The conceptual plans of an academic center for nanotechnology research were developed in the spring of 2009. The research center is a 60 × 40 m (200 × 130 ft) two-story, 11 m (36 ft) high building surrounded by an atrium that is enclosed by a structural facade architecturally designed to represent the lattice structure of a carbon nanotube (Fig. 1).

The 134 m (440 ft) long structural facade consists of 7.3 m (24.0 ft) high and 500 mm (20 in.) thick prefabricated structural elements of C60 (8700 psi) white concrete that are aligned along two curves partially surrounding the rectangular footprint of the building. The structural facade is constructed of 53 prefabricated units of three different types and encloses an unobstructed atrium space of approximately 1185 m² (12,750 ft²) around the two-story building. The approximate surface area of the atrium is 3000 m² (32,000 ft²). The atrium is capped by an insulated concrete roof with an area of 1800 m² (19,400 ft²) and weight of 560 kg/m² (945 lb/yd²) that is supported on steel beams spanning the variable atrium space. The span is a maximum of 15 m (49 ft) and cantilevers over the structural facade at a maximum extension of 7 m (23 ft).

The structure is located in the highly seismic Marmara region of Western Anatolia in Turkey. The area requires design for absolute ground acceleration value of 0.4g, where g is acceleration due to gravity. The structure is exposed to winter conditions with possible ice formation and snow loads of 100 kg/m² (20 lb/ft²). The annual ambient temperatures vary from -5°C (23°F) to 40°C (104°F), and the

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**This paper describes the design and construction of a prefabricated structural concrete facade of a university nanomaterials laboratory.**

**The precast concrete lattice pattern on the facade mimics the nanostructure of the materials.**

**The structural design met the requirements of seismic resistance, as was demonstrated by a minor earthquake that occurred during construction.**
prefabricated structural facade that also formed the architectural boundary of the atrium are discussed in this paper. The structural facade of the research center is aligned along two curves that are defined from five focal points. The entrance to the atrium is through the overlapped parts of the two curves. The facade elements were developed as a lattice structure. The 500 mm (20 in.) thick prefabricated panels not only house thermally insulated layered windows at middepth but also function as a tilted light funnel that maximizes the infusion of natural daylight into the atrium.

Functional characteristics

The research center was built using several different construction materials and techniques. The two-story rectangular building is a cast-in-place reinforced concrete structure of C30 (4400 psi) concrete with interior reinforced masonry walls. The atrium consists of prefabricated, insulated C60 (8700 psi) white concrete facade elements. The roof is made of C30 concrete reinforced with lattice steel overlying galvanized ribbed steel decking that spans between steel beams. The steel beams rest on steel belt beams that are continuously supported along the top of the facade elements. The connections between the steel roof beams and the steel belt beams and the connections between the steel belt beams and the prefabricated concrete structural facade elements allow rotation between the connected elements.

Multiglass panels in thermally insulated aluminum frames are embedded within the prefabricated and insulated facade elements. Due to the differences in techniques and materials used in its construction, the design and construction of the research center was completed in stages by different contractors. The design and construction of the prefabricated structural facade that also formed the architectural boundary of the atrium are discussed in this paper.

The structural facade of the research center is aligned along two curves that are defined from five focal points. The entrance to the atrium is through the overlapped parts of the two curves.

The facade elements were developed as a lattice structure. The 500 mm (20 in.) thick prefabricated panels not only house thermally insulated layered windows at middepth but also function as a tilted light funnel that maximizes the infusion of natural daylight into the atrium.

In addition to its important structural function as a support element for the roof, the 1000 m² (10,800 ft²) prefabricated facade also forms the vertical face of the structure. Therefore, the design of the facade incorporated architectural, functional, and structural requirements. The architectural requirement for providing reflectivity and visibility of the facade through the structural element required the design of the concrete mixture to satisfy not only structural and durability requirements but also the functional requirement of reflectivity. Due to its low iron oxide content, white cement is up to two times more reflective than gray cement. The use of white cement in the concrete provides the required architectural reflectivity without a special reflective coating.
The structural facade had to be transparent enough to allow the infusion of daylight while having the strength and ductility to support the lateral and vertical loads due to the heavy roof. The solution to these two conflicting requirements resulted in 30% of the total facade area dedicated to the solid structural elements, with the remaining 70% made up of the windows.

**Structural characteristics**

The structural facade of the research center consists of 53 prefabricated elements, of which there are three types. Following their production and shipment, the prefabricated structural elements were installed into sockets that were constructed as a continuous strip foundation for the total length of the facade. Figure 2 shows the three types of prefabricated facade elements that constitute the facade.

There are thirty 3.0 m (10 ft) wide type 1 elements, twenty 1.5 m (4.9 ft) wide type 2 elements, and three triangular type 3 elements with a maximum width of 4.5 m (14.8 ft). The type 3 elements are the bounding elements of the two curves along which the facade is positioned. Type 1 and type 3 elements weigh approximately 70 kN (15.5 kip) each, and type 2 elements weigh 35 kN (7.7 kip) each.

The beams that support the roof plate are supported by the prefabricated facade elements every 3 m (10 ft) along the facade. The roof plate that spans between the two-story structure and the facade has variable spans. Therefore, the load on each of the precast concrete structural facade elements is different. Rather than designing and producing each of the 53 prefabricated concrete facade elements for different load conditions, the elements were designed for four maximum loading values that were determined by dividing the roof slab into four separate regions with a special consideration for the roof along the region where the two facade curves overlap.

The roof loads that are collected by the radial beams are transferred to the belt beam that is aligned along the top of the facade. The belt beam and rods limit relative movement between the precast concrete units during construction. They also limit the degrees of freedom for vibrations to occur through the facade and provide redundancy for the transfer of loads among the prefabricated elements.

The steel roof beams that are composite with the reinforced concrete roof slab are supported by the prefabricated structural facade elements. The connection allows for the rotation of the beams, preventing the development of fixity moments at the top of the cantilevered facade elements.

Figure 3 shows the cross section of the roof slab. The beam connections allow free rotation under the action of service loads. The bolts engage axially under the possible action of pounding in the event of an earthquake and provide partial fixity limited by the yield strength of the bolts in the event of ultimate loading on the roof.

The erection of the prefabricated structural facade elements had an in-plane erection tolerance of 5 mm (0.20 in.) and an out-of-plane erection tolerance of 0.02 rad. During the design phase, the individual elements were designed to resist loads individually.

The design of the prefabricated structural elements not only included the service loads and ultimate loads but also loads generated during construction and installation.

Moments, shears, and axial loads are generated within the prefabricated structural facade elements under the action of imposed axial and lateral loading from the roof. In addition to the bending moments, shear, and axial loads, torsional stresses were expected due to the diagonal positioning of the lattice elements between the nodes of the prefabricated facade elements.
To provide the required strength and ductility, a doubly reinforced solution was used that increased the allowable steel ratio of the particular cross section to 6.5%.

The prefabricated facade elements not only had to be strong enough to support the vertical loads imposed by the roof and lateral loads imposed by the deflection of the roof attached to the main building but also to allow the housing of windowpanes within limited deflections and prevent pane fracture under the construction and service loads.

Durability considerations led to limited service load stresses and associated crack widths because the elements were not to be coated for durability and protection. The

Tight confinement of the concrete was critical due to the anticipated triaxial stresses. With the effective depth of the section at 160 mm (6.3 in.) along the axis where the in-plane bending takes place, the shear reinforcement spacing was limited to a maximum of 80 mm (3 in.).

The continuous reinforcement, each with a total length of 9.2 m (30 ft), was bent at 10 locations to follow the configuration of the prefabricated structural facade elements. Longitudinal structural continuity was necessary because the concrete cross section did not accommodate reinforcement overlap. The vertical reinforcement was coupled at the nodes by ties along the full widths of the nodes. Figure 4 shows the elevation views of the reinforcement for the three types of prefabricated structural facade elements that were produced. Figure 5 shows a typical cross section of a lattice element, excluding the shear stud, and a typical plan view of a node.

The diameter of the longitudinal reinforcement varied from 22 to 26 mm (0.87 to 1.02 in.) depending on the load imposed on the prefabricated elements along the facade.

Each layer of the facade was designed such that the concrete was not stressed beyond 50% of the compressive strength under service load. The moment design values required the use of doubly reinforced sections because the required ductility could not be provided by single side reinforced sections. The maximum steel ratio for a single side reinforced C60 (8700 psi) concrete section was found to be 3.6%, which was less than the required steel ratio of 4.7%.

To provide the required strength and ductility, a doubly reinforced solution was used that increased the allowable steel ratio of the particular cross section to 6.5%.

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Durability considerations led to limited service load stresses and associated crack widths because the elements were not to be coated for durability and protection. The
highlight of prefabrication for this project was the one-step production of prefabricated concrete elements that met the structural, architectural, and functional requirements.

With the development of high-strength materials and advanced prefabrication methods, slender prefabricated elements with limited stiffness can be designed and produced. Therefore, the explicit service load design of reinforced concrete elements, which is considered by the author to be implicitly and insufficiently covered in ultimate strength-based reinforced concrete design guidelines, becomes all the more important in the design and construction of precast concrete.

Another important aspect of prefabrication in this project was the strength development of the concrete. The production and shipment of precast concrete elements occur before the concrete reaches the design strength. A particularly important phase of prefabrication is the removal of the forms. The economically optimal use of formwork and rate of production required that demolding take place as soon as possible. Early strength as well as 28-day strength of the concrete was closely monitored to rapidly mold and demold the prefabricated products.

Figure 6 shows the variation of strength based on the collected samples for the statistical strength evaluation of the prefabricated facade elements that were designed for a strength of 60 MPa (8.7 ksi).

The high-early-strength white cement allowed the attainment of C20 (2900 psi) class within 24 hours, sufficient to permit form removal.

To estimate displacements at different times, the development of the elastic modulus of the concrete was considered according to the variation of the concrete elastic modulus suggested by the Turkish standard Requirements for Design and Construction of Reinforced Concrete Structures, TS-500.1

The installation of the windowpanes was a critical step that was carefully coordinated with the glass producer so that they could be installed any time after the erection of the prefabricated elements. Therefore, the glass producer had to be supplied with the estimated deformations of the loaded prefabricated facade elements.

The architectural design yielded a limited cross-sectional area with which to meet the serviceability and strength requirements. The limit where the design concrete strength and design crack width is reached has been extended and improved with the addition of compression steel that eases, redistributes, and reduces the development and value of stress in the concrete and limits the crack widths.

The sections were designed so that they were not stressed under service loads beyond 50% of the compressive strength of the concrete. It was determined from numerous compression tests for the concrete used in the design that the behavior of the high-strength concrete was linear up to approximately 55% of its compressive strength. The limited compressive stresses under service loads coupled with double reinforcement around the perimeter of the cross section (Fig. 7) yielded a ductile and strong structural cross section that satisfied the structural requirements for all of the stages of production and
installation. The design reinforcement requirements resulted in the use of 26 mm (1 in.) reinforcing bars along the 200 mm (8.0 in.) wide part of the section and 16 mm (0.63 in.) reinforcing bar along the 100 mm (4.0 in.) wide part.

Based on the cross-sectional dimensions in Fig. 7, the strength design requirements based on the heaviest weak axis design moment value were determined to be:

\[
M_n = 0.9M_u = 0.9\rho f_ybd^2 \left[ 1 - \frac{\beta(0.9)f_y}{\alpha(0.85)f_c} \right]
\]

\[
M_n = (3383)(100) = 0.9\rho(4.285)(138)(138)^2 \left[ 1 - \frac{(0.325)(0.9)(4.285)}{(0.56)(0.85)(610)} \rho \right]
\]

\[
\rho_{\text{required}} = 0.039
\]

where

- \(M_u\) = ultimate moment
- \(M_n\) = nominal moment strength
- \(\rho\) = steel ratio
- \(f_y\) = yield strength of steel
- \(b\) = width of equivalent concrete section
- \(d\) = depth of equivalent concrete section
- \(\beta\) = average depth constant of the resultant compressive force with respect to the concrete strength
\( \alpha \) = average stress constant with respect to the concrete strength

\( f'_c \) = compressive strength of concrete

\( \rho_{\text{required}} \) = required steel ratio

Balanced steel ratio:

\[
\rho_b = \frac{\alpha f'_c}{f_y} \frac{\varepsilon_u}{(\varepsilon_u + \varepsilon_y)} = \frac{(0.56)(610)(0.003)}{4.285 \left( \frac{0.003 + 0.002}{0.003 + 0.002} \right)} = 0.0478
\]

where

\( \rho_b \) = balanced steel ratio

\( \varepsilon_u \) = ultimate concrete strain

\( \varepsilon_y \) = steel strain at yield

Considering the strain hardening of the reinforcement, ACI’s Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05) limit for the steel ratio was found to be at 75% of the balanced value and the TS-500 limitation to be at 70% of the balanced value determined based on the elastic–perfectly plastic behavior.

\[
\rho_{\text{lim(TS-500)}} = 0.70 \rho_u = (0.70)(0.0478) = 0.033
\]

\[
\rho_{\text{lim(ACI-318-05)}} = 0.75 \rho_u = (0.75)(0.0478) = 0.036
\]

where

\( \rho_{\text{lim(TS-500)}} \) = limiting steel ratio according to the TS-500

\( \rho_{\text{lim(ACI-318-05)}} \) = limiting steel ratio according to ACI 318-05

The required reinforcement ratio was greater than that allowed by TS-500 and ACI 318-05. It was decided by the designer to doubly reinforce the section, reduce the concrete compressive stresses, and increase the section ductility.

\( \rho_{\text{required}} = 0.039 > \rho_{\text{lim(TS-500)}} = 0.033 \)

where

\( \rho_{\text{lim(TS-500)}} \) = balanced TS-500 steel ratio

Because the compression steel was close enough to the neutral axis that at ductile failure the compressive stress was below the yield stress of steel, the balanced steel ratio for the doubly reinforced section had to be determined by the compression steel stress value below the yield stress.

\[
\rho_{sb} = 0.7 \rho_{\text{singlesided}} + \rho_{\text{compression}} \frac{f'_c}{f_y}
\]

where

\( \rho_{\text{singlesided}} \) = steel ratio for singly reinforced concrete section

\( \rho_{\text{compression}} \) = steel ratio for doubly reinforced concrete section

\( f'_c \) = steel compressive stress

\[
= (0.003)(200,000) \left( \frac{68 - 37}{68} \right) = 273 \text{ MPa (39.6 ksi)}
\]

where

\( E_i \) = elastic modulus of steel

\( c \) = distance between the outermost part of compressive section and the neutral axis

\( c_p \) = distance between the compressive steel and the neutral axis

\( \rho_{\text{lim(TS-500)}} = (0.7)(0.0478) + (0.556) \frac{2791}{4285} = 0.063 \)

0.063 > \( \rho_{\text{required}} = 0.039 \)

Another reason for incorporating the compressive steel was to increase stiffness for serviceability concerns. The amount of double reinforcement used was more than required in terms of strength design, providing double reinforcement with a limit higher than the required reinforcement level of 3.9%. The service design of slender prefabricated structural elements needed to be evaluated for service load considerations. The exposed concrete structure not only had to satisfy structural design requirements but also architectural and durability requirements. Therefore cracks had to be limited, both in size and number, and the deformations had to be limited to required tolerances. Figure 8 illustrates equivalent cross sections of a doubly reinforced uncracked section and the cracked section.

The neutral axis of the cracked section stressed within its elastic limits is determined to be

\[
\frac{b(kd)^2}{2} + (n - 1)A_{s,top}(kd-c) = nA_{s,bot}(d-kd)
\]

where

\( k \) = uncracked depth factor for singly reinforced section
\[ I = \frac{d^3 b}{12} + b d \left( \frac{d}{2} - k_d d \right)^2 + (n-1) A_{\text{top}} (k_d d - c)^2 + r_{\text{top}}^3 (n-1) \frac{A_{\text{top}}}{12 r_{\text{top}}} + (n-1) A_{\text{bot}} (d - k_d d)^2 + r_{\text{bot}}^3 (n-1) \frac{A_{\text{bot}}}{12 r_{\text{bot}}} \]

\[ = \frac{(138)^3 138}{12} + (138)(138) \left[ \frac{138}{2} - (0.543)(138) \right]^2 + (5.2-1)(1060)[(0.543)(138) - 37]^2 + (26)^3 (5.2-1) \frac{1060}{(12)(26)} \times [138 - (0.543)(138)]^2 + (26)^3 (5.2-1) \frac{1060}{(12)(26)} \]

\[ = 5534 \times 10^7 \text{ mm}^4 (133 \text{ in.}^4) \]

where

\[ I = \text{moment of inertia} \]

\[ k_d = \text{uncracked depth factor for doubly reinforced section} \]

\[ r_{\text{top}} = \text{diameter of top reinforcement} \]

\[ r_{\text{bot}} = \text{diameter of bottom reinforcement} \]

The moment of inertia of the doubly reinforced section cracked up to the neutral axis that occurs when the concrete varies linearly up to its elastic limit can be determined from

\[ I = \frac{(kd)^3 b}{12} + bkd \left( \frac{kd}{2} \right)^2 + (n-1) A_{\text{top}} (kd - c)^2 + r_{\text{top}}^3 (n-1) \frac{A_{\text{top}}}{12 r_{\text{top}}} + (n-1) A_{\text{bot}} (d - kd)^2 + r_{\text{bot}}^3 (n-1) \frac{A_{\text{bot}}}{12 r_{\text{bot}}} \]

\[ = \frac{(0.467)(138)^3 (138)}{12} + (138)(0.467)(138) \left[ \frac{(0.467)(138)}{2} \right]^2 + (5.2-1)(1060)[(0.467)(138) - 37]^2 + (26)^3 (5.2-1) \frac{1060}{(12)(26)} \times [138 - (0.467)(138)]^2 + (26)^3 (5.2-1) \frac{1060}{(12)(26)} \]

\[ = 4583 \times 10^7 \text{ mm}^4 (110 \text{ in.}^4) \]

The limited cracking required by the use of double reinforcement resulted in a stiffness reduction due to an 18% reduction in the moment of inertia. The extra reinforcement was used to limit the theoretical service load cracking and the associated losses in cross-sectional moment of inertia. Without the use of the compression reinforcement, the stiffness reduction was determined to be higher than 30%, which raised serviceability concerns.

**Prefabrication and installation**

The design, production, and construction of the structure took place between March 2009 and August 2010. The design of the prefabricated structural elements took four months, with many revisions and consideration of several core design ideas. The formwork producer participated later in the design stage in generating the construction drawings for the prefabricated structural facade elements. Composite fiber-reinforced plastic formwork was chosen for its ductility and ease of repair for repeated use. The C60 (8700 psi) concrete mixture included white cement and white limestone aggregates with a maximum size of 10 mm (0.39 in.); it had a slump of 150 mm (5.9 in.).

A continuous reinforcement cage without overlaps was a challenge to place into a slender formwork within the tolerances. After a couple of trials, the construction of a complete and continuous reinforcement cage was achieved. A crew of six workers was able to bend and tie the cage for one prefabricated structural facade element in one work day. The reinforcing bar content per cubic meter of concrete in the doubly reinforced sections varied between 250 and 350 kg/m³ (420 and 590 lb/yd³). The fabrication of a single reinforcing cage for each element allowed ease of maneuvering and placement into the composite formwork. **Figure 9** shows the reinforcement cage for type 1 facade elements.

The composite formwork was constructed of a series of smaller forms that were bolted together at the nodes of
Concrete was placed following the placement of the top portion of the formwork. The formwork geometry and material were such that 85% of the surface of the cast product was covered by the formwork, which facilitated curing. Without the use of any special curing schemes, the forms were removed after 24 hours at a strength grade of C20 (2900 psi), which was sufficient for the elements to be lifted and stored for shipment. Each prefabricated unit had four specially designed anchor points that were placed with respect to the center of gravity of the facade element. The units were lifted by an overhead mobile crane and transferred to the storage area (Fig. 11).

Concrete was placed following the placement of the top portion of the formwork. The formwork geometry and material were such that 85% of the surface of the cast product was covered by the formwork, which facilitated curing.

Prior to the production of the facade elements, insulation layers were precision cut to the dimension required between the two layers of concrete. Each facade element had two reinforcement cages. Following the placement of one cage into the formwork, the top plates were installed to be connected to the belt beam during installation and the concrete was placed followed by the placement of the insulation layers. Figure 10 shows a typical prefabrication sequence of the initial layer of the prefabricated structural facade element.

Following the placement of the insulation layer, the second layer of the reinforcement cage was positioned and shear studs were installed through the insulation layer. This created thermal bridging but provided composite action between the two layers of the prefabricated structural facade element.

The setup of the formwork, which included the cleaning, bolting, and fine tuning by installing the specially produced sloped shims, required a full work day for four workers.

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The prefabricated structural facade elements were shipped to the site 20 km (12 mi) from the factory in groups of
construction. Late in November 2010, an earthquake of moment magnitude 4 occurred in the region. During the earthquake, the installed facade elements showed no sign of distress. **Figure 13** shows the installation of the type 1 and type 3 facade elements into the sockets.

Following their installation, the prefabricated structural facade elements were bolted along the points of contact with adjacent elements (**Fig. 14**) to prevent relative lateral translation between adjacent elements.

Following completion of the facade, the steel beams that span between the building and the facade over the atrium space were installed. **Figure 15** shows the completed facade and the placement of the steel belt beams and the radial beams. The beams had a fixed connection to the building and were simply supported on the facade. Following the placement of the beams, corrugated steel decks were placed between the beams with shear studs that penetrated into the cast-in-place concrete roof slab. The construction of the roof was complete with the placement of insulation and a sloped topping. **Figure 16** shows the construction sequence of the roof beams and the composite roof deck.

The installation of the facade elements took approximately two months, followed by the construction of the roof slab and the atrium (**Fig. 17**).

Following the construction of the facade and during the construction of the roof, the window embedded frames were placed into the facade elements. Deformations under the imposed loadings were carefully estimated within the design stage of the prefabricated elements and submitted to the windowpane producer to allow for the deformations and expected movements in the design and construction of the window frames. **Figure 18** shows a perspective of the completed atrium a year before the structure’s inauguration.

three units (**Fig. 12**). At the site they were unloaded and placed into socketed strip foundations and later grouted. During this phase of construction, the facade elements were cantilevered. While the roof support was not present, the facade would have to be supported by its foundation in the event of an earthquake. Therefore the elements were designed to respond as cantilevers in an earthquake during
Figure 12. Transfer of the prefabricated facade elements to the construction site.

Figure 13. Installation of prefabricated facade elements into foundation sockets.
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Conclusion

Design of the prefabricated structural facade with the structural and architectural requirements presented a unique challenge. The prefabricated solution served to advance structural design and construction techniques as well as provide a construction solution that met the architectural and structural requirements of a complex facade system with a single-step prefabricated construction process.

Figure 14. Connection between the neighboring prefabricated facade elements.

Figure 15. Complete prefabricated facade structure.
References


2. ACI (American Concrete Institute) Committee 318. 2005. Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05). Farmington Hills, MI: ACI.

Figure 16. Installation of the radial steel roof beams and cast-in-place composite concrete roof deck.

Figure 17. Interior view of the complete atrium space before internal architectural works.
Notation

\[ I \] = moment of inertia

\[ k \] = uncracked depth factor for singly reinforced section

\[ k_2 \] = uncracked depth factor for doubly reinforced section

\[ M_n \] = nominal moment strength

\[ M_u \] = ultimate moment

\[ n \] = modular ratio

\[ r_{bot} \] = diameter of reinforcement above the neutral axis

\[ r_{top} \] = diameter of reinforcement below the neutral axis

\[ f_c \] = compressive strength of concrete

\[ f_s \] = steel compressive stress

\[ f_y \] = yield strength of steel

\[ g \] = acceleration due to gravity

\[ \alpha \] = average stress constant with respect to the concrete strength

\[ \beta \] = average depth constant of the resultant compressive force with respect to the concrete strength

\[ \varepsilon_u \] = ultimate concrete strain

---

Figure 18. Perspective of the facade.
$\varepsilon_y$ = steel strain at yield

$\rho$ = steel ratio

$\rho_b$ = balanced steel ratio

$\rho_{TS-500}$ = balanced TS-500 steel ratio

$\rho_{\text{compension}}$ = steel ratio for doubly reinforced section

$\rho_{\text{lim(ACI-318)}}$ = limiting steel ratio according to ACI-318

$\rho_{\text{lim(TS-500)}}$ = limiting steel ratio according to TS-500

$\rho_{\text{required}}$ = required steel ratio

$\rho_{\text{singlesided}}$ = steel ratio for singly reinforced section

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**About the author**

Özgür Bezgin, PhD, is chief of the research and development department for Yapı Merkezi Prefabrication Inc. in Istanbul, Turkey. He received his BS in civil engineering from Polytechnic University in Brooklyn, N.Y., after an education in physics engineering at Hacettepe University in Ankara and MS in structural engineering and PhD in substructural and geotechnical engineering from Rutgers, the State University of New Jersey. Throughout his graduate and professional studies, Bezgin conducted research and design work in the field of mechanics of solids, finite element modeling, soil-structure interaction analysis and design and investigation of deep foundations and deep excavation support structures. Bezgin is also a part-time lecturer in Bosphorus University in Istanbul.

**Abstract**

This paper describes the design and construction of a prefabricated structural concrete facade of a university laboratory to study nanomaterials. The precast concrete lattice pattern on the building facade mimics the nanostructure of these materials. The structural design met the requirements of seismic resistance, as was demonstrated by a minor earthquake that occurred during construction. Architectural requirements included maximizing natural light inside the laboratory and a high degree of reflectance. The result is a building with a delicate appearance that is resistant to the seismicity of its location.

**Keywords**

Composite, doubly reinforced, facade, high-strength concrete, prefabrication, white cement.

**Review policy**

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute’s peer-review process.

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