

Lessons Learned from Design and Test of Latticed Steel Transmission Towers

ABSTRACT

Design of latticed steel transmission towers is a challenging task which involves lengthy modeling and detailing of significant amount of steel members and connections. Testing of the towers designed validates consistency between modeling and detailing and ensures the towers are designed to withstand design loads without premature failures.

This paper presents lessons learned from design and testing of latticed steel towers in a recent project of developing a family of nine (9) 500kV transmission towers. The paper provides a discussion about the process of selecting bidders for design, detailing and fabrication of latticed steel towers, and a brief discussion of determining materials used in the towers. The paper also discusses comparisons of different tower weights for various tower base widths and foundation reactions to achieve a cost effective design combining both the tower and its foundations. Examples of modeling tower members using beam elements in PLS-TOWER are presented to illustrate the importance of fully understanding the main purpose of using beam elements in TOWER. Variations between tower modeling and detailing are presented with specific attention to member and connection eccentricities. Finally, lessons learned from six (6) tower tests are discussed.

INTRODUCTION

The majority of 500kV transmission lines in Georgia were constructed in 1970s. All latticed steel towers used in the 500kV transmission lines in Georgia were originally developed by Georgia Power Company (GPC) in the late 1960s and early 1970s. The towers were not designed with working clearances for hot line nor helicopter maintenance.

During the 1980s and 1990s, there was not much demand for building new 500kV transmission lines in Georgia. However, in early 2000s, construction of several hundred miles of new 500kV transmission lines was anticipated. In light of the construction anticipated, studies of 500kV structures deemed necessary in order to provide clearance required for hot line and helicopter maintenance, to comply with the latest national design specifications, and to achieve the most economic design.

BACKGROUND

In early 2004, Georgia Transmission Corporation (GTC) expected to construct approximately 90 miles of 500kV transmission line in next five to seven years. GTC understood that GPC, another member of the Georgia Integrated Transmission

System (ITS), anticipated a similar need of 500kV transmission line construction. GTC also found that GPC was in process of updating its existing horizontal configuration tower family to comply with the latest National Electric Safety Code (NESC). GTC discussed with GPC on several occasions about jointly developing a new family of 500kV structures to meet the demand of the new construction of 500kV lines in Georgia. GPC and GTC agreed that it would be beneficial for both companies to use same 500kV structure standards in the future construction of 500kV transmission lines in Georgia. At the time, Southern Company transmission line standards team was working on the study of a 500kV Structure Family Evaluation to establish a standard 500kV structure family within Southern Company. GTC joined the study in late 2004 after received an invitation from GPC. The study involved evaluating design and construction criteria and costs for a multitude of 500kV structure families using 35 miles of Mostellar Spring – McGrau Ford 500kV line as a basis for comparison. The study considered the following structural families:

- Existing GPC horizontal configuration towers
- Existing Alabama Power Company (APC) delta configuration self-supporting and guyed towers
- Delta configuration single poles
- Horizontal configuration guyed H-frames

The tangent structures in each family considered are shown in Figure 1.

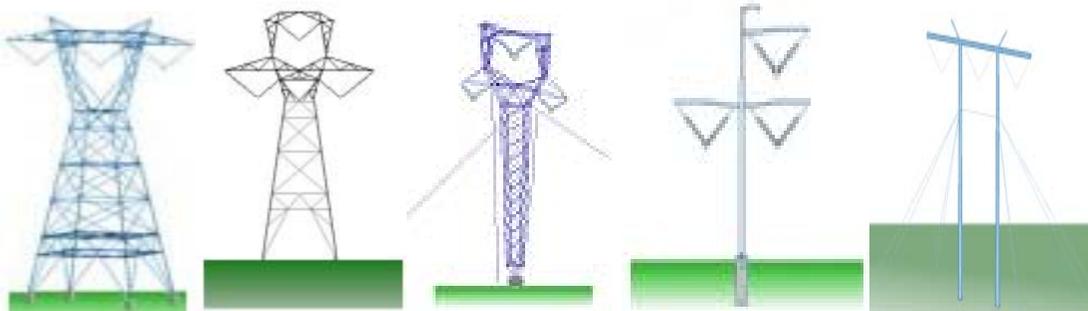


FIGURE 1

After the extensive studies, it was concluded that an upgraded existing APC delta configuration tower family is the desired 500kV standard structures going forward. The new family of structures will have following improvements to the existing structures studied:

- Provide adequate clearance required for hot line maintenance
- Provide adequate phase spacing between shield wires and conductors required for helicopter maintenance
- Provide engineered buzzard shield for protection of insulators from buzzard contamination
- Design an additional angle structure to provide efficient line design with a more completed family of structures
- Accommodate more effectively with 150 feet rights-of-way width under the various loading conditions encountered
- Use more readily available angle sizes and provide interchangeable angles between AISC and metric angles in design and detailing
- Provide better electric behaviors

Based on the assessment of work involved in developing the new family of 500kV structures, a working team consisting of members from Southern Company and GTC was formed. The team was responsible for coordinating design requirements of Southern Company and GTC, setting design parameters, determining bidding process, selecting winning bidder and reviewing design and details submitted by the bidder.

BIDDING PROCESS

In 2005, the working team started the pre-bidding process. It began with initial contact with consulting firms in the United States which have known experience in design and analysis of 500kV transmission lines. After review of the information collected in the initial contact, considering the scope of the project and capacity of the consulting firms, the following four (4) firms were selected for further consideration:

- Black & Veatch
- Commonwealth Associates Inc.
- Burns & McDonald
- Power Engineers

The working team conducted face to face interviews with each of above consulting firms. The interview started with the consultant's presentation on experience of latticed steel tower design, recent 500kV transmission line work, and design software, particularly PLS-TOWER (TOWER). Then detailed discussions on project scope, schedule, quality control process, tower detailing and tests, tower fabrication, tower erection and other related topics were carried out. It was intended initially to have a consulting firm perform all engineering work including design, analysis and detailing of the family of tower. However during the interview and discussions, it was found out that the consultants have either very limited or no capacity at all for detailing of latticed steel tower. After much deliberation, it was decided that the project would be better carried out in two steps. First, the team would solicit proposals from the consulting firms to develop design specifications for the latticed steel towers. Then a tower supplier would be selected to provide services in design, analysis, detailing and testing of the towers based on the design specifications developed. The tower supplier selected would also supply 157 steel towers for GTC's thirty-eight (38) miles of Thomson – Warthen 500kV Transmission Line Project.

After proposals for developing the design specifications from the consulting firms were evaluated, Black & Veatch was selected. In order to incorporate requirements for hot line and helicopter maintenance in design specifications, a large meeting was called for soliciting advice and recommendations from personnel of helicopter companies specialized in transmission line maintenance, transmission line construction contractors with extensive experience in 500kV transmission line construction, suppliers of hotline maintenance tools, and Southern Company and GTC maintenance. A detailed design specification was developed for use to solicit proposals from tower suppliers for design, analysis, detailing, testing and supply of latticed steel towers. Following suppliers were contacted initially for pre-qualification:

- Fort Worth Tower Inc. – USA
- Locweld Inc. – Canada
- Falcon Steel Company – USA
- Fabrimet – Canada
- Thomas & Betts – USA
- SAE Towers – Mexico

- Sisttemex, Inc. – Mexico
- Formet – Mexico
- KEC – India
- Kalpataru – India
- Mitas – Turkey

After reviewing information submitted by the tower suppliers in the pre-qualification, it was found that no American suppliers have engineering and detailing capability to undertake the project. An invitation for proposal was then sent to the remaining suppliers. The following suppliers submitted proposals:

- Thomas & Betts, Fabrimet and Comemsa
- SAE Towers
- Sisttemex, Inc.
- Formet
- Kalpataru

All bidders were evaluated commercially and technically by GTC and Southern Company. Evaluations included contract terms and conditions, engineering capability, pricing, and review of facilities. SAE Towers and Kalpataru were short listed for further evaluation. Their proposals and preliminary designs were further scrutinized, which included a numerical weighted evaluation, an exchange of additional information, and a formal interview conducted by conference calls. SAE Towers was finally selected as the winning bidder.

TOWER MATERIALS AND DESIGN

Tower materials were required to be new and undamaged and conform to AISC (AISC, 2005) and ASTM standard specifications such as ASTM A36 or ASTM A572, ASTM A394 or ASTM A325. The use of different steel grades were considered in the design. The capacity of tension members are directly proportional to steel grade used while the capacity of compression member is dependant on slenderness ratio KL/r . For long and slender compression members, it is common to use base quality structural steel ASTM A36. In general, weight is the dominant factor in the cost of a latticed steel tower (EPRI, 1990). The design that weighs the least will in general cost the least. However, many other factors such as detailing, bracing schemes, fabrication, handling, shipping and erection needed to be evaluated. Another cost factor that needed to be considered was member duplication. Since fabrication of latticed steel tower is a production line type operation, time and labor spent on setup on the shearing and punching equipment for each member could well impact the bottom line of total costs. It was noticed in the evaluation that designs submitted by Kalpataru, India have significantly more pieces than designs submitted by other bidders or similar designs done by domestic consultants or tower suppliers. This could be because labor costs in fabrication and erection does not have significant impact on cost in the designs. Each piece in latticed steel towers must be handled a number of times during fabrication, shipping and erection, therefore the number of pieces in a tower design is a significant cost factor. In addition, any operation that requires hand work or individual inspection such as welding, drilling, or bending is more costly than the production operations of shearing and punching. These factors should be considered in the economic evaluations.

The bolt size and grade are critical factors in the structural strength and economics of

tower design. It is common practice in the United States to use only one diameter and grade of bolt in any single tower design. ASTM A394 Type 0 and Type 1 bolts are the most commonly used bolts for transmission line towers in North America. However, considering more and more transmission line towers are designed and/or supplied by overseas fabricators, it may be desirable to specify ASTM A307 and A325 bolts for transmission line towers. Because ASTM A307 and A325 bolts are widely used in steel structural joints of buildings and bridges, the ASTM A307 and A325 specifications are better known internationally. These bolts are more readily available and generally have better quality assurance and quality control process.

Transmission tower connections are designed as bearing type connections. Decision on the design of connections with threads included or excluded from the shear plane could have significant impact on costs. If the connections are designed with threads included, then all threaded bolts could be used to provide ease of construction. It would also eliminate concerns about possible mistake of installing bolt threads in the shear plane which could lead to significant strength reduction of the connection when the design called for excluding threads in the shear plane. However, connections designed with threads included would require more bolts in the connection and increase number of gusset plates used in the connections. Gusset plates are always expensive, and sometimes they will increase the eccentricities in the connections. In addition to the cost of furnishing, fabricating, handling and erecting the additional piece, each gusset plate usually requires a few extra bolts and fills or washers which contribute more adverse economic comparison (EPRI, 1990). It should be a major goal in design and detailing to minimize the number of gusset plates used in a tower.

It was required in the design specifications that the transmission line structures to be designed in accordance with the ASCE 10-97. The loads, configuration and electrical clearances were provided to the bidders in loading and configuration drawings. The structures furnished under the design specifications included a family of following latticed steel towers:

- DSS-T 50 – 125 ft, self-supporting tangent structure, 0°-5°
- DSS-HT 50 – 135 ft, self-supporting heavy tangent structure, 0°-5°
- DSS-SA 50 – 120 ft, self-supporting small angle structure, 5°-15°
- DSS-LA 50 – 120 ft, self-supporting large angle structure, 15°-30°
- DSS-DE1 65 – 125 ft, self-supporting deadend structure, 0°-45°
- DSS-DE2 65 – 125 ft, self-supporting deadend structure, 45°-115°
- DGM-T 50 – 125 ft, guyed mast tangent structure, 0°-5°
- DGM-HT 50 – 135 ft, guyed mast heavy tangent structure, 0°-5°

After reviewed the preliminary designs submitted by the bidders, it was found that the structure DSS-DE2 was extremely heavy because it was designed to accommodate a very large line design angle. Past experience in Southern Company and GTC 500kV transmission line designs shows it is very rare in line design that any structure would have line angle greater than 90°. Therefore it was decided to reduce design line angle of the structure DSS-DE2 to 90° and add another design of DSS-DE3 structure with line angle 90°-115°. This would provide more economic utilization of all structures.

It was expected that all bidders would put sufficient effects on designing the lightest towers possible in their proposals. However, it is extremely important for the owner to keep in mind that an economical tower design is a compromise between tower costs and foundation costs. The basic guidance is that a narrower base will lead to a lighter tower, but larger foundation reactions. While a wider base will in general reduce the foundation reactions, but may require longer, more costly tower bracing members and lead to a more expensive tower.

After initial evaluation of design proposals submitted by the bidders, it was found that further design optimization would be necessary taking into consideration of foundation reactions and costs in the tower designs. Request for additional design information was made to the two short listed bidders. The two bidders redesigned three towers selected by GTC to provide information on tower weights and foundation reactions under various tower base widths. After receiving the information on the revised tower base width and foundation reactions, foundation designs using EPRI’s CUFAD (Compression and Uplift Foundation Analysis and Design) were performed. A cost comparison for different widths of tower base taking into consideration of both tower and foundation cost was then carried out. The data used is summarized below.

- The foundation designs were based on two soil borings obtained in a project area where two 500kV transmission lines will be built in the near future. The first soil boring was categorized as "Good Soil" with a blow count $N = 22 - 34$, unit soil weight = $120 - 125 \text{ lb/ft}^3$, soil friction angle = $32 - 37$ degrees and no water table. The second soil boring was categorized as "Average/Poor Soil". The top forty-seven feet soil has blow count $N = 8 - 35$, dry unit soil weight = $115 - 120 \text{ lb/ft}^3$, soil friction angle = $30 - 32$, water table depth at 10 feet.
- Tower base widths considered: Tangent tower: 36 ft, 40 ft and 45 ft
 Large angle tower: 40 ft and 48 ft Deadend tower: 46 ft, 50 ft and 52 ft
- Tower costs:
 - Tangent tower steel: \$0.843 / lb Deadend tower steel: \$0.951 / lb
 - Large angle tower steel: \$0.910 / lb Steel erection cost: \$0.900 / lb
- Concrete pier foundation cost was assumed to be \$600 per cubic yard, including concrete, reinforcement and contractor labor and materials.
- Uplift and compression foundations were designed differently for angle and deadend towers. The minimum foundation diameter considered was 3 feet and maximum burial depth was 40 feet. Under same loading and soil condition, a smaller diameter and deeper foundation is in general cheaper than a larger diameter and shallower foundation.

The cost comparison results were summarized in the table below.

Table 1: Cost Comparison for Different Widths of Tower Base

Tower Type		Tangent		Large Angle	Deadend	
Base Width Increase (ft)		36 to 40	36 to 45	40 to 48	46 to 50	46 to 52
"Good Soil"	Cost (\$) Steel & Erection	1339 (↑)	4468 (↑)	3244 (↑)	2701 (↑)	4081 (↑)
	Foundation	-720 (↓)	-1680 (↓)	-11160 (↓)	-11160 (↓)	-20400 (↓)
Total Cost Difference (\$)		619 (↑)	2788 (↑)	-7916 (↓)	-8459 (↓)	-16319 (↓)
"Avg./Poor Soil"	Cost (\$) Steel & Erection	1339 (↑)	4468 (↑)	3244 (↑)	2701 (↑)	4081 (↑)
	Foundation	-1680 (↓)	-7680 (↓)	-25800 (↓)	-23040 (↓)	-33840 (↓)
Total Cost Difference (\$)		-341 (↓)	-3212 (↓)	-22556 (↓)	-20339 (↓)	-29759 (↓)

The cost comparison shows that increasing tower base width to reduce foundation reactions will result in total savings except for tangent tower in “good soil” area. The savings could be quite significant for large angle and deadend towers located in “average/poor” soil areas. It was found that the deadend tower with 46 feet base width would require the uplift foundations in “average/poor” soil area to be designed with a diameter of 10-11 feet and a burial depth of over 40 feet, which would be a very expensive and impractical construction.

A guidance to determine the width of tower base could be to set the slope of the post legs of tower so the theoretical intersection of their extensions is at the elevation close to the centroid of the horizontal loads of the worst and the second worst design loading conditions. This design would provide following advantages (EPRI 1990):

- The post legs of the tower body form almost a true “A” frame and the horizontal shear bracing will be minimal.
- Forces in the post legs will be fairly uniform from ground line to tower waist.
- Foundation reactions will be fairly constant from the shortest to the tallest tower.

TOWER MODELING

It is a standard practice to model latticed towers as ideal elastic three-dimensional trusses with pin connection at joints. Tower members are considered as axially loaded tension or compression truss elements. Moment due to eccentricities is assumed to be small and is not calculated in the analysis.

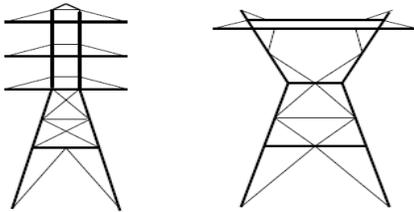


Figure 2 Beam Elements (Thick Lines)

PLS-TOWER is a specialized computer program for analysis of latticed transmission towers and is widely used by utilities around the world because of its compatibility to transmission line design software PLS-CADD. All computer programs used in analysis of

latticed transmission towers almost exclusively modeled the tower members as truss elements until recently, PLS-TOWER now recommends the use of beam elements to stabilize planar joints and mechanisms. PLS-TOWER recommends that all tower members be modeled as beam elements, except diagonals and single horizontal struts as shown in Figure 2 (PLS-TOWER, 2008).

PLS-TOWER emphasized that the main purpose of using beam elements is to stabilize planar joints and mechanisms. Beam elements, in addition to axial tension and compression can carry shears and moments, but they are not intended to be loaded in bending and there is no design check for moments and no moment report, except for the moment results table that can be generated using “**Model/ Results/ Moments for Angles Modeled as Beams**”. For design purposes, the beam members are still assumed to be loaded axially. The beam elements can be used to stabilize the model, but the model should still be triangulated (PLS-TOWER, 2008).

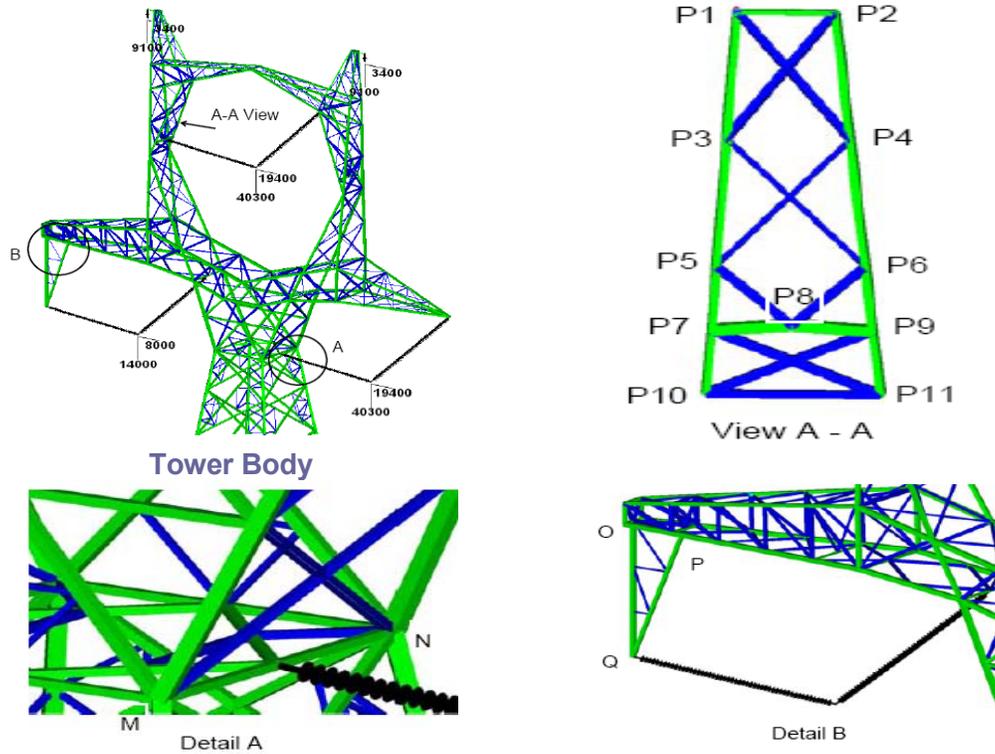


Figure 3 Tower Modeling Examples

Modeling examples in Figure 3 illustrate that it is extremely important for anyone using PLS-TOWER design latticed structures to have a complete understanding of the purpose of beam elements used in PLS-TOWER. It is equally important for anyone designing latticed structures to have understanding and knowledge of structural behavior when creating a valid model. Because beam elements in TOWER can carry shears and moments, and no design check for shears and moment is performed, mistakes of misusing beam elements in TOWER could be disastrous. It is an imperative to check the moment results generated in “**Model/ Results/ Moments for Angles Modeled as Beam**” and any other warnings in TOWER to ensure that no beam elements are carrying large moment.

Figure 3 shows an upper part of a small angle tower. Most members were modeled as beam elements. In “Detail A” the member MN (L3 x 3 x 3/16) was modeled as a beam element. The mid joint of the member is the insulator attachment point and has no support in vertical direction, which would generate significant moments in the member. Analysis of the member with significant moment is beyond the capabilities of TOWER. In “Detail B” members OQ (double angle L4 x 4 x 1/4) and PQ (double angle L3½ x 3½ x 3/8) in the swing bracket were modeled as beam elements and attached to members that were modeled as beam elements as well. This model simulated the swing bracket as a rigid frame which was not what the design intended. It is obvious that the model had flaws and violated “the beam elements are not intended to be loaded in bending...” stated in the TOWER user’s manual. When the model was analyzed with TOWER in late 2007, the output showed that the member MN was loaded to 76% of its capacity and member PQ was at 56% of its capacity under the worst design loading. No warning

or any other message was given in the output. However, when the tower was tested in early 2008, both members MN and PQ failed prematurely.

The misuse of beam element shown in the examples of “Detail A” and “Detail B” are pretty obvious for someone with experience in tower design. But sometimes it can be quite intricate. In “View A-A” of Figure 3, horizontal and leg members were modeled as beam elements except members P10P11, X-bracing and V-bracing were modeled as truss elements. At the attachment point P8, two V-bracing at inside and outside panel was formed to provide support at all direction. The model was properly triangulated. No obvious flaw or violation to the recommendations and guidelines provided by the TOWER was found. However the result of modeling horizontal and leg members as beam elements is significantly different to the result when these members are modeled as truss elements. Table 2 shows the result of member force in term of its capacity.

Table 2 Member Capacity Comparison: Beam Elements vs. Truss Elements

	Member Force as % of Its Capacity										
Beam Element Model	P1P2	P1P3 P2P4	P3P5 P4P6	P5P7 P6P9	P7P10 P9P11	P1P4 P2P3	P3P6 P4P5	P5P8 P6P8	P7P11 P9P10	P7P8 P8P9	P10P11
	87	48	36	57	97	83	91	33	21	57	33
Truss Element Model	P1P2	P1P3 P2P4	P3P5 P4P6	P5P7 P6P9	P7P10 P9P11	P1P4 P2P3	P3P6 P4P5	P5P8 P6P8	P7P11 P9P10	P7P8 P8P9	P10P11
	238	112	100	80	140	220	213	65	93	179	192

The result in Table 2 shows that member design is within its allowable capacity in beam element model that simulates a rigid frame panel, whereas in truss element model majority of members are significantly overstressed. When the bracing is insufficiently stiff to provide proper support, the members modeled as beam elements shown carried moment and shear, but no design check for shear and moment is performed in TOWER. This had raised concerns about using beam elements in tower models, especially when the tower was modeled by someone with limited experience in modeling. The concerns were expressed to PLS in early 2008. Suggestions were made to PLS for adding a warning or a pop-up message when TOWER detects significant moments in members modeled as beam elements. The TOWER program released after April 2008 has an additional warning now listed when beam elements modeled are carrying significant moment. A graphic warning was also added in 2008. The graphic warning is a very helpful tool to ensure that the beam elements were used properly in TOWER. The tower designer should exercise extra caution and heed any warnings issued when beam elements are used in the tower model.

DETAILING

The tower designer is responsible for ensuring structural integrity when the drawings are created by detailers. The drawings used for fabrication of tower should be consistent with the model. Moments due to framing eccentricities are generally not calculated in the design model. The detailer should minimize the framing eccentricities in detailing processes so that no significant moments would be generated. When framing eccentricities are unavoidable at some connections, proper design checks and calculations shall be performed to adjust the model and member selection accordingly.

Figure 4 shows a typical deadend tower body model. The “Detail A” shows detail of insulator attachment. The loads applied at point O will generate moments on member P and the bracing connected to the joint. Proper member design taking into consideration of moments due to eccentricities is necessary.

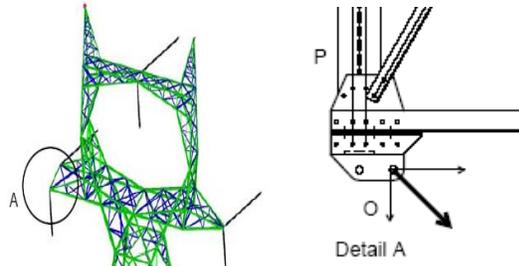


Figure 4

The quality of details is a major factor contributing to the strength of a tower. The designer and the detailer should continuously work together during the preparation of the fabrication drawings which would largely determine the economy of a design.

TESTING

Full scale structural tests were performed for six out of nine towers designed. The other three towers (DGM-T, DGM-HT and DSS-DE3) will be tested later when they are used in a project. The tests were set up to conform to the design conditions and to validate the adequacy of the individual members and their connection designs to withstand the specified loading conditions. The tower tests perform as the acceptance checks. If there is no premature failure, the design is assumed to meet the minimum strength required.

Nine design loading conditions were specified for testing all six towers. The loading sequence for each tower test was determined so that the load cases having the least influence on the results of successive tests were performed first. Simplifying the testing operation was also considered in selecting the loading sequence. Testing loads were applied to 50, 75, 90, 95 and 100% of the factored design loads. At 50, 75 and 90% the load were held for two minutes, and at 95 and 100%, the loads were held for five minutes. All six towers were required to be tested to destruction at 115% of the factored design loads selected as the last testing load. Five towers tested passed 115% of the worst loading case selected. One tower failed at approximately 110% of the worst loading case selected. It was learned that five to six test loading conditions should be sufficient to validate the towers designed, and the nine loading conditions specified were not necessary for the tests of single circuit latticed steel towers.

It was required the test structure be constructed of the same material and by the same type of fabrication as in the production run. The maximum yield for ASTM A572/A572M Steel (Grade 50) members used in the test structure was 58 ksi. At the time of preparing the material for the test structures, the tower supplier was reportedly unable to find material for some tower members with a maximum yield lower than that specified. It was agreed that as a minimum, the requirement of a maximum yield of 58 ksi for ASTM A572/A572M Steel (Grade 50) shall apply to the members designed for only tension loading, and compression members with KL/r less than 120, and the members were designed to a stress over 80% of their capacity.

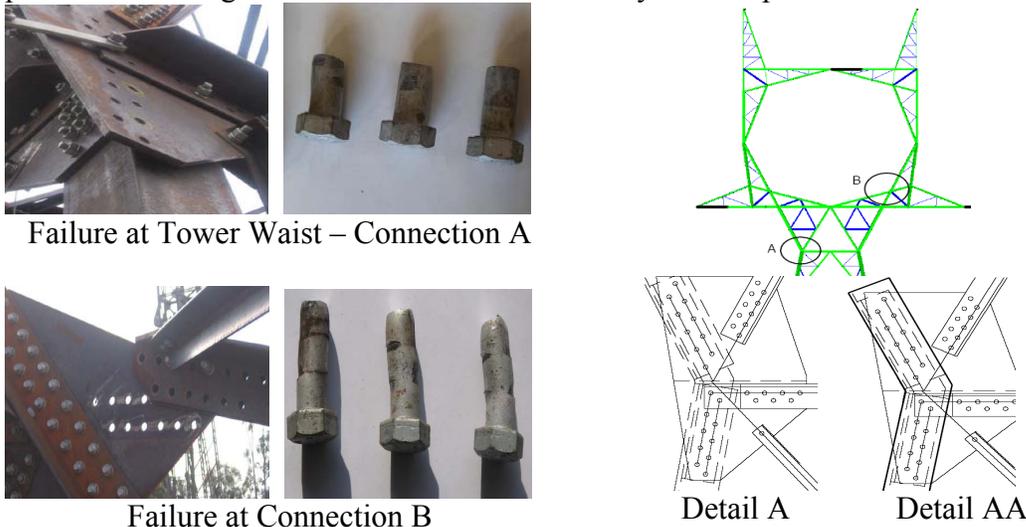
In all six towers tested, the first two tangent towers passed the tests with no premature

failure. Over a dozen premature failures occurred in the tests of next four towers, which were more than it would expect normally. All failures can be attributed to either detailing or modeling errors. Detailing errors occurred when double angles were used in the analysis, but not properly detailed contributed to several failures.



Figure 5

Figure 5 shows one failure where V-bracing was designed with back-back double angles, but the detail in View A-A did not properly design the sufficient bolts at connections between the V-bracing and the gusset plates. There was also a premature buckling failure when double angles were used in X-bracing, but only a single angle was shown on the drawings. Detailers should pay special attention when double angles are used. The tower designer should work closely with the detailers and perform a thorough review of all details when they are completed.



Failure at Tower Waist – Connection A

Failure at Connection B

Figure 6

Figure 6 shows two failures occurred at two different connections when testing a deadend tower. The failed bolts in Connection A showed clear cut-off surfaces at the shear plane, which indicates the connection was significantly under designed. The connection was designed using AISC method for eccentric loads on fastener groups assuming each bolt group would withstand the force that connected to the member. Since the leg member was not continuous at the waist (see Detail A), the above assumption was not accurate. The combined forces from different members on each bolt group should have been considered. Detail AA shows an improved detailing with leg continuous above the tower waist.

The failure occurred in Connection B was quite different. The failed bolts showed clear necking at the failed surface. Deformation at shear planes was observed, which could be due to lack of hardness of the bolts used, or the shear planes slipped significantly when the test loads applied. The failure appeared to be tension failure due to prying action. Tower connections are designed as bearing type connection. To avoid prying action at connection, it would be a good practice to use high strength bolts in connections that connect high strength steel and are heavily loaded.

SUMMARY

The proliferation of demands for reinforcing extra high voltage transmission line networks in recent years and for complying with updated codes and standards requires many utilities in the United States to develop new families of latticed steel structures. Design of latticed steel transmission towers is a challenging task which involves lengthy modeling and detailing of significant amount of steel members and connections. There are very limited resources in the United States capable of carrying out the detailing and design of latticed steel towers. Selecting experienced designers and detailers are critical for the success and economy of the project.

An economical tower design is a compromise of tower and foundation costs. The foundation costs should be considered when developing tower designs. It is extremely important to thoroughly understand the capability of the software used in the design and modeling of the structures, and to have knowledge of structural behavior. It is the responsibility of tower designer and detailer to ensure structural integrity of the towers designed. Full scale tower tests provide an indispensable tool to validate the adequacy of the structures designed, particularly when the structures were designed by someone with limited experience in the subject.

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