

NUMERICAL STUDY OF DBT CONNECTIONS FOR SEISMIC MOMENT RESISTING FRAMES

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Abstract

The research reported herein corresponds to the numerical study of the response of a Double Built-up Tee (DBT) beam moment connection, when subjected to monotonic loading. The objective of this research was to identify the failure modes associated with this type of connection.

A subset of all the available beam sizes from the AISC Manual was selected, considering that there are no experimental results for beams under the W21 series.

Four prototype buildings were designed according to the Chilean seismic code for buildings, considering the flexibility of the connection, and complying with the seismic compactness and strong column-weak beam requirements set forth by the AISC seismic provisions. The structures designed had the same 36 meters by 54 meters floor plan, uniform story height of 3.5 meters and distance between columns of 9 meters, but different number of stories (three, six, nine, and twelve).

A few beam/column combinations were selected from the prototype buildings and different connection configurations were designed for each combination to represent several failure modes reported in the literature. These configurations were represented numerically through 3D finite element models, using ANSYS. The models were subjected to monotonic loading up to the full plastification of the beam cross section in flexure.

The results show that it is not possible to draw general design and modeling recommendations for the connection based only on the response of the isolated T stub, and that the conditions imposed by the beam must be considered. The results of the numerical models will be validated with the results of full size beam column-connection specimens to be tested.

Keywords: numerical model; failure modes; partially restrained connection.



1.Introduction

Prior to the earthquakes of Northdridge (1994) and Kobe (1995), the welded unreinforced flange (WUF) connection was the most popular beam-to-column moment connection in the United States [2]. However, once these events occurred, a series of brittle failures were detected in this type of connection (Fig. 1). These unexpected failures prompted the Federal Emergency Managment Agency (FEMA) to issue a call to the engineering community to develop improved seismic design criteria for buildings structured with moment resistant frames (MRFs), which culminated with the development of FEMA 350 [3].





Fig. 1: Pre-Northridge connection and types of failures observed post earthquake [2].



FEMA 350 includes several connection configurations that were prequalified to be used as part of the seismic resistant system. One of these connections is the Double Split Tee (DST) moment connection. Consistent with the practice in the USA, all the research conducted to prequalify the connection was developed usinghot-rolled sections for beams, columns, and Tees. An ongoing research program at the University of Chile is looking at the possibility of using built-up instead of hot-rolled T-stubs. The use of built-up tees presents advantages in terms of material utilization, freedom of sizing, and availability, but it can introduce brittle failure modes associated with the weld between the flange and the stem of the T. As part of this research, the response of an isolated built up T representing the connector element was studied numerically and experimentally, concluding that if the weld is properly executed, the behavior of the connection is analogous to that with a hot-rolled T section [2]. In addition, dynamic analyses were performed on models of 2D frames representing buildings where moment frames with this type of connections were the main lateral force resisting system, including the rotational stiffness of the connection, concluding that the structure must be designed for serviceability and not resistance, because when considering a partially restrained (PR) connection, the drift increases [4].

The contribution of this paper is to analyze numerically using 3D finite element model in ANSYS [5] the various failure modes established in the literature of a series of connection models and compare these results with those obtained theoretically when the design procedure of FEMA 350 is used.

2.Failure modes considered

A number of failure modes or limit states can develop in the column, beam and connection. Table 1 shows all the failure modes normally considered for the connection and the beam, in the case of the DST connection. Fracture of the tension or shear bolts is not considered in this study because they are brittle failure modes than have to be avoided by design. Although net section fracture and block shear of the T web are also fracture related failure modes, they are included in the study because they provide a significant amount of energy dissipation before they occur, through bearing and slip of the bolts under shear and they usually cannot be prevented to occur by design.

Failure Modes	Capacity	Туре	OBS
Fracture of the tension bolts	M _{ftb}	Brittle	Ultimate
Plastic bending of the Tee flange	M_{pb}	Ductile	Design
Fracture of the shear bolts	M_{fsb}	Brittle	Ultimate
Net section fracture of the Tee web	M_{nsf}	Brittle	Ultimate
Block shear failure of the Tee web	M_{bs}	Brittle	Ultimate
Plastic hinging of the beam	M_{phb}	Ductile	Design
Net section fracture of the beam flange	M _{nsfb}	Brittle	Ultimate

Table 1- DBT connection and beam failure modes

Initially, the application of the "Net section fracture of the beam flange" limit state prevented a significant number of sections from being used as beams in the lateral load resisting system. Therefore, it was decided to explore the likelihood of occurrence of the failure mode. To do this, the data from the specimens tested by Swanson [6], which were the base of the FEMA 350 recommendations, was used to conduct the evaluation. Using the geometric and material information available from the tests, the capacities associated with each failure mode was evaluated following FEMA 350 (Table 2).



Specimen	\mathbf{M}_{phb}	M _{nsfb}	M _{nsf}	M _{shear}	$\mathbf{M}_{\mathbf{pb}}$	M _{tensile}	M _{bs (*)}	Fail	ure mode
	[tonf-m]	[tonf-m]	[tonf-m]	[tonf-m]	[tonf-m]	[tonf-m]	[tonf-m]	FEMA 350	Experimental [6]
FS03	77.6	61.8	81.2	98.5	162.6	119.0	70.4	N.S.F.B.	N.S.F.
FS04	78.4	60.5	89.2	129.2	299.9	155.3	81.9	N.S.F.B.	N.S.F.
FS05	110.3	89.9	146.5	141.6	1283.3	163.2	136.8	N.S.F.B.	P.H.B.
FS06	109.5	86.9	132.0	147.4	-5905.3	208.1	120.8	N.S.F.B.	P.H.B.
FS07	110.3	89.9	157.4	141.6	1358.6	167.1	147.2	N.S.F.B.	N.S.F.B.
FS08	80.9	78.7	154.7	185.9	-4105.8	211.8	166.8	N.S.F.B.	P.H.B.

Table 2- Failure modes using FEMA 350 and experimental

(*)Moment capacity not extrapolated to the face of the column

The results in Table 2 show that theoretically all specimens should have failed by net section fracture of the flange of the beam (M_{nsfb}). However, only specimen FS07 exhibited this type of failure and it appeared only after the beam had achieved its plastic capacity and significant bucking had developed in the beam flange. If this failure mode is not considered all specimens should theoretically have failed by plastic hinge formation on the beam (M_{phb}). This was the case for all specimens except FS03 and FS04, which failed by net section fracture of the connection (M_{nsf}), but the capacities of both failure modes were close enough so that a small variation in the strength of the beam would cause the net section fracture of the T control.

3. Numerical model description and validation.

The finite element model developed is an extension of the model presented by Bravo [2], which represented one half of the T stub. To simplify the model, the column was replaced by a rigid surface with compression-only supports. Solid rectangular 20-node and tetrahedral 10-node elements were used to mesh the T stub and bolts. The beam was divided in three segments indicated in different tones in Fig. 4: a segment at the connection, a plastic hinge segment, and a nominally elastic segment. A denser mesh was used in the segment where the plastic hinge in the beam is expected. The analysis was conducted under displacement control. The load was applied in two stages; first, the bolts were subjected to a pretension, followed by the application of a vertical displacement at the tip of the beam.

Bilinear stress-strain curves were used to represent the response of the different parts of the model. The nonlinear behavior of the material was modeled using a Von Mises yield criterion with isotropic hardening.

A number of contact conditions were imposed to describe the interaction between the different parts of the model, namely, T stub, bolts, and beam. The model can capture the bearing of the shear bolts on the holes and the ensuing local deformation of the plate around the bearing area. The friction between the beam flange and the T stem was modeled using a "slip-critical standard" contact condition, with a friction coefficient μ of 0.33, assuming Class A surface conditions according to AISC [11].

To validate the numerical model, specimen FS04 tested by Swanson [6] was modelled and analyzed under monotonic loading. The reported material properties were used for the beam, T stubs, and columns. For the other parts of the model, nominal values were used. The specimen was selected because, although it was built with hot-rolled T stubs, the damage was concentrated in the beam, hence, the effect of the weld should not be relevant in the response of the specimen. Because the column was not modelled, it was included as an additional flexibility causing additional vertical displacement at the tip of the beam, assuming elastic response. The model captures adequately the yielding and capacity of the test specimen. The difference in stiffness can be attributed to all the flexibilities in the test setup and the test specimen that cannot be captured by the numerical model, for instance the panel zone deformation, bending of the column flanges, and deformation at the supports.

Figure 2 shows an isometric view of the numerical model and a comparison of the response determined numerically, under monotonic loading, and experimentally, under cyclic loading.





Fig. 2: ANSYS numerical model and validation

4. Design of prototype buildings.

The connections and beams to be analyzed numerically were obtained from the design of prototype buildings. Four prototype buildings were designed according to the requirements of the Chilean seismic building code NCh433 [7]. The buildings had the same floor plan with moment resisting frames located on the perimeter, as shown in Figure 3, a constant story height of 3.5 m, and they differed only in the number of stories (3, 6, 9, and 12). The buildings were destined for housing and assumed to be located on soil type A and seismic zone I, as per NCh433.



Fig. 3: Typical floor plan and elevation of the prototype structures

Table 3 presents the sizes of the beams resulting from the design of the prototypes. The last two columns on the table present the total number of stories and the specific stories of the prototype where the beam size was used,



respectively. Only two sizes of beams were selected for each prototype. No effort was made to optimize the design, because the purpose was to find reasonable beam and connections sizes to study in detail numerically and experimentally.

Beam size	d	b _f	t _f	t _w	Prototype	Levels
	mm	mm	mm	mm	stories	used in
W33x118	900	540	110	90	6	1-4
W36x182	900	540	110	90	9	1-7
W36x194	900	540	110	90	12	1-10
W21x101	800	440	60	17	3	All
W24x94	800	440	60	15	6	5,6
W24x94	800	440	60	15	9	8,9
W24x84	800	440	60	13	12	11,12

Table 3- Dimensions of beams used in the prototype buildings

Table 4 lists the results for natural period of the structure, design base shear and minimum base shear required by NCh433 [11], and the ratio between minimum and design base shear. The maximum allowed elastic drift for the design spectrum (7 mm according to NCh433) is always larger or equal to the drift of the prototypes. The strong column/weak beam requirement is met for all the prototypes, with ratios ranging from 1.3 for the 12 story prototype to 4 for the 3 story prototype.

Table 4- Dynamic and design quantities for the prototype structures

Response quantities	Units	Prototype					
		3 story	6 story	9 story	12 story		
Fundamental period, T*	[s]	0.53	1.32	1.74	2.31		
Design base shear, V _{b_design}	[kN]	278	261	294	305		
Minimum base shear, $V_{b_{min}}$	[kN]	417	938	1509	2118		
V _{b_min} / V _{b_design}	-	1.50	3.60	5.14	6.95		

The use of built up tees allows the designer to size the connection as a fully rigid connection, considering that the sizes of the flange and the stem of the T can be freely selected. However, Piluso et al. [8] showed that depending on the relative sizes of the tee, three failure modes (shown in Figure 4) can occur, namely: yielding of the tee flange (mode 1), yielding of the tee flange combined with fracture of the tension bolts (mode 2), and fracture of the bolts alone (mode 3).



Fig. 4: Failure modes identified by Piluso et al [8]

Designing the connection to be fully rigid would induce mode 3 (fracture of the tension bolts) to occur, which is a brittle and hard to predict failure mode. Therefore, for this study it was decided to design the connection as partially restrained.



Four different connection configurations were designed for each beam size, each one looking to achieve a specific limit state, according to FEMA 350 (FEMA, 2000). These designs are shown in Table 5, whereby the first design for each beam size corresponds to the one that induces the formation of the plastic hinge on the beam. The variables in Table 5 correspond to the nomenclature used in FEMA 350, reproduced in Figure 3.



Fig. 5: Geometric parameters according to FEMA 350 (FEMA, 2000)

This study only shows the comparison of numerical and theorical results for DBT1 through DBT 12 connections. In almost all cases 1 3/8" diameter bolts were used, except for DBT5 where 1 1/2" diameter bolts were used. In all this cases, four shear bolts and four tension bolts per row were considered, respecting the minimum distance between hole and edge set in the AISC 2010 [11]. A distance between the edge of the flange and the center of the hole of 71 mm resulted for DBT1 through DBT8, while for DBT9 through DBT 12, 69 mm was used. Table 6 shows the values for each parameter of the connection and the bending moment values for each failure mode.

ID	Story	Beam	\mathbf{s}_1	s ₂	g	t _{stem}	н	t _f	s	Failure mode	M _{phb}	M _{nsf}	M bs	M tfb	EI _{eq} /
DBT		Size	[cm]	[cm]	[cm]	[cm]	[cm]	[cm]	[cm]	[8]	[tonf-m]	[tonf-m]	[tonf-m]	[tonf-m]	EI
1	6,9	24x94	12,4	10,5	13,0	2,0	26,0	2,4	1,2	1	163,6	231,4	219,5	285,22	0,70
2		24x94	13,6	12,50	13,0	1,2	30,0	3,6	1,2	2	167,5	137,5	152,3	176,61	0,60
3		24x94	13,6	10,50	13,0	1,2	30,0	3,6	1,2	2	164,3	137,5	131,7	176,61	0,59
4		24x94	12,2	10,50	13,0	2,0	30,0	2,2	1,2	1	163,5	231,3	219,5	74,08	0,44
5	12	24x84	12,8	11,50	13,0	2,4	30,0	2,8	1,6	1	147,3	272,7	278,3	168,41	0,67
6		24x84	13,6	12,50	13,0	1,0	32,0	3,6	1,0	2	149,0	113,3	125,9	126,01	0,58
7		24x84	13,6	10,50	13,0	1,0	32,0	3,6	1,0	2	146,1	113,3	108,8	126,01	0,57
8		24x84	12,3	10,50	13,0	2,2	32,0	2,3	1,4	1	145,5	253,3	239,5	64,43	0,44
9	3	21x101	12,4	10,50	16,0	2,2	25,5	2,4	1,4	1	167,7	225,8	229,7	374,51	0,70
10		21x101	13,0	13,00	16,0	1,4	26,5	3,0	1,4	1	172,1	141,9	171,7	236,63	0,60
11		21x101	13,0	10,50	13,0	1,4	26,5	3,0	1,4	1	168,0	141,9	135,2	236,63	0,59
12		21x101	12,1	10,50	16,0	2,0	26,5	2,1	1,4	1	167,5	204,4	208,8	143,39	0,60

Table 5- Geometric parameters of the connection for W24x94, W24x84 and W21x101 beams and failure modes



This section shows the results of Von Mises stresses for each failure mode considered and comments for DBT1 to DBT 12 connections.

All numerical models were subjected to a step by step displacement in the free end of the beam to reach 2% of rotation of this, at which point yielding flange beam is initiated. This situation occur in the all numerical models but only in the DBT1, DBT4, DBT5, DBT8, DBT9 and DBT 12 connections occurs the full plastification of the beam cross section in flexure (Figure 6).

Nominal properties were used for the base material for the beams ($F_y = 250$ [MPa] and $F_u = 400$ [MPa]) and expected properties for the Tees ($F_y = 320$ [MPa] and $F_u = 480$ [MPa]); both classified as ASTM A 36. For the shear and tension bolts ASTM A490 was considered, whose mechanical properties were determined in the study of Salas [10] from samples extracted from three 1 3/8"diameter bolts (*E*=195815 [MPa], F_y =1149 [MPa] and F_u =1246 [MPa]).



Fig.6: Full plastification of the beam cross section in flexure for the DBT1, DBT4, DBT5, DBT8, DBT9 y DBT 12 connections.

Figure 7 shows the stress distribution for connection identified as DBT 3 which, according to Table 6 was theoretically designed to fail by block shear. In this case the stress distribution is not homogeneous in the plane of fault shear. This situation is typical for all connections designed to fail in this way unless at the design stage the minimum spacing between bolts and bolt to the edge, they are less than those established in the AISC 2010. Then, the expected failure for DBT3, DBT 7 and DBT11 connections will be the net section fracture of the Tee.





Fig. 7: Stress distribution in the plane of fault shear for the DBT3 connection

Table 6 shows the results of stresses obtained for each failure mode omitting the block shear failure.

Table 6 shows as expected that in all cases the failure for shear and tension bolt it does not occur, reaching a maximum of the capacity of 82% and 48%, respectively.

To achieve the failure mode of net section fracture, the beam must yield imposing a displacement of the free end of the beam such that rotation of this is greater than 2%, because, the maximum stress of the stem is 375 [MPa]. The reason why the first failure for net section fracture net and not by plastic bending is due to brittle behavior of the stem.

Regarding the plastic bending of the T flange, it occured for DBT8 and DBT4 at 25 mm and for DBT12 at 54.4 mm, i.e., before reaching the 2% of rotation of the beam, reaching 294 [MPa] capacity on average. This value is very close to 320 [MPa], which indicates that the expected failure mode will be this.

For the failure mode for plastic hinge beam,net section fracture and plastic bending of the T flange, the maximum variation with respect to the formulation set by the FEMA 350 was -7.7%, -14.5% and 21.9%, respectively.



ID	$\sigma_{\rm sb}$	σ _{sb} /748	σ_{tb}	σ _{tb} /1149	σ_{tT}	$\sigma_{\rm pb}$	$\sigma_{\rm b}$	σ _b /320	M numerical	Theorical Fail	M FEMA 350	Variation with respect to FEMA 350
	[MPa]		[MPa]		[MPa]	[MPa]	[MPa]		[tonf-m]		[tonf-m]	[%]
DBT1	492	0.66	470	0.41	320	265	322	1.01	157	P.H.B.	164	-4.3
DB12	398	0.53	495	0.43	370	260	325	1.02	119	N.S.F.	138	-13.8
DB13	485	0.65	205	0.48	370	257	326	1.02	118	N.S.F.	138	-14.5
DB14 DBT5	404 576	0.62	305	0.27	318	262	330	1.03	1/1	P.B. at 25 mm	14	10.2
DB15 DBT6	330	0.44	400	0.34	316	261	320	1.04	99.5	NSF	147	-4.1
DBT7	392	0.52	380	0.33	375	267	318	0.99	98.4	N.S.F.	113	-12.9
DBT8	439	0.59	348	0.30	311	303	327	1.02	78	P.B. at 25 mm	64	21.9
DBT9	523	0.70	335	0.29	321	206	326	1.02	155	P.H.B.	168	-7.7
DBT10	476	0.64	424	0.37	357	243	319	1	124	N.S.F.	142	-12.7
DBT11	563	0.75	427	0.37	365	243	327	1.02	122	N.S.F.	142	-14.1
DBT12	613	0.82	388	0.34	326	280	324	1.01	130	P.B. at 54.4 mm	143	-9.1
Where:	σ_{sb} , max	timum shea	ar bolt stre	SS.								
	σ_{tb} , max	timun tensi	ón bolt str	ess.								
	σ_{tT} , aver	rage net see	ction fractu	ure Tee stro	ess.							
	σ_{pb} , min	imun plast	ic bending	Tee stress								
	$\sigma_{\rm b}$, flang	ge beam sti	ess.									
Where:	P.H.B.,	Plastic Hin	ige Beam.									
	N.S.F., 1	Net section	fracture o	of the Tee.								
	P.B., Plastic bending of the Tee.											

Table 6- Failure modes for DBT1 to the DBT12 connections

6.Conclusions

The details of a research program being conducted to study the performance of Double built-up T moment connections have been presented. The focus of the study is the numerical simulation of full size connection subassemblages.

Four prototype buildings were designed following current seismic design provisions in Chile. The design is feasible, although the sizes of beams and columns result large, due to the stringent displacement limits and minimum base shear requirements.

A finite element model capable of reproducing the response of the connection/beam assemblage has been developed and shown to predict adequately the yield and ultimate capacity of the connection.

The numerical analysis results show that the distribution of stresses in the shear plane of the web of the T is not homogeneous. Therefore, it is extremely unlikely that the block shear failure would occur.

Full or partial plastification of the beam occurs in all models at 2% rotation. At this level of rotation, fracture of the T stem had not yet occurred; however, the tendency to present this failure mode was evident after some more deformation was imposed to the model.

The capacity associated with plastic bending of the T flange is significantly lower than the beam capacity. Therefore, designing the connection for the full strength of the beam while the controlling failure mode is plastic bendong of the t flange is not possible.

Finally, as expected from the design procedure the tensions generated in the shear and tension bolts are not high enough to generate a brittle failure.



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