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Stronger.
SteelDay.

AISC Live Webinars

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Course Description

The Structural Stability Game Show!
September 25, 2020

This webinar will feature a panel of engineers and academics, who will present their views on the root cause of three structural collapse case studies. The audience will then be given an opportunity to vote on which cause was most likely. Finally, the moderator will reveal and explain the true nature of the collapse.

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Learning Objectives

- Identify sources of instability for several structural configurations.
- Describe methods for how to approach stability analysis computationally and experimentally.
- Show how the structural factor of safety can be significantly impacted if one neglects considering stability.
- Explain the importance of communication during the design and construction of a project with case studies.

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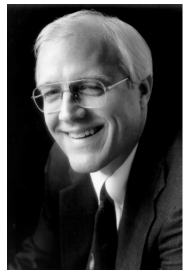
The Structural Stability Game Show!

Cliff D. Bishop
Exponent

Patricia Clayton
UT Austin



Larry Griffis
Walter P. Moore

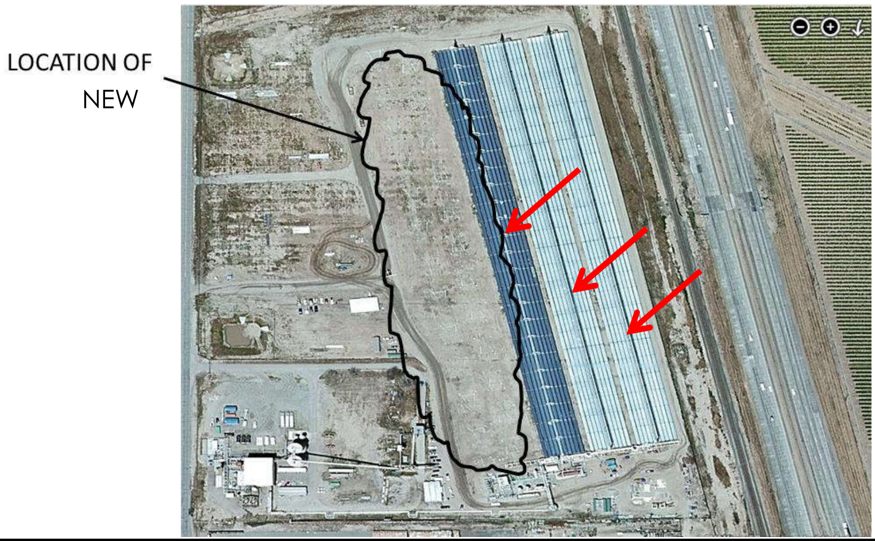
John D. Hooper
Magnusson Klemencic

Ron Ziemian
Bucknell University

Case 1: Solar power support structure

8

Background – Site and Structure



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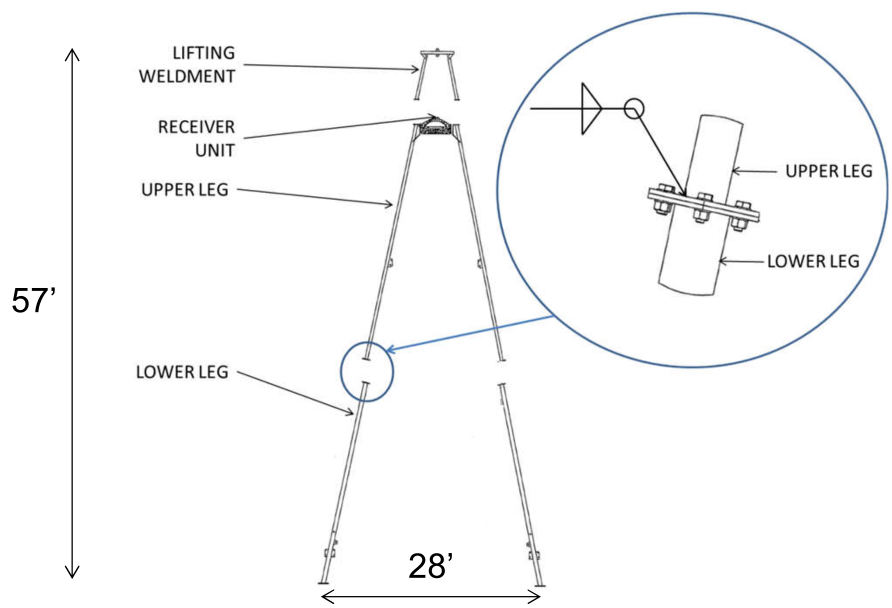
Background – Site and Structure



10

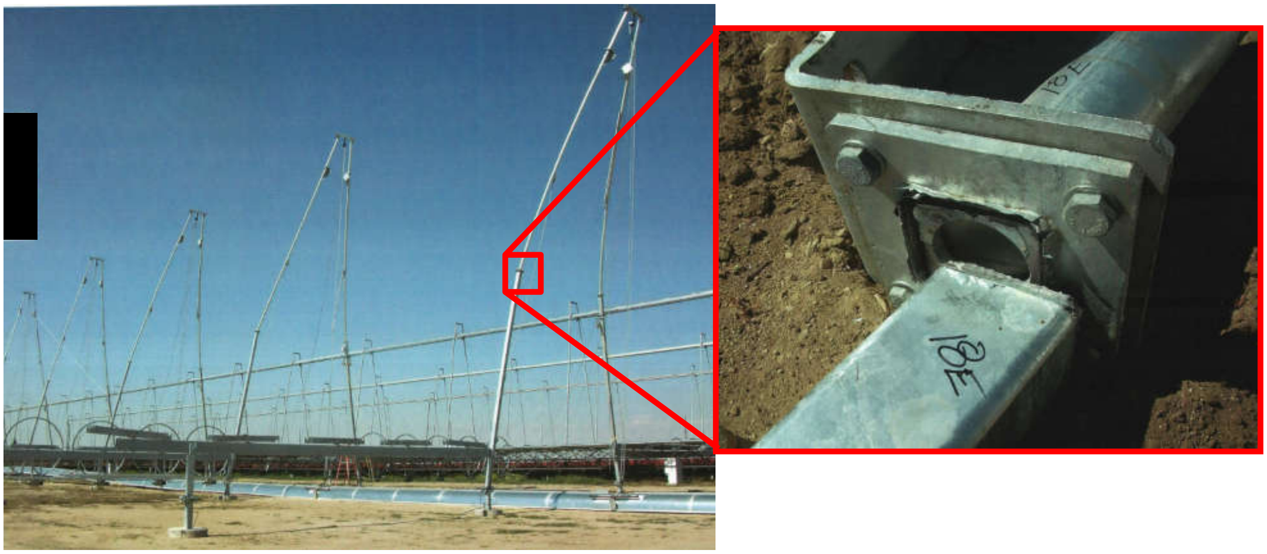
Background – Erection Process

11

Background – The Failure

12

Background – The Failure



13

Contestants’ discussion of root cause

14

Contestants' discussion of root cause

A. Weld failure 

Contestants' discussion of root cause

A. Weld failure 

B. Inadequate design 

Contestants’ discussion of root cause

A. Weld failure 

B. Inadequate design 

C. Poor construction sequencing 

17

Contestants’ discussion of root cause

A. Weld failure 

B. Inadequate design 

C. Poor construction sequencing 

D. Extreme weather 

18

What was the root cause?

A. Weld failure 

B. Inadequate design 

C. Poor construction sequencing 

D. Extreme weather 

Time to Vote!

19

What was the root cause?

A. Weld failure 

B. Inadequate design 

C. Poor construction sequencing 

D. Extreme weather 

Select your answer!

20

What was the root cause?

A. Weld failure 

B. Inadequate design 

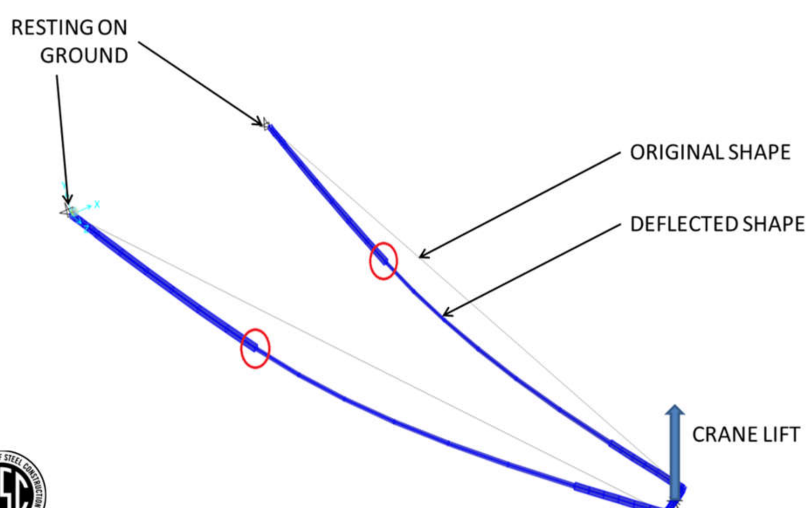
C. Poor construction sequencing 

D. Extreme weather 

21

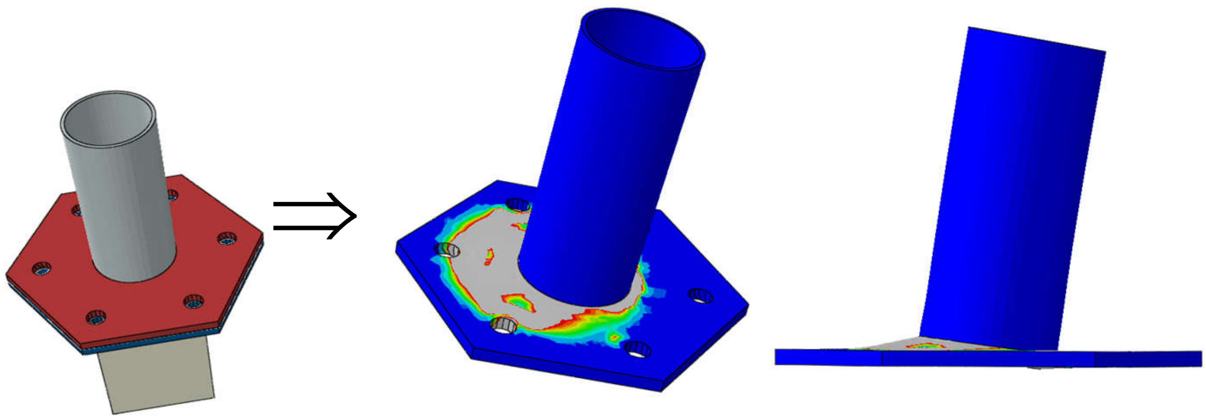
Sufficient weld

max. moment at welds between upper and lower weldment is 95 kip-in.

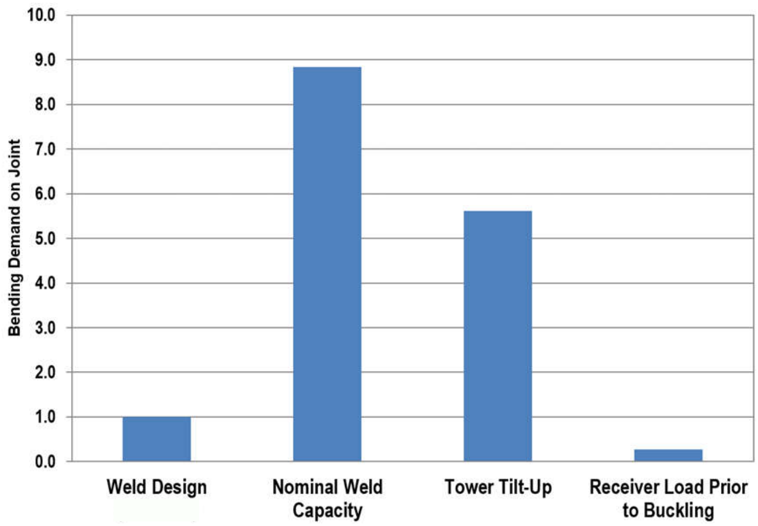


22

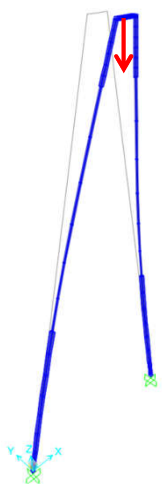
Sufficient weld

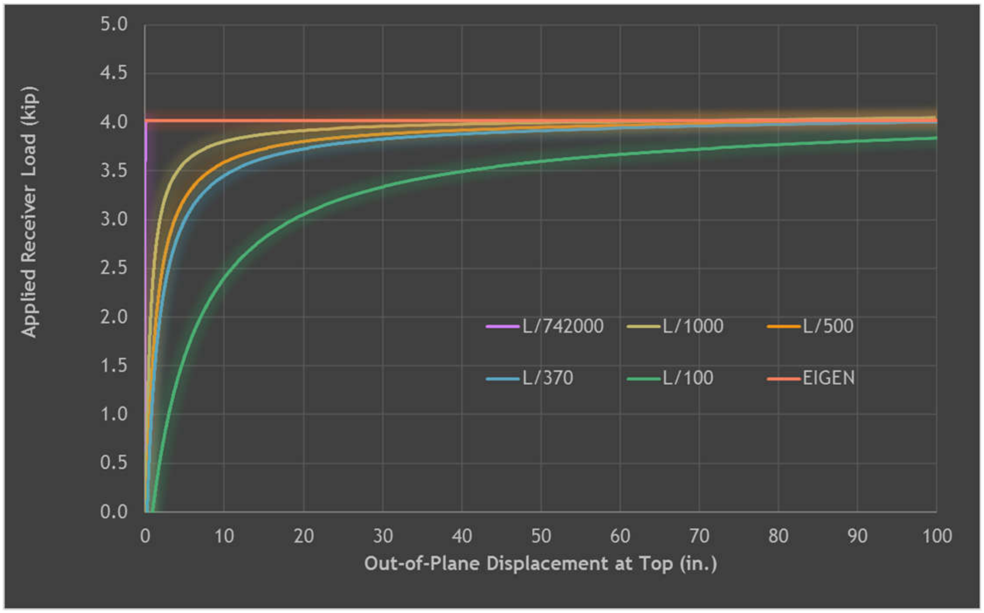


Sufficient weld



Adequate design





25

Adequate design

Model	Top Bracing?	Total Load (kips)	
		Eigen	L-D
Case 1: Self-weight only	Unbraced	17.8	-
Case 2: Receiver + self-weight	Unbraced	6.1	5.6
Case 3: Receiver + self-weight	Fully Braced	30	-

weight of tower + receiver
4.8 kips + 2.1 kips = 6.9 kips

26

Adequate design

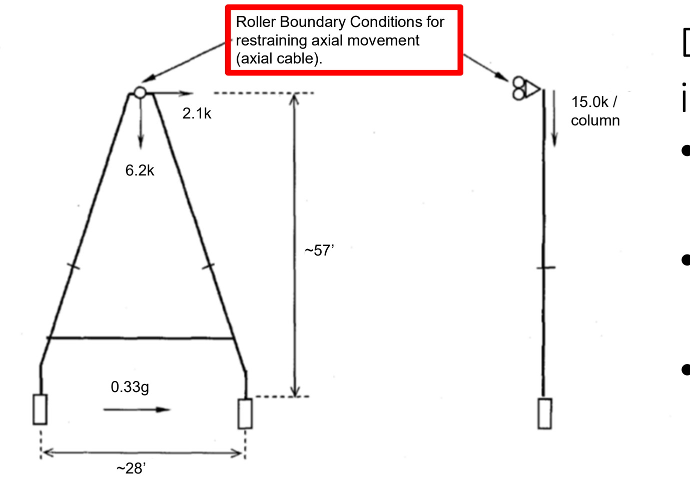


Figure 6.1 – Tower Buckling (G + Seismic X) *Figure 6.2 – Tower Buckling (G + Seismic Y)*

Designer's CD's and calc's included:

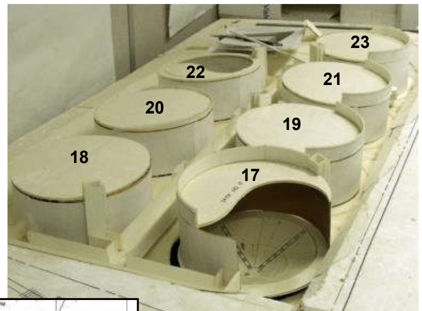
- Analysis of *braced* structure for lifting
- Analysis of *unbraced* for wind only
- Explicit instructions for *inadequate* bracing

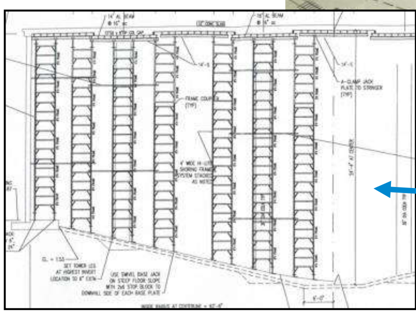
27

Case 2: Dome scaffolding collapse

28

Background





- Project required construction of 7 digester tanks
- Tanks were numbered from 17 to 23
- Dimensions
  - 125-ft diameter concrete tank
  - 50-ft tall
  - 1-ft thick wall and slab
- Base of tanks slopes down towards center
- Aluminum falsework used to support formwork and deck pour

29

Background

At the Time of the Accident:

- Tanks 17 and 18 had already been constructed; roof decks poured without incident
- Tank 19 was ready for concrete deck pour
- Tanks 20 and 21 had the falsework almost fully installed

30

Background

At the Time of the Accident:

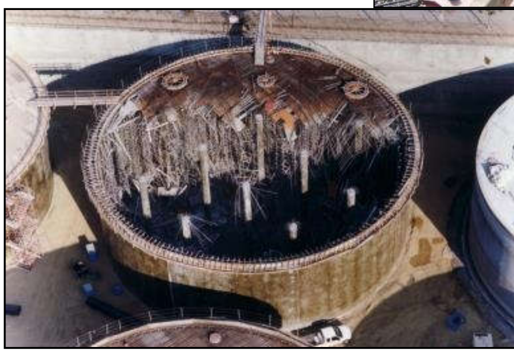
- Concrete roof was being placed in Tank 19, moving south to north
- A little over half of the concrete had been placed

31

The failure

32

The Failure

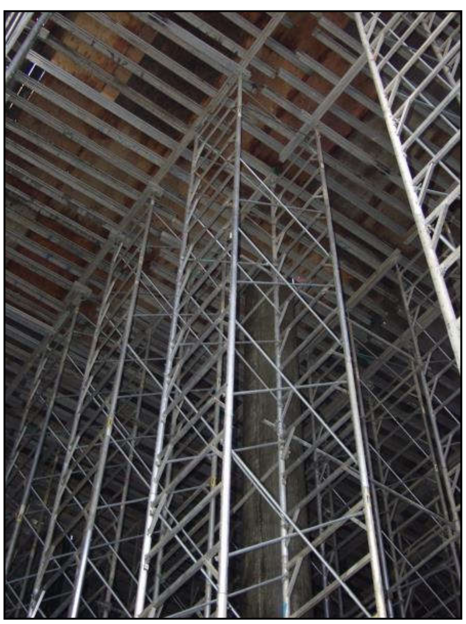


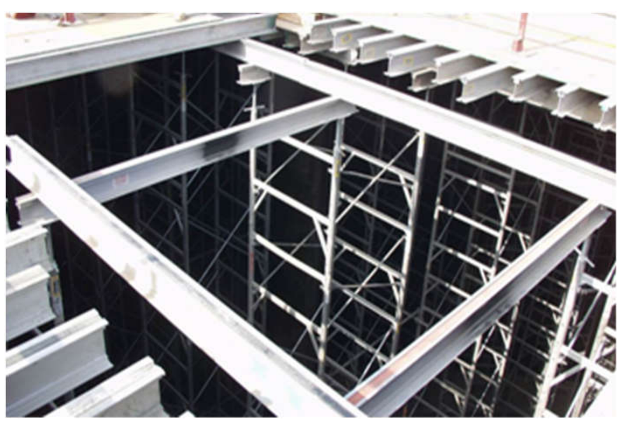
- Falsework supporting the plywood in Tank 19 collapsed
- 29 persons on the roof deck
- 14 workers injured; 4 serious; none fatally injured
- Tanks 19, 20, and 21 were immediately put under Cal OSHA Order to Preserve, pending investigation

33

Scaffold Layout

- Plywood formwork for concrete deck was supported on aluminum beams
- Layers of aluminum beams supported on aluminum shoring towers

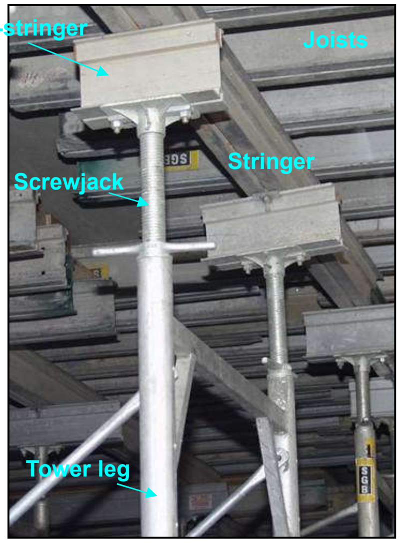


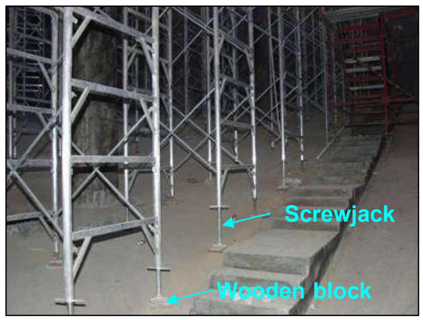


34

Scaffold Layout

- Top and bottom of the towers had screwjacks for adjustable extension
- Many of the towers also had 12-inch or 16-inch long sub-stringers at the top





35

Scaffold Layout

36

Observations – Tank 19

So… What caused the failure?

37

Contestants' discussion of root cause

38

Contestants' discussion of root cause

A. Stringer web buckling 

39

Contestants' discussion of root cause

A. Stringer web buckling 

B. Screwjack extensions 

40

Contestants' discussion of root cause

A. Stringer web buckling 

B. Screwjack extensions 

C. Inadequate bracing 

41

Contestants' discussion of root cause

A. Stringer web buckling 

B. Screwjack extensions 

C. Inadequate bracing 

D. Too much concrete 

42

What was the root cause?

A. Stringer web buckling 

B. Screwjack extensions 

C. Inadequate bracing 

D. Too much concrete 

Time to Vote!

43

What was the root cause?

A. Stringer web buckling 

B. Screwjack extensions 

C. Inadequate bracing 

D. Too much concrete 

Select your answer!

44

What was the root cause?

A. Stringer web buckling 

B. Screwjack extensions 

C. Inadequate bracing 

D. Too much concrete 

45

Failure Modes and Contributors Considered

- Longer screwjacks
- Missing cross-braces
- Slip at base due to missing/failed wood shims at base
- Different joist layout
- No outer frame bracing
- Bracing not tied to the wall
- Weld failure in frame
- Failure of couplers between frames
- Sub-stringer buckling
- 12-inch sub-stringers
- Eccentric installation of sub-stringers
- Overloading of tower legs
- Uneven load distribution
- Unequal screwjack extensions
- Swivel vs. fixed screwjack ends

46

Shoring System Design

TABLE 4 - TOWER CAPACITIES WITH JACKS ONLY OR EQUIVALENT

For 1 to 3 frame (up to 20')high towers, with 12" maximum screwjack extension top and bottom or with jacks extended 12" at one end and extension tubes at the other, extended not more than 14-1/2" using 5 and 6 foot high frames.
NOTE: For towers using only 4' high frames up to four frames high, loads in this table can be increased by 15%.

HEIGHT OF TOWER	SUPER HI-LITE FRAMES C/W INTERFRAME CROSSBRACES ALLOWABLE LOAD PER LEG	SUPER HI-LITE FRAMES WITHOUT INTERFRAME CROSS BRACING ALLOWABLE LOAD PER LEG
One tier	9,500 lb. (4320 kg.)	9,500 lb. (4320 kg.)
Two tier	11,600 lb. (5200 kg.)	9,200 lb. (4180 kg.)
Three tier	12,000 lb. (5443 kg.)	8,800 lb. (4000 kg.)

TABLE 5 - TOWER CAPACITIES WITH JACKS AND EXTENSION TUBES AS INDICATED

For 2 and 3 frame high towers using 4' and 6' high frames completely interbraced with extension tubes and jacks therein at one end extended 12" and with jacks extended 12" or extension tubes extended 14-1/2" in the other end replacing the jacks. (See maximum extension column.)
NOTE: One tier of 5' high frames can also be used in a tower and still interbraced if used as the bottom frame.

If towers are not completely interbraced, reduce all capacities by 25%. For towers only one frame high with extension tubes and jacks as indicated below reduce all load capacities by 15% (as one frame cannot be interbraced). For towers using only 4' high frames. 2, 3, and 4 frames high completely interbraced capacities can be increased by 15%.

	LENGTH EXTENSION TUBES EXTENDED COMPLETE WITH JACKS IN TUBES EXTENDED 12" AT ONE END AND JACKS ONLY OR EXTENSION TUBES EXTENDED 14-1/2" AT THE OTHER END	MAX. EXTENSION	SAFE WORKING LOAD FOR EACH LEG USING 30" EXTENSION TUBES PLUS JACKS EXTENDED 12"	48" EXT. TUBES PLUS JACKS EXTENDED 12"
A	EXT. TUBES 8"+24" of 2-jacks =	32"	10,500 lb (4760kg.)	10,500 lb. (4760kg
	EXTENSIONS 14"+24" of 2-jacks =	38"	9,500 lb.(4300kg.)	9,700 lb. (4400kg
	AT ONE END 20"+24" of 2-jacks =	44"	7,500 lb.(3400kg.)	8,500 lb. (3860kg
	ONLY 26"+24" of 2-jacks =	50"		8,000 lb. (3630kg
	32"+24" of 2-jacks =	56"		7,500 lb. (3400kg
	The following "B" Section is for towers with extension tubes and jacks at both ends of a tower or if no jacks are used at one end, the extension tubes can be extended 14" more replacing the jacks, providing they are long enough.			
B	EXT. TUBES 8"+ 8"+24" of 2-jacks =	40"	9,500 lb.(4310kg.)	10,000 lb. (4535kg
	EXTENSIONS 14"+14"+24" of 2-jacks =	48"	7,000 lb.(3175kg.)	7,500 lb. (3400kg
	AT BOTH 20"+20"+24" of 2-jacks =	64"	5,000 lb.(2270kg.)	5,700 lb. (2585kg
	ENDS 26"+26"+24" of 2-jacks =	76"		4,500 lb. (2040kg
			Not recommendable without auxiliary bracing.	

NOTE: 30" EXTENSION TUBES MUST NOT BE EXTENDED MORE THAN 20-1/2".
48" EXTENSION TUBES MUST NOT BE EXTENDED MORE THAN 32-1/2".
SAFETY FACTOR = 2.5 BASED ON ULTIMATE TEST RESULTS

- Maximum applied load per leg = 9,000 lb
- Allowable load per leg = 8,800 lb
- From Table 4 in Appendix G
  - 3 tier tower
  - Maximum 12-inch screwjack extension
- Factor of safety = 2.5
- Minimum ultimate capacity per leg = 22,000 lb

47

Shoring System Design

ALLOWABLE LOAD PER LEG

1 Frame High	8,100#
2 Frames High	7,000#
3 Frames High	7,500#

IGN AND GENERAL NOTES, SPECIFICATIONS AND TYPICAL DETAILS
FOR SUPER HI-LITE HEAVY DUTY ALUMINUM SHORING SYSTEM

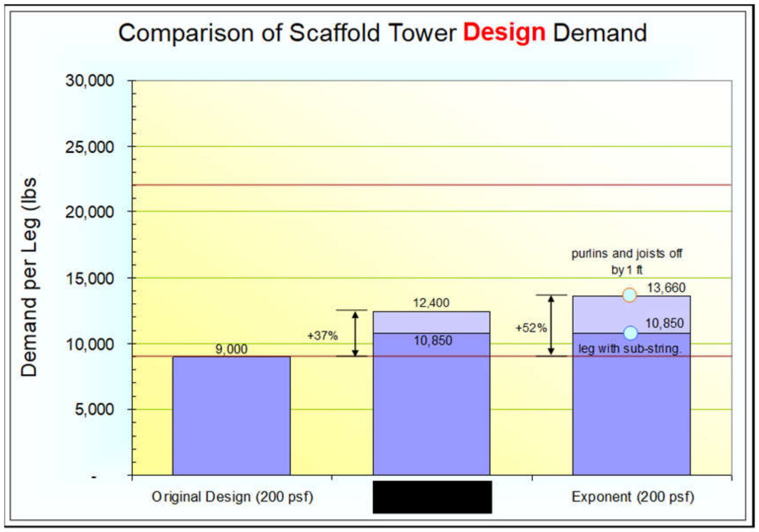
Patent Construction Systems

RECOMMENDATIONS AND INSTRUCTIONS NO.
SS-660

- Design drawings specified
  - up to 9 tier towers with no outer frame bracing
  - 16-inch long sub-stringers on top of tower legs
- The effect of the sub-stringers on load capacity of the towers was not calculated.

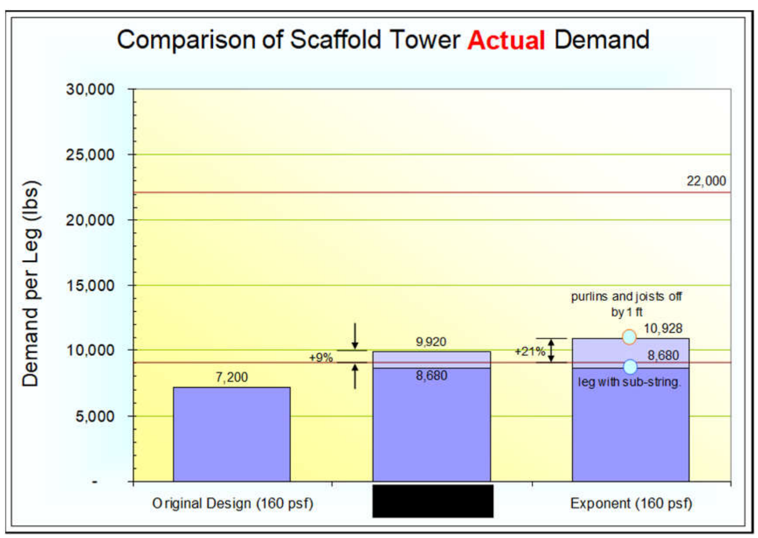
48

Design Loads (200 psf)



Design load included a 40 psf live load in addition to concrete and equipment weight

Actual Loads (160 psf)



Actual loads still considerably higher than original design load of 9,000 lbs

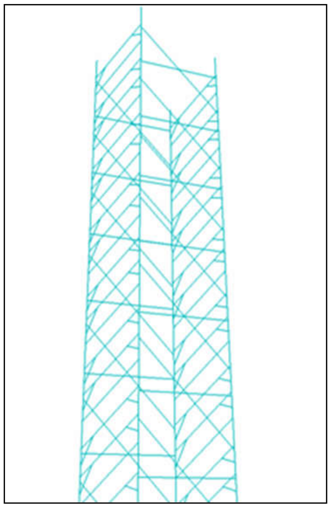
Full-Scale Field Testing

- Conducted full-scale field testing of exemplar shoring
- Towers were incrementally loaded to failure
- Failure due to sub-stringer web buckling leading to global buckling of scaffold towers
- The peak applied loads on legs with sub-stringers were
  - 14,850 lbs (Tower 1)
  - 13,750 lbs (Tower 2)
- Failure loads significantly below design ultimate of 22,000 lbs

51

52

Finite-Element Analysis

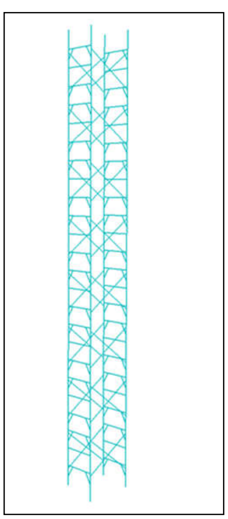


A finite element model of shoring tower was prepared in ABAQUS

The model was used to study the effect of various factors on the strength and stability of the shoring towers

53

Finite-Element Analysis

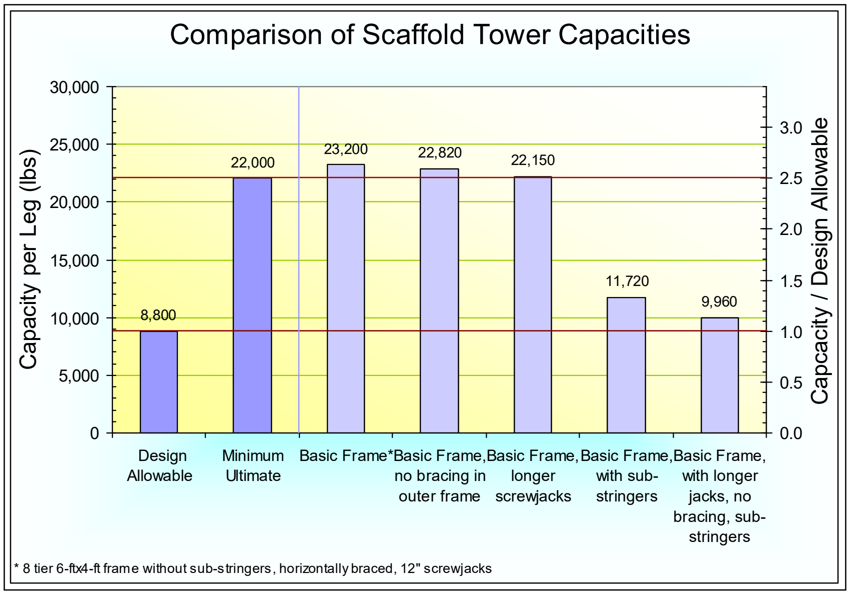


Factors investigated using F-E model

- Longer screwjacks
- No outer frame bracing
- Sub-stringer buckling
- Uneven load distribution
- Missing cross-brace
- Unequal screwjack extensions
- Missing/failed wood shims at base
- Weld failure in frame
- Failure of frame couplers

54

Finite-Element Analysis



Sub-stringers are the primary contributor to reduction in scaffold capacity

55

Case 3: Building roof collapse

56

Background

57

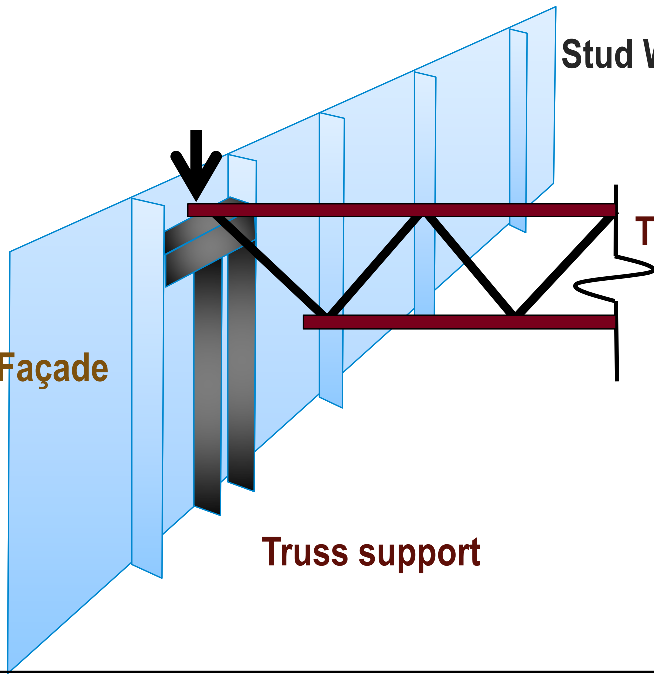
The Failure

58

The Failure

59

Design



60

As-Built

61

The Failure

62

Contestants' discussion of root cause

Contestants' discussion of root cause

A. Freak storm (overload) 

Contestants’ discussion of root cause

A. Freak storm (overload) 

B. Missing support (improper construction) 

65

Contestants’ discussion of root cause

A. Freak storm (overload) 

B. Missing support (improper construction) 

C. Local member effects 

66

Contestants' discussion of root cause

A. Freak storm (overload) 

B. Missing support (improper construction) 

C. Local member effects 

D. Improper design 

What was the root cause?

A. Freak storm (overload) 

B. Missing support (improper construction) 

C. Local member effects 

D. Improper design 

[Time to Vote!]

What was the root cause?

A. Freak storm (overload) 

B. Missing support (improper construction) 

C. Local member effects 

D. Improper design 

Select your answer!

What was the root cause?

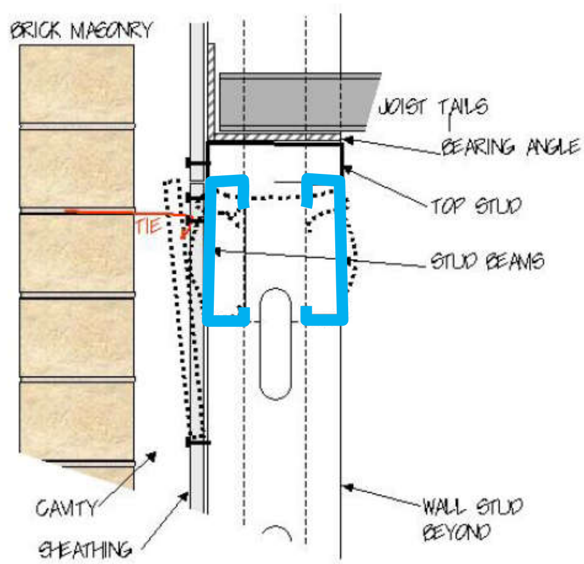
A. Freak storm (overload) 

B. Missing support (improper construction) 

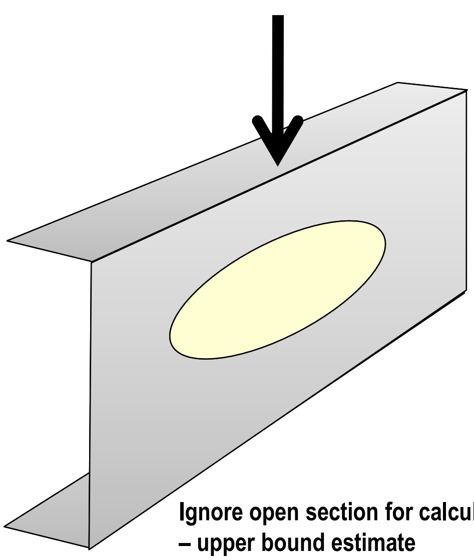
C. Local member effects 

D. Improper design 

Failure Mechanism – web crippling



Failure Mechanism – web crippling



Ignore open section for calculations – upper bound estimate

Description	P (kip)
Allowable Load (ASD)	1.9
Exponent Calculated Capacity	3.5
Non-Factored Loads (service loads)	4.0
BOCA Requirements[1]	6.4
"Expert" Report	7.4 (??)

Building Officials and Code Administrators requires assemblies to withstand 2.5 times live load for testing

Opposing Expert report

[redacted], Ph.D., P.E.
and Associates, PLLC
Professional Engineering Services

June 8, 2006

[redacted]

RE: Supplemental Report, Broadway Business P

Dear Mr. Commander:

This report concerns the ongoing investigation of the r
Broadway Business Park, Oklahoma City, Oklahoma.
Bldg. 240, which is consistent with my prior reports on
collapsed during hail storms and rainstorms as documen
occurred principally at the steel stud wall assemblies th
roof trusses. Subsequent damage was caused to the ma
support the masonry veneer.

The roofs were supported by open web, bar joists that were in turn supported by bearing assemblies built into the steel stud framing walls. Three of these bearing assemblies were removed from the buildings and tested for capacity. The test results indicate that the bearing assemblies as constructed were sufficient to support the design loads that were required at the time of construction of the building. The tests indicate that the capacity of the bearing assemblies was at least 7,429 lbs, which is in my opinion sufficient to support design loads consistent with the governing building code.

Removal of the Bearing Assemblies
In May of 2006, we excavated the wall stud bearing assemblies from three sites within the two buildings at Broadway Business Park. The assemblies are labeled as follows, which are consistent with my original report dated October 4, 2004:

- X S 13 from Building 200
- X N 24 from Building 240
- X N 13 from Building 240

Photographs were taken to document the condition of the wall studs and the bearing assemblies prior to their removal from the buildings. The bearing assembly from N 24, Bldg. 240 is pictured in Fig. 1. In the photograph, one can view water damage to the exterior sheathing that is consistent with the photographs taken in July of 2004.

The bearing assemblies for testing were chosen in that these were assemblies with relatively minor damage compared to most of the bearing assemblies in the two buildings. Even so, all of the bearing assemblies that were chosen and removed exhibited varying degrees of damage prior to testing.

BP 3461

[redacted]

The roofs were supported by open web, bar joists that were in turn supported by bearing assemblies built into the steel stud framing walls. Three of these bearing assemblies were removed from the buildings and tested for capacity. The test results indicate that the bearing assemblies as constructed were sufficient to support the design loads that were required at the time of construction of the building. The tests indicate that the capacity of the bearing assemblies was at least 7,429 lbs, which is in my opinion sufficient to support design loads consistent with the governing building code.

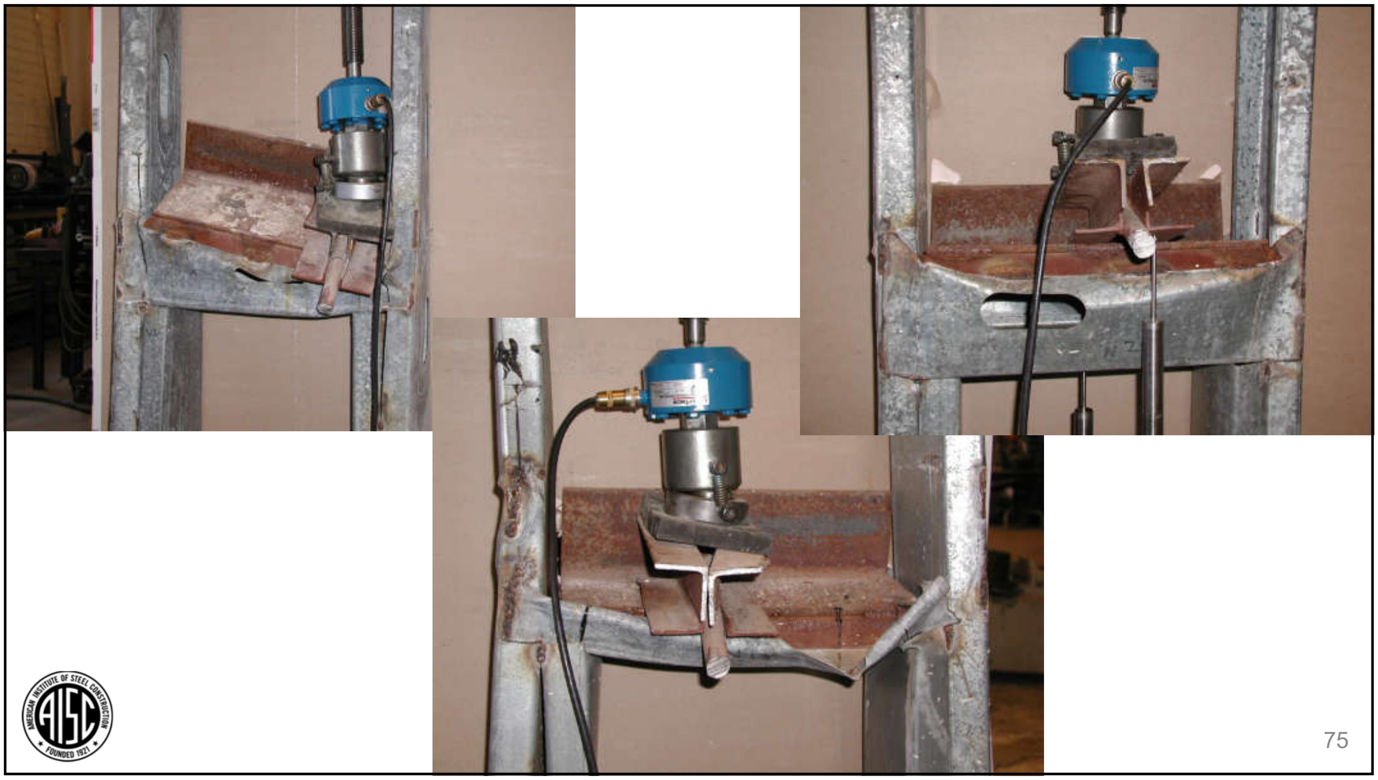
73

Opposing Expert Report

Table 1. Results of Capacity Tests on Bearing Assemblies

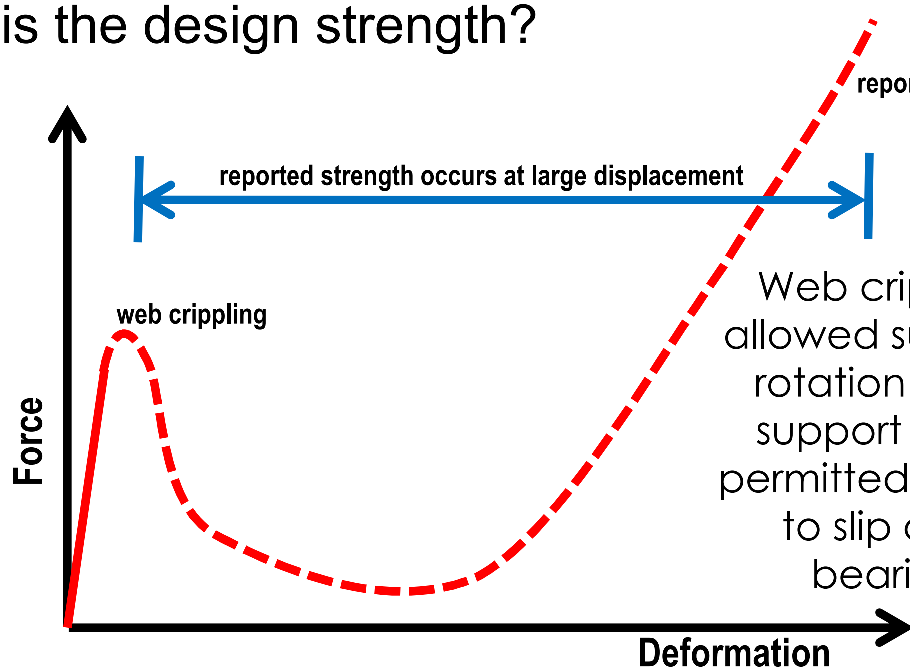
Design Criteria				Test Results		
Design Dead Load (DL) (psf)	Design Live Load (LL) (psf)	Total Design Load (DL + LL) (psf)	Boca 1981 Test Load (psf)	Bearing Assembly	Test Load (lbs)	Equivalent Total Load per Square Foot (psf)
12	20.8	32.8	52	N 13	7429	61
12	20.8	32.8	52	N 24	5167 ??	42
12	20.8	32.8	52	S 13	3942 ?	32

74



75

What is the design strength?



Web crippling allowed sufficient rotation of the support which permitted the joist to slip off its bearing.

76

"For since the fabric of the universe is most perfect, and is the work of a most wise Creator, nothing whatsoever takes place in the universe in which some relation of maximum and minimum does not appear."

Leonhard Euler in *Lineas Curvas (Elastic Curves)*, 1744

Thank you for your participation!

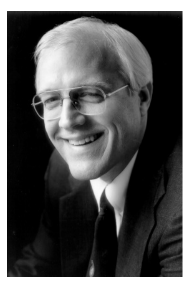
Exponent

SSRC — Structural Stability Research Council — Founded 1944

The Structural Stability Game Show!

Cliff D. Bishop
Exponent

Patricia Clayton
UT Austin



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Magnusson Klemencic

Ron Ziemian
Bucknell University

SteelDay Everywhere

September 25, 2020
aisc.org/steelday

AISC | Questions?

Smarter.
Stronger.
SteelDay.

PDH Certificates

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

Smarter.
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PDH Certificates

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

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AISC | Thank you

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