stiffener was always assumed to extend from tension to compression flanges, however, edge stiffeners length was reduced to extend only to one half of the web depth at haunch tips. The percentage reduction in the elastic buckling load and plastic limit load due to reduction in stiffeners length compared to the case of full stiffener length was computed to be 18 % and 0.5%, respectively. This indicated that reducing the edge stiffener length to half of web depth had minor effect on moment capacity when failure takes place by yielding. Effect of stiffener thickness on moment capacity was outlined in a previous work where no significant reduction in moment capacity occurred when the stiffener thickness was reduced down to 0.7 times the stiffener thickness deduced from the conventional design method given that local buckling requirements were satisfied. Moreover, no gain in strength was achieved when the stiffener thickness exceeded its conventional value [5].

3. Proposed Design Equations:

In order to establish suitable design equations for tapered-haunched connections with different stiffener configurations, the parametric analysis results were fed into a linear regression analysis. The maximum moment

capacity of the connection, M_{max} , was written in terms of the plastic moment, M_p , of the beam section at haunch tip. Results are as follows:

Unstiffened Connections:

$$M_{max}/M_p = 0.4119 + 0.0367 \tan(L_h/b_f) + 0.0031\beta - 0.0029h_w/t_{wh} + 0.0185b_f/2t_f \quad (10)$$

Semi-stiffened Connections:

$$M_{max}/M_p = 0.9855 - 0.0127L_h/b_f - 0.0183\beta + 0.0154b_f/2t_f - 8.162 \times 10^{-6} (h_w/t_{wh})^2 \quad (11)$$

Fully-stiffened Connections:

$$M_{max}/M_p = 0.8489 - 0.0146L_h/b_f + 4.924 \times 10^{-4}\beta - 0.0076b_f/2t_f + 0.0014h_w/t_{wh} \quad (12)$$

It is to be noted herein that if the value of M_{max} exceeds M_p , this indicates that the connection can develop, numerically, the required plastic strength of the adjacent beam. However, in dimensioning the connection components, the value of M_{max} should not exceed M_p . Figures 16 to 19 verify the proposed design equations by comparison with the finite element results.

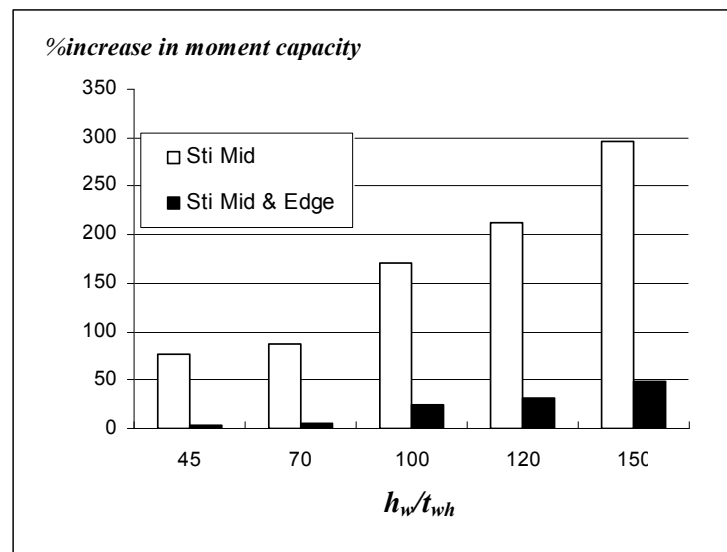


Fig. 14 Effect of adding stiffeners on moment capacity for various h_w/t_{wh} values

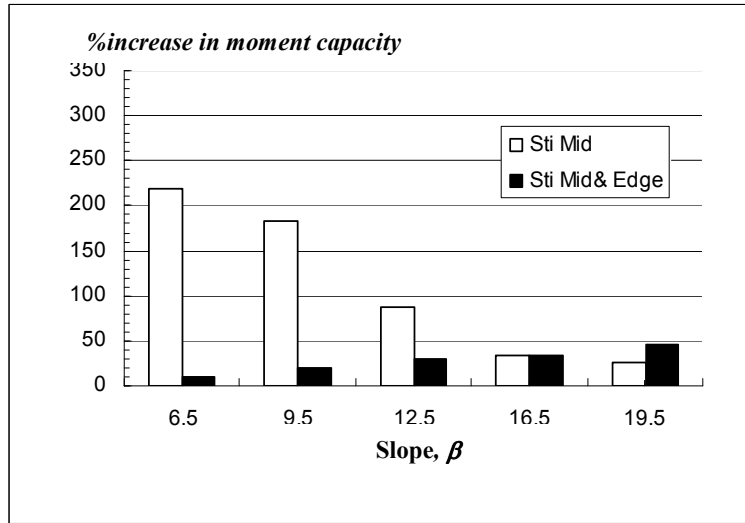


Fig. 15 Effect of adding stiffeners on moment capacity for various β values

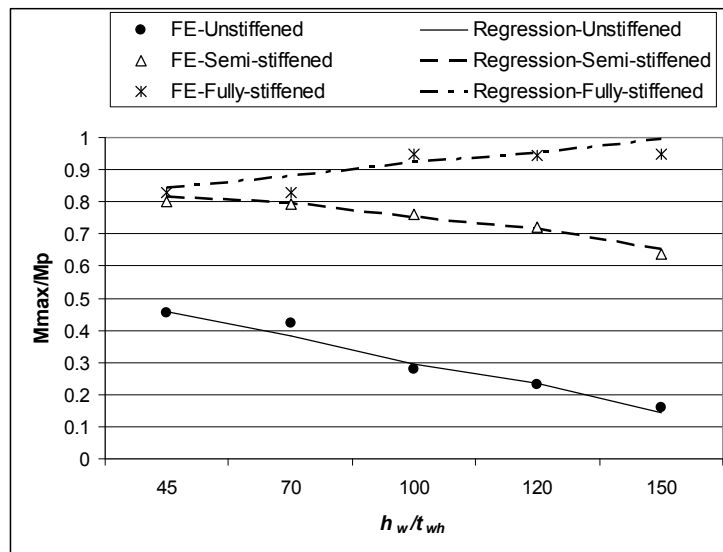


Fig. 16. Verification of design equations for various h_w/t_{wh} values

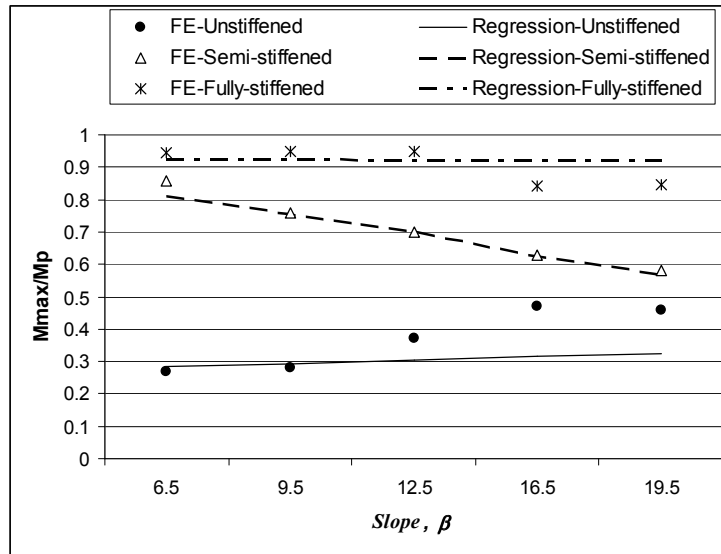


Fig. 17. Verification of design equations for various β values

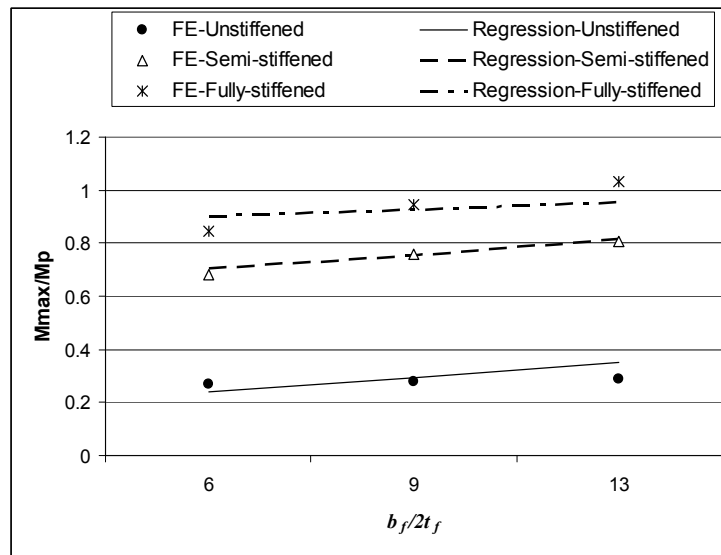


Fig. 18. Verification of design equations for various $b_f/2t_f$ values

4. Summary and Conclusions:

In this work, a comprehensive parametric analysis of tapered-haunched connections was conducted to investigate the effect of the connection geometric parameters on moment capacity and mode of failure corresponding to different stiffeners configurations. Proposed design equations for each stiffeners configuration were established by regression analysis of finite

element results. The analysis indicated that the connection moment capacity and failure mode was greatly dependent on stiffeners configuration, however, the connection strength was nearly insensitive to the length of the edge stiffeners allowing the use of shorter stiffeners. Moreover, a decrease in the stiffeners thickness compared to the thickness deduced from the conventional method could be also achieved without any loss in the connection strength.

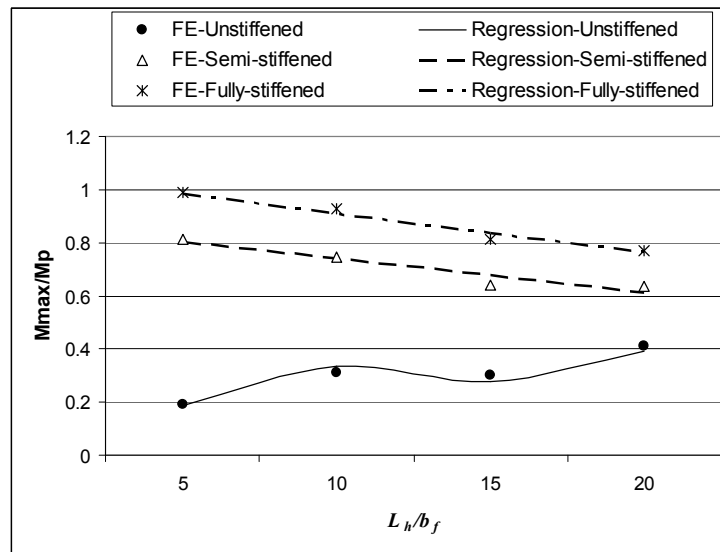


Fig. 19. Verification of design equations for various L_h/b_f values

REFERENCES:

1. Fisher, J.W., Lee, G.C., Yura, J.A. and Driscoll, J.r., "Plastic Analysis and Tests of Haunched Corner Connections" WRC Bulletin No. 91, New York, 1969.
2. Machaly, E.B., "Behavior, Analysis and Design of Steel Work Connections", 4th Edition, 2000.
3. Osgood, W.R., "A Theory of Flexure for Beams with Non Parallel Extreme Fibers" Journal of Applied Mechanics, ASME, 1939.
4. Bleich, F., "Design of Rigid Framed Knees", American Institute of steel Construction, Chicago, IL, 1943.
5. Machaly, E.B., Safar, S.S., El-Banna, A.A., "Analysis of Tapered-Haunched Connections", Journal of Engineering and Applied Science, Faculty of Engineering, Cairo University, 2006.
6. Desalvo, G.J., and Gorman, R.W., "ANSYS User's Manual", Swanson Analysis Systems, Houston, PA, 1989.
7. Salmon, C.G. and Johnson, J.E., "Steel Structures: Behavior and Design", Harper and Row Publishers, 4th Edition, 1996.

8. El-Banna, A.A., "Analysis and Design of Tapered and Curved Beam-to-Column Connections", M.Sc Thesis, Cairo University, 2005.
9. Egyptian Code of Practice for Steel Construction and Bridges (Allowable Stress Design), Code no. (205), 1st Edition 2001.

دراسة المتغيرات و التصميم لوصلات الإطارات ذات القطاعات المسلوية

الملخص:

تستخدم الوصلات المسلوية بكثرة فى الاطارات المعدنية ذات البجور المتسعة و المتوسطة الاتساع . و لقد ظهرت الحاجة لاستنتاج طرق بسيطة ، و لكن دقيقة ، لتصميم هذه الوصلات . لهذا الغرض ، تم اجراء دراسة المتغيرات باستخدام برنامج طريقة العناصر المحددة ANSYS لدراسة تأثير العوامل المختلفة المؤثرة على مقاومة الوصلة . و لقد أخذ فى الاعتبار تأثير العوامل المختلفة و التى اشتملت على نحافة الوتر، نحافة الشفة ، ميل الجزء المسلوب ، النسبة ما بين طول الجزء المسلوب و عرض الشفة ، تخانات و طول أعصاب التقوية و كذلك التوزيعات المختلفة لهذه الأعصاب . استخدمت نتائج الدراسة البارامترية فى اجراء دراسة احصائية لاستنتاج معادلات تصميم يمكن استخدامها فى تحديد مقاومة الوصلة المناظرة لتوزيعات مختلفة لأعصاب التقوية . كما تم وضع توصيات اضافية متعلقة بتحديد أبعاد هذه الأعصاب .