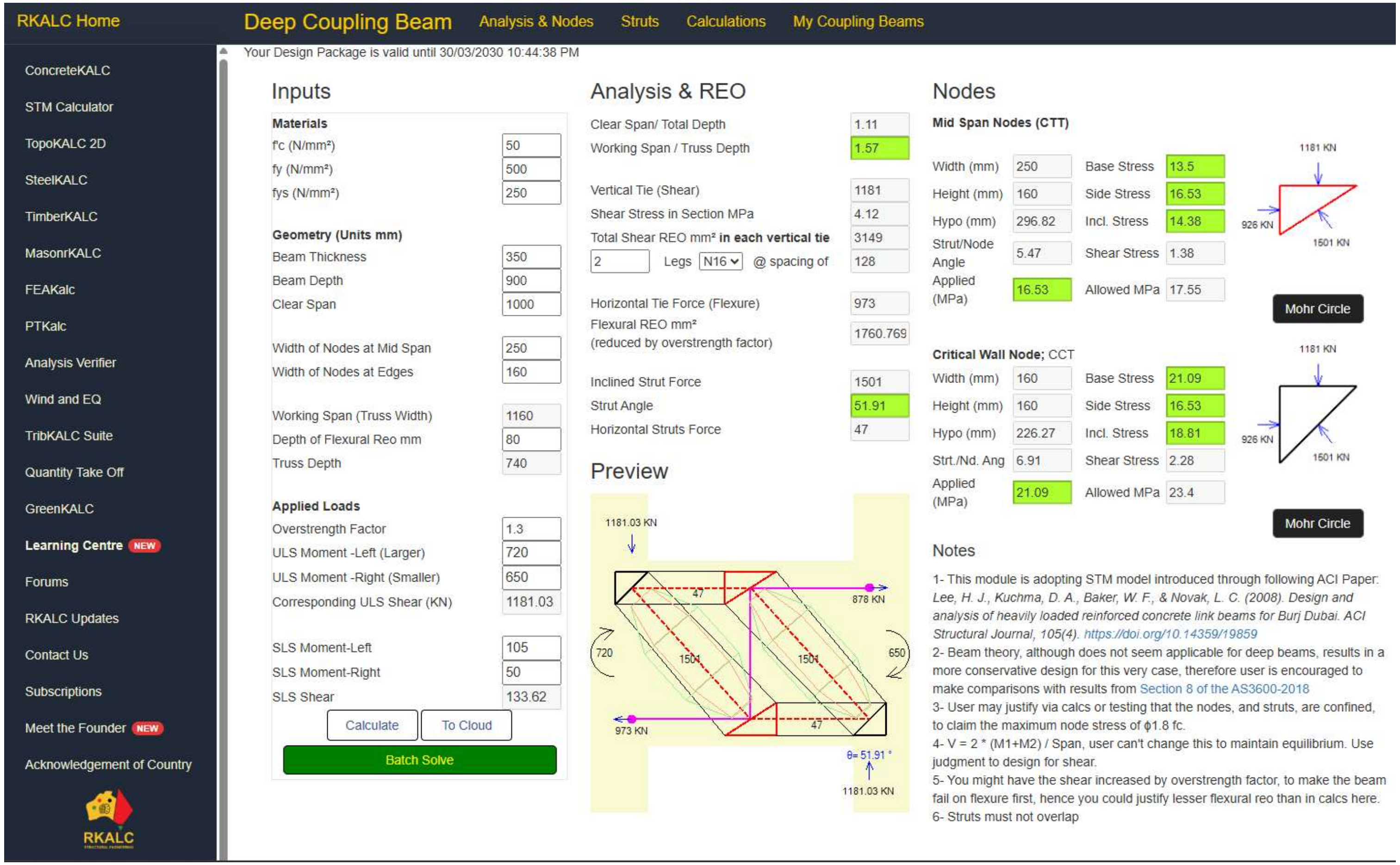
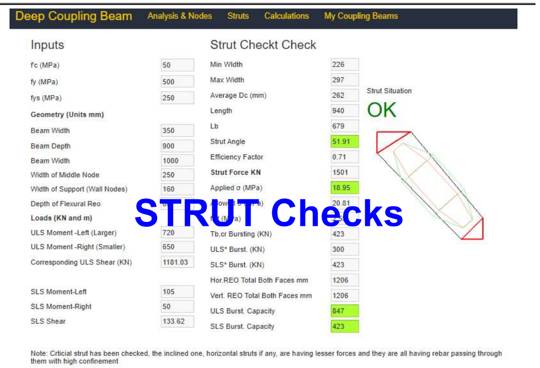
LEARNING CENTRE	Coupling Beam Design	Coupling Beams	By	RK
		STM Calculator	Sheet No. 1	Date 7/26

Sample Structure

1- Coupling Beam Behaviours
2- Deep Beam STM Model
3- Shear Design procedure
4- Flexural Design
5- Nodal and Strut Checking
6- Batch Solving - RKALC way



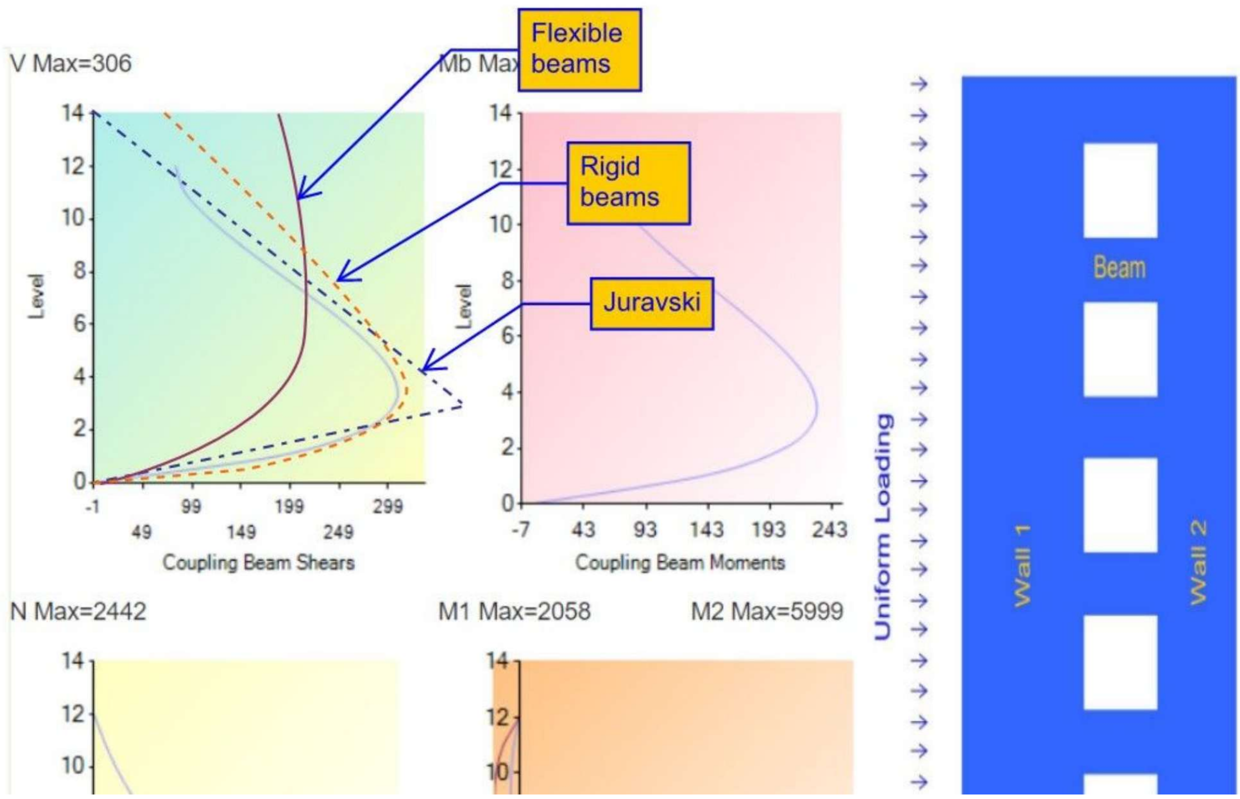


Educational Example Only – Refer to current Australian Standards and project-specific requirements before design.

The issue of coupled shear walls is one of the most debated topics among structural engineers. It's not surprising to see five people with twenty different opinions, each of valid points. Ove #Arup once said on this:

"The more you look, the more you see,
And that's why experts disagree.
For some look here, and some look there,
But no one can look everywhere.
For if they did, it seems to me
That they would hardly be experts, you see.
According to their point of view,
What they say may well be true,
But looking from another angle,
We tend to get into a tangle.
Which of the views is then correct?
That is not easy to suspect."

Many of us might appreciate that the flexibility of the beams can affect the distribution of force within height. There's an old rule of thumb that states coupling beams experience maximum shear within the bottom quarter of the wall height (around 0.2 to 0.3 of H).



According to a wise man by the name Juravski, a rigid beam takes shear represented by Vh/z. Here, V stands for the total shear applied to the wall, h represents the typical storey height, and z is the ratio of the wall length in plan (0.7L for single walls and approximately 0.85 for walls forming part of the core).

In cases where the geometry doesn't allow for squat-like or stiff beams with minimum depth and the span is relatively large, we refer to this as a flexible beam with an aspect ratio greater than, let's say, 4 (span to depth). In such scenarios, the maximum force travels from the bottom quarter to approximately 0.5 to 0.7 of H, resulting in a fairly uniform distribution of force across all beams. The magnitude, in this case, becomes a fraction of Juravski's formula, roughly ranging from 30% to 50%.

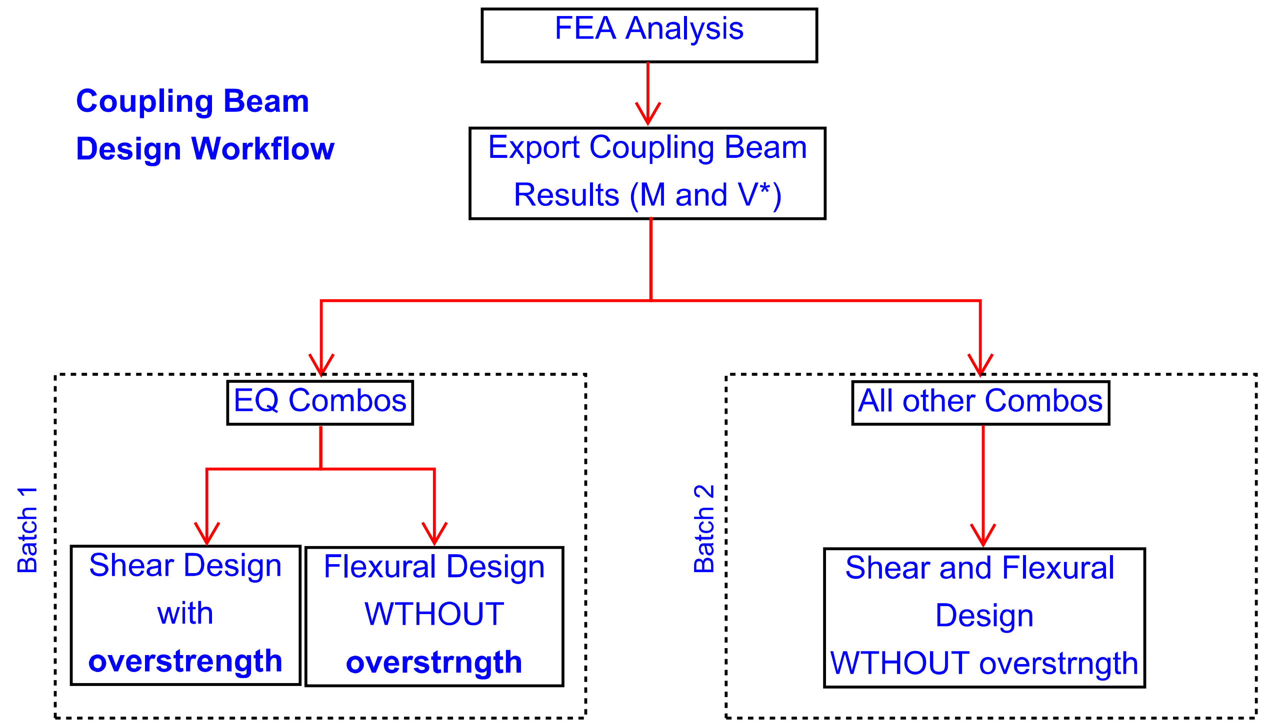
There's also a sweet spot, a range of medium stiffness with an aspect ratio between 2 and 4. Within this range, the shear suffered is approximately 75% of Juravski's formula, occurring at 30% to 50% of the height.

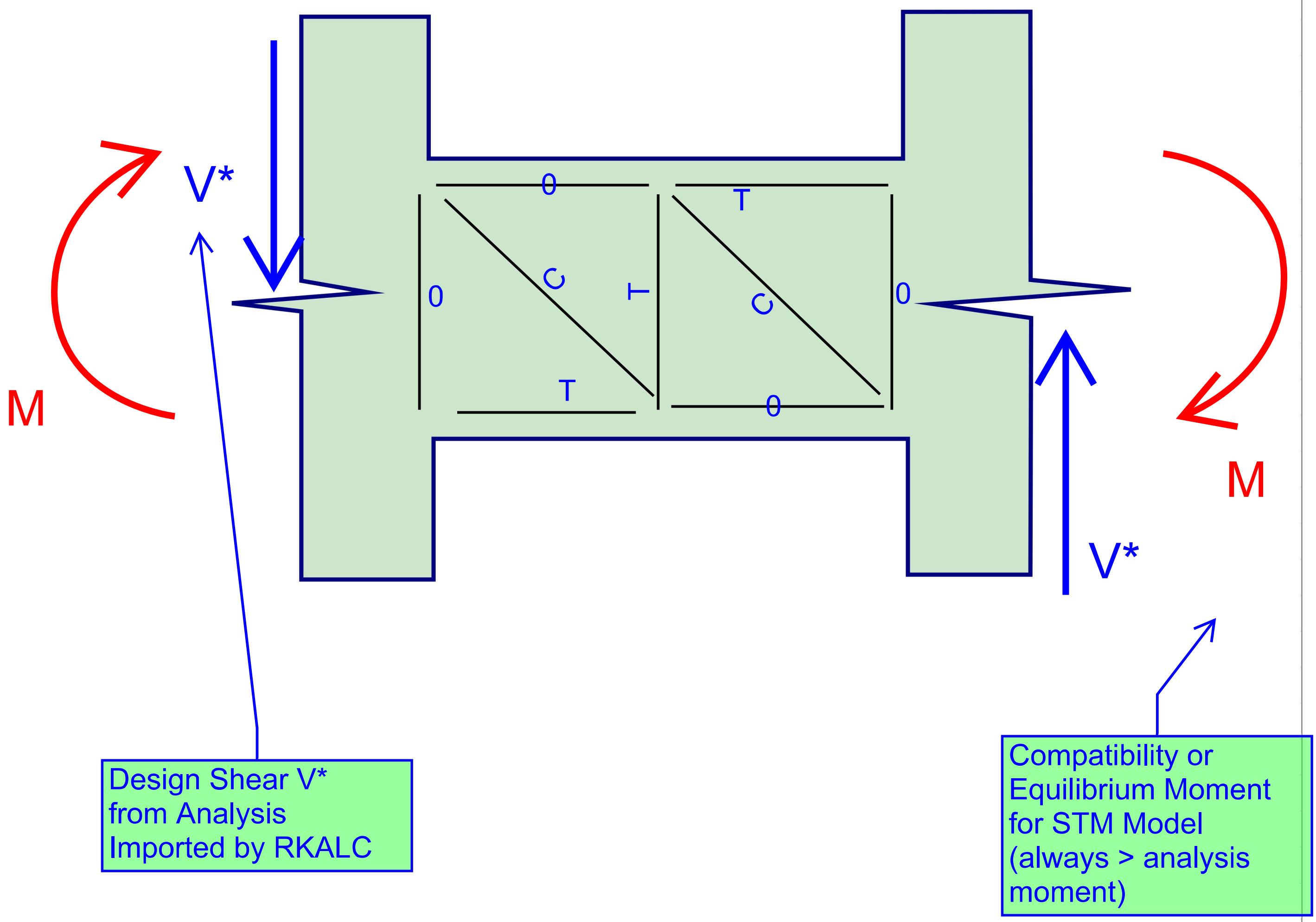
In any of the three cases mentioned above, the total sum of shears taken by the beams would be equal to the total reaction exerted by the wall pier connected to the group of beams.

It is unfavorable to have no coupling beams and solely rely on the slab as the coupling element. However, there may be situations where space constraints force such a configuration, and in those cases, the global behavior of the wall can be manipulated by transferring the force from one level to another.

Irrespective of beam depth or rigidity, some of us try to make sense of what we observe in ETABS. For instance, when adjusting beam stiffness to 40%, 70% etc., we are addressing a relevant matter. This involves artificially simulating post-ultimate loading event behavior by inducing cracking, a matter of judgement and thinking. Try all of these scenarios at our RKALC calculator.

Coupling Beam
Design Workflow





Overstrength factor for coupling beam has been not very clear in the current AS3600, many provide full u/sp as factor which is > 2 (overkill), others measure it based on provided flexural reinforcement. recommended value of 1.6 for current standard is conservative.
clause 14.6.6 do not cal the coupling beam by the name so this overstrength appreach applies to walls and to beams, unlike ACI318 for example.

Current Australian Standard AS3600-2018

14.6.6 In-plane shear

In determining the required shear strength of a structural wall and supporting foundations under earthquake design actions, allowance for the effects of flexural over-strength and dynamic amplification shall be included. This requirement is satisfied when the wall shear capacity (ϕV_u) at a cross-section is not less than:

A2

$$\phi V_u \geq \min\left[\left(\frac{1.6 M_u}{M^*}\right)V^*; \left(\frac{\mu}{S_p}\right)V^*\right] \quad \ldots 14.6.6$$

≈ 1.60

Proposed standard , finally, is calling the beams by name :) and provides less conservative

Proposed Change AS3600-2026

14.6.9 Coupling beams

Coupling beams across the entire height of structural walls that are integral to ensuring the desired plastic mechanism can be developed in the wall system shall be portioned and detailed to ensure a flexural mechanism can be developed without failing in shear. This requirement is satisfied when the coupling beam shear capacity (ϕV_u) is not less than —

$$\phi V_u \geq \min\left[\left(\frac{\Omega_v M_u}{M^*}\right)V^*; 1.5V^*\right] \quad 14.6.9$$

≈ 1.3

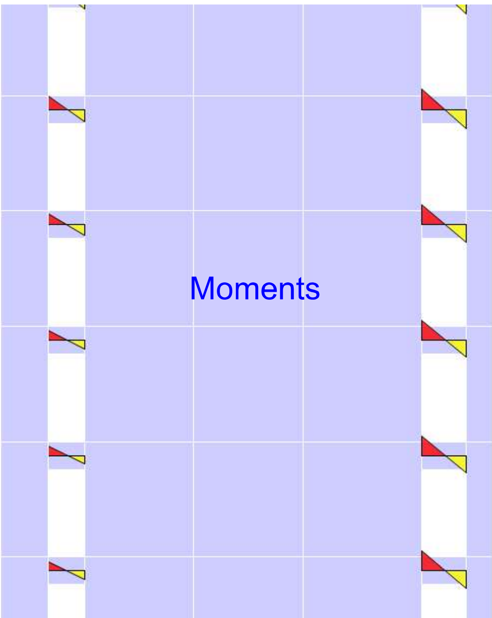
where

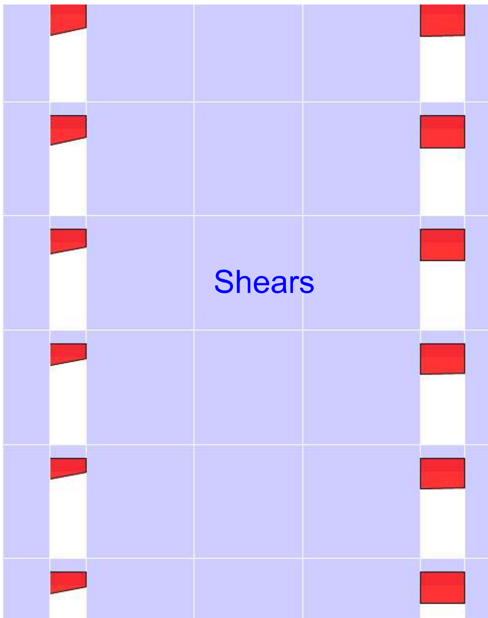
Ω_v = overstrength factor of the wall, which shall be taken as 1.2

M_u = moment capacity of the coupling beam

M^* = bending moment in the coupling beam

Typical procedure is to export ETABS results outside and design them using excel sheets because ETABS design is complying with AS3600 - or at least last time i checked.
Deep coupling beams must be designed as STM elements since span to depth ratio < 3





	A	B	C	D	E	F	G	H	I	J	K
1	TABLE: Spandrel Forces										
2	Story	Spandrel	utput Cas	Case Type	Location	P	V2	V3	T	M2	M3
3						kN	kN	kN	kN-m	kN-m	kN-m
4	TOP	S02	SW	LinStatic	Left	5	-7	0.02577	-0.00712	0.009975	-6
5	TOP	S02	SW	LinStatic	Right	5	-2	0.02577	-0.00712	-0.01579	-2
6	TOP	S02	Live	LinStatic	Left	0.1174	-0.2215	0.004716	0.000661	6.58E-05	-0.1066
7	TOP	S02	Live	LinStatic	Right	0.1174	-0.2215	0.004716	0.000661	-0.00465	0.1149
8	TOP	S02	SDL	LinStatic	Left	1	-3	0.002997	0.00167	-0.00115	-2
9	TOP	S02	SDL	LinStatic	Right	1	-3	0.002997	0.00167	-0.00415	1
10	TOP	S02	FACADE/P.	LinStatic	Left	0.4929	-1	0.00198	-0.00021	0.00055	-1
11	TOP	S02	FACADE/P.	LinStatic	Right	0.4929	-1	0.00198	-0.00021	-0.00143	0.4093
12	TOP	S02	SCREED	LinStatic	Left	0.3975	-1	0.00119	0.000385	-0.00024	-1
13	TOP	S02	SCREED	LinStatic	Right	0.3975	-1	0.00119	0.000385	-0.00143	0.3314
14	TOP	S02	STORAGE	LinStatic	Left	-0.0041	0.00839	2.31E-05	1.97E-06	-1.4E-06	0.00523
15	TOP	S02	STORAGE	LinStatic	Right	-0.0041	0.00839	2.31E-05	1.97E-06	-2.4E-05	-0.00316
16	TOP	S02	EP	LinStatic	Left	4.19E-05	-7.5E-05	0	1.18E-06	0	-3.9E-05
17	TOP	S02	EP	LinStatic	Right	4.19E-05	-7.5E-05	0	1.18E-06	0	3.58E-05
18	TOP	S02	1.2G+1.5Q	Combinat	Left	8	-14	0.04543	-0.00533	0.01105	-11
19	TOP	S02	1.2G+1.5Q	Combinat	Right	8	-8	0.04543	-0.00533	-0.03438	-0.1343
20	TOP	S02	1.35DL	Combinat	Left	9	-15	0.04311	-0.00711	0.01233	-12
21	TOP	S02	1.35DL	Combinat	Right	9	-9	0.04311	-0.00711	-0.03079	-0.3396
22	TOP	S02	0.9DL+W3:	Combinat	Left	6	-10	0.02874	-0.00474	0.008217	-8
23	TOP	S02	0.9DL+W3:	Combinat	Right	6	-6	0.02874	-0.00474	-0.02053	-0.2264
24	TOP	S02	0.9DL+W3‹	Combinat	Left	6	-10	0.02874	-0.00474	0.008217	-8
25	TOP	S02	0.9DL+W3‹	Combinat	Right	6	-6	0.02874	-0.00474	-0.02053	-0.2264
26	TOP	S02	0.9DL+W3!	Combinat	Left	6	-10	0.02874	-0.00474	0.008217	-8
27	TOP	S02	0.9DL+W3!	Combinat	Right	6	-6	0.02874	-0.00474	-0.02053	-0.2264
28	TOP	S02	0.9DL+W3(	Combinat	Left	6	-10	0.02874	-0.00474	0.008217	-8
29	TOP	S02	0.9DL+W3(	Combinat	Right	6	-6	0.02874	-0.00474	-0.02053	-0.2264

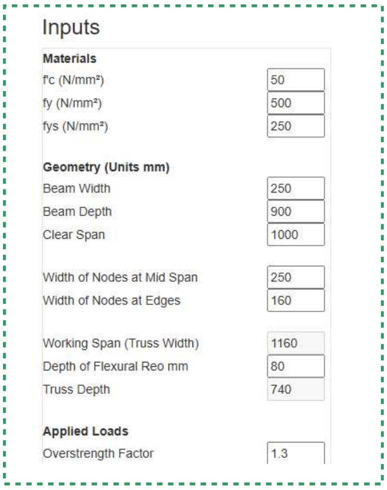
Results Exported from ETABS

RKALC STM
Workflow

Set Beam Parameters

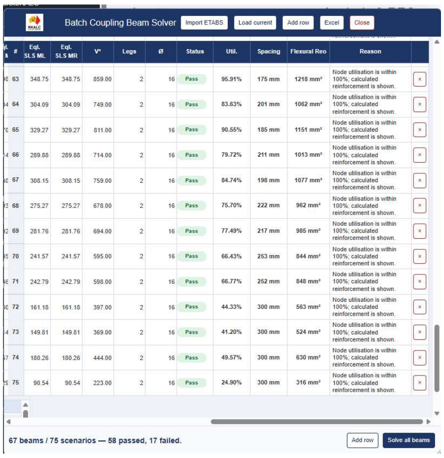
Materials:
(fc / fy / fysls)

Geometry
(depth / width / thick
reo depth / nodes &
Overstrength)

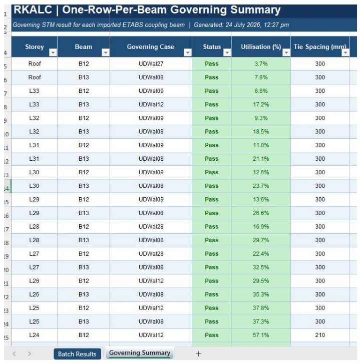


Import ETABS results
or other software
One Wall at a time
or group all beams that
have same span + depth

etabs wont tell
span & thickness



Batch Solve



Export Results to Excel
&
Review Results

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ACI STRUCTURAL JOURNAL TECHNICAL PAPER

Title no. 105-S43

Design and Analysis of Heavily Loaded Reinforced Concrete Link Beams for Burj Dubai

by Ho Jung Lee, Daniel A. Kuchma, William Baker, and Lawrence C. Novak

This paper presents a study on the shear capacity and behavior of reinforced concrete link beams designed for the Burj Dubai Tower, which is the tallest building in the world and will be ready for occupancy in 2009. Several thousand reinforced concrete link beams were used in this structure to interconnect structural walls; in some cases, the factored shear forces in these link beams were up to three times the traditional nominal ACI shear force strength limit. This study presents an examination of the factors that control the design and behavior of heavily loaded reinforced concrete link beams. Nonlinear finite element analysis methods were used to validate and inform the design of the reinforced concrete link beams for Burj Dubai and to examine if the ACI nominal sectional shear force limit is appropriate for this type of member. The results illustrate the undue conservatism of the ACI design provisions and the role of nonlinear analyses in design.

Keywords: beam; design; nonlinear finite element analysis; reinforced concrete; shear; strut-and-tie model.

INTRODUCTION

The Burj Dubai Tower, when completed, will be the world's tallest structure. Whereas the final height of this multi-use skyscraper is a "well-guarded secret," it will comfortably exceed the current record holder of 509 meter (1671 ft) tall Taipei 101. The 280,000 m^2 (3,000,000 ft^2) reinforced concrete tower will be used for retail, an Armani hotel, residences, and offices. The goal of the Burj Dubai Tower is not simply to be the world's highest building—it is to embody the world's highest aspirations.

Designers purposefully shaped the structural concrete for the Burj Dubai to be Y-shaped in plan to reduce the wind forces as well as to keep the structure simple and foster constructibility. The structural system can be described as a buttressed core, as shown in Fig. 1 and 2. Each wing, with its own high-performance concrete core and perimeter columns, buttresses the others via a six-sided central core, or hexagonal hub. The result is a tower that is extremely stiff torsionally. The design team purposely aligned all the common central core and column elements to form a building with no structural transfers.

Each tier of the building steps back in a spiral pattern that causes the tower's width to change at each setback. The advantage of this stepping and shaping is to "confuse the wind." The wind vortexes never become organized because at each new tier the wind encounters a different building shape that reduces the overall wind loads on the structure. Due to the tapering of the tower, the primary demand on the link beams is from gravity load redistribution, flow from the taller core to the perimeter of the structure. The 280,000 m^2 (3,000,000 ft^2) tower and 185,000 m^2 (2,000,000 ft^2) podium structures are currently under construction, as shown in Fig. 2. The project is scheduled for completion in 2009.

The center hex reinforced concrete core walls provide the torsional resistance of the structure similar to a closed pipe or axle, as shown in Fig. 3. The center hex walls are

Fig. 1—Tower rendering.

Fig. 2—Construction photo of tower.

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Ho Jung Lee *is an Engineer with SC Solutions, Inc., Sunnyvale, CA. He received his BS and MS from the Seoul National University, Seoul, South Korea, and his PhD from the University of Illinois at Urbana-Champaign, Urbana, IL.*

Daniel A. Kuchma, *FACI, is an Assistant Professor in the Department of Civil and Environmental Engineering at the University of Illinois at Urbana-Champaign. He received his PhD from the University of Toronto, Toronto, ON, Canada. He is a member of ACI Subcommittee 318-E, Shear and Torsion.*

William Baker *is a Partner of Skidmore, Owings & Merrill LLP, Chicago, IL.*

Lawrence C. Novak *is an Associate Partner of Skidmore, Owings & Merrill LLP.*

buttressed by the wing walls and hammer head walls, which behave as the webs and flanges of a beam to resist the wind shears and moments. Outriggers at the mechanical floors allow the columns to participate in the lateral load resistance of the structure; hence, all of the vertical concrete is used to support both gravity and lateral loads. The walls had

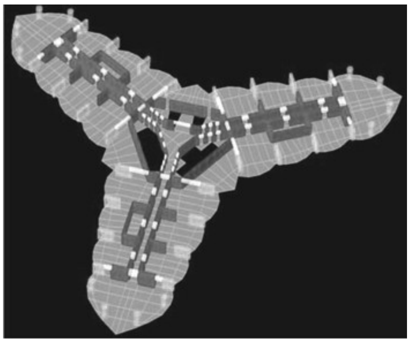


Fig. 3—Three-dimensional view of single floor.

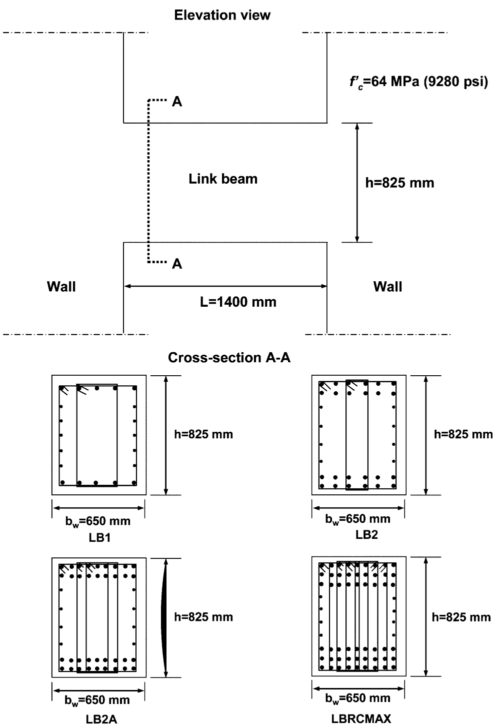


Fig. 4—Design details for analyzed link beams. (Note: 1 mm = 0.0394 in.)

concrete strengths ranging from C80 to C60 cube strength and contained portland cement and fly ash. Local aggregates were used for the concrete mixture design. The wall and column sizes were optimized using virtual work/LaGrange multiplier methods.[1] This results in a very efficient structure.

The structure was analyzed for gravity (including P-Δ analysis and creep and shrinkage), wind, and seismic loads by a three-dimensional analysis model that consisted of the reinforced concrete walls, link beams, slabs, mats, piles, and the spire structural steel system. The model consisted of over 73,500 shells and 75,000 nodes.

RESEARCH SIGNIFICANCE

This paper examines methods for the design and analysis for reinforced concrete link beams that are cast integral with wall piers. In particular, the appropriateness of the strut-and-tie method and shear design stress limits for this class of member are reviewed. The results of this investigation indicate that much higher shear stress levels should be permitted in ACI 318, as this would greatly extend the utility of this class of member.

OVERVIEW OF LINK BEAM DESIGN

The demands on the link beams vary greatly and are dependent on the location of the link beam relative to a setback with the largest shear forces being generated in the beams closest to a setback. The typical link beams used in the Burj Dubai are quite stocky with a shear-span ratio ($l/2h$) of 0.85, a width of 650 mm (2.13 ft), and a height of 825 mm (2.7 ft). For the design of reinforced concrete link beams, the conventional deep beam design method in the ACI 318-99[2] and the strut-and-tie method in ACI 318-02[3] were used, with Appendix A enabling the design of link beams somewhat beyond the conventionally designed maximum deep beam stress limit of $10\sqrt{f'_c}$ in psi ($0.83\sqrt{f'_c}$ in MPa), which is based on ACI 318-99,[2] Section 11.8.4, as will be discussed in the following. In the case of members subjected to very large shear forces, embedded built-up structural steel sections were provided within the core of the concrete link beams to carry the entire shear and flexure demand.

This study was principally conducted during the structural design of the Burj Dubai and used to check and inform the design by the different methods and to examine the condition and stiffness of the reinforced concrete link beams under service and factored loads. In addition, a study was made of the appropriateness of nominal shear stress limits for link beams. A series of nonlinear analyses were conducted that can account for the influence of many factors on response including the amount of flexural reinforcement, the distribution of vertical and horizontal web reinforcement, the span-depth ratio, and the confinement provided by walls at the ends of the link beams. The nonlinear finite element analysis tools used in this investigation were ABAQUS,[4] ADINA,[5] and VecTor2.[6]

DESIGN DETAILS OF LINK BEAMS

The geometry, factored loads, and design methods of four Burj Dubai link beams, LB1 to LB4, are shown in Table 1. These link beams, which have the same external dimensions, capture the range of typical shear design force levels for which different design solutions were used. Table 2 and Fig. 4 present details on Link Beams LB1 and LB2, as well as Link Beams LB2A and LBRCMAX that are more heavily reinforced and hypothetical link beams whose behavior is evaluated in the nonlinear analyses. Link Beam LB1 was designed by the deep design method specified in ACI 318-99.[2] Link Beam LB2 was designed by the strut-and-tie model in Appendix A of ACI 318-02.[3] Vertical shear or tie reinforcement in Link Beams LB1 and LB2 was determined using the selected design approaches to support their factored design loads. Horizontal web reinforcement close to the minimum amount suggested for deep beams in ACI 318-02[3] was used in Link Beams LB1 and LB2. The design of Link Beam LB2 using the strut-and-tie method in the ACI 318-02[3] provisions, unchanged in ACI 318-05,[7] is described in the next section. As previously mentioned, a pure reinforced design concrete solution was not possible for all members by ACI 318-02,[3] or would be by ACI 318-05,[7] such that composite members with steel-embedded sections were used to support the shear design in members subjected to very large shear and flexural forces, including Link Beams LB3 and LB4. An objective of this study was to investigate whether or not it was possible to develop a pure reinforced concrete solution to support the very large shear forces in Link Beams LB3 and LB4. To this end, the behavior of more heavily reinforced members, Link Beams LB2A and LBRCMAX, as described in Table 2 and Fig. 4, will also be examined.

The concrete cylinder compressive strength used in the design of these link beams was f_c' = 64 MPa (9280 psi); a concrete cube strength of 80 MPa (11600 psi) was specified and actual cube and cylinder breaks indicate considerably stronger concrete. The design yield strength of flexural reinforcement used in the link beams was 460 MPa (67 ksi) and of the vertical stirrups and horizontal web reinforcement was 420 MPa (61 ksi). Reinforcement with a yield strength of 460 MPa (67 ksi) was actually provided, but in accordance with Section 11.5.2 of ACI 318-02,[3] the effective strength was taken as 420 MPa (61 ksi). Nonlinear finite element analyses of these reinforced concrete link beams (LB1, LB2, LB2A, and LBRCMAX) were performed as will be presented.

STRUT-AND-TIE MODEL USED IN LINK BEAM

The strut-and-tie method has recently developed as a rational method in the design of discontinuity (D)-regions in structural concrete such as deep beam, squat walls, pile caps, and other elements in which plane sections do not remain plane. The strut-and-tie method provides a conceptually simple design methodology based on the lower-bound theorem of limit analysis.[8] Provisions for using the strut-and-tie method were included as Appendix A in ACI 318-02.[3]

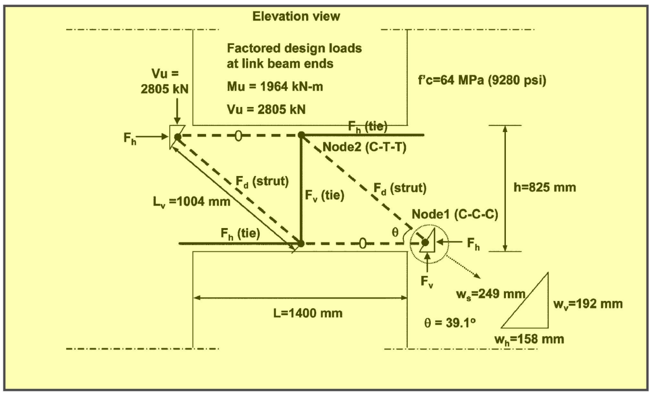


Fig. 5—Strut-and-tie model used in design of Link Beam LB2. (Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 psi = 6.89 × 10^{-3} MPa; and 1 ft-kip = 1.356 kN-m.)

The strut-and-tie model used for the design of Link Beam LB2 is shown in Fig. 5. The design yields the required amount of horizontal and vertical tie reinforcement and the required strength of diagonal concrete struts and nodal zones. Minimum horizontal web reinforcement was provided in the link beams even though it was not specifically required when the strut-and tie design procedure is used. In this link beam design, the amount of horizontal and vertical ties can simply be increased to support larger shear loads until the strength is limited by the strength of the diagonal struts and nodal zones.

The use of Appendix A in ACI 318-02[3] and ACI 318-05[7] provides for a direct design of the diagonal strength of struts so as to avoid a diagonal compressive failure. This permits a member to be designed for a higher shear stress than the limit for deep beams in ACI 318-99.[2] This deep beam stress limit was also set to guard against diagonal compression failures but is not considered to be necessary for deep beams as will be discussed in the following. In the design of Link Beam LB2 by Appendix A of ACI 318-02, the conservative assumption of narrow bottle-shaped diagonal struts were made even though the results of the analyses indicate that there is a uniform field of diagonal compression throughout these members. Thus, it was considered that the nominal capacity calculated for this strut-and-tie model by ACI 318-02 would lead to a conservative design.

MAXIMUM SHEAR STRENGTH SPECIFIED IN CODE PROVISIONS

There is a large variation in the nominal shear design stress limit that is specified in codes of practice even though the reason for this limit is the same as guarding against a diagonal

Table 1—Geometry, loading, and design methods for link beams

	Geometry			Factored loads		
Beam ID	Width, mm	Depth, mm	Span, mm	Shear, kN	Moment kN-m	Design method used
LB1	650	825	1400	1705	1194	Conventional (ACI 318-99, Section 11.8)
LB2	650	825	1400	2805	1164	Strut-and-tie (ACI 318-02, Appendixes A and C)
LB3	650	825	1400	3750	2625	Steel plate
LB4	650	825	1400	5250	3675	Built-up steel I-beam

Notes: Factored loads are equal for both ends of link beams. Ratio of ultimate load to sustained day-to-day (gravity only) service loads is approximately 2.5. Walls adjacent to link beams are 650 mm thick and are typically reinforced with a minimum of T20mm at 350 mm on each vertical and horizontal face. 1 in. = 25.4 mm; 1 kip = 4.448 kN; and 1 ft-kip = 0.356 kN-m.

Table 2—Reinforcing details of link beams analyzed in study

	Reinforcement			Stirrups		
Beam ID	Top bars	Bottom bars	Side bars each face	Size	Spacing, mm	Type
LB1	5-T32	5-T32	5-T12	T16	150	Two hoops
LB2	12-T32	12-T32	4-T12	T16	125	Two hoops
LB2A	18-T32	18-T32	4-T12	T16	80	Three hoops
LBRCMAX	27-T32	27-T32	4-T12	T16	75	Five hoops

Notes: T32, T20, T16, and T12 are deformed reinforcing bars with respective diameters of 32, 20, 16, and 12 mm. In LB2, top and bottom bars were used in two layers each. LB2A and LRCMAX were not used in Burj Dubai project. They are included for purpose of examining appropriateness of current ACI 318-05 limit on maximum shear stress. LB2A has significantly more longitudinal tension reinforcement and transverse reinforcement than LB2. LBRCMAX is analyzed to figure out maximum shear capacity of reinforced concrete link beam. In LB2A and LBRCMAX, top and bottom bars were placed in two layers and three layers each, respectively. 1 in. = 25.4 mm.

Inputs

Materials	
f'c (N/mm²)	50
fy (N/mm²)	500
fys (N/mm²)	250
Geometry (Units mm)	
Beam Thickness	350
Beam Depth	900
Clear Span	1000
Width of Nodes at Mid Span	250
Width of Nodes at Edges	160
Working Span (Truss Width)	1160
Depth of Flexural Reo mm	80
Truss Depth	740
Applied Loads	
Overstrength Factor	1.3
ULS Moment -Left (Larger)	720
ULS Moment -Right (Smaller)	650
Corresponding ULS Shear (KN)	1181.03
SLS Moment-Left	105
SLS Moment-Right	50
SLS Shear	133.62

V=(ML+MR)/L

Calculate | To Cloud

Batch Solve

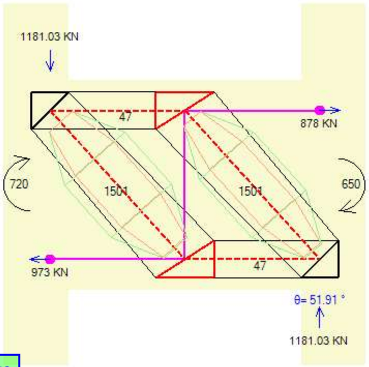
Inputs

Analysis & REO

Clear Span/ Total Depth	1.11
Working Span / Truss Depth	1.57
Vertical Tie (Shear)	1181
Shear Stress in Section MPa	4.12
Total Shear REO mm² **in each vertical tie**	3149
2 Legs N16 @ spacing of	128
Horizontal Tie Force (Flexure)	973
Flexural REO mm² (reduced by overstrength factor)	1760.769
Inclined Strut Force	1501
Strut Angle	51.91
Horizontal Struts Force	47

reinforcements

Preview

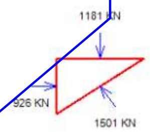


Nodes

Nodal Checks

Mid Span Nodes (CTT)

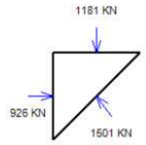
Width (mm)	250	Base Stress	13.5
Height (mm)	160	Side Stress	16.53
Hypo (mm)	296.82	Incl. Stress	14.38
Strut/Node Angle	5.47	Shear Stress	1.38
Applied (MPa)	16.53	Allowed MPa	17.55



Mohr Circle

Critical Wall Node; CCT

Width (mm)	160	Base Stress	21.09
Height (mm)	160	Side Stress	16.53
Hypo (mm)	226.27	Incl. Stress	18.81
Strt./Nd. Ang	6.91	Shear Stress	2.28
Applied (MPa)	21.09	Allowed MPa	23.4



Mohr Circle

Notes

1- This module is adopting STM model introduced through following ACI Paper: *Lee, H. J., Kuchma, D. A., Baker, W. F., & Novak, L. C. (2008). Design and analysis of heavily loaded reinforced concrete link beams for Burj Dubai. ACI Structural Journal, 105(4). https://doi.org/10.14359/19859*

2- Beam theory, although does not seem applicable for deep beams, results in a more conservative design for this very case, therefore user is encouraged to make comparisons with results from Section 8 of the AS3600-2018

3- User may justify via calcs or testing that the nodes, and struts, are confined, to claim the maximum node stress of ϕ1.8 fc.

4- V = 2 * (M1+M2) / Span, user can't change this to maintain equilibrium. Use judgment to design for shear.

5- You might have the shear increased by overstrength factor, to make the beam fail on flexure first, hence you could justify lesser flexural reo than in calcs here.

6- Struts must not overlap

Inputs

f'c (MPa)	50
fy (MPa)	500
fys (MPa)	250
Geometry (Units mm)	
Beam Width	350
Beam Depth	900
Beam Width	1000
Width of Middle Node	250
Width of Support (Wall Nodes)	160
Depth of Flexural Reo	80
Loads (KN and m)	
ULS Moment -Left (Larger)	720
ULS Moment -Right (Smaller)	650
Corresponding ULS Shear (KN)	1181.03
SLS Moment-Left	105
SLS Moment-Right	50
SLS Shear	133.62

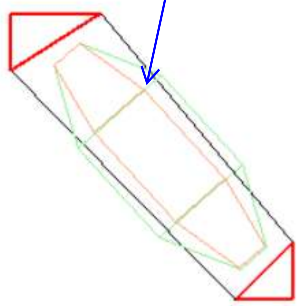
Strut Checkt Check

Min WIdth	226
Max Width	297
Average Dc (mm)	262
Length	940
Lb	679
Strut Angle	51.91
Efficiency Factor	0.71
Strut Force KN	1501
Applied σ (MPa)	18.95
Allowed σ (MPa)	20.81
f'ct (MPa)	2.55
Tb,cr Bursting (KN)	423
ULS* Burst. (KN)	300
SLS* Burst. (KN)	423
Hor.REO Total Both Faces mm	1206
Vert. REO Total Both Faces mm	1206
ULS Burst. Capacity	847
SLS Burst. Capacity	423

if very keen, design for struts, but note, beam is super confined

Strut Situation

OK



Coupling Beam STM Design - AS3600-2018 - A2

Strut-and-tie model analysis and reinforced concrete design

Generated
22 July 2026, 6:35 PM

Calculations by
admin@rkalc.com

1. Introduction

This document provides analysis of non-flexural element design in accordance with the Australian Standard AS3600-2018. It covers both the ultimate and service conditions

This module is adopting STM model introduced through following ACI Paper:
Lee, H. J., Kuchma, D. A., Baker, W. F., & Novak, L. C. (2008). Design and analysis of heavily loaded reinforced concrete link beams for Burj Dubai. ACI Structural Journal, 105(4). https://doi.org/10.14359/19859

The following outcomes are provided:

RKALC Calculation sheet

- Flexural Reinforcement
- Nodal Stresses and compliance with the Standard
- Strut Checks, including bursting

The design ensures structural integrity and compliance with the standard under the specified loading conditions. For detailed guidelines and requirements, refer to the relevant sections of AS3600-2018-A2.

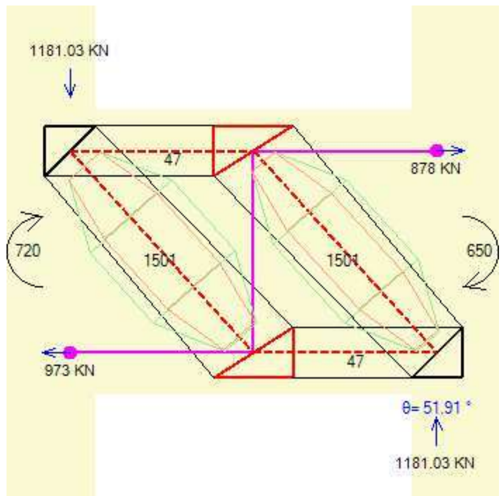
> This calculation can be reproduced and checked independently. For more details, refer **RKALC website**.

2. Material Properties

Property	Value	Unit
Concrete Strength f'c	**50**	MPa
Rebar fy	**500**	MPa
SLS Rebar stress fys	**250**	MPa

3. Geometry

Geometry Parameter	Value	Unit
Beam Width	**350**	mm
Beam Depth	**900**	mm
Beam Span	**1000**	mm
Width of Nodes at Mid Span	**250**	mm
Width of Nodes at Edges	**160**	mm
Working Span (Truss Width)	**1160**	mm
Depth of Flexural Reo	**80**	mm
Truss Depth	**740**	mm



4. Applied Loads

Applied Load	Value	Unit
ULS Moment -Left (Larger)	**720**	KN
ULS Moment -Right (Smaller)	**650**	KN
Corresponding ULS Shear (KN)	**1181.03**	mm
SLS Moment-Left	**105**	mm
SLS Moment-Right	**50**	mm
SLS Shear	**133.62**	mm

5. Nodal Assumptions

Mid Span Nodes (Red in Figure)

Node Width (total)	**250**	mm
Node Depth	**160**	mm
Node Thickness	**350**	mm

Critical Wall Nodes (the ones near at beam ends)

Critical Wall Node Width	**160**	mm
Critical Wall Node Depth	**160**	mm
Node Thickness	**350**	mm

6. Analysis and Required Reinforcement

Analysis Result	Value	Unit
Span-Depth Ratio of the STM Model	**1.11**	
Working Span / Truss Depth	**1.57**	mm
Vertical Tie (Shear)	**1181**	KN
Shear Stress in Section	**4.12**	MPa
Total Shear REO	**3149**	mm^2
Horizontal Tie Force (Flexure)	**973**	KN
Flexural REO	**2289**	mm^2
Inclined Strut Force	**1501**	KN
Strut Angle	**51.91**	Degrees
Horizontal Struts Force	**47**	KN

liquidate total shear reo on each faces,
and spread the reinforcement
long the span

Ensure the reinforcement is reasonable,
not heavy, and spread the reinforcement
so the centre of its geometry
is passing by the centre of the nodes at the bottom

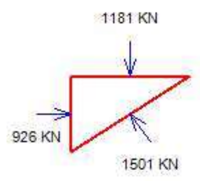
7. Nodal Capacity Check

Mid Span Node

Left part

Check	Value	Unit
Width	**250**	mm
Base stress	**350**	N/mm^2
Side stress	**16.53**	N/mm^2
Inclined stress	**14.38**	N/mm^2
Shear stress	**1.38**	N/mm^2
Allowed Stress (CTT Node)	**17.55**	N/mm^2

Status Pass

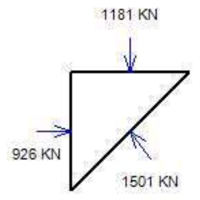


Critical Wall Node

Check	Value	Unit
Base stress	**21.09**	N/mm^2
Side stress	**16.53**	N/mm^2
Inclined stress	**18.81**	N/mm^2
Shear stress	**2.28**	N/mm^2
Allowed Stress (CCT Node)	**23.4**	N/mm^2

Note: if M1 and M2 are not equal, node becomes CCT, otherwise CCC

Status Pass



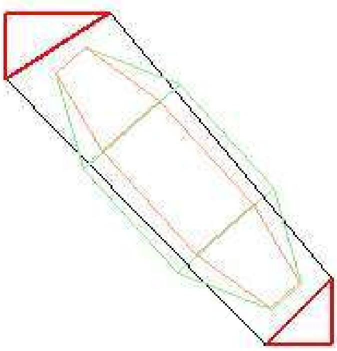
8. Notes

1. Beam theory, although does not seem applicable for deep beams, results in a more conservative design for this very case, therefore user is encouraged to make comparisons with results from Section 8 of the AS3600-2018
2. User may justify via calcs or testing that the nodes, and struts, are confined, to claim the maximum node stress of ϕ1.8 fc.

3. V = 2 * (M1+M2) / Span, user can't change this to maintain equilibrium. Use judgment to design for shear.
4. You might have the shear increased by overstrength factor, to make the beam fail on flexure first, hence you could justify lesser flexural reo than in calcs here.

Left Strut

Strut Parameter	Value	Unit
Average Dc	**262**	mm
Lb	**679**	mm
Angle	**51.91**	Degrees
Efficiency Factor	**0.71**	
Applied MPa	**18.95**	
Allowed MPa	**20.81**	
fct	**2.55**	MPa
Tb,cr Bursting	**423**	MPa
SLS* Bursting	**423**	KN
ULS* Burst	**300**	KN
Hor.REO Total Both Faces mm	**1206**	mm^2
Vert.REO Total Both Faces mm	**1206**	mm^2
ULS Burst. Capacity	**847**	KN
SLS Burst. Capacity	**423**	KN
Strut Status	**OK**	



Note: Crticial strut has been checked, the inclined one, horizontal struts if any, are having lesser forces and they are all having rebar passing through them with high confinement