2 GF Mount St, Cirencester
Gloucestershire, UK,
GL7 1TJ

Mobile: +44 (0)7706 895172
Skype: davide,tonelli
Email: davide.tonelli86@gmail.com
Website: www.dic.unipi.it/davide.tonelli/

Davide Tonelli

Portfolio

In the following it is shown a list of the major tasks I have been involved with while working at OPS Ltd.

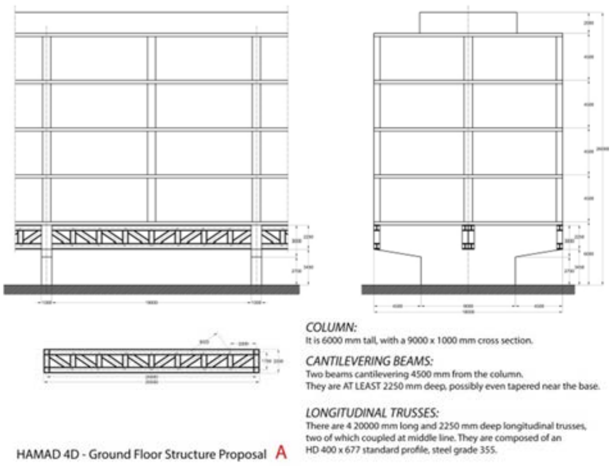
The works are arranged in chronological order, starting from March 2015 up to now.

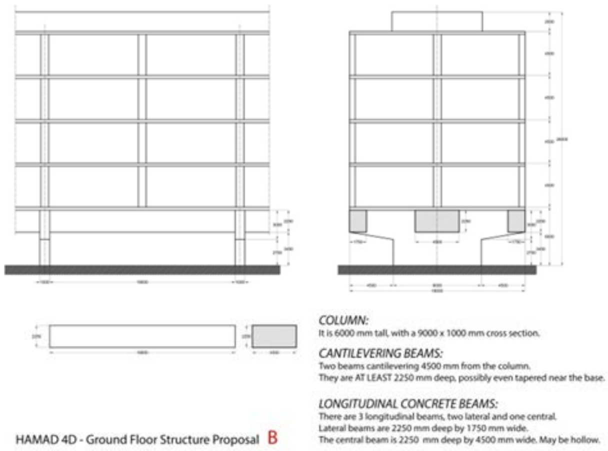
For each work a general description is provided which illustrates ONLY my personal contributions to the project, followed by a meaningful list of pictures and/or report excerpts.

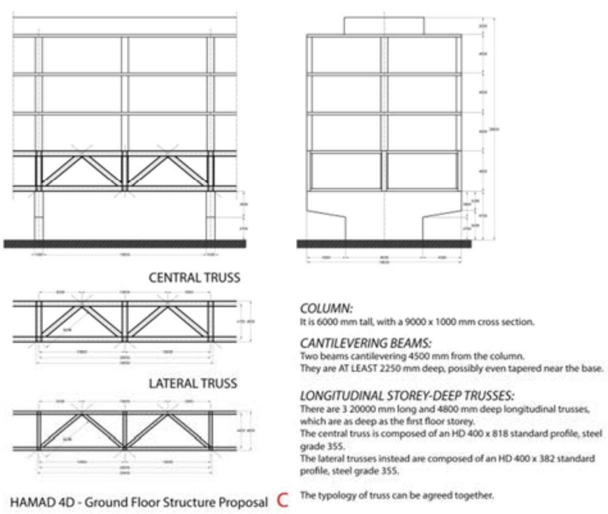
1 *Scheme design for a complex of 6 RC multi-storey buildings in Qatar*

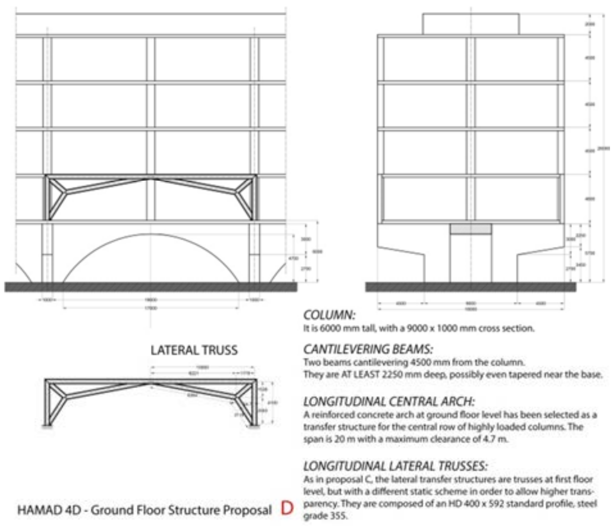
Being provided with architectural floor plans I started with sizing columns, cores and slabs. At a later stage, as usual during scheme design, I had to develop a number of options for prospective transfer structures at GF.
Also, as the building site has a high water table, I had to design different arrangements of tension piles at foundation level.

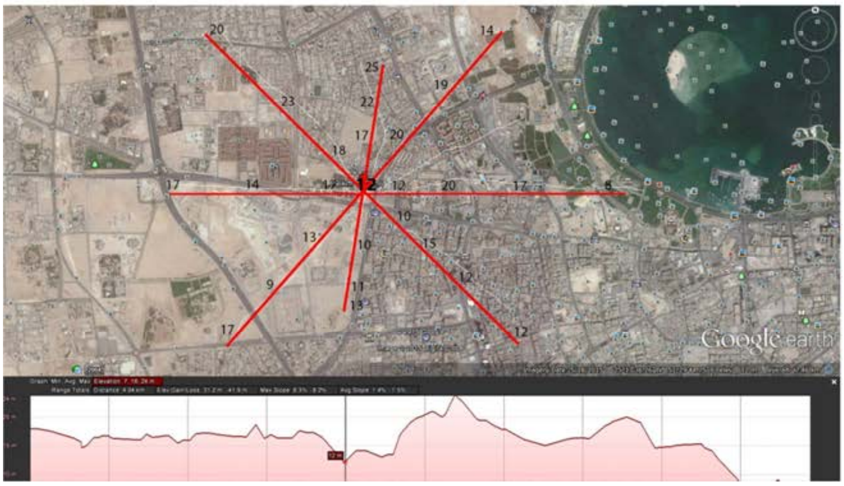
Transfer structures at GF

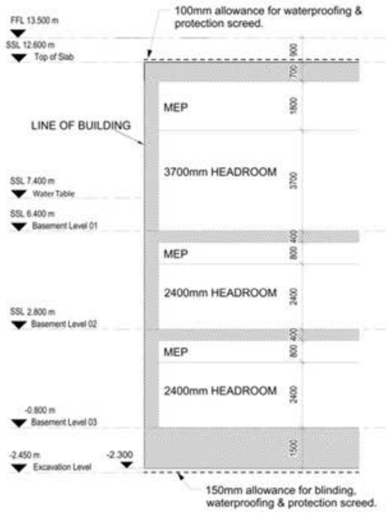










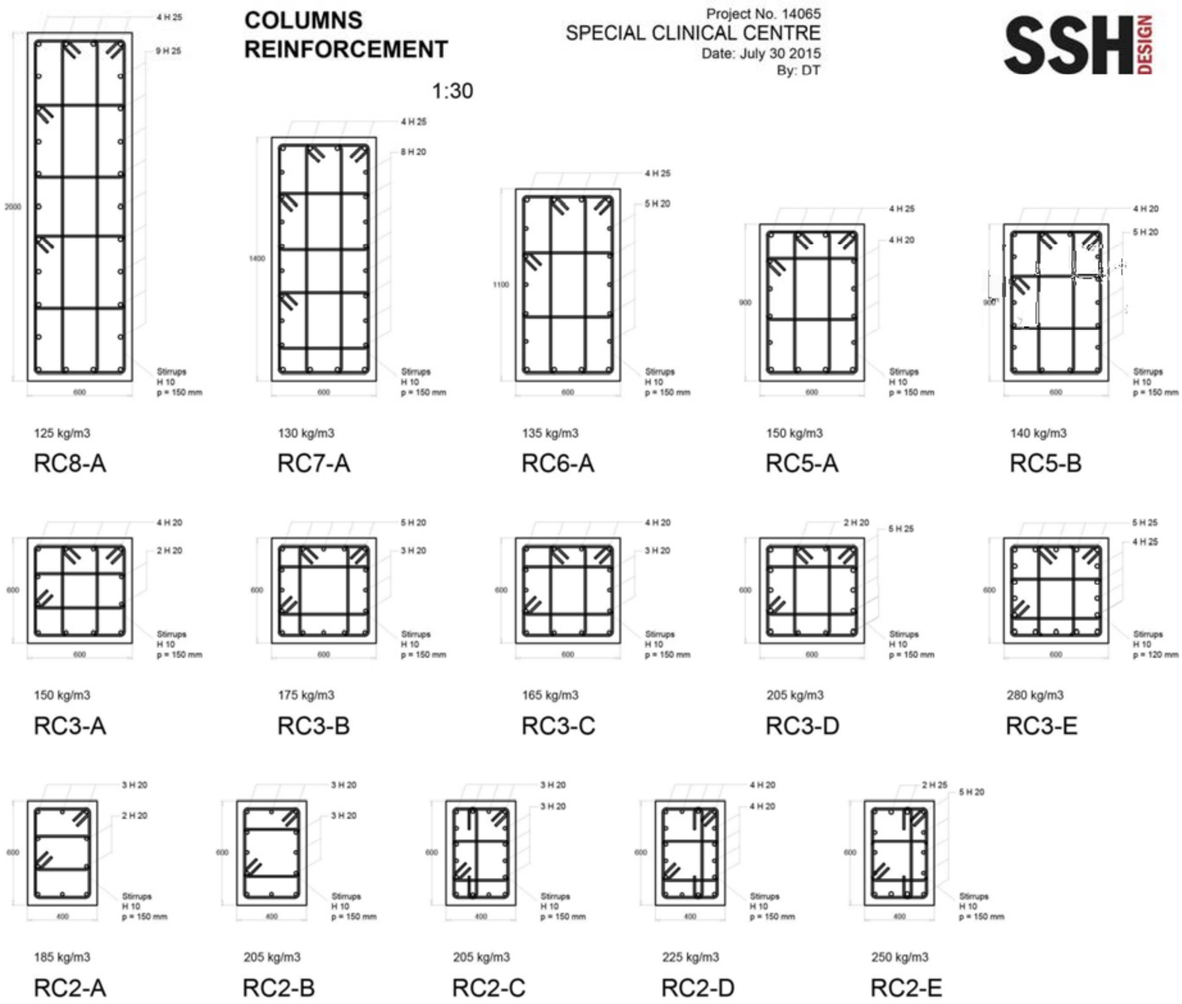


Altitude study of the site (left) and water table on basement

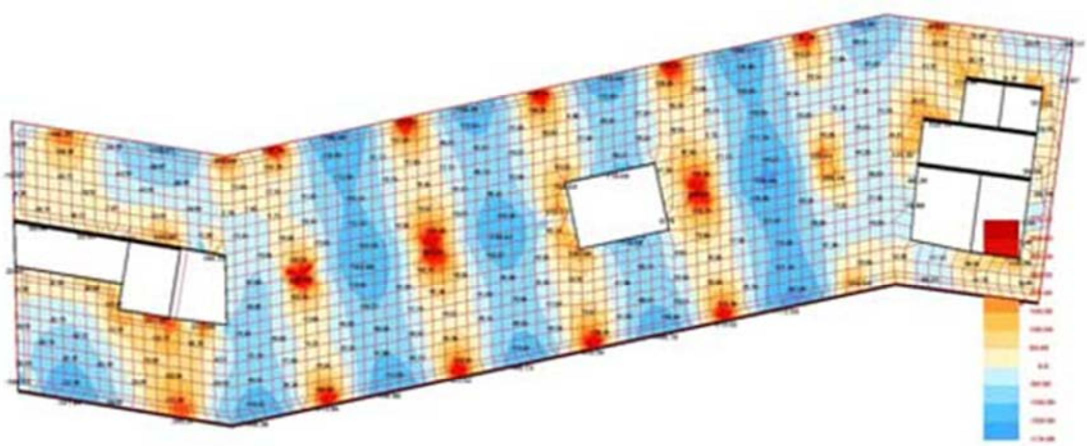
Basement raft with tension piles: general arrangement

2 *Detailed design for a RC multi-storey buildings in Qatar*

At a later stage I carried out the detailed design for the concrete members of one of the aforementioned six buildings.
The whole building was modelled in the F.E. program Robot, then tailor-made spread-sheets were set up in order to carry out the structural design.



Columns cross sections with reinforcement rate and disposition



Screenshot of a floor slab modelled in Robot

SSH DESIGN

Project No. 14065
SPECIAL CLINICAL CENTRE
Date: July 30 2015
By : DT

SLABS - BENDING MAIN REINFORCEMENT + CRACK WIDTH CHECK

	P1 - P4							
	Transverse Column Strip - A,B,C					Longitudinal Column Strip - A,E,H		
	A	B	C			A	E	H
	First Support	First Span	"0" Support			Typical Support	Typical Span	Typical Support
EFA	-385.8	174.9	-		EFA	-259.9	127.5	-259.9
FEM	-380	175	-150		FEM	-330	155	-330
	Transverse Middle Strip - E,F,G					Longitudinal Middle Strip - B,F,I		
	E	F	G			B	F	I
	First Support	First Span	"0" Support			Typical Support	Typical Span	Typical Support
EFA	-121.8	135.6	-		EFA	-86.7	104.4	-86.7
FEM	-105	108	50		FEM	-90	105	-90

	P1 - P4 - chosen							
	COLUMN STRIPS							
	Transverse Column Strip - A,B,C					Longitudinal Column Strip - A,E,H		
	A	**B**	**C**			**A**	**E**	**H**
	First Support	First Span	"0" Support			Typical Support	Typical Span	Typical Support
kNm	-437	201	-172		kNm	-480	232	-480
Strength	T 25 @ 100	T 20 @ 200	T 20 @ 200	Top	Strength	T 25 @ 100	T 20 @ 175	T 25 @ 100
Cracks	2 lyrs T20+T20 und.col.	T 20 @ 150	T 20 @ 175		Cracks	2 lyrs T20+T20 und.col.	T 20 @ 125	2 lyrs T20+T20 und.col.
	MIDDLE STRIPS							
	Transverse Middle Strip - E,F,G					Longitudinal Middle Strip - B,F,I		
	E	**F**	**G**			**B**	**F**	**I**
	First Support	First Span	"0" Support			Typical Support	Typical Span	Typical Support
kNm	-157	162	75	Top	kNm	135	157	135
Strength	T 16 @ 175	T 16 @ 150	T 16 @ 200		Strength	T 16 @ 200	T 16 @ 150	T 16 @ 200
Cracks	T 16 @ 125	T 16 @ 125	ok		Cracks	T 16 @ 150	T 16 @ 125	T 16 @ 150

Green = Input data from FEA (Robot). Yellow = Results.

Slab bending design

SSH DESIGN

Project No. 14065
SPECIAL CLINICAL CENTRE
Date: July 30 2015
By : DT

SLABS - BENDING MAIN REINFORCEMENT + CRACK WIDTH CHECK - AUXILIARY CALC SHEET

M_{sd} (kNm) = 157

M_{tmax} (kNm) = 812.8512

T 20 @ 100 = T 25 @ 150

ϕ (mm) =	16
spacing (mm) =	125
no. ϕ /m =	8
As (mm2) =	1608.5
T (kN) =	691.7
y (mm) =	38.1
d (mm) =	336.0
z (mm) =	320.7
Mrd (kNm) =	**221.8**
n =	15
b (mm) =	1000
b* (mm2) =	48.3
c* (mm2) =	16213.6
Δ* (mm) =	259.2
y_{II} (mm) =	105.5
I_{II} (mm4) =	1673310155
σ_{II}^{c} (N/mm2) =	6.4
σ_{II}^{S} (N/mm2) =	**210.9**

Transverse =	1.15
Longitudinal =	1.51
SLS / ULT =	0.65
w_lim (mm) =	0.3
fck (Mpa) =	40.0
fcd (Mpa) =	22.7
fyk (Mpa) =	460.0
fyd (Mpa) =	430.0
thk (mm) =	400.0
c (mm) =	40.0
B (mm) =	600
H (mm) =	600
be (mm) =	1200
δ_sls (mm) =	5
L / δ =	600

Coefficients to multiply FEM values with, to take into account patch loading (not modeled) - see next TABLE

Legend:

These colours contain stuff to be changed

Equivalent diameter bundles

20	**20**
314.1592654	314.1592654
628.3185307	
28	

Tables 7.2, 7.3 EC2, with w_k = 0.3	
σ_S (Mpa)	Max. ϕ (mm)
160	32
200	25
220	20
240	16
σ_S (Mpa)	Max. sp. (mm)
160	300
200	250
220	200
240	150

	DL	LL	LL/DL	Transv. (+%)	Longit. (+%)
Roof	13.25	7.5	0.566	20.9	74.2
P1 to P4	12.75	5	0.392	14.5	51.4
GF	19.95	5	0.251	9.3	32.8
B02, B01	11.1	2.5	0.225	8.3	29.5

Slab bending design

SSH DESIGN

Project No. 14065
SPECIAL CLINICAL CENTRE
Date: July 30 2015
By : DT

SLABS - PUNCHING SHEAR CHECK EC2 6.4

	Central Column	Edge Column	CornerColumn
N (kN) =	2638	1131	816
B (mm) =	600	600	600
H>B (mm) =	600	600	600
A_s^T (mm2/m) =	1795	2513	2513
A_s^L (mm2/m) =	1795	3141	1795
d (mm) =	330	330	330
u_0 (mm) =	2400.0	1200.0	600.0
u_1 (mm) =	6546.9	3273.5	1636.7
β =	1.15	1.4	1.5
v_{ed}^{0} (Mpa) =	3.83	4.00	6.18
v_{ed}^{1} (Mpa) =	**1.40**	**1.47**	**2.27**
ρ_T =	0.0045	0.0063	0.0063
ρ_L =	0.0045	0.0079	0.0045
ρ =	0.0045	0.0070	0.0053
$C_{rd,c}$ =	0.12	0.12	0.12
k =	1.78	1.78	1.78
v =	0.50	0.50	0.50
$v_{rd,max}$ (Mpa) =	5.712	5.712	5.712
$v_{rd,c}$ (Mpa) =	0.559	0.649	0.591
Studs ϕ (mm) =	12	12	12
Sr (mm) =	247.5	247.5	247.5
no. studs/row =	3	3	3
φ rows =	**30**	**30**	**30**
no. radial rows =	**12**	**7**	**4**
A_{sw} (mm2) =	4071.5	2375.0	1357.2
$f_{yw,ed}$ (Mpa) =	332.5	332.5	332.5
$v_{rd,cs}$ (Mpa) =	**1.672**	**1.949**	**2.114**

LEGEND

Ast and Asl refer to a full width column strip, also on the edge and corner columns!

HOLES

If there are holes near a column, the control perimeter must be reduced accordingly. The part of control perimeter contained between the tangents to the outline of the hole - drawn from the column centre - must be computed and taken out from u1. This can be computed by hand and subtracted from u1 of the selected type of column, one-off. This being done, the rest of the check keeps unvaried.

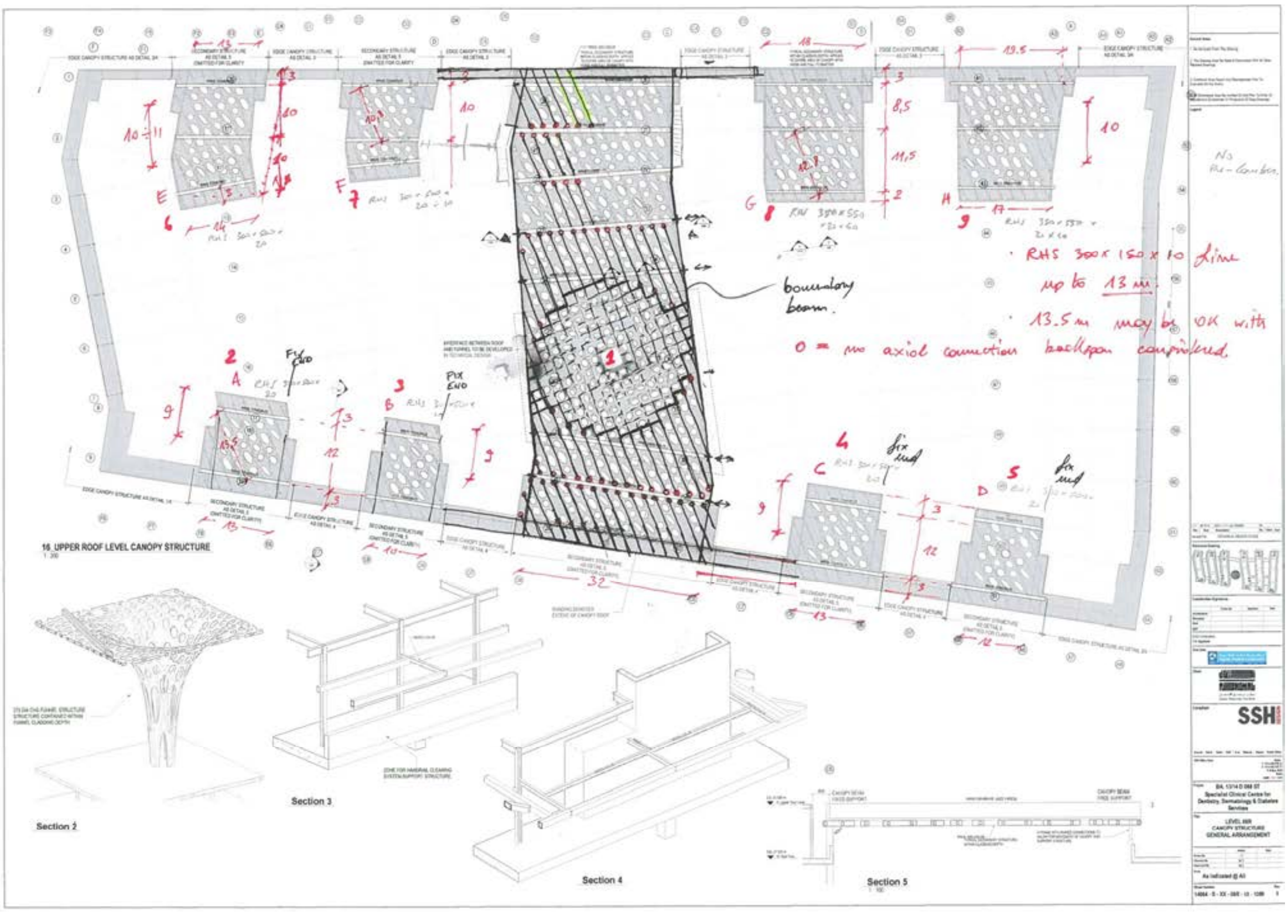
Green = Input data.
Yellow = Results.

Slab punching shear checks

3 *Detailed design for a long span steel canopy in Qatar*

All the buildings composing the Hamad Medical Centre are linked by a lightweight steel canopy with a non-trivial geometry.
I developed its design from the very early stage up to the technical design phase, in close collaboration with the architects.
The central part of the canopy is roughly 100 m long by 30 m wide and is supported by transversal fabricated box beams. At its center a vortex structure connects it to the ground, right in front of the main building entrances.
The whole geometry was modelled in Rhino. Automated spread-sheets were then necessary in order to carry out the structural checks, due to the considerable number of members and load combinations.

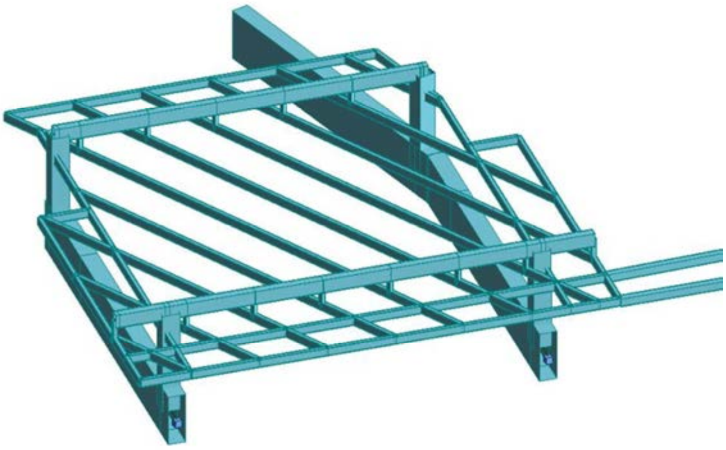
Canopy: overall plan view



Canopy: overall plan view

Main canopy and funnel: structural model in Robot

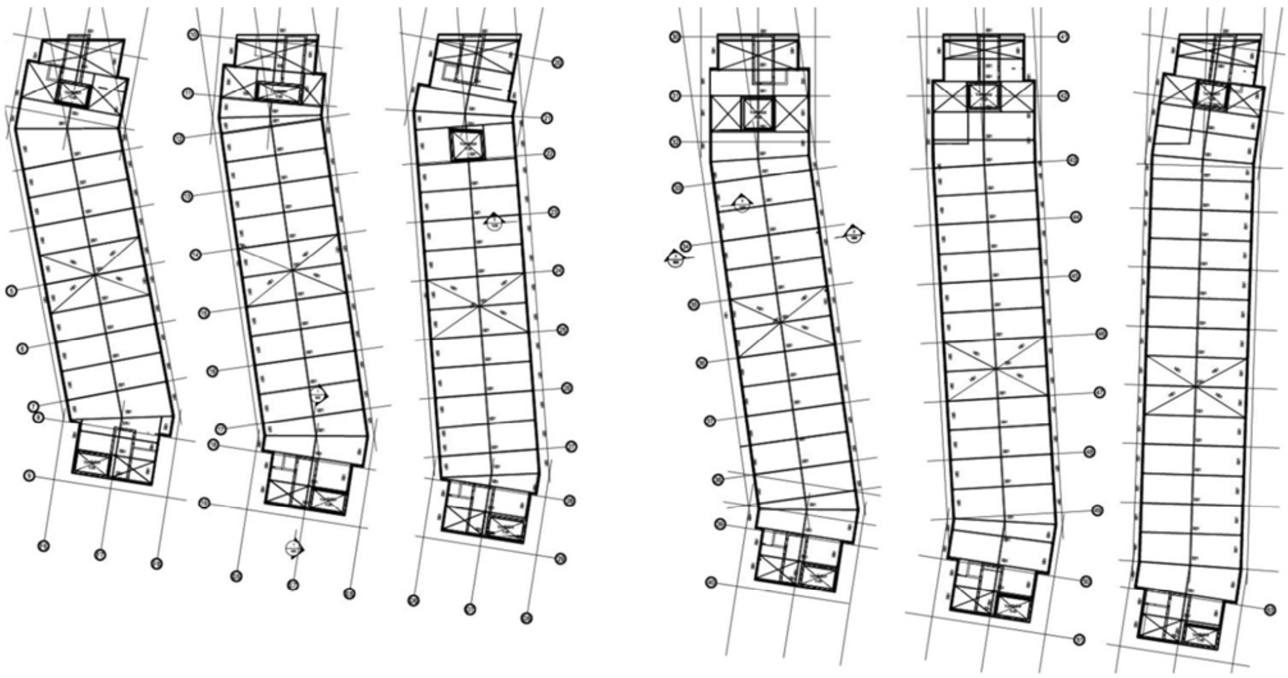
Main canopy and funnel: rendered view with GRP cladding



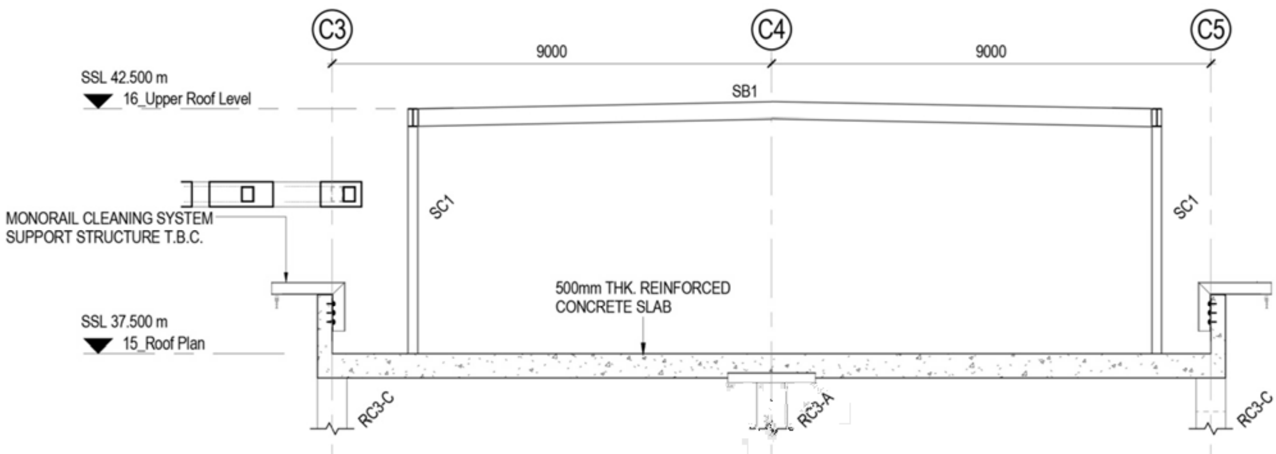
Secondary bits of canopy: close-up

4 *Detailed design for the steel enclosure roofs of 6 RC multi-storey buildings in Qatar*

Each of the roof slabs of the buildings composing the Hamad Medical Centre hosts essential MEP machinery and hence needs a bespoke enclosure roof.
I designed lightweight steel frames to support them. Although similar to each other, separate design had to be carried out because of cutbacks and recesses that made each of them unique. These cutbacks were required by the architect and led to the introduction of a number of additional bracings at the ends of the roof structures.



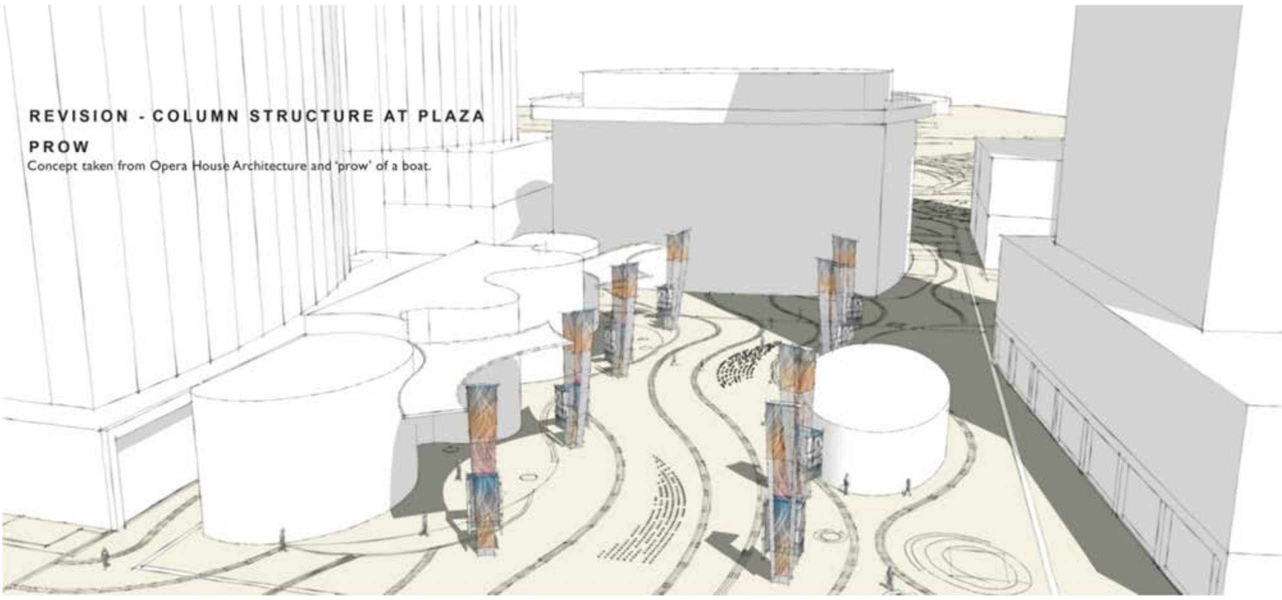
Medical Centre enclosure roofs – overall view



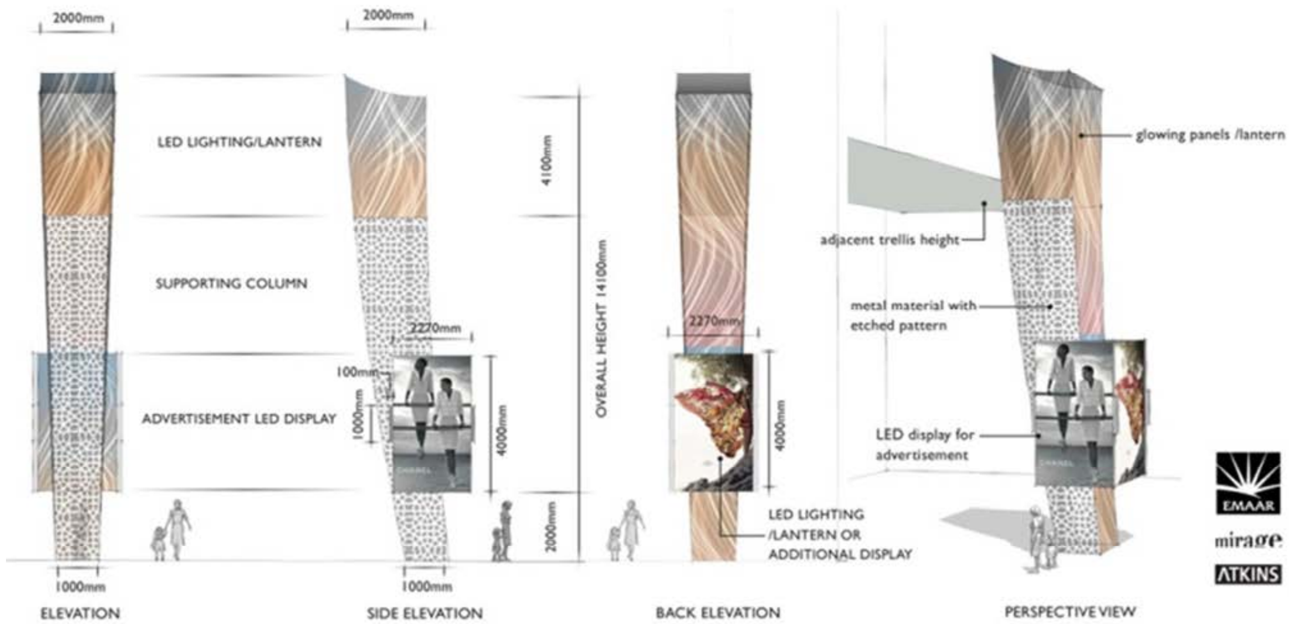
Enclosure roof - typical frame

5 *Detailed design of a 15 m tall totem structure in the Opera House area, Dubai*

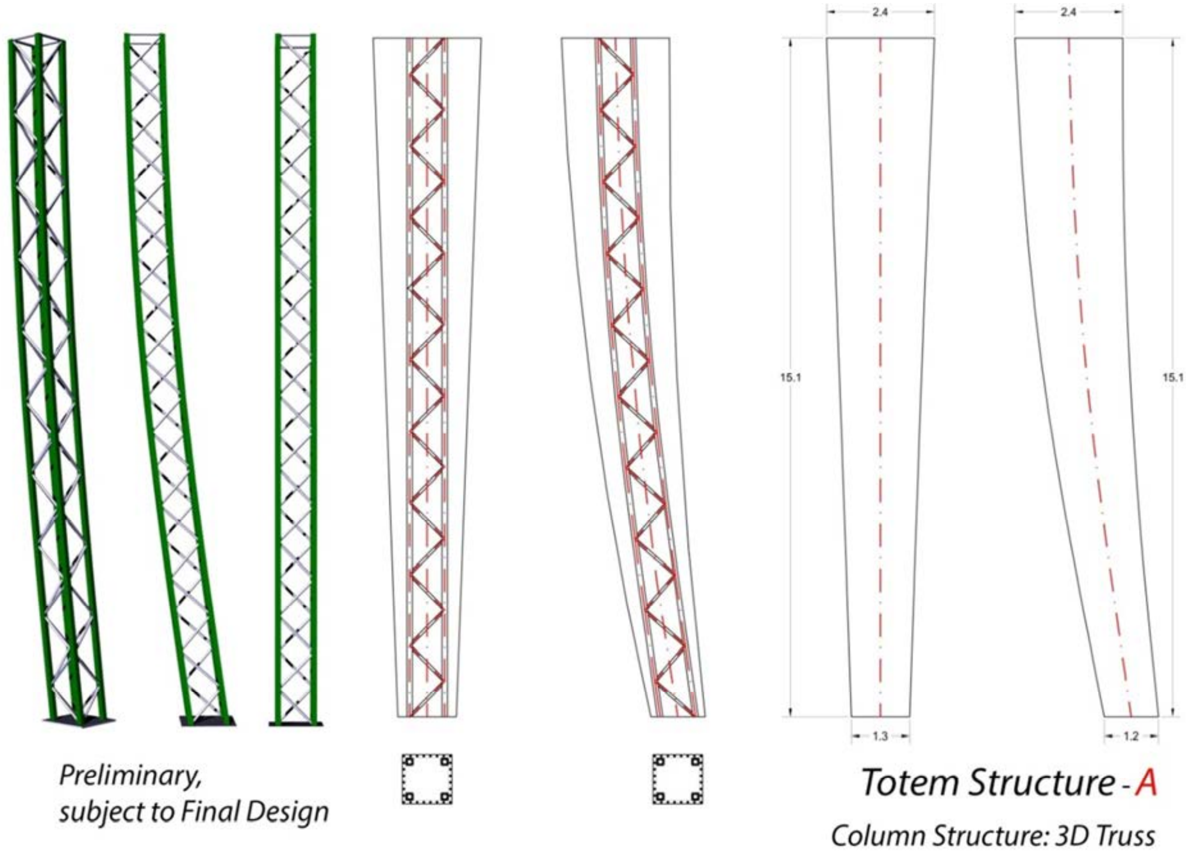
The project of the Opera House in Dubai envisages a set of 8 "totems" within the plaza adjacent the main Opera House building.
These totems are intended to be lightweight steel columns endowed with secondary frames devised to connect provisional, interchangeable installations and posters.
As the figures below show, the totems are roughly 15 m tall and 1 m wide at the base, whereas their centre line is curved.
I developed three scheme options for their structure, one of which is reported below. Additionally, I handed out the holding down bolts arrangement for construction.



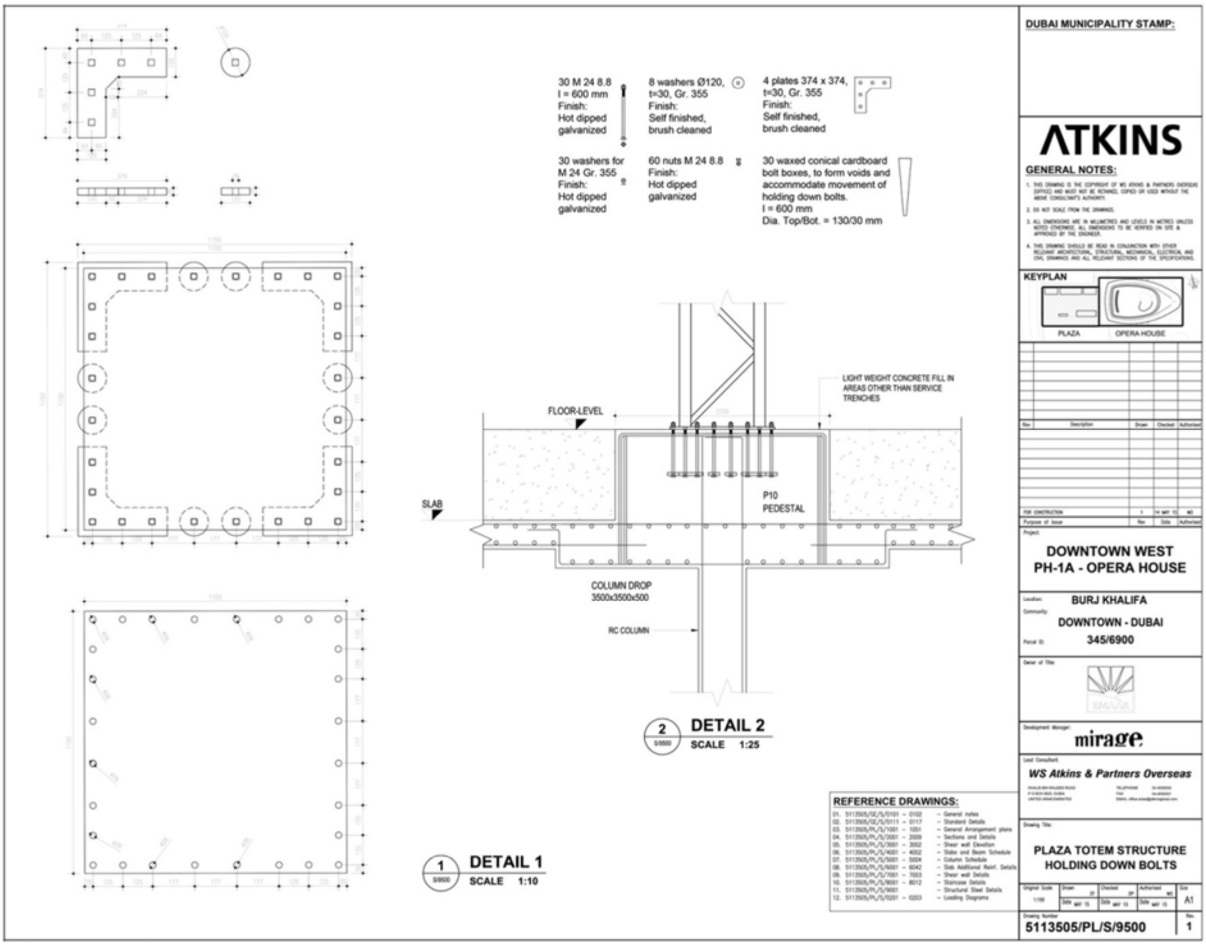
Totems location within the Opera House Plaza



Totems architectural design intent



Totems structure: option A, preliminary design



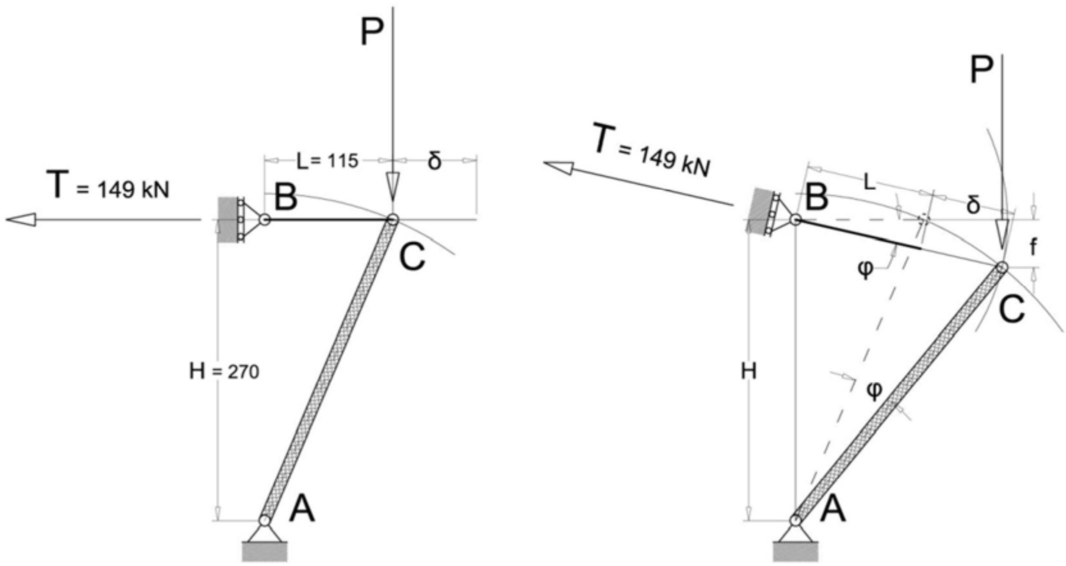
Holding down bolts: arrangement issued for construction

6 *Technical review of the remedial works for a multi-storey car park in Oman*

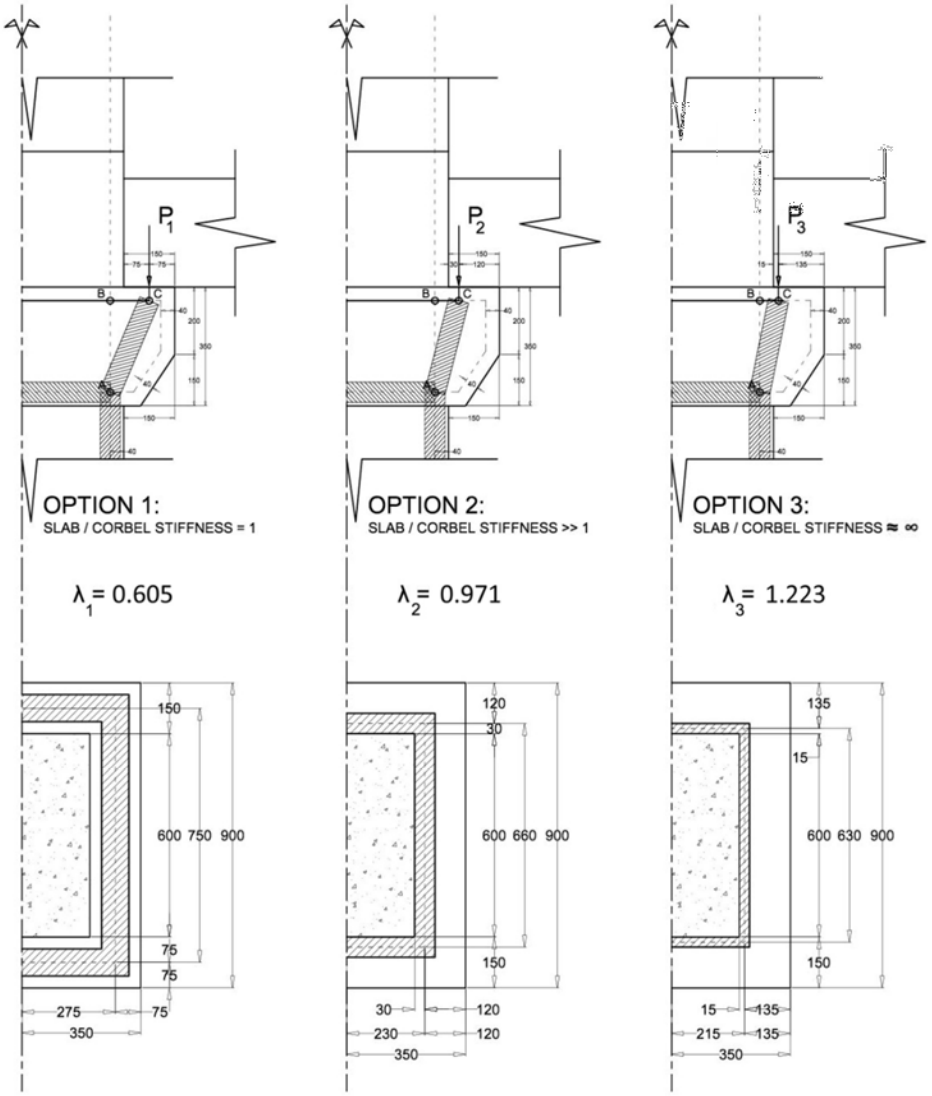
I produced a number of technical reports over a period of 8 months regarding the technical review of remedial works for a multi-storey concrete car park in Oman.
As shown in the figure below, the starter bars in the P2 columns were not properly anchored. As a consequence the construction was stopped at P2 level and a third party consultant was appointed to devise remedial works. They decided to demolish the P2 columns, their footprint within the PT slab as well as the P1 column heads. Then a corbel was built on the slab underside to carry the storey shear and the columns were re-cast with appropriate reinforcement.
OPS Ltd. were appointed for the technical review of the aforementioned remedial works. I carried out several checks according to various codes (ACI 318M-11, EC2, BS 8110) as well as by applying the plasticity theorems, in order to prove that the reinforcement provided in the corbel was not sufficient to provide the required factor of safety.
OPS Ltd. and the third party consultant could not reach an agreement and the consultant decided to take full responsibility over it.

Wrong anchoring of P2 starter bars

Corbel reinforcement



Application of the plasticity theorems in order to work out the corbel ULS load factor



Parametric analysis on the relationship between slab-corbel relative stiffness and corbel load bearing capacity

7 *Scheme design for a multi-storey car park in Qatar*

Halfway in the design process of the Hamad Medical Centre in Qatar, the client decided to build a multi-storey car park in a plot next to the buildings site, in order to increase the parking capacity already provided by the basement car-park.
As a peculiarity, because the space near the site was limited the car park had to overhang an existing MEP building.
I carried out the scheme design, providing three general options involving RC, PT and PC slabs, respectively. I also worked out four alternative options for the area overhanging the existing building, namely 21 x 25 m in plan, where the columns could only be located along the perimeter. Among the options there were either a steel girder or a truss at P2 level, or composite beams at all floors, all of them being 9 m spaced apart.
The design of the composite beams proved to be very challenging as the long span (21 m) and the high load and beams spacing stretched the design to its limit. Therefore, in order to fine tune the beams design, I set up bespoke spread-sheets.

OPS
STRUCTURAL ENGINEERING

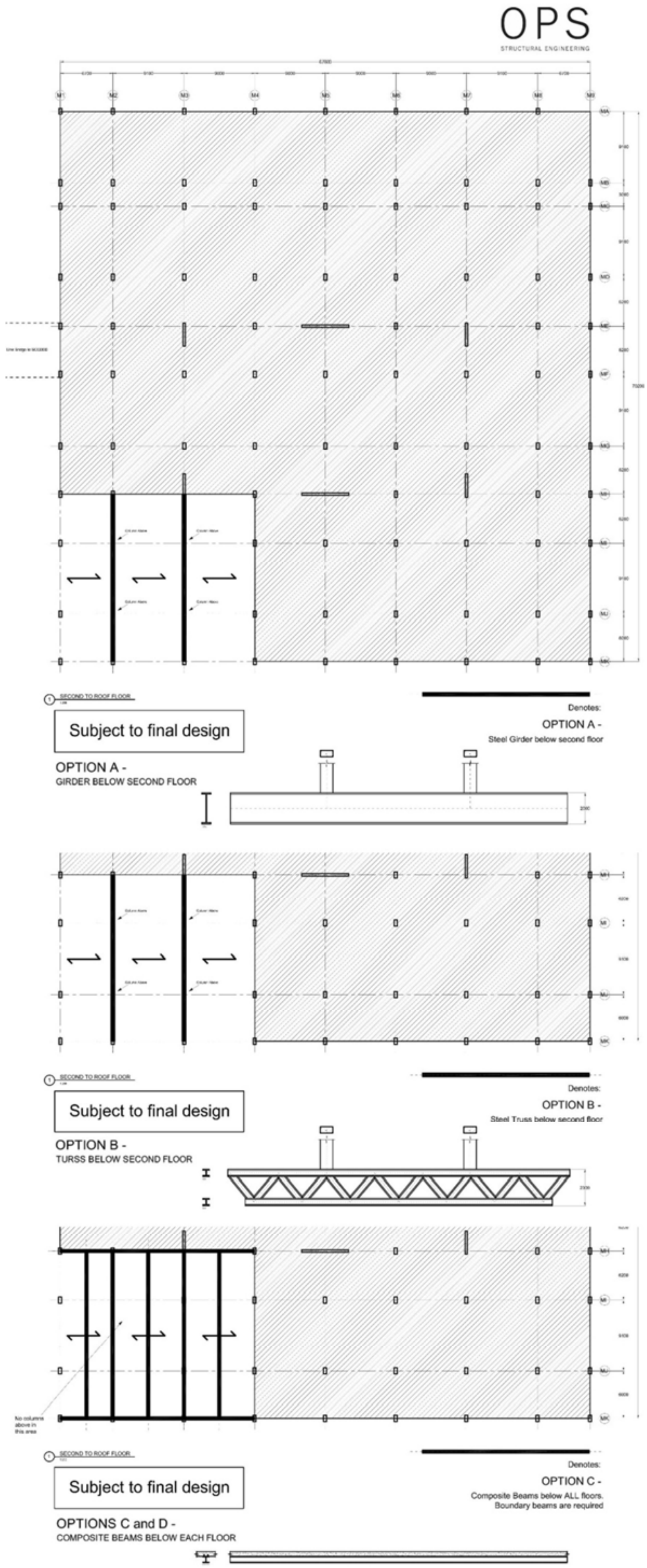
HAMAD EXTERNAL CAR PARK, PRELIMINARY DESIGN RESULTS - OVERALL CAR PARK									
	Option 1			Option 2			Option 3		
	2-ways RC flat slab			2-ways PT flat slab			1-way Pre Cast slab		
Storey	B_{COL} (mm)	H_{COL} (mm)	Slab Thk (mm)	B_{COL} (mm)	H_{COL} (mm)	Slab Thk (mm)	B_{COL} (mm)	H_{COL} (mm)	Slab Thk (mm)
6	400	400	300	400	400	225	400	400	250
5	400	400	300	400	400	225	400	400	250
4	400	500	300	400	450	225	400	450	250
3	400	650	300	400	600	225	400	600	250
2	400	800	300	400	750	225	400	750	250
1	400	950	300	400	900	225	400	900	250
0	400	1100	300	400	1050	225	400	1050	250
							This option needs a 1-way **downstand beam**: 600 x 750 mm		

HAMAD EXTERNAL CAR PARK, PRELIMINARY DESIGN RESULTS - BIT OVERHANGING THE EXISTING BUILDING									
TRANSFER STRUCTURE OPTIONS									
Transfer structures are necessary to make up for the lack of support in the south-east corner of the car-park, where it overhangs over an existing building.									
	OPTION	H (mm)	Spacing (mm)	w_1 * (kg/m^2)	W_{TOT} * (ton)	Beams at each floor	Columns above 2nd floor	Boundary beams	Additional Columns
A	GIRDERS below second floor	2000	9100	250	100	NO	YES	NO	NO
B	TRUSSES below second floor	2300	9100	250	100	NO	YES	NO	NO
C	COMPOSITE beams at each floor - 1	450	4550	760	265	YES	NO	YES	NO
D	COMPOSITE beams at each floor - 2	450	4550	450	220	YES	NO	NO	YES

* w_1 is the overall weight of **steel** transfer structures, summed up at all levels in cases of transfers at each floor such as options C and D, per square meter of car-park footprint.
* W_{TOT} is the overall tonnage of steel transfer structures

All COLUMNS below second floor to be 600 x 800 mm.

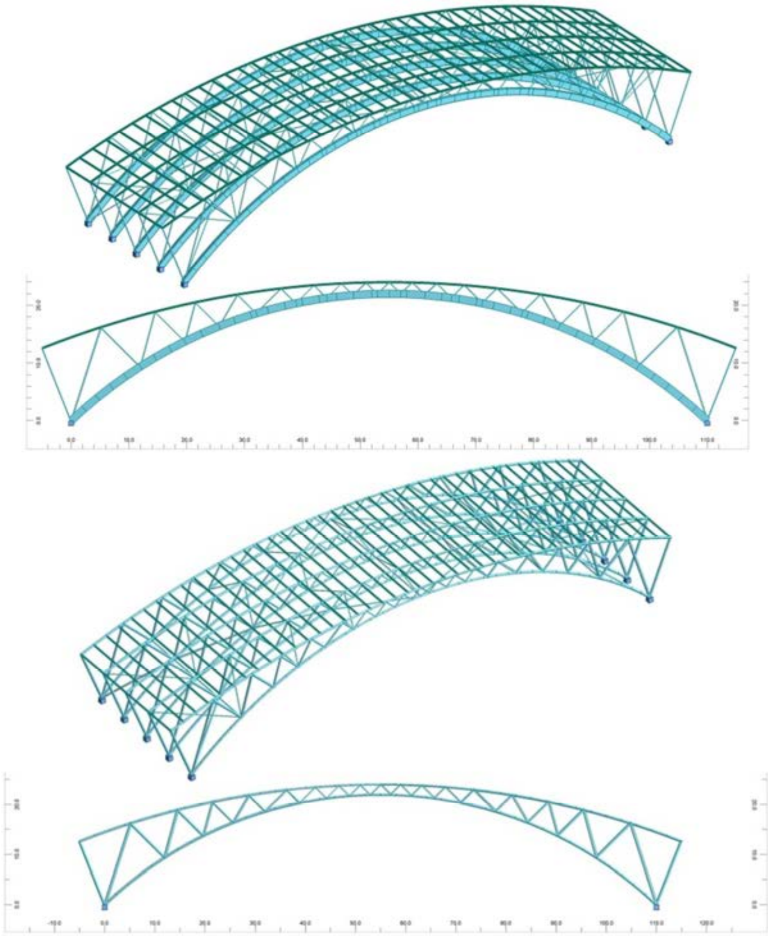
Summary of all the proposed options: RC, PT and PC for the slab, together with different steel beams to support the area overhanging the existing building



Three different options for supporting the area overhanging the existing building

8 *Scheme design for Riyadh Western Metro Station*

OPS Ltd. were appointed for the concept design of the Riyadh western metro station.
The architectural project involved two long span roofs, the longest of which being in excess of 110 m.
I produced two options involving a two hinged arch with fabricated box section and a trussed two hinged arch. The latter turned out to be more effective as the secondary members necessary to support the roof (as per architectural design) could be effectively used as truss diagonals, thus reducing the overall structural mass.



Two options to cover the station 110 m span

RIYADH METRO WESTERN STATION CANOPY - FEASIBILITY STUDY

BOXED ARCH - 1				
λ	δ (mm)	f_1 (Hz)	W_{TOT} (ton)	W_1 (kg/m^2)
collapse factor	max. serv. displ.	first fund. freq.	Tonnage	Unitary weight
3.5	50	1.2	817	240

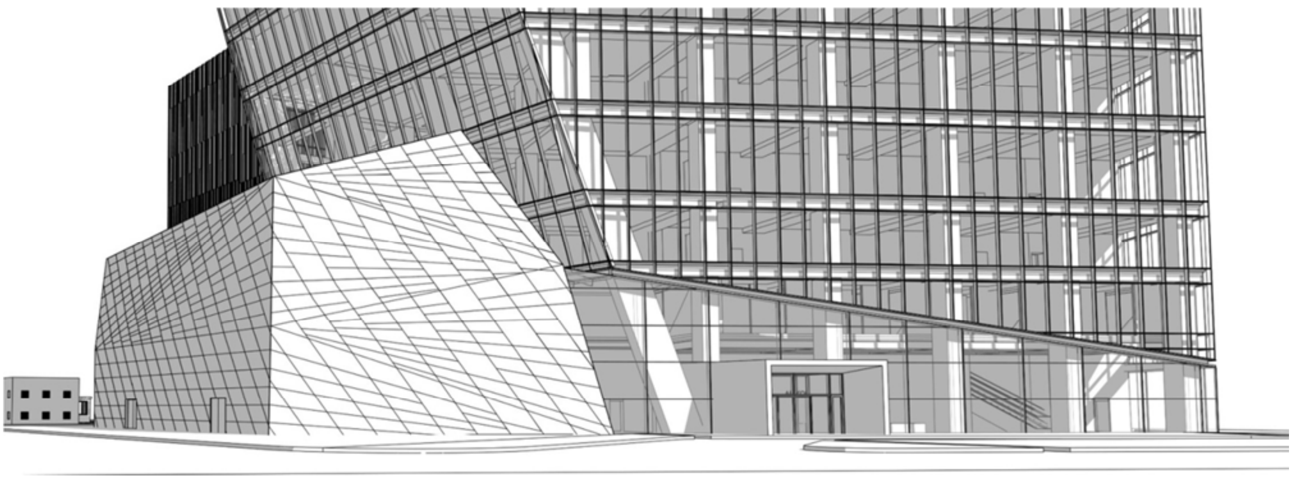
TRUSSED ARCH - 2				
λ	δ (mm)	f_1 (Hz)	W_{TOT} (ton)	W_1 (kg/m^2)
collapse factor	max. serv. displ.	first fund. freq.	Tonnage	Unitary weight
4.6	50	1.5	486	140

ARCHES GEOMETRICAL DATA		LOADING	
L (m) =	110	G1 =	self-weight
h (m) =	22	G2 (kN/m^2) =	1.25
spacing (m) =	8	Sand (kN/m^2) =	0.6
		Wind (kN/m^2) =	-0.8

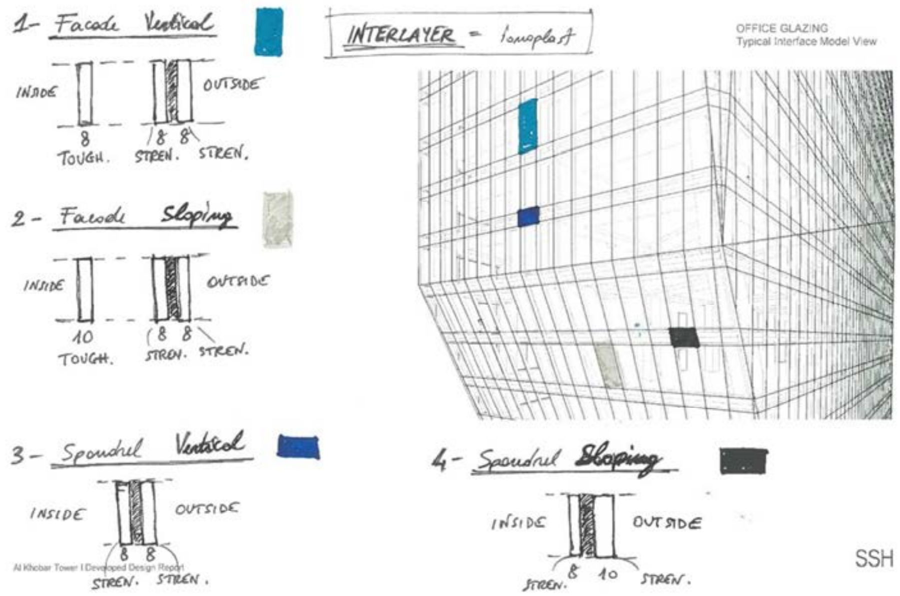
Results summary and cost comparison of the two options

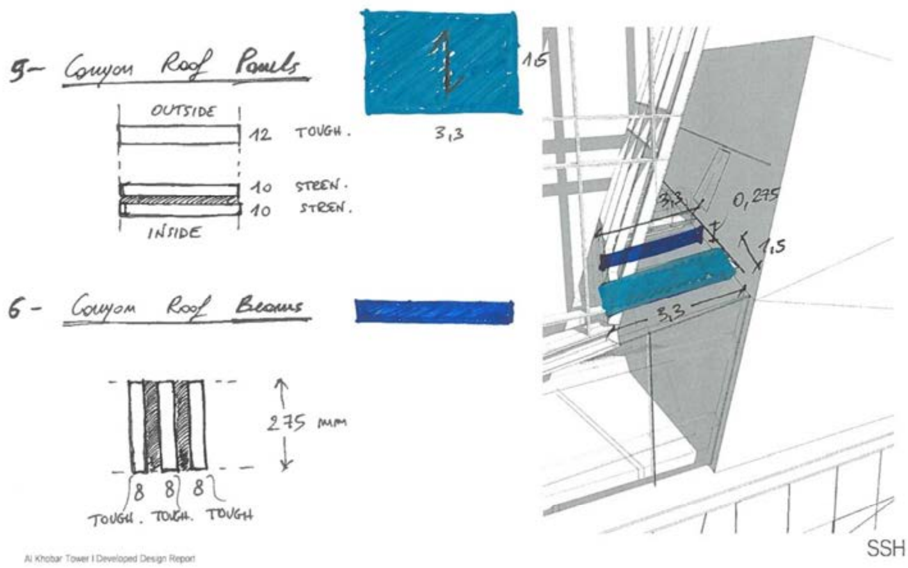
9 *Detailed design for the façade of a 150 m tall tower in Saudi Arabia*

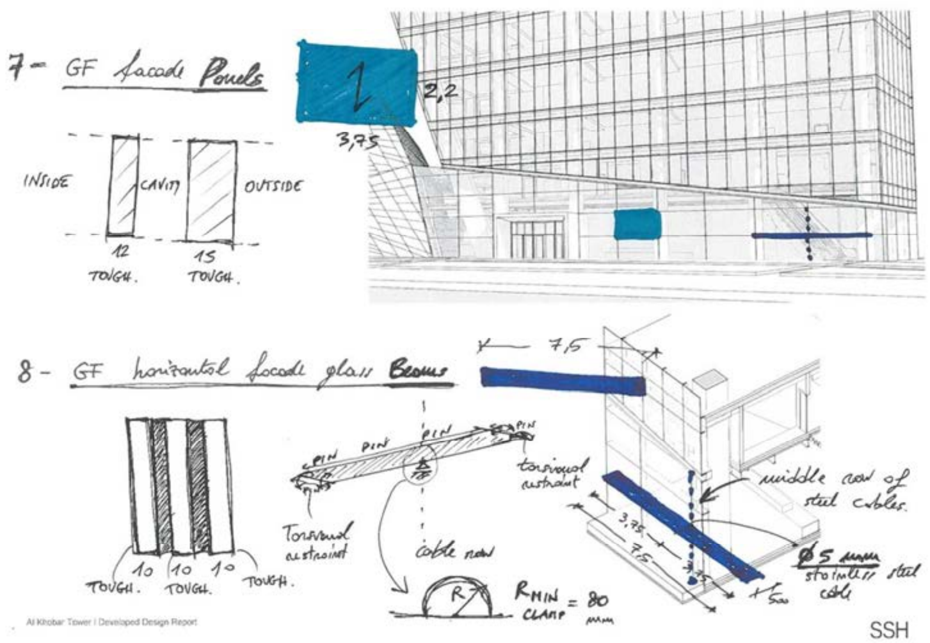
OPS Ltd. were appointed for the design of a tower in Saudi Arabia, 150 m tall.
I collaborated with a senior façade engineer to develop the tower façade from scheme to detailed design.
First thing the façade system was selected, with vertical mullions 1.5 m spaced apart and 1-way spanning glazings. Then I sized all the glass elements according to the publication "Structural use of glass in buildings" by the Institution of Structural Engineers and the EC code draft prEN 13474-2012. The elements involved IGUs, spandrel laminated panels, vertical and horizontal beams. For each of them I set up a tailored spread-sheet.
Then I schemed and designed all of the connections between glass elements and building structure (see figures below).
Eventually, as the back and roof of the tower must accommodate PV panels, I designed a modular frame 6 x 3 m wide to support them. I also selected the connection devices to attach the PV panels to the frame, in such a way that the whole frame is assembled in plant and then hauled on site, ready to be fastened onto the building structure (see last figure).



Tower architectural project and façade close-up







Summary of results of façade glass elements scheming

OPS

Project: Al Khobar Tower
Structure: Canyon roof IGUs
Calc by: DT, **Date:** 25/11/2015

Job ref: 15101
Checked by: DP

MATERIAL DESIGN STRENGTH

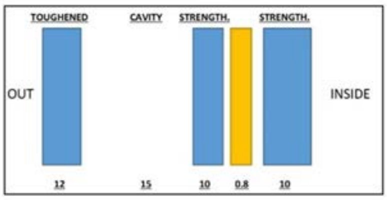
$$f_{g,d} = \frac{k_{mod} k_{sp} f_{g,k}}{\gamma_{Ma}} + \frac{k_v (f_{b,k} - f_{g,k})}{\gamma_{Mv}}$$

k_{mod} =	0.29	50 years dead load		
	0.44	sand load mid term		
	0.74	10 minutes multiple gusts (storm)		
k_{sp} =	1	factor for glass surface profile		
$f_{g,k}$ (N/mm^2) =	45	annealed glass		
$f_{b,k}$ (N/mm^2) =	70	heat strengthened glass		
$f_{b,k}$ (N/mm^2) =	120	thermally thoughened glass		
γ_{Ma} =	1.6	annealed glass partial factor		
γ_{Mv} =	1.2	strengthened partial factor		
	DEAD	MAINTEN.	WIND	
$f_{g,d}$ (N/mm^2) =	29.0	33.2	41.6	strengthened
$f_{g,d}$ (N/mm^2) =	70.7	74.9	83.3	toughened
ω_{DEAD} =	0.00	dead load shear coeff		
$\omega_{MAINTENANCE}$ =	0.20	sand load mid term shear coeff		
ω_{WIND} =	0.60	wind load shear coeff		
$\omega_{PRESSURE}$ =	0.10	pressure load shear coeff		

WIND AND POINT LOADING

p_{DEAD} (kN/m^2) =	**0.81**	dead load
p_{MAINT} (kN/m^2) =	**1.50**	maintenance load
p_{WIND} (kN/m^2) =	**2.50**	wind load
P_{POINT} (kN) =	3.00	point load
r_{eq} (mm) =	28	Equivalent circ. Footprint

FLOOR PANEL GEOMETRY - 1-way spanning



Panels geometry		*Panels components*	
L (m) =	3.3	Single Pane (mm) =	12
W (m) =	1.5	Cavity (mm) =	15
Span (m) =	1.5	Lam upp pane (mm) =	10
		Interlayer (mm) =	0.76
		Lam bott pane (mm) =	10
		IGU H_{TOT} (mm) =	32.76
		Lam H_{TOT} (mm) =	20.76
		$h_{m,UPPER}$ (mm) =	5.38
		$h_{m,BOTTOM}$ (mm) =	5.38
		Post-failure pane (mm) =	10

EFFECTIVE THICKNESS FOR DEFLECTIONS

$$h_{ef,w} = \sqrt[3]{\sum_k h_k^3 + 12\,\omega \sum_i h_i h_{m,i}^2}$$

	DEAD	MAINT	WIND	PRESSURE
$h_{ef,w}$ (mm) =	12.6	15.0	18.3	13.9

EFFECTIVE THICKNESS FOR STRESSES under DEAD LOAD - MINOR AXIS

$$h_{ef,\sigma} = \sqrt{\frac{h_{ef,w}^3}{h_j + 2\,\omega\, h_{m,j}}}$$

you get different stiffnesses for panes with different thicknesses

	DEAD	MAINT	WIND	PRESSURE
$h_{ef,\sigma\ UPPER}$ (mm) =	14.1	16.7	19.4	15.6
$h_{ef,\sigma\ BOTTOM}$ (mm) =	14.1	16.7	19.4	15.6

GEOMETRICAL PROPERTIES FOR DEFLECTIONS

	DEAD	MAINT	WIND	PRESSURE
I_{SINGLE} (mm^4) =	144,000	144,000	144,000	144,000
I_{LAM} (mm^4) =	166,667	282,444	513,999	224,555

IGU LOAD SHARING

	DEAD	MAINT	WIND	PRESSURE
Single load share =	0.366	1.000	0.300	0.391
Lamin load share =	0.634	0.000	0.781	0.609

LOAD COMBINATIONS

$$ULS = \gamma_G G + \gamma_Q Q_{k1} + \gamma_Q \psi_0 Q_{ki} \qquad \text{eq.n (6.10) EC1}$$

$$SLS = G + \psi_{1,1} Q_{k1} + \psi_{2,1} Q_{ki} \qquad \text{eq.n (6.14a) EC1}$$

Table A1.1 - accidental loads on buildings' roofs

ψ_0 =	0.7
ψ_1 =	0.5
ψ_2 =	0.2

Table A1.2(B) - design values of actions, STR/GEO approach

γ_G =	1.35
γ_Q =	1.5

$$ULS = 1.35\ Dead + 1.5\ Wind + 1.5 * 0.7 * Maintenance$$

$$SLS = 1.0\ Dead + 0.5\ Wind + 0.2\ Maintenance$$

BENDING MOMENTS

	DEAD	MAINT	WIND	PRESSURE	TOT
ULS (kN/m^2) =	1.09	1.58	3.75	0.19	6.60
SLS (kN/m^2) =	0.81	0.30	1.25	0.14	2.50
M_{ULS} (kNm) =	0.31	0.44	1.05	0.05	1.86
$M_{POST\text{-}FAIL}$ (kNm) =	0.23	0.42	0.70	0.04	1.39

GEOMETRICAL PROPERTIES FOR STRESSES

	DEAD	MAINT	WIND	PRESSURE
Z_{SINGLE} (mm^3) =	24,000	24,000	24,000	24,000
$Z_{LAM\ UPPER}$ (mm^3) =	33,333	46,485	62,470	40,548
$Z_{LAM\ BOTTOM}$ (mm^3) =	33,333	46,485	62,470	40,548

PANELS LOADS due to VARIATIONS of P

d_{SINGLE} / d_{LAM} =	1.56	See IGU design.xlsx
d_{LAM} (mm) =	0.59	for further reference
d_{SINGLE} (mm) =	0.91	
$q_{LAM\text{-}EQ}$ (kN/m^2) =	0.140	
$q_{SINGLE\text{-}EQ}$ (kN/m^2) =	0.140	

DEFLECTIONS

	DEAD	MAINT	WIND	PRESS	TOT
δ_{SINGLE} (mm) =	1.9	2.0	2.5	0.9	*7.3*
δ_{LAM} (mm) =	2.9	0.0	1.8	0.6	*5.3*
L (mm) =	1500				
L / 250 =	6				

STRESSES on INTACT IGU

	DEAD	MAINT	WIND	PRESSURE	TOT
σ_{SINGLE} (Mpa) =	4.7	18.5	13.2	2.2	
Strenghtened	0.161	0.556	0.317	0.067	*1.100*
Toughened	0.066	0.247	0.158	0.030	*0.500*
σ_{UPPER} (Mpa) =	5.8	0.0	13.2	1.3	
Strenghtened	0.201	0	0.317	0.039	*0.557*
Toughened	0.082	0	0.158	0.017	*0.258*
σ_{BOTTOM} (Mpa) =	5.8	0.0	13.2	1.3	
Strenghtened	0.201	0	0.317	0.039	*0.557*
Toughened	0.082	0	0.158	0.017	*0.258*

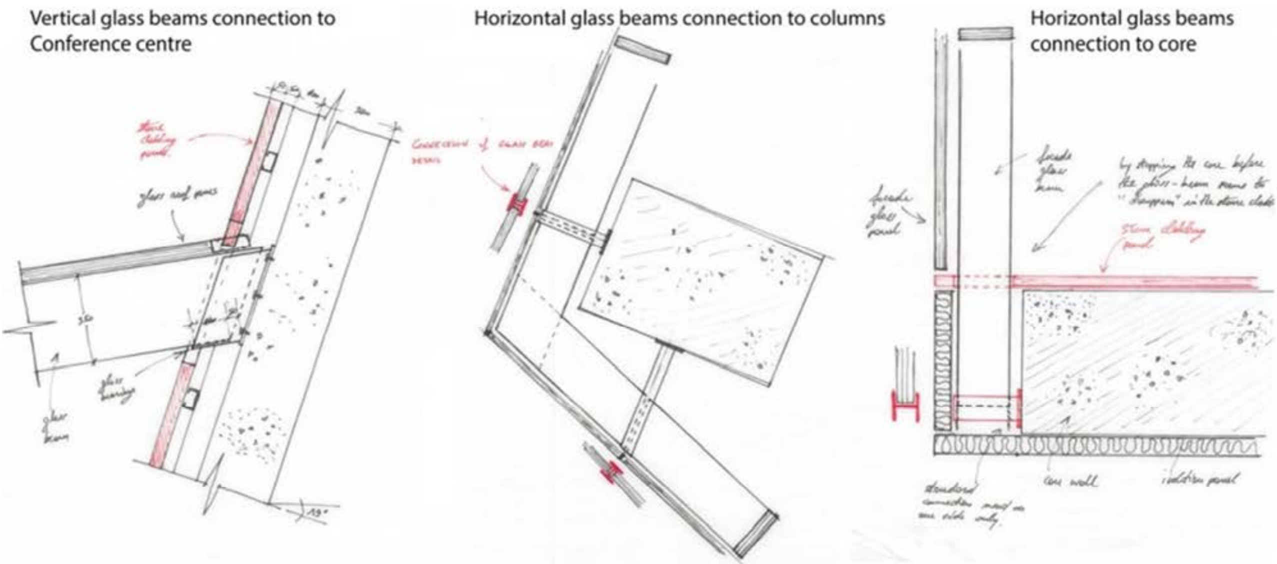
Point load on single pane

Roarke's ed.6, eqn 8 Tab 26 chap 10

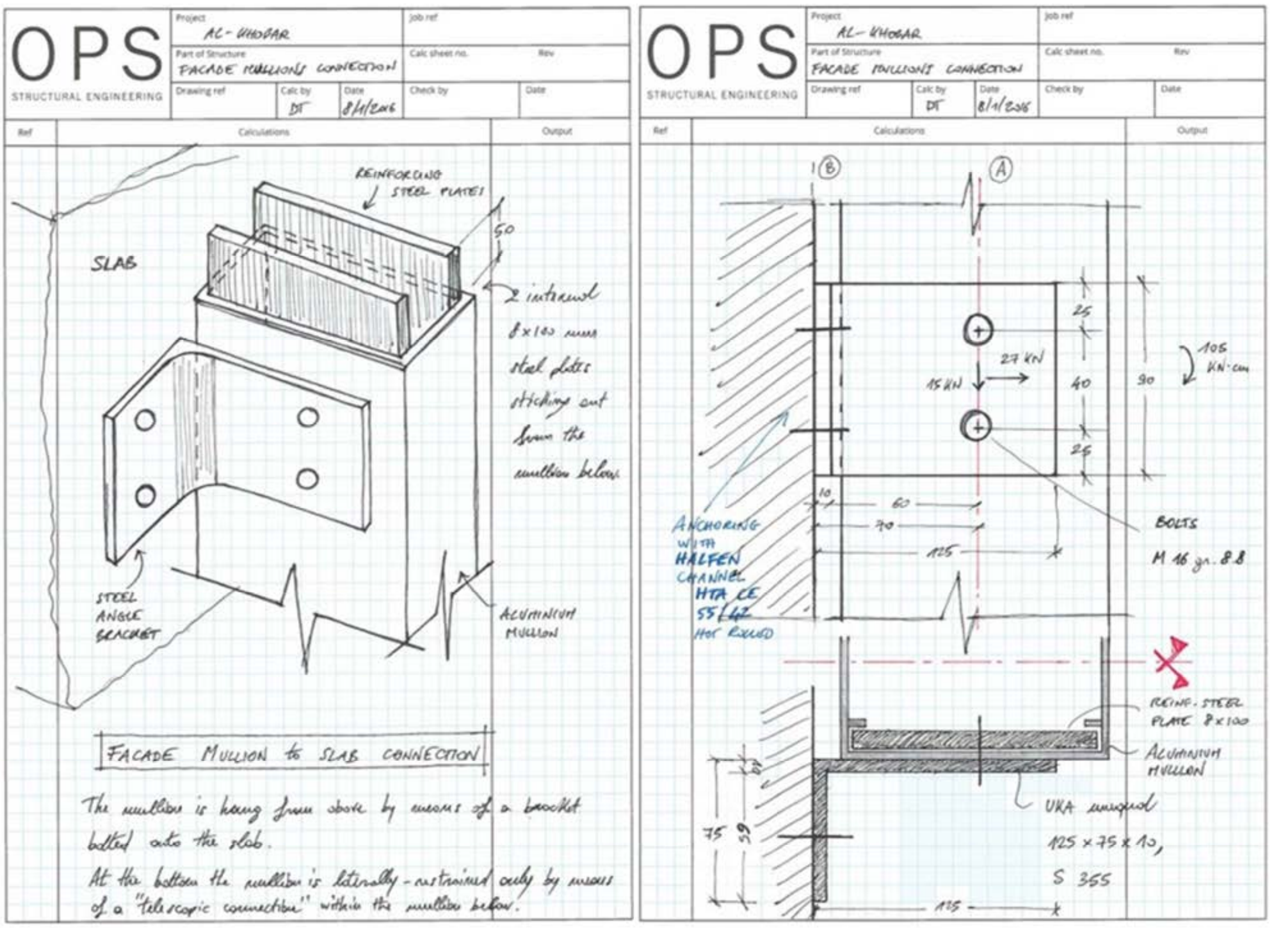
$$\sigma = \frac{3P}{2\pi t^2}\left[(1+\upsilon)\ \ln\frac{2a}{\pi r_{eq}} + \beta\right] \qquad \beta = \quad 0.068$$

	DEAD	POINT	PRESSURE	
σ_{SINGLE} (Mpa) =	4.7	65.3	2.2	
Strenghtened	0.161	1.56707	0.067	*1.795*
Toughened	0.066	0.78334	0.030	*0.879*

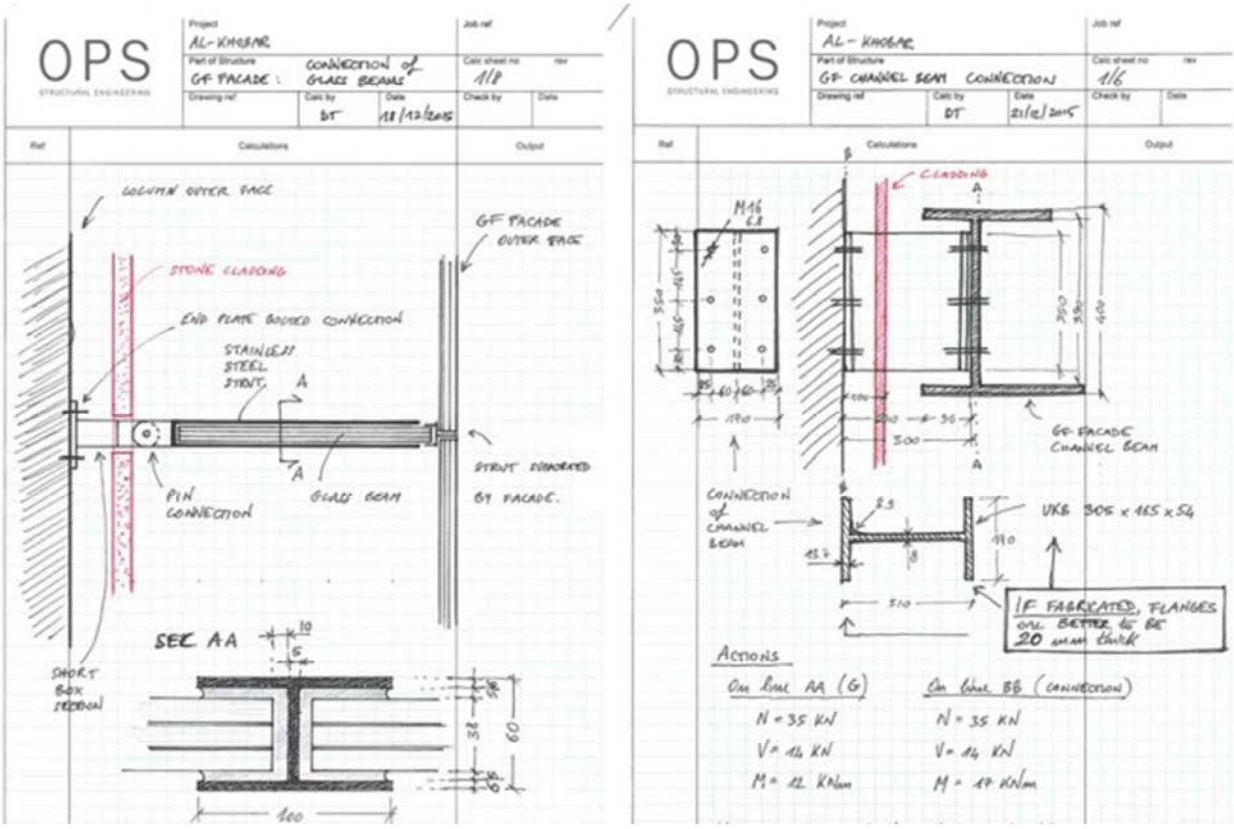
IGU tailored design spread-sheet excerpt



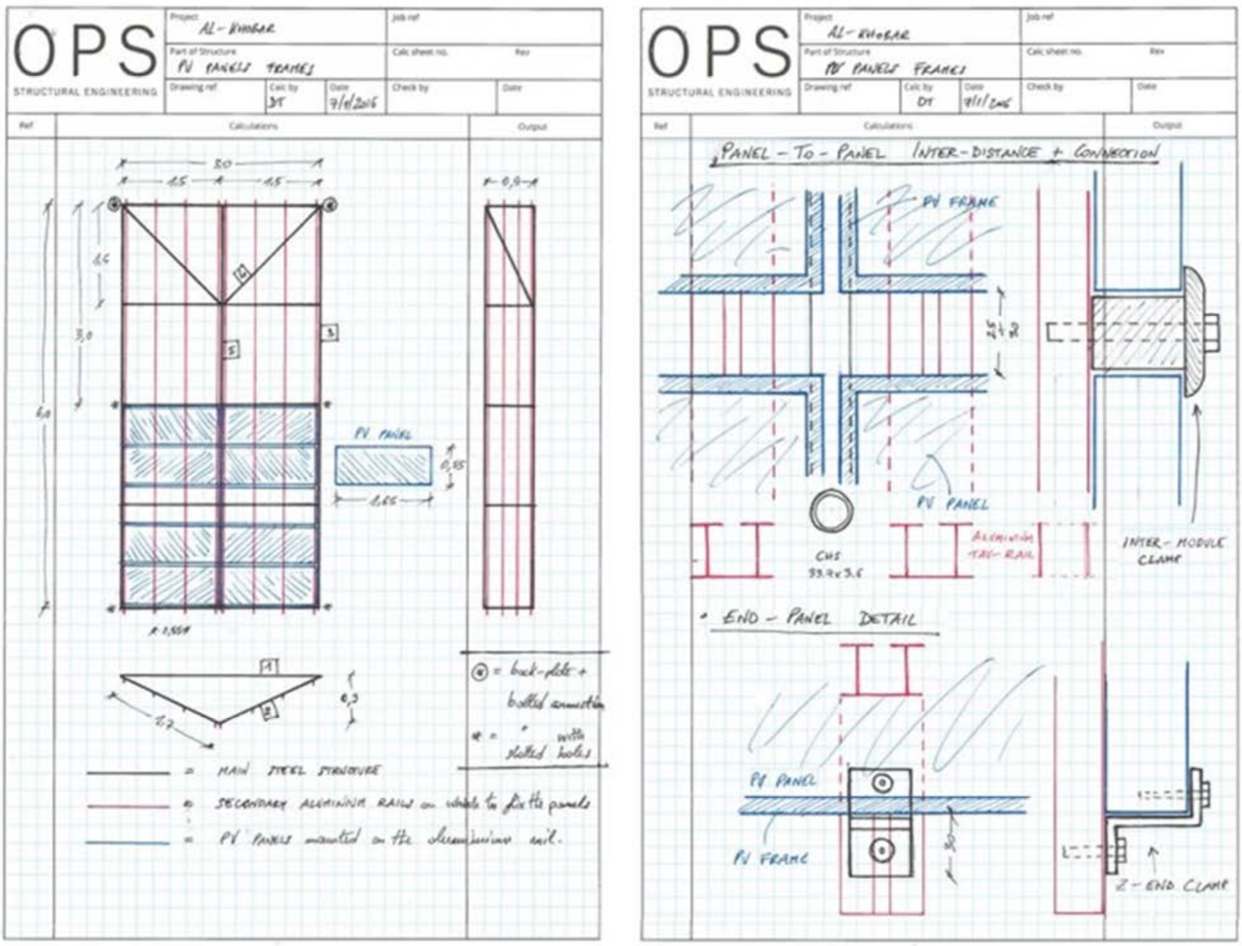
Sketches of glass beams to concrete connection details



Excerpt from design calc-sheets for the façade mullions connection



Excerpt from design calc-sheets for façade elements connections



Excerpt from design calc-sheets for PV panels frames

10 *Checks and remedial works for a complex titanium cladding in Iraq*

A new Cultural Centre has recently been built in Iraq. The complex is made up of four buildings with a faceted skin, covered by a very elaborate cladding.

At a very late stage in the construction process the cladding system adopted proved to be unsuitable for the purpose. In particular both the structural decking and the cladding panels appeared to be too flexible and the connection device between them badly conceived. Additionally, the overall quality of the cladding work installed looked poor.

OPS Ltd. were appointed to investigate the root causes of the poor performances of the cladding and to find solutions. I was entrusted with the task.

I started by identifying the main reasons of concern: i.e. the capacity of the metal decking, the pedestal screwed connection onto the deck, the stiffness of the cladding panels, the glued connection between cladding panels and stiffeners, the stiffeners position and the quality of the workmanship.

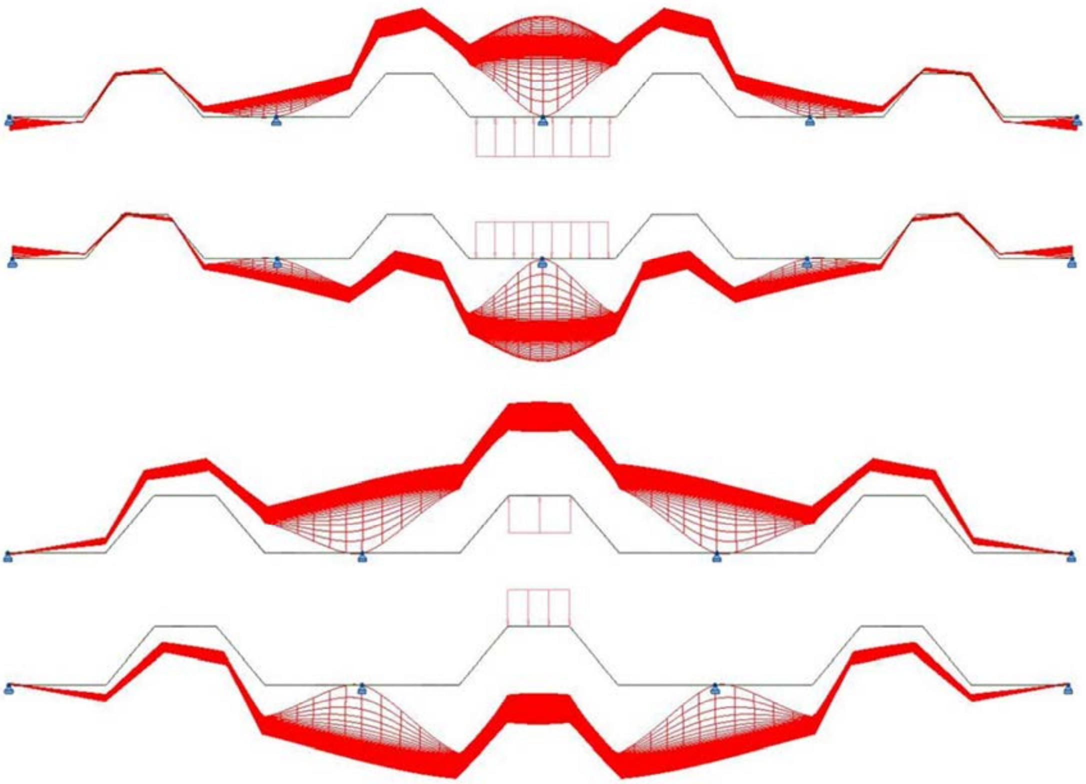
Regarding the metal decking, I first worked out its effective width under point load from the pedestal (see figure below). Then I computed its effective section under pressure and suction loads according to EC 1993. Eventually I carried out numerical buckling tests with the constrained finite element method and devised physical testing to validate the results.

This is an extensive work and it is still ongoing.

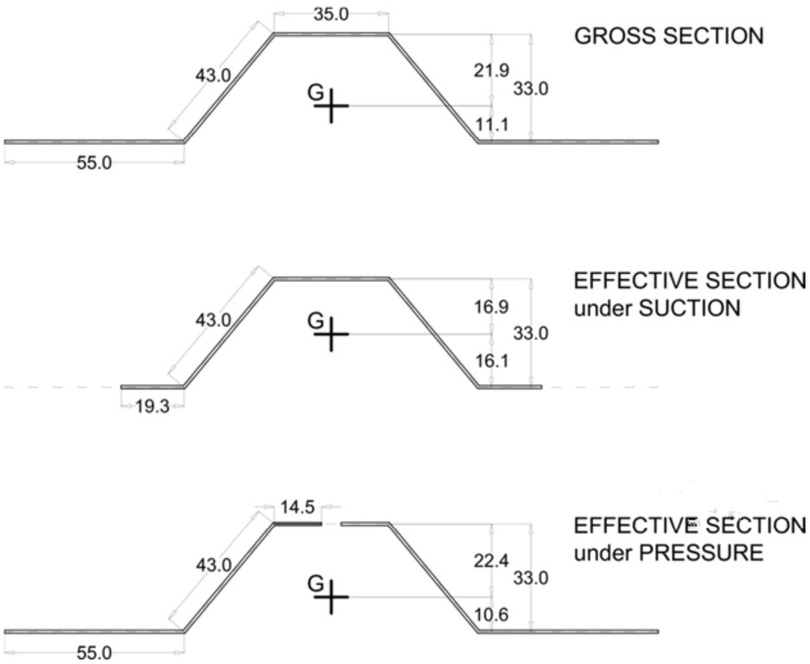
Cultural Centre architectural project and cladding close-up

Cassette bracket fixings

Pedestal to metal decking connection

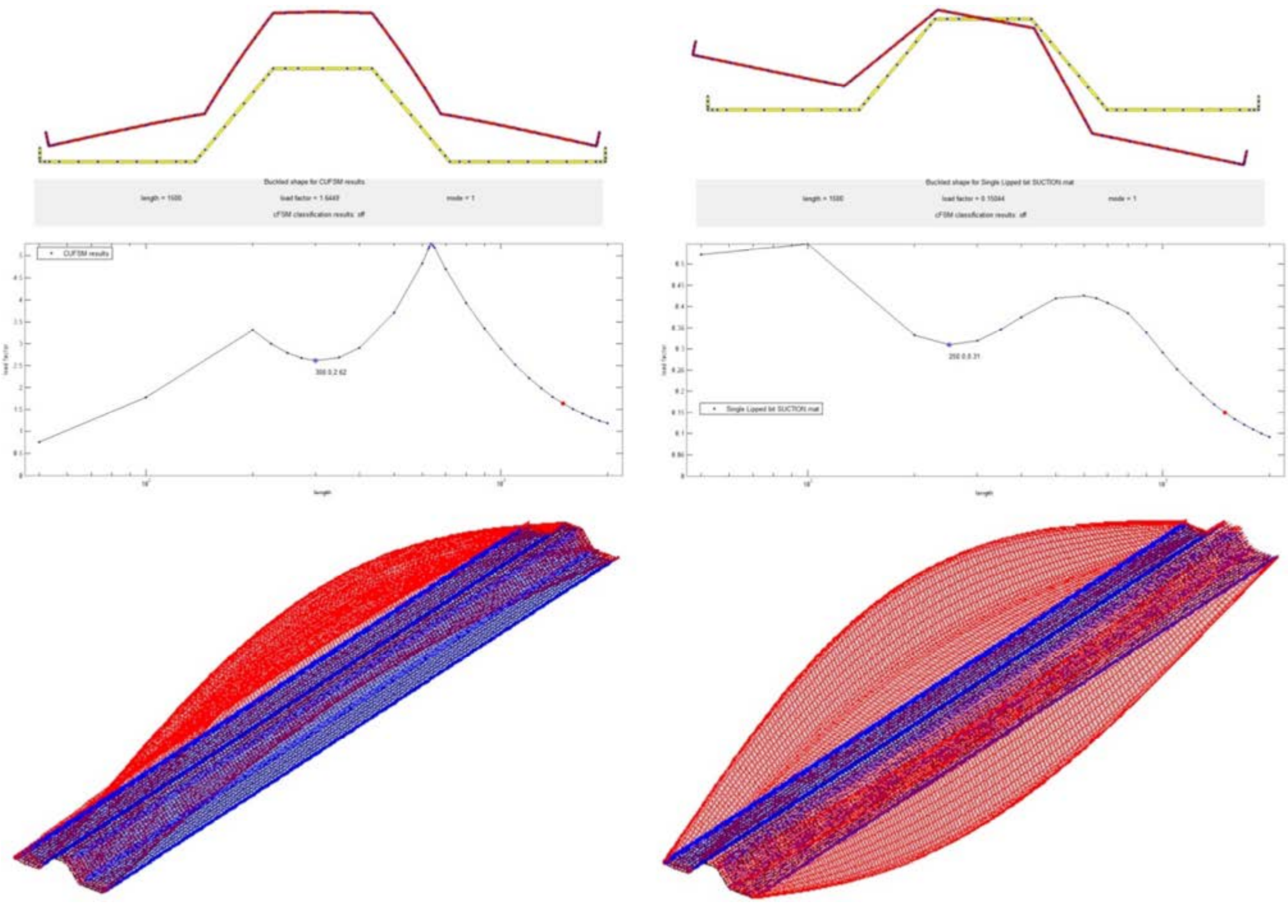


Evaluation of deck effective width under pedestal point load



	M_{ED} (kNm)	$M_{B,RD}$ (kNm)	M_{ED} / $M_{B,RD}$
SUCTION	-3.16	-0.99	3.2
PRESSURE	3.45	1.01	3.4

Evaluation of effective section according to EC 1993-1-3 and 1993-1-5

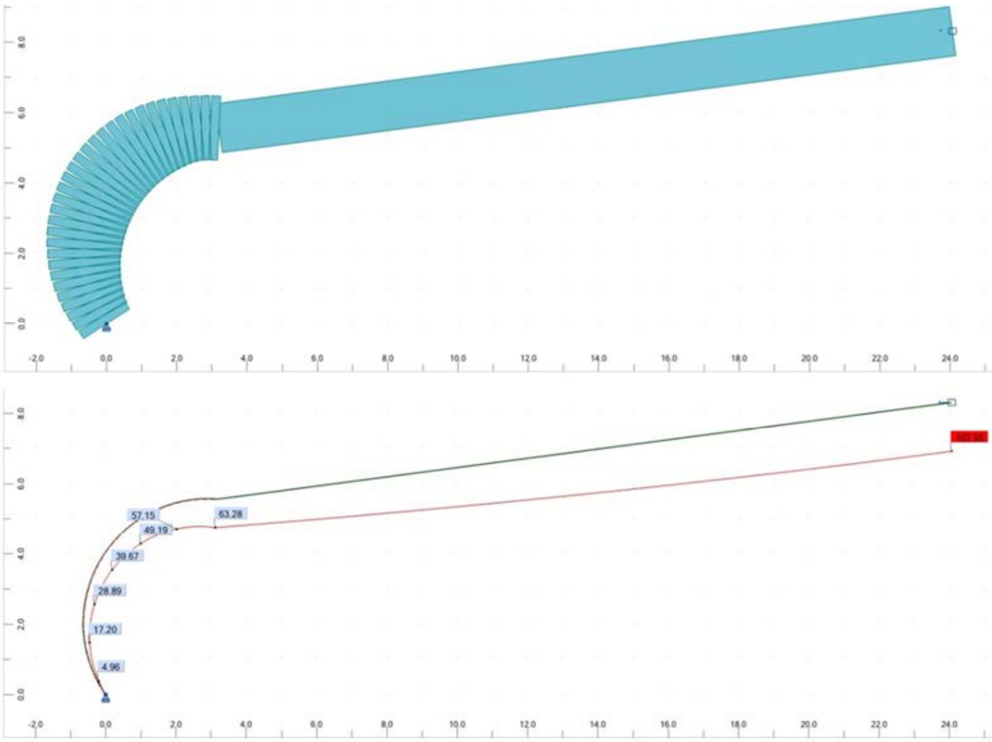


Deck buckling analysis under pressure (right) and suction load (left), using the constrained finite element method

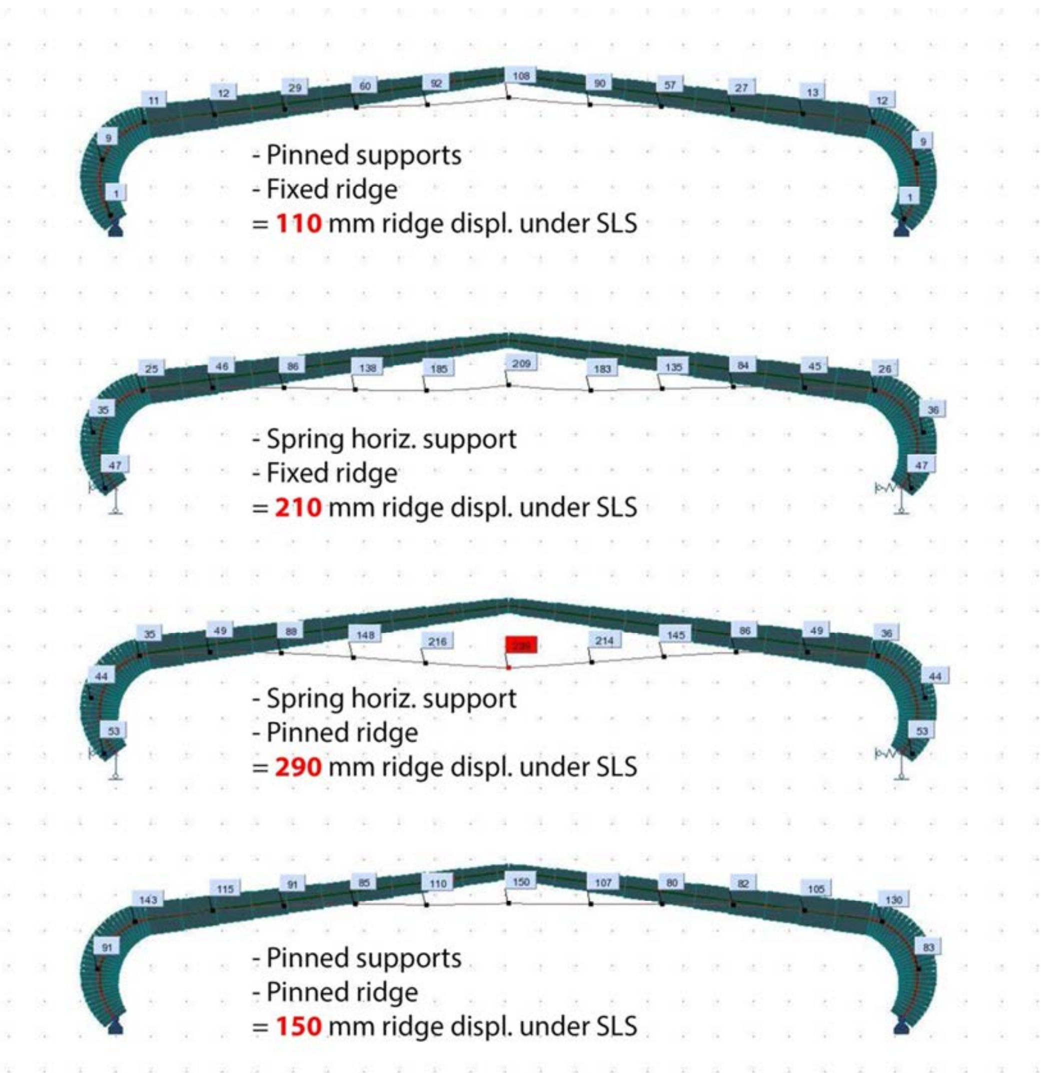
11 *Technical review of a long span timber roof of a pool in UK*

A swimming pool covered by a series of 50 m span glulam frames has recently been built in UK. Not long ago the completion of works the roof started exhibiting high as well as unexpected permanent deformations at the ridge, the causes of which are not clear yet.
OPS Ltd. were appointed to investigate the root causes and to find solutions.
I went through all the designer and contractor documentation and carried out numerical parametric testing in order to point out the effect of several parameters such as shrinkage, change in moisture content, manufacturing tolerances, foundations stiffness, etc... onto the structure's behaviour.
The work is still ongoing.

Glulam two hinged arches



Analysis under uniform shrinkage due to a variation in moisture content



Parametric study on the effect of the support and ridge connection stiffnesses