JÖRG SCHLAICH
SCHLAICH BERGERMANN UND PARTNER, STUTTGART

THE CHALLENGE AND JOY OF
STRUCTURAL ENGINEERING

THURSDAY, OCTOBER 4, 2012
EDWARD AND MARY ALLEN LECTURE IN STRUCTURAL DESIGN

Established in 2012, the annual Edward and Mary Allen Lecture aims to bring the world’s leading structural designers to MIT to speak about engineering within the realm of architecture, design, and creativity and to interact closely with students in their design work. The program includes a public lecture in the Architecture Lecture Series as well as a workshop reviewing graduate student structural design projects.

For the inaugural Allen Lecture on October 4, 2012, the eminent German structural engineer, educator, and designer Jörg Schlaich visited MIT to speak about his decades of inspired design work, including long-span roofs, innovative bridges, and multi-story facades, as well as his more recent efforts in solar energy. Along with Edward Allen, Schlaich also worked with graduate students in architecture and engineering on their designs for an enclosed skybridge over Vassar Street on the MIT campus, a project which he had also developed conceptual designs for several years prior.

This booklet commemorates Schlaich’s visit with photographs of his lecture and from his review of student projects, and also contains a survey of the student structural design work.
Jörg Schlaich

Prof. Dr. Ing. Drs. h.c. Jörg Schlaich is a German structural engineer. He studied architecture and civil engineering from 1953 to 1955 at Stuttgart University before completing his studies at the Technical University of Berlin in 1959 (Dr. Ing.). He spent 1959-60 at the Case Western Reserve University in Cleveland, Ohio, USA (MSc.).

Jörg Schlaich was made a partner of the structural engineers, Leonhardt und Andrä and was responsible for the Olympic Stadium Roof, Munich in 1968-72.

From 1974 to 2000 he was full professor and director of the Institute for Concrete Structures (Institut für Massivbau), later called the Institute for Structural Design (Konstruktion und Entwurf) at the University of Stuttgart. In 1980 he and his partner, Dr. Rudolph Bergermann, founded their own firm, Schlaich Bergermann und Partner in Stuttgart.

Most of his projects, as well of that of his company with offices in Stuttgart, Berlin, New York, Sao Paulo and Shanghai, are documented on their website (www.sbp.de). The work of Schlaich Bergermann und Partner focuses on the three main themes of building, surveying and solar energy.

Jörg Schlaich is also the developer of the Solar Updraft Tower for large scale solar energy generation. He is credited with advancing the strut and tie model for reinforced concrete with his seminal 1987 paper, “Toward a Consistent Design of Structural Concrete” in *PCI Journal.*
ABOVE: Jörg Schlaich talking to students about bridge design
RIGHT: Jörg Schlaich with John Ochsendorf
ABOVE: Jörg Schlaich with graduate students in architecture and engineering
LEFT: Jörg Schlaich with John Ochsendorf and Edward Allen
ABOVE AND RIGHT: Jörg Schlaich reviewing student projects with Edward Allen and John Ochsendorf
ABOVE: Jörg Schlaich with Edward Allen and John Ochsendorf at the end of the student project review
RIGHT: Two skybridge designs proposed by Schlaich Bergermann und Partner for the MIT campus
STUDENT DESIGN WORK

The following pages showcase the projects reviewed by Jörg Schlaich during his visit to MIT. The student design work comes from a second-year structural design course in the Masters of Architecture program at MIT, taught by John Ochsendorf and Andrea Love with assistance from Caitlin Mueller. Each presentation proposes a conceptual structural design developed over the course of four weeks for an enclosed skybridge connecting two existing buildings on the MIT campus across Vassar Street.

Before the review, Jörg Schlaich presented the students with two of his own designs for the same project developed several years earlier.
In devising a scheme for a skybridge between buildings 36 and 46, our main architectural objective was to design a visually minimal bridge that did not compete with the surrounding buildings but was still interesting in its own right. The bridge bends in plan to allow for views of the Stata Center and the envelope is clad with folded plates of glass. After several iterations, we decided upon a cable-stayed structural system anchored in both buildings.

**ANT BRIDGE**
SUK LEE & SUSANNA PHO

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**MATERIAL PROPERTIES**
- Concrete Deck: 40 lbs/ft²
- Structural Glass: 35 lbs/ft²

**TREIBURY AREA**
- 175 ft or 53.3 m

**UNIFORM LIVE LOAD**
- 9 ft x (80 lbs/ft² + 30 lbs/ft²) = 990 lbs/ft

**STRUCTURE DEAD LOAD**
- (1041 lbs/ft x 5 ft) + (1090 lbs/ft x 5 ft)

**AXIAL LOAD**
- Crushing:
  \[ A = \frac{F}{\sigma} = \frac{89.94 k}{3 \text{ ksi}} = 29.97 \text{ in}^2 \]
  \[ h = 3.74 \text{ in} \text{ or } 95 \text{ mm} \]
- Buckling:
  \[ I = \frac{(kL)^2 \times P_{cr}}{\pi^2 E} = \frac{((74 \text{ ft x 12 in}))^2 \times 89.94 k}{\pi^2 \times 3,000 \text{ ksi}} = 2395.29 \text{ in}^4 \]
  \[ bhv/12 = 2395.29 \text{ in}^4 \text{ (b=width=8 in)} \]
  \[ h = 15.32 \text{ in} \text{ or } 389.19 \text{ mm} \]
- Moment:
  \[ M = \frac{wL^2}{16} = \frac{3.12 k/\text{ft}}{7.4 \text{ ft}}^2/16 = 10.68 k-\text{ft} \text{ = } 128.16 k-\text{in} \]
  \[ S = \frac{M}{\sigma} = \frac{128.16 k-\text{in}}{3 \text{ ksi}} = 42.72 \text{ in}^3 \]
  \[ S = \frac{I}{y} = \frac{6}{h^3} \]
  \[ h = 0.52 \text{ in} \text{ 13.20 mm} \]

**LOAD CONDITIONS**

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**AXIAL LOAD CONDITIONS**

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**GIRDERSIZING**

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**CABLE SIZING**

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Stainless steel rod is used instead of tendon cable to reduce the amount of possible sag.
In devising a scheme for a skybridge between buildings 36 and 46, our main architectural objective was to design a visually minimal bridge that did not compete with the surrounding buildings but was still interesting in its own right. The bridge bends in plan to allow for views of the Sta Center and the envelope is clad in folded plates of glass. After several iterations, we decided upon a cable-stayed structural system anchored in both buildings.

### Force Polygon

#### Bridge B

- Maximum Tension Member B: 7.8 k

#### Bridge A

- Maximum Tension Member A: 7.8 k

### Axial Load Analysis

**B** 109.20 k

**M** 485.75 k

### Deflection Analysis

**INCREASED OVERALL DEPTH OF BEAM TO PREVENT DEFLECTION.**

### Girder Sizing

- **Concrete deck:** 40 lbs/ft²
- **Structural glass:** 35 lbs/ft²
- **Tributary area:** 5 ft or 1.52 m
- **Uniform Live Load:** 9 ft x (80 lbs/ft² + 30 lbs/ft²) = 990 lbs/ft
- **Structure Dead Load:** (1041 lbs/ft x 5 ft) + (1090 lbs/ft x 5) = 128.16 k

### Deflection Analysis

Increased overall depth of beam to prevent deflection.

### Cross Section

**Crushing:**

- **A** = \( F / \sigma \)
- h = 3.74 in or 95 mm

**Buckling:**

- **I** = \( (kL)^2 \times P_{cr} \) / (\( \pi^2E \))
- **bhv/12** = 2395.29 in⁴
- h = 15.32 in or 389.19 mm

**Moment:**

- **M** = \( wL^2/16 \)
- S = **M** / **\sigma** = 128.16 k-in / 3 ksi = 42.72 in³
- S = **I** / **y** = 6/h³
- h = 0.52 in 13.20 mm

### Point Load

- (990 lbs/ft x 5 ft) + (2131 lbs/ft x 5 ft) = 15.6 k
- Point Load on one side of structure: 15.6 k / 2 = 7.8 k or 34.7 kn

### Tension Rods

- **Maximum Tension force on Tension Rods:** 12.28 k or 54.62 kn
- Diameter = 0.5 in, Stainless Steel Rod with allowable force: 12.56 k
- A = F / \( \sigma \) = 12.28 k / 15 ksi = 0.82 in²
- Diameter = 1.02 in or 25.91 mm

Stainless Steel rods with 125 IN or 31.75 MM diameter will be used to carry tension force.

EXPOSED EMBEDDED STEEL PLATE CUSTOMIZED TO ACT AS CABLE ANCHORING SYSTEM

STEEL REINFORCEMENT

CABLE-DECK CONNECTION DETAIL

CABLE-DECK CONNECTION DETAIL

MAXIMUM TENSION FORCE ON TENSION RODS: 12.28 K OR 54.62 KN

DIAMETER = 0.5 IN STAINLESS STEEL ROD WITH ALLOWABLE FORCE: 12.56 K

A = F / \( \sigma \) = 12.28 k / 15 ksi = 0.82 IN²

DIA = 1.02 IN OR 25.91 MM

STAINLESS STEEL RODS WITH 125 IN OR 31.75 MM DIAMETER WILL BE USED TO CARRY TENSION FORCE.
FLOATING FOOT BRIDGE

LOADS
- Floor dead load: 0.244 kip/ft [0.004 kN/m]
- Floor live load: 0.800 kip/ft [0.012 kN/m]
- Roof dead load: 0.329 kip/ft [0.006 kN/m]

TOTAL:
- 1.800 kip/ft [0.026 kN/m]

LOAD CASE 1 UNIFORM LOADING
- Free body diagram
- Sizing the cable
- From the cable sizing table:
  - 1-7/8" [47.625 mm] diameter, class "A" coating throughout
  - Breaking strength: 216.0 tons

LOAD CASE 2 ASYMMETRICAL LOADING
- Free body diagram
- Sizing the girder/strut
- Sizing the secondary strut
- Curtain wall system:
  - 1/4" Glass Roof
  - 1/4" Glass Floor
  - 2" Steel Diagrid Mesh
  - 1.5 x 1.5 x 3/16 HSS
  - 1/4" Glass Cladding

DETAIL
- Cable connection
- Section
- Floor system
- Roof system
- Facade system

SHIMURA YANG & YAN-PING WANG
FLOATING FOOT BRIDGE

LOAD CASE 1

LOADS

- 0.61 m [C = 1.89 kips]

SIZING THE SECONDARY STRUT

- Force polygon
  - 0.329 kip/ft
  - roof dead load: 0.800 kip/ft
  - floor dead load: 0.004 kN/m

- 8 ft
  - 2 ft

206.58 tons (1837 kN)

180 kips

- \( \frac{7.15 \text{kips}}{wL^2} \)
- Assume 18 inch strut depth
  - \( d = 0.75 \text{ ft} \)

- \((1 \cdot 110 \text{ ft} \cdot 12 \text{ in})^2\)

UNIFORM LOADING

- 835.38 kN
- 187.8 kips

- \( \text{Steel} = 15 \text{ ksi} \)
- 180 kips
- \( \text{breaking strength: 216.0 tons} \)

- Class “A” coating throughout

- \( 38.1 \text{ mm} \times 38.1 \text{ mm} \times 4.76 \text{ mm} \)
  - 1½ in x 1½ in x 3/16 in

- 38.1 mm

- 38.1 mm

- 2.133 m

- 0.46 kip/ft

- 1.89 kips

- 180 kips

- \[9.78 \text{ cm}^4\]

- \[5.42 \text{ cm}^2\]

- \( A = 0.84 \text{ in}^2 \)

- \( 0.124 \text{ kip/ft} \)

- Cladding: 0.002 kN/m
- Roof live load: 0.004 kN/m

- 17.81 cm²

- \( 0.026 \text{ kN/m} \)

- \( 0.02 \text{ m}^2 \)

- \( 0.02 \text{ m}^2 \)

- \( \Delta = (2 \cdot 30\text{ in}^2 \cdot (52\text{ in})^2) + (2 \cdot 2.76\text{ in}^2 \cdot (0\text{ in})^2) + (2 \cdot 20\text{ in}^2 \cdot (78\text{ in})^2) \)

- \( M_{\text{max}} = (\text{Estee} \cdot 2900 \text{ k/in}^2) \)
- Length = 1320 in

- \( I = \frac{\pi d^4}{16} \)

- \( \sigma = \frac{1.8 \text{ kips}}{y=0} \)

- 64 in

- 12 in

- 1 ft

- \( \sigma = \frac{5(0.09 \text{k/in})^4}{0.0006 \text{ m}^4} \)

- \( 340 \text{ kip•ft} \)

- 3.7 in

- \( \geq \)

- \( 3.46 \text{ in} \)

- \( \leq 3.7 \text{ in} \)

- \( \Delta \)

- Section

- FACADE SYSTEM

- ROOF SYSTEM

- FLOOR SYSTEM

- Steel Diagrid Mesh

- 1 7/8” Steel Cable

- 1/4” Glass Cladding

- 1.5 x 1.5 x 3/16 HSS

- 1/4” Glass Roof

- 160 lb/ft
- 164 kg/m

- 150 lb/ft
- 51 kg/m

- 34 lb/ft
- 34 lb/ft

- 32500 mm

- 51 kg/m

- 51 kg/m

- 38.10

- 28.54

- 6.35

- 38.10

- 50.80

- 12.5

- 152.40

- 508.00

- 50.80

- \( \text{Section Properties} / \text{Self Weight} \)

- RENA YANG & YAN-PING WANG
DESIGN APPROACH::

Vassar Street is one of MIT's prominent streets located on the north edge of MIT campus. Nearby the site is Frank Gehry's Stata Center and Charles Correa's Brain and Cognitive Sciences Building. Despite many well known buildings, Vassar Street lacks a visual key urban element that brings people to this street. With these observations, his proposal attempts to achieve the following criteria:

• Create a visual focal point on Vassar St. using an archway as an indicator of a grand passageway;
• Maximize transparency to maintain the openness that Vassar Street currently has;
• Create a comfortable and visually engaging experience for the skybridge users;

GLOBAL SHAPE:

Using force polygon

LOADS:

Live Load:

- 30 lb/ft²
- 80 lb/ft²
- 110 lb/ft²

Dead Load:

- (density of steel= 490 lbs/ft³)
  assuming that we use 2" of steel (1/6 of a foot),
  × 490 lbs/ft³ × 1/6 = 82 lbs/ft²

- (density of concrete= 150 lbs/ft³)
  assuming that we use 6" of concrete (1/2 of a foot),
  × 150 lbs/ft³ × 1/2 = 75 lbs/ft²

- (density of glass= 162 lbs/ft³)
  assuming that we use 2" of glass (1/3 of a foot),
  × 162 lbs/ft³ × 1/3 = 54 lbs/ft²

- 15 lb/ft²

UNIFORM LOADING CASE (w):

Live Load+ Dead Load = 336 lbs/ft²

336 lbs/ft² × 107 ft = 33600 lbs = 3.36 kips/ft

3.36 kips/ft for two arches = 1.68 kips/ft for 1 arch

1.68 kips/ft × 107 ft = 179.76 kips

> 179.76 kips/19 segments = 9.46 kips/seg

Previous Iterations

Final Force Polygon
The proposal attempts to achieve the following criteria:

- Create a comfortable and visually engaging experience for the skybridge users;
- Maximize transparency to maintain the openness that Vassar Street currently has;
- Create a visual focal point on Vassar St. using an archway as an indicator of a grand passageway;
- His proposal attempts to achieve a visual key urban element that brings people to this street.

Vassar Street is one of MIT's prominent streets located on the north edge of MIT campus. Nearby the site is Frank Gehry's Stata Center and Charles Correa's Brain and Cognitive Sciences Building. Despite many well-known buildings, Vassar Street lacks

**DESIGN APPROACH:**

1. **Primary Member Sizing:**
   - **Steel Cables in Tension**
     - 15 ksi = 10 kips / A
     - A = 0.67 in²
     - \( \sigma = 15 \text{ ksi} = 10 \text{ kips} / \text{A} \)
   - **Steel Column in Compression**
     - 15 ksi = 10 kips / A
     - A = 0.67 in²
     - \( \sigma = 15 \text{ ksi} = 10 \text{ kips} / \text{A} \)

2. **Secondary Member Sizing:**
   - **Steelf Cables in Tension**
     - 15 ksi = 10 kips / A
     - A = 0.67 in²
     - \( \sigma = 15 \text{ ksi} = 10 \text{ kips} / \text{A} \)
   - **Steel Column in Compression**
     - 15 ksi = 10 kips / A
     - A = 0.67 in²
     - \( \sigma = 15 \text{ ksi} = 10 \text{ kips} / \text{A} \)

3. **Dead Load**
   - \( w = 0.11 \text{ kips/ft}^2 \times 10 \text{ ft} / 2 \text{ arch} = 0.55 \text{ kft} / \text{A} \)
   - \( \sigma = 15 \text{ ksi} = 97 \text{ kips} / \text{A} \)
   - \( I_{\text{required}} = 1836.5 \text{ in}^4 \)

4. **Live Load**
   - \( w = 110 \text{ lbs/ft}^2 \times 10 \text{ ft} + 107 \text{ ft} = 117.7 \text{ kips} \)
   - \( \sigma = 15 \text{ ksi} = 97 \text{ kips} / \text{A} \)

5. **Deflection due to Asymmetrical Loading**
   - \( w = 226 \text{ lbs/ft}^2 \times 226 \text{ lbs/ft}^2 + 0.26 \text{ kips/ft} \)
   - \( M_{\text{Max}} = 145.5 \text{ kip-ft} = 194 \text{ kip-ft} / \text{m} \)

6. **Maximum Moment Due to Asymmetrical Loading**
   - \( M_{\text{Max}} = (wLL^2)/32 \)
   - \( \sigma = P/A \)
   - \( \sigma = 15 \text{ ksi} = 97 \text{ kips} / \text{A} \)

7. **Tension**
   - \( T = 145.5 \text{ kip-ft}/(12 \text{ in/ft}) / 6 \text{ in} = 291 \text{ kips} \)
   - \( \sigma = P/A \)
   - \( \sigma = 15 \text{ ksi} = 97 \text{ kips} / \text{A} \)

8. **Compression**
   - \( \sigma = P/A \)
   - \( \sigma = 15 \text{ ksi} = 97 \text{ kips} / \text{A} \)

9. **Torsion**
   - \( T = 145.5 \text{ kip-ft}/(12 \text{ in/ft}) / 18 \text{ in} = 97 \text{ kips} \)

When we used HSS20X18X5/8 as sectional properties in Multiframe software to calculate, the highest distance of deflection was 3.867 inches as shown in the right graph. Therefore, we had to increase the sectional dimensions in order to accommodate \( \Delta_{\text{Max}} = 5 \text{ in} \). Given \( \Delta_{\text{Max}} = 5 \text{ in} \), the new max deflection \( \Delta_{\text{Max}} = 2.37 \text{ inches} \) as shown on the right graph and new section is HSS22X20X5/8.
2-in-1

EVELYN TING, JIE ZHANG

LOAD ANALYSIS
1. Main truss cord member
   Size = 6 x 6 x 3/16 inch
   Weight = 42.06 lbs/ft
   Cross sectional area = 6.18 in^2
   S = 28 Sl (ft^2) / 304.8 in^4

2. Main truss web / secondary truss compression member
   Size = 2 x 2 x 1/8 inch
   Weight = 0.54 lbs/ft
   Cross sectional area = 0.13 in^2

3. Secondary truss tension member
   Size = 2.5 x 2.5 x 3/8 inch
   Weight = 2.00 lbs/ft
   Cross sectional area = 0.47 in^2

4. Tertiary beam
   Size = 3 x 3 x 1/2 inch
   Weight = 1.50 lbs/ft
   Cross sectional area = 0.48 in^2

5. Floor system
   Composite floor system of 2.25 inch thickness (max. thickness of concrete = 1.5 in,
   max. corrugation thickness of metal decking = 1.5 in)
   Density of Concrete = 150 lbs/cubic foot
   Weight of Metal/Decking = 30 lbs/ft

6. Cladding material
   Glass for thermal insulation
   Metal mesh for architectural expression

BUILDING SYSTEMS

ENVELOPE
Desired effects: varying opacity, openness, and
Primary material: steel mesh
TEXTURE TO EXPRESS THE ARCHITECTURAL FIGURE

CONCEPT
This design was conceived as an intersection of two bridges. Two lookouts branch off the bridge, sloping up and down on either side to provide views out onto the surroundings. They create specific views and invite visual dialogue with street traffic, making the bridge both a circulation path and a gathering space. However, the use of glass and metal mesh attempts to bring a sense of lightness and transparency to the site.

UNDER UNIFORM LOAD
TRUSSES FOR LOOKOUT POINTS
For Truss 1
Truss self weight = 111.0 kips / 350 sf = 318.5 kips / 10 ft
Total Load = 207.4 kips (0.35 lbs/sf) + 2.65 kips = 33.1 kips
Reaction force (ground) = 15.6 kips / 100 ft = 15.6 kips
Max. axial tension force = 16.0 kips
Max. cross sectional area = 1.26 kips / 10.3 kips
Max. compression force = 16.83 kips over 35.09 sf
Bag = 0.704 in / 42.7 in^2
Structure for truss 1 is sound.

For Truss 2
Truss self weight = 16.5 kips
Total load = 61.25 kips (110 lbs/sf) + 10 kips = 10 kips
Reaction force (ground) = 15.1 kips / 100 ft = 15.1 kips
Member sizes are sufficient for Truss 2 with significantly smaller loads.

TRUSSES FOR MAIN BRIDGE
Self-weight of one truss = 37.6 kips / 110 sf = 347.3 kips / 110 sf
Total load = 1.52 kips / 110 lbs/sf + 0.55 kips = 10 kips

BEAM ANALOG METHOD FOR VERIFYING CORD MEMBER SIZING
Maximum moment = 1750 kips / 10 ft
Tension/compression force = 17.5 kips ( allowable)
Crushing = 0 lbs / 10 ft
Crushing force = 0.3 kips / (allowable)
Surface area = 0.75 in^2 (allowable)

MULTIFRAME ANALYSIS FOR VERIFYING DIAGONAL MEMBER SIZING
Maximum axial stress in webs = 78.96 kips (tension) / 67.63 kips (compression)
Maximum axial stress in cords = 136.24 kips (tension) / 134.56 kips (compression)
Maximum moment on diagonal bracing = 907.80 kips ft; required sectional modules
Multiframe results are consistent with hand calculations. Steel sections will not crush or buckle under given load. The structure is...
Plan Connection Detail 1 - Floor

Total load = Dead Load + Live Load

- Other systems’ self-weight = Floor + Ceiling + Facades = 45 lbs/sf
- Approximate facade weight = 15 lbs/sf
- Ceiling system is estimated to weigh half of the floor system = 10 lbs/sf, or 10 lbs/sf

Total Load (floor) = 20 lbs/sf, or 202 lbs/ft

Total Load (floor beam) = 3.05 lbs/ft x 10 ft x 16 = 0.488 kips; distributed = 4.4 lbs/ft

Floor deck consists of 16 short diagonal beams and 1 long diagonal beam (counted under Truss self-weight).

S = 0.486 in^3

Size = 2 x 2 x 1/8 inch

Metal mesh for architectural expression

Glass for thermal insulation

Cladding material

Weight of Metal Decking = 1 lbs/sf

Max. corrugation thickness of metal decking = 1.5 in

Composite floor system of 2.25 inch thickness (max. thickness of concrete = 1.5 in,

Load Analysis

Snow load = 30 lbs/sf

Total height = 13 ft

K (fixity) = 1

Allowable Stress (concrete) = 1.5 ksi

Modulus of Elasticity (steel) = 29,000 ksi

Allowable Stress (steel) = 15 ksi

Steel sections will not crush or buckle under given load; The structure is sound.

Multiframe results are consistent with hand calculations;

Maximum moment on diagonal bracing = 93.978 kips x ft; required sectional modulus = 6.27 < 8.67 in^3

Maximum axial stress in webs = 78.96 kips (tension) / 67.63 kips (compression)

Multiframe analysis for verifying diagonal member sizing

Tension/compression force in cords = 1770 kips x ft / 13 ft = 136.1 kips

Max. moment = 1770 kips x ft

Uniform Load = 15.23 kips/110 ft + 0.45 kips (other systems) + 1.1 kips/ft (live loads) = 1.7 kips/ft

Floor System (steel beam)

- Total load = 30 lbs/sf x 300 sf = 9000 lbs
- Total load = 110 lbs/sf x 300 sf = 33,000 lbs
- Total load = 0.55 kips = 1110 lbs

- Structure for truss 1 is sound.

- Req. I = 10.44 in^4 < 21.7 in^4

- Max. axial compression force = 16.83 kips over 35.09 ft

- Req. cross sectional area = 1.26 in^2 < 1.51 in^2

Floor System (composite slab)

- Total load = 20 lbs/sf x 300 sf + 80 lbs/sf x 300 sf = 30 kips

- Tributary Area = 300 sf

- FLOOR SYSTEM (steel beam)

- Secondary truss tension member

- Cross sectional area = 5.00 in^2

- Size = 8 x 8 x 3/8 inch

- Weight = 22.37 lbs/ft

- Cross sectional area = 6.18 in^2

- Size = 2 x 2 x 1/4 inch

- 3. Secondary truss tension member

- Cross sectional area = 8.67 in^3

- Size = 2 x 2 x 1/8 inch

- Weight = 11 lbs/ft

- Cross sectional area = 1.51 in^2

- Size = 2 x 2 x 1/8 inch

- 4. Tertiary beam

- Cross sectional area = 13.4 in^2

- Size = 12 x 12 x 5/16 inch

- 2. Main truss web

- Cross sectional area = 24.9 in^3

- Size = 8 x 8 x 3/8 inch

- 1. Main truss cord member

- Cross sectional area = 5.00 in^2

- Size = 2 x 2 x 1/8 inch

- TRUSSES FOR MAIN BRIDGE

- Tributary area = 61.25 sf x (45 + 110) lbs/sf + 0.55 kips = 10 kips

- FLOOR SYSTEM (composite slab)

- The composite slab is rested between the main cords, and the tertiary beams effectively shorten the span of the slab, allowing the thickness of the slab to be reduced.

- Concrete allowable stress = 1 ksi

- M = (wL^2)/8 = (0.77 k/ft x 35 ft x 35 ft) / 8 = 131.79 k*ft

- Req. Section Modulus of beam A = (M/L) / (f/sq in) = 13.1 sq/in

- Req. Section Modulus of beam B = 8.42 sq/in; 0.446 sq/in

- UNDER ASYMMETRICAL LOAD MULTIFRAME ANALYSIS

- The trusses are braced by diagonal floor and ceiling beams and act as rigid tubes under asymmetrical load.

- MULTIFRAME ANALYSIS

- Axial stress = 15 kips = allowable stress.

- The truss structure is sound under asymmetrical load.

- Deflection analysis - Multiframe analysis

- Maximum deflection = 3.687 in < 0.7 in (allowed deflection)

- Hand calculations are inaccurate as the application of the equation assumes a well-braced truss whereas in actuality, diagonal bracings are compromised at the openings to allow circulation into lookout spaces.
FAT BELLY

INTERIOR VIEW

PLAN

LONGITUDINAL SECTION

LOADING

PRIMARY STRUCTURE (LONGITUDINAL SECTION)

ASYMMETRICAL LOADING DEFLECTION

EXPLODED AXONOMETRIC

DEAD LOAD

Steel tube load (x) = 52,200.0 pounds

carried at floor weight per unit area = 80.0 pounds per square foot

carried at roof weight per unit area = 30.0 pounds per square foot

skylight glass体积=174.0 cubic feet

+ 40'-3" A.S.L.

F.F.L.

+ 50'-3" A.S.L.

T.O. SIDEWALK

+ 20'-3" A.S.L.

w=2.5kips / linear foot

165.9 kips 737.8

load per segment = 25.2 kips 112.1

segment length = 10.3 feet 3.1 meters

weight by volume = 150.0 pounds per cubic foot

111.3 kips 495.0

HSS

18x12x5/8 13019.0 lbs

load allowance = 41,730.0 pounds

DEAD LOAD

2.5 kips/feet 35.8

total span = 113.0 feet 34.4 meters

weight = 56,500.0 pounds

weight = 1186.3 pounds

area = 1391.0 square feet

imperial units
metric units

= (wL^2)/8

Maximum Moment

kN/meters

PRIMARY STRUCTURE (LONGITUDINAL SECTION)

ASYMMETRICAL LOADING DEFLECTION

EI/(kL)^2

π

σ

required cross sectional area (A) = 28.3 square inches 18236.5 square mm

MOMENT DIAGRAM

Mmax

CALCULATION

FOR AVERAGE GLOBAL

steel allowable stress

for finding centroid

arbitrary y axis,

d1, d2, d3

22.0

87.28 "

87.3 in Location of centroid

22.0

67.87 " 19.41 "

67.9 in Distance between Column Center and Cable Center

22.0

3.1

rod diameter = 1.3 inches 32.7 mm

Maximum Moment (Mmax) =

87.28 "

67.87 "

15.0 kips/square inch (safety factor included)

MAXIMUM ALLOWED DEFLECTION

Max Deflection

Length (L) = 1320.0 in

Mmax

Length (L) = 1320.0 in

AREA A1= in2

= F/A

Area A2= in2

= F/A

moment of inertia (I) = 25.4 inches^4 10590753.2 Mm^4

Area A3= in2

= F/A

required moment of inertia (I)= 33,459

Max Deflection (max) =

required cross sectional area (A) = 7.3 square inches 4719.7 square mm in compression and tension blocks

number of compression blocks = 1.0 members

rod diameter = 1.3 inches 32.7 mm

maximum axial load (F) = 109.7 kips 488.1 (from force polygon)

working load per cable = 245.0 kips 1089.8

total max axial load (P) = 22.7 kips 101.0 (from force polygon)

total max axial load (P) = 424.0 kips 1886.0 (from force polygon)

total max axial load (F) = 424.0 kips 1886.0 (from force polygon)

tension rod- axial

buckling safety factor = 3.0

number of members = 1.0 members

buckling length (L) = 123.0 inches 3124.2 mm

area per member = 1.5 square inches 976.3 square mm

COMPRESSION –

steel allowable stress

maximum axial stress = 490.0 kips 2179.6 (from force polygon)

26.6 KIPS (C)

22.0

19.4 in

67.9 in

87.3 in

67.9 in

3.1

area per member = 1.5 square inches 976.3 square mm

required cross sectional area (A) = 1.5 square inches 976.3 square mm

15.0 kips/square inch (safety factor included)

steel allowable stress

number of members = 1.0 members

rod diameter = 1.3 inches 32.7 mm

Maximum Moment (Mmax) =

87.28 "

67.87 "

15.0 kips/square inch (safety factor included)

26.6 KIPS (C)
**STRUCTURE - LONGITUDINAL SECTION**

- **1.8 IN. DIAMETER STEEL CABLE**
- **2-7/8" STEEL CABLE**

**SECONDARY STRUCTURE - CROSS SECTION**

- **HSS 18x12x5/8**
- **6" CONCRETE OF 3" METAL DECK**

**CABLE AXIAL STRESS**

- Steel rod diameter: 3.81 mm
- Steel allowable stress: 15.0 kips/square inch (safety factor included)

**AXIAL STRESS**

- Cross sectional area: 1.3 square inches 838.7 square mm
- Maximum axial load: 22.7 kips 101.0 (from force polygon)

**TOTAL MAX AXIAL LOAD**

- Total max axial load: 424.0 kips 1886.0 (from force polygon)

**BUCKLING**

- Fixity: 1.5 fixed/pin
- Buckling safety factor: 3.0
- Buckling length: 123.3 inches 3124.2 mm

**DEAD LOAD**

- Weight per foot: 9.5 pounds per foot
- Total span: 113.0 feet 34.4 meters
- Weight: 26100.0 pounds

**DISTRIBUTED LIVE LOAD**

- Segment length: 10.3 feet 3.1 meters
- Weight: 1886.0 pounds
- Load segments: 11.0 segments
- Load allowance: 41730.0 pounds

**REINFORCED CONCRETE**

- Area: 1391.0 square feet
- Moment of inertia: 8484.1 kip-feet

**SECONDARY STRUCTURE (CROSS SECTION)**

- Area: 3.1 square inches 18236.5 square mm
- Moments of inertia: 22.0 sq.in, 3.1 in^4

**CALCULATION**

- Maximum Moment
  - for finding centroid
  - arbitrary y axis,
  - maximum moment

**DEFLECTION**

- Calculating centroid for finding the centroid
  - Distance from centroid
  - Moment

**STRESS**

- Axial Load (T, C) = 109.7 kips
- Moment (M) = 109.7 kip*ft

**STRAIN**

- Load length (L) = 56.5 ft
- Depth (d) = 1.0 feet

**ALLOWED DEFLECTION**

- Maximum deflection
  - Maximum allowed deflection
  - Deflection (Δ) = L/360

**HYPERBOLIC AREA**

- Area A1 = 28.3 square inches 18236.5 square mm
- Area A2 = 3.1 square inches 18236.5 square mm
- Area A3 = 22.0 square inches 18236.5 square mm

**CALCULATION CENTRE**

- Distance between column and cable center
  - Distance between column and cable center

**REQUIRED MOMENT OF INERTIA**

- Applied live load
  - Length (L)
  - Moment (M)

**AVERAGE GLOBAL MOMENT OF INERTIA**

- Moment of inertia

**MINIMUM ALLOWED DEFLECTION**

- If an Error in Δmax...
The gray, monolithic concrete buildings on Vassar street dominate the view of the people passing through. Therefore, we strived for a design that had minimal visual impact on the site. Also, since the Stata Center already has a very strong presence on the street, a simple bridge would not interfere with the view so highly sought after by tourists and also would not clutter up an already packed street.

### Loads

- **Snow Load**: 30 lb/ft, 80 lb/ft, 42 lb/ft, 20 lb/ft, 2 lb/ft, 58 lb/ft, 232 lb/ft
- **People Load**: 0.4 kN/m, 1.1 kN/m, 0.6 kN/m, 0.3 kN/m, 0.03 kN/m, 0.8 kN/m, 3.4 kN/m

### Force Polygon

- **Total Distributed Load**: 249 K, 1106 kN
- **Total Load**: 12.5 K, 55.6 kN

### Member Sizing

#### Primary Compression Member Sizing

- **Check for Building over 1/8**: 1/2 = 0.53 ft
- **Check for Snow Loading Condition**: 109k < 1290 (0.09)/(1320 in)4 < 23470 cm3
- **Check for Snow Loading Conditions**: A = 30.8 ft x 80 lb/ft = 110 lb/ft = 0.11 k/ft
- **Check for Load**: X = 0.11 k/ft = 0.17 k/in, Y = 0.11 k/ft = 0.17 k/in
- **Check for Max Allowable Stress**: E = 29,000 ksi, P = (2EI)/(kL)2
- **Check for Max Allowable Stress**: Largest force = 158 K

#### Secondary Compression Member Sizing

- **Check for Building**: A = 18.7 in2, I = 702 in4
- **Check for Max Allowable Stress**: Largest force = 164 K

#### Primary Tension Member Sizing

- **Check for Building**: A = 18.7 in2, I = 702 in4
- **Check for Max Allowable Stress**: Largest force = 10.5 k

#### Secondary Tension Member Sizing

- **Check for Building**: A = 11.6 in2, I = 80.5 in4
- **Check for Max Allowable Stress**: Largest force = 6.5 k
PRIMARY TENSION MEMBER SIZING

The view so highly sought after by tourists and also would not clutter up an already packed street. Stata Center already has a very strong presence on the street, a simple bridge would not interfere with through. Therefore, we strived for a design that had minimal visual impact on the site. Also, since the gray, monolithic concrete buildings on Vassar street dominate the view of the people passing through.

FORCE POLYGON

Snow Load 30 lb/ft

LOADS

Steel Struts (x10) Steel Arch (x4)

3/4" Glass Cladding

People Load Deck

Total Load Total Distributed Load

MEMBER SIZING

CONCEPT

Transparency

Site Plan

17.8cm x 17.8cm x 1.3cm
Choose 7"x7"x1/2" Square HSS

Area req = 164.3K / 15ksi = 10.9 in²

Largest Force = 164.3K = Checking for Max Allowable Stress

Ix = 702 in⁴ Iy = 452 in⁴

Area req = 158K / 15ksi = 10.5 in² = 67.7 cm²

15 ksi = 158K / area

Stress = force / area

Largest force = 158K = Checking for Max Allowable Stress

M = □L

= 30lb/ft + 80 lb/ft = 110 lb/ft = 0.11 k/ft
ω

P = □

E = 29,000 ksi

L/2 = 53.6 ft

S = 31.6 in

σ = 15ksi = 474 k*in/S

σ □ = 15 ksi = M/S

158K = □

k = 1 (pin-pin condition)

M = [(0.11k/ft)(107.2ft)²]/32 = 474 k*in

Ireq = 228 in⁴ = 9490 cm⁴ --> 684 in⁴ = 28470 cm⁴ (safety factor)

Sx = 87.7 in³ Sy = 75.3 in³

40.6cm x 30.5cm x 0.9cm
Choose 16"x12"x3/8" Rectangular HSS

= Check for Buckling over L/2

= Checking for Uneven Loading Conditions

SECONDARY COMPRESSION MEMBER SIZING

Iy = 3.2 in⁴

Ix = 4.9 in⁴

Choose member 4"x3"x3/16" 10.2cm x 7.6cmx 0.5cm

Area req = 9.7 K/15ksi = 0.65 in²

15 ksi = 9.7 K/Area req

Largest force = 9.7 K = Check for Max Allowable Stress

Ireq = 0.97 in⁴ = 40.4 cm⁴ --> 2.91 in⁴ = 121 cm⁴ (safety factor)

9.7 K = □

2(29,000 ksi)I/(1(144 in))²

P = □

2EI/(kL)

E = 29,000 ksi

= Check for Buckling

Moment Diagram

Shear Diagram

606 K*ft

9.7 ft

62.5 K 62.5 K

278 kN

12.5K 12.5K

55.6 kN

12.5K 12.5K 12.5K

232 lb/ft

58 lb/ft

2 lb/ft

20 lb/ft

42 lb/ft

80 lb/ft

3.4 kN/m

0.8 kN/m

0.03 kN/m

0.3 kN/m

0.6 kN/m

1.1 kN/m

0.4 kN/m

L

11 10 9 8 7 6 5 4 3 2 1

1/4 inch = 1 kip

Force in vertical strut = 9.7K = 46kN

Maximum Moment = 1818 K*ft

K 0

I

H

G

F

E

D

C

B

A

158K 703K

12.5K

55.6 kN

12.5K 12.5K 12.5K 12.5K 12.5K 12.5K 12.5K 12.5K

0 K*ft

-121 K*ft

-242 K*ft

-364 K*ft

-485 K*ft

-606 K*ft

KA B C D E F I J

DEFLECTION OF STRUCTURE

I = I + A d²

PARALLEL AXIS THEOREM

I = 2*[702 in⁴ + 18.7 in²*(23.1 in)²] + 2*[80.5 in⁴ + 11.6 in²*(33.6 in)²]

I = 47713.9 in⁴

ω L⁴) / 384*EI

max = (5*(0.09 K)(1320in)⁴) / (384*(29000 K/in²)(47713.9 in⁴))

PRIMARY COMPRESSION MEMBER SIZING

Concrete Deck

Tension Cable

Steel Structure for Cladding

Sky Illusion
The skywalk is supported by 2 barrel vaults along the edges. The trusses are a series of arches on the top. The barrel vault shape of the bridge is then wrapped with sheets.

Dead load distributed evenly, live load on half.

Distributed load for 18' span

\[ A = \frac{3.66k}{30.64k} \]

\[ B = \frac{3.66k}{30.64k} \]

\[ C = \frac{3.66k}{30.64k} \]

\[ D = \frac{6k}{30.64k} \]

\[ E = \frac{6k}{30.64k} \]

\[ F = \frac{6k}{30.64k} \]

\[ G = \frac{6k}{30.64k} \]

\[ H = \frac{6k}{30.64k} \]

\[ I = \frac{6k}{30.64k} \]

\[ J = \frac{6k}{30.64k} \]

\[ K = \frac{6k}{30.64k} \]

\[ L = \frac{6k}{30.64k} \]

\[ M = \frac{6k}{30.64k} \]

\[ N = \frac{6k}{30.64k} \]

\[ O = \frac{6k}{30.64k} \]

\[ P = \frac{6k}{30.64k} \]

\[ Q = \frac{6k}{30.64k} \]

\[ R = \frac{6k}{30.64k} \]

\[ S = \frac{6k}{30.64k} \]

\[ T = \frac{6k}{30.64k} \]

\[ U = \frac{6k}{30.64k} \]

\[ V = \frac{6k}{30.64k} \]

\[ W = \frac{6k}{30.64k} \]

\[ X = \frac{6k}{30.64k} \]

\[ Y = \frac{6k}{30.64k} \]

\[ Z = \frac{6k}{30.64k} \]

\[ P_{cr} = \frac{P_{cr}}{P_{cr}} \]

\[ I_{req} = \frac{14.3 \text{ in}^4}{3 \times 173.85 \text{k}} \]

\[ I_{req} = \frac{37.1 \text{ in}^4}{3 \times 301.94 \text{k}} \]
TRUSSES RUNNING ARE CONNECTED BY THIS CREATES A BRIDGE. THE TRUSSES CONSIST OF STEEL.

Distributed load for 18' span

3.66k 3.66k 3.66k 3.66k

6k 6k 6k 6k

Dead load distributed evenly, live load on half.
Precast Concrete Deck (Local Bending)

Load table for deck:

<table>
<thead>
<tr>
<th>Compressive Forces</th>
<th>Tensile Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A = \frac{F}{2}$</td>
<td>$A = \frac{F}{2}$</td>
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<tr>
<td>$A = \frac{20}{2} = 10$</td>
<td>$A = \frac{20}{2} = 10$</td>
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<td>$A = 10$</td>
<td>$A = 10$</td>
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</tr>
</tbody>
</table>

Compressive Flange: $20$ cm$^2$

Tension Flange: $20$ cm$^2$

20 cm deep I-Beams

Network Node

5 cm Thick Reinforced Concrete Deck

25.4 cm Diameter 22 mm Thick

Double-Pane Low-E Glass

60 cm$^2$ Hollow Steel

Hollow Steel Arch

Insulated Panels

Arch Tension Tie

Reaction

Arch Self (17.6 kN)

Precast Concrete Deck (Local Bending)

Under asymmetrical loading conditions

Member sizing based on sectional moment of section for the Bottom Flange

Desired Outer radius: $23.2$ cm

Total Flange loads: $277480$ kN + $13988$ kN

Force along each point: $409.5$ kN (7.15 kN/m)

Max Moment: $399$ kN-m

30.5 m $2.24$ cm $103421$ kN/m$^2$

Neutral Axis Finding

Max Deflection: $399$ kN m

Reaction

Roof (32.6 kN)

Lattice Members (Bending)

Load table for lattice:

Cross sectional area of tension and compression flanges

Cross-sectional area of tension and compression flanges

Moment of Inertia

$I = \sum A \times d^2$

Load in tributary area Compression/Tension

Concrete Thickness

(derived in Multiframe and compared with hand calculations - not shown)

Global moment under full asymmetrical loading

$T = C = M$

Local Moment

$\sum I = \frac{2}{3} (\pi r^4) + \frac{4}{3} (\pi (r^3 - r^3))$

Neutral Axis

Max Deflection: $59$ mm

$\Delta = 0.011 m = 11 mm$

Cross sectional Area

$A = \frac{F}{S} = \frac{203468 \times 10^3}{76209.15} = 26.6 \text{ cm}^2$

$A = \frac{F}{S} = \frac{203468 \times 10^3}{76209.15} = 26.6 \text{ cm}^2$

$0.066 m^2 = 60 \text{ cm}^2$

Arch Tension Tie

Horizontal component of force $= \frac{203468}{26.6} = 76209.15$

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BARRY BEAGEN + TRYGVE WASTVEDT
TRIBUTARY AREA
THE HANDSHAKE
10'

---

**SIZING MEMBERS**

**ARCH AND CABLE**

- **SECONDARY**
  - A E F G H I J K L M

- **ARCH AND FIN SYSTEM**
  - 63.42 kips
  - 25.67 kips

- **ARCH 1**
  - Point load = 5ft x 1.6 kips/ft / 2 = 4 kips
  - 26.6 kips

- **ARCH 2**
  - 87.0 kips
  - 387 kN
  - 80.9 kips
  - 20.1 kips

- **REACTION ON BUILDING 36**
  - 56.9 kips

- **REACTION ON BUILDING 46**
  - 60.8 kips

- **CABLE 1**
  - 87.0 kips
  - 387 kN

- **CABLE 2**
  - 21.2 kips

- **FRAME 1**
  - 6.0 x 0.312
  - DL + Snow Load = 0.4 kips/ft
  - DL + Live Load

- **FRAME 2**
  - 37.5 kips

---

**DETAIL A**

- **Total Load** = 1.6kips/ft
- **Total Weight of Glass** = 0.325 kips/ft
- **Total Weight of Steel** = 0.12 kips/ft

- **DL** = .445 kips/ft = 0.5 kips/ft
- **LL** = 800lb/ft = 0.8 kips/ft

- **Width of Deck** = 10ft

- **DETAIL B**
  - **MAXIMUM DEFLECTION**
    - Column Side = -0.86 in
    - Cable Side = -3.3 in

---

**DEFLECTION ANALYSIS**

- **MAXIMUM DEFLECTION**
  - Column Side = -0.86 in
  - Cable Side = -3.3 in

---

**LOADS**

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- **LL** = 800lb/ft = 0.8 kips/ft

- **Width of Deck** = 10ft

- **Primary Members**
  - **Primary Comp. Member (C1)**
  - **Primary Tension Member (T1)**
  - **Compression Strut (C2)**

- **Secondary Members**
  - **Tension Cable (T2)**

---

**SIZING MEMBERS**

- **TOTAL LOAD**
  - **DL + Snow Load** = 0.4 kips/ft
  - **DL + Live Load**

- **WIDTH OF DECK** = 10ft

---

**DEFLECTION ANALYSIS**

- **MAXIMUM DEFLECTION**
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**LOADS**

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Iguana Bridge

Frame development

effort determined by need to reconcile center of gravity in relation to the possible torsion of the connection between the arch and the deck

Calculating Dead load

Frame Load
Steel = 2.6 lbs/in^3
Dead Load steel = 355.28 lbs 257.96N

Assuming 13 identical panels @ 5.6 ft^2

Wall and Roof Load
Glass = 2.5 lbs/in^2
Dead Load glass = 20335 lbs 45.36N

Floor Load
Concrete = 152.6 lbs/in^2
Dead Load Concrete = 66167 lbs 294.36N

Calculating Point Loads
Floor load Concrete: 66167 lbs @ (8 ft) = 75 psf
Steel: 2 x 3.5 = 60 kips

Calculating Point Loads
Floor Dead load: 66167 lbs/80 ft = 825 psf
Steel: 2 x 3.5 = 60 kips

Wall Dead load: 20335 lbs/10 ft = 2034 psf
Steel: 2 x 3.5 = 60 kips

Steel Dead load: 20335 lbs/10 ft = 2034 psf
Steel: 2 x 3.5 = 60 kips

Concrete: 66167 lbs/10 ft = 6617 psf
Concrete: 2 x 3.5 = 60 kips

Line load Roof = 20 kips
Line load Floor: 80 kips
Total Load = 20,000 lbs
Total 20 kips/13 = 1.5 kips
Frame development

offset determined by need to reconcile center of gravity in relation to the possible torsion of the connection between the arch and the deck

\[ F_{\text{max}} = 165 \text{ kips} \]

\[ o = \frac{F}{A} \]

\[ o = 15 \text{ ksi} \]

\[ F = 340 \text{ kips} \]

\[ o = \frac{165 \text{ kips}}{A} \]

\[ A = \frac{165 \text{ kips}}{15 \text{ ksi}} \]

\[ A = 11 \text{ in}^2 \]

Rectangular section with \( A > 22.7 \):

- 14" x 12" x 1/2"

\[ I_x = 678 \text{ in}^4 \]

\[ I_y = 536 \text{ in}^4 \]

\[ S_x = 96.9 \]

\[ S_y = 89.3 \]

Rectangular Arch Beam sized as follows

Checking for Buckling

since the arch is not being completely stabilized by the connection to the deck it is necessary for us to check for buckling at two areas. The most conservative span is halfway along the arch above the deck. Because the force is highest on the left side of the arch we assume that buckling will occur over the first half. A less conservative estimate check:

\[ P_{cr} = \frac{\pi^2 (E I)}{k L^2} \]

\[ E = 29000 \text{ ksi} \]

\[ I = I_x = 678 \text{ in}^4 \]

\[ L = 75' \]

\[ P_{cr} = \frac{\pi^2 (29000 \text{ ksi} \times 678 \text{ in}^4)}{75' (12 \text{ inch/ft})^2} \]

\[ P_{cr} = 239.6 \text{ kips} \]

Less conservative estimate check:

working backward we get the actual buckling length

\[ L = \sqrt{\frac{\pi^2 (E I)}{P_{cr} \times 3}} \]

\[ L = 436 \text{ in} \text{ or about } 36 \text{ ft} \]

Calculating Dead load

9'

15'

8x10.5

assuming 11 identical panels @ 5.1 ft^2

Panel size:

- 2 @ 110 ft x 8 ft x 0.25 in
- 1 @ 110 ft x 10 ft x 0.25 in

Floor size:

110 ft x 8 ft x 0.5 ft

- .1'

\[ \text{Dead Load steel} = 35519 \text{ lbs} \]

\[ \text{Dead Load glass} = 10335 \text{ lbs} \]

Frame Load

2k

2k

2k

2k

2k

2k

2k

4k

load greater on arch because deck not supported on the left side

10k

Bending and asymmetrical load

By finding the maximum bending moment we can increase the size of the arch in order to stiffen it for asymmetrical loading

\[ M_{\text{max}} = \frac{w L^2}{32} \]

\[ w = \text{live load (estimate to be about } 110 \text{ psf or } 1.1 \text{ ksf}) \]

\[ M_{\text{max}} = \frac{1.1 \text{ ksf} \times (90 \text{ ft})^2}{32} \]

\[ M_{\text{max}} = 278 \text{ kf} \]

\[ o = \frac{M_{\text{max}}}{S_x} \]

\[ o = 15 \text{ ksi} \]

\[ S_x = \frac{M_{\text{max}}}{o} \]

\[ S_x = \frac{278 \text{ kf} \times (12 \text{ in/ft})}{15 \text{ ksi}} \]

\[ S_x = 222.4 \text{ in}^3 \]

The rectangular section that has a section modulus of at least 223 and keeps the width and thickness of the current beam is 12" x 20" x 1/2"

section that has a section modulus of at least 223 and keeps the width and current beam is 12" x 12" x 1/2"

\[ S = \frac{BH^2}{6} - \frac{bh^3}{6} \]

\[ S = \frac{12 \text{ in} \times 24 \text{ in}^2}{6} - \frac{11 \text{ in} \times 23 \text{ in}^3}{24 \text{ in} \times 6} \]

\[ S = 222.5 \text{ in}^3 \]

The beam section is large enough because the two Section modulus values are equal

Bueno our Arch is properly sized because 1.483” is less than the 3.5” for deformation of this arch

Chris Martin & Robert White
The inspiration for this bridge is a Guy Mordenassus pedestrian bridge in New Haven, CT which utilizes a curved, perforated steel sheet as support for the handrail.

### Calculations

#### Assumptions and Loads
- **Average width = 98 ft**
- **Height of steel = 50 ft**
- **6" steel = 2600 psi, 125 ksi**
- **L = Length = 110 ft**
- **Depth of web = 98 in**
- **W2 = Dead Load over entire beam**
- **W1 = Live Load over ½ beam**

#### Plan
- **∑M = 0**
- **∑F = 0**

#### Reactions
- Assume a live load of 3.85 k distributed over one half of the beam or L/2.
- **Stress = 84/(5/6)(.50 in)(112 in)**
- **V = 84 k**
- **V from shear diagram – average shear over length**
- **Teq = (117.75 in)(.125 in)/(354 in)**
- **h = 112 in**
- **Stress = V/Av = must be under 15 ksi**
- **T = 163 k OK**

#### Additional Calculations
- **Shear Calculation for Webs**
  - Dead Load: 118 k/line ft
  - Live Load: 177 k = 1.5 k/linear ft

#### Deflection Calculation
- **Beam = 6" deep STL box**
- **L = 118 ft**
- **Allowable Stress of steel = 438 ksi**
- **Teq = (720 in)(.125 in) / 354 in**
- **Teq = (length of corrugation)(width) / length of chunk**
- **h = 108 in**
- **Av = (5/6)(teq)(h)**
- **Stress = V/Av = must be under 15 ksi**

#### The Corrugator
- **Cross Section**
- **Area under shear diagram: 1317 k ft**
- **VA = (.56)(118) + (.39)(59) - 39**
- **VB = .575 + 33.04**
- **VB + VA = W1(L) + W2(L/2)**

### Notes
- **Compression = Upper deck**
- **Shear = Force/Area**
- **Stress = 1.80 ksi OK**
- **Stress in the critical value.**
- **Factor is increased from 1/3 the critical value to 2/3**
- **For the purposes of calculating shear only, the continue cross sectional area is used.**
- **Teq = (length of corrugation)(width) / length of chunk**
- **Stress = Force/Area**
- **Stress = 438/47  = 8.1 ksi OK**
- **Teq = (720 in)(.125 in) / 354 in**
- **Teq = (length of corrugation)(width) / length of chunk**

### Diagrams
- **EXPLODED ISOMETRIC 1/16" = 1 ft**
- **IMPROPERLY LOADED**
- **MINIMUM MOMENT**
- **MAXIMUM MOMENT**
- **ASYMOMETRICAL LOADING**

---

**The Cross Section**
**Design Revisions for Future**

- **Top Flange (Steel)**
- **Steel and Insulated Glass Web**
- **Bottom Flange (Steel, Aluminum Cladding)**

**Perforated Web**

- Steel and insulated glass web; steel is perforated
- Depth and frequency of curvature varies along elevation

**Coded Isometric**

- 1/16” = 1’
- 1:192

**Coded Section**

- 1/8” = 1’
- 1:96

**Details**

- Aluminum Framing
- Glass Roof
- Top Flange (Box Girder)
- Interior Roofing
- Corrugated Aluminum Decking
- Tribory Steel Support Ribs
- Bottom Flange (steel)
- Aluminum Cladding and Insulation Cavity

**Design Details**

- Perforated, Corrugated Steel Web
- Insulated Glass Roof Drains to Gutter
- Sloped Rigid Insulation
- Steel Support for Glass Roof (Beyond) @ 48” O.C.
- Insulated Glass Roof Drains to Gutter
- 18” Deep Box Flange with Wall Thickness of 0.13, Variable Width
- Perforated, Corrugated Steel Web
- (5) Layers Insulated Glass
**Dual-Restrained Arch Skybridge**

Sean T. Tang
Andrew Sang

**SECTION A-A**

**SECTION B-B**

**CALCULATIONS**

**Floor Tributary Area**

Tributary Area = 3m x 3m
- Concrete, Glass: 445 kN
- Roof: 365 kN

Total = 810 kN

- Concrete Strength: 3 ksi
- Steel Rebar Strength: 40 ksi

**Floor Uniform Load**

- Load from Floor: 445 kN
- Load from Roof: 365 kN

**Roof Tributary Area**

Tributary Area = 3m x 3m
- Concrete, Glass, Snow: 274 kN
- Roof: 365 kN

Total = 639 kN

- Concrete Strength: 3 ksi
- Steel Rebar Strength: 40 ksi

**Roof Uniform Load**

- Load from Floor: 445 kN
- Load from Roof: 365 kN

**Equal horizontal forces allow forces to negate each other at their connection points**

**Force Polygon**

**Column Sizing**

Load from Roof = 406 kN
Load from Floor = 810 kN

Concrete Strength: 3 ksi

- Top: 254 mm
  - No.6 (US) Bars, As = 1500 mm
  - Steel Ratio = 0.013
- Bottom: 439 mm
  - No.6 (US) Bars, As = 1500 mm
  - Steel Ratio = 0.006

**Floor Beam Sizing**

Mmax = 76 kips * in

- Primary Member Sizing
  - W10X17: S = 16.2 in.
  - E = 29,000 ksi
  - Pcr = 5700 kN >> 406 kN
  - No chance of buckling

- Secondary Member Sizing
  - W10X15: S = 13.8 in.
  - E = 29,000 ksi
  - Section Modulus = 31.6 in

**Roof Beam Sizing**

Mmax = 203 kips * in

- Primary Member Sizing
  - W10X15: S = 13.8 in.
  - E = 29,000 ksi
  - Pcr = 5700 kN >> 406 kN
  - No chance of buckling

- Secondary Member Sizing
  - W10X17: S = 16.2 in.
  - E = 29,000 ksi
  - Section Modulus = 15.5 in

**New Primary Member Sizing**

**No.6 (US) Bars**

Steel Ratio = 0.013
As = 1700 mm

- Assymetric Loading Floor: 13.6 ksi < 15 ksi OK
- Assymetric Loading Roof: 13.8 ksi < 15 ksi OK

- Additional Stress = 4.4 ksi
- Additional Stress = 7.12 ksi

**New Primary Member Sizing**

**Assymetric Loading Floor**

- People Load: 80 psf -> 800 lbs/ft
- Snow Load: 30 psf -> 300 lbs/ft

- Uniform Load Stress: 9.36 ksi

- New Primary Member Sizing

**Assymetric Loading Roof**

- People Load: 80 psf -> 800 lbs/ft
- Snow Load: 30 psf -> 300 lbs/ft

- Uniform Load Stress: 6.52 ksi

- New Primary Member Sizing
virendeel bridge  dicle uzunayla & sayjel patel

CONCEPT

BEAM ANALYSIS

MULTIFRAME ANALYSIS

Fabrication

PARAMETRIC LOGIC
Axial Loads (P) kN

Moment (z) kN*m

10 6 C
15 6 13 6
8 13 7 37 31 21 12 3 7 32 5 40 9 12 8 9 5 20

T 3 8 15 6 13
8 12 9 0

C 48 T 3 8 55 T 58 T
0 C 4 4 31 2 2 58 C
0 0 9 8 5 2 4

84 kip 84 kip

DL + LL = 3.2 kip/ft

Bridge Width
30.00%

Ft 3
Area Required
33,600

117

Box of steel sections serve
4 and ceiling:

Prefabricated vertical elements are
welded steel plates

Red deck are perforated to
the weight of the structure
vary ribs
Vassar Skybridge
Structural Design
[4.463]Building Structural Systems II
Professor: John Ochsendorf Andrea Love
TA: Caitlin Tobin Mueller
Laura Renee Schmitz
Emid Xuezhu Tian

CONCEPT
The Vassar Skybridge will allow for visibility down the street and by its slanted geometry it will allow for people on the bridge to lean over and see the street life below.

ESTIMATED LOADS
Floor Dead Load: 3.11 kN/m² (65 psf)
Floor Live Load: 3.83 kN/m² (80 psf)
Roof Dead Load: 1.44 kN/m² (30 psf)
Roof Live Load: 2.15 kN/m² (45 psf)
Cladding Load: 0.48 kN/m² (10 psf)

DESIGN ASSUMPTIONS
Bridge length: 32.54 m (106.75 ft)
Bridge Width: 3.05 m (10 ft)
Bridge Height: 4.57 m (15 ft)

ALLOWABLE STRESSES (σ)
Steel: 103 N/mm² (15 ksi)
MODULUS OF ELASTICITY (E)
Steel: 29000 ksi

POINT LOADS
Tributary Area - Floor
(65 psf+80 psf)(10 ft)(9.7 ft)=14.065 kips
Tributary Area - Roof
(30 psf+45 psf)(10 ft)(9.7 ft)=10.67 kips
Tributary Area - Cladding
(10 psf)(15 ft)(9.7 ft)(2)=2.91 kips
Vertical Point Loads (P)
27.645 kips =122.97 kN

DISTRIBUTED LOADS
w=(80 psf+45 psf)(10 ft)
=1250 psf=2.1161 mm²

SIZING THE CABLES
Point Loads for Cable (P<sub>n</sub>)
P<sub>n</sub>=P/ cos(20°)=29.419 kips=130.86 kN

Maximum Force in Cable (P<sub>m</sub>)
P<sub>m</sub>=245.7 kips=1092.93 kN

Gross Required Area for 2 Cables (A<sub>g</sub>)
A<sub>g</sub>= P<sub>m</sub>/ σ=245.7 kips/15 ksi=16.4 in²

Required Area for 1 Cable (A)
A= 1/2 A<sub>g</sub>=8.2 in²=5280.31 mm²
A= m²
r=1.41 in=35.81 mm

Required Area for Secondary Cables (A<sub>s</sub>)
A<sub>s</sub>= A/ m²
r<sub>s</sub>=0.79 in= 20.06 mm

MODULUS OF ELASTICITY (E)
Steel: 29000 ksi

DEFLECTION
\[ \Delta_{max} = \frac{5wL^4}{384EI} \]
\[ L=106.75 \text{ ft}/2=53.375 \text{ ft} \]
We calculated for the deflection of half of the bridge for asymmetrical loading
\[ \Delta_{max} \leq \frac{L}{360} = \frac{53.375 \text{ ft}}{360} = 640.5 \text{ in}/360 = 1.779 \text{ in} \]
wLL=(80 psf)(10 ft)+(45psf)(19.5 ft)=1677.5 plf =1.6775 k/ft=0.139 k/in
E=29000 k/in²
I ≥5 wLLL⁴/384E \[ \Delta_{max} = \frac{5(0.139 \text{ k/in})(640.5 \text{ in})^4}{384(29000 \text{ k/in}^2)(1.779 \text{ in})} = 5094.1 \text{ in}^4 \]
\[ I= \sum I_i= \sum A_i d_i \]
A<sub>1</sub>=A<sub>2</sub>=A<sub>3</sub>=A<sub>4</sub>= 5.74 in²
\[ d<sub>1</sub>=d<sub>2</sub>=d<sub>3</sub>=d<sub>4</sub>=1.5 \text{ ft}=18 \text{ in} \]
\[ I=4(5.74 \text{ in}^2)(18 \text{ in})^2=7439.04 \text{ in}^4 \]
7439.04 in⁴> 5094.1 in⁴

Dimensions and Sectional Properties of Round HSS for hollow steel tube, d=6.625 in
\[ \Delta_{max} = \frac{5wL^4}{384EI} \]
\[ L=106.75 \text{ ft}/2=53.375 \text{ ft} \]
We calculated for the deflection of half of the bridge for asymmetrical loading
\[ \Delta_{max} \leq \frac{L}{360} = \frac{53.375 \text{ ft}}{360} = 640.5 \text{ in}/360 = 1.779 \text{ in} \]
wLL=(80 psf)(10 ft)+(45psf)(19.5 ft)=1677.5 plf =1.6775 k/ft=0.139 k/in
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\[ I= \sum I_i= \sum A_i d_i \]
A<sub>1</sub>=A<sub>2</sub>=A<sub>3</sub>=A<sub>4</sub>= 5.74 in²
\[ d<sub>1</sub>=d<sub>2</sub>=d<sub>3</sub>=d<sub>4</sub>=1.5 \text{ ft}=18 \text{ in} \]
\[ I=4(5.74 \text{ in}^2)(18 \text{ in})^2=7439.04 \text{ in}^4 \]
7439.04 in⁴> 5094.1 in⁴

Vassar Skybridge
SIZING THE TRUSSES

Maximum Moment in Truss (\(M_{\text{max}}\))
\[
M_{\text{max}} = \frac{wL^2}{24}
\]
Maximum Tension and Compression Forces in Truss (\(T/C\))
\[
T = C = \frac{M_{\text{max}}}{d}
\]

Gross Required Area for 2 Trusses \(A_g\)
\[
\sigma = \frac{T}{A_g}
\]

Required Area for 1 Truss (\(A\))
\[
A = \frac{1}{2} A_g = 5.74 \text{ in}^2 = 370.32 \text{ mm}^2
\]

Required Area for Secondary Cables (\(A_s\))
\[
A_s = \frac{1}{2} A = 2.87 \text{ in}^2 = 185.16 \text{ mm}^2
\]

Maximum Moment in Truss (\(M_{\text{max}}\))
\[
M_{\text{max}} = \frac{wL^2}{72}
\]

DEFLECTION
\[
\Delta = \frac{5wL^4}{384EI}
\]

Dimensions and Sectional Properties of Round HSS for hollow steel tube,
\(d = 6.625 \text{ in} \)
\(I = 1973 \text{ in}^4 \)

Structural Design
[4.463] Building Structural Systems II
Professor: John Ochsendorf  Andrea Love
TA: Caitlin Tobin Mueller
Laura Renee Schmitz
Enid Xuezhu Tian
Load Calculations:

\[
\omega_{ds} = 957.6 \text{ Pa} \\
\text{Beam Depth:} \quad d = 1.4 \text{ m} \\
\omega_{ll} = 4.8 \times 10^3 \text{ Pa} \\
\text{Beam Length:} \quad L = 34.1 \text{ m} \\
\text{Live Load on Deck:} \quad \text{Factored Load} = 1.2 \times \omega_{ds} \\
\text{Factored Load} = 1.2 \times \omega_{ll} \\
\text{Moment max} = \text{Factored Load} \times \text{Beam Length}^2 \\
\text{Tension max} = \frac{\text{Moment max}}{\text{Beam Depth}^2} \\
\text{Compression max} = \frac{\text{Tension max}}{\text{Beam Depth}^2} \\
\text{Size Tension Members:} \quad \text{Material Strength Reduction} = 0.9 \\
\text{Material Yield Strength} = 3.4 \times 10^8 \text{ Pa} \\
\text{Ag} = \text{Tension max} \times \text{Material Yield Strength} \\
\text{Size Compression Members:} \quad \text{Material Strength Reduction} = 0.9 \\
\text{Effective Length Factor} = 0.7 \\
\text{Member Radius} = 32.4 \text{ cm} \\
\text{Material Young’s Modulus} = 2 \times 10^{11} \text{ Pa} \\
\text{Length of Compression Member} = 5.2 \text{ m} \\
\text{Fe} = \pi E KL^2 r \frac{\text{Ag}}{\text{Compression max}} \\
\text{Fe} = 37.8 \text{ cm}^2 \\
\text{HSS 12.750 x 0.375} \quad \text{gross area} = 87.1 \text{ cm}^2 \\
\text{Pipe Diameter:} \quad 32.4 \text{ cm} \\
\text{Pipe Thickness:} \quad 1 \text{ cm} \\

Vassar Street Skywalk

**Project Background**

- A unique feature of MIT's campus is its system of interconnected buildings that allow students and faculty to travel across campus indoors.
- The Brain and Sciences Complex was completed in 2005, and has no connection to this interconnected network. This requires students and faculty to cross Vassar Street if they wish to go to buildings on the main campus.
- MIT would like to connect the recently built Brain and Cognitive Sciences Building to the Main Campus via a Skywalk over Vassar Street.

**Schematic Plan**

**Load Path**
Vassar Street Skywalk

A system of interconnected buildings which cross campus indoors. Completed in 2005, and has no connection to the Brain and Cognitive Sciences Building to cross Vassar Street.

**Calculations**

### Load Calculations:
- Superimposed Dead Load on Deck: 
  \[ \omega_{sd} = 977.6 \text{ Pa} \]
- Live Load on Deck: 
  \[ \omega_{ll} = 4.8 \times 10^5 \text{ Pa} \]
- Beam Depth: 
  \[ d = 1.4 \text{ m} \]
- Deck Width: 
  \[ w = 3 \text{ m} \]
- Beam Length: 
  \[ L = 34.1 \text{ m} \]

### Factored Load on Deck:
**Factored Load**

\[ 1.2 \cdot \omega_{ds} + 1.6 \cdot \omega_{ll} + 2.7 \times 10^6 \text{ N} \]

**Moment**

\[ \frac{M_{max}}{d} = 2.7 \times 10^6 \text{ N} \]

**Tension**

\[ T_{max} = 8.8 \times 10^6 \times 1.4 \times 10^5 \text{ N} \]

**Compression**

\[ C_{max} = 2.7 \times 10^6 \times 1.4 \times 10^5 \text{ N} \]

### Size Tension Member:
- Material Strength Reduction: 
  \[ \phi_t = 0.9 \]
- Material Yield Strength: 
  \[ F_y = 3.4 \times 10^8 \text{ Pa} \]

\[ A_g = \frac{80.9 \text{ cm}^2}{\phi_t F_y} \]

**HSS 12.750 x 0.375 gross area = 87.1 cm²**

### Size Compression Member:
- Material Strength Reduction: 
  \[ \phi_c = 0.7 \]
- Member Radius: 
  \[ r = 32.4 \text{ cm} \]
- Length of Compression Member: 
  \[ L_c = 5.2 \text{ m} \]

\[ A_g = \frac{37.8 \text{ cm}²}{E} \]

**HSS 12.750 x 0.375 gross area = 87.1 cm²**
BIRDS PORTCHOUTH RUSSUM
Plashet School Footbridge

SCHLAICH BERGERMANN UND PARTNER
Ripshorst Bridge

CONNECTION
DETAIL PRECEDENTS

ELEVATION 1:100

PLAN 1:200

SECTION 1:200

1.52 m typ.
6.2 m
33 m
6.2 m
1.52 m typ.
**Loads**

<table>
<thead>
<tr>
<th>Roof</th>
<th>Dead Load 2,179 kN/m² + Live Load 0,958 kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cladding</td>
<td>Dead Load 1,436 kN/m²</td>
</tr>
<tr>
<td>Floor</td>
<td>Dead Load 3,787 kN/m² + Live Load 3,830 kN/m²</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>12,190 kN/m² → 41,532 kN/m</strong></td>
</tr>
</tbody>
</table>

**Structure Precedents**

**Connection Details**

**Schlaich Bergermann und Partner**

**Ripshorst Bridge**

**Birds Portmouth Russum**

**Plashet School Footbridge**

**Elevation 1:100**

**Plan 1:200**

**Section 1:200**

**Loads**

**Primary Structure Sizing**

**Axial**

\[ A_{\text{req}} = \frac{\sigma_{\text{allow}}}{F_{\text{max}}} = 18000 \text{ mm}^2 \]

**Buckling**

\[ P_{cr} = \frac{\pi^2 (EI)}{(KL)^2} = 153098 \text{ kN} \]

\[ M_{\text{max}} = \frac{w(L/4)^2}{8} = 321 \text{ kNm} \rightarrow S = \frac{M_{\text{max}}}{\sigma_{\text{allow}}} = 3027600 \text{ mm}^3 \]

**Stiffness**

\[ P_{cr} > 3F_{\text{max}} \checkmark \]

**Secondary Structure Sizing**

**Bending + Axial**

\[ \sigma_{\text{allow}} = \frac{M_{\text{max}}}{S} + \frac{F_{\text{max}}}{A} \]

\[ F_{\text{max}} = 32 \text{ kN} \]

\[ P_{cr} > 3F_{\text{max}} \checkmark \]

**Multiframe Model Assumptions → 2D**

Primary  
Secondary  
Deck

2D assumptions based on equal areas

1 tube  
1 bar

**Deflection**

**Result**

\[ \Delta_{\text{max}} < \Delta_{\text{allow}} \rightarrow \text{Arch stiff enough} \checkmark \]
“The experience of having Jörg Schlaich in our class was particularly great: not only was he able to pinpoint the flaws in the structural logic, but he also gave very good advice in the program and aesthetics of the design, which I appreciated very much as an architect. It was really great to learn from him; thanks very much!”

– Shiyu Wei, Master of Architecture Student