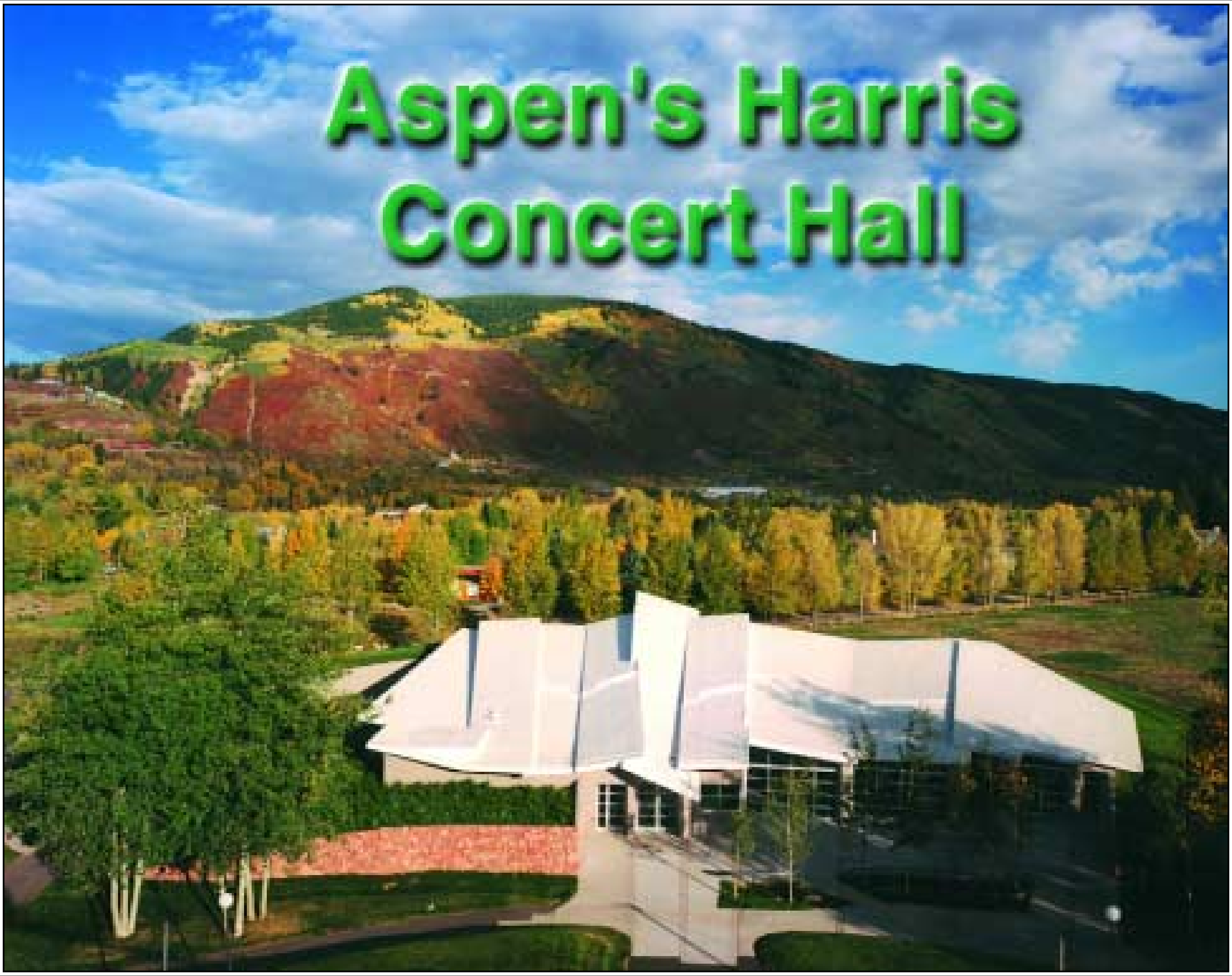
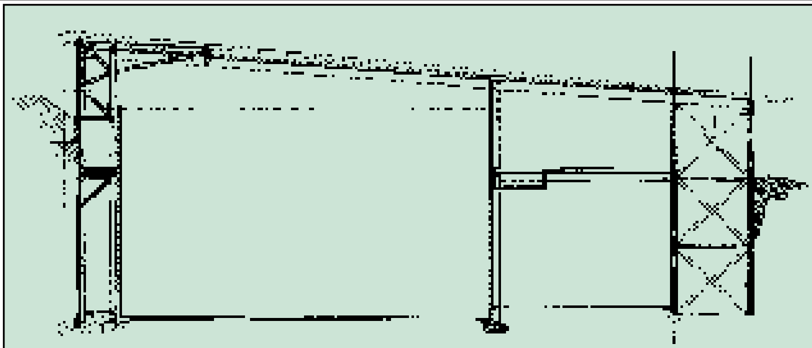


Increasing steel tonnage while simplifying fabrication and erection resulted in a more economical design

By Gregory P. Luth, Ph.D., S.E., and M. Douglas Rutledge

THE JOAN AND IRVING HARRIS CONCERT HALL FOR THE MUSIC ASSOCIATES OF ASPEN was conceived as a multi-purpose building to support the activities that take place during the city's annual music festival. At this annual event notable musicians and students from all over the world congregate to study, practice, and give concerts. The concert venue is an open-air tent structure that contributes to the unique atmosphere but which is unsuitable for high quality recording. This, along with the need for practice facilities that are contiguous with the concert site while being acoustically isolated from the outdoor activities provided the motivation for the new facility.

As the program evolved, the needs of the Music Associates become more focused and it became clear that this would be an unusual facility that would challenge the abilities of the architects, engineers, and acousticians. In addition to serving as a rehearsal facility, the new hall was expected to provide an excellent acoustical environment comparable to the finest available elsewhere for an audience of 500 people and for groups of musicians ranging from small ensembles up to a 150 piece orchestra. The acoustical isolation of the hall had to be of sufficient quality to accommodate commercial quality recording activities that normally require a completely isolated "quiet" room.



Building Section

The city of Aspen exercises a considerable amount of control over new facilities in an effort to preserve the character of a small town in the Colorado Rockies. City planners had two primary concerns: that the design for the new facility be architecturally unique and that the facility have a low profile and be unobtrusive to mesh well with the current pastoral setting of the site, which is surrounded by residential areas.

In addition to the myriad of special constraints imposed by the program and regulatory agencies, the Music Associates of Aspen had a tight budget for the project. Last, but not least, to avoid interrupting the summer festival activities, all construction would have to be completed in one off-season, which meant wintertime construction in the mountains. Specifically, the contractor would be able to move on site in the latter part of August and would have to complete construction by the following June-in less than nine months—constructing the entire facility during the fall and winter months.

Program Development

Due to the unusually stringent regulatory process in Aspen, planners had to develop the architectural concepts to the point where the exterior envelope of the building was frozen before the project feasibility could be established. Harry Teague Architects, of Aspen, worked with the owner and the acoustical consultant, Cohen Acoustical, of Los Angeles, to develop the architectural concepts that were submitted for the approval of the city planning commission. Although the overall height of the proposed building was 50’ on one side with the roof sloping down to an overall height of approximately 35’ on the opposite side, the visual impact was minimized by placing the floor of the concert hall 25’ below the existing grade and then building up a 15’-tall berm in the shape of a dome.

The roof is formed by a series of nine planes that are truncated segments of a circle generated by radial lines originating at the center of the existing tent. The individual planes have varying slopes in two directions so that in the general case there is a variable step along the interface between planes. The design that resulted won both city approval and an award from the National Endowment for the Arts, while satisfying the conceptual constraints imposed by the acousticians.

By the time the contractor and structural engineer were on board, the following hard constraints had evolved:

- Sound isolation requirements dictated that there be a 6” normal weight concrete roof slab.
- The roof had to span the 65’ width of the hall and carry a suspended ceiling consisting of 2” of cement plaster (25 psf) and a snow load, including drifts, of as much as 115 psf.
- The structural depth over the hall had to be kept to a minimum. (The architectural concepts anticipated a folded plate structure.)
- The 50’-tall foundation wall on the east side of the building had to carry as much as 40’ of earth resulting in a uniform service load of 1,000 psf. The total thrust due to soil loads on the east side of the building is 3,800 kips. To put this load in perspective, it is roughly equivalent to the wind load on a 50-story building the same width as the hall and is three times a Zone 4 seismic load for the structure.

- The exterior envelope of the building was fixed, precluding the possibility of simplifying the geometry.

Schematic Design

After the approval of the design concept by the city of Aspen the owner contracted independently with the structural engineers to provide structural design and with the general contractor, Shaw Construction, of Grand Junction, Colorado, to provide value engineering services during the remainder of the design process. At the first meeting, the design/construction team developed a set of secondary constraints as follows:

- The building system had to accommodate a fast winter construction schedule.
- To the extent possible given the geometric constraints of the architectural concepts, the structural concepts should result in repetition.
- The structural system should allow for a significant amount of prefabrication.
- The structural concepts should minimize the time required to enclose the building.

A series of 10 different conceptual designs were prepared, including cast-in-place, precast, and steel schemes, and evaluated for both cost and schedule implications. The major subsystems that were considered were the roof structure, the foundation walls, and the lateral system.

Although the architect had contemplated a cast-in-place folded plate structure for the roof with interior concrete walls to resist soil pressures, that was not the most economical system nor could it be built within the schedule constraint. Precast was not economical due to the lack of repetition. The steel schemes included a variety of framing options based on two fundamental systems of load paths: one with primary girders oriented radially; the other with primary girders oriented tangentially. Discussions with the acousticians indicated that the slab thickness constraint could be met with an average thickness of 6", allowing the use of 4½" of stone concrete over 3" deck. At the widest spot, the spacing of the radial grids exceeds the unshored span of commonly available decks requiring a significant penalty either in the deck or in shoring. As a result, the scheme that appeared to offer the best combination of constructibility and economy utilized a system of radial cantilever/drop-in girders and tangential beams with 4½" of normal weight concrete over 3", 18 gauge, composite metal deck.

On the high side of the hall there is a 5'-wide corridor at ground level that is not capable of carrying the full wall reaction, while on the opposite side of the hall there is a relatively substantial floor slab at grade. The options for the 50'-tall foundation walls included conventional 24" walls spanning vertically 50' or 18" walls below and 14" walls above ground level spanning vertically to the corridor slab. The corridor slab would be supported either on moment frames located on the radial grids approximately 13' apart or by post-tensioned tie-backs anchored to a continuous "dead man" cast at existing grade. The latter scheme was judged to be the most economical based on preliminary estimates.

At this stage in the design process, the structural engineer and general contractor began to examine the detailed construction sequence and concluded that there would be an advantage in being able to backfill a portion of the foundation walls early in the construction schedule to help with the weather enclosure. It was decided to design the foundation walls on the high side to carry the first 25' of soil in a cantilevered condition, allowing back-filling operations to commence as soon as the walls reached existing grade. The "active" soil pressures for this temporary condition are significantly less than the "at rest" pressures used for the design of the permanent condition and the wall thickness of 18" could be maintained. The cantilevered wall did require a substantial continuous "L" footing.

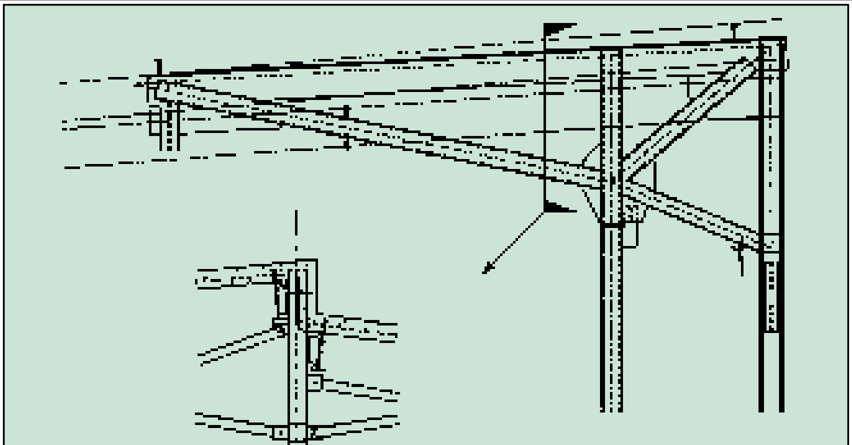
The following detailed construction sequence evolved as the result of the coordination efforts of the design/construction team:

1. Excavate to foundation elevation and cast spread footings while the steel frame is being fabricated.
2. Erect the steel frame and metal roof decking on the spread footings.
3. Place concrete on the roof deck.
4. Install temporary waterproofing on the concrete roof deck.
5. Simultaneously start foundation wall construction.
6. Commence backfilling as soon as the foundation walls are waterproofed to grade.
7. Install temporary enclosures between the roof and the completed sections of walls to allow interior construction to proceed.
8. Install tieback system at grade.
9. Complete construction of the walls.
10. Install permanent roofing after connection between the walls and roof diaphragm is complete.

The schematic wall and footing designs were modified to accommodate the cantilevered design condition. Based on the modified schematic design the general contractor prepared a conceptual estimate which indicated that the project was significantly over budget with roughly 30% of the total cost attributable to the structure.

The structural steel costs accounted for 30% of the structural cost while the concrete portion of the work accounted for 60%. The tieback system accounted for 10% of the total structural cost. There were inherent risks with the tieback system due to the unique use of a single continuous "dead man" at grade. There is very little published information on analysis/design methodologies for such a system. What little information is available was written for guyed towers and is extremely conservative, e.g., the methodologies are based on passive resistance on the face of the anchor block resulting in anchor block depths on the order of 10' for the 10 klf reaction from the walls.

Aside from the questions surrounding the correct design procedures, the tieback system envisioned offered little redundancy and the entire system would have to be test loaded simultaneously to conform to standard practice for tieback installations, since the tiebacks are not independent. A conventional tieback system in which the individual tiebacks are test loaded as they are installed was not considered due to the presence of cobbles in the glacial till on which the building would be founded and because the costs would have been prohibitive.


Typical Truss

Design Development

Because of the budget considerations and the questions regarding the design of the tiebacks, the structural engineer began to explore an arch concept to support both the roof and the wall. The thrust from the arch could provide the necessary support for the wall at an elevation considerably lower than the 50' height of the roof. These studies resulted in completely new design for the structure between schematic design and design development (DD).

As the result of continuing coordination between the acousticians and the architect, the ceiling of the hall had assumed a more regular configuration remaining at a uniform elevation in the transverse direction along any radial line. This created the possibility of a very deep structure on the high side of the roof. While an arch was not possible due to a number of architectural constraints, it was possible to approximate the behavior of the arch by creating a deep truss on the high side of the roof that would cantilever 15' out over the hall to pick up the main roof beams. This truss was formed integrally with a vertical "pony" truss that would extend down to a point roughly 8' above the corridor slab. In the absence of soil thrust loads, the moment from the horizontal cantilever could be resisted by a couple provided by a pair of columns spaced at approximately 5'. In the presence of soil thrust loads, the eccentricity of the thrust at the bottom of the vertical section of truss and the resisting thrust at the elevation of the roof slab counteracts the gravity moment. If the full design soil load is present without the design live load of 115 psf, the couple in the lower columns actually reverses.

With the new scheme, the radial roof girders were replaced with a pair of beams flanking the truss. This allows the truss to occupy a vertical plane while the beams on either side can be sloped to match the planes that they support, simplifying the fabrication of both the beams and the trusses. In this condition, the geometric irregularities are confined to the connection between the trusses and the beams while the fabrication of the primary members is straightforward. The preliminary design of the beams assumed a two span condition with a support at the tip of the truss. The preliminary design was based on LRFD methods and considered composite action with the heavy slab at midspan and with heavy reinforcing in the slab over the supports. For these assumptions, strength and stiffness under construction loads control the beam sizes. This design approach resulted in W16X31 grade 50 beams spanning the 65' hall with 5 #9 reinforcing bars over the supports at each end. The ability to use such a slender section on the long spans enabled the architect to maintain the overall exterior geometry that had already been approved while satisfying the detailed acoustical constraints on the shape and volume of the hall. Even with the modest structural depths required, there is virtually no room to spare in the final roof sandwich. The scheme offered the additional benefit of reducing the span of the deck, eliminating the need for the tangential beams - all of which would have been different.

To take advantage of the strength provided by the trusses on the radial lines, the foundation walls were designed to span horizontally to concrete pilasters that encase the outboard column on the radial grid. This scheme uses the architectural steps that occur in the exterior wall at these locations to create a flanged concrete section with excellent stiffness and strength characteristics. In the new configuration the typical wall section was reduced to 12" with a proportional reduction in the continuous wall footing. Designing the pilaster/truss system to carry the soil loads in the construction condition accommodated the construction sequence.

As a result of the reconfiguration, the cost of the tiebacks was eliminated and the concrete costs were reduced by 30%. Since the schematic estimate was based on an average unit price per ton, it was not possible to determine whether the new scheme reduced the overall steel cost. However, the feedback from fabricators who looked at the two schemes indicated that the constructibility of the new scheme was superior to the original.

Detailed Design/Construction Engineering

Given the tight schedule and complex steel fabrication, Music Associates of Aspen accepted the recommendation of the general contractor and structural engineer that the steel detailing be overlapped with the final design of the structure. Rather than retain the services of a conventional detailer, the owner retained the engineer-of-record who in turn subcontracted with Douglas Rutledge & Company (DRC) of Loveland, CO, to provide detailing services in order to get the benefit of constructibility input during the final design. DRC's advantage is that they also have experience with fabrication.

The impact of the decision to retain the services of a detailer with fabrication knowledge during final design was felt immediately. The first tasks were to evaluate the DD structure for constructibility from the point of view of detailing (minimizing the risk and/or impact of detailing errors), fabrication (repetition and uniformity of concept, size of pieces to maximize shop assembly), erection (stability, ease of making field connections, number, and order of pieces), construction sequence (particularly the effect of unbalanced concrete loads during the placement of concrete on the deck), and schedule. DRC completed the initial review in a week and precipitating a number of significant changes in the design.

- It was determined that all of the trusses could be made identical by setting the slope of the top chord and the overall depth to accommodate the lowest roof elevation and then allowing the beams to slope to be independent of the truss.
- By setting the truss bearing points appropriately, the trusses could be shop assembled.
- Varying the lengths of the column pairs supporting the trusses could accommodate elevation differences between radial lines. The column pairs them-

selves could be shop assembled as a single piece.

• Only one of the radial lines requires an extension of the vertical truss below the bottom of the typical truss. On that line the vertical truss could be included with the column pair assembly with the exception of a single loose diagonal which would be field welded in place.

• The question of whether the beams should be placed with their webs vertical, a more stable configuration for bare steel erection, or perpendicular to the plane of the roof, which is more conducive to the composite action was studied. Composite action is required to maintain the size and weight of the beam, which is critical to the architectural and acoustical constraints. After weighing the costs and benefits of each approach, it was decided to place the beams perpendicular to the roof planes but add bridging to stabilize the top flanges prior to the deck being welded in place.

• To provide continuity of the bridging, diaphragms consisting of a single ½" plate attached by means of two field bolts to standard stiffeners on the beam pairs were provided in line with the bridging. A similar detail was used to provide support for the bottom flanges of the beams in the negative moment regions.

• The beam connections at the columns use a pair of plates in the form of a tee. A vertical plate attached to the column is cut on a bias to accommodate the slope of the beam webs. A second plate parallel to the beam web is welded to the first at a slope that matches the beam slope along the radial grid. The bolt holes in the beam align with the axes of the member, confining the geometric irregularities to the pair of plates. (It was felt that this would result in the easiest fabrication and at the same time be easily modified in the field if necessary.)

To increase the stiffness of the beams while minimizing the number of pieces, they were made continuous from the outside column on the truss side to the end of the cantilever on the far side of the hall, resulting in three connections to the truss. The typical beams lengths were set at just under 60' to accommodate the standard mill length. This dictated the length of the cantilevers on the far side of the hall. In order to accommodate the camber in the erected beams it was necessary to detail the connections at the three points along the truss for the cambered geometry, which, in turn made it necessary to define the method of cambering, i.e., center or third points.

The modifications above can be considered "refinements" of the original concept. However, the review by DRC identified areas where significant economies or better constructibility could be achieved by changing the design concept. Among these were the following:

• The transition from a single beam to a double beam on the opposite side of the hall from the truss was problematic from the point of view of connection complexity. It would also present stability problems during concrete placement since the beam would be subjected to torsional loads when the concrete was present on only one side. There was added complexity due to the need for additional ledger angles, miscellaneous plates, and associated hardware required to accommodate the varying slopes and elevation differences that occur along the radial grids. Although it would add tonnage, it was decided to use a double beam all the way across the building. As a result of this decision, the connections to the columns are similar at all points along the grids.

• The areas on the north and south sides of the hall approach rectangular geometry. At design development a rectangular framing scheme was adopted in these areas. At the suggestion of DRC the framing in these areas was changed to match the balance of the roof, resulting in uniformity of the structural concept throughout.

• The architectural concept requires eaves that cantilever beyond the walls from 2' to 10' while presenting a thin (7½") profile. The DD documents contemplated the use of metal deck supported on TEES with stems turned up into the slab to form the eaves. DRC identified the miscellaneous support steel as a costly item. In coordination with the architect and general contractor a new detail was developed. The new detail not only eliminated the costly steel, but also solved the sequence problem of how to make the final connection between the walls and the roof slabs (which are built out of sequence) while eliminating all of the soffit material. The result is more elegant and allows the architecture to follow the original design intent.

• As the result of value engineering using the "unit cost" approach during schematic design, the ground floor had been framed using joists spanning to masonry bearing walls. It was assumed that the ground floor would be built essentially as a mezzanine inside an existing building. On closer examination, it seemed likely that the erector and fabricator would want to stage the joists at final elevation during the initial erection. Doing this would necessitate the addition of beams on which to stage the joists. Due to the geometry of the building, there were few joists that would be identical under any circumstance. This, coupled with the implications of working joist shop drawings, fabrication, and delivery into an already tight construction schedule, tipped the balance in favor of composite beam framing on the ground floor.

While it appeared that the above modifications would greatly simplify the fabrication and erection of the project, there was no denying that the tonnage of the project had increased. All of the thought that had gone into the decision making would be wasted if the bidders were forced to use a unit cost approach due to the lack of time to retrace the steps the design/detailing team had already taken. To give the bidders the benefit of the thinking, preliminary shop drawings for the trusses were issued along with the bid documents.

Construction

Construction began on schedule in late August of 1992. One of the worst winters in memory arrived with a roar several weeks later when the first snow of the season blanketed the nearly complete excavation. The contractor battled the elements for a number of weeks before he completed the first third of the foundations and was ready to receive the initial shipment of steel.

The first load of steel arrived on site in late October and was erected in 5 days, substantially bettering the three weeks that had been allowed due to the complexity. This pattern repeated itself throughout the erection process. The detailing and fabrication were virtually error free.

Once the steel was erected, and concrete slabs on deck placed, the contractor draped visqueen from the partially completed roof, installed temporary heat and proceeded with the balance of the construction in the relatively pleasant environment of an enclosed building.

Throughout the construction process, the general contractor utilized CAD files provided by the structural engineer to monitor and verify geometry. This turned out to be an enormous benefit due to the complexity of the geometry. Much of the time that might have been lost in the conventional RFI process was saved, allowing both the contractor and the engineer to focus on the more substantive issues that arise on a fast-tracked project being built in severe weather conditions.

The sequence that had been developed cooperatively by the design and construction team during the preliminary design worked like a charm. The final finishes inside the hall were being applied as the last foundation wall was completed. The summer music festival went ahead on schedule in June, 1993. The inaugural concert at the Joan and Irving Harris Hall was held on schedule in August, 1993, approximately 11 months after construction began.

Project Team

Owner:	Music Associates of Aspen, CO
Architect:	Harry Teague Architects, Aspen, CO
Structural Engineer:	Krawinkler, Luth & Associates, Inc., Menlo Park, CA
General Contractor:	Shaw Construction, Denver
Steel Detailer:	Douglas Rutledge & Company, Loveland, CO

Gregory P. Luth, Ph.D., S. E., is the president and a principal engineer at Krawinkler, Luth & Associates, a structural engineering firm specializing in performance based design and construction engineering services, headquartered in Menlo Park, CA.

Educated as an engineer, M. Douglas Rutledge spent 20 years managing a steel fabrication business. In 1995, he joined and now manages the KL&A office in Loveland, CO, which focuses on integrated design/construction activities.