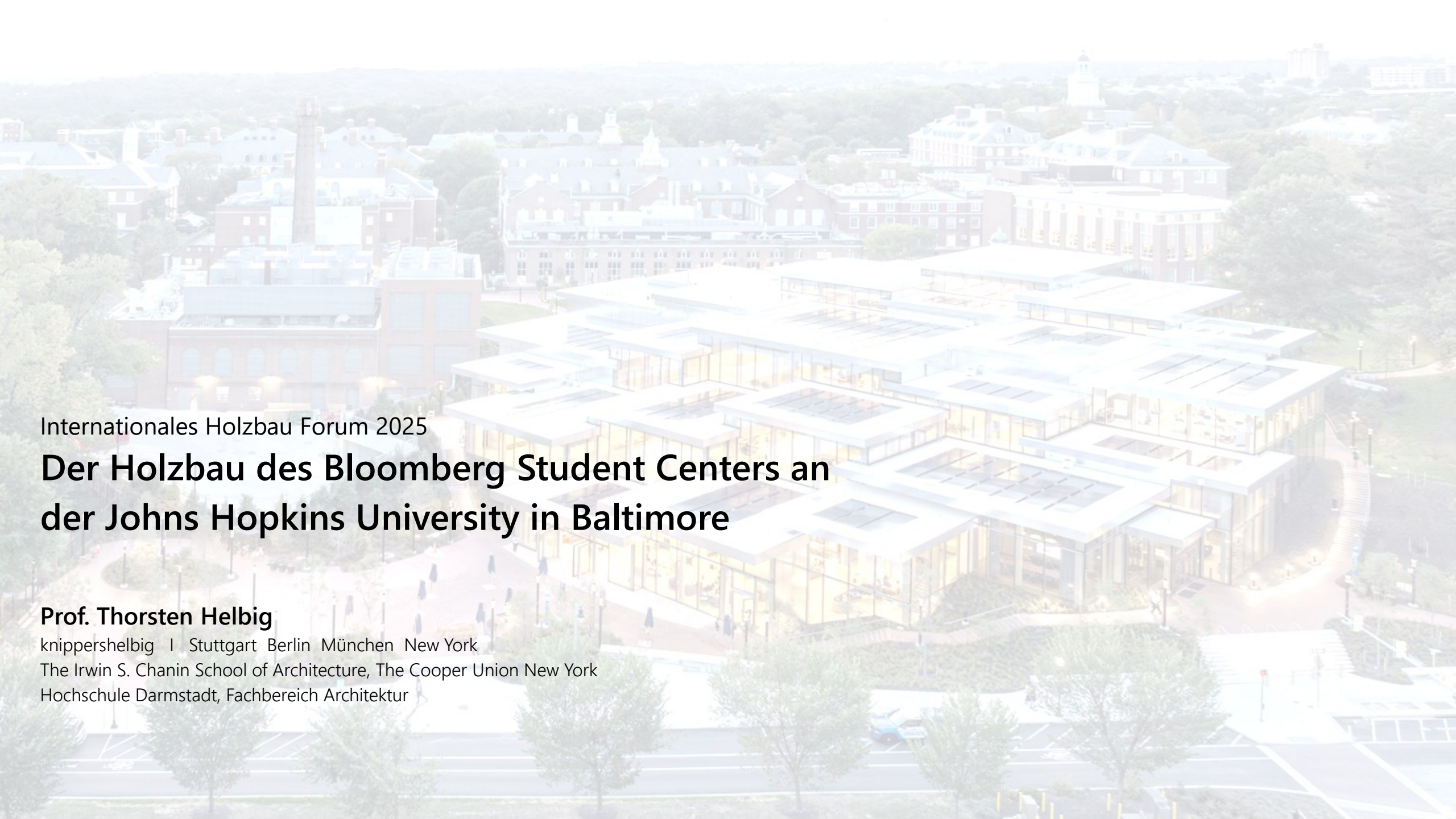


Internationales Holzbau Forum 2025

Der Holzbau des Bloomberg Student Centers an der Johns Hopkins University in Baltimore

Prof. Thorsten Helbig
knippershelbig I Stuttgart Berlin München New York
The Irwin S. Chanin School of Architecture, The Cooper Union New York
Hochschule Darmstadt, Fachbereich Architektur

Peek & Cloppenburg Kaufhaus, Köln
Renzo Piano Building Workshop

Holzparkhaus Wendlingen
Hermann+bosch architekten

The Cradle, Düsseldorf
HPP Architekten

Jüberg-Turm, Hemer
Birk, Heilmeyer und Frenzel

Alnatura, Campus 360, Darmstadt
haascookzemmrich

Betriebsstätte Hald & Grunewald, Rottenburg
rundzwei Architekten

Studierendenhaus Braunschweig
Gustav Düsing, Max Hacke

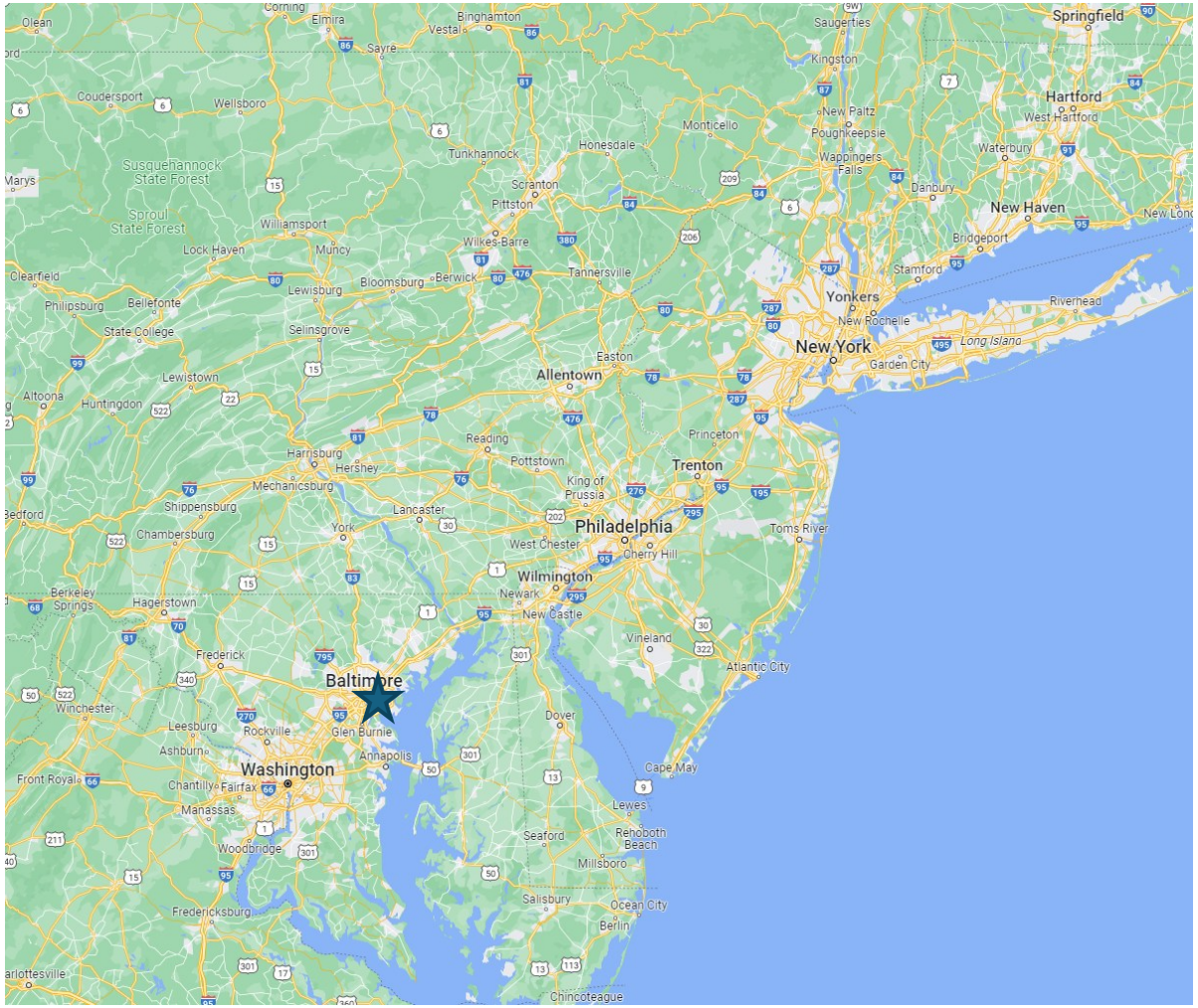
Remstalbrücken
Cheret Bozic Architekten

REWE Wiesbaden
ACME

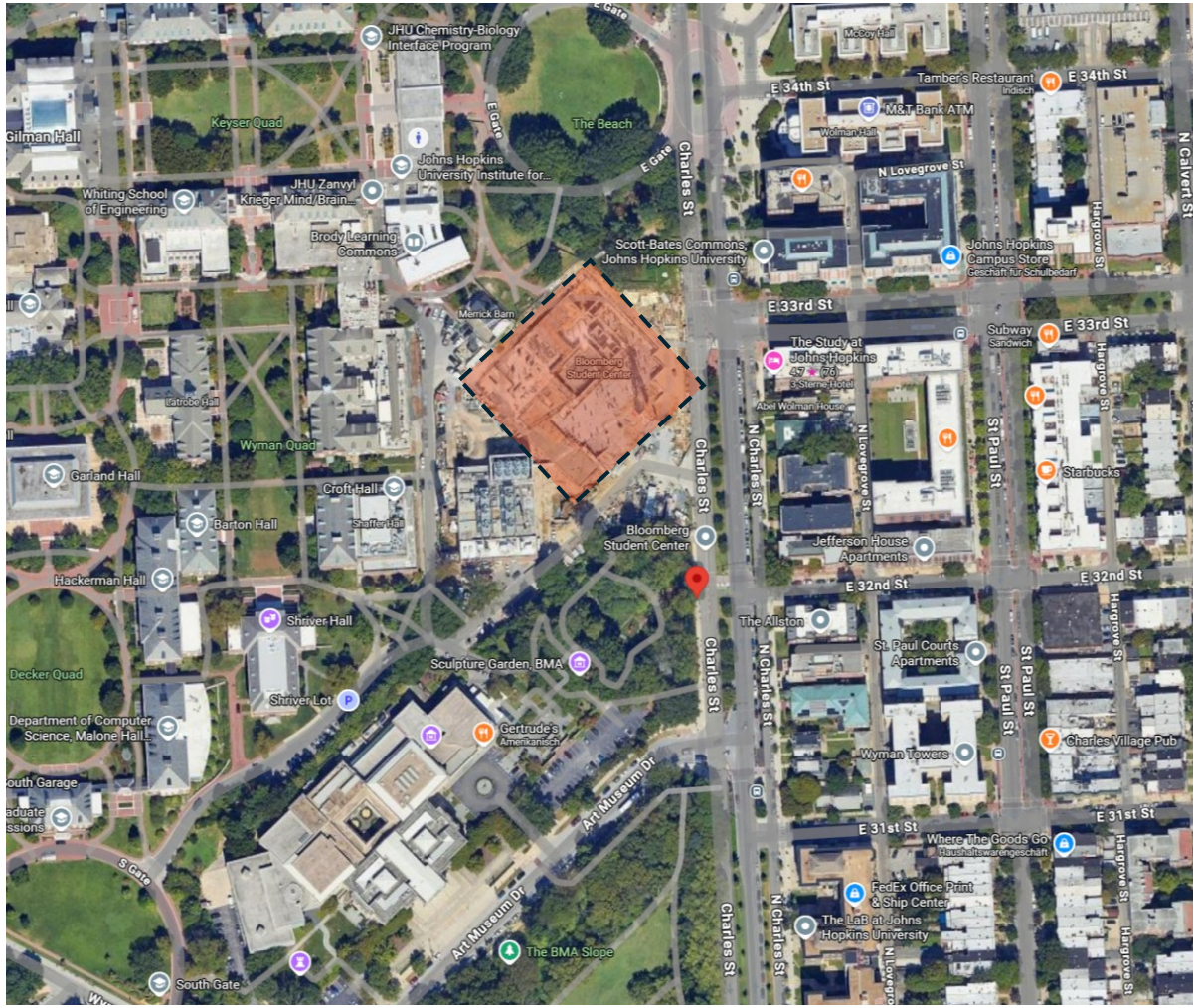
Innovationsfabrik 2.0, Heilbronn
Wächter + Wächter Architekten

U5-Haltestelle Arenen Volkspark, Hamburg
BLRM

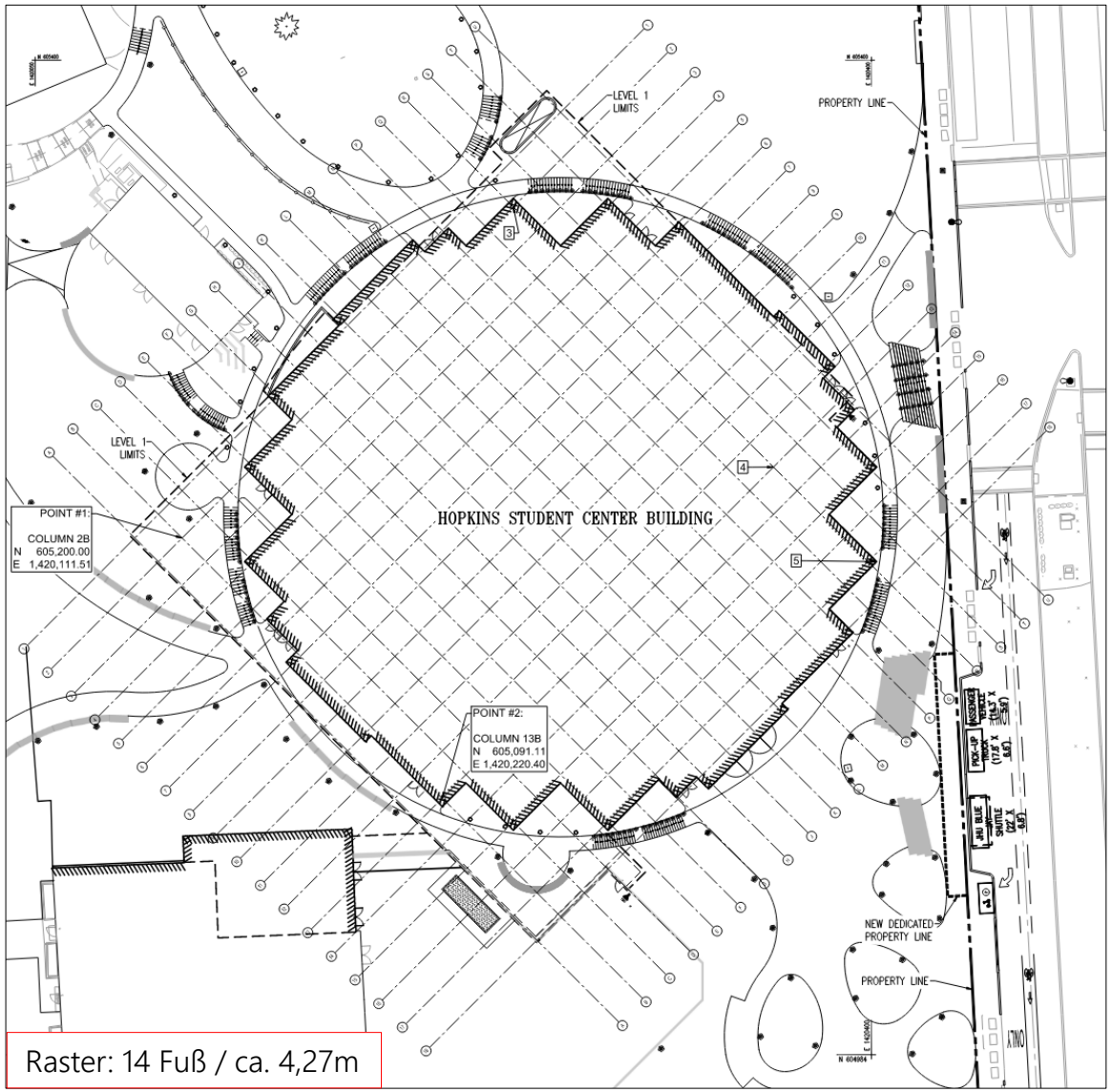
Burj Zanzibar, Tansania
OMT / Birk Heilmeyer und Frenzel



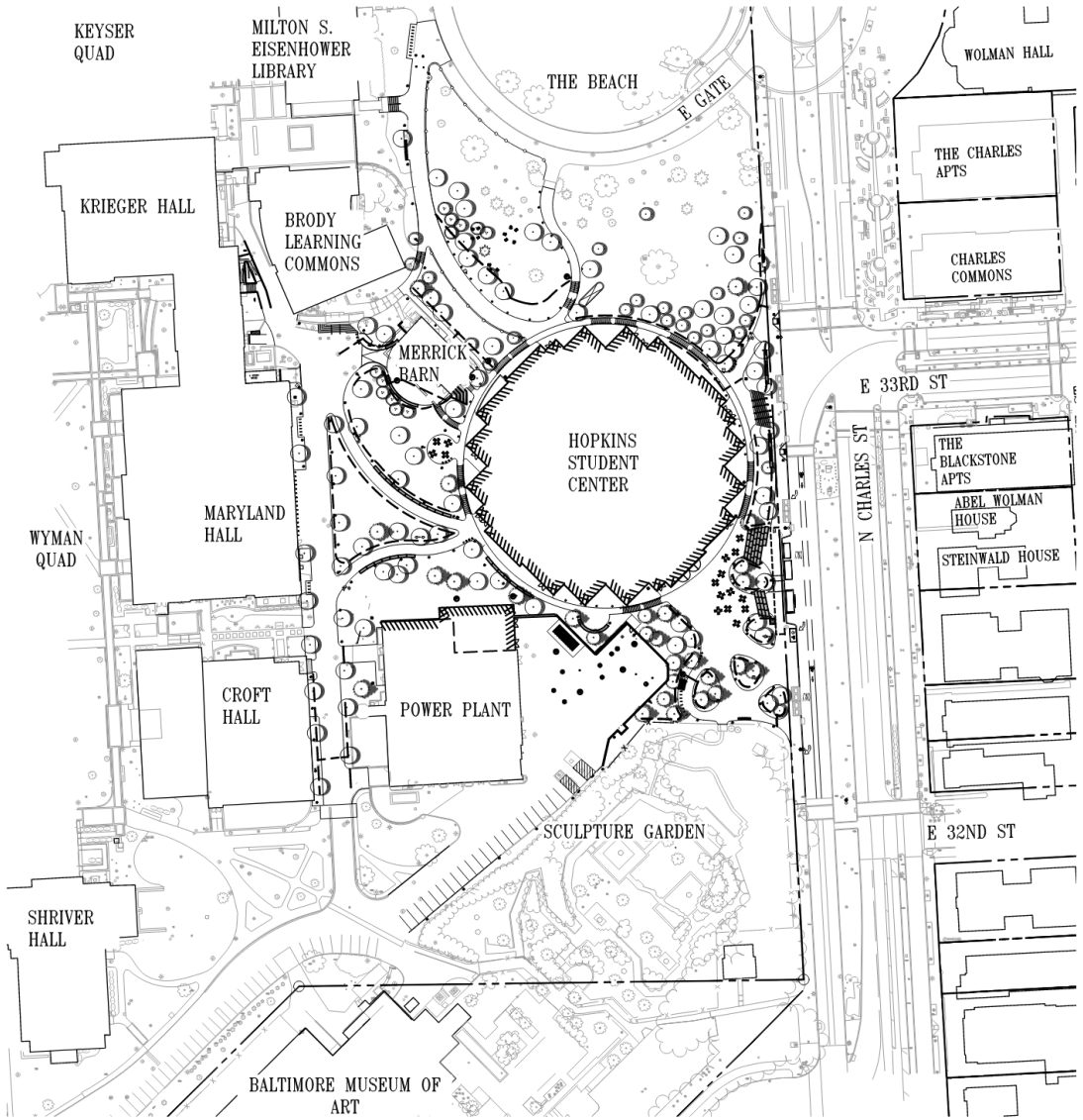
3200 North Charles Street, Baltimore, Maryland

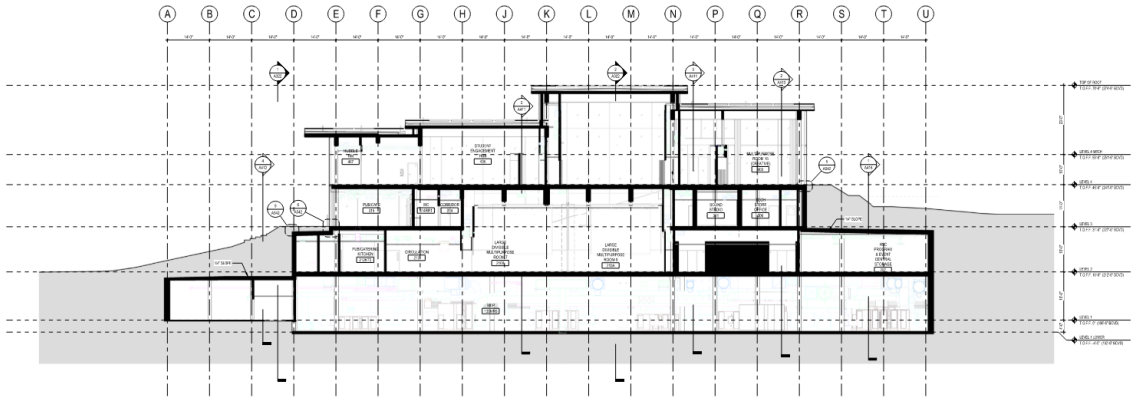


Luftbild (Google Maps)

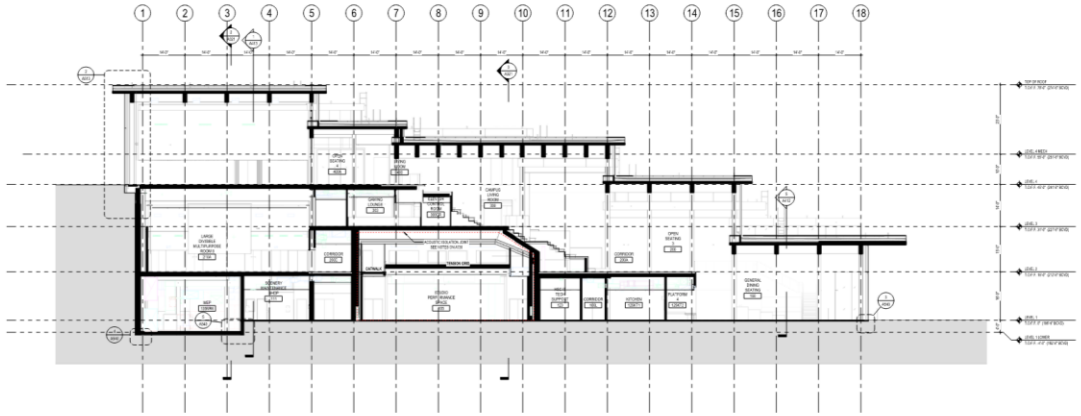


Raster: 14 Fuß / ca. 4,27m

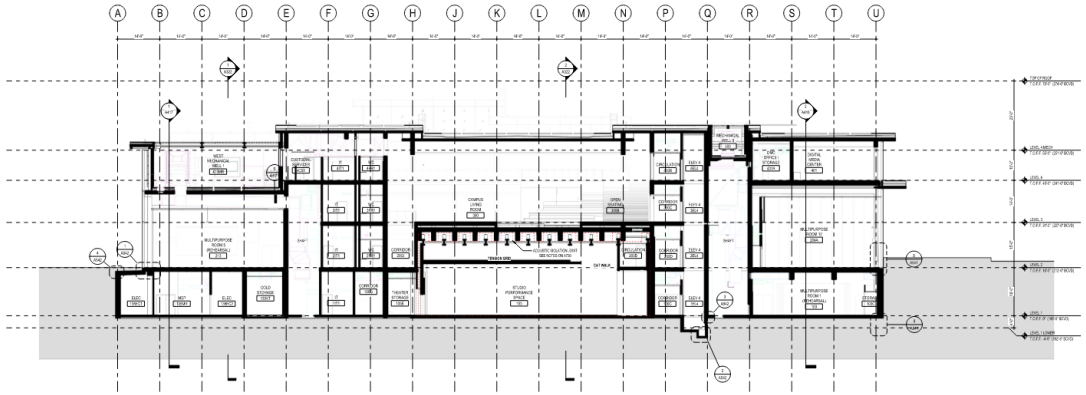




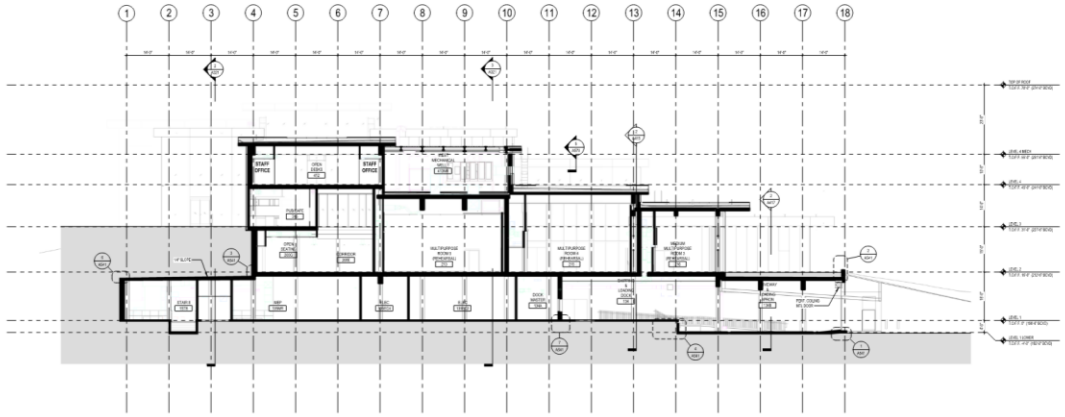
2 E/W SECTION B
SCALE 3/32" = 1'-0"



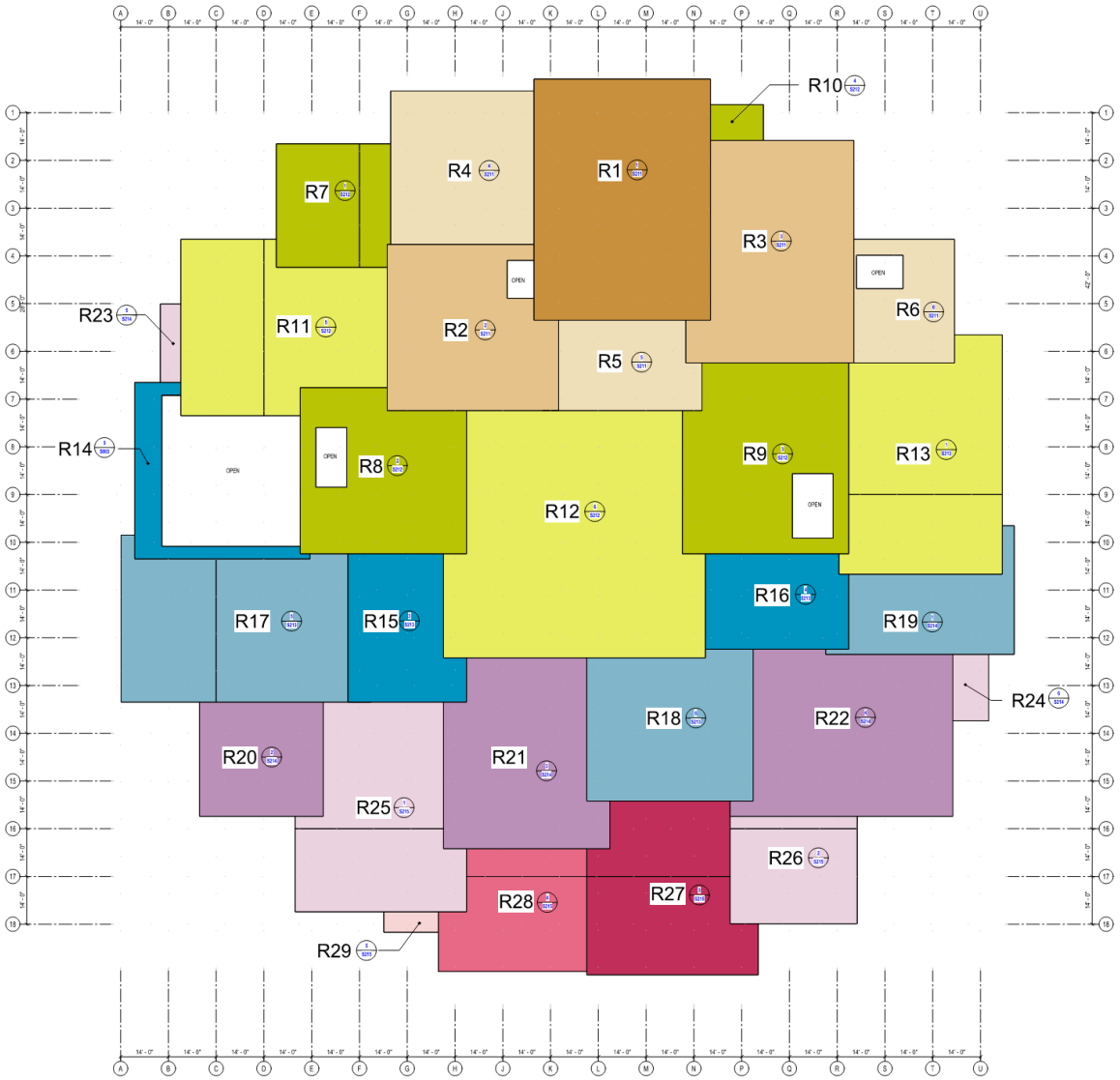
2 N/S SECTION B
SCALE 3/32" = 1'-0"



3 E/W SECTION A
SCALE 3/32" = 1'-0"



1 N/S SECTION A
SCALE 3/32" = 1'-0"



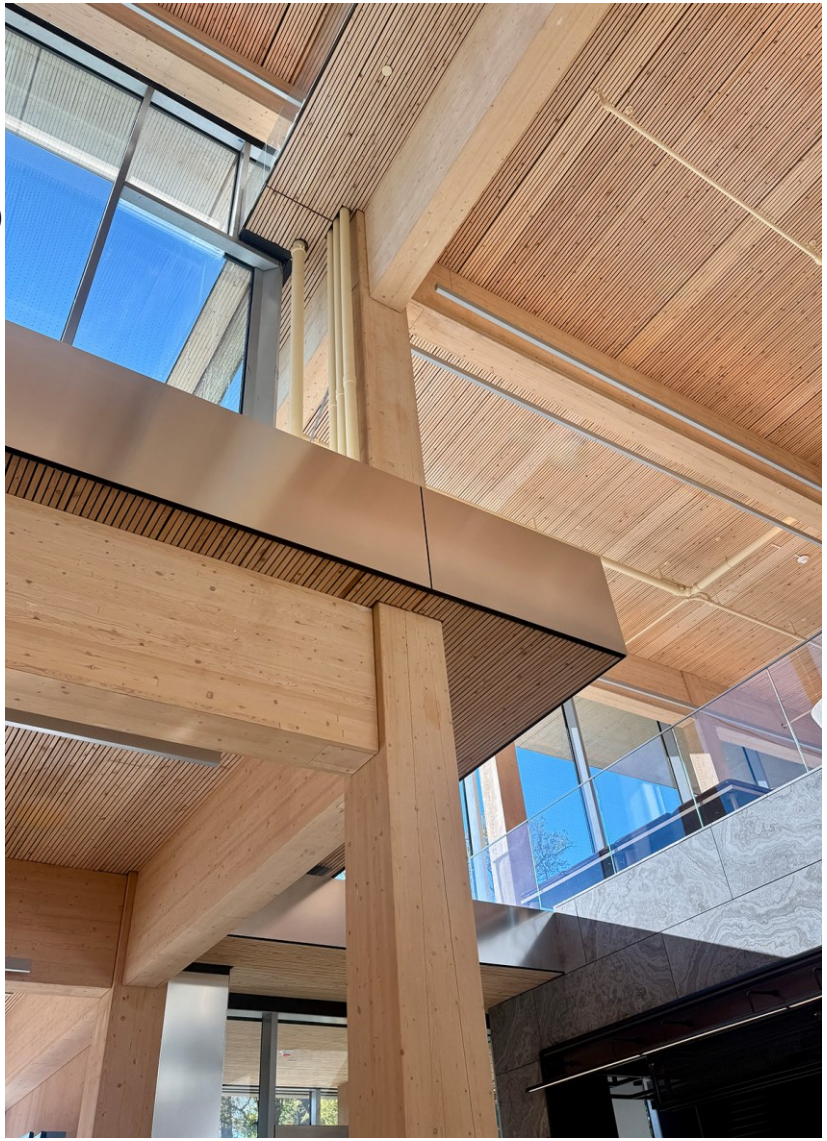
Dübelholz(DLT)
Innenseite: Akustik (NRC 0,7)

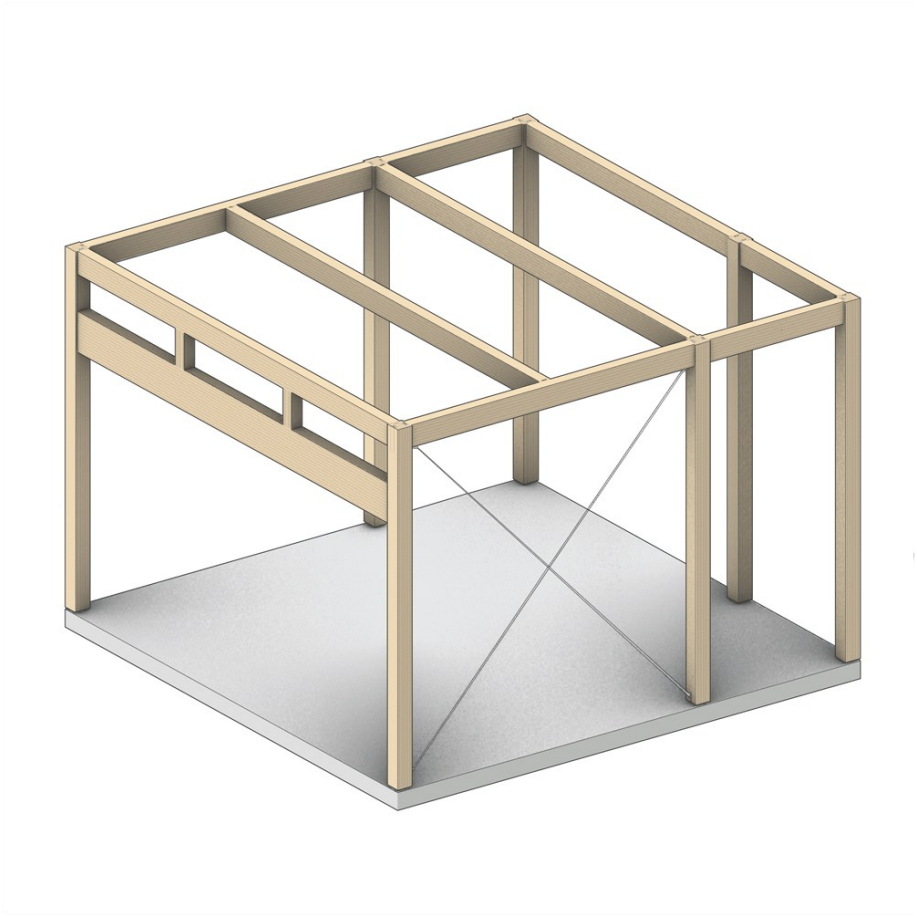
Sperrholzplatten
auf der Oberseite
(Dachscheibe)

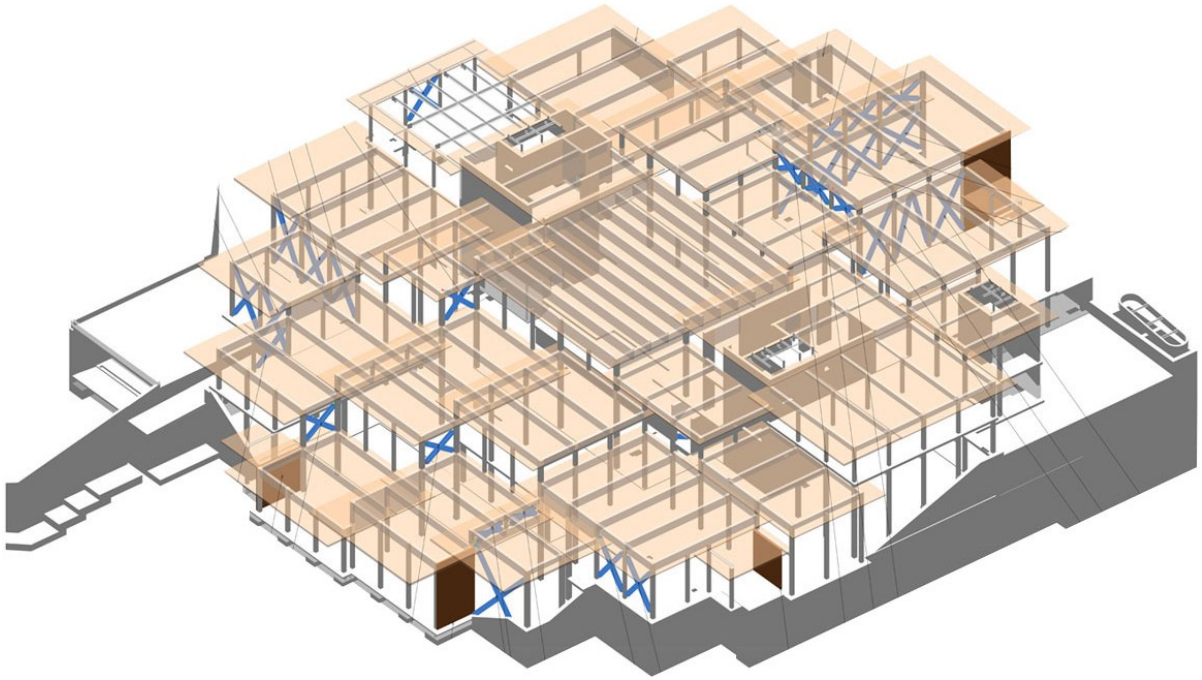
BSH Träger
30cm / 40 ... 137cm
Spannweite: 8,4 ... 21m

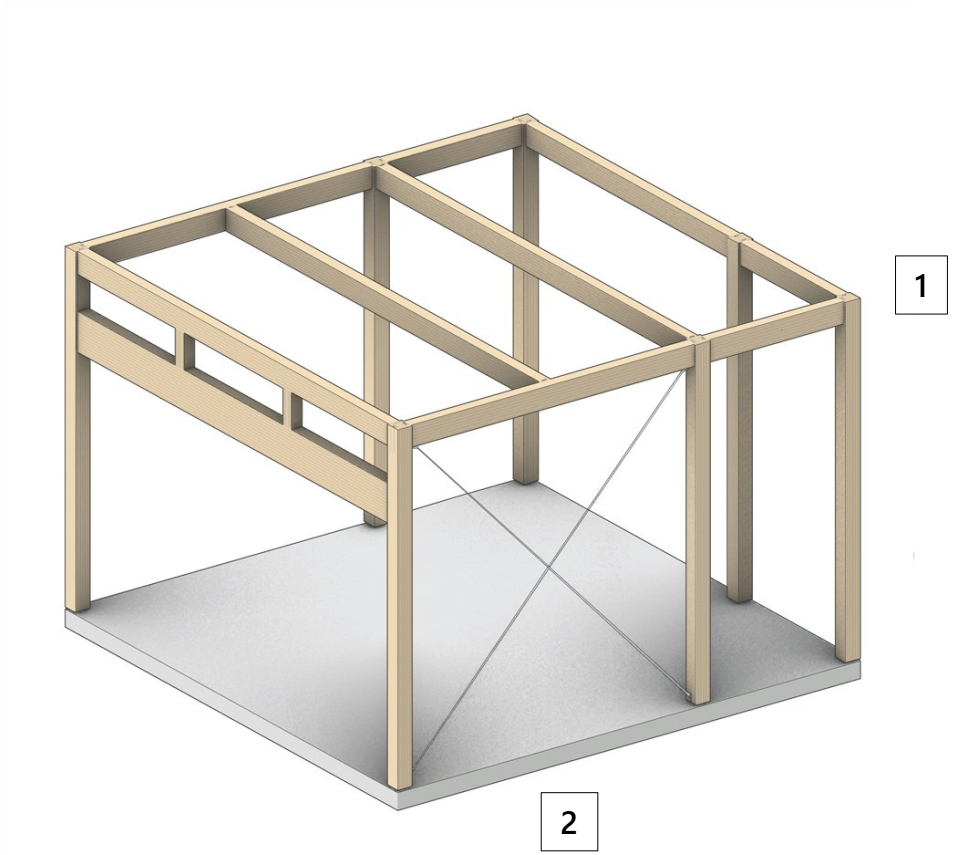
BSH Stützen
30x30 ... 45x45 cm

Kabeltrassen
mit DLT Abdeckpaneelen



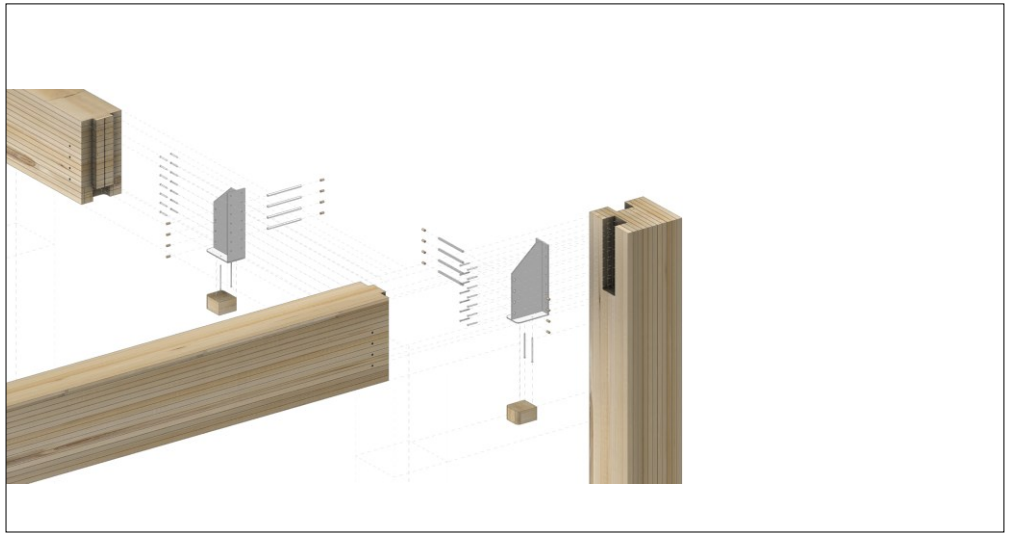




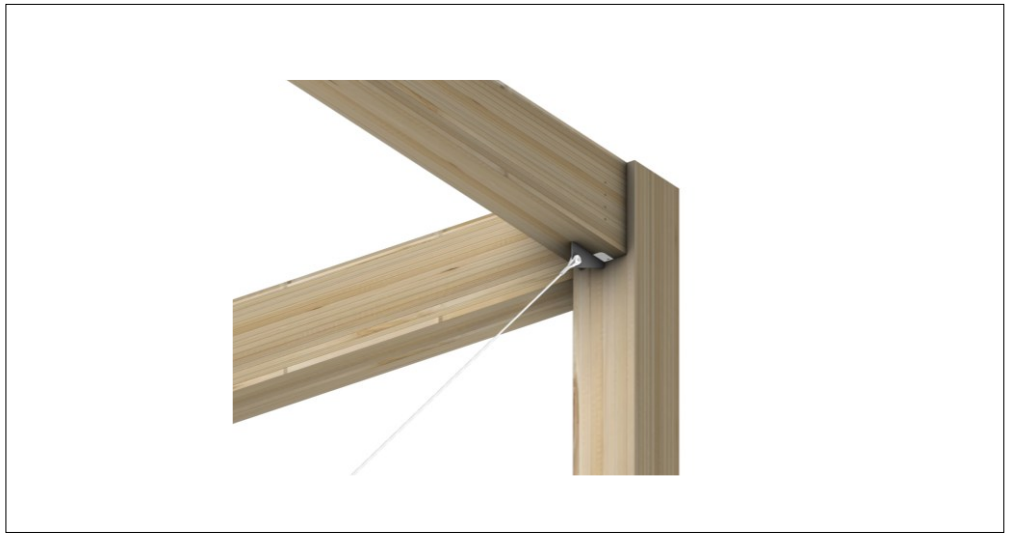


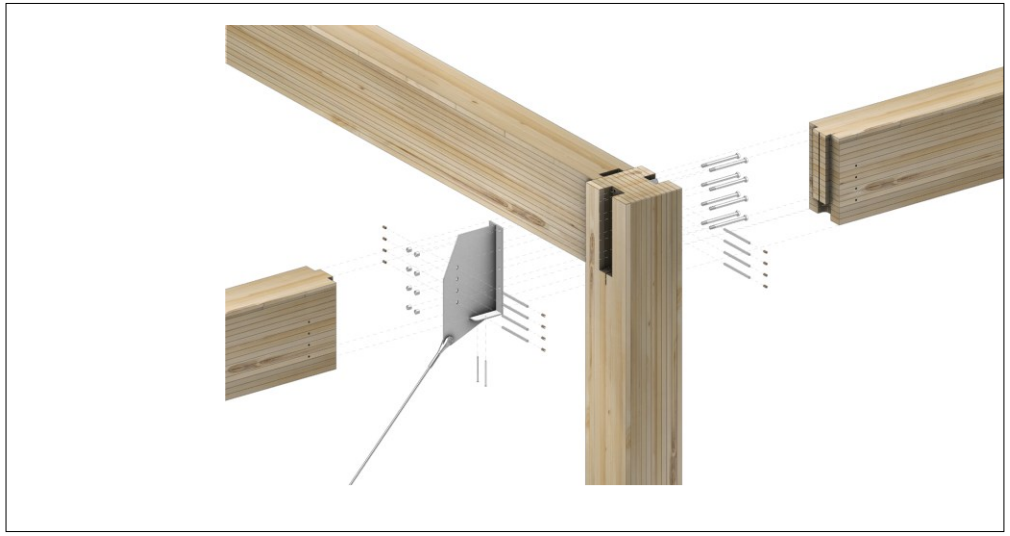
1

2

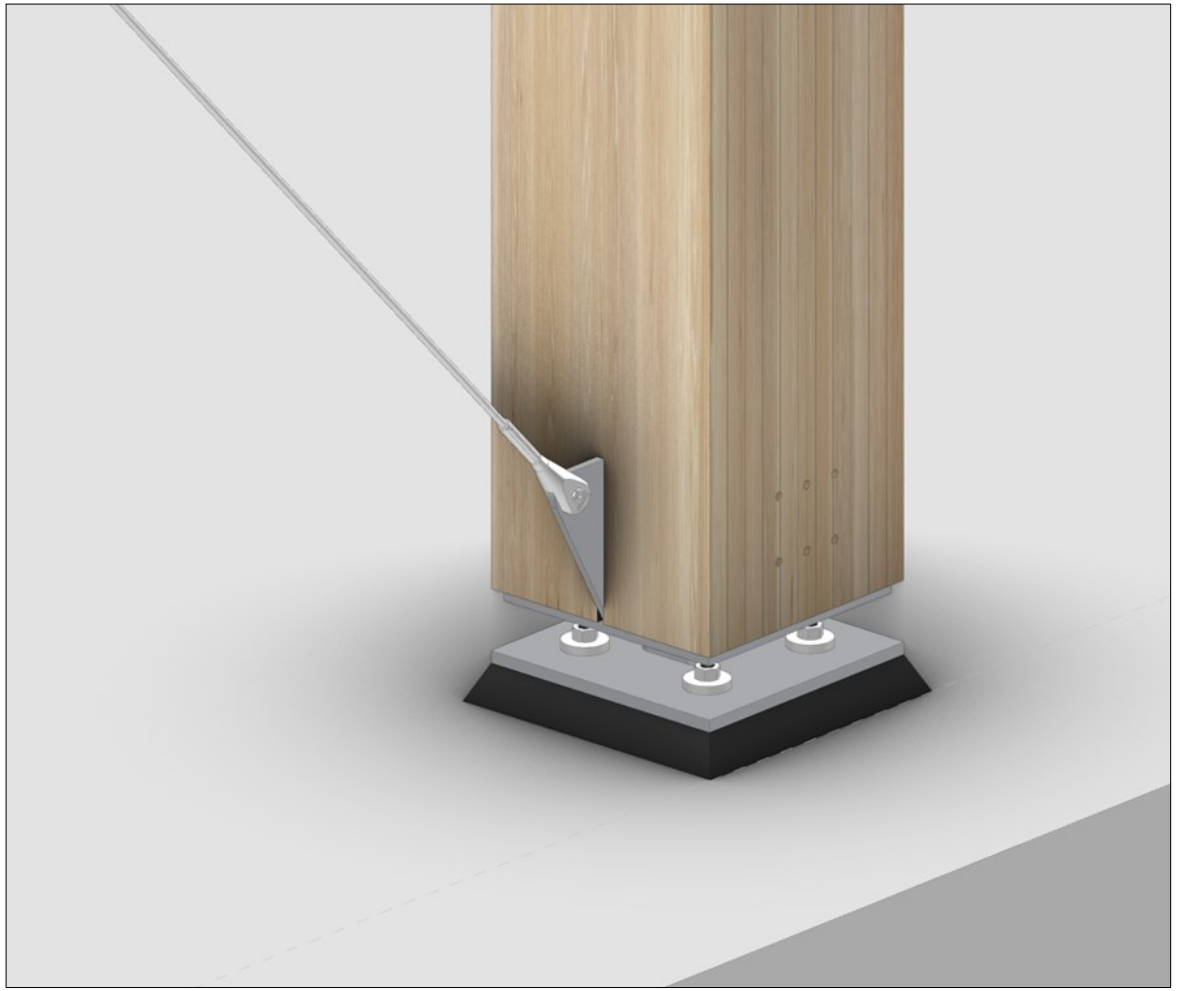


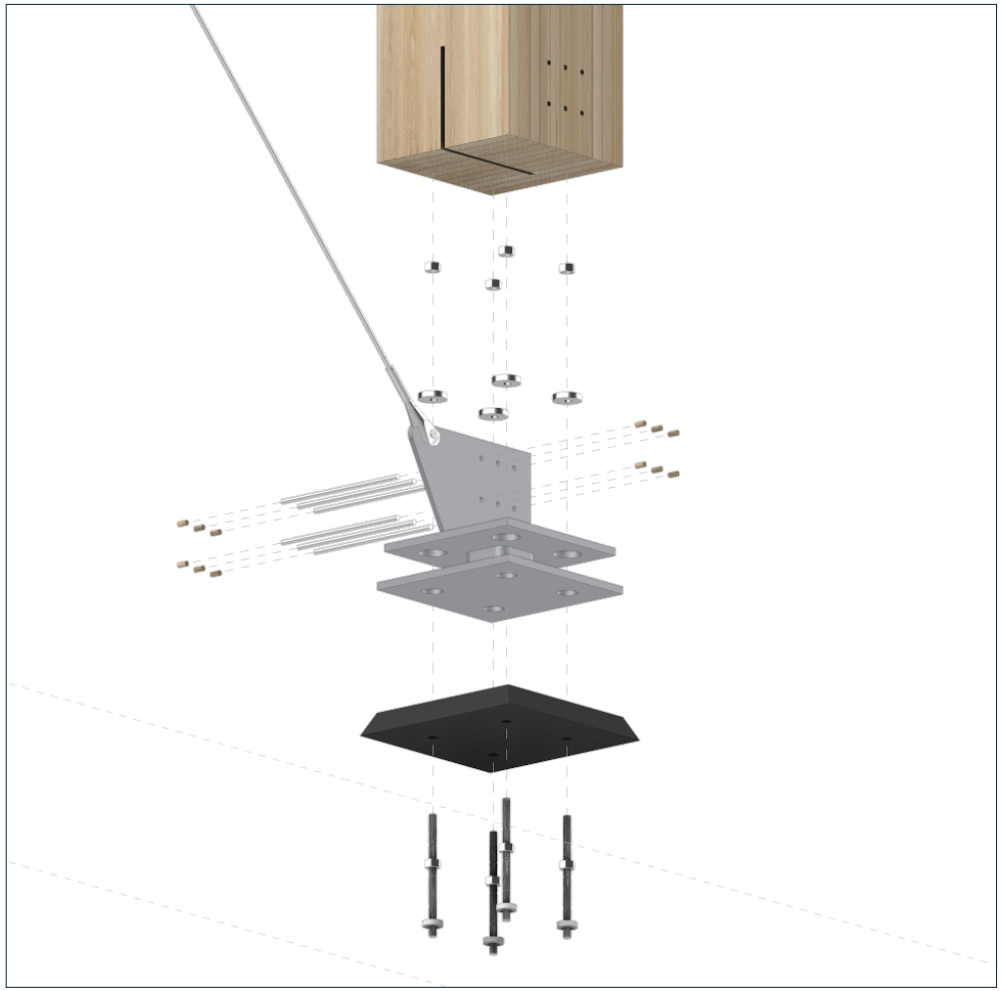
1

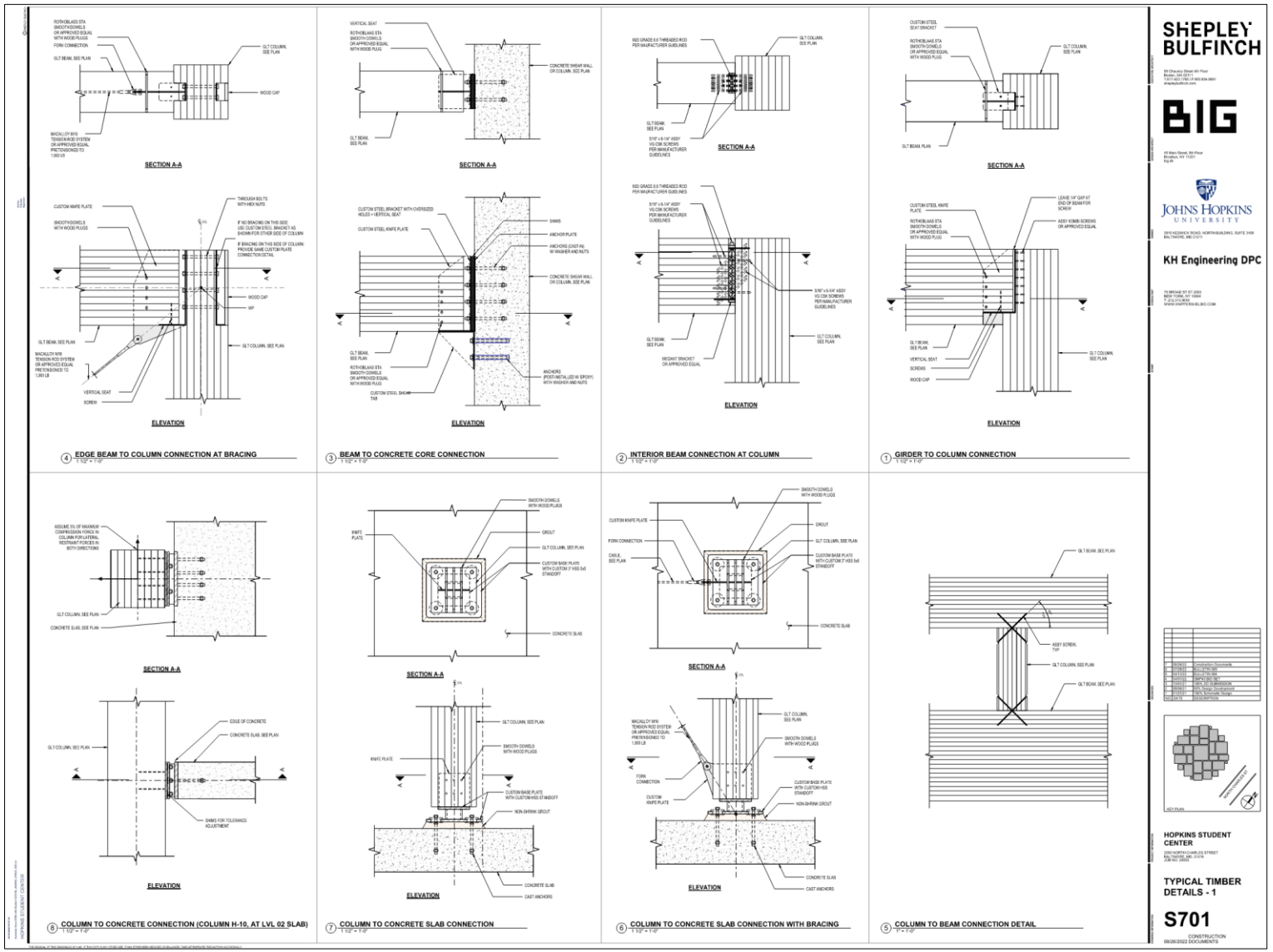


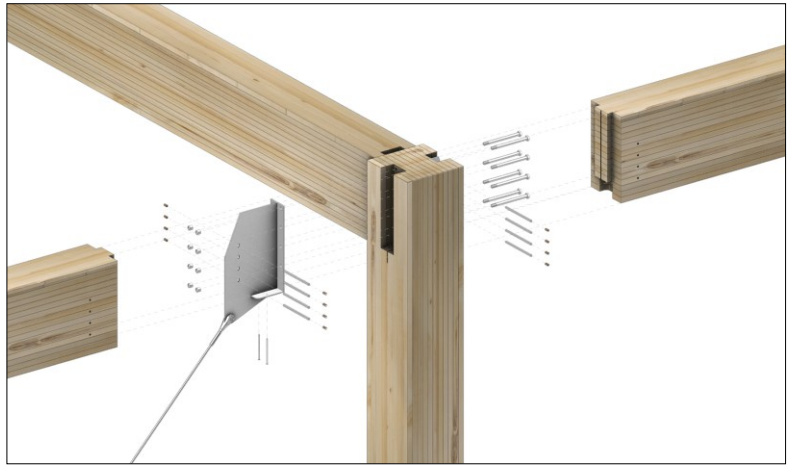


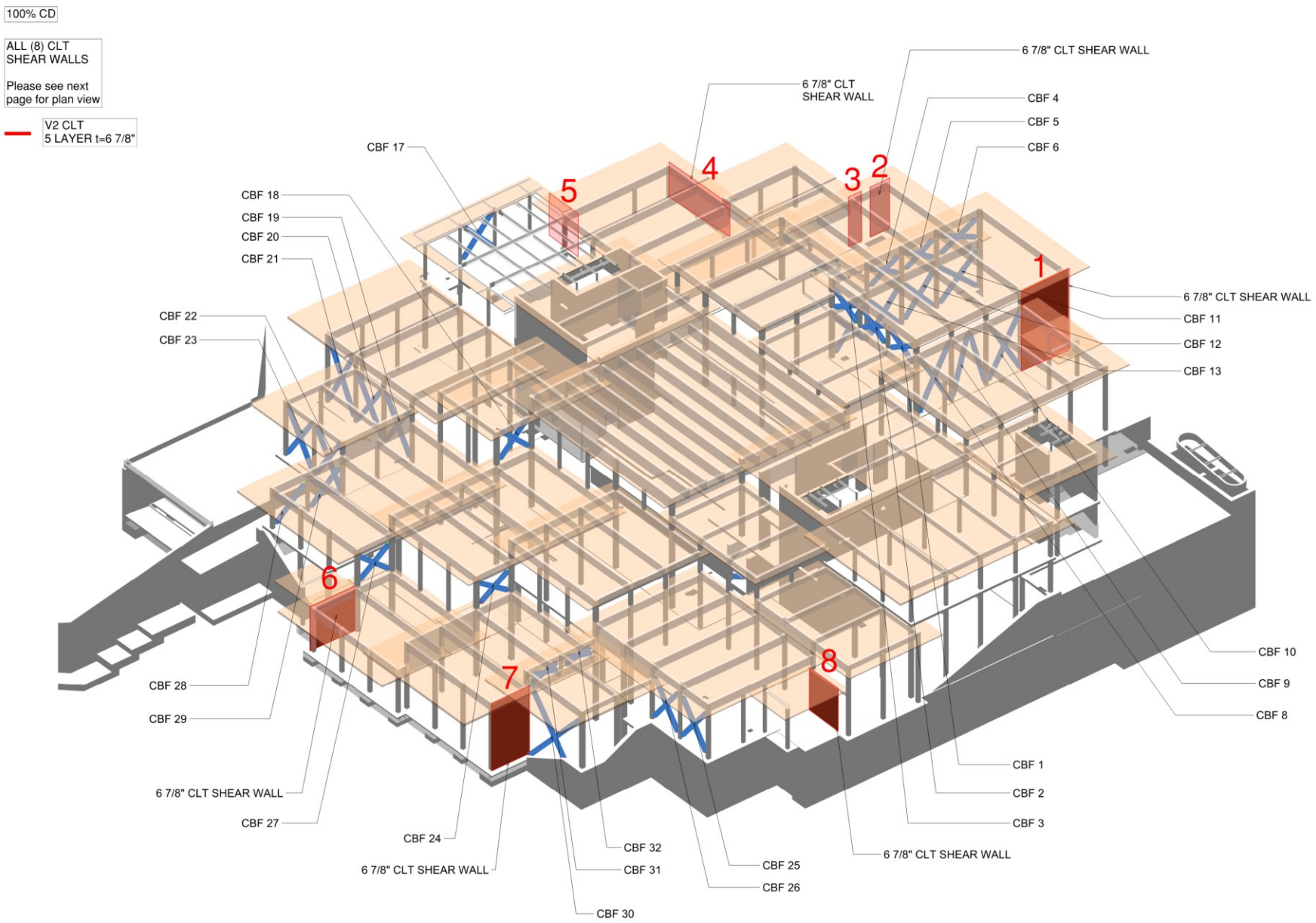
2





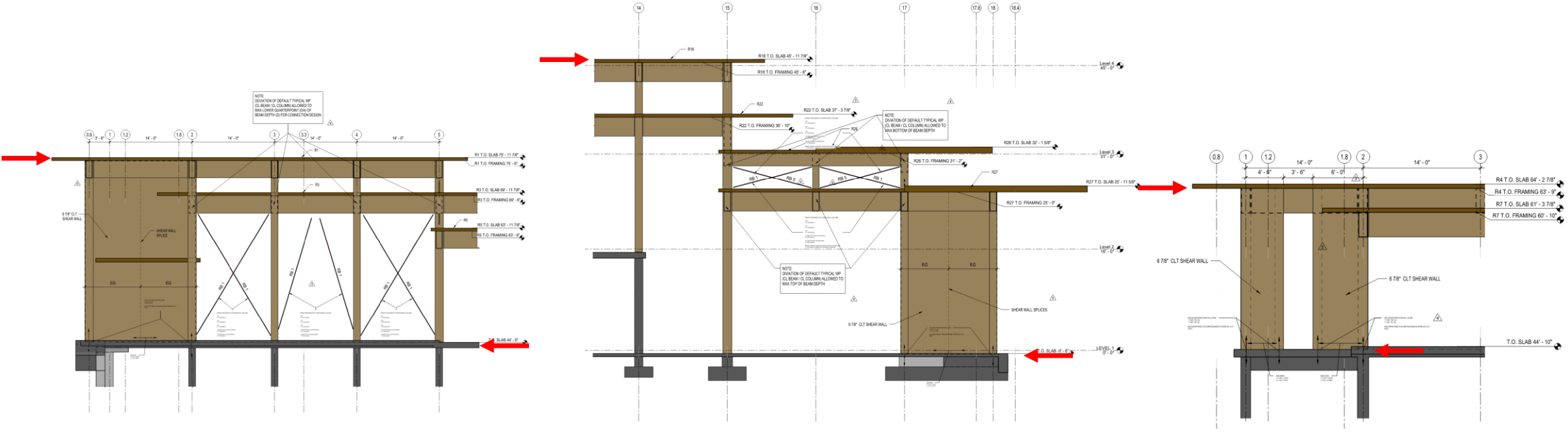

SECTION A-A
ELEVATION
4 EDGE BEAM TO COLUMN CONNECTION AT BRACING
1 1/2" = 1'-0"
3 BEAM TO CONCRETE CORE CONNECTION
1 1/2" = 1'-0"
2 INTERIOR BEAM CONNECTION AT COLUMN
1 1/2" = 1'-0"
1 GIRDER TO COLUMN CONNECTION
1 1/2" = 1'-0"
8 COLUMN TO CONCRETE CONNECTION (COLUMN H-10, AT LVL 02 SLAB)
1 1/2" = 1'-0"
7 COLUMN TO CONCRETE SLAB CONNECTION
1 1/2" = 1'-0"
6 COLUMN TO CONCRETE SLAB CONNECTION WITH BRACING
1 1/2" = 1'-0"
5 COLUMN TO BEAM CONNECTION DETAIL
1" = 1'-0"
SHEPLEY BULFINCH
BIG
JOHNS HOPKINS UNIVERSITY
KH Engineering DPC
HOPKINS STUDENT CENTER
TYPICAL TIMBER DETAILS - 1
S701
CONSTRUCTION
06/26/2022 DOCUMENTS





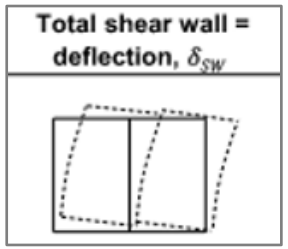
Horizontal-Lastabtrag

29 Furnierschichtholz-Dachscheiben
3 Stahlbetonkerne
8 aussteifende BSP-Wände
Stahlstab-Auskreuzungsverbände



exemplarische Darstellung BSP-Wände und Stahl-Aussteifungsverbände (Auszüge Structural Drawing Set knippershelbig)

Analyseschritte:



Schritt 1:

Definition der normativen Anforderungen, die der SPDWS-21* an BSP-Wände stellt

Schritt 2:

Aufsetzung eines Excelkalkulations-blatts, nach FEMA/AWC-Rechenbeispiel, zur Ermittlung der erwarteten, analytischen Wandverformung

Schritt 3:

Isolierung der CLT-Wände aus dem Globalmodel, iterative Steifigkeitsanpassung bis Verformung = analytischer Erwartungswert

Schritt 4:

Übertrag der ermittelten CLT-Steifikgeiten zurück ins Globalmodel

Schritt 5:

Schnittgrößener-mittlung im Globalmodel, Vergleich CLT- und Stahlverbandlasten vor vs. nach der Modifizierung

Verifizierter, steifigkeits-entsprechender Horizontallastabtrag

** SPDWS-21: Special Design Provisions for Wind and Seismic (ANSI/AWC, 2021)*

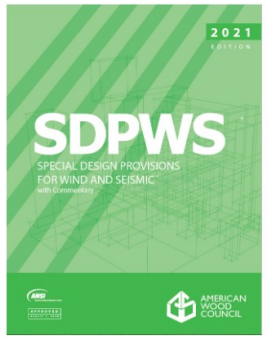
Diagramm: Ermittlung und Integration verifizierter BSP-Wand-Steifigkeitsparameter

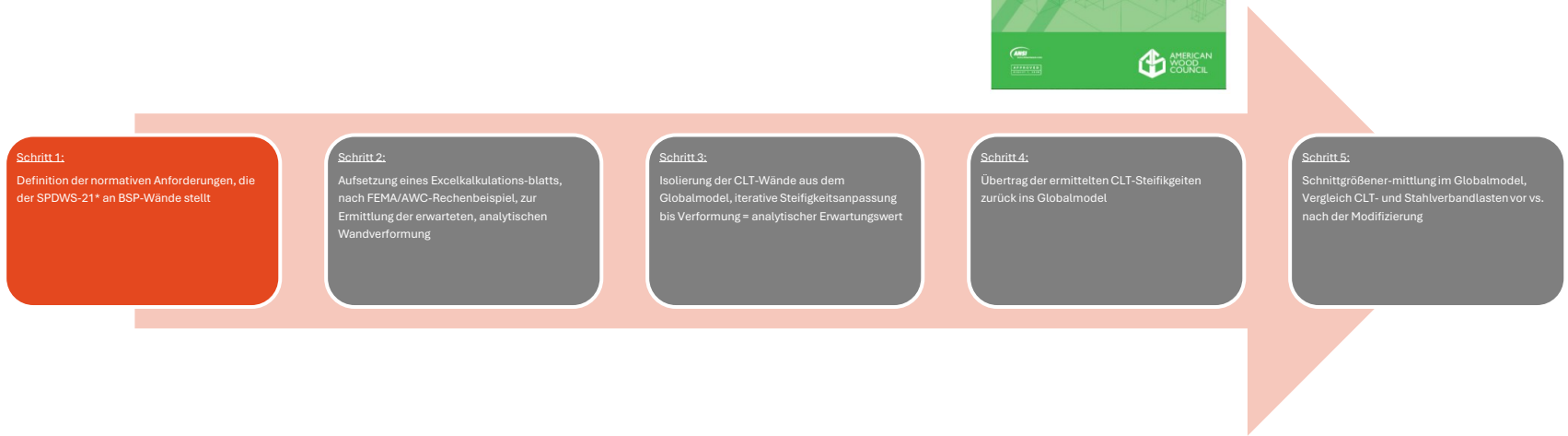
Schritt 1:
Definition der normativen Anforderungen, die der SDPWS-21 an BSP-Wände stellt

Anforderungen an die BSP-Schubelemente

- Höhen-Breiten-Verhältnisse (multi panel shear wall aspect ratio): $2 < h/b_s < 4$
- Wandelementierung
- Anzahl und Anforderungen an Verbindungsmittel
- Faktorisierung, bzw. Verhältnis Einwirkung / Widerstand

→ *z.B. Bemessung Zuganker, s. rechts*





B.3.4 Hold-down System

Each end of each shear wall shall be provided with a hold-down system. Hold-down systems shall comply with the following:

1. Hold-down systems shall consist of continuous tie-rod systems or conventional hold-down devices that use threaded anchor rods and nail, screw or bolt attachment to the CLT panel.
2. Where continuous tie-rod systems are used, rods at each level shall be designed for cumulative overturning tensile forces and bearing plates shall be provided at the floor level above each story Tie-rod elongation or conventional hold-down device deformation for each story shall not exceed 0.185 in. when applying strength design load combinations of ASCE 7.
3. The hold-down system including anchorage to the foundation shall be designed to resist not less than 2.0 times the forces associated with the design shear capacity of the CLT shear wall determined in accordance with 4.1.4.1 for seismic and 1.5 times the forces associated with the design shear capacity of the CLT shear wall in accordance with 4.1.4.2 for wind. Connections between the hold-down device and CLT panel shall comply with net section tension rupture, row tear-out, group tear-out in accordance with NDS Appendix E. The anchorage to foundation shall be designed in accordance with ACI 318.

Prescriptive Building Design

anstelle individueller, komplexer Berechnungsansätze werden spezifische Anforderungen an Bauteile oder Methoden gestellt, bei deren Einhaltung vereinfachte Berechnungsansätze angewendet werden dürfen.

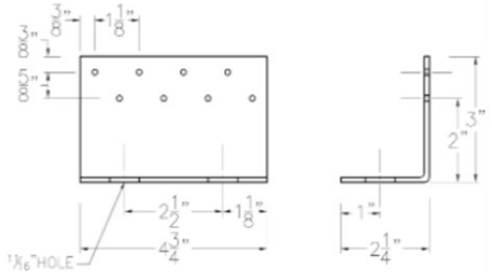
Hintergrund
oft experimentell ermittelte Werte, die auf spezifischen Bauteile und Verbindungen beruhen)

Nachteil
strenge Konformität erforderlich, wenig Flexibilität im Entwurf

spezifisch für aussteifende CLT-Wände gilt z.B.:

- Stahl-Winkelabmessungen sind exakt vorgegeben
- Zugehörige Schubtragfähigkeit gegeben (s. rechts)

Figure B.3.2 Top and Bottom of Wall Angle Connector



B.5 Nominal Unit Shear Capacity

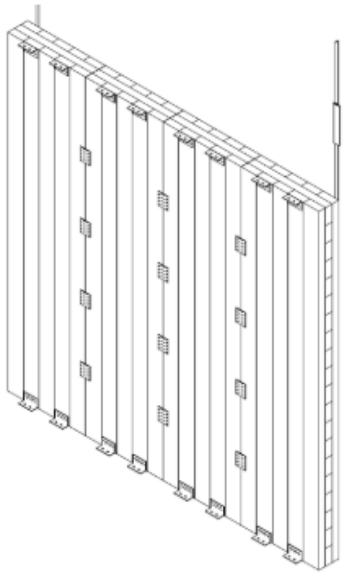
Nominal unit shear capacity, v_s, shall be in accordance with Equation B-2. Where both faces of a panel are provided with angle connectors in accordance with B.3.2, the nominal unit shear capacity shall be permitted to be taken as the sum of the nominal unit shear capacities of each face.

$$v_s = n\left(\frac{2605}{b_s}\right)C_G \qquad \text{(B-2)}$$

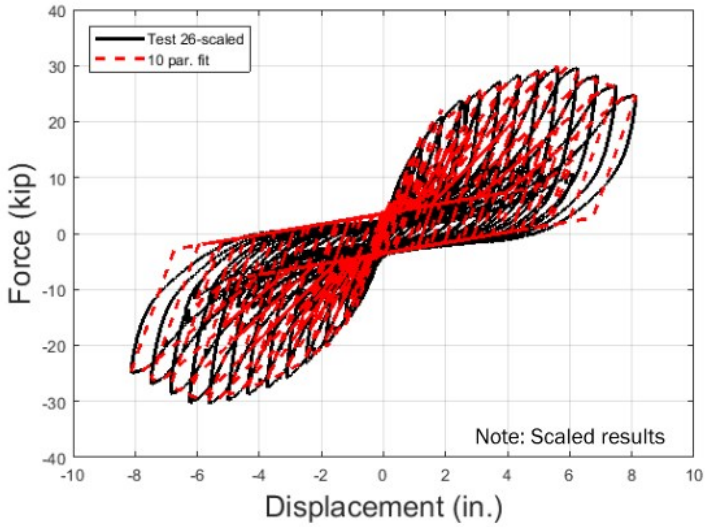
where:

n = number of angle connectors along bottom of panel face
2605 = connector nominal shear capacity in accordance with NDS (lbs)
b_s = individual CLT panel length (ft)
C_G = CLT panel specific gravity adjustment factor. $C_G = 1.0$ for $G \geq 0.42$. $C_G = 0.86$ for $G = 0.35$. Linear interpolation shall be permitted to be used to determine values of C_G for G between 0.35 and 0.42.

Auszug SDPWS-21, Kapitel B.5



From SDPWS Figure C-B.1



FEMA Building Seismic Safety Council nehrp

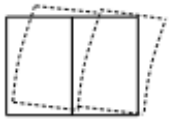
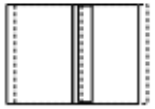
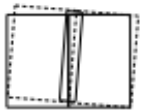
Federal Emergency Management Agency 11

Auszüge aus: Philip Line, M. Omar Amini: 2020 NEHRP Provisions Training Materials: Cross Laminated Timber (CLT) Shear Wall Example)

Schritt 2:
Analyse nach FEMA/AWC-Rechenbeispiel zur Ermittlung der erwarteten, analytischen Wandverformung

$$\delta_{SW} = \frac{576 v b_s h^3}{EI_{eff(in-plane)}} + \frac{vh}{GA_{eff(in-plane)}} + 3\Delta_{nail\ slip,h} + 2\Delta_{nail\ slip,v}\left(\frac{h}{b_s}\right) + \Delta_a \frac{h}{\sum b_s}$$

(SDPWS-21 Eq. B-1)

Total shear wall = deflection, δ_{SW}	Panel bending and shear +	Sliding +	Panel rotation +	Rigid body rotation
				

Schritt 1:
Definition der normativen Anforderungen, die der SPDWS-21* an BSP-Wände stellt

Schritt 2:
Aufsetzung eines Excelkalkulations-blatts, nach FEMA/AWC-Rechenbeispiel, zur Ermittlung der erwarteten, analytischen Wandverformung

Schritt 3:
Isolierung der CLT-Wände aus dem Globalmodel, iterative Steifigkeitsanpassung bis Verformung = analytischer Erwartungswert

Schritt 4:
Übertrag der ermittelten CLT-Steifikgeiten zurück ins Globalmodel

Schritt 5:
Schnittgrößener-mittlung im Globalmodel, Vergleich CLT- und Stahlverbandlasten vor vs. nach der Modifizierung

Verformungsanteile BSP-Scheibenelemente

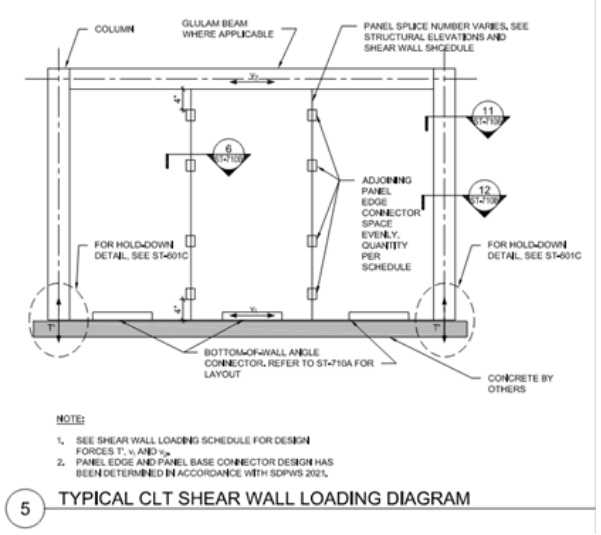
- Biegung und Schub in Scheibenebene
- Gleiten der Einzelelemente
- Rotation der Einzelelemente
- Starrkörperrotation der Gesamtscheibe

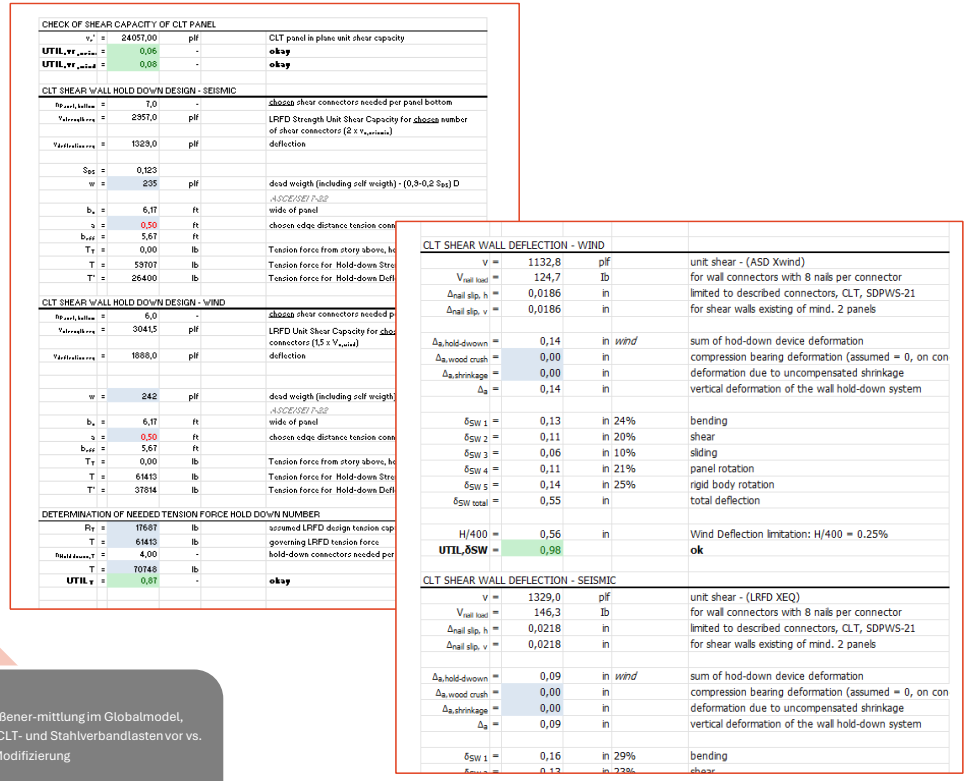
FEMA: 202 NEHRP Recommended Seismic Provisions: Design Examples, Training Materials, and Design Flow Charts, FEMA P-2192-V1/November 2021

Schritt 2:
Analyse nach FEMA/AWC-Rechenbeispiel zur Ermittlung der erwarteten, analytischen Wandverformung

Analytische Verformung der BSP-Wandscheibe abhängig:

- Anzahl, Lage, Ausbildung der Verbindungsmittel
- Breite der Einzelelemente
- Lagenaufbau CLT





CHECK OF SHEAR CAPACITY OF CLT PANEL				
v_r'	=	24057,00	plf	CLT panel in plane unit shear capacity
UTIL.vr	=	0,06	-	**okay**
UTIL.vr	=	0,08	-	**okay**
CLT SHEAR WALL HOLD DOWN DESIGN - SEISMIC				
	=	7,0	-	chosen shear connectors needed per panel bottom
	=	2357,0	plf	LRFD Strength Unit Shear Capacity for chosen number of shear connectors (2 x $v_{r,seismic}$)
	=	1329,0	plf	deflection
S_{DS}	=	0,123		
w	=	235	plf	dead weigth (including self weigth) - (0,3-0,2 S_{DS}) D
				ASCE/SEI 7-22
b_v	=	6,17	ft	wide of panel
a	=	0,50	ft	chosen edge distance tension conn
b_{eff}	=	5,67	ft	
T_T	=	0,00	lb	Tension force from story above, h
T	=	59707	lb	Tension force for Hold-down Stre
T'	=	26400	lb	Tension force for Hold-down Defl
CLT SHEAR WALL HOLD DOWN DESIGN - WIND				
	=	6,0	-	chosen shear connectors needed p
	=	3041,5	plf	LRFD Unit Shear Capacity for cho connectors (1,5 x $v_{r,seismic}$)
	=	1888,0	plf	deflection
w	=	242	plf	dead weigth (including self weigth)
				ASCE/SEI 7-22
b_v	=	6,17	ft	wide of panel
a	=	0,50	ft	chosen edge distance tension conn
b_{eff}	=	5,67	ft	
T_T	=	0,00	lb	Tension force from story above, h
T	=	61413	lb	Tension force for Hold-down Stre
T'	=	37814	lb	Tension force for Hold-down Defl
DETERMINATION OF NEEDED TENSION FORCE HOLD DOWN NUMBER				
R_T	=	17687	lb	assumed LRFD design tension cap
T	=	61413	lb	governing LRFD tension force
	=	4,00	-	hold-down connectors needed per
T	=	70748	lb	
UTIL.T	=	0,87	-	**okay**

CLT SHEAR WALL DEFLECTION - WIND					
v	=	1132,8	plf		unit shear - (ASD Xwind)
$V_{nail\ load}$	=	124,7	lb		for wall connectors with 8 nails per connector
$\Delta_{nail\ slip,\ h}$	=	0,0186	in		limited to described connectors, CLT, SDPWS-21
$\Delta_{nail\ slip,\ v}$	=	0,0186	in		for shear walls existing of mind. 2 panels
$\Delta_{a,hold\text{-}dwown}$	=	0,14	in	*wind*	sum of hod-down device deformation
$\Delta_{a,wood\ crush}$	=	0,00	in		compression bearing deformation (assumed = 0, on con
$\Delta_{a,shrinkage}$	=	0,00	in		deformation due to uncompensated shrinkage
Δ_a	=	0,14	in		vertical deformation of the wall hold-down system
$\delta_{SW\ 1}$	=	0,13	in	24%	bending
$\delta_{SW\ 2}$	=	0,11	in	20%	shear
$\delta_{SW\ 3}$	=	0,06	in	10%	sliding
$\delta_{SW\ 4}$	=	0,11	in	21%	panel rotation
$\delta_{SW\ 5}$	=	0,14	in	25%	rigid body rotation
$\delta_{SW\ total}$	=	0,55	in		total deflection
H/400	=	0,56	in		Wind Deflection limitation: H/400 = 0.25%
UTIL.δSW	=	0,98			**ok**
CLT SHEAR WALL DEFLECTION - SEISMIC					
v	=	1329,0	plf		unit shear - (LRFD XEQ)
$V_{nail\ load}$	=	146,3	lb		for wall connectors with 8 nails per connector
$\Delta_{nail\ slip,\ h}$	=	0,0218	in		limited to described connectors, CLT, SDPWS-21
$\Delta_{nail\ slip,\ v}$	=	0,0218	in		for shear walls existing of mind. 2 panels
$\Delta_{a,hold\text{-}dwown}$	=	0,09	in	*wind*	sum of hod-down device deformation
$\Delta_{a,wood\ crush}$	=	0,00	in		compression bearing deformation (assumed = 0, on con
$\Delta_{a,shrinkage}$	=	0,00	in		deformation due to uncompensated shrinkage
Δ_a	=	0,09	in		vertical deformation of the wall hold-down system
$\delta_{SW\ 1}$	=	0,16	in	29%	bending
		0,13	in	23%	shear

Schritt 1:
Definition der normativen Anforderungen, die der SPDWS-21* an BSP-Wände stellt

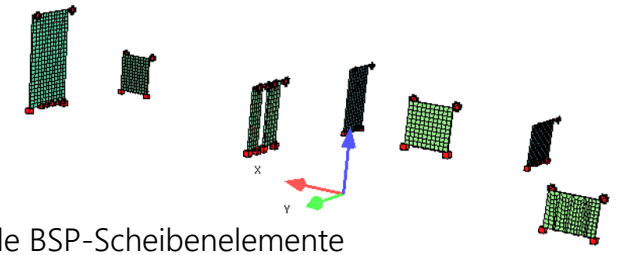
Schritt 2:
Aufsetzung eines Excelkalkulations-blatts, nach FEMA/AWC-Rechenbeispiel, zur Ermittlung der erwarteten, analytischen Wandverformung

Schritt 3:
Isolierung der CLT-Wände aus dem Globalmodel, iterative Steifigkeitsanpassung bis Verformung = analytischer Erwartungswert

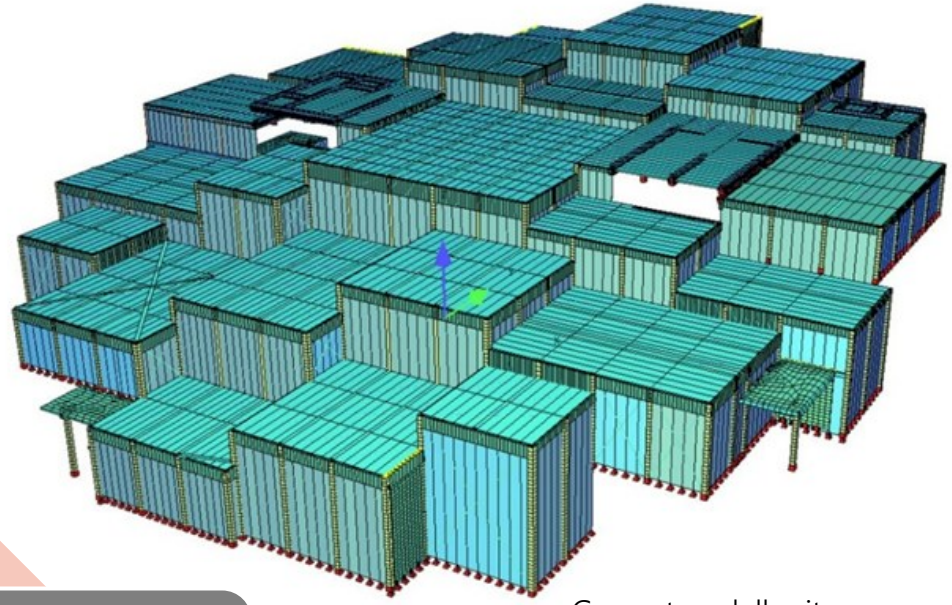
Schritt 4:
Übertrag der ermittelten CLT-Steifikgeiten zurück ins Globalmodel

Schritt 5:
Schnittgrößener-mittlung im Globalmodel, Vergleich CLT- und Stahlverbandlasten vor vs. nach der Modifizierung

Schritt 3:
Steifigkeitsanpassung der BSP-Wände unter Schubbelastung gemäß analytisch ermittelter Werte (aus Schritt 2) anhand isolierter BSP Wandmodelle



Einzelmodelle BSP-Scheibenelemente



Gesamtmodell mit angepassten Systemsteifgkeiten in SOFiSTiK

Schritt 4:
Übertrag der angepassten BSP Scheibenmodelle in das Gesamtmodell

Schritt 1:
Definition der normativen Anforderungen, die der SPDWS-21* an BSP-Wände stellt

Schritt 2:
Aufsetzung eines Excelkalkulations-blatts, nach FEMA/AWC-Rechenbeispiel, zur Ermittlung der erwarteten, analytischen Wandverformung

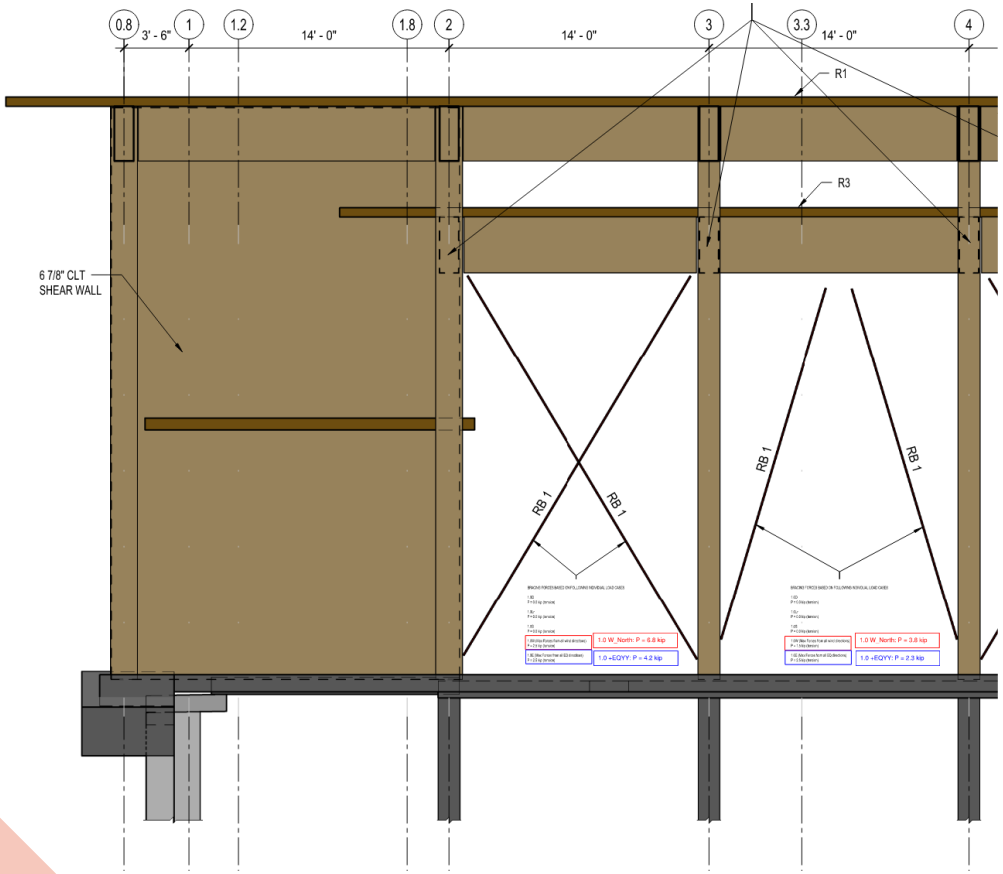
Schritt 3:
Isolierung der CLT-Wände aus dem Globalmodel, iterative Steifigkeitsanpassung bis Verformung = analytischer Erwartungswert

Schritt 4:
Übertrag der ermittelten CLT-Steifikgeiten zurück ins Globalmodel

Schritt 5:
Schnittgrößener-mittlung im Globalmodel, Vergleich CLT- und Stahlverbandlasten vor vs. nach der Modifizierung

Schritt 5:

Schnittgrößenermittlung der aussteifenden Systeme,
Sensitivitätsanalysen der Auswirkungen unterschiedlicher Steifigkeiten
der konkurrierenden Aussteifungssysteme



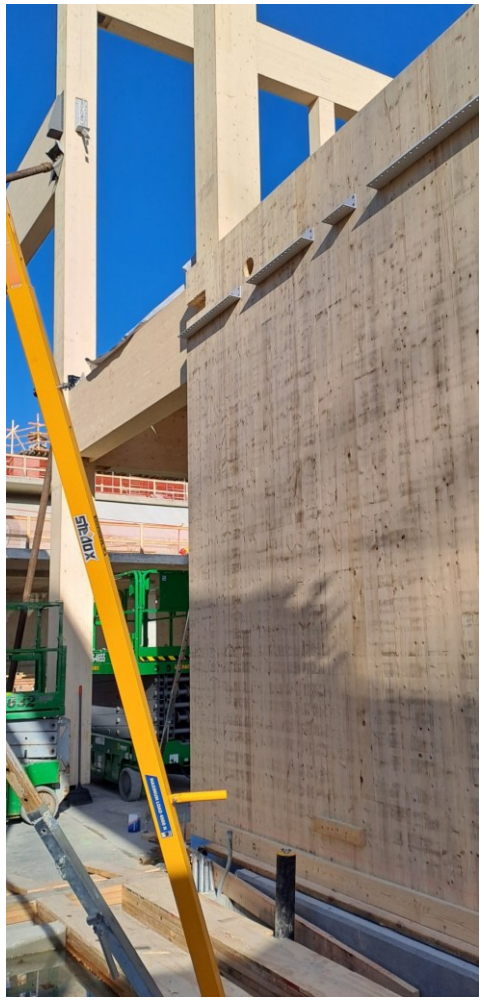
Schritt 1:
Definition der normativen Anforderungen, die der SPDWS-21* an BSP-Wände stellt

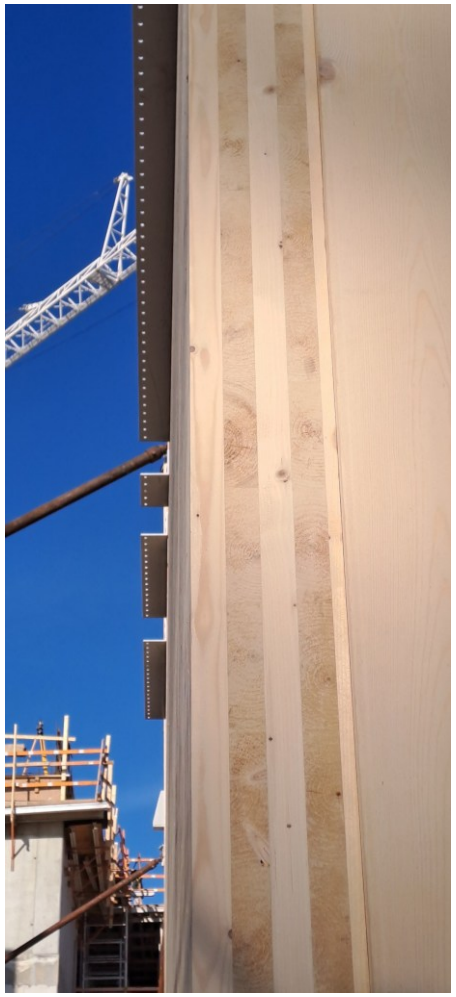
Schritt 2:
Aufsetzung eines Excelkalkulations-blatts, nach FEMA/AWC-Rechenbeispiel, zur Ermittlung der erwarteten, analytischen Wandverformung

Schritt 3:
Isolierung der CLT-Wände aus dem Globalmodel, iterative Steifigkeitsanpassung bis Verformung = analytischer Erwartungswert

Schritt 4:
Übertrag der ermittelten CLT-Steifikgeiten zurück ins Globalmodel

Schritt 5:
Schnittgrößenermittlung im Globalmodel, Vergleich CLT- und Stahlverbandlasten vor vs. nach der Modifizierung





Holzbaumontage – aufgestellte BSP-Wände Nr. 6 (links), Nr. 7 (rechts)

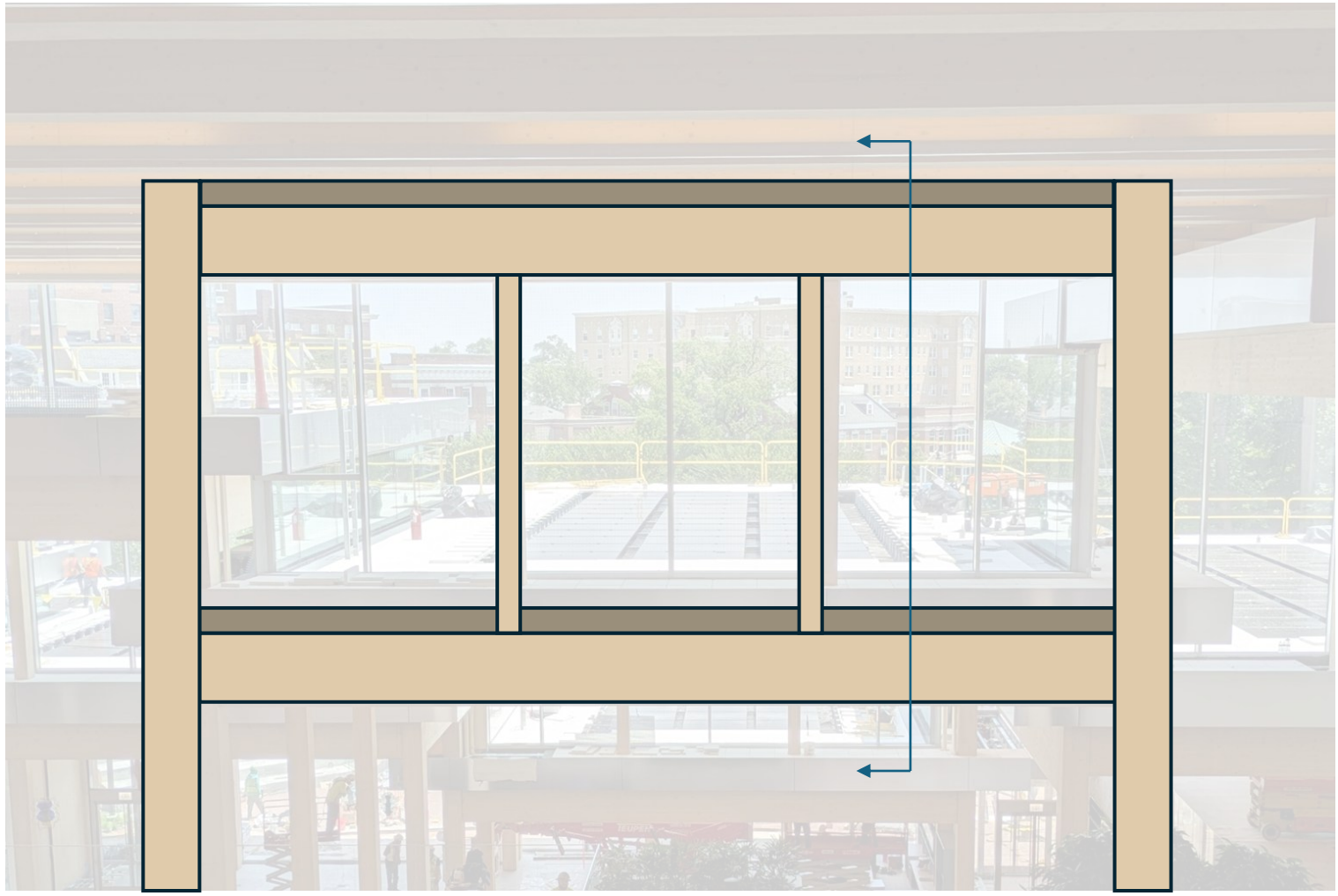
70 ft (21 m) langer Balken (10,000 lb oder 4535 kg schwer) - 21m Dachfeld

BSH Randträger mit 86” (218 cm) Höhe

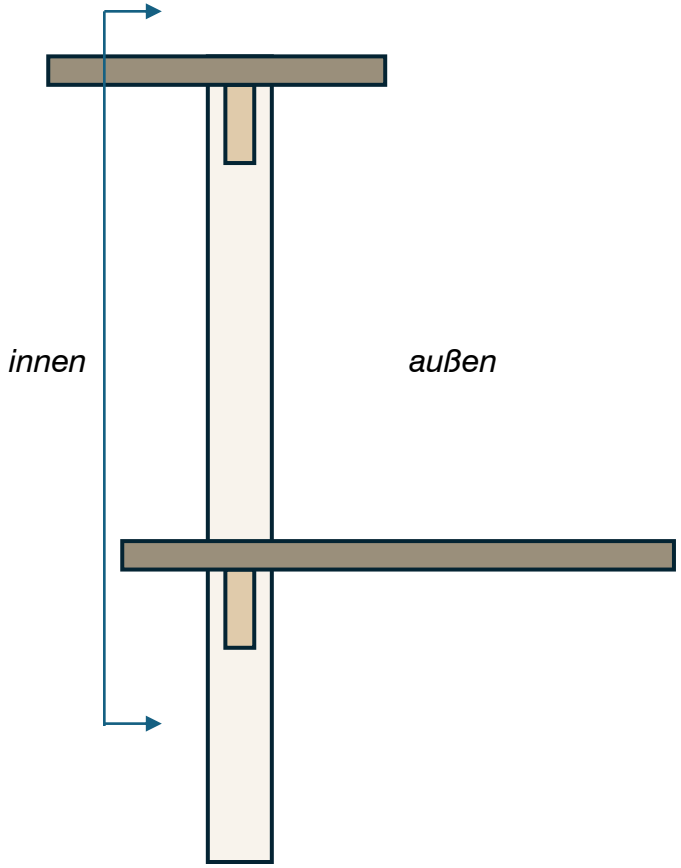
Einhub, temporäre Festhalterungen für den 21m weit spannenden Träger

54” (137 cm) hoher Balken des 21 m weit spannenden Dachfeldes

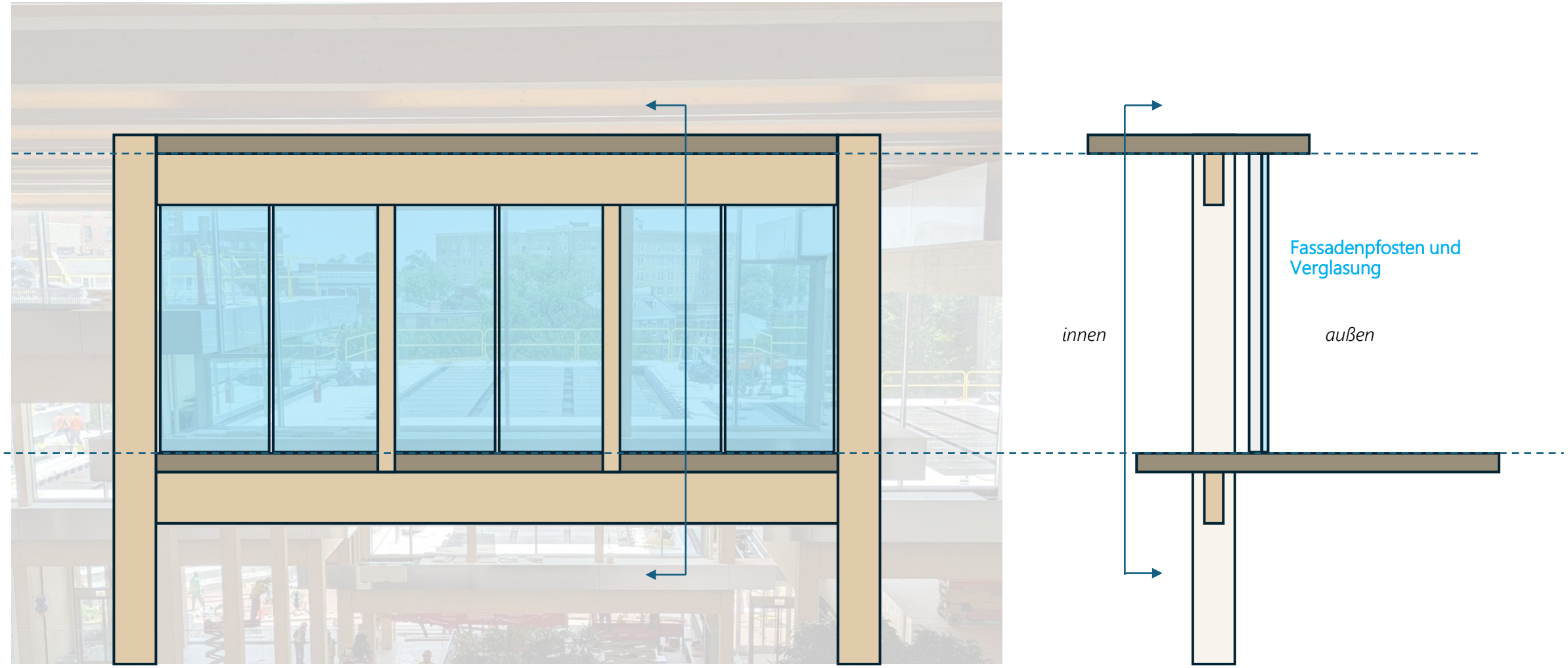
Fassade: Atrium - Oberlicht



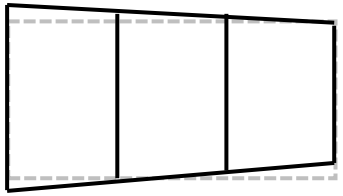
Oberlichter mit integrierter Fassade, Ansicht von Innen



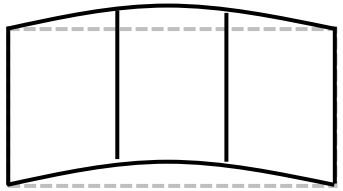
Schnitt


Fassadenpfosten und Verglasung
innen
außen

Effekte aus Bautoleranzen

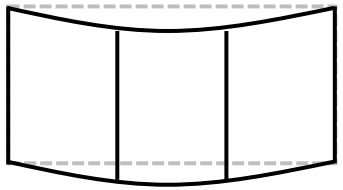


Berücksichtigung der statischen Überhöhung der Balken und dessen Fertigungstoleranz



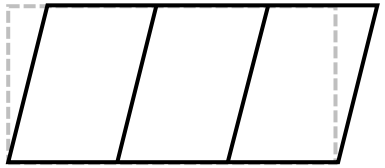
Effekte aus vertikaler Verformung des Tragwerks

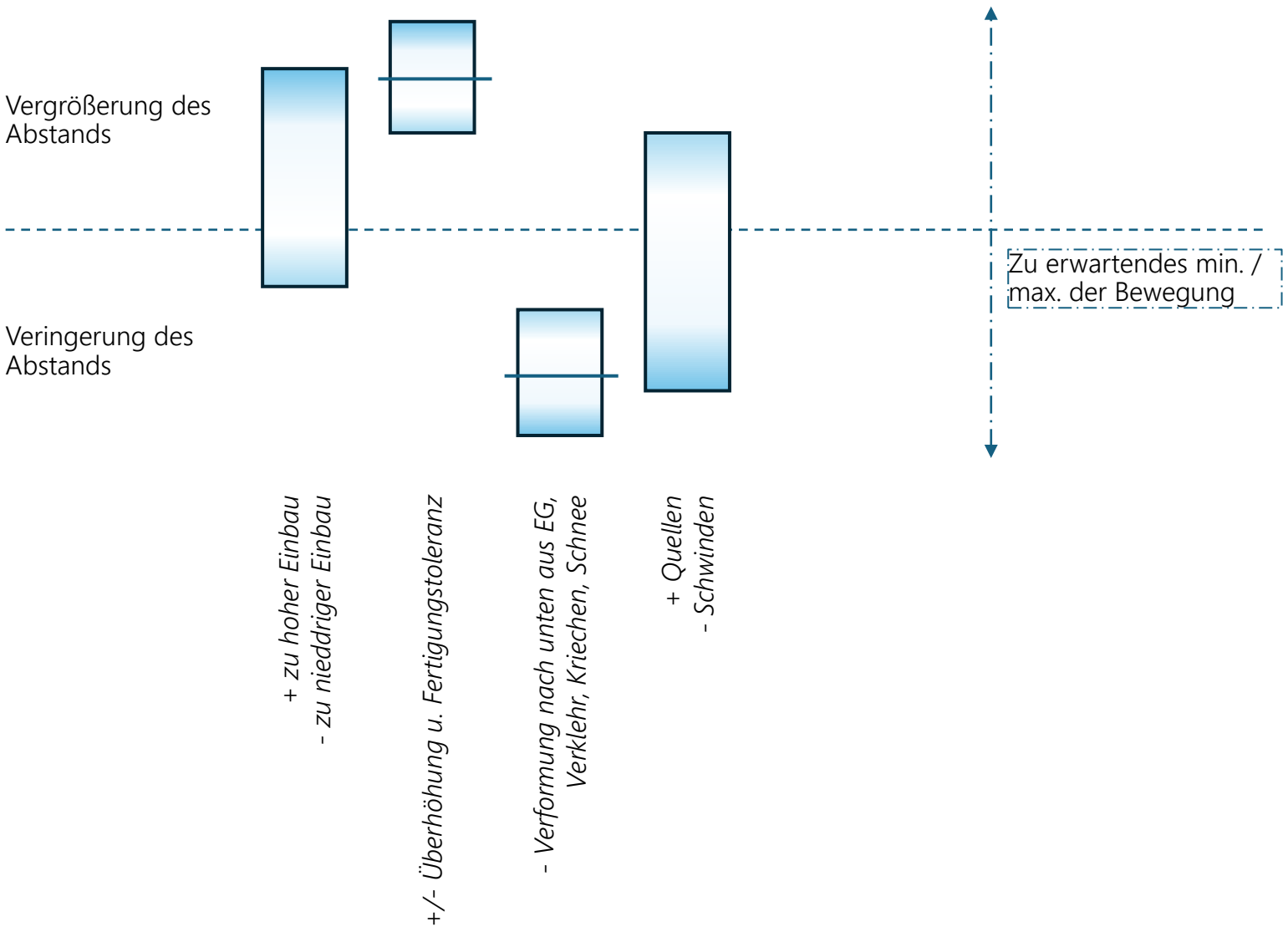
(Kriechen, Verkehr, Schnee)



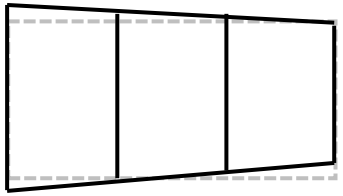
Effekte aus Quellen und Schwinden

Effekte aus horizontaler Verformung des Tragwerks (Wind, Erdbeben)

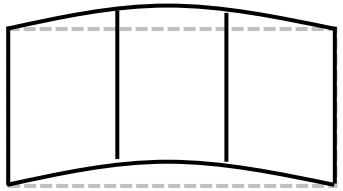




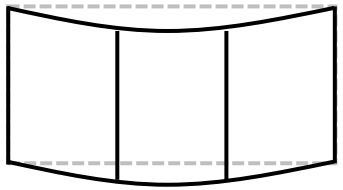
Effekte aus Bautoleranzen



Berücksichtigung der statischen Überhöhung der Balken und dessen Fertigungstoleranz

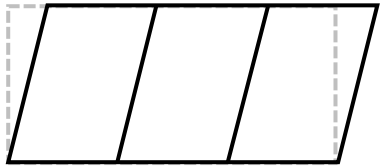


Effekte aus vertikaler Verformung des Tragwerks (EG, Kriechen, Verkehr, Schnee)



Effekte aus Quellen und Schwinden

Effekte aus horizontaler Verformung des Tragwerks (Wind, Erdbeben)



Anstelle einer “worst case” Überlagerung der Bewegungen und unnötig großer Fugenbreiten:

- Präzise Ermittlung der auftretenden Bewegungen in der Fuge aus den genannten Mechanismen
- Ausgleich von Toleranzen aus Bau- und Fertigung bereits im Moment des Einbaus (z.B. mit Unterfütterung etc.)
- strategische Platzierung des Bolzens in der Bewegungsfuge, um etwaige zukünftige Auf- oder Abbewegungen *nach Einbau* zu kompensieren

Type A Bracket analysis - Mid span - Clerestory

Installation and fabrication tolerance		
Upward tolerance	Tol_{upward}	-1,25 in
Downward tolerance	$Tol_{downward}$	0,5 in

Moisture content		
Maximum moisture content at installation		16%
Anticipated moisture at fabrication		12%
Minimum moisture content in service		7%
Change in moisture content service		-9%
Change in moisture content installation		4%
Beam depth from bracket (d-3in)		51 in
Maximum beam shrinkage	$\Delta_{shirnkage,MC}$	**0,87 in**
Surelevation due to swelling at installation	$\Delta_{swelling,MC}$	**-0,39 in**

Specified precamber		
Specified precamber RELATIVE	PC_{spec}	**0 in**

Allowance for variable loads& long term perm. after installation	$\Delta_{var,long\ term}$	**0,125** in

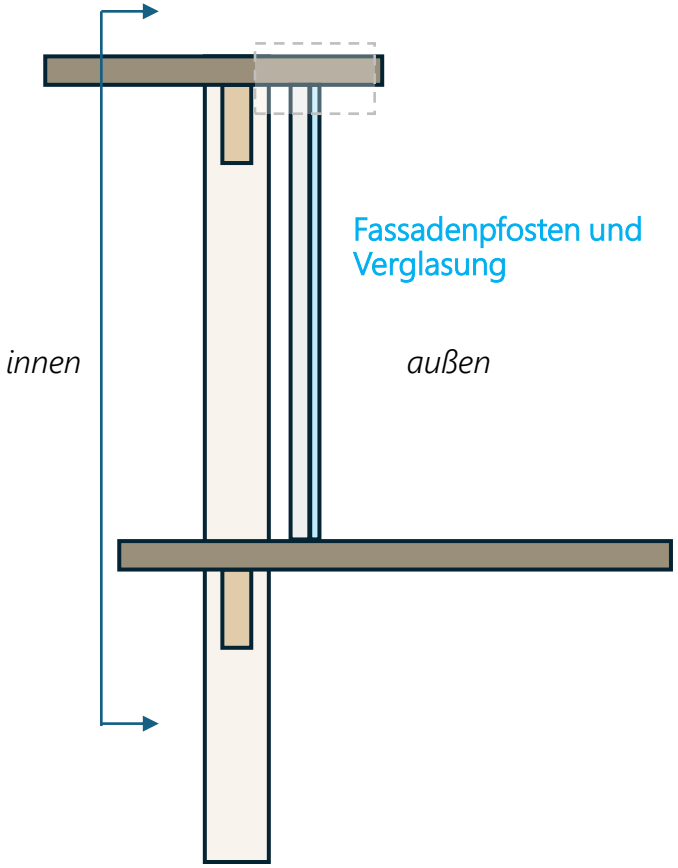
Determination of minimum slot dimension			
Downard	$\Delta_{var,long\ term} + \Delta_{shrinkage,MC}$	**1,00 in**	**25,3 mm**
Upward	$-\Delta_{var,long\ term}$	**-0,13 in**	**-3,2 mm**

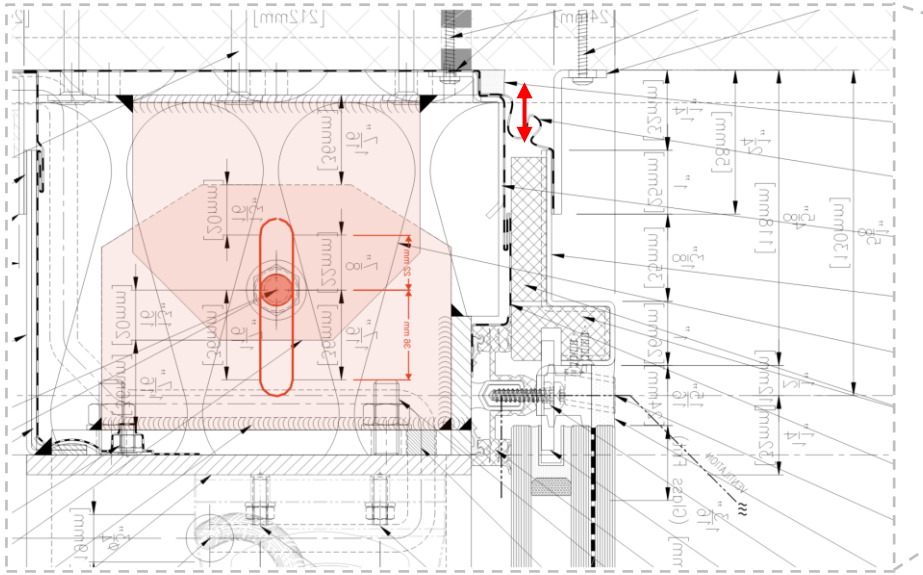
All negative values are upward, all positive values are downward

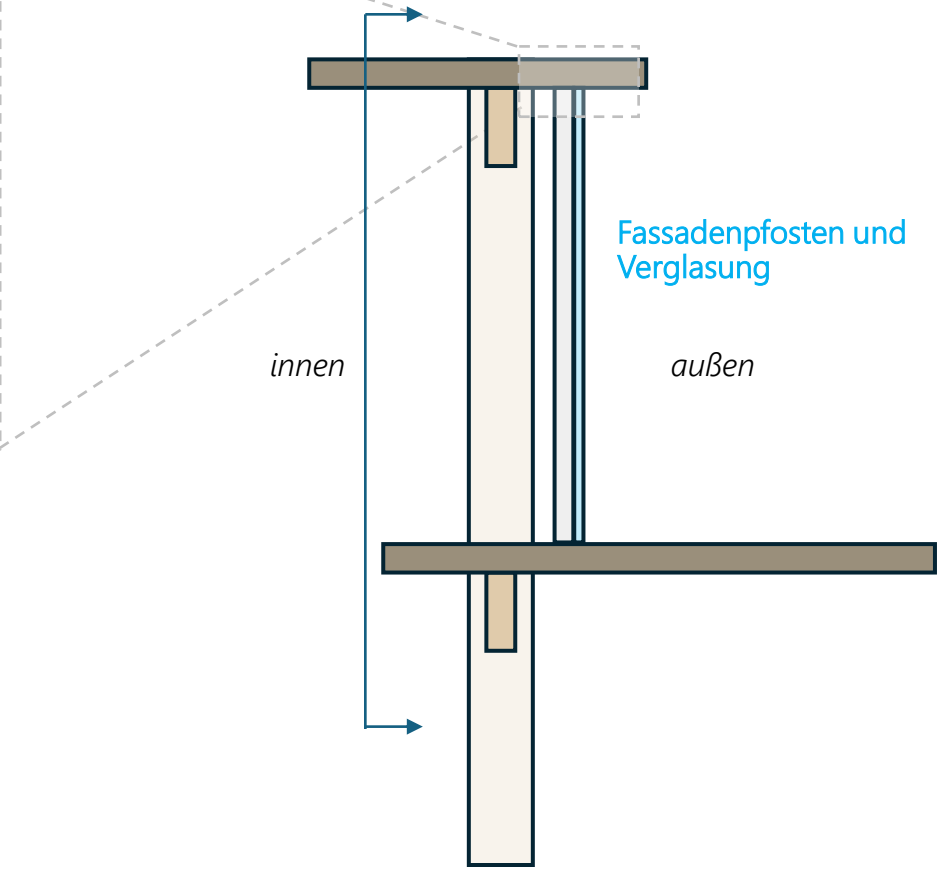
Determination of shim height			
Maximum	$\Delta_{swelling,MC} + Tol_{upward}$	**1,64 in**	**41,6 mm**
Minimum	$Tol_{downward}$	**-0,50 in**	**-12,7 mm**

Min. thickness assumes superimposed dead load is applied after façade installation

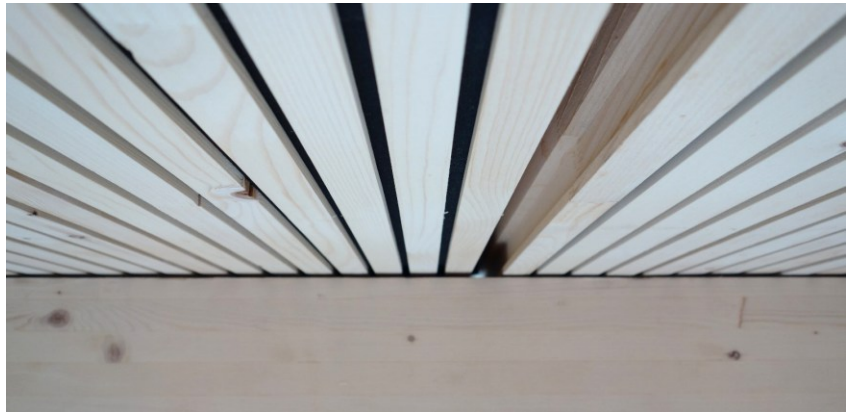
Negative shim values represent a lower position of the bolt within the slot compared to shop drawing zero position

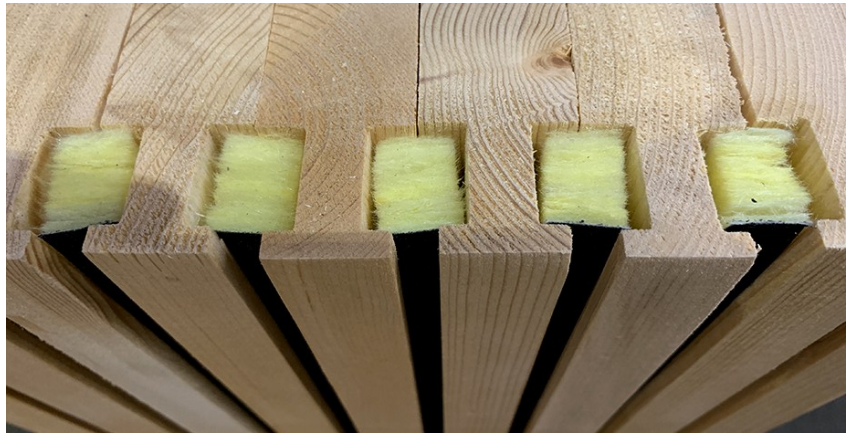




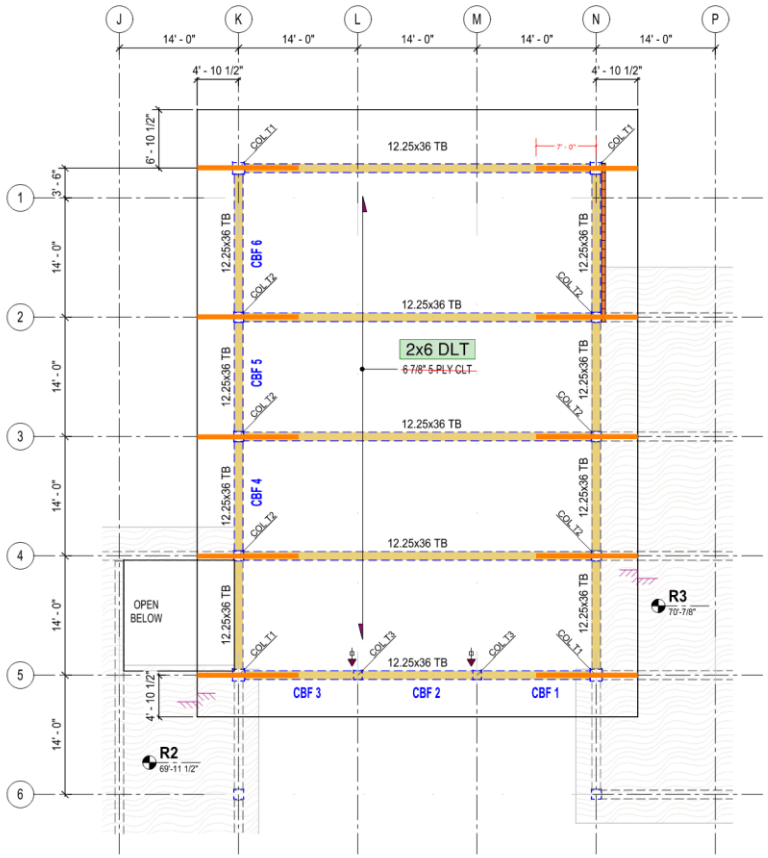


Bewegungsfuge am oberen Anschluss des Fassadenpfosten



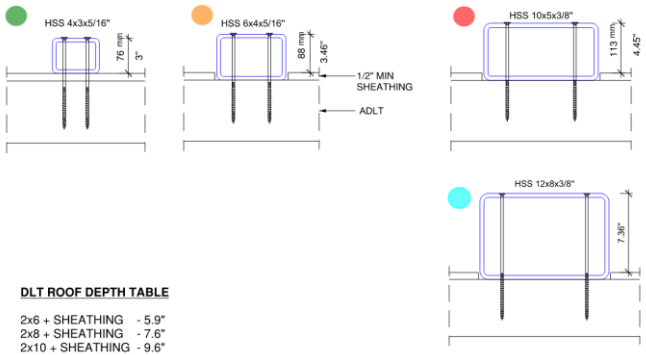


Mock-Up des Dübelholz-Elements mit **innenliegendem Bereich** (links) inkl. Akustik-Einlagen (NRC 0,7),
Übergang Innen/Außen (Mitte) und **außenliegender Bereich** (rechts),
Abdichtung der Stoßfugen der Brettstapelelemente



LEGEND:

- (SMALL HSS): For the 3'-4'' overhang: HSS 4x3x5/16'' on top of the sheathing, backspan 5ft
- (MEDIUM HSS): For the 4'-10'' overhang: HSS 6x4x5/16'' on top of ADLT, backspan 7ft
- (LARGE HSS): For the 6'-10'' overhang: HSS 10x5x3/8'' on top of ADLT, backspan 10ft
- HSS on side of the column or outrigger attached to the face of the column
- (X-LARGE HSS): For the 10'-4'' overhang: HSS 12x8x3/8'' on top of ADLT, backspan 14ft (only roof #22)



DLT ROOF DEPTH TABLE

2x6 + SHEATHING - 5.9"
2x8 + SHEATHING - 7.6"
2x10 + SHEATHING - 9.6"
2x12 + SHEATHING - 11.6"

Note, 1/2'' sheathing has been assumed. This depth is to be confirmed once diaphragm loading and deflection criteria are provided by KH.

StructureCraft

Project	JHSC Student Center
Subject	Preliminary Roof Sizing

DHL
StructureCraft

April 2023

August 2023

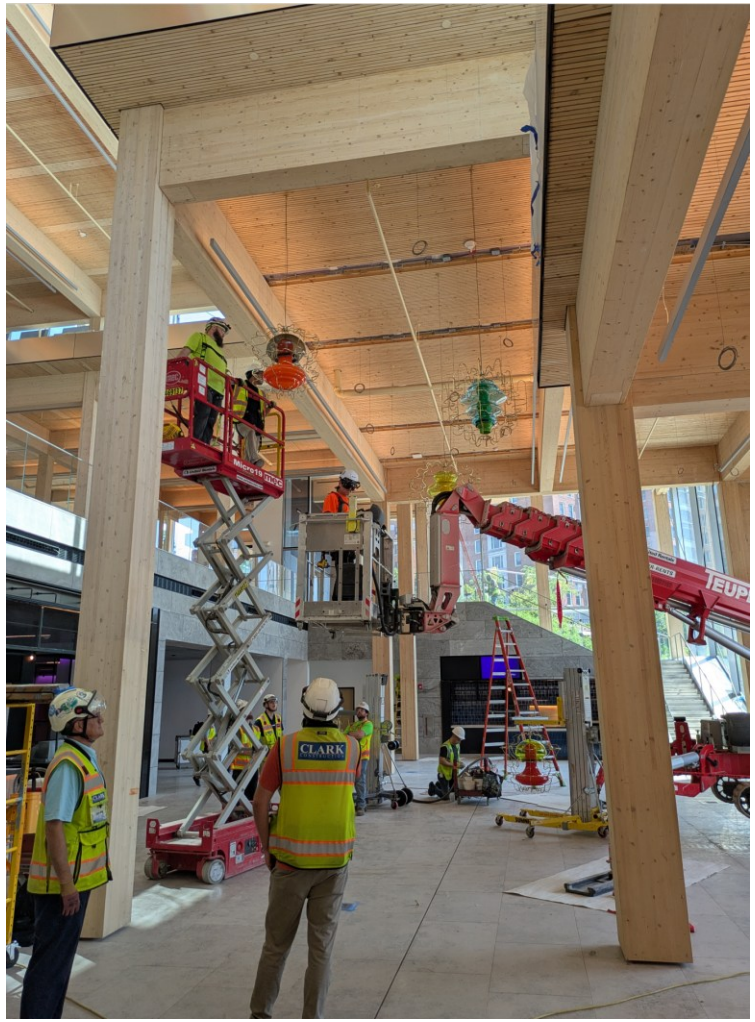
Oktober 2023

Dezember 2023

März 2024

Dezember 2024

März 2025



Juli 2025

Bloomberg Student Center
Johns Hopkins University

Client
Johns Hopkins University, Baltimore

Design Architect
Bjarke Ingels Group, New York

Executive Architect
Shepley Bulfinch, Boston

Structural and Facade Engineering
knippershelbig, New York

Team
Florian Meier, Roman Schieber, Sabrina Machanek,
Nils Lange, Shamil Lallami, Asaka Rieser, Alex Cheng
Chris Rieser (Engineer of Record)

General Contractor
Clark Construction

Sub-Contractor Timber Construction
Design Assist / Review Competition Phase
Structure Craft, Vancouver

Sub-Contractor Facade
Design Assist
Lindner Group

Internationales Holzbau Forum 2025

Der Holzbau des Bloomberg Student Centers an der Johns Hopkins University in Baltimore