Design and Construction of High Multitier Shoring Towers: Case Study

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Abstract: Shoring towers are the common formwork solution for high-clearance construction, but there are not many documented cases of extremely high towers. This paper reports on a project in which 60-m-(200-ft-) high shoring towers were used for slab formwork. The paper describes the design and construction of the formwork, with a focus on aspects unique to high multitier towers. The various considerations made are presented, and data and information are provided that may assist practitioners facing similar engineering undertakings. Special attention is given to the organization of the work, in light of the scarcity of data pertaining to shoring towers of such heights. Measured assembly and disassembly work inputs are presented and analyzed, and the validity of a model to predict work inputs in multitier tower erection is examined.

INTRODUCTION

Scaffold-type support towers are the common vertical-shoring solution in formwork for high-clearance concrete construction. Practical aspects of design and construction with shoring towers are covered in the formwork literature (Hurd 1995; Bennett and D'Alessio 1996; Peurifoy and Oberlender 1996; McAdam and Lee 1997), as well as in manufacturers' technical publications. Some more unique aspects of shoring-tower-based design and construction have been the subject of several recent studies (Shapira 1995a,b, 1998; Shapira and Goldfinger 2000). The literature, however, lacks detailed reports on specific cases focusing on shoring towers, in particular the use of exceptionally high towers.

This paper reports on such a case, in which shoring towers were erected to what may be regarded as the upper limit of their height range, that is 60 m (200 ft) (Johnston 1996). The project on which the towers described here were erected is not a typical project that requires a large number of high-rise towers, such as may be expected in a large public or industrial facility, or in bridge construction. On the contrary, the slab to be formed in this case was of small area in a residential building, requiring only a limited number of towers. But the adoption of what may appear initially to be an unlikely solution, together with the exceptional tower height, render the case worth reporting.

This paper is based on a close observation of the case, through all parties involved, including the general contractor (GC) management, site management, site engineering, and formwork supplier and designer. Work studies to determine work inputs were conducted, site functionaries (the project manager, project engineer, and site superintendent) were interviewed, and design and construction shop drawings and other documents were studied.

THE PROJECT

The building is a 25-story-high reinforced concrete structure consisting of two underground parking floors, one entrance floor, 21 residential floors, and one upper mechanical floor. The total height above ground level is 78 m (256 ft). The building maintains a basically square shape throughout most of its height, with footprint dimensions of 29 × 26 m (95 × 86 ft). The walls of the east facade recess 3.7 m (12 ft) along a 12-m- (40-ft-) long section in the middle of the facade, from the ground floor through the 17th residential floor. This recess stops at the 18th floor, where a 12-m-long wall section projects beyond the facade's main vertical plane in a curved line, for a maximum distance of 5.5 m (18 ft) relative to the 17th floor. Thus a 12 × 5 m (40 × 16 ft) cantilevered slab is created 60 m (200 ft) above ground level (Figs. 1 and 2). The critical issue was how to execute this concrete slab. The remaining floors on the east facade, up to the building's top, have the same geometry as the 18th floor.

Other relevant project data are as follows:

- On the east side, the contour of the underground parking floors extends out 20 m (66 ft) beyond the upper structure's footprint. Thus the concrete ceiling of that part of the substructure—at ground level—serves as a staging area during construction.
- A 92-m- (300-ft-) high tower crane is located next to the east side of the building, alongside the middle of the facade, at a distance of 5 m (16 ft) from what would be the projecting wall of the 18th floor. Because of its height, the crane is anchored to the walls of the rising structure. At the point in time when the 18th floor is constructed, anchors are located between the 12th and the 13th floors (Figs. 2 and 3).

ALTERNATIVES

Three solutions were initially considered by the GC for the concreting of the projecting 60-m- (200-ft-) high slab: (1) Precast concrete elements, which would eliminate the need for formwork; (2) cast-in-place concrete, with the forming deck supported by cantilevered trusses one floor below; and (3) cast-in-place concrete, with the forming deck supported by full-height towers. The precast solution was based on using the adjacent stairway shaft concrete structure. A cantilevered wall was to be cast with the shaft that would later serve as support for the precast slab. As was the case, construction of the stairway shaft was independent of and preceded that of the connecting floors; hence it would be at least 2 months before the slab could be attached to the wall, during which time the wall would remain unrestrained. This situation was deemed unacceptable by the structural engineer, because of stability concerns, which led to the rejection of the precast solution at the preliminary stage of the decision-making process.

The two other structurally feasible alternatives were then analyzed and assessed in technical detail, before considering the economic question. The major advantages and disadvan-

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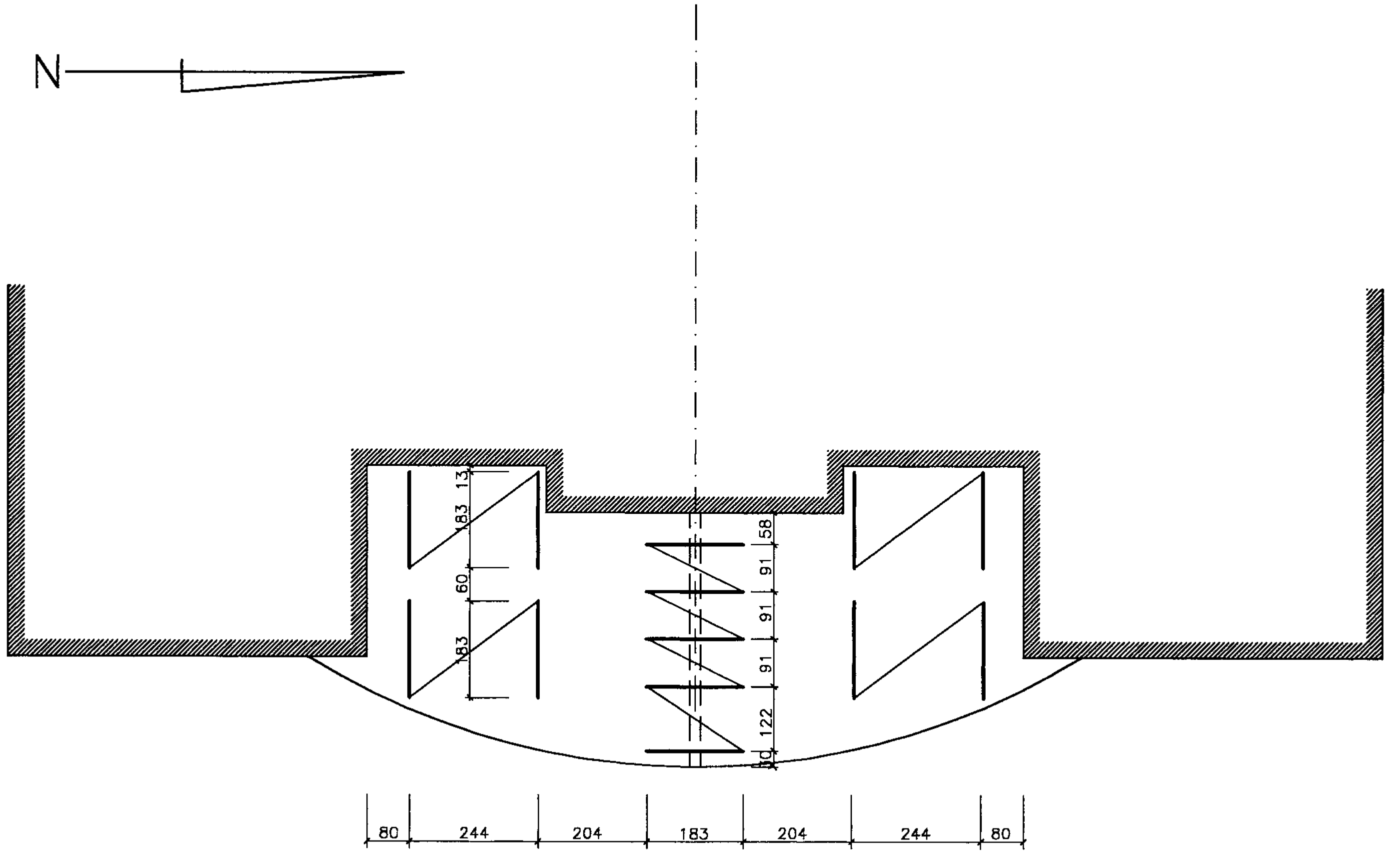

FIG. 1. Slab Dimensions and Tower Layout (Measurements in cm)

tages are summarized in Table 1. Following this comparison, neither alternative appeared to have a clear advantage over the other; therefore each of the two alternatives were taken to a level of design allowing for a preliminary cost estimate, with the aim of selecting the more economical alternative.

The cantilevered truss solution was based on four 12.75-m- (42-ft-) long and 2.5-m- (8-ft-) high steel trusses. The weight of each truss was computed to be 3,500 kg (7,700 lb) or 14,000 kg (30,800 lb) for all four trusses. With the additional weight of the connecting crossbars and the horizontal and diagonal braces, estimated at 6,000 kg (13,200 lb), the overall weight of this temporary structure would reach 20,000 kg (44,000 lb). Based on $1.5/kg ($0.68/lb), including material and labor, the cost of this solution—excluding formwork for the slab itself—was thus estimated at $30,000. All included, the total cost for this solution was estimated at $35,000. Since no reuse of the trusses on another project was expected, and as the residual value for recycling was deemed negligible, 100% of this total would have to be charged to the project. Thus $35,000 was set as the benchmark for the examination of the shoring towers solution.

Formwork companies submitted two different shoring-tower proposals to the GC. The competing tower systems were (a) steel towers, with a working load capacity of 45 kN (10,000 lb) per tower leg; and (b) aluminum towers, with a working load capacity of 80 kN (18,000 lb) per tower leg. In principle, both rental of towers (i.e., material) and labor costs should have been taken into account. The GC, however, had subcontracted the placement of the concrete for a fixed unit price per cubic meter of concrete, regardless of the type of element concreted, while providing all material and equipment. Thus, only rental costs (including design and transportation) were to be included in the comparisons between the two formwork proposals and the cantilevered trusses. Here, in fact, lies the attractiveness—in the present case and circumstances—of the labor-intensive towers-based solution (in which the GC essentially bears only material costs) over the trusses-based solution (in which the GC bears all costs, as the temporary prefabricated steel structure is considered equipment).

At the same time, the concrete subcontractor was expected to negotiate with the GC for some compensation due to the unusual nature of the exceptionally high slab and the resulting high labor cost per cubic meter of that slab. Consequently, the GC had an advantage in selecting towers that would, overall, require lower work inputs. In that respect, the aluminum towers had an added advantage over the steel towers: (1) because of their higher working load capacity, the required number of aluminum towers would be less than that of the steel towers; and (2) work studies conducted with these two tower types, albeit at lower heights and without crane assistance, show that assembly and disassembly work inputs (worker-hours per tower at a given height) are lower for the aluminum towers than for the steel towers (Shapira and Goldfinger 2000).

The two proposals submitted were based on a rental duration of 3 months with a monthly rental rate in the range of 2–3% of the towers' purchase price. With the price of steel towers being considerably lower than that of aluminum towers, this gave steel an initial advantage. However, because the aluminum solution required considerably fewer towers than the steel solution, the aluminum supplier's proposal of approximately $13,000 (transportation included) was 20% lower than that of the steel supplier. With the additional labor advantage, as mentioned above, the aluminum towers were therefore favored for the job.

Although no work input data for the erection of such high towers were available, the GC felt that the $22,000 difference between the two alternatives—trusses and towers—left considerable room for any compensation that might be negotiated by the concrete subcontractor. Consequently, the high towers were selected for the project.

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FIG. 2. General View

DESIGN

Objectives and Constraints

Because of the extreme height of the towers, the major design objective was to produce a temporary structure that would use the minimum number of towers, and yet comply with all strength and stability requirements. Given the expected high erection work inputs, along with the contractor's need to rent the towers, such a solution would initially make this alternative economically competitive, compared with other alternatives. Successful implementation and contractor satisfaction would also pave the way for future application of the solution in other projects having similar characteristics. While this may always be true, it was regarded even more so in the present case, in which the solution concept was likely to elicit skeptical reactions.

Other design conditions and constraints are as follows:

- High load-bearing capacity of the towers: Although the towers will have to be restricted to bearing capacities lower than their maximum allowable loads due to their exceptional height, the aluminum tower's 80-kN (18,000-lb) per-leg working capacity gives this tower type an advantage in securing a high de facto safety factor. Indeed, other extra heavy duty tower types exist that offer much higher working loads, but these much heavier and costlier towers are not used for regular building construction. Leg capacity for common heavy-duty steel and aluminum tower types having the same configuration as the aluminum towers referred to here is in the range of 40–60 kN (9,000–13,300 lb) (Shapira and Raz 2000).
- Crane on site: A tower crane would be available to assist in assembly and disassembly operations. However, its location in close proximity to the designated location of the tower-based temporary structure, together with its anchors, created a geometric constraint that had to be considered in determining the support tower layout.
- The floor on which the towers were to be erected: With proper reshoring, the concrete ceiling of the underground parking floors was considered a suitable rigid surface to support the loads exerted by the towers.
- Winds: The location of the project only a few hundred meters from the coast line and its resulting exposure to strong west winds could be a potential hazard, worsened by the exceptional height of the towers. However, since the towers were to be erected adjacent to the east facade, the permanent structure's walls would act as a windshield for the temporary structure. At the same time, it was necessary to consider provisions against possible suction effects.

Design Stages

Design generally followed the recommended tower-based formwork process (Shapira 1995a), as follows:

Element Selection

The aluminum manufacturer offers tier heights of 150, 180, and 210 cm (5, 6, and 7 ft, respectively). The higher the tower, the greater economic advantage of using the maximum available tier height, since the number of tiers composing the tower affects work inputs considerably more than tower height (Shapira and Goldfinger 2000). Therefore, 210-cm-high tiers were selected. As for deck forming, the manufacturer's proprietary aluminum stringers and joists were used.

Tower Layout

Often, the boundary conditions and the geometry of the slab to be formed govern tower layout, particularly in the case of small-area slabs (Shapira 1995a). In the present case, the shape and dimensions of the slab, together with the constraining location of the crane ties and the need for the smallest possible number of towers, produced (after several trial-and-error iterations) the tower layout shown in Fig. 1. The layout consists of four separate 244 × 183 cm (8 × 6 ft) towers, the largest size suitable geometrically and the second largest size available, and one continuous 395 × 183 cm (13 × 6 ft) 10-leg tower, made up of three 91-cm (3-ft) and one 122-cm (4-ft) modules (see frame detail in Fig. 4).

Another consideration, in determining the tower layout, was to achieve a uniform distribution of the loads on the towers' legs. The exceptional tower height raises a problem, normally ignored in more common height situations, of tower shortening under compression. Uniform tower-leg loading would guarantee uniform shortening, which can be anticipated and handled.

Statical Analysis

On the basis of the proposed tower layout, joist spacing was determined, stringers were selected, and actual loads on each tower leg computed. Statical analysis was performed according to the Israeli formwork standard SI 904 [Standards Institute of Israels (SII) 1998], in a manner very similar to ACI 347R guidelines American Concrete Institute (ACI) 1994]. [Note that unlike ACI 347R, which uses the permissible-stress and working-load method, SI 904 introduces the concept of partial safety factors (LRFD) and limit state design, now adopted by most European countries as well (Shapira 1999); however,

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FIG. 3. Close-up View of Shoring Towers and Bracings

TABLE 1. Technical Comparison of Solutions

Solution (1)	Advantages (2)	Disadvantages (3)
Cantilevered trusses	Simple formwork (one-floor high, above lower deck) Lower deck enables finishing works of concreted slab Stone-covered facade walls remain intact	Extremely heavy structure (loads transferred indirectly, by cantilever) Complex and hazardous assembly operation Difficult dismantling after concreting of slab
Full-height shoring towers	Simple structure (vertical loads transferred directly) Accommodates platforms for finishing works of concreted slab Gradual disassembly can be utilized for repair work and clean up of stone-covered facade walls	Labor intensive Lack of experience with such high towers (design and construction) Need to prevent damage to stone-covered facade walls (from bracings)

proprietary systems, such as the aluminum system under consideration and other tower-based forming systems, are treated in the same manner in both SI 904 and ACI 347R, by the working-load method.] Dead loads were taken as the combined two floors (18th and 19th) to be supported by the towers before formwork dismantling. Calculated leg load was in the range of 48–54 kN (10,700–12,000 lb), and the resulting preelevation of the forming deck, to compensate for tower shortening, was 4 cm (1.5 in.).

Design Miscellaneous

Self-Weight of Forms

While the weight of forms is always a nominal component of the vertical loads on formwork, it is in practice frequently neglected, especially when small in relation to the weight of the concrete plus live load (Hurd 1995). In other cases it is often taken as an additional percentage (commonly 5%) of the combined concrete weight and live load. In the present case, however, the weight of the towers was expected to add significantly more to the vertical loads, and this was an important consideration, due to the reshoring requirements for the underground floor on which the towers were to be erected.

The proposed 183 $\times$ 210 cm (6 $\times$ 7 ft) frame weighs 21 kg (46.2 lb), and one pair of cross braces for a 244 cm (8 ft) wide tower weighs 9.3 kg (20.5 lb). Thus, one tier composed of two frames and two pairs of cross braces weighs 60.6 kg (133.4 lb). One set of four tier connectors, including pins, weighs 3.2 kg (7 lb), bringing the total of one complete tier in a multitier tower to 63.8 kg (140.4 lb), or 1,786 kg (3,930 lb) for a 28-tier tower. One set of four base plates, four tower heads, and eight screw jacks adds 58 kg (127.6 lb), for a total tower weight (lateral bracing not included) of 1,844 kg (4,057 lb), or the addition of 4.6 kN (1,014 lb) to the vertical load on each tower leg. For an average leg load of 50 kN (11,000 lb), as calculated for the present case, this means a 10% increase in the combined concrete weight and live load.

Bracing

Bracing design for lateral stability and buckling control followed the manufacturer's recommendations and general bracing design principles (Brand 1975). Bracing was based on two subsystems: a horizontal one as primary bracing, and a diagonal one as secondary bracing.

Primary braces consisted of 10 horizontal arrays of interconnected 4-cm- (1.5-in.-) diameter steel tubes, positioned

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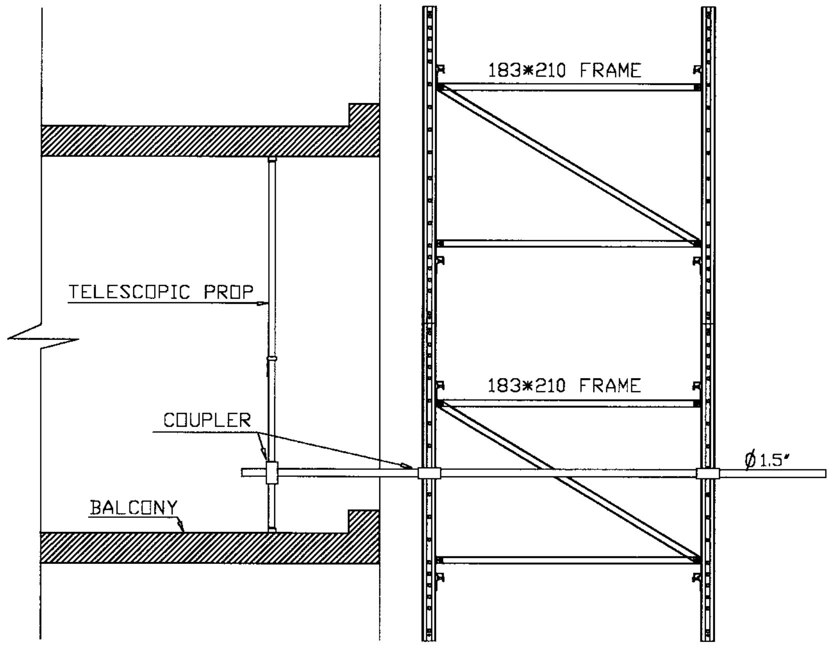

FIG. 4. Detail of Shoring Tower Tied to Building (Frame Sizes in cm)

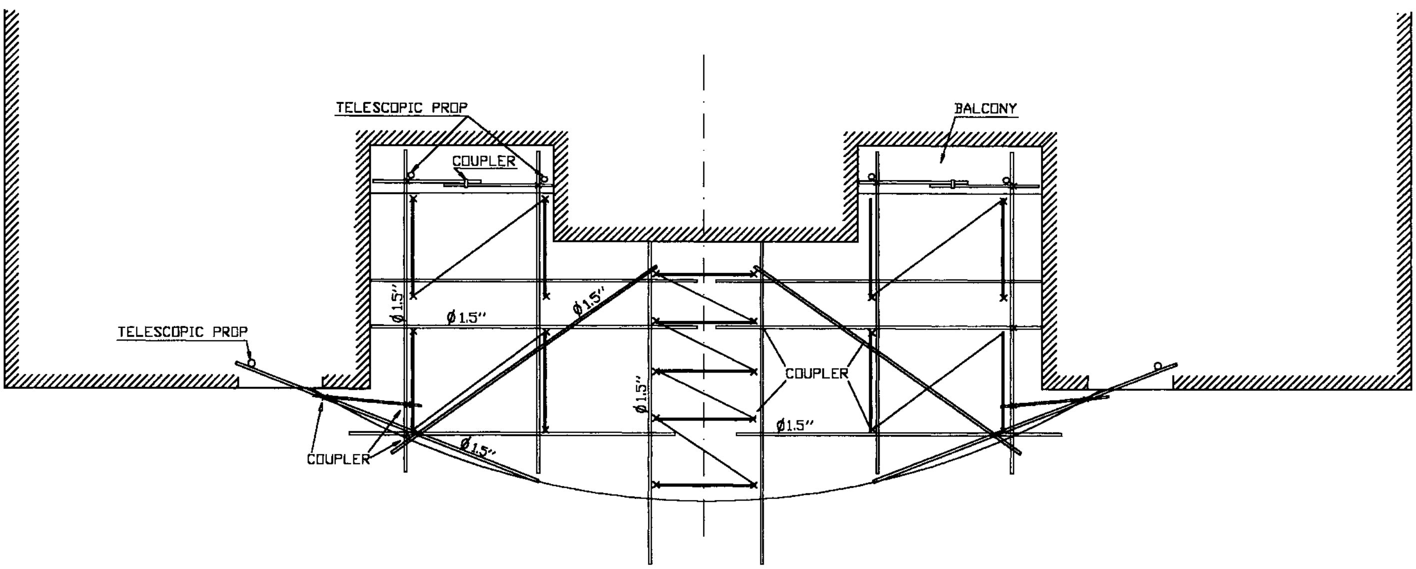

FIG. 5. Bracing System: Horizontal Array

every second floor or 6 m (18 ft), which were also connected to the tower frames and tied to the permanent structure. All tube-tube and tube-frame connections used proprietary couplers, including special couplers for the connection of circular steel tubes to aluminum sections. Fig. 5 shows the horizontal array, which includes tubes parallel and perpendicular to the building, as well as tubes running diagonally across the orthogonal tubes. Anchoring to the building was by tubes connected through wall openings (windows and balconies) to single telescopic props restrained by the floors (see detail in Fig. 4), as well as by welded ties to screw rods inserted into the external, stone-covered walls.

Secondary bracing consisted of 4-cm- (1.5-in.-) diameter steel tubes running diagonally in vertical planes parallel to the building facade and connected to intersecting horizontals.

Fig. 3 is a view of a segment of the tower and bracing array, showing details of horizontals, diagonals, ties to the building, and connections.

Reshoring

The two underground parking floors underneath the carrying towers were reshored in the course of tower erection, so that they could support the load of ~50 kN (11,000 lb) exerted by each tower leg. The reshoring used the same type of towers, and the layout and locations were kept as close as possible to those of the upper tower assembly, to ensure direct transfer of loads. Where the geometry and dimensions of the parking floors constrained placement of reshoring, single telescopic props were added to compensate for the direct tower loading. Reshore buckling did not pose a problem due to its minimum vertical dimension.

CONSTRUCTION

Work Organization

The tall shoring towers were erected by a crew of four workers. To achieve high productivity and quality of work, the

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composition of the crew was maintained throughout the entire construction, in terms of both individuals and personal tasks. Assembly was supported by the use of the on-site tower crane, which significantly contributed to the speed of tower erection. On the other hand, work was slowed by strict safety measures, particularly the harnessing of workers to the rising braced towers.

The assembly steps are as follows:

1. Erection of lower part of towers, tiers 1–3, up to ~6 m (20 ft) in height:
   - Preparatory works, including marking of tower locations on the floor and assembly of timber footings (sills) underneath tower base plates. Screw jacks were supplied to the site preoiled by the tower rental company; hence on-site jack oiling—often included in this stage—was not required.
   - Assembly of tower bases, including base plates, bottom screw jacks, and initial leveling.
   - In-place assembly of tower tiers, one on top the other, without crane assistance. These first three tiers were assembled in a horizontal sequence, by which the entire tower array is erected tier after tier [as opposed to vertical sequence, by which each tower is assembled to its full height before work progresses to the next tower; work sequence is mentioned because it may affect work inputs in tower erection (Shapira and Goldfinger 2000)].
2. Erection of main body of towers, tiers 4–28, up to ~60 m (200 ft) in height:
   - Preassembly of tower tiers, three at a time, on the ground, by the entire crew. For speed and convenience of work, tiers were assembled horizontally, with frames in parallel upright position, connected by lower and upper cross braces. Timber boards were used as temporary diagonals to brace the 6-m- (20-ft-) long assemblies for tilt-up and lifting by the crane. While this procedure was suitable for the four separate 4-leg towers, the large continuous 10-leg tower had to be split up into two assemblies (4- and 6-leg ones) for ease of erection.
   - Connection of the preassembled tiers onto the tower, including the addition of cross braces to connect the two parts of the large continuous tower. Two workers were responsible for rigging the preassembled tiers on the ground, and two other workers were in charge of unrigging the preassembly and securing it to the tower. These latter two workers originally were located on the ground, helping with assembly, but have now moved to the top of the tower. To move between the ground and top of the tower the workers used the passenger hoist serving the building and also temporary passageways from the building to the adjoining towers. As with the lower part of the towers, here, the entire tower array also was erected by horizontal sequence, but utilizing a three-tier-at-a-time assembly.
   - Primary tower bracing. The two workers, earlier in charge of ground rigging, have now moved to the building to serve bracing elements to the other two, who, having finished assembly, were responsible for the brace work. Bracing elements were transferred to the building floors between assembly cycles, depending on crane free time. This step utilized two workers for rigging, and the two others for unrigging.
   - Assembly of tower heads, including top screw jacks, final leveling, and adjustment of tower height (as prescribed by soffit elevation).
3. Forming of upper deck: aluminum stringers and joists, plywood sheathing.
4. Reshoring of two floors underneath the towers (prior to the completion of Steps 2 and 3).
5. Tying of towers to adjacent walls (prior to the completion of Steps 2 and 3).

Disassembly of towers was carried out by either a two- or three-worker crew, and involved the following tasks:

1. Lowering of top screw jacks and striking of deck formwork; forming elements transferred by hand to the nearby floor.
2. Dismantling of towers, including bracing members and wall ties; all elements transferred by hand to the floors for interim storage.
3. Tower-crane-assisted removal of all forms, tower sections, and bracing elements from the floors down to the staging area.
4. Preparation of elements for shipping to next project, including cleaning, sorting, and bundling (i.e., return to rental company).

Work Inputs

Field Results

Tower assembly and disassembly were carried out during the spring and summer of 1999. Weather, as a factor potentially affecting work progress and productivity, was inconsequential, with temperatures in the range of 15–30 degrees centigrade (~60–85°F), spring lows to summer highs, and scanty light rains (in spring only). Typical workdays, Sunday through Thursday, were 11 h long, with only 6 work hours on Fridays.

Total work inputs were 1,217 worker-hours: 644 worker-hours over 16 days for assembly and 573 worker-hours over 21 days for disassembly (the latter lasting longer than the former because of finishing works to the bottom of the slab and repair and clean-up works to the adjacent stone-covered facade walls). The breakdown of these inputs by tasks, given in Table 2, is more telling than the total. Of interest are mainly work inputs in terms of worker-hours per tower (Column 4). Upper

TABLE 2. Work Inputs for 60-m- (200-ft-) High Towers

Task (1)	Work days[a] (2)	Worker-Hours: Per job (3)	Worker-Hours: Per tower[b] (4)
(a) Assembly[c]			
Lower part of towers (up to 6 m in height)	1	44	6.77
Main body of towers (up to 60 m in height); crane-assisted	12	468	72.00
Upper deck	1	44	—
Reshoring	1	44	—
Bracing to permanent structure	1	44	6.77
[Total assembly]	16	644	85.54[e]
(b) Disassembly[d]			
Upper deck (element storage on adjacent floor)	1	33	—
Towers and bracing (element storage on adjacent floors)	12	310	47.69
Removal of elements from floors; crane-assisted	4	92	14.15
Preparation of elements for shipping to next project	4	138	21.23
[Total disassembly]	21	573	83.07[f]
[Total assembly and disassembly]	37	1,217	168.61[e,f]

[a]Mostly lasting 11 work hours.
[b]Based on 6.5 towers (the 10-leg continuous tower assembly considered as 2.5 towers).
[c]Carried out by a four-worker crew.
[d]Carried out by a two- or three-worker crew.
[e]Upper deck and lower reshoring assembly work not included in this value.
[f]Upper deck disassembly work not included in this value.

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deck forming and lower reshoring work are not included in the totals, as they are essentially not dependent on tower height. Total assembly work inputs obtained for the 60-m- (200-ft-) high towers are ~86 worker-hours/tower. With a four-laborer crew working 11 h/day, this input means 2 days for the complete erection of one such tower, including bracing and tying to the permanent structure. It took about the same time to dismantle the towers and prepare elements for reuse.

While for cost estimates the total of the assembly and disassembly input results suffice, for work planning purposes it is necessary to look at assembly and disassembly separately. In any event, worker-hours per tower, and not crew-days per tower, should be the basis in further reference to the results given here. As an example, if regular 8-h (instead of 11-h) work days were the case, then a four-worker crew using the same construction method would be expected to require 2.7, or nearly 3 days for the assembly of one 60-m- (200-ft-) high tower.

If daily output, rather than input per tower, is of interest, then the entire tower array of 6.5 towers was raised, on average, at the rate of two tiers a day, including primary bracing. In practice this output reflects an erection speed of three tiers a day, for the entire array, plus several days of work focused solely on bracing.

Prediction

Because published work input data on high shoring towers are scarce, it might be impossible to find other sources of estimation data for similar tall towers. No data could be found with which to compare the above-listed results. But the question facing the constructor is how to predict work inputs for a project prior to beginning of construction.

A previously developed model (Shapira and Goldfinger 2000) was restricted to towers up to 30 m (100 ft) in height. That model did not take into account the special bracing systems required for higher towers, but only included standard stability provisions prescribed by tower manufacturers. Such a limitation could render the results predicted by the model for such tall towers inaccurate. However, with the field results of work inputs for the 60-m- (200-ft-) high aluminum towers at hand, it was of interest to examine how much the actual field data could differ from results obtained by using the model. This was made possible, as the very same aluminum towers had been one of the tower types used to develop the model. The summary of this comparison is presented in Table 3. The rounded measurement data in Column 2 are from Table 2 (Column 4). The model results in Table 3 (Column 3) were computed using the followng model formulas for the aluminum towers (Shapira and Goldfinger 2000):

Assembly:

$$y = 0.0035x^3 + 0.024x^2 + 0.36x + 0.30 \qquad (1)$$

Disassembly:

TABLE 3. Measured versus Predicted Work Inputs for 60-m- (200-ft-) High Towers

Task (1)	Work Inputs (worker-hours): Field measurement (2)	Work Inputs (worker-hours): Model results[a] (3)	Difference, measured versus predicted (%) (4)
Assembly	86	106	−19
Disassembly	83	78	+6
[Total]	169	184	−8

[a]Shapira and Goldfinger (2000).

$$y = 0.0028x^3 + 0.016x^2 + 0.15x + 0.01 \qquad (2)$$

where x = number of tiers (in this case: x = 28); and y = assembly or disassembly work inputs [(1) and (2), respectively], in worker-hours. Column 4 in Table 3 shows the percentage difference between predicted and measured results. The 19% difference in assembly work inputs (measured results lower than predicted results) may suggest that crane assistance in the current case, as opposed to all-manual work as assumed for the model, was more significant in speeding erection than was the requirement for extra bracing work in slowing the operation. Hence, an affirmation of the benefits of crane-assisted assembly. (Note that crane utilization for disassembly was not as significant in curbing extra-bracing-related work inputs as in assembly, with measured results 6% higher than predicted results.) Overall it appears that the 8% total difference indicates a close approximation, and the model equations should therefore be useful, in the absence of other data sources, even for towers in the range of 30–60 m (100–200 ft) in height.

Cost Comparison Revisited

On the basis of the actual work input results, it is possible to complete the cost comparison between the truss-based and the tower-based solutions, thus establishing how they would have compared had labor in tower erection not been disregarded. With a total of 1,217 worker-hours at a cost of $7/hour (local rate), the labor cost component would add $8,500 to the tower solution, bringing its total cost to $21,500. The cost of the crane assistance would take it slightly higher, but the cost of crane time was not taken into account in the truss solution as well. Thus, even if labor costs had not been disregarded, the 60-m- (200-ft-) high towers solution would still be nearly 40% less costly than the cantilevered truss solution. This difference appears to be large enough to cover potential variations in the cost comparison that may result from possible changes in cost data (e.g., worker hourly rate, monthly rental rates for equipment, and cost of steel in trusses) due to market fluctuations, or even different market conditions. In the U.S. market, the labor-to-material ratio differs greatly from that considered here, and the situation might be as follows. Using a $25 worker hourly rate and $1.00/lb for structural steel (*Building* 1997), the corresponding total costs would be approximately $50,000 for the trusses and $45,000 for the towers (i.e., the latter are still 10% lower than the truss solution).

CONCLUSIONS

Although multitier shoring towers are widely used in high-clearance construction, cases illustrating their design and construction have not been adequately documented and reported for the benefit of the professional public. This paper has attempted to partially fill the gap with a report on exceptionally high multitier towers. The report aims at contributing more than mere information, as it presents the project's specific conditions and constraints and the effect these had on the decisions made. It also indicates solutions that may be considered under different conditions. In addition, the report examines the validity of a model in predicting work inputs in multitier tower assembly and disassembly, for which data are scarce. This examination shows that the formulas provided, originally intended for towers of up to 30 m (100 ft), may be used in the absence of more accurate data to compute work inputs for significantly higher towers, even as high as 60 m (200 ft).

In principle, the use of high-rise towers does not differ greatly from that of mid-rise towers. However, as the lesson learned from the case reported here shows, several design and construction aspects should be addressed more carefully as the height of the tower increases. These include the following:

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1. Design: Layout with a minimal number of towers under the given loads and stability requirements; opting for uniform tower-leg loading; bracing against tower buckling, lateral forces, and wind effects; using the highest available frames to minimize the number of tiers; self-weight of towers; and compensation for tower shortening.
2. Construction: Crane assistance in assembly and disassembly (note that not all tower models can be lifted by crane); work organization, in terms of work processes and labor crew composition; convenience of using the support towers as access scaffolds for finishing and repair works; number of tiers rather than tower height as the basis for estimating work inputs; and mode of contractor workers' employment and payment basis as a factor affecting tower type to be favored by the contractor.

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