

An 11-story cube of shimmering glass will soon float over the streets of downtown Los Angeles—and it isn't a Hollywood special effect.



Mark Sarkisian is a partner, **Eric Long** is associate director, **Andrew Krebs** is an associate and **Alessandro Beghini** is an associate, all with Skidmore, Owings and Merrill, LLP, in San Francisco.

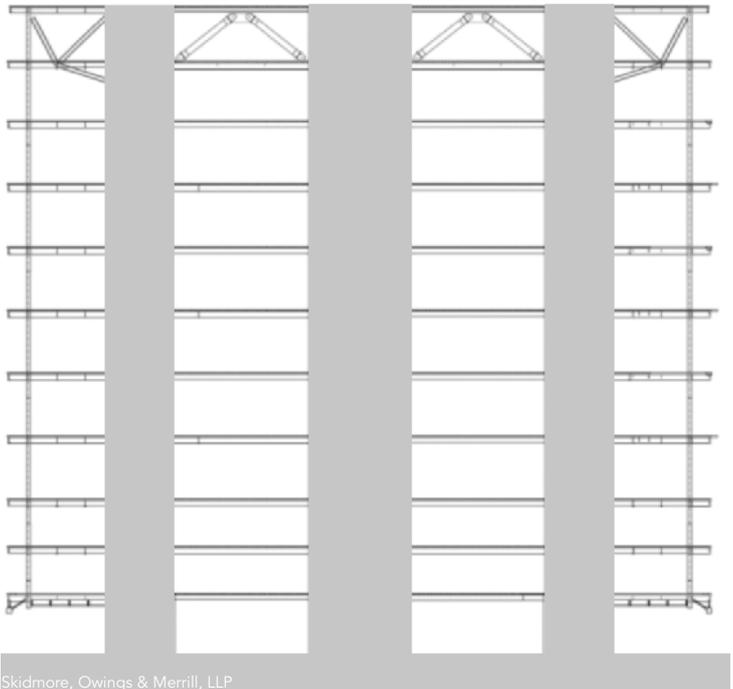
- ▶ The 633,000-sq.-ft facility, scheduled to open next July, appears to float over its surroundings in downtown L.A. thanks to a lack of ground-level perimeter columns.



Clark Construction Group



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- ▲ A cross-section view of the building's framing.
- ▼ A temporary shoring column being removed.

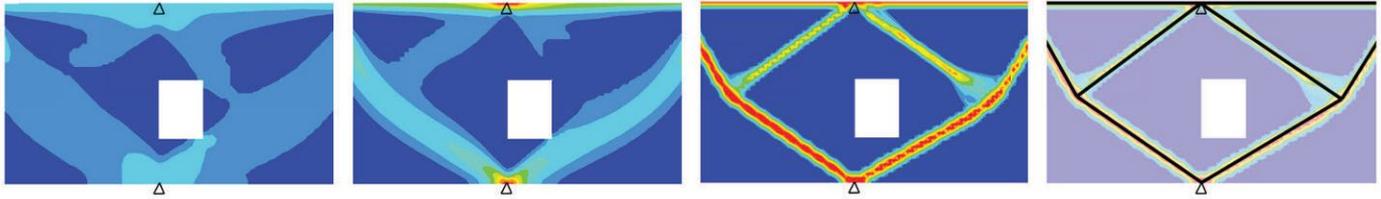
THE CONCEPT OF "FLOATING" isn't typical of courthouses.

The norm for such facilities is to project authority while staying rooted in the foundations of justice. But the new United States Courthouse in Los Angeles, currently under construction, takes a different approach.

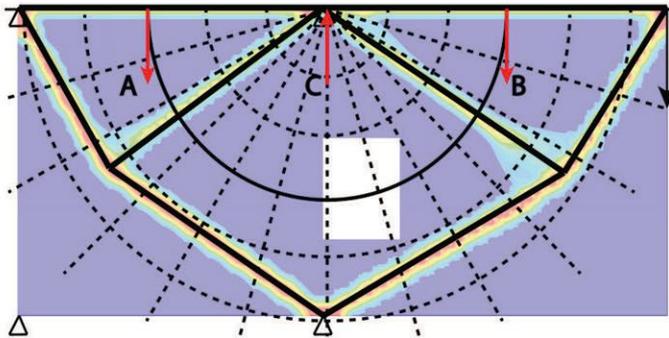
Developed in close collaboration with the Clark Construction Group, the design concept for the 633,000-sq.-ft facility, scheduled to open next July, is based on a novel idea of elevating the building above a large civic plaza by removing all vulnerable ground-level perimeter columns and supporting the entire structure on hardened-concrete shear wall cores. Since none of the perimeter columns extend to the ground level, the exterior loads are support by a frame designed for redundancy around the entire building perimeter. The open plaza area provides greater standoff (an additional 33 ft) from the



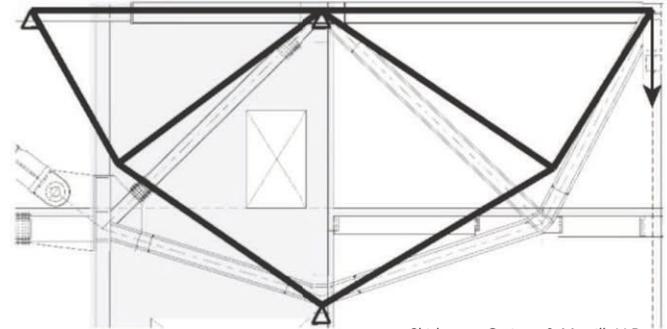
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Optimization Iterations



Overlay of Optimal Michell Truss



Final Truss with Optimized Overlay

- ▲ The top row illustrates several iterations of a topology optimization study. Based on the results, it is possible to infer an optimal truss geometry (bottom right). There is a resemblance of the optimization results with the optimal layout proposed by Michell for the problem of a single point load supported by a simply supported truss in a half space (bottom left).
- The nearly complete truss.

neighboring streets and allows the cube to stand clear as an object. The core assemblies are designed to act as organizing elements for stairs and mechanical rooms and correspond directly to the organization of the floor layout. These cores provided excellent opportunities to support the building and provide lateral stiffness from the foundations through the entire height. The shear wall cores are linked with 16 steel buckling restrained braces (half with a capacity of 2,000 kips and half with 1,500 kips), manufactured by CoreBrace, to not only provide greater lateral load resistance but also provide greater long-term resiliency, given that the braces can be replaced if required following a major seismic event.

The core elements also provide support for a three-dimensional steel truss system at the roof level that supports the vertical loads beyond the cores. This allows the outer 33 ft of the building to be supported from the truss above and also lets the cubic massing appear as a singular form, hovering above the plaza. The trusses are designed with an innovative concept that is based on optimization principles. The final geometric form is similar to a bicycle wheel and resulted in material savings of over 15% when compared to the most efficient conventional trusses.

The Structural System

The approximately 240-ft-tall courthouse, which contains 24 courtrooms and 32 judges' chambers, is square in plan, with dimensions of 222 ft by 222 ft, and the typical floor-to-floor height is 20 ft. The lateral system consists of four primary reinforced concrete shear wall cores, with additional shear walls in the north-south direction. The shear walls use vertical W12x40 to W12x87 sections in the boundary zones as longitudinal reinforcement, which allowed the steel framed floors to be erected ahead of concrete pouring during construction. The shear wall cores extend from foundation to roof and are interconnected with ductile reinforced



coupling beams at openings required for doorways and corridors.

The gravity framing system consists typically of conventional 3-in. metal deck topped with 3¼-in.-thick lightweight concrete supported by conventional steel wide-flange beams. The steel floor framing members span to the steel columns embedded in the concrete shear walls within the center portion of the building and to steel columns at the perimeter of the building. These perimeter steel columns, 24 in all, are suspended from 12 one-story-deep structural steel trusses at the roof, one at each end of each truss. The truss system is comprised of wide-flange sections and cantilevers from the internal reinforced concrete shear walls out to the perimeter steel columns. The roof trusses extend through the interior of the plan and act as coupling elements between the reinforced concrete shear walls, and BRBs are used for the diagonal truss members between shear walls. Herrick provided 24 42-in.-diameter by 48-ft-long temporary steel columns, which were used in compression to assist in construction and were removed upon completion of superstructure construction. These were located on top of the basement slab, penetrated through the Level 1 concrete podium slab and extended up to the underside of each perimeter column, which began at Level 2.

Each corner of the building is completely column-free, with a cantilever of more than 30 ft in each direction. These cantilevered corners were accomplished using a "layered" cantilevered beam framing approach that was used to control displacements



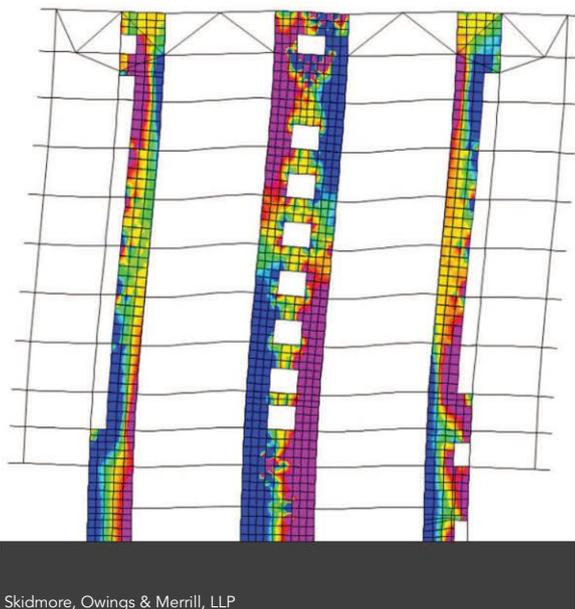
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◀ ▼ Column-free cantilevered corner framing.



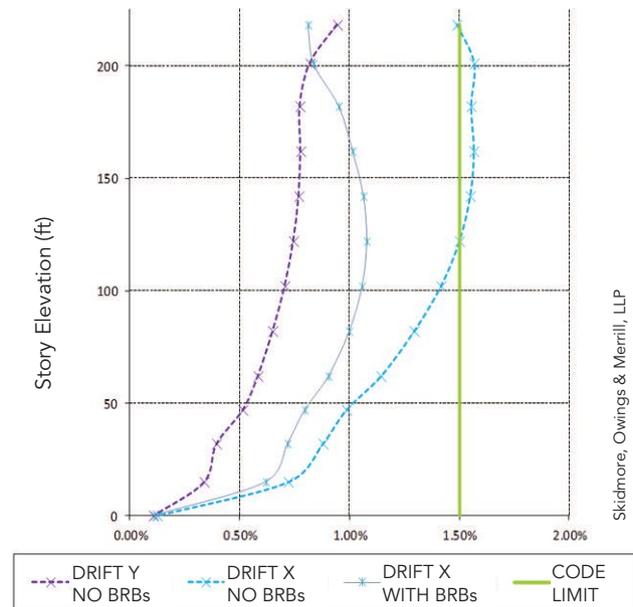
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▼ The linking truss: induced double curvature deformation (left) and influence in weak axis (X) drift (right).



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Story Drift Ratio Under Seismic Action



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while minimizing the steel needed. Temporary angles were used during steel erection to support the cantilever corners to hold the steel at the proper elevation until the final moment welds could be made.

The Bicycle Wheel

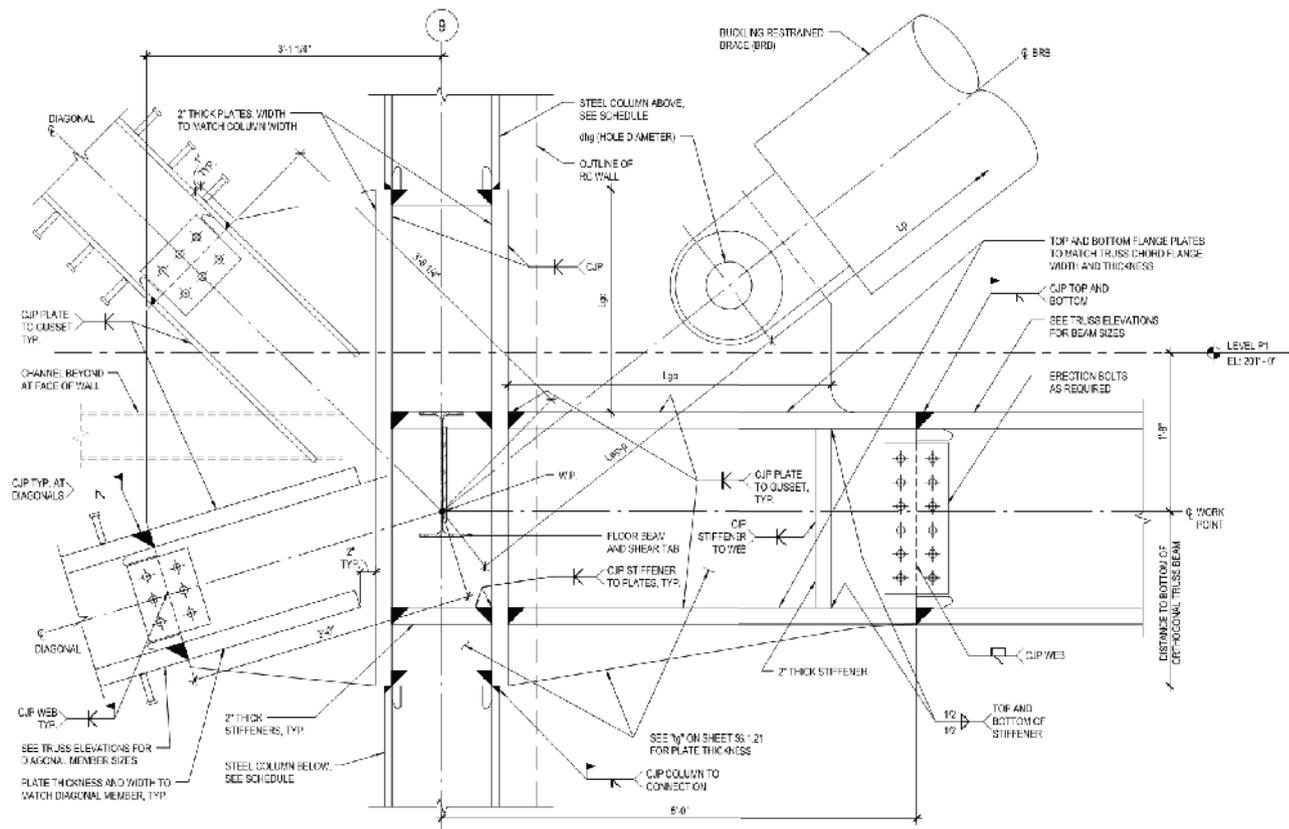
One of the project's most significant design challenges involved the gravity loads from the perimeter columns needing to be carried back to the reinforced concrete core elements through a steel roof truss system. The overall depth and configuration of the roof truss members was critical to optimizing the steel weight given the strength and deflection requirements. The roof truss member configuration was inspired by the results of the topology optimization and evaluated for efficiency using Maxwell's Theorem of Load Paths. This theorem is at the foundation of Michell's early 20th century work on frames of least weight, also known as Michell trusses. These truss systems represent the stiffest layouts for the least amount of material in a continuum and were chosen for the roof trusses on the courthouse due to the high floor-to-floor elevations, as well as for their ability to coordinate well with the MEP layout. (See the previous page for an illustration of the iterations that led to the final truss design.)

Another challenge was the interface between the truss steel and the reinforcing steel inside of the core walls, in terms of fitting everything inside the walls and working around the large member sizes. Tekla was used to model the steel and Altair HyperWorks was used to model the rebar, which assisted with identifying areas where they clashed and the subsequent finalization of the reinforcing steel details.

The Linking Trusses

Yet another challenge was that the layouts of the shear wall core elements were best suited as rectangles given the organization of the floor program. This led to a "weak" direction and a "strong" direction. In the weak direction (north-south), behavior was identified that exceeded the allowable story drift. But rather than a conventional optimization process of increasing wall thicknesses, the idea to use the roof truss as a "mega" coupling beam at the top story was explored and incorporated in the design.

The introduction of linking diagonals produced a change in the lateral mode of deformation in the weak direction, from the single curvature cantilever typical of shear wall buildings to a double curvature mode of deformation more typical of an outrigger system. With the coupled system, lateral drifts were satis-



▲ A BRB-to-column connection drawing.

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factory and contained slightly over 1.0%, and as a consequence, the structure became strength-controlled rather than stiffness-controlled. Fabricator Herrick Steel assisted SOM in the final detailing of the truss connections and node joint configuration, providing iterative comments in working sessions to provide the most economical and efficient joints.

BRBs were the most efficient elements for the coupling system. As they are not intended to be the main energy dissipating mechanism, the linking braces are designed to remain essentially elastic under the elastic demands produced for the design earthquake. The BRBs were erected in sequence with the structural steel framing. However, only the top pin in each BRB was installed at that time. To ensure that minimal dead loads were carried by the BRBs, the hole for the bottom pin was bored through the gusset plate in the field, and all welding was completed after the concrete was completed through the roof level.

From Compression to Tension

In order to shorten the construction schedule, a bottom-up procedure was followed by temporarily shoring the perimeter columns until the roof truss was built; more than 400 chevron braces were installed and removed as the concrete floors were poured. Building the reinforced concrete walls from ground to roof first, then erecting the roof trusses, erecting the floor framing below and pouring the slab on metal decks would have more than doubled the schedule for the superstructure and increased the cost of construction beyond feasibility. Early collaboration between SOM, Clark Construction Group, Herrick and the concrete contractor was critical to the success of this sequencing and the development of the re-

moval process; SOM provided the design parameters and expected deflections while Herrick and steel erection consultant Hassett Engineering developed the temporary column sizes, support connections, jacking scheme and temporary column removal plan. The transfer of compression in the temporary shoring columns was facilitated by a jacking system in the basement, which allowed for the removal of 2 in. of steel shims from under the temporary shoring columns. Nonlinear staged construction analyses were performed using ETABS 2013, and deflections at critical stages were tabulated. Relative elastic deflection between the perimeter columns and the core walls was studied at each level in combination with creep and shrinkage analyses. Corrections in floor elevations at the perimeter locations were determined for construction, with the perimeter cantilever poured to thickness, and the slab between the core walls poured to design elevation. ■

Owner

United States General Services Administration

Design-Build Contractor

Clark Construction Group

Architect and Structural Engineer

Skidmore, Owings and Merrill, LLP

Erection Consultant

Hassett Engineering

Steel Team

Fabricator and Erector

The Herrick Corporation (AISC Member/Certified)



Detailer

SNC Engineering, Inc. 