โครงการพัฒนาพื้นที่ส่วนขยาย
ศูนย์ราชการเฉลิมพระเกียรติ ๘๐ พรรษา ๕ ธันวาคม ๒๕๕๐ พื้นที่โซน C
บริษัท ธนารักษ์พัฒนาสินทรัพย์ จำกัด

แบบรายละเอียดฉบับสมบูรณ์ (Final Detailed Design)
สัญญาก่อสร้างอาคารสนับสนุน
(ส่วนงานโครงสร้าง งานสถาปัตยกรรม และงานระบบหลัก)

รายงานการออกแบบ Design Note รายการคำนวณ
งานโครงสร้าง

(REVISION 0)

จัดทำโดย

GCDT
GOVERNMENT CENTER
DESIGN TEAM

21 เมษายน 2564

สำหรับปรับรองรายการคำนวณออกแบบ อาคารสนับสนุน
โครงการพัฒนาพื้นที่ส่วนขยาย ศูนย์รายการเฉลิมพระเกียรติ 80 พรรษา
5 ธันวาคม 2550 พื้นที่โซน C

สำเนาถูกต้อง

07 เมษายน 2564

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1. เกณฑ์กำหนดในการออกแบบ
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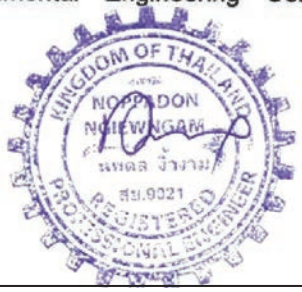
หมวดที่ 1
ข้อกำหนดที่ใช้ในการออกแบบ (Design Criteria)

1. **มาตรฐาน พระราชบัญญัติ และข้อบัญญัติที่ใช้อ้างอิง (Standards and Codes)**

ในการออกแบบโครงสร้างคอนกรีตเสริมเหล็กใช้ทฤษฎีกำลังประลัย (Ultimate Strength Design) สำหรับโครงสร้างเหล็กใช้วิธีหน่วยแรงที่ยอมให้ (Allowable Strength Design) โดยจะยึดถือตามกฎหมายที่เกี่ยวข้อง และมาตรฐานของสมาคมวิชาชีพ ดังนี้

- พระราชบัญญัติควบคุมอาคาร พ.ศ 2522 และกฎกระทรวงที่ออกตามความในพระราชบัญญัติฉบับนี้
- ข้อบัญญัติกรุงเทพมหานคร เรื่องควบคุมอาคาร พ.ศ. 2544
- มาตรฐานสำหรับอาคารคอนกรีตเสริมเหล็กโดยวิธีกำลังของวิศวกรรมสถานแห่งประเทศไทยในพระบรมราชูปถัมภ์
- มาตรฐานสำหรับอาคารเหล็กรูปพรรณ ของวิศวกรรมสถานแห่งประเทศไทยในพระบรมราชูปถัมภ์
- มาตรฐานประกอบการออกแบบอาคารเพื่อต้านทานการสั่นสะเทือนของแผ่นดินไหว มยผ.1302-52 กรมโยธาธิการและผังเมือง, กระทรวงมหาดไทย
- มาตรฐานประกอบการออกแบบอาคารเพื่อต้านทานการสั่นสะเทือนของแผ่นดินไหวปรับปรุงครั้งที่ 1 มยผ.1301-54 กรมโยธาธิการและผังเมือง, กระทรวงมหาดไทย
- มาตรฐานการออกแบบอาคารเพื่อต้านทานการสั่นสะเทือนของแผ่นดินไหว มยผ.1301/1302-61
- มาตรฐานการคำนวณแรงลมและการตอบสนองของอาคาร, มยผ.1311-50 กรมโยธาธิการและผังเมือง , กระทรวงมหาดไทย
- มาตรฐานการออกแบบอาคารต้านทานการสั่นสะเทือนของแผ่นดินไหว มยผ. 1301/1302-61
- American Society of Civil Engineer, Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7)
- American Society for Testing and Material (ASTM)
- Precast-Prestressed Concrete Institute (PCI)
- ACI Committee 318, "Building Code Requirement for Reinforced Concrete" (ACI 318)
- American Institute of Steel Construction, "Steel Construction Manual" (AISC)
- Uniform Building Code (UBC)
- ACI Committee 350, "Code Requirements for Environmental Engineering Concrete Structures"
- International Building Code (IBC)



2. กำลังวัสดุที่ใช้ในการออกแบบ

กำลังวัสดุที่ใช้ในการคำนวณหรือควบคุมคุณภาพวัสดุ จะใช้ค่าต่าง ๆ ดังต่อไปนี้

(1) คอนกรีต ค่ากำลังต้านทานแรงอัด (Compressive Strength: fc') ของคอนกรีตรูปทรงกระบอกมาตรฐานเส้นผ่าศูนย์กลาง 0.15 ม. สูง 0.30 ม. ที่อายุครบ 28 วัน มีค่าดังนี้

- คอนกรีตอัดแรง fc' ไม่น้อยกว่า 350 กก./ตร.ซม.
- คอนกรีตเสริมเหล็กทั่วไป fc' ไม่น้อยกว่า 350 กก./ตร.ซม.

(2) เหล็กเสริมและเหล็กรูปพรรณ ค่าแรงดึงที่จุดคลาก (Yield Strength: fy) ของเหล็กมีค่าดังนี้

- เหล็กข้ออ้อย ตามมาตรฐานผลิตภัณฑ์อุตสาหกรรม (มอก.) 24-2559 ของกระทรวงอุตสาหกรรม ชั้นคุณภาพ SD-40 มีค่า fy ไม่น้อยกว่า 4,000 กก./ตร.ซม.ซม. สำหรับเหล็กเสริม ขนาดเส้นผ่าศูนย์กลาง 12 มม. ถึง 28 มม.
- เหล็กข้ออ้อย ตามมาตรฐานผลิตภัณฑ์อุตสาหกรรม (มอก.) 24-2559 ของกระทรวงอุตสาหกรรม ชั้นคุณภาพ SD-50 มีค่า fy ไม่น้อยกว่า 5,000 กก./ตร.ซม. สำหรับเหล็กเสริม ขนาดเส้นผ่าศูนย์กลาง 32 มม.
- เหล็กเส้นกลม ตามมาตรฐานผลิตภัณฑ์อุตสาหกรรม (มอก.) 20-2559 ชั้น คุณภาพ SR-24 มีค่า fy ไม่น้อยกว่า 2,400 กก./ตร.ซม.
- เหล็กที่เกลียวกำลังสูงชนิด 7 เส้น (PC Strand) ตามมาตรฐานผลิตภัณฑ์อุตสาหกรรม (มอก.) 420-2540
- เหล็กรูปพรรณชนิด Hot-rolled of Cold rolled Mild Steel ชั้นคุณภาพ SS-400 ตามมาตรฐานอุตสาหกรรมไทย มอก.1227-2558 โดยมีค่าแรงดึงที่จุดคราก (Yield Strength: Fy) มีค่าไม่น้อยกว่า 2,350 กก./ ตร.ซม.
- เหล็กรูปพรรณชนิด Hot-rolled of Cold rolled Mild Steel ชั้นคุณภาพ HS-41 ตามมาตรฐานอุตสาหกรรมไทย มอก.107-2533 โดยมีค่าแรงดึงที่จุดคราก (Yield Strength: Fy) มีค่าไม่น้อยกว่า 2,350 กก./ ตร.ซม.

3. น้ำหนักบรรทุกใช้งานในการออกแบบ (Design Load)

(1) น้ำหนักบรรทุกคงที่ (Dead Load: DL)

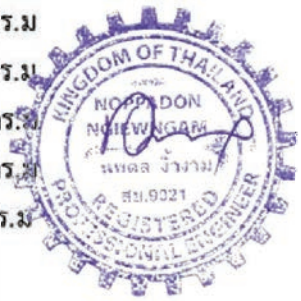
- น้ำหนักคอนกรีต	=	2400	กก./ลบ.ม.
- น้ำหนักเหล็กรูปพรรณ	=	7850	กก./ลบ.ม.
- น้ำหนักผนังก่ออิฐมวลเบา	=	100	กก./ตร.ม
- น้ำหนักผนังก่ออิฐบล็อก	=	180	กก./ตร.ม
- น้ำหนักผนังกระจก	=	100	กก./ตร.ม
- น้ำหนักผนังก่ออิฐมอญเต็มแผ่น	=	360	กก./ตร.ม
- น้ำหนักผนังก่ออิฐมอญครึ่งแผ่น	=	180	กก./ตร.ม
- กระเบื้องยางปูพื้น	=	5	กก./ตร.ม
- กระเบื้องเซรามิคปูพื้น	=	15-20	กก./ตร.ม
- ฝ้าเพดานยิบซั่มบอร์ดโครงเคร่าโลหะ	=	30	กก./ตร.ม
- น้ำหนักเครื่องจักรฯ รับวัสดุอื่น ๆ ให้คำนวณจากข้อมูลจริง			

(2) น้ำหนักบรรทุกคงที่เพิ่มเติม (Superimposed Dead Load: SDL)

- พื้นที่ลานอเนกประสงค์	=	120	กก./ตร.ม
- พื้นที่สำนักงาน	=	120	กก/ตร.ม
- ห้องประชุม, ห้องเอนกประสงค์, ภัตตาคาร	=	120	กก/ตร.ม

(3) น้ำหนักบรรทุกจร (Live Load: LL)

- หลังคาเบา	=	50	กก./ตร.ม
- กันสาดหรือหลังคาคอนกรีตเสริมเหล็ก	=	200	กก./ตร.ม
- พื้นที่สำนักงาน	=	300	กก./ตร.ม
- ห้องประชุม, ห้องอเนกประสงค์, ภัตตาคาร	=	400	กก./ตร.ม
- บันได ช่องทางเดินห้องประชุม, ภัตตาคาร	=	500	กก./ตร.ม
- ห้องเก็บของ (ไม่น้อยกว่า)	=	500	กก./ตร.ม
- ที่จอดรถยนต์	=	400	กก./ตร.ม
- พื้นที่อาคารศูนย์ประชุม	=	500	กก./ตร.ม
- ห้องไฟฟ้า	=	300*	กก./ตร.ม
- ห้อง AHU	=	500*	กก./ตร.ม
- ห้องหม้อแปลง, เครื่องกำเนิดไฟฟ้า,	=	2,000*	กก./ตร.ม
- Chiller, Cooling Tower			
- ห้องเครื่องลิฟต์	=	2,000*	กก./ตร.ม



หมายเหตุ น้ำหนักบรรทุกจร * ที่เสนอจะต้องตรวจสอบอีกครั้งเมื่อมีข้อมูลน้ำหนักที่แท้จริงของอุปกรณ์แล้ว

(4) แรงลม (Wind Load, WL)

4.1 ตามข้อบัญญัติกรุงเทพมหานคร เรื่องควบคุมอาคาร พ.ศ. 2544 กำหนดให้ค่าหน่วยแรงลมออกแบบมีความสัมพันธ์กับความสูงของอาคารและมีค่าไม่น้อยกว่าค่าที่ระบุในตารางต่อไปนี้

ความสูงของอาคารหรือส่วนของอาคาร	หน่วยแรงลมอย่างน้อย กิโลกรัมต่อตารางเมตร
ส่วนของอาคารที่สูงไม่เกิน 10 เมตร	50
ส่วนของอาคารที่สูงกว่า 10 เมตร แต่ไม่เกิน 20 เมตร	80
ส่วนของอาคารที่สูงกว่า 20 เมตร แต่ไม่เกิน 40 เมตร	120
ส่วนของอาคารที่สูงเกิน 40 เมตร แต่ไม่เกิน 80 เมตร	160
ส่วนของอาคารที่สูงเกินกว่า 80 เมตร	200

4.2 ตามมาตรฐานการคำนวณแรงลมและการตอบสนองของอาคาร (มยผ. 1311-50)

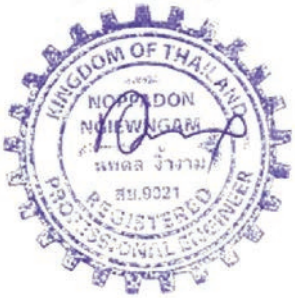
(5) การลดน้ำหนักบรรทุกจรของอาคาร

ในการคำนวณน้ำหนักที่ถ่ายลงเสาและฐานราก ให้ใช้น้ำหนักของอาคารเต็มอัตราจริงส่วนน้ำหนักบรรทุกจร (Live Load) ให้ลดส่วนลงได้ตามชั้นของอาคารดังต่อไปนี้

ตารางที่ 1 อัตราการลดน้ำหนักบรรทุกบนพื้นแต่ละชั้นของอาคาร

การรับน้ำหนักของพื้น	อัตราการลดน้ำหนักบรรทุกบนพื้นแต่ละชั้นเป็นร้อยละ
หลังคาหรือดาดฟ้า	0
ชั้นที่หนึ่งถัดจากหลังคาหรือดาดฟ้า	0
ชั้นที่สองถัดจากหลังคาหรือดาดฟ้า	0
ชั้นที่สามถัดจากหลังคาหรือดาดฟ้า	10
ชั้นที่สี่ถัดจากหลังคาหรือดาดฟ้า	20
ชั้นที่ห้าถัดจากหลังคาหรือดาดฟ้า	30
ชั้นที่หกถัดจากหลังคาหรือดาดฟ้า	40
ชั้นที่เจ็ดถัดจากหลังคาหรือดาดฟ้า และชั้นต่อลงไป	50

สำหรับห้องประชุม, ห้องเอนกประสงค์, ห้องเก็บของ, ที่จอดรถยนต์ให้คิดน้ำหนักบรรทุกเต็มอัตราทุกชั้น



(6) แรงแผ่นดินไหว (Earthquake Load, E)

ตามกฎกระทรวง พ.ศ. 2550 ได้กำหนดการรับน้ำหนัก ความต้านทาน ความคงทนของอาคารและพื้นดินที่รองรับอาคารในการต้านแรงสั่นสะเทือนของแผ่นดินไหว ตามความในพระราชบัญญัติควบคุมอาคาร พ.ศ. 2522 และมีผลบังคับใช้ตั้งแต่วันที่ 30 พฤศจิกายน 2550 ซึ่งกฎกระทรวงดังกล่าวได้แก้ไข เพิ่มเติมและยกเลิกกฎกระทรวงฉบับที่ 49 (พ.ศ. 2540) ตามความในพระราชบัญญัติควบคุมอาคาร พ.ศ. 2522 ว่าด้วยการออกแบบโครงสร้างจะต้องออกแบบรับแรงแผ่นดินไหว เนื่องจากโครงการอยู่ในเขตที่อาคารออกแบบอาคารรับแรงเนื่องจากแผ่นดินไหว ซึ่งตามกฎกระทรวงที่กำหนดให้พื้นที่หรือบริเวณเฝ้าระวัง บริเวณที่ 1 และบริเวณที่ 2 สำหรับอาคารที่จำเป็นต่อความเป็นอยู่ของสาธารณชนจะต้องออกแบบโครงสร้างต้านทานแรงแผ่นดินไหว

"บริเวณเฝ้าระวัง" หมายถึง พื้นที่หรือบริเวณที่อาจได้รับผลกระทบจากแผ่นดินไหว ได้แก่ จังหวัดกระบี่ จังหวัดชุมพร จังหวัดพังงา จังหวัดภูเก็ต จังหวัดระนอง จังหวัดสงขลา และจังหวัดสุราษฎร์ธานี

"บริเวณที่ 1" หมายถึง พื้นที่หรือบริเวณที่เป็นดินอ่อนมากที่อาจได้รับผลกระทบจากแผ่นดินไหวระยะไกล ได้แก่ กรุงเทพมหานคร จังหวัดนนทบุรี จังหวัดปทุมธานี จังหวัดสมุทรปราการ และจังหวัดสมุทรสาคร

"บริเวณที่ 2" หมายถึง พื้นที่หรือบริเวณที่อยู่ใกล้รอยเลื่อนที่อาจได้รับผลกระทบจากแผ่นดินไหว ได้แก่ จังหวัดกาญจนบุรี จังหวัดเชียงราย จังหวัดเชียงใหม่ จังหวัดตาก จังหวัดน่าน จังหวัดพะเยา จังหวัดแพร่ จังหวัดแม่ฮ่องสอน จังหวัดลำปาง และจังหวัดลำพูน

ซึ่งโครงการก่อสร้างนี้ตั้งอยู่ในจังหวัดกรุงเทพมหานคร เป็นเขตเสี่ยงต่อแผ่นดินไหวอยู่ใน บริเวณที่ 1 จำเป็นต้องออกแบบระบบโครงสร้างที่คำนึงถึง แรงเนื่องจากแผ่นดินไหวด้วย โดยการออกแบบอาคารให้อ้างอิงตามมาตรฐานประกอบการออกแบบอาคารเพื่อต้านการสั่นสะเทือนของแผ่นดินไหว ปี พ.ศ. 2552 (มยผ. 1302-52) , ปี พ.ศ. 2554 (มยผ. 1301-54) และ ปี พ.ศ. 2561 (มยผ. 1301/1302-61)

(7) แรงเนื่องจากการทรุดตัวที่แตกต่างกัน (Differential Settlement, S)

เป็นแรงกระทำต่อโครงสร้างเนื่องจากการทรุดตัวที่แตกต่างกันของฐานรากอาคาร ซึ่งในการออกแบบโครงสร้างอาคารนี้จะพิจารณาถึงผลอันเนื่องมาจากการทรุดตัวที่แตกต่างกันของฐานรากที่ไม่ก่อให้เกิดการแตกร้าวของโครงสร้างอาคาร

(8) แรงดันดิน (H)

เป็นแรงกระทำต่อโครงสร้างทางด้านข้างในการออกแบบโครงสร้างอาคารจะพิจารณาถึงผลกระทบที่เกิดจากแรงดันดินดังกล่าว ดังนี้

For Sand or Sandy Clay

Ka = tan2 (45 - ϕ/2) Coefficient to active earth pressure.

Ko = tan2 (45+ ϕ/2) Coefficient of passive earth pressure.

Kp = 1 – sin ϕ Coefficient of earth pressure at rest.

ϕ = Angle of internal friction

For clay, ϕ = 0

Active soil pressure, Pa = $\gamma h - 2C$

Passive soil pressure, Pp = $\gamma h + 2C$

γ = Density of Soil

C = Co-efficient of Cohesion

Unless otherwise noted, the lateral soil pressure coefficients are:

Active soil pressure coefficient, Ka = 0.33

At-rest soil pressure coefficient, Ko = 0.50

Passive soil pressure coefficient, Kp = 3.00

(9) แรงเนื่องจากการเปลี่ยนแปลงของอุณหภูมิ (Temperature Change, T)

เป็นแรงกระทำที่ทำให้เกิดการยืดหดตัวของโครงสร้าง โดยการเปลี่ยนแปลงของอุณหภูมิจะอยู่ที่ ±15 องศาเซลเซียส ในการออกแบบโครงสร้างอาคารซึ่งเป็นอาคารช่วงยาว จะพิจารณาถึงผลที่เกิดจากการเปลี่ยนแปลงของอุณหภูมิที่ไม่ก่อให้เกิดการแตกร้าวของโครงสร้างอาคาร

4. การรวมน้ำหนักบรรทุกต่าง ๆ

(1) การออกแบบอาคารคอนกรีตเสริมเหล็กโดยวิธีกำลัง (Ultimate Strength Design) ตามกฎกระทรวงและมาตรฐาน (ACI 318)

สัญลักษณ์ (Symbols)

DL	=	ผลที่เกิดจากน้ำหนักบรรทุกคงที่ (Dead Load)
LL	=	ผลที่เกิดจากน้ำหนักบรรทุกจร (Live Load)
WL	=	ผลที่เกิดจากแรงลม (Wind Load)
E	=	ผลที่เกิดจากแรงแผ่นดินไหว (Seismic Load Effects)

กำลังที่ต้องการ

U	=	1.7DL + 2.0 LL	(กฎกระทรวงฉบับที่ 6 พ.ศ.2527)
U	=	0.75(1.7DL + 2.0LL + 2.0WL)	(กฎกระทรวงฉบับที่ 6 พ.ศ.2527)
U	=	0.90DL + 1.30WL	(กฎกระทรวงฉบับที่ 6 พ.ศ.2527)
U	=	0.75(1.7DL + 2.0LL + 2.0EQ)	(กฎกระทรวง พ.ศ.2550)
U	=	0.90DL + 1.3EQ	(กฎกระทรวง พ.ศ.2550)
U	=	1.2DL + 1.0LL + 1.0EQ	(มยผ.1302-52 และ มยผ. 1302-54)
U	=	0.75(1.4DL + 1.7LL) + 1.0EQ	(มยผ.1301/1302-61)
U	=	0.90DL + 1.0EQ	(มยผ.1302-52,มยผ. 1302-54 และ มยผ. 1301/1302-61)
U	=	1.4DL + 1.7 LL	(EIT 1008-38)
U	=	0.75(1.4DL + 1.7LL + 1.7WL)	(EIT 1008-38)
U	=	0.75(1.4DL + 1.7LL + 1.87EQ)	(EIT 1008-38)
U	=	0.90DL + 1.43EQ	(EIT 1008-38)

ตัวคูณลดกำลัง (Strength Reduction Factor)

- สำหรับแรงดัด	0.90
- สำหรับแรงเฉือนและแรงบิด	0.85
- สำหรับแรงอัดตามแนวแกน, เหล็กปลอกเดี่ยว	0.70
- สำหรับแรงอัดตามแนวแกน, เหล็กปลอกเกลียว	0.75

Flexural Strength Design

$$\rho_b = 0.85 \times \beta_1 \times \frac{fc'}{f_y} \times \frac{6120}{(6120 + f_y)}$$

$$\beta_1 = 0.85 - 0.05 \times \frac{(fc' - 280)}{70}; \beta_1 < 0.85$$

$$\rho_{max} = 0.75\rho_b$$

$$\rho_{min} = \frac{14}{f_y}$$; for beam and footing

$$= 0.0018$$; for slab where Grade SD40 deformed bars are used

$$= 0.0025$$; for slab where Grade SR24 round bars are used

$$R = 0.75\rho_b f_y \left(1 - 0.44\rho_b \frac{f_y}{fc'}\right)$$

$$\phi M_n = \phi R b d^2$$

For Single reinforcement: $(M_u < \phi M_n)$

$$\rho = 0.85\frac{fc'}{f_y} \times \left(1 - \sqrt{1 - \frac{236 M_u}{\sqrt{\phi b d^2 fc'}}}\right)$$

Shear Strength Design

For beam shear strength

$$V_c = 0.53\sqrt{fc'}.bd$$

For punching shear strength

$$V_c = 1.06\sqrt{fc'}.b_o d$$

Compressive Strength Design

$$\phi V_n = 0.80\phi\left(0.85fc'(A_g - A_s) + A_s f_y\right)$$

Where

ϕ = Strength reduction factor

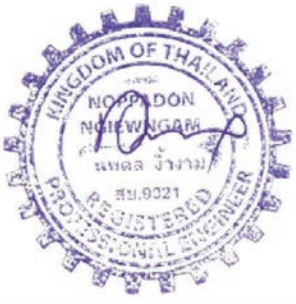
= 0.90 for flexure

= 0.85 for shear

= 0.70 for axial compression

Where

Ag	=	Gross area of member (cm^2)
As	=	Area of reinforcing steel (cm^2)
M_u	=	Ultimate moment (kg-m)
M_n	=	Nominal flexural strength (kg-m)
P_n	=	Nominal Compressive strength (kg)
R	=	Flexural resistance factor (kg/cm^2)
V_u	=	Ultimate shear force (kg)
V_c	=	Concrete shear strength (kg)
B	=	member width (cm)
b_o	=	Critical perimeter (cm)
d	=	Effective depth of member (cm)
fc'	=	Compressive strength of concrete (kg/cm^2)
fy	=	Yield strength of reinforcing steel (kg/cm^2)
h	=	Depth of member (cm)
β_1	=	Concrete stress block parameter
ρ	=	Reinforcement ratio = As / bd
ρ_b	=	Balanced steel ratio



(2) การออกแบบโครงสร้างเหล็กโดยวิธีหน่วยแรงที่ยอมให้ (Allowable Stress Design-ASD) ตามกฎกระทรวงและมาตรฐาน (AISC)

สัญลักษณ์ (Symbols)

DL	=	ผลที่เกิดจากน้ำหนักบรรทุกคงที่ (Dead Load)
LL	=	ผลที่เกิดจากน้ำหนักบรรทุกจร (Live Load)
WL	=	ผลที่เกิดจากแรงลม (Wind Load)
E	=	ผลที่เกิดจากแรงแผ่นดินไหว (Seismic Load Effects)

กำลังที่ต้องการ

U	=	DL + LL	
U	=	0.75(DL + LL + WL)	
U	=	0.75(DL + LL + EQ)	
U	=	1.0DL + 0.7EQ	(มยผ.1302-52 และ มยผ. 1301/1302-61)
U	=	1.0DL + 0.75LL + 0.525EQ	(มยผ.1302-52 และ มยผ. 1301/1302-61)
U	=	0.6DL + 0.7EQ	(มยผ.1302-52 และ มยผ. 1301/1302-61)

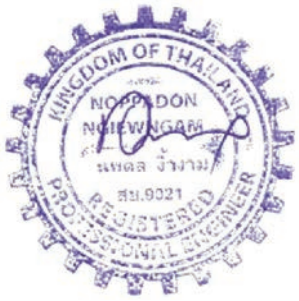
Allowable Flexural Stress

1. Members with Unbraced length not exceed the value of L_c

$$F_b = 0.66F_y$$

Where $L_c = \frac{637b_f}{\sqrt{F_y}}$ or $\frac{1406000}{(d/A_f)F_y}$

Whichever is smaller



2. Members with Unbraced length greater than Lc

When $\sqrt{\frac{717x10^4C_b}{F_y}} < \frac{I}{r_T} < \sqrt{\frac{3585x10^4C_b}{F_y}}$

$$F_b = \left(\frac{2}{3} - \frac{F_y(I/r_T)^2}{10755x10^4C_b}\right)F_y < 0.60F_y$$

When $\frac{I}{r_T} = \sqrt{\frac{3585x10^4C_b}{F_y}}$

$$F_b = \frac{1195x10^4C_bF_y}{(I/r_T)^2} < 0.60F_y$$

Allowable Shear Stress

$$F_v = 0.40F_y$$

Allowable Compressive Stress

When $\frac{KL}{r} < C_c$

$$F_a = \frac{\left(1-\frac{\left(\frac{KL}{r}\right)^2}{2C_c^{\,2}}\right)F_y}{\frac{5}{8}+\frac{3}{8}\frac{\frac{KL}{r}}{C_c}-\frac{\left(\frac{KL}{r}\right)^3}{8C_c^{\,3}}}$$

Where $C_c = \sqrt{\frac{2\pi^2E}{F_y}}$

When $\frac{KL}{r} > C_c$

$$F_a = \frac{12\pi^2E}{23\left(\frac{KL}{r}\right)^2}$$

Allowable Tensile Stress

$F_t = 0.60F_y$

Where

A_f	=	Area of the compression flange (cm^2)
C_b	=	$1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2$, but not more than 2.3, where M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M1/M2, the ratio of end moments, is positive when M1 and M2 have the same sign and negative when they are of opposite signs. When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the value of C_b shall be taken as unity.
E	=	Modulus of elasticity of steel (kg/cm^2)
F_a	=	Allowable axial compressive stress (kg/cm^2)
F_b	=	Allowable bending stress (kg/cm^2)
F_t	=	Allowable axial tensile stress (kg/cm^2)
F_v	=	Allowable shear stress (kg/cm^2)
Fy	=	Specified minimum yield stress of the type of steel being used (kg/cm^2)
K	=	Effective length factor
b_f	=	Flange width of rolled beam (cm)
d	=	depth of beam (cm)
l	=	Unbraced length of member (cm)
r	=	radius of gyration (cm)
r_T	=	radius of gyration of a section comprising the compression flange plus 1/3 of the compression web area, taken about an axis in the plane of the web (cm).

2. รายการคำนวณเสา, กำแพง

TABLE: Column Forces C56 (2nd Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
2 SFL	C21	AComb31-1	-161	0	0	4	0
2 SFL	C84	AComb2-1	-1596	26	12	38	44
2 SFL	C38	AComb2-1	-1152	52	113	322	149
2 SFL	C38	AComb2-1	-1152	52	113	322	149
2 SFL	C38	AComb2-1	-1152	52	113	322.474	149
2 SFL	C38	AComb2-1	-1152	52	113	322	149

TABLE: Column Forces C56 (5th Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
5 SFL	C15	AComb31-1	-44	0	0	0	0
3 SFL	C64	AComb2-1	-1489	0	1	1	0
3 SFL	C42	AComb2-1	-1332	70	0	0	133
3 SFL	C41	AComb2-1	-1021	0	96	201	0
3 SFL	C41	AComb2-1	-1021	0	96	201.019	0
3 SFL	C42	AComb2-1	-1332	70	0	0	133

TABLE: Column Forces C56 (Roof Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
8 SFL	C36	AComb2-1	15	10	48	8	6
6 SFL	C64	AComb2-1	-936	0	41	57	0
8 SFL	C84	AComb2-1	-276	94	94	210	104
7 SFL	C63	AComb2-1	-342	0	118	418	0
7 SFL	C83	AComb2-1	-320	18	115	513.201	87
7 SFL	C29	AComb2-1	-243	62	4	3	343

spColumn

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Computer program for the Strength Design of Reinforced Concrete Sections
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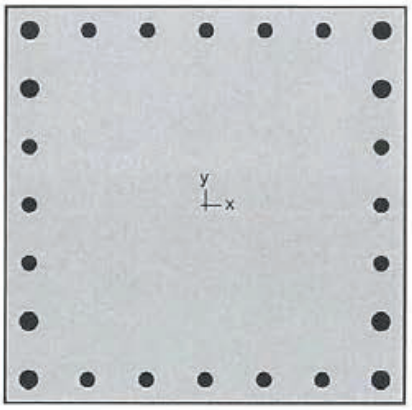

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List of Figures

Figure 1: Column section 4

1. General Information

File Name	c:\193018\gcdt\spcloumn\10-11-63(c...\c56-5th.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	35 MPa
E_c	27805.6 MPa
f_c	29.75 MPa
ε_u	0.003 mm/mm
β_1	0.786192

2.2. Steel

Type	Standard
f_y	400 MPa
E_s	200000 MPa
ε_{yt}	0.002 mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴
r_x	288.675 mm
r_y	288.675 mm
X_o	0 mm
Y_o	0 mm

3.2. Section Figure

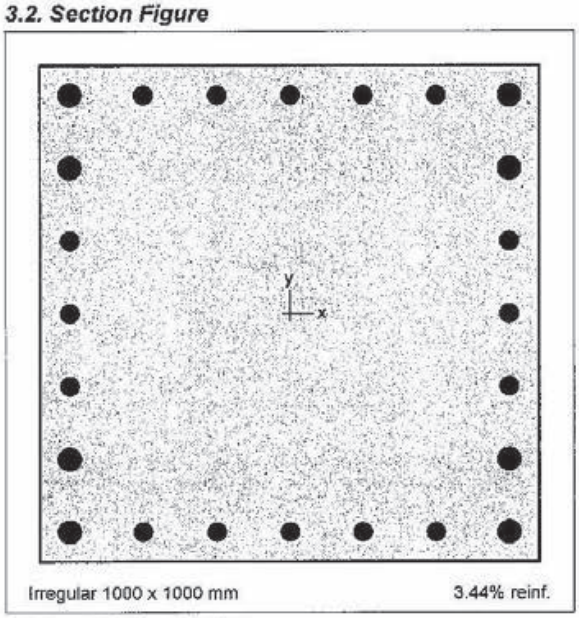


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-500.0	-500.0	2	500.0	-500.0	3	500.0	500.0
4	-500.0	500.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---

Bars	---
Total steel area, A_s	34440 mm²
Rho	3.44 %
Minimum clear spacing	98 mm

4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
1845.0	-440.2	440.2	1230.0	-293.5	440.2	1230.0	-146.7	440.2
1230.0	0.0	440.2	1230.0	146.7	440.2	1230.0	293.5	440.2
1845.0	440.2	440.2	1845.0	440.2	293.5	1230.0	440.2	146.7
1230.0	440.2	0.0	1230.0	440.2	-146.7	1845.0	440.2	-293.5
1845.0	440.2	-440.2	1230.0	293.5	-440.2	1230.0	146.7	-440.2
1230.0	0.0	-440.2	1230.0	-146.7	-440.2	1230.0	-293.5	-440.2
1845.0	-440.2	-440.2	1845.0	-440.2	-293.5	1230.0	-440.2	-146.7
1230.0	-440.2	0.0	1230.0	-440.2	146.7	1845.0	-440.2	293.5

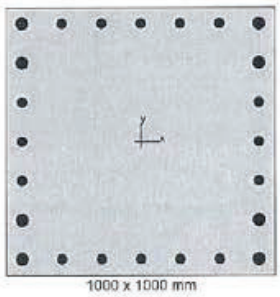
5. Factored Loads and Moments with Corresponding Capacity Ratios

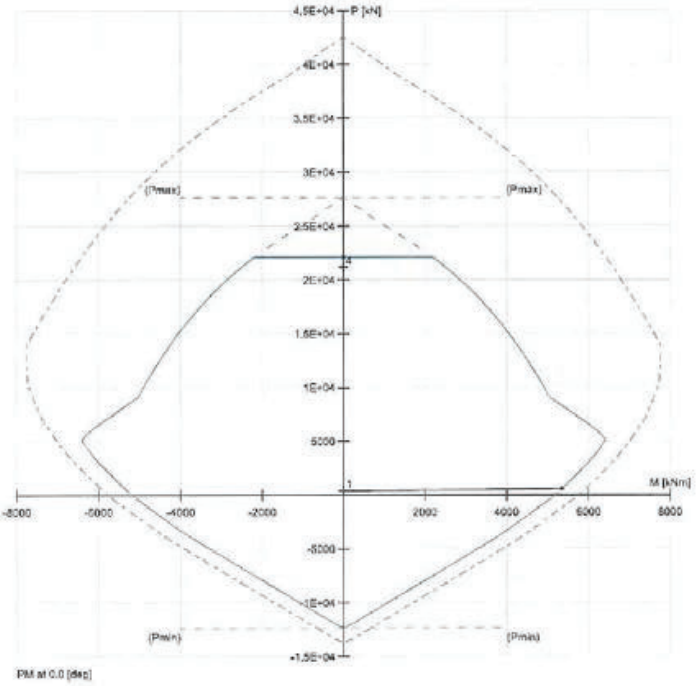
NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	NA Depth mm	ε_t	ϕ	
1	430.00	0.00	0.00	563.79	5352.52	0.00	218	0.00994	0.900	0.17
2	15950.00	383.00	440.00	22100.73	383.00	440.00	1326	0.00000	0.650	0.72
3	11470.00	3300.00	1530.00	12300.44	3721.09	1725.23	868	0.00139	0.650	0.90
4	21190.00	0.00	0.00	22100.73	0.00	0.00	1081	-0.00039	0.650	0.96

6. Diagrams

6.1. PM at θ=0 [deg]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	34440 mm²
Rho	3.44 %
Min. clear spacing	98 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	21190.0	0.0	0.0	22100.73	0.00	0.00	0.96
1	430.0	0.0	0.0	563.79	5352.52	0.00	0.17

Max. Capacity Ratio: 0.96

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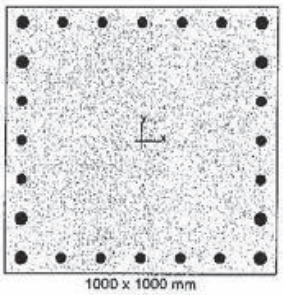
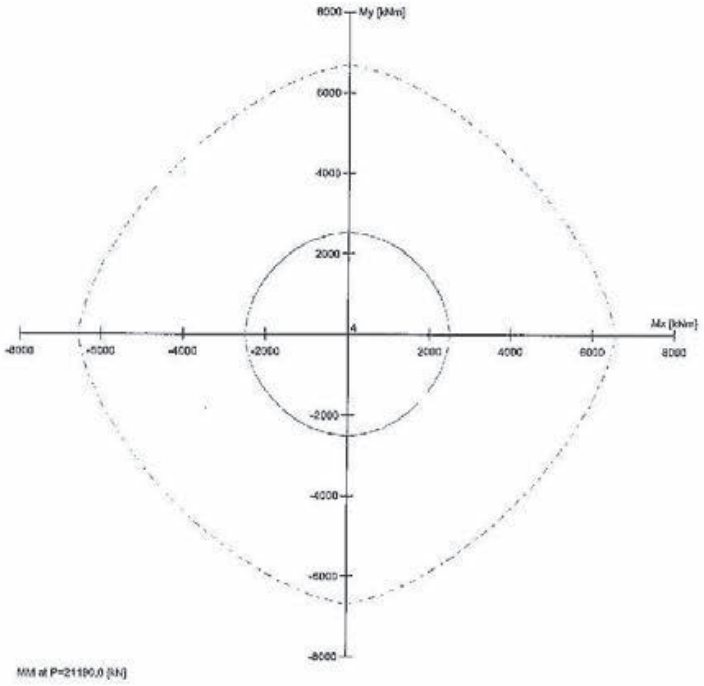
6.2. MM at P=21190 [kN]





General Information	
Project	—
Column	—
Engineer	—
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

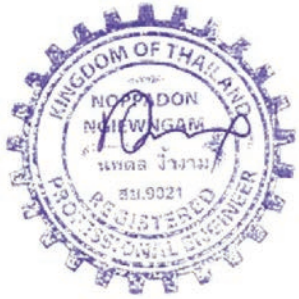
Section	
Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	—
Cover to	—
Clear cover	—
Bars	—

Confinement type	Tied
Total steel area, A_s	34440 mm²
Rho	3.44 %
Min. clear spacing	98 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	21190.0	0.0	0.0	22100.73	0.00	0.00	0.96

Max. Capacity Ratio: 0.96



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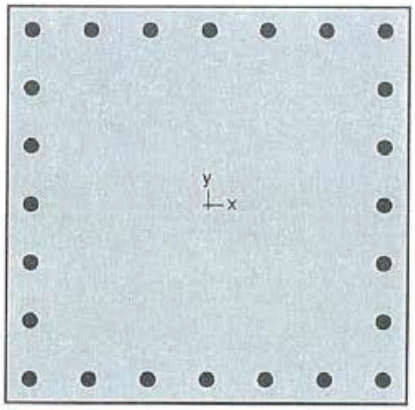

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1. General Information

File Name	c:\193018\gcdt\spcloumn\10-11-63(...\c56-roof.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

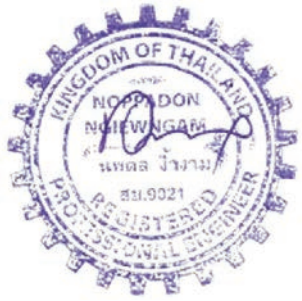
2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	1e+006	mm²
I_x	8.33333e+010	mm⁴
I_y	8.33333e+010	mm⁴
r_x	288.675	mm
r_y	288.675	mm
X_o	0	mm
Y_o	0	mm



3.2. Section Figure

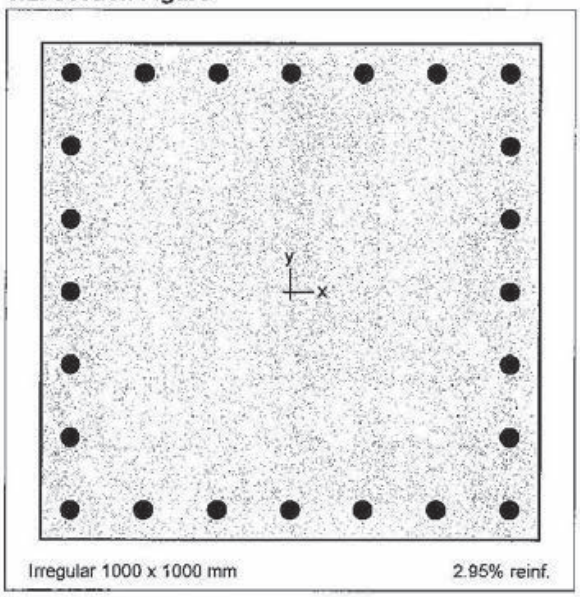


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-500.0	-500.0	2	500.0	-500.0	3	500.0	500.0
4	-500.0	500.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---

Bars	---
Total steel area, A_s	29520 mm²
Rho	2.95 %
Minimum clear spacing	107 mm

4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
1230.0	-440.2	440.2	1230.0	-293.5	440.2	1230.0	-146.7	440.2
1230.0	0.0	440.2	1230.0	146.7	440.2	1230.0	293.5	440.2
1230.0	440.2	440.2	1230.0	440.2	293.5	1230.0	440.2	146.7
1230.0	440.2	0.0	1230.0	440.2	-146.7	1230.0	440.2	-293.5
1230.0	440.2	-440.2	1230.0	293.5	-440.2	1230.0	146.7	-440.2
1230.0	0.0	-440.2	1230.0	-146.7	-440.2	1230.0	-293.5	-440.2
1230.0	-440.2	-440.2	1230.0	-440.2	-293.5	1230.0	-440.2	-146.7
1230.0	-440.2	0.0	1230.0	-440.2	146.7	1230.0	-440.2	293.5

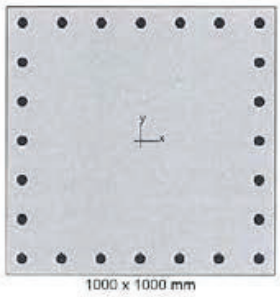
5. Factored Loads and Moments with Corresponding Capacity Ratios

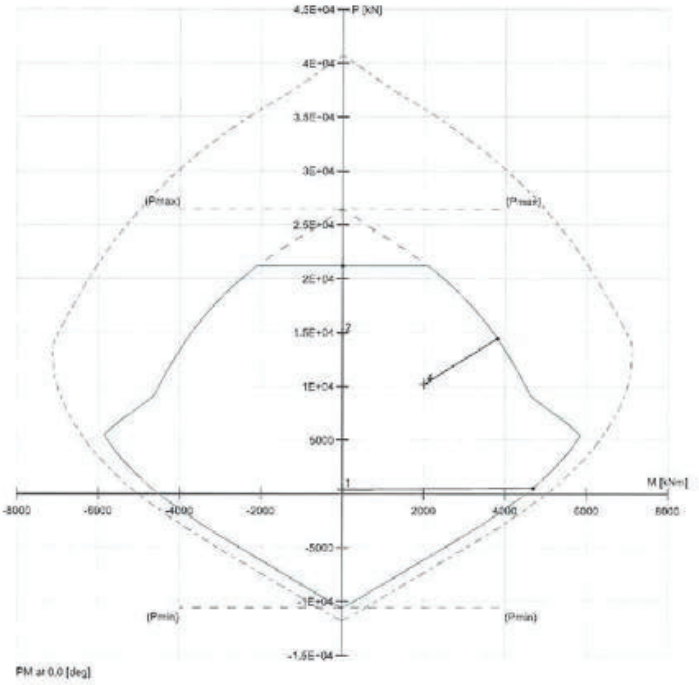
NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	NA Depth mm	ε_t	ϕ	
1	440.00	0.00	0.00	539.63	4697.32	0.00	203	0.01088	0.900	0.20
2	14890.00	0.00	0.00	21153.48	0.00	0.00	1071	-0.00037	0.650	0.70
3	13320.00	0.00	1330.00	18675.79	0.00	2856.14	949	-0.00003	0.650	0.64
4	10210.00	2010.00	0.00	14427.17	3804.18	0.00	760	0.00071	0.650	0.63

6. Diagrams

6.1. PM at θ=0 [deg]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

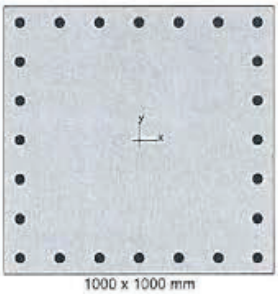
Section	
Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

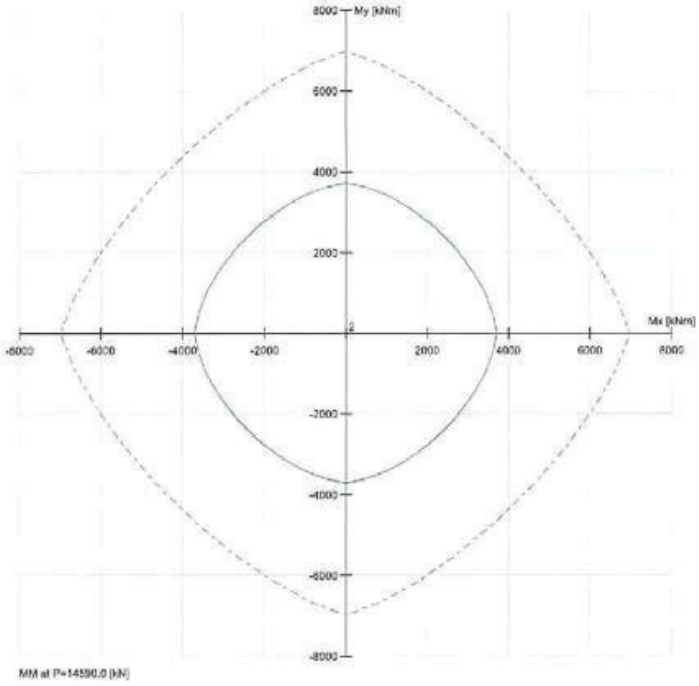
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	29520 mm²
Rho	2.95 %
Min. clear spacing	107 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
2	14890.0	0.0	0.0	21153.48	0.00	0.00	0.70
4	10210.0	2010.0	0.0	14427.17	3804.18	0.00	0.63
1	440.0	0.0	0.0	538.63	4697.32	0.00	0.20

Max. Capacity Ratio: 0.70

6.2. MM at P=14890 [kN]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	29520 mm²
Rho	2.95 %
Min. clear spacing	107 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
2	14890.0	0.0	0.0	21153.48	0.00	0.00	0.70

Max. Capacity Ratio: 0.70

TABLE: Column Forces C57 (2nd to Roof Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
3 SFL	C55	AComb31-1	107	20	0	3	28
2 SFL	C54	AComb2-1	-726	25	0	1	53
8 SFL	C54	AComb30-1	-78	42	2	1	91
R	C58	AComb30-1	-42	3	41	69	11
R	C58	AComb30-1	-32	3	41	81	1
8 SFL	C54	AComb30-1	-78	42	2	1	91

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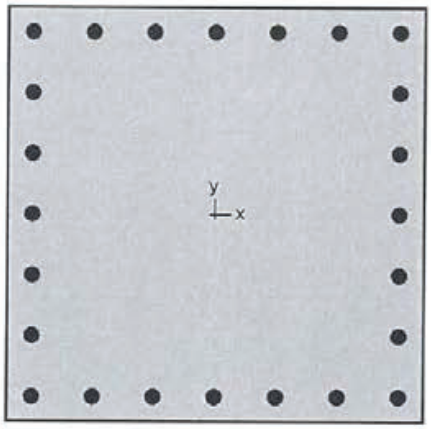

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1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63(...\C57-Roof.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	1e+006	mm^2
I_x	8.33333e+010	mm^4
I_y	8.33333e+010	mm^4
r_x	288.675	mm
r_y	288.675	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure

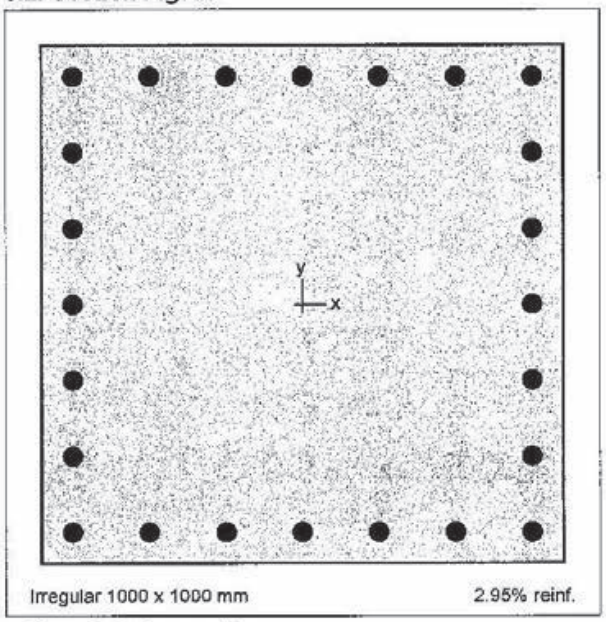


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-500.0	-500.0	2	500.0	-500.0	3	500.0	500.0
4	-500.0	500.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---

Bars	—
Total steel area, A_s	29520 mm²
Rho	2.95 %
Minimum clear spacing	107 mm

4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
1230.0	-440.2	440.2	1230.0	-293.5	440.2	1230.0	-146.7	440.2
1230.0	0.0	440.2	1230.0	146.7	440.2	1230.0	293.5	440.2
1230.0	440.2	440.2	1230.0	440.2	293.5	1230.0	440.2	146.7
1230.0	440.2	0.0	1230.0	440.2	-146.7	1230.0	440.2	-293.5
1230.0	440.2	-440.2	1230.0	293.5	-440.2	1230.0	146.7	-440.2
1230.0	0.0	-440.2	1230.0	-146.7	-440.2	1230.0	-293.5	-440.2
1230.0	-440.2	-440.2	1230.0	-440.2	-293.5	1230.0	-440.2	-146.7
1230.0	-440.2	0.0	1230.0	-440.2	146.7	1230.0	-440.2	293.5

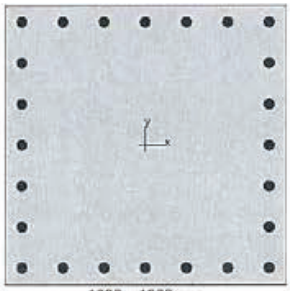
5. Factored Loads and Moments with Corresponding Capacity Ratios

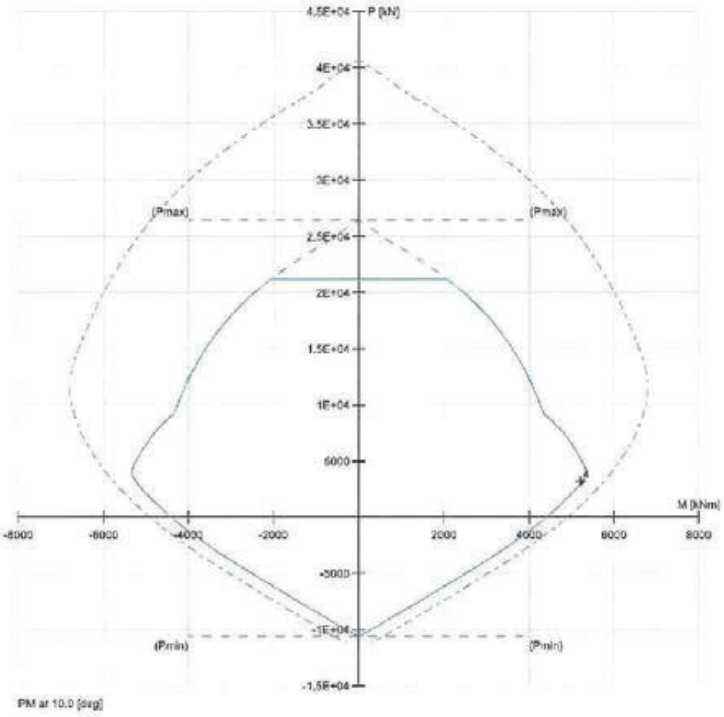
NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	NA Depth mm	ε_t	ϕ	
1	150.00	80.00	60.00	0.00	3439.96	2579.97	470	0.00537	0.900	0.06
2	9360.00	570.00	0.00	21153.48	570.00	0.00	1071	-0.00037	0.650	0.44
3	2760.00	2100.00	1040.00	0.00	3881.30	1922.17	419	0.00591	0.900	0.56
4	3200.00	5130.00	870.00	3088.69	5164.32	875.82	391	0.00542	0.900	0.99
5	2430.00	30.00	3430.00	0.00	39.44	4509.71	190	0.01209	0.900	0.78

6. Diagrams

6.1. PM at θ=10 [deg]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials		
f'_c	35	MPa
E_c	27805.6	MPa
f_y	400	MPa
E_s	200000	MPa

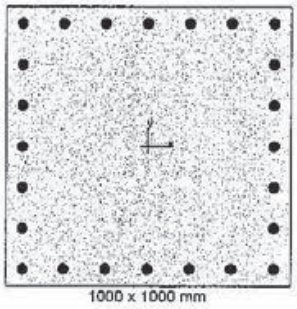
Section		
Type	Irregular	
A_g	1e+006	mm²
I_x	8.33333e+010	mm⁴
I_y	8.33333e+010	mm⁴

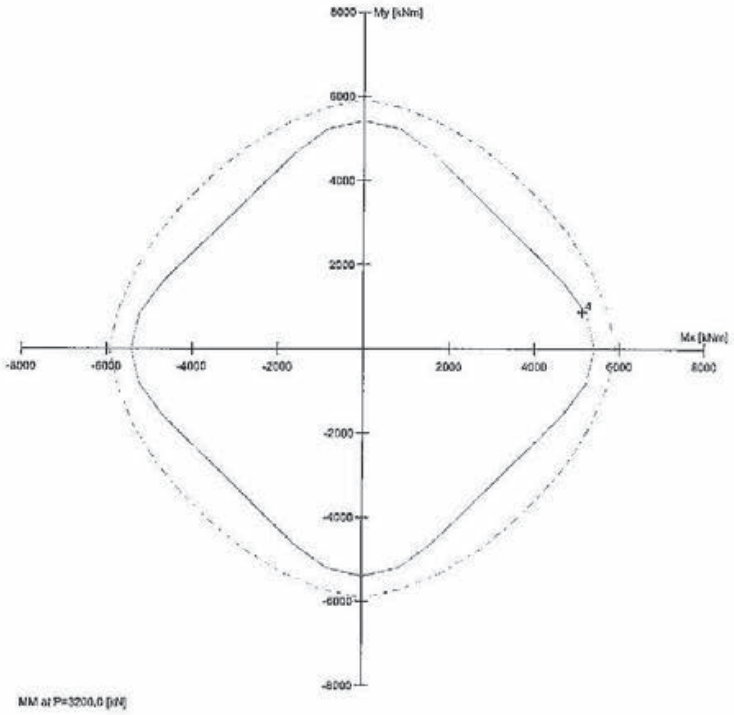
Reinforcement		
Pattern	Irregular	
Bar layout	---	
Cover to	---	
Clear cover	---	
Bars	---	
Confinement type	Tied	
Total steel area, A_s	29520	mm²
Rho	2.95	%
Min. clear spacing	107	mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	3200.0	5130.0	870.0	3088.69	5164.32	875.82	0.99

Max. Capacity Ratio: 0.99

6.2. MM at P=3200 [kN]





General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials

f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section

Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	29520 mm²
Rho	2.95 %
Min. clear spacing	107 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	3200.0	5130.0	870.0	3088.69	5164.32	875.82	0.99

Max. Capacity Ratio: 0.99

TABLE: Column Forces C58 (2nd to Roof Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
2 SFL	C57	AComb34-1	173	4	0	0	5
B2	C49	AComb2-1	-1454	1	0	0	0
3 SFL	C86	AComb30-1	-96	32	0	0	51
2 SFL	C48	AComb2-1	-972	15	28	61	25
2 SFL	C48	AComb2-1	-972	15	28	61	25
R	C48	AComb2-1	-127	28	6	9	62

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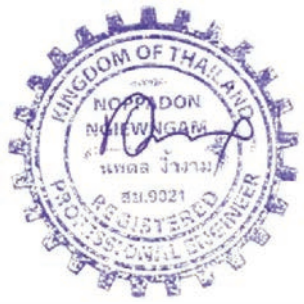


1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63(...\C58-Roof.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slendemess	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	35 MPa
E_c	27805.6 MPa
f_c	29.75 MPa
ε_u	0.003 mm/mm
β_1	0.796192

2.2. Steel

Type	Standard
f_y	400 MPa
E_s	200000 MPa
ε_{yt}	0.002 mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴
r_x	288.675 mm
r_y	288.675 mm
X_o	0 mm
Y_o	0 mm

3.2. Section Figure

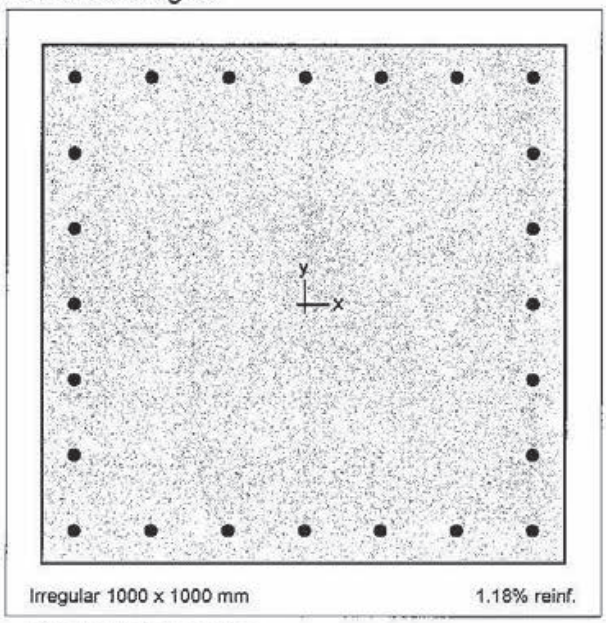


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-500.0	-500.0	2	500.0	-500.0	3	500.0	500.0
4	-500.0	500.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---

Bars	---
Total steel area, A_s	11784 mm²
Rho	1.16 %
Minimum clear spacing	121 mm

4.4. Bars Provided

Area	X	Y	Area	X	Y	Area	X	Y
mm²	mm	mm	mm²	mm	mm	mm²	mm	mm
491.0	-437.5	437.5	491.0	-291.7	437.5	491.0	-145.8	437.5
491.0	0.0	437.5	491.0	145.8	437.5	491.0	291.7	437.5
491.0	437.5	437.5	491.0	437.5	291.7	491.0	437.5	145.8
491.0	437.5	0.0	491.0	437.5	-145.8	491.0	437.5	-291.7
491.0	437.5	-437.5	491.0	291.7	-437.5	491.0	145.8	-437.5
491.0	0.0	-437.5	491.0	-145.8	-437.5	491.0	-291.7	-437.5
491.0	-437.5	-437.5	491.0	-437.5	-291.7	491.0	-437.5	-145.8
491.0	-437.5	0.0	491.0	-437.5	145.8	491.0	-437.5	291.7

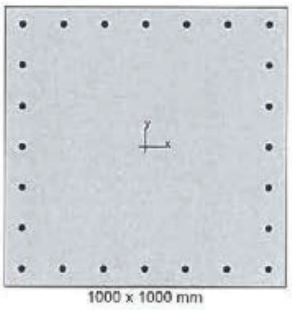
5. Factored Loads and Moments with Corresponding Capacity Ratios

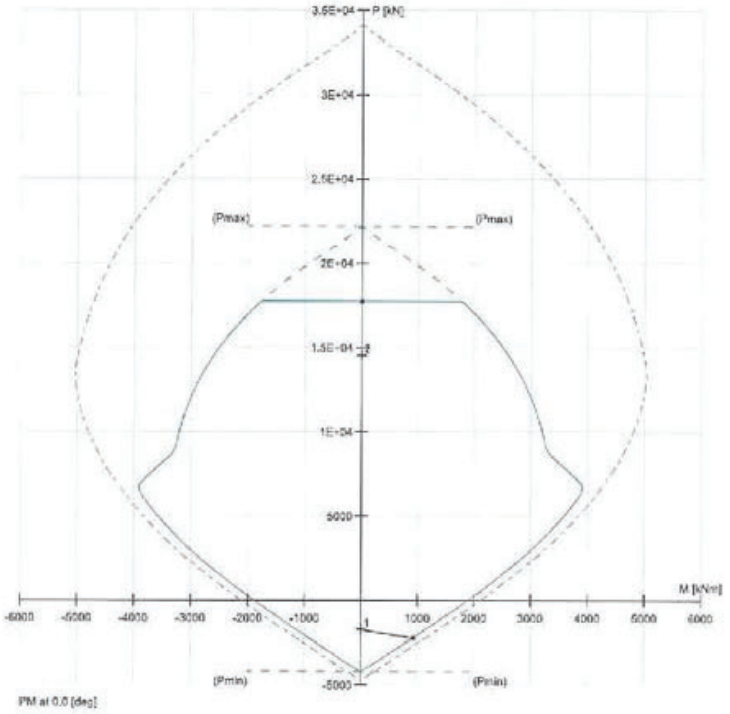
NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kN	kNm	kNm	kN	kNm	kNm	mm			
1	-1730.00	0.00	0.00	-2216.81	932.83	0.00	53	0.05021	0.900	0.74
2	14540.00	0.00	0.00	17738.77	0.00	0.00	1035	-0.00028	0.650	0.82
3	9720.00	610.00	250.00	17738.77	610.00	250.00	1186	0.00001	0.650	0.55
4	1270.00	90.00	620.00	0.00	271.85	1872.78	158	0.01792	0.900	0.64

6. Diagrams

6.1. PM at θ=0 [deg]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

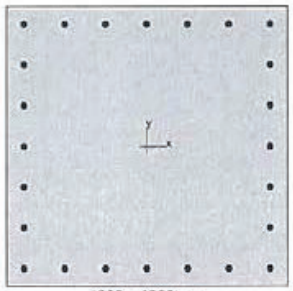
Section	
Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

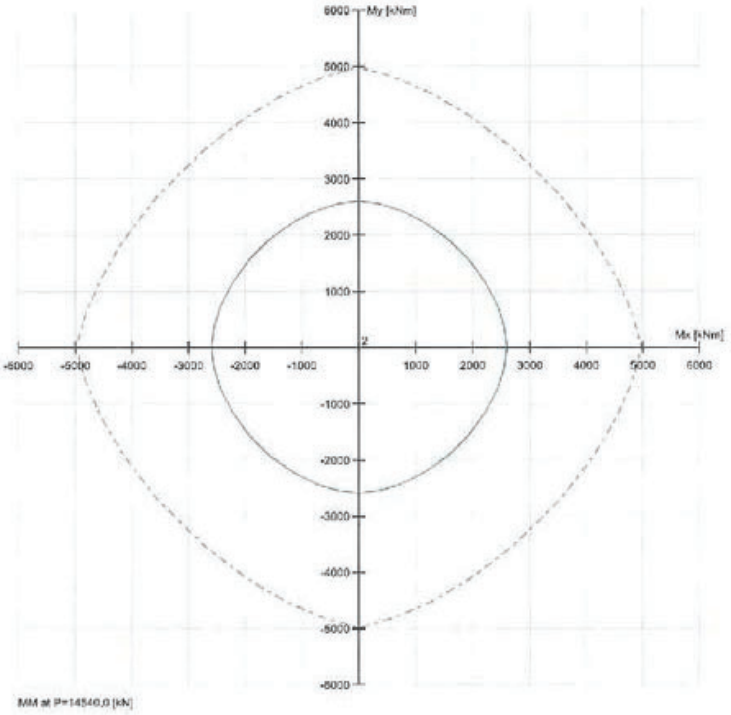
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	11784 mm²
Rho	1.18 %
Min. clear spacing	121 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
2	14540.0	0.0	0.0	17738.77	0.00	0.00	0.82
1	-1730.0	0.0	0.0	-2216.81	932.83	0.00	0.74

Max. Capacity Ratio: 0.82

6.2. MM at P=14540 [kN]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1e+006 mm²
I_x	8.33333e+010 mm⁴
I_y	8.33333e+010 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	11784 mm²
Rho	1.18 %
Min. clear spacing	121 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
2	14540.0	0.0	0.0	17738.77	0.00	0.00	0.82

Max. Capacity Ratio: 0.82

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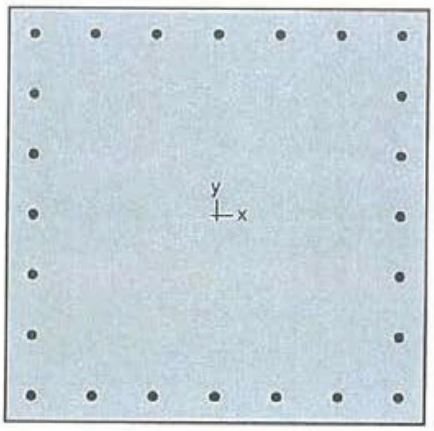

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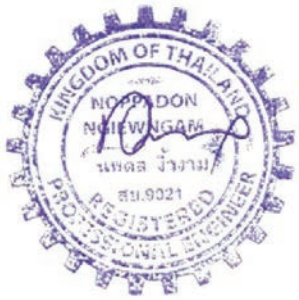
TABLE: Column Forces C59 (2nd to Roof Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
R	C47	AComb33-1	-65	13	24	70	35
2 SFL	C46	AComb2-1	-496	12	22	198	108
R	C47	AComb2-1	-257	44	80	63	36
R	C46	AComb2-1	-257	44	81	67	35
R	C47	AComb2-1	-232	44	80	233	127
R	C46	AComb2-1	-232	44	81	231	127



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Computer program for the Strength Design of Reinforced Concrete Sections
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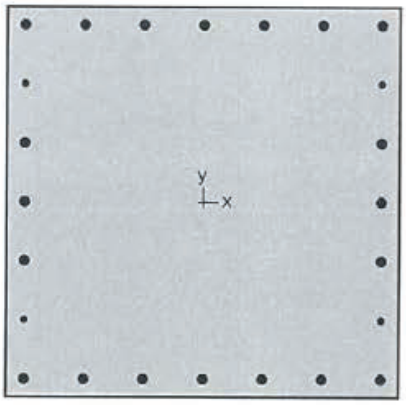

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1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63(...\C59-Roof.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	1.96e+006	mm^2
I_x	3.20133e+011	mm^4
I_y	3.20133e+011	mm^4
r_x	404.145	mm
r_y	404.145	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure

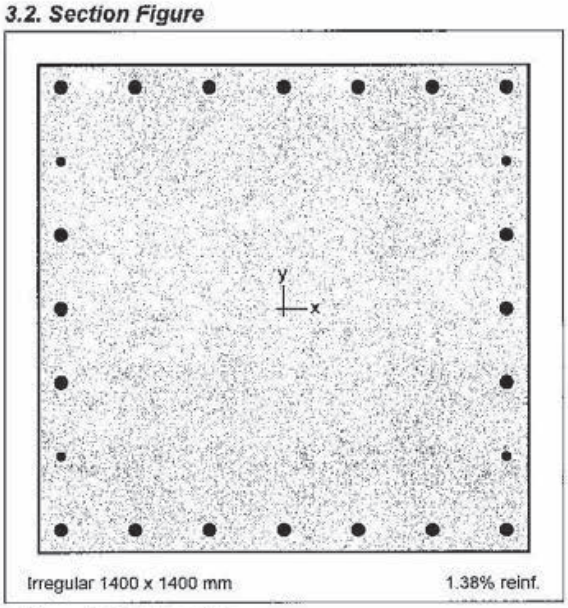


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-700.0	-700.0	2	700.0	-700.0	3	700.0	700.0
4	-700.0	700.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---

Bars	---
Total steel area, A_s	27060 mm²
Rho	1.38 %
Minimum clear spacing	172 mm

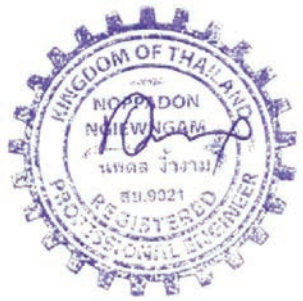
4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
1230.0	-636.0	636.0	1230.0	-424.0	636.0	1230.0	-212.0	636.0
1230.0	0.0	636.0	1230.0	212.0	636.0	1230.0	424.0	636.0
1230.0	636.0	636.0	615.0	636.0	424.0	1230.0	636.0	212.0
1230.0	636.0	0.0	1230.0	636.0	-212.0	615.0	636.0	-424.0
1230.0	636.0	-636.0	1230.0	424.0	-636.0	1230.0	212.0	-636.0
1230.0	0.0	-636.0	1230.0	-212.0	-636.0	1230.0	-424.0	-636.0
1230.0	-636.0	-636.0	615.0	-636.0	-424.0	1230.0	-636.0	-212.0
1230.0	-636.0	0.0	1230.0	-636.0	212.0	615.0	-636.0	424.0

5. Factored Loads and Moments with Corresponding Capacity Ratios

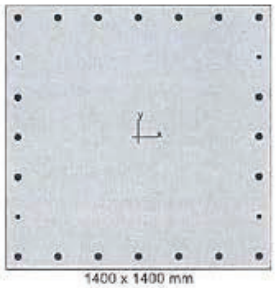
NOTE: Calculations are based on "Critical Capacity" Method.

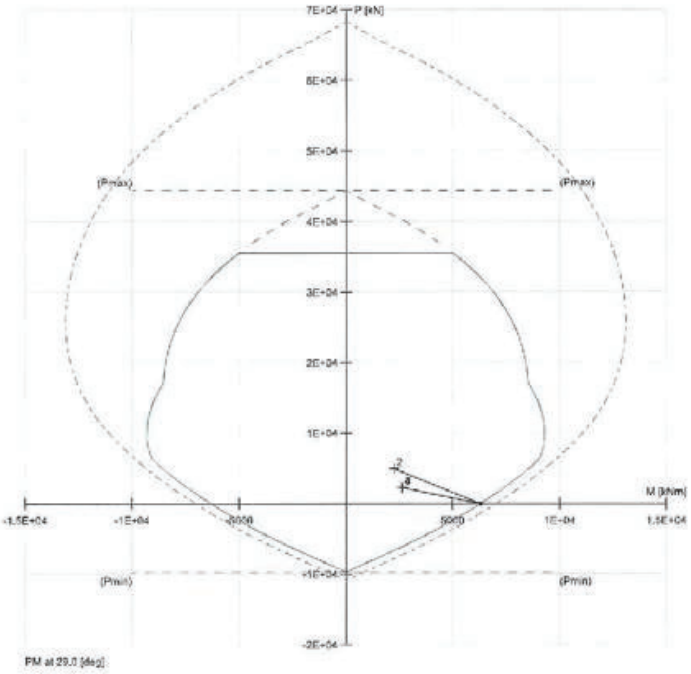
No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	NA Depth mm	ε_t	ϕ	
1	650.00	700.00	350.00	0.00	5662.96	2831.48	429	0.00903	0.900	0.41
2	4960.00	1980.00	1080.00	0.00	5563.64	3034.71	448	0.00874	0.900	0.54
3	2320.00	2330.00	1270.00	0.00	5564.48	3033.00	448	0.00875	0.900	0.60
4	2320.00	2310.00	1270.00	0.00	5554.36	3053.70	450	0.00872	0.900	0.60



6. Diagrams

6.1. PM at θ=29 [deg]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1.96e+006 mm²
I_x	3.20133e+011 mm⁴
I_y	3.20133e+011 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	27060 mm²
Rho	1.38 %
Min. clear spacing	172 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
3	2320.0	2330.0	1270.0	0.00	5564.48	3033.00	0.60
4	2320.0	2310.0	1270.0	0.00	5554.36	3053.70	0.60
2	4960.0	1980.0	1080.0	0.00	5563.64	3034.71	0.54

Max. Capacity Ratio: 0.60

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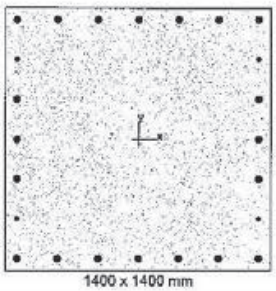
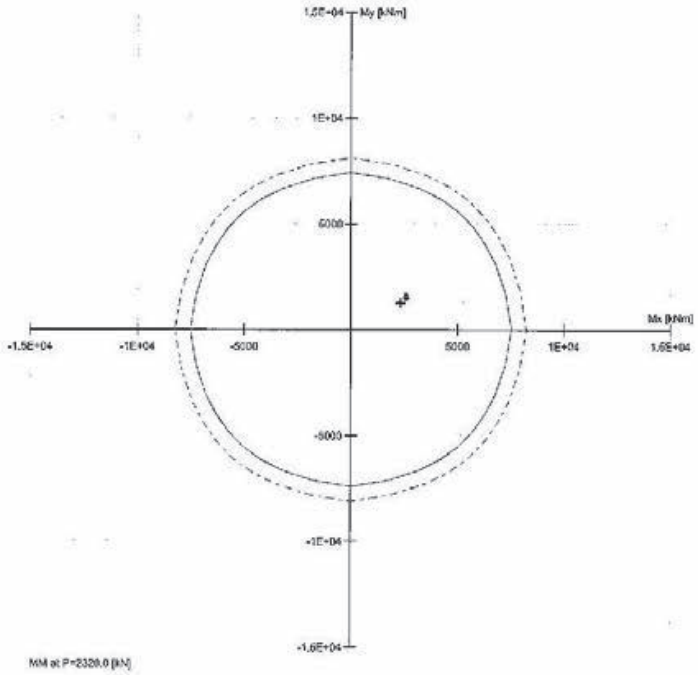
11/26/2020
3:29 PM

6.2. MM at P=2320 [kN]



1400 x 1400 mm



MM at P=2320.0 [kN]

General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1.96e+006 mm²
I_x	3.20133e+011 mm⁴
I_y	3.20133e+011 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---

Confinement type	Tied
Total steel area, A_s	27060 mm²
Rho	1.38 %
Min. clear spacing	172 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
3	2320.0	2330.0	1270.0	0.00	5564.48	3033.00	0.60
4	2320.0	2310.0	1270.0	0.00	5554.38	3053.70	0.60

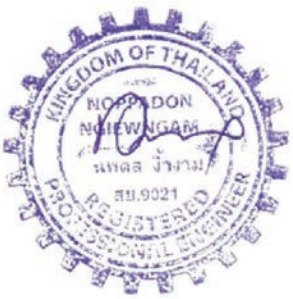
Max. Capacity Ratio: 0.60

TABLE: Pier Forces W5 (2nd -3rd Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
3 SFL	AComb33-1	-1469	1	63	30	1413	153
2 SFL	AComb2-1	-4735	21	61	10	233	441
2 SFL	AComb27-1	-3603	274	45	52	40	10006
2 SFL	AComb29-1	-3614	21	135	37	1822	530
3 SFL	AComb27-1	-3184	254	37	122	22	9041
2 SFL	AComb29-1	-3706	21	135	37	2336	610
2 SFL	AComb27-1	-3695	274	45	52	131	11049

TABLE: Pier Forces W5 (4th -Roof Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
R	AComb33-1	-123	7	43	69	78	1
4 SFL	AComb2-1	-3647	4	88	12	234	328
5 SFL	AComb27-1	-2252	466	104	1528	61	5031
5 SFL	AComb29-1	-2263	18	191	105	575	358
5 SFL	AComb27-1	-2252	466	104	1528	61	5031
4 SFL	AComb29-1	-2858	11	166	77	1692	502
4 SFL	AComb27-1	-2847	352	66	933	144	9041



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Computer program for the Strength Design of Reinforced Concrete Sections
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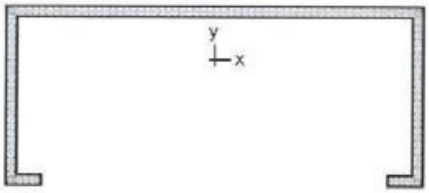

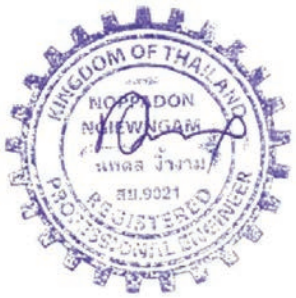

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Figure 1: Column section 4

1. General Information

File Name	c:\193018\gcdt\spcloumn\10-11-63(c2...\w5 2-3.col
Project	Zone C DD
Column	LW No.2
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard
f'_c	35 MPa
E_c	27805.6 MPa
f_c	29.75 MPa
ε_u	0.003 mm/mm
β_1	0.796192

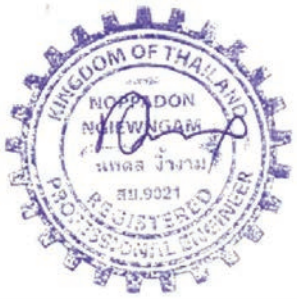
2.2. Steel

Type	Standard
f_y	400 MPa
E_s	200000 MPa
ε_{yt}	0.002 mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular
A_g	6.84181e+006 mm²
I_x	2.11589e+013 mm⁴
I_y	1.47929e+014 mm⁴
r_x	1758.58 mm
r_y	4649.87 mm
X_o	0.00172118 mm
Y_o	-0.000112251 mm



3.2. Section Figure

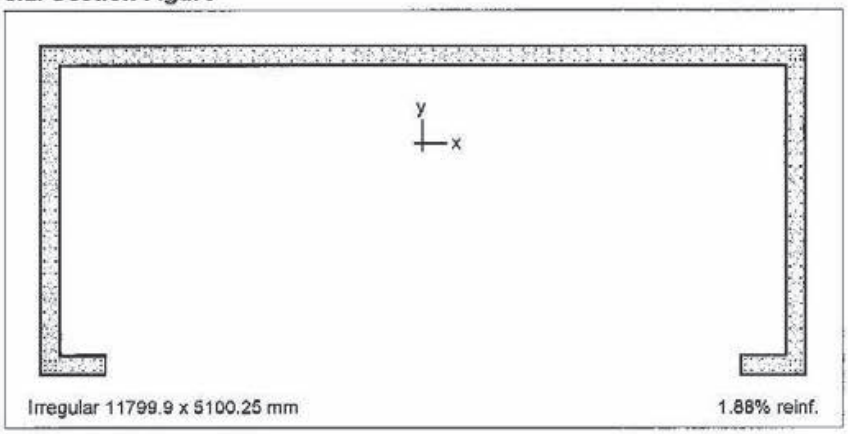


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-5599.4	-3281.9	2	-5599.4	1218.0	3	5600.5	1218.0
4	5600.5	-3281.9	5	4900.4	-3281.9	6	4900.4	-3581.9
7	5900.5	-3581.9	8	5900.5	1518.0	9	-5899.4	1518.3
10	-5899.4	-3581.9	11	-4899.3	-3581.9	12	-4899.3	-3281.9

4. Reinforcement

4.1. Bar Set: User-defined

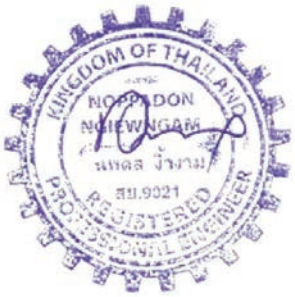
Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#20	20.00	314.16						

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#20 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	128642 mm²
Rho	1.88 %
Minimum clear spacing	42 mm



4.4. Bars Provided

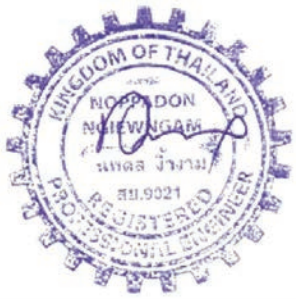
Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	-4969.3	-3351.6	491.0	-4969.3	-3512.1	491.0	-4969.3	-3431.6
491.0	-5829.4	-3511.4	491.0	-5749.4	-3511.4	491.0	-5749.4	-3361.4
491.0	-5829.4	-3361.4	491.0	-5829.4	-3436.4	491.0	-5669.4	-3511.4
491.0	-5669.4	-3361.4	491.0	-5669.4	-3436.4	491.0	-5829.4	1447.6
491.0	-5749.4	1447.6	491.0	-5749.4	1297.6	491.0	-5829.4	1297.6
491.0	-5829.4	1372.6	491.0	-5669.4	1447.6	491.0	-5669.4	1297.6
491.0	-5669.4	1372.6	491.0	4970.4	-3351.6	491.0	4970.4	-3512.1
491.0	4970.4	-3431.6	491.0	5830.5	-3511.4	491.0	5750.5	-3511.4
491.0	5750.5	-3361.4	491.0	5830.5	-3361.4	491.0	5830.5	-3436.4
491.0	5670.5	-3511.4	491.0	5670.5	-3361.4	491.0	5670.5	-3436.4
491.0	5830.5	1447.6	491.0	5750.5	1447.6	491.0	5750.5	1297.6
491.0	5830.5	1297.6	491.0	5830.5	1372.6	491.0	5670.5	1447.6
491.0	5670.5	1297.6	491.0	5670.5	1372.6	491.0	-5567.3	1455.8
491.0	-5369.6	1455.8	491.0	-5171.9	1455.8	491.0	-4974.2	1455.8
491.0	-4776.6	1455.8	491.0	-4578.9	1455.8	491.0	-4381.2	1455.8
491.0	-4183.5	1455.8	491.0	-3985.8	1455.8	491.0	-3788.2	1455.8
491.0	-3590.5	1455.8	491.0	-3392.8	1455.8	491.0	-3195.1	1455.8
491.0	-2997.4	1455.8	491.0	-2799.8	1455.8	491.0	-2602.1	1455.8
491.0	-2404.4	1455.7	491.0	-2206.7	1455.7	491.0	-2009.0	1455.7
491.0	-1811.4	1455.7	491.0	-1613.7	1455.7	491.0	-1416.0	1455.7
491.0	-1218.3	1455.7	491.0	-1020.6	1455.7	491.0	-822.9	1455.7
491.0	-625.3	1455.7	491.0	-427.6	1455.7	491.0	-229.9	1455.7
491.0	-32.2	1455.7	491.0	165.5	1455.7	491.0	363.1	1455.7
491.0	560.8	1455.7	491.0	758.5	1455.7	491.0	956.2	1455.7
491.0	1153.9	1455.7	491.0	1351.5	1455.6	491.0	1549.2	1455.6
491.0	1746.9	1455.6	491.0	1944.6	1455.6	491.0	2142.3	1455.6
491.0	2339.9	1455.6	491.0	2537.6	1455.6	491.0	2735.3	1455.6
491.0	2933.0	1455.6	491.0	3130.7	1455.6	491.0	3328.4	1455.6
491.0	3526.0	1455.6	491.0	3723.7	1455.6	491.0	3921.4	1455.6
491.0	4119.1	1455.6	491.0	4316.8	1455.6	491.0	4514.4	1455.6
491.0	4712.1	1455.6	491.0	4909.8	1455.5	491.0	5107.5	1455.5
491.0	5305.2	1455.5	491.0	5502.8	1455.5	491.0	-5567.5	1280.5
491.0	-5369.9	1280.5	491.0	-5172.4	1280.5	491.0	-4974.9	1280.5
491.0	-4777.3	1280.5	491.0	-4579.8	1280.5	491.0	-4382.3	1280.5
491.0	-4184.8	1280.5	491.0	-3987.2	1280.5	491.0	-3789.7	1280.5
491.0	-3592.2	1280.5	491.0	-3394.7	1280.5	491.0	-3197.1	1280.5
491.0	-2999.6	1280.5	491.0	-2802.1	1280.5	491.0	-2604.5	1280.5
491.0	-2407.0	1280.5	491.0	-2209.5	1280.5	491.0	-2012.0	1280.5
491.0	-1814.4	1280.5	491.0	-1616.9	1280.5	491.0	-1419.4	1280.5
491.0	-1221.9	1280.5	491.0	-1024.3	1280.5	491.0	-826.8	1280.5
491.0	-629.3	1280.5	491.0	-431.8	1280.5	491.0	-234.2	1280.5
491.0	-36.7	1280.5	491.0	160.8	1280.5	491.0	358.4	1280.5
491.0	555.9	1280.5	491.0	753.4	1280.5	491.0	950.9	1280.5
491.0	1148.5	1280.5	491.0	1346.0	1280.5	491.0	1543.5	1280.5
491.0	1741.0	1280.5	491.0	1938.6	1280.5	491.0	2136.1	1280.5
491.0	2333.6	1280.5	491.0	2531.2	1280.5	491.0	2728.7	1280.5
491.0	2926.2	1280.5	491.0	3123.7	1280.5	491.0	3321.3	1280.5
491.0	3518.8	1280.5	491.0	3716.3	1280.5	491.0	3913.8	1280.5
491.0	4111.4	1280.5	491.0	4308.9	1280.5	491.0	4506.4	1280.5
491.0	4704.0	1280.5	491.0	4901.5	1280.5	491.0	5099.0	1280.5
491.0	5296.5	1280.5	491.0	5494.1	1280.5	491.0	5663.0	1192.9
491.0	5663.0	999.5	491.0	5663.0	806.0	491.0	5663.0	612.6
491.0	5663.0	419.1	491.0	5663.0	225.7	491.0	5663.0	32.3
491.0	5663.0	-161.2	491.0	5663.0	-354.6	491.0	5663.0	-548.1
491.0	5663.0	-741.5	491.0	5663.0	-935.0	491.0	5663.0	-1128.4
491.0	5663.0	-1321.8	491.0	5663.0	-1515.3	491.0	5663.0	-1708.8
491.0	5663.0	-1902.2	491.0	5663.0	-2095.7	491.0	5663.0	-2289.1
491.0	5663.0	-2482.5	491.0	5663.0	-2676.0	491.0	5663.0	-2869.4

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	5663.0	-3062.9	491.0	5663.0	-3256.3	491.0	5838.0	-3256.3
491.0	5838.0	-3062.7	491.0	5838.0	-2869.0	491.0	5838.0	-2675.3
491.0	5838.0	-2481.6	491.0	5838.0	-2288.0	491.0	5838.0	-2094.3
491.0	5838.0	-1900.6	491.0	5838.0	-1706.9	491.0	5838.0	-1513.2
491.0	5838.0	-1319.6	491.0	5838.0	-1125.9	491.0	5838.0	-932.2
491.0	5838.0	-738.5	491.0	5838.0	-544.9	491.0	5838.0	-351.2
491.0	5838.0	-157.5	491.0	5838.0	36.2	491.0	5838.0	229.8
491.0	5838.0	423.5	491.0	5838.0	617.2	491.0	5838.0	810.9
491.0	5838.0	1004.5	491.0	5838.0	1198.2	491.0	5062.6	-3344.4
491.0	5233.1	-3344.4	491.0	5403.5	-3344.4	491.0	5574.0	-3344.4
491.0	5066.4	-3519.4	491.0	5234.3	-3519.4	491.0	5402.3	-3519.4
491.0	5570.3	-3519.4	491.0	-5836.9	-3204.7	491.0	-5836.9	-3005.8
491.0	-5836.9	-2807.0	491.0	-5836.9	-2608.1	491.0	-5836.9	-2409.2
491.0	-5836.9	-2210.4	491.0	-5836.9	-2011.5	491.0	-5836.9	-1812.6
491.0	-5836.9	-1613.8	491.0	-5836.9	-1414.9	491.0	-5836.9	-1216.1
491.0	-5836.9	-1017.2	491.0	-5836.9	-818.3	491.0	-5836.9	-619.5
491.0	-5836.9	-420.6	491.0	-5836.9	-221.8	491.0	-5836.9	-22.9
491.0	-5836.9	176.0	491.0	-5836.9	374.8	491.0	-5836.9	573.7
491.0	-5836.9	772.6	491.0	-5836.9	971.4	491.0	-5836.9	1170.3
491.0	-5661.9	1171.7	491.0	-5661.9	972.4	491.0	-5661.9	773.2
491.0	-5661.9	574.0	491.0	-5661.9	374.7	491.0	-5661.9	175.5
491.0	-5661.9	-23.7	491.0	-5661.9	-223.0	491.0	-5661.9	-422.2
491.0	-5661.9	-621.5	491.0	-5661.9	-820.7	491.0	-5661.9	-1019.9
491.0	-5661.9	-1219.2	491.0	-5661.9	-1418.4	491.0	-5661.9	-1617.6
491.0	-5661.9	-1816.9	491.0	-5661.9	-2016.1	491.0	-5661.9	-2215.3
491.0	-5661.9	-2414.6	491.0	-5661.9	-2613.8	491.0	-5661.9	-2813.0
491.0	-5661.9	-3012.3	491.0	-5661.9	-3211.5	491.0	-5604.9	-3344.4
491.0	-5426.1	-3344.4	491.0	-5247.4	-3344.4	491.0	-5068.7	-3344.4
491.0	-5595.2	-3519.4	491.0	-5418.6	-3519.4	491.0	-5242.1	-3519.4
491.0	-5065.5	-3519.4						

5. Factored Loads and Moments with Corresponding Capacity Ratios

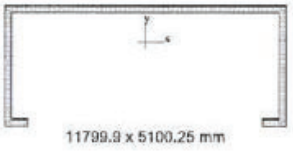
NOTE: Calculations are based on "Moment Capacity" Method.

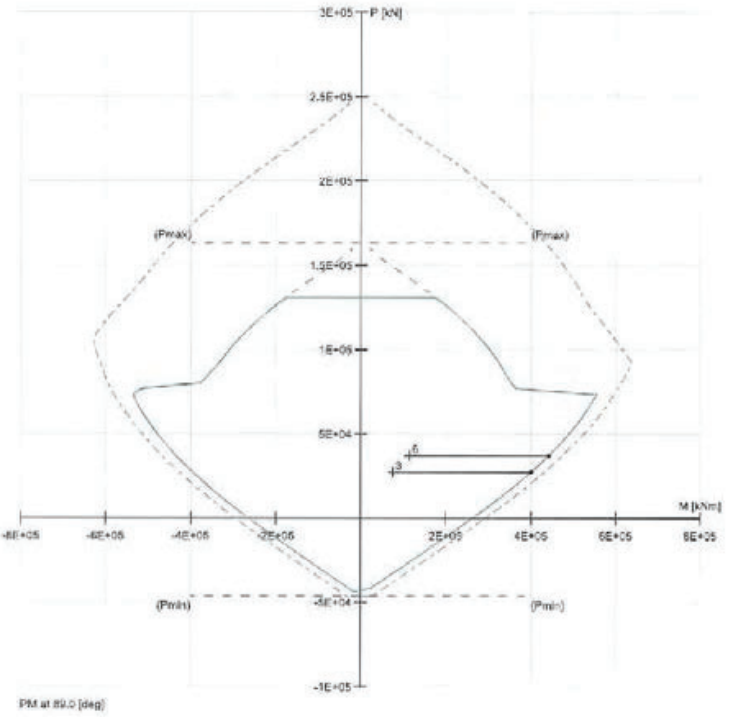
No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	NA Depth mm	ε_t	ϕ	
1	5300.00	1600.00	920.00	5300.00	140486.33	80779.66	2208	0.00574	0.900	0.01
2	47340.00	3750.00	4640.00	47340.00	113042.72	139871.55	5425	0.00125	0.650	0.03
3	27270.00	1170.00	75730.00	27270.00	6182.88	400197.88	2178	0.01464	0.900	0.19
4	16180.00	1450.00	3420.00	16180.00	97168.19	229182.94	4142	0.00321	0.750	0.01
5	37060.00	24810.00	6550.00	37060.00	134844.00	35599.69	3785	0.00150	0.650	0.18
6	36940.00	2370.00	114780.00	36940.00	9125.81	441967.34	2666	0.01141	0.900	0.26



6. Diagrams

6.1. PM at θ=89 [deg]





PM at 89.0 [deg]

General Information	
Project	Zone C DD
Column	LW No.2
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

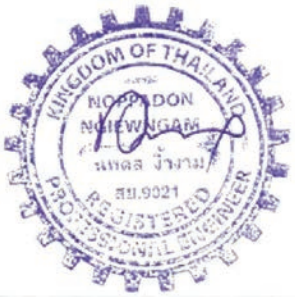
Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	6.84181e+006 mm²
I_x	2.11589e+013 mm⁴
I_y	1.47929e+014 mm⁴

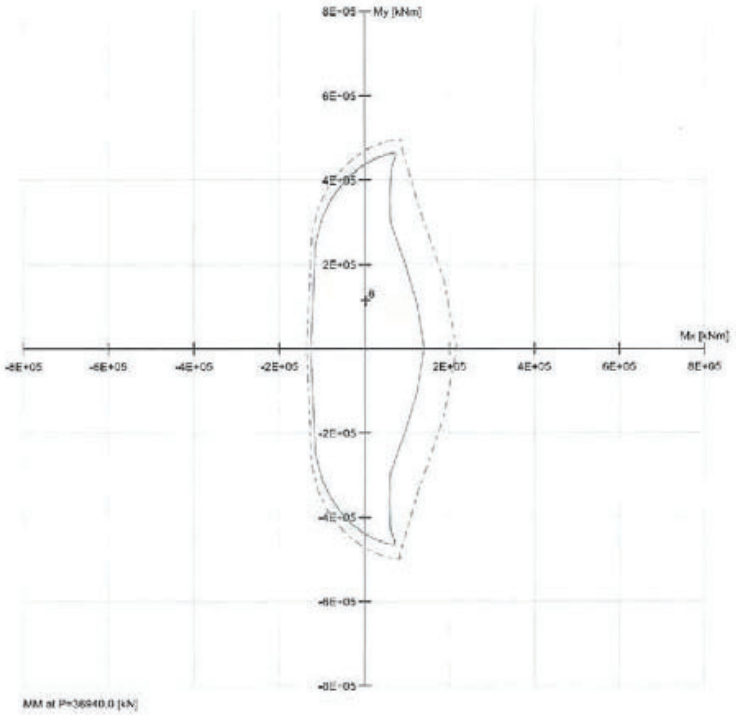
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	128642 mm²
Rho	1.88 %
Min. clear spacing	42 mm

No.	P_n kN	M_{nx} kNm	M_{ny} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
6	36940.0	2370.0	114780.0	36940.00	9125.81	441967.34	0.26
3	27270.0	1170.0	75730.0	27270.00	6182.86	400197.88	0.19

Max. Capacity Ratio: 0.26



6.2. MM at P=36940 [kN]



General Information	
Project	Zone C DD
Column	LW No.2
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	6.84181e+006 mm²
I_x	2.11589e+013 mm⁴
I_y	1.47929e+014 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	128642 mm²
Rho	1.88 %
Min. clear spacing	42 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
6	36940.0	2370.0	114780.0	36940.00	9125.81	441967.34	0.26

Max. Capacity Ratio: 0.26

spColumn v6.50
Computer program for the Strength Design of Reinforced Concrete Sections
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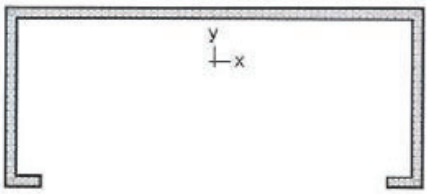

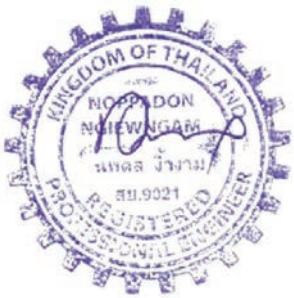

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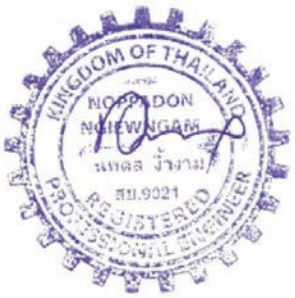


1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63...\W5 4-Roof.col
Project	Zone C DD
Column	LW No.2
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	6.84181e+006	mm^2
I_x	2.11589e+013	mm^4
I_y	1.47929e+014	mm^4
r_x	1758.58	mm
r_y	4649.87	mm
X_o	0.00172118	mm
Y_o	-0.000112251	mm

3.2. Section Figure

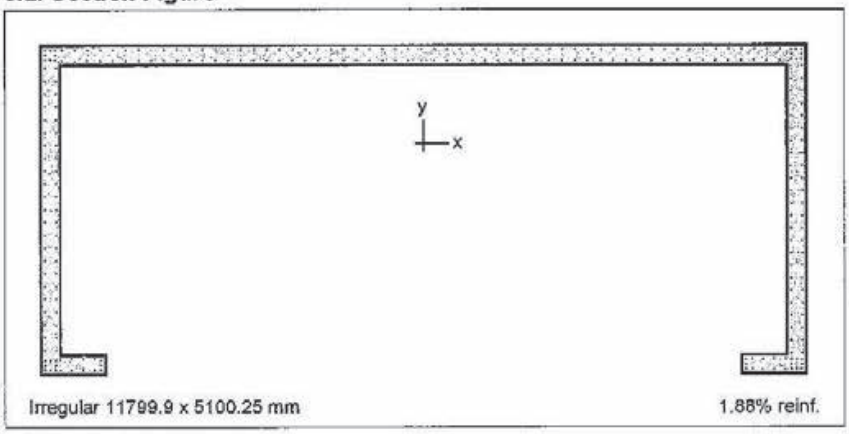


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-5599.4	-3281.9	2	-5599.4	1218.0	3	5600.5	1218.0
4	5600.5	-3281.9	5	4900.4	-3281.9	6	4900.4	-3581.9
7	5900.5	-3581.9	8	5900.5	1518.0	9	-5899.4	1518.3
10	-5899.4	-3581.9	11	-4899.3	-3581.9	12	-4899.3	-3281.9

4. Reinforcement

4.1. Bar Set: User-defined

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#20	20.00	314.16						

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#20 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	128642 mm²
Rho	1.88 %
Minimum clear spacing	42 mm

4.4. Bars Provided

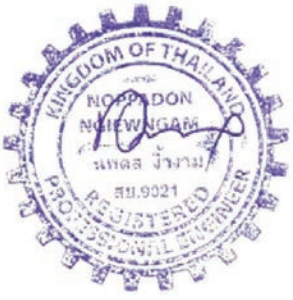
Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	-4969.3	-3351.6	491.0	-4969.3	-3512.1	491.0	-4969.3	-3431.6
491.0	-5829.4	-3511.4	491.0	-5749.4	-3511.4	491.0	-5749.4	-3361.4
491.0	-5829.4	-3361.4	491.0	-5829.4	-3436.4	491.0	-5669.4	-3511.4
491.0	-5669.4	-3361.4	491.0	-5669.4	-3436.4	491.0	-5829.4	1447.6
491.0	-5749.4	1447.6	491.0	-5749.4	1297.6	491.0	-5829.4	1297.6
491.0	-5829.4	1372.6	491.0	-5669.4	1447.6	491.0	-5669.4	1297.6
491.0	-5669.4	1372.6	491.0	4970.4	-3351.6	491.0	4970.4	-3512.1
491.0	4970.4	-3431.6	491.0	5830.5	-3511.4	491.0	5750.5	-3511.4
491.0	5750.5	-3361.4	491.0	5830.5	-3361.4	491.0	5830.5	-3436.4
491.0	5670.5	-3511.4	491.0	5670.5	-3361.4	491.0	5670.5	-3436.4
491.0	5830.5	1447.6	491.0	5750.5	1447.6	491.0	5750.5	1297.6
491.0	5830.5	1297.6	491.0	5830.5	1372.6	491.0	5670.5	1447.6
491.0	5670.5	1297.6	491.0	5670.5	1372.6	491.0	-5567.3	1455.8
491.0	-5369.6	1455.8	491.0	-5171.9	1455.8	491.0	-4974.2	1455.8
491.0	-4776.6	1455.8	491.0	-4578.9	1455.8	491.0	-4381.2	1455.8
491.0	-4183.5	1455.8	491.0	-3985.8	1455.8	491.0	-3788.2	1455.8
491.0	-3590.5	1455.8	491.0	-3392.8	1455.8	491.0	-3195.1	1455.8
491.0	-2997.4	1455.8	491.0	-2799.8	1455.8	491.0	-2602.1	1455.8
491.0	-2404.4	1455.7	491.0	-2206.7	1455.7	491.0	-2009.0	1455.7
491.0	-1811.4	1455.7	491.0	-1613.7	1455.7	491.0	-1416.0	1455.7
491.0	-1218.3	1455.7	491.0	-1020.6	1455.7	491.0	-822.9	1455.7
491.0	-625.3	1455.7	491.0	-427.6	1455.7	491.0	-229.9	1455.7
491.0	-32.2	1455.7	491.0	165.5	1455.7	491.0	363.1	1455.7
491.0	560.8	1455.7	491.0	758.5	1455.7	491.0	956.2	1455.7
491.0	1153.9	1455.7	491.0	1351.5	1455.6	491.0	1549.2	1455.6
491.0	1746.9	1455.6	491.0	1944.6	1455.6	491.0	2142.3	1455.6
491.0	2339.9	1455.6	491.0	2537.6	1455.6	491.0	2735.3	1455.6
491.0	2933.0	1455.6	491.0	3130.7	1455.6	491.0	3328.4	1455.6
491.0	3526.0	1455.6	491.0	3723.7	1455.6	491.0	3921.4	1455.6
491.0	4119.1	1455.6	491.0	4316.8	1455.6	491.0	4514.4	1455
491.0	4712.1	1455.6	491.0	4909.8	1455.5	491.0	5107.5	1455
491.0	5305.2	1455.5	491.0	5502.8	1455.5	491.0	-5567.5	1280
491.0	-5369.9	1280.5	491.0	-5172.4	1280.5	491.0	-4974.9	1280.5
491.0	-4777.3	1280.5	491.0	-4579.8	1280.5	491.0	-4382.3	1280.5
491.0	-4184.8	1280.5	491.0	-3987.2	1280.5	491.0	-3789.7	1280.5
491.0	-3592.2	1280.5	491.0	-3394.7	1280.5	491.0	-3197.1	1280.5
491.0	-2999.6	1280.5	491.0	-2802.1	1280.5	491.0	-2604.5	1280.5
491.0	-2407.0	1280.5	491.0	-2209.5	1280.5	491.0	-2012.0	1280.5
491.0	-1814.4	1280.5	491.0	-1616.9	1280.5	491.0	-1419.4	1280.5
491.0	-1221.9	1280.5	491.0	-1024.3	1280.5	491.0	-826.8	1280.5
491.0	-629.3	1280.5	491.0	-431.8	1280.5	491.0	-234.2	1280.5
491.0	-36.7	1280.5	491.0	160.8	1280.5	491.0	358.4	1280.5
491.0	555.9	1280.5	491.0	753.4	1280.5	491.0	950.9	1280.5
491.0	1148.5	1280.5	491.0	1346.0	1280.5	491.0	1543.5	1280.5
491.0	1741.0	1280.5	491.0	1938.6	1280.5	491.0	2136.1	1280.5
491.0	2333.6	1280.5	491.0	2531.2	1280.5	491.0	2728.7	1280.5
491.0	2926.2	1280.5	491.0	3123.7	1280.5	491.0	3321.3	1280.5
491.0	3518.8	1280.5	491.0	3716.3	1280.5	491.0	3913.8	1280.5
491.0	4111.4	1280.5	491.0	4308.9	1280.5	491.0	4506.4	1280.5
491.0	4704.0	1280.5	491.0	4901.5	1280.5	491.0	5099.0	1280.5
491.0	5296.5	1280.5	491.0	5494.1	1280.5	491.0	5663.0	1192.9
491.0	5663.0	999.5	491.0	5663.0	806.0	491.0	5663.0	612.6
491.0	5663.0	419.1	491.0	5663.0	225.7	491.0	5663.0	32.3
491.0	5663.0	-161.2	491.0	5663.0	-354.6	491.0	5663.0	-548.1
491.0	5663.0	-741.5	491.0	5663.0	-935.0	491.0	5663.0	-1128.4
491.0	5663.0	-1321.9	491.0	5663.0	-1515.3	491.0	5663.0	-1708.8
491.0	5663.0	-1902.2	491.0	5663.0	-2095.7	491.0	5663.0	-2289.1
491.0	5663.0	-2482.5	491.0	5663.0	-2676.0	491.0	5663.0	-2869.4

Area	X	Y	Area	X	Y	Area	X	Y
mm²	mm	mm	mm²	mm	mm	mm²	mm	mm
491.0	5663.0	-3062.9	491.0	5663.0	-3256.3	491.0	5838.0	-3256.3
491.0	5838.0	-3062.7	491.0	5838.0	-2869.0	491.0	5838.0	-2675.3
491.0	5838.0	-2481.6	491.0	5838.0	-2288.0	491.0	5838.0	-2094.3
491.0	5838.0	-1900.6	491.0	5838.0	-1706.9	491.0	5838.0	-1513.2
491.0	5838.0	-1319.6	491.0	5838.0	-1125.9	491.0	5838.0	-932.2
491.0	5838.0	-738.5	491.0	5838.0	-544.9	491.0	5838.0	-351.2
491.0	5838.0	-157.5	491.0	5838.0	36.2	491.0	5838.0	229.8
491.0	5838.0	423.5	491.0	5838.0	617.2	491.0	5838.0	810.9
491.0	5838.0	1004.5	491.0	5838.0	1198.2	491.0	5062.6	-3344.4
491.0	5233.1	-3344.4	491.0	5403.5	-3344.4	491.0	5574.0	-3344.4
491.0	5066.4	-3519.4	491.0	5234.3	-3519.4	491.0	5402.3	-3519.4
491.0	5570.3	-3519.4	491.0	-5836.9	-3204.7	491.0	-5836.9	-3005.8
491.0	-5836.9	-2807.0	491.0	-5836.9	-2608.1	491.0	-5836.9	-2409.2
491.0	-5836.9	-2210.4	491.0	-5836.9	-2011.5	491.0	-5836.9	-1812.6
491.0	-5836.9	-1613.8	491.0	-5836.9	-1414.9	491.0	-5836.9	-1216.1
491.0	-5836.9	-1017.2	491.0	-5836.9	-818.3	491.0	-5836.9	-619.5
491.0	-5836.9	-420.6	491.0	-5836.9	-221.8	491.0	-5836.9	-22.9
491.0	-5836.9	176.0	491.0	-5836.9	374.8	491.0	-5836.9	573.7
491.0	-5836.9	772.6	491.0	-5836.9	971.4	491.0	-5836.9	1170.3
491.0	-5661.9	1171.7	491.0	-5661.9	972.4	491.0	-5661.9	773.2
491.0	-5661.9	574.0	491.0	-5661.9	374.7	491.0	-5661.9	175.5
491.0	-5661.9	-23.7	491.0	-5661.9	-223.0	491.0	-5661.9	-422.2
491.0	-5661.9	-621.5	491.0	-5661.9	-820.7	491.0	-5661.9	-1019.9
491.0	-5661.9	-1219.2	491.0	-5661.9	-1418.4	491.0	-5661.9	-1617.6
491.0	-5661.9	-1816.9	491.0	-5661.9	-2016.1	491.0	-5661.9	-2215.3
491.0	-5661.9	-2414.6	491.0	-5661.9	-2613.8	491.0	-5661.9	-2813.0
491.0	-5661.9	-3012.3	491.0	-5661.9	-3211.5	491.0	-5604.9	-3344.4
491.0	-5426.1	-3344.4	491.0	-5247.4	-3344.4	491.0	-5068.7	-3344.4
491.0	-5595.2	-3519.4	491.0	-5418.6	-3519.4	491.0	-5242.1	-3519.4
491.0	-5065.5	-3519.4						

5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kN	kNm	kNm	kN	kNm	kNm	mm			
1	1230.00	780.00	10.00	1230.00	143322.20	1837.48	1284	0.00886	0.900	0.01
2	36470.00	2340.00	3280.00	36470.00	109470.87	153446.34	4940	0.00177	0.650	0.02
3	28580.00	16920.00	5020.00	28580.00	136420.45	40474.64	3432	0.00200	0.650	0.12
4	28470.00	1440.00	90410.00	28470.00	6458.29	405482.28	2239	0.01418	0.900	0.22



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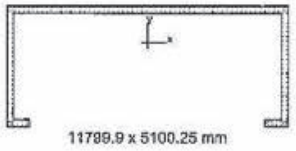
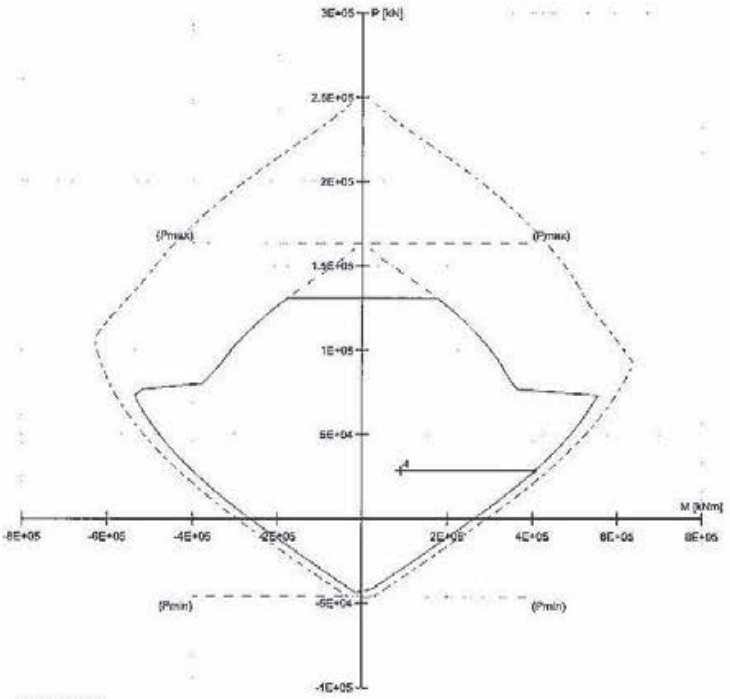
3:31 PM

6. Diagrams

6.1. PM at θ=89 [deg]





PM at 89.0 [deg]

General Information	
Project	Zone C DD
Column	LW No.2
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials		
f'_c	35	MPa
E_c	27805.6	MPa
f_y	400	MPa
E_s	200000	MPa

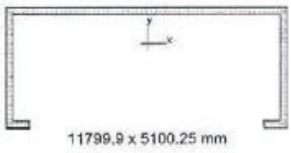
Section		
Type	Irregular	
A_g	6.84181e+006	mm²
I_x	2.11589e+013	mm⁴
I_y	1.47929e+014	mm⁴

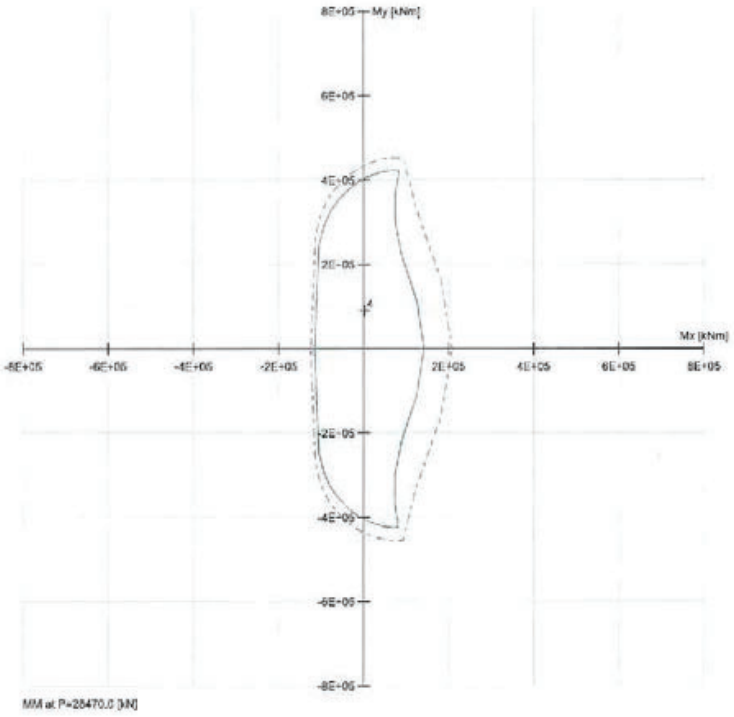
Reinforcement	
Pattern	Irregular
Bar layout	—
Cover to	—
Clear cover	—
Bars	—
Confinement type	Tied
Total steel area, A_s	128642 mm²
Rho	1.88 %
Min. clear spacing	42 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	28470.0	1440.0	90410.0	28470.00	6458.29	405482.28	0.22

Max. Capacity Ratio: 0.22

6.2. MM at P=28470 [kN]





General Information	
Project	Zone C DD
Column	LW No.2
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials		
f'_c	35	MPa
E_c	27805.6	MPa
f_y	400	MPa
E_s	200000	MPa

Section		
Type	Irregular	
A_g	6.84181e+006	mm²
I_x	2.11589e+013	mm⁴
I_y	1.47929e+014	mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	128642 mm²
Rho	1.88 %
Min. clear spacing	42 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	28470.0	1440.0	90410.0	28470.00	6458.29	405482.28	0.22

Max. Capacity Ratio: 0.22

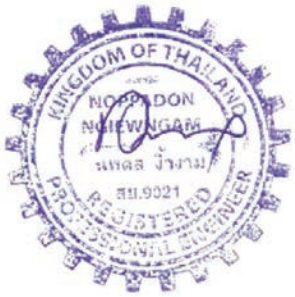
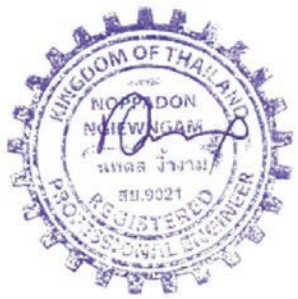


TABLE: Pier Forces W6\D3 (2nd -3rd Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
3 SFL	AComb34-1	-1652	135	609	3088	9923	1606
2 SFL	AComb2-1	-4844	124	45	105	974	50
2 SFL	AComb28-1	-3693	636	208	282	3214	8126
2 SFL	AComb29-1	-3595	197	664	3061	12125	1447
2 SFL	AComb30-1	-3886	6	585	3188	10907	2230
2 SFL	AComb29-1	-3739	197	664	3061	14647	2194
2 SFL	AComb32-1	-2068	580	199	176	3810	10643

TABLE: Pier Forces W6\D3 (4th -Roof Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
R	AComb32-1	-87	15	9	89	171	41
4 SFL	AComb2-1	-3756	100	14	228	2	24
4 SFL	AComb28-1	-2797	502	130	247	1543	4273
4 SFL	AComb34-1	-1404	120	587	3069	7084	1082
4 SFL	AComb30-1	-2941	20	575	3183	6882	1512
4 SFL	AComb34-1	-1544	120	587	3069	9960	1669
4 SFL	AComb32-1	-1624	453	155	53	2422	6756



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Computer program for the Strength Design of Reinforced Concrete Sections
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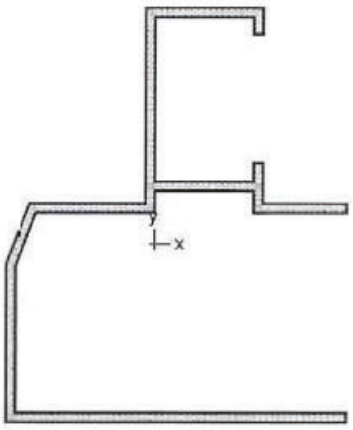

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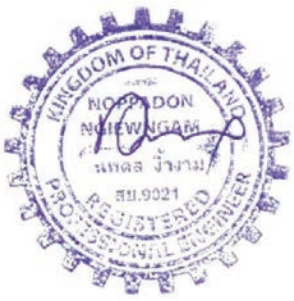


1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63...\W6-D3 2-3.col
Project	Zone C DD
Column	LW Nos.1&3
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	1.1895e+007	mm²
I_x	2.41724e+014	mm⁴
I_y	1.17672e+014	mm⁴
r_x	4507.94	mm
r_y	3145.24	mm
X_o	-0.00021904	mm
Y_o	-0.000860867	mm

3.2. Section Figure

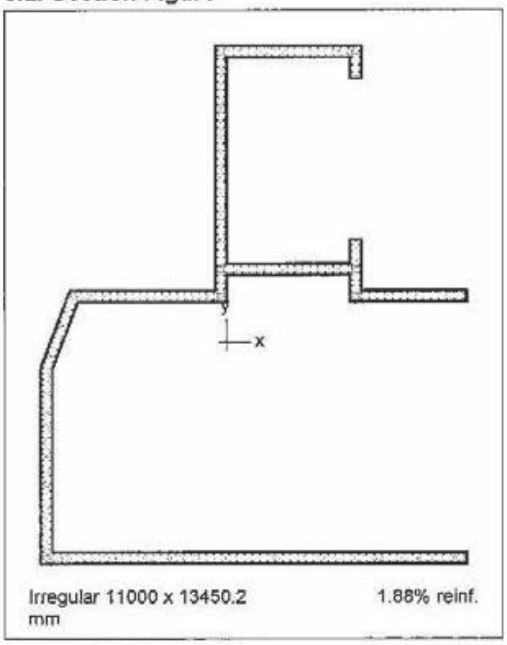


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	5.9	1737.9	2	5.9	1037.9	3	-3835.5	1037.9
4	-4494.1	-707.2	5	-4494.1	-5462.3	6	6205.8	-5462.0
7	6205.8	-5762.0	8	-4794.1	-5762.3	9	-4794.2	-652.5
10	-4042.9	1337.9	11	-294.0	1337.9	12	-294.0	7687.9
13	3506.0	7687.9	14	3506.0	6837.9	15	3206.0	6837.9
16	3206.0	7387.7	17	6.0	7387.7	18	6.0	2037.9
19	3206.0	2037.9	20	3206.0	2662.9	21	3506.0	2662.9
22	3506.0	1337.9	23	6205.9	1337.9	24	6205.8	1037.9
25	3206.0	1037.9	26	3206.0	1737.9			

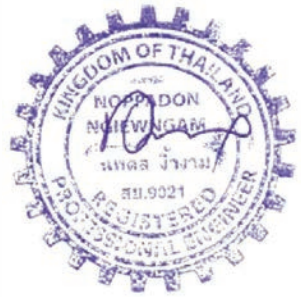
4. Reinforcement

4.1. Bar Set: User-defined

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#20	20.00	314.16						

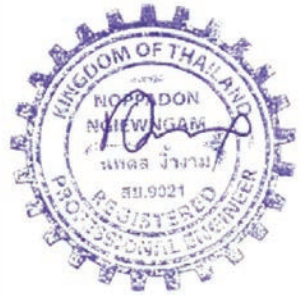
4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#20 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65



4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	223399 mm²
Rho	1.88 %
Minimum clear spacing	20 mm



4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
314.2	6135.8	-5532.1	314.2	6135.8	-5612.0	314.2	6135.8	-5692.0
314.2	-4724.1	-5691.8	314.2	-4564.1	-5691.8	314.2	-4644.1	-5691.9
314.2	-4724.1	-5541.9	314.2	-4564.1	-5541.9	314.2	-4644.1	-5541.9
314.2	-4724.1	-5616.8	314.2	-4564.1	-5616.8	314.2	-3884.0	1108.2
314.2	-3969.5	1108.2	314.2	-4054.8	1108.7	314.2	-3894.8	1268.1
314.2	-3995.1	1266.9	314.2	-4024.0	1190.3	314.2	-3854.5	1186.4
314.2	-64.1	1108.3	314.2	-144.1	1108.3	314.2	-224.1	1108.3
314.2	-224.1	1183.3	314.2	-64.1	1183.3	314.2	-64.1	1258.3
314.2	-144.1	1258.3	314.2	-224.1	1258.3	314.2	-64.1	1808.3
314.2	-144.1	1808.3	314.2	-224.1	1808.3	314.2	-64.1	1883.3
314.2	-224.1	1883.3	314.2	-224.1	1958.3	314.2	-144.1	1958.3
314.2	-64.1	1958.3	314.2	3276.0	1808.3	314.2	3356.0	1808.3
314.2	3436.0	1808.3	314.2	3276.0	1883.3	314.2	3436.0	1883.3
314.2	3436.0	1958.3	314.2	3356.0	1958.3	314.2	3276.0	1958.3
314.2	3436.0	2592.9	314.2	3276.0	2592.9	314.2	3356.0	2592.9
314.2	-64.1	7617.8	314.2	-144.1	7617.8	314.2	-224.1	7617.8
314.2	-64.1	7542.7	314.2	-224.1	7542.7	314.2	-224.1	7467.7
314.2	-144.1	7467.7	314.2	-64.1	7467.7	314.2	3436.0	6907.9
314.2	3276.0	6907.9	314.2	3356.0	6907.9	314.2	3276.0	7617.9
314.2	3356.0	7617.9	314.2	3436.0	7617.9	314.2	3276.0	7542.9
314.2	3436.0	7542.9	314.2	3436.0	7467.9	314.2	3356.0	7467.9
314.2	3276.0	7467.9	314.2	6135.8	1107.9	314.2	6135.8	1187.9
314.2	6135.8	1267.8	314.2	3276.0	1108.3	314.2	3356.0	1108.3
314.2	3276.0	1183.3	314.2	3276.0	1258.3	314.2	3356.0	1258.3
314.2	3435.9	1258.3	314.2	3435.9	1183.3	314.2	3435.9	1108.3
491.0	-4731.6	-663.9	491.0	-4731.6	-813.2	491.0	-4731.6	-962.5
491.0	-4731.6	-1111.8	491.0	-4731.6	-1261.1	491.0	-4731.6	-1410.4
491.0	-4731.6	-1559.7	491.0	-4731.6	-1709.0	491.0	-4731.6	-1858.3
491.0	-4731.6	-2007.6	491.0	-4731.6	-2156.9	491.0	-4731.6	-2306.2
491.0	-4731.6	-2455.5	491.0	-4731.6	-2604.8	491.0	-4731.6	-2754.1
491.0	-4731.6	-2903.4	491.0	-4731.6	-3052.7	491.0	-4731.6	-3201.9
491.0	-4731.6	-3351.2	491.0	-4731.6	-3500.5	491.0	-4731.6	-3649.8
491.0	-4731.6	-3799.1	491.0	-4731.6	-3948.4	491.0	-4731.6	-4097.7
491.0	-4731.6	-4247.0	491.0	-4731.6	-4396.3	491.0	-4731.6	-4545.6
491.0	-4731.6	-4694.9	491.0	-4731.6	-4844.2	491.0	-4731.6	-4993.5
491.0	-4731.6	-5142.8	491.0	-4731.6	-5292.1	491.0	-4731.6	-5441.4
491.0	-4682.0	-532.2	491.0	-4632.3	-400.6	491.0	-4582.6	-268.9
491.0	-4532.9	-137.2	491.0	-4483.2	-5.5	491.0	-4433.5	126.1
491.0	-4383.8	257.8	491.0	-4334.1	389.5	491.0	-4284.4	521.1
491.0	-4234.7	652.8	491.0	-4185.0	784.5	491.0	-4135.3	916.2
491.0	-4085.6	1047.8	491.0	-4556.7	-695.8	491.0	-4556.6	-893.7
491.0	-4556.6	-1091.6	491.0	-4556.6	-1289.5	491.0	-4556.6	-1487.4
491.0	-4556.6	-1685.3	491.0	-4556.6	-1883.2	491.0	-4556.6	-2081.2
491.0	-4556.6	-2279.1	491.0	-4556.6	-2477.0	491.0	-4556.6	-2674.9
491.0	-4556.6	-2872.8	491.0	-4556.6	-3070.7	491.0	-4556.6	-3268.6

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	-4556.6	-3466.5	491.0	-4556.6	-3664.4	491.0	-4556.6	-3862.3
491.0	-4556.6	-4060.2	491.0	-4556.6	-4258.1	491.0	-4556.6	-4456.0
491.0	-4556.6	-4653.9	491.0	-4556.6	-4851.8	491.0	-4556.6	-5049.7
491.0	-4556.6	-5247.6	491.0	-4556.6	-5445.5	491.0	-4492.6	-526.2
491.0	-4428.6	-356.7	491.0	-4364.6	-187.1	491.0	-4300.6	-17.5
491.0	-4236.6	152.1	491.0	-4172.6	321.7	491.0	-4108.6	491.3
491.0	-4044.6	660.8	491.0	-3980.6	830.4	491.0	-3916.6	1000.0
491.0	-3763.8	1275.4	491.0	-3572.9	1275.4	491.0	-3382.0	1275.4
491.0	-3191.1	1275.4	491.0	-3000.3	1275.4	491.0	-2809.4	1275.4
491.0	-2618.5	1275.4	491.0	-2427.6	1275.4	491.0	-2236.7	1275.4
491.0	-2045.9	1275.4	491.0	-1855.0	1275.4	491.0	-1664.1	1275.4
491.0	-1473.2	1275.4	491.0	-1282.4	1275.4	491.0	-1091.5	1275.4
491.0	-900.6	1275.4	491.0	-709.7	1275.4	491.0	-518.9	1275.4
491.0	-328.0	1275.4	491.0	-3768.6	1100.4	491.0	-3576.9	1100.4
491.0	-3385.2	1100.4	491.0	-3193.5	1100.4	491.0	-3001.9	1100.4
491.0	-2810.2	1100.4	491.0	-2618.5	1100.4	491.0	-2426.8	1100.4
491.0	-2235.1	1100.4	491.0	-2043.4	1100.4	491.0	-1851.8	1100.4
491.0	-1660.1	1100.4	491.0	-1468.4	1100.4	491.0	-1276.7	1100.4
491.0	-1085.0	1100.4	491.0	-893.4	1100.4	491.0	-701.7	1100.4
491.0	-510.0	1100.4	491.0	-318.3	1100.4	491.0	-231.5	1355.7
491.0	-231.5	1541.5	491.0	-231.5	1727.3	491.0	-56.6	1352.1
491.0	-56.6	1536.2	491.0	-56.6	1720.2	491.0	-231.5	2000.0
491.0	-231.5	2197.9	491.0	-231.5	2395.8	491.0	-231.5	2593.8
491.0	-231.5	2791.7	491.0	-231.5	2989.6	491.0	-231.5	3187.5
491.0	-231.5	3385.5	491.0	-231.5	3583.4	491.0	-231.5	3781.3
491.0	-231.5	3979.2	491.0	-231.5	4177.1	491.0	-231.5	4375.1
491.0	-231.5	4573.0	491.0	-231.5	4770.9	491.0	-231.5	4968.8
491.0	-231.5	5166.8	491.0	-231.5	5364.7	491.0	-231.5	5562.6
491.0	-231.5	5760.5	491.0	-231.5	5958.5	491.0	-231.5	6156.4
491.0	-231.5	6354.3	491.0	-231.5	6552.2	491.0	-231.5	6750.1
491.0	-231.5	6948.1	491.0	-231.5	7146.0	491.0	-231.5	7343.9
491.0	-56.5	2000.0	491.0	-56.5	2197.7	491.0	-56.5	2395.5
491.0	-56.5	2593.2	491.0	-56.5	2790.9	491.0	-56.5	2988.7
491.0	-56.5	3186.4	491.0	-56.5	3384.1	491.0	-56.5	3581.9
491.0	-56.5	3779.6	491.0	-56.5	3977.4	491.0	-56.5	4175.1
491.0	-56.5	4372.8	491.0	-56.5	4570.6	491.0	-56.5	4768.3
491.0	-56.5	4966.0	491.0	-56.5	5163.8	491.0	-56.5	5361.5
491.0	-56.5	5559.2	491.0	-56.5	5757.0	491.0	-56.5	5954
491.0	-56.5	6152.4	491.0	-56.5	6350.2	491.0	-56.5	65[illegible]
491.0	-56.5	6745.6	491.0	-56.5	6943.4	491.0	-56.5	1[illegible]
491.0	-56.5	7338.8	491.0	0.0	1975.4	491.0	198.4	1975.4
491.0	396.7	1975.4	491.0	595.1	1975.4	491.0	793.4	1975.4
491.0	991.8	1975.4	491.0	1190.1	1975.4	491.0	1388.5	1975.4
491.0	1586.8	1975.4	491.0	1785.2	1975.4	491.0	1983.6	1975.4
491.0	2181.9	1975.4	491.0	2380.3	1975.4	491.0	2578.6	1975.4
491.0	2777.0	1975.4	491.0	2975.3	1975.4	491.0	3173.7	1975.4
491.0	0.0	1800.4	491.0	198.7	1800.4	491.0	397.3	1800.4
491.0	596.0	1800.4	491.0	794.7	1800.4	491.0	993.4	1800.4
491.0	1192.0	1800.4	491.0	1390.7	1800.4	491.0	1589.4	1800.4
491.0	1788.0	1800.4	491.0	1986.7	1800.4	491.0	2185.4	1800.4
491.0	2384.1	1800.4	491.0	2582.7	1800.4	491.0	2781.4	1800.4
491.0	2980.1	1800.4	491.0	3178.7	1800.4	491.0	0.0	7625.4
491.0	198.9	7625.4	491.0	397.8	7625.4	491.0	596.6	7625.4
491.0	795.5	7625.4	491.0	994.4	7625.4	491.0	1193.3	7625.4
491.0	1392.2	7625.4	491.0	1591.1	7625.4	491.0	1789.9	7625.4
491.0	1988.8	7625.4	491.0	2187.7	7625.4	491.0	2386.6	7625.4
491.0	2585.5	7625.4	491.0	2784.4	7625.4	491.0	2983.2	7625.4
491.0	3182.1	7625.4	491.0	0.0	7450.2	491.0	199.1	7450.2
491.0	398.3	7450.2	491.0	597.4	7450.2	491.0	796.5	7450.2

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	995.7	7450.2	491.0	1194.8	7450.2	491.0	1393.9	7450.2
491.0	1593.1	7450.2	491.0	1792.2	7450.2	491.0	1991.3	7450.2
491.0	2190.5	7450.2	491.0	2389.6	7450.2	491.0	2588.7	7450.2
491.0	2787.9	7450.2	491.0	2987.0	7450.2	491.0	3186.1	7450.2
491.0	3268.5	7393.7	491.0	3268.5	7196.8	491.0	3268.5	7000.0
491.0	3443.5	7000.0	491.0	3443.5	7196.8	491.0	3443.5	7393.7
491.0	3268.5	2000.0	491.0	3268.5	2175.2	491.0	3268.5	2350.3
491.0	3268.5	2525.5	491.0	3443.5	2000.0	491.0	3443.5	2174.2
491.0	3443.5	2348.4	491.0	3443.5	2522.6	491.0	3268.5	1326.8
491.0	3268.5	1465.3	491.0	3268.5	1603.8	491.0	3268.5	1742.4
491.0	3443.5	1326.8	491.0	3443.5	1464.3	491.0	3443.5	1601.9
491.0	3443.5	1739.5	491.0	3515.2	1275.4	491.0	3711.2	1275.4
491.0	3907.2	1275.4	491.0	4103.2	1275.4	491.0	4299.2	1275.4
491.0	4495.2	1275.4	491.0	4691.2	1275.4	491.0	4887.2	1275.4
491.0	5083.2	1275.4	491.0	5279.2	1275.4	491.0	5475.2	1275.4
491.0	5671.2	1275.4	491.0	5867.2	1275.4	491.0	6063.2	1275.4
491.0	3517.5	1100.4	491.0	3713.7	1100.4	491.0	3910.0	1100.4
491.0	4106.3	1100.4	491.0	4302.5	1100.4	491.0	4498.8	1100.4
491.0	4695.1	1100.4	491.0	4891.3	1100.4	491.0	5087.6	1100.4
491.0	5283.9	1100.4	491.0	5480.1	1100.4	491.0	5676.4	1100.4
491.0	5872.7	1100.4	491.0	6068.9	1100.4	491.0	-4471.5	-5524.8
491.0	-4451.9	-5699.8	491.0	-4273.9	-5524.8	491.0	-4254.7	-5699.8
491.0	-4076.4	-5524.8	491.0	-4057.5	-5699.8	491.0	-3878.8	-5524.8
491.0	-3860.3	-5699.8	491.0	-3681.2	-5524.8	491.0	-3663.1	-5699.8
491.0	-3483.6	-5524.8	491.0	-3465.9	-5699.7	491.0	-3286.0	-5524.7
491.0	-3268.7	-5699.7	491.0	-3088.5	-5524.7	491.0	-3071.5	-5699.7
491.0	-2890.9	-5524.7	491.0	-2874.3	-5699.7	491.0	-2693.3	-5524.7
491.0	-2677.1	-5699.7	491.0	-2495.7	-5524.7	491.0	-2479.9	-5699.7
491.0	-2298.2	-5524.7	491.0	-2282.7	-5699.7	491.0	-2100.6	-5524.7
491.0	-2085.4	-5699.7	491.0	-1903.0	-5524.7	491.0	-1888.2	-5699.7
491.0	-1705.4	-5524.7	491.0	-1691.0	-5699.7	491.0	-1507.9	-5524.7
491.0	-1493.8	-5699.7	491.0	-1310.3	-5524.7	491.0	-1296.6	-5699.7
491.0	-1112.7	-5524.7	491.0	-1099.4	-5699.7	491.0	-915.1	-5524.7
491.0	-902.2	-5699.7	491.0	-717.6	-5524.7	491.0	-705.0	-5699.7
491.0	-520.0	-5524.7	491.0	-507.8	-5699.7	491.0	-322.4	-5524.7
491.0	-310.6	-5699.7	491.0	-124.8	-5524.7	491.0	-113.4	-5699.7
491.0	72.7	-5524.7	491.0	83.8	-5699.7	491.0	270.3	-5524.7
491.0	281.0	-5699.7	491.0	467.9	-5524.7	491.0	478.2	-5699.7
491.0	665.5	-5524.7	491.0	675.4	-5699.7	491.0	863.0	-5524.7
491.0	872.6	-5699.7	491.0	1060.6	-5524.7	491.0	1069.9	-5699.7
491.0	1258.2	-5524.7	491.0	1267.1	-5699.6	491.0	1455.8	-5524.6
491.0	1464.3	-5699.6	491.0	1653.3	-5524.6	491.0	1861.5	-5699.6
491.0	1850.9	-5524.6	491.0	1858.7	-5699.6	491.0	2048.5	-5524.6
491.0	2055.9	-5699.6	491.0	2246.1	-5524.6	491.0	2253.1	-5699.6
491.0	2443.6	-5524.6	491.0	2450.3	-5699.6	491.0	2641.2	-5524.6
491.0	2647.5	-5699.6	491.0	2838.8	-5524.6	491.0	2844.7	-5699.6
491.0	3036.4	-5524.6	491.0	3041.9	-5699.6	491.0	3233.9	-5524.6
491.0	3239.1	-5699.6	491.0	3431.5	-5524.6	491.0	3436.3	-5699.6
491.0	3629.1	-5524.6	491.0	3633.5	-5699.6	491.0	3826.7	-5524.6
491.0	3830.7	-5699.6	491.0	4024.2	-5524.6	491.0	4027.9	-5699.6
491.0	4221.8	-5524.6	491.0	4225.1	-5699.6	491.0	4419.4	-5524.6
491.0	4422.4	-5699.6	491.0	4617.0	-5524.6	491.0	4619.6	-5699.6
491.0	4814.5	-5524.6	491.0	4816.8	-5699.6	491.0	5012.1	-5524.6
491.0	5014.0	-5699.6	491.0	5209.7	-5524.6	491.0	5211.2	-5699.6
491.0	5407.3	-5524.6	491.0	5408.4	-5699.6	491.0	5604.8	-5524.6
491.0	5605.6	-5699.6	491.0	5802.4	-5524.6	491.0	5802.8	-5699.6
491.0	6000.0	-5699.5	491.0	6000.0	-5524.6			

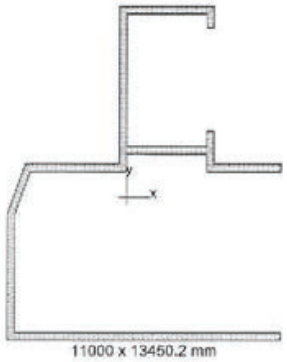
5. Factored Loads and Moments with Corresponding Capacity Ratios

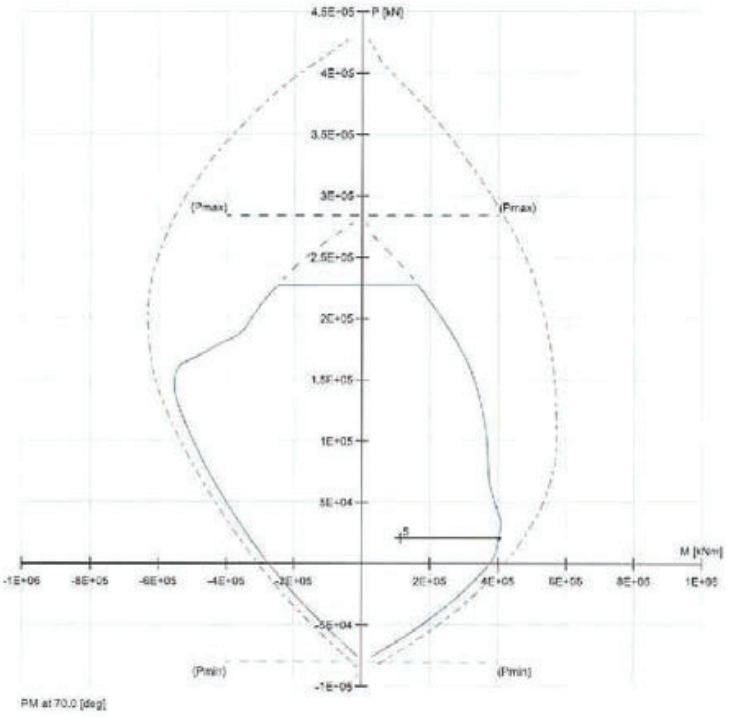
NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kN	kNm	kNm	kN	kNm	kNm	mm			
1	16520.00	99230.00	16060.00	16520.00	557188.38	90178.87	633	0.11603	0.900	0.18
2	48440.00	9740.00	500.00	48440.00	728695.69	37407.38	946	0.05605	0.900	0.01
3	36930.00	32140.00	81260.00	36930.00	148435.59	375291.72	5598	0.00327	0.756	0.22
4	37390.00	146470.00	21940.00	37390.00	671629.19	100604.51	1449	0.05925	0.900	0.22
5	20680.00	38100.00	106430.00	20680.00	135771.73	379270.03	4686	0.00445	0.854	0.28

6. Diagrams

6.1. PM at θ=70 [deg]





PM at 70.0 [deg]

General Information	
Project	Zone C DD
Column	LW Nos.1&3
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

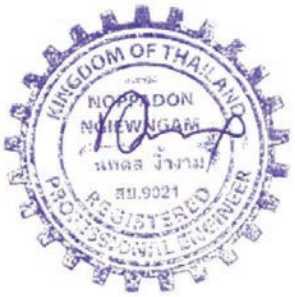
Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1.1895e+007 mm²
I_x	2.41724e+014 mm⁴
I_y	1.17672e+014 mm⁴

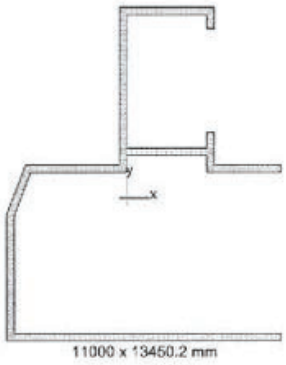
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	223399 mm²
Rho	1.88 %
Min. clear spacing	20 mm

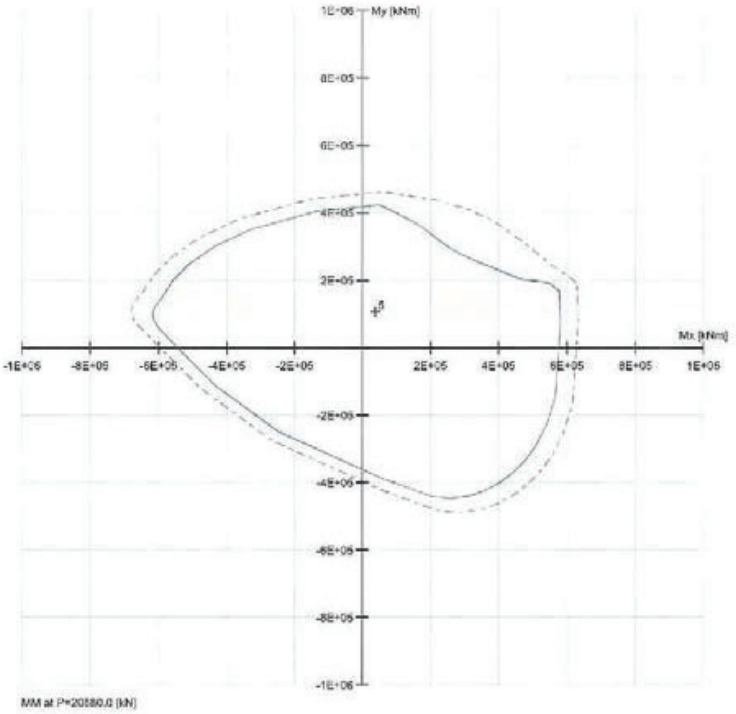
No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
5	20680.0	38100.0	106430.0	20680.00	135771.73	379270.03	0.28

Max. Capacity Ratio: 0.28



6.2. MM at P=20680 [kN]





General Information	
Project	Zone C DD
Column	LW Nos.1&3
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1.1895e+007 mm²
I_x	2.41724e+014 mm⁴
I_y	1.17672e+014 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	—
Cover to	—
Clear cover	—
Bars	—
Confinement type	Tied
Total steel area, A_s	223399 mm²
Rho	1.88 %
Min. clear spacing	20 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
5	20680.0	38100.0	106430.0	20680.00	135771.73	379270.03	0.28

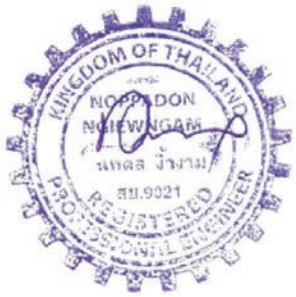
Max. Capacity Ratio: 0.28

TABLE: Pier Forces W6\D12 (2nd -3rd Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
3 SFL	AComb34-1	-1613	143	749	3927	13063	1737
2 SFL	AComb2-1	-4749	122	57	112	865	523
2 SFL	AComb27-1	-3629	633	178	470	2263	8706
2 SFL	AComb29-1	-3520	197	780	3732	15744	2021
3 SFL	AComb34-1	-1613	143	749	3927	13063	1737
2 SFL	AComb29-1	-3664	197	780	3732	18707	2771
2 SFL	AComb27-1	-3774	633	178	470	2939	11111

TABLE: Pier Forces W6\D12 (4th -Roof Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
R	AComb34-1	-86	67	118	899	379	129
4 SFL	AComb2-1	-3682	119	5	224	29	580
4 SFL	AComb27-1	-2746	527	111	453	877	4677
4 SFL	AComb34-1	-1373	135	718	3846	9743	1302
4 SFL	AComb30-1	-2876	11	680	3936	9618	1256
4 SFL	AComb34-1	-1513	135	718	3846	13261	1963
4 SFL	AComb27-1	-2933	527	111	453	1422	7259



spColumn

spColumn v6.50
Computer program for the Strength Design of Reinforced Concrete Sections
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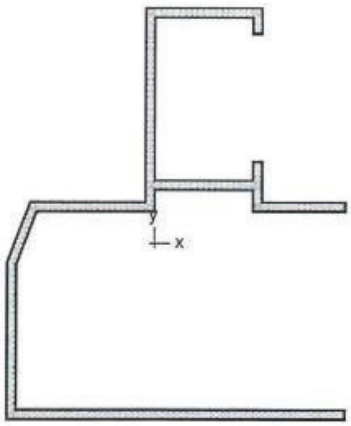

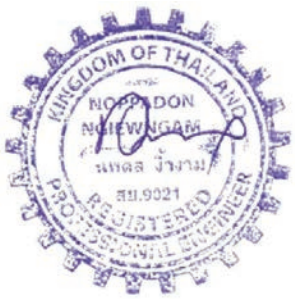

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Figure 1: Column section 4

1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-6...\W6-D12 2-3.col
Project	Zone C DD
Column	LW Nos.1&3
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

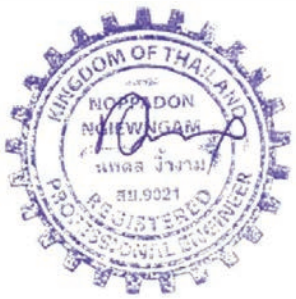
2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	1.1895e+007	mm²
I_x	2.41724e+014	mm⁴
I_y	1.17672e+014	mm⁴
r_x	4507.94	mm
r_y	3145.24	mm
X_o	-0.00021904	mm
Y_o	-0.000860867	mm



3.2. Section Figure

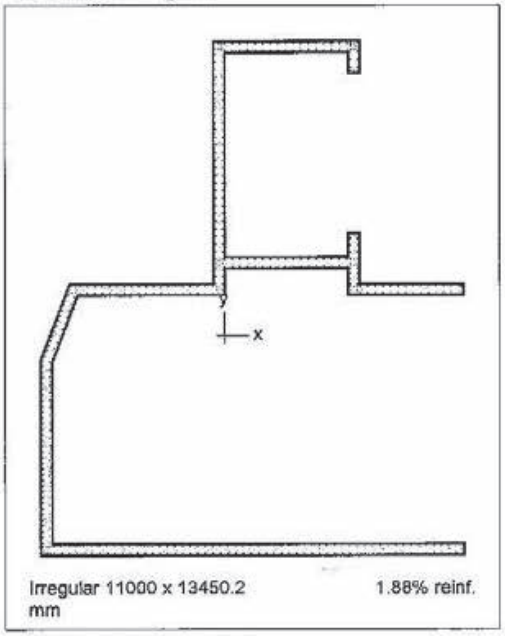


Figure 1: Column section

3.3. Exterior Points

Points	X mm	Y mm	Points	X mm	Y mm	Points	X mm	Y mm
1	5.9	1737.9	2	5.9	1037.9	3	-3835.5	1037.9
4	-4494.1	-707.2	5	-4494.1	-5462.3	6	6205.8	-5462.0
7	6205.8	-5762.0	8	-4794.1	-5762.3	9	-4794.2	-652.5
10	-4042.9	1337.9	11	-294.0	1337.9	12	-294.0	7687.9
13	3506.0	7687.9	14	3506.0	6837.9	15	3206.0	6837.9
16	3206.0	7387.7	17	6.0	7387.7	18	6.0	2037.9
19	3206.0	2037.9	20	3206.0	2662.9	21	3506.0	2662.9
22	3506.0	1337.9	23	6205.9	1337.9	24	6205.8	1037.9
25	3206.0	1037.9	26	3206.0	1737.9			

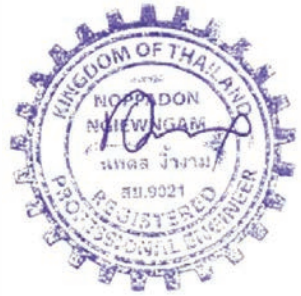
4. Reinforcement

4.1. Bar Set: User-defined

Bar	Diameter mm	Area mm²	Bar	Diameter mm	Area mm²	Bar	Diameter mm	Area mm²
#20	20.00	314.16						

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#20 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65



4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	223399 mm²
Rho	1.88 %
Minimum clear spacing	20 mm

4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
314.2	6135.8	-5532.1	314.2	6135.8	-5612.0	314.2	6135.8	-5692.0
314.2	-4724.1	-5691.8	314.2	-4564.1	-5691.8	314.2	-4644.1	-5691.9
314.2	-4724.1	-5541.9	314.2	-4564.1	-5541.9	314.2	-4644.1	-5541.9
314.2	-4724.1	-5616.8	314.2	-4564.1	-5616.8	314.2	-3884.0	1108.2
314.2	-3969.5	1108.2	314.2	-4054.8	1108.7	314.2	-3894.8	1268.1
314.2	-3995.1	1266.9	314.2	-4024.0	1190.3	314.2	-3854.5	1186.4
314.2	-64.1	1108.3	314.2	-144.1	1108.3	314.2	-224.1	1108.3
314.2	-224.1	1183.3	314.2	-64.1	1183.3	314.2	-64.1	1258.3
314.2	-144.1	1258.3	314.2	-224.1	1258.3	314.2	-64.1	1808.3
314.2	-144.1	1808.3	314.2	-224.1	1808.3	314.2	-64.1	1883.3
314.2	-224.1	1883.3	314.2	-224.1	1958.3	314.2	-144.1	1958.3
314.2	-64.1	1958.3	314.2	3276.0	1808.3	314.2	3356.0	1808.3
314.2	3436.0	1808.3	314.2	3276.0	1883.3	314.2	3436.0	1883.3
314.2	3436.0	1958.3	314.2	3356.0	1958.3	314.2	3276.0	1958.3
314.2	3436.0	2592.9	314.2	3276.0	2592.9	314.2	3356.0	2592.9
314.2	-64.1	7617.8	314.2	-144.1	7617.8	314.2	-224.1	7617.8
314.2	-64.1	7542.7	314.2	-224.1	7542.7	314.2	-224.1	7467.7
314.2	-144.1	7467.7	314.2	-64.1	7467.7	314.2	3436.0	6907.9
314.2	3276.0	6907.9	314.2	3356.0	6907.9	314.2	3276.0	7617.9
314.2	3356.0	7617.9	314.2	3436.0	7617.9	314.2	3276.0	7542.9
314.2	3436.0	7542.9	314.2	3436.0	7467.9	314.2	3356.0	7467.9
314.2	3276.0	7467.9	314.2	6135.8	1107.9	314.2	6135.8	1187.9
314.2	6135.8	1267.8	314.2	3276.0	1108.3	314.2	3356.0	1108.3
314.2	3276.0	1183.3	314.2	3276.0	1258.3	314.2	3356.0	1258.3
314.2	3435.9	1258.3	314.2	3435.9	1183.3	314.2	3435.9	1108.3
491.0	-4731.6	-663.9	491.0	-4731.6	-813.2	491.0	-4731.6	-962.5
491.0	-4731.6	-1111.8	491.0	-4731.6	-1261.1	491.0	-4731.6	-1410.4
491.0	-4731.6	-1559.7	491.0	-4731.6	-1709.0	491.0	-4731.6	-1858.3
491.0	-4731.6	-2007.6	491.0	-4731.6	-2156.9	491.0	-4731.6	-2306.2
491.0	-4731.6	-2455.5	491.0	-4731.6	-2604.8	491.0	-4731.6	-2754.1
491.0	-4731.6	-2903.4	491.0	-4731.6	-3052.7	491.0	-4731.6	-3201.9
491.0	-4731.6	-3351.2	491.0	-4731.6	-3500.5	491.0	-4731.6	-3649.8
491.0	-4731.6	-3799.1	491.0	-4731.6	-3948.4	491.0	-4731.6	-4097.7
491.0	-4731.6	-4247.0	491.0	-4731.6	-4396.3	491.0	-4731.6	-4545.6
491.0	-4731.6	-4694.9	491.0	-4731.6	-4844.2	491.0	-4731.6	-4993.5
491.0	-4731.6	-5142.8	491.0	-4731.6	-5292.1	491.0	-4731.6	-5441.4
491.0	-4682.0	-532.2	491.0	-4632.3	-400.6	491.0	-4582.6	-268.9
491.0	-4532.9	-137.2	491.0	-4483.2	-5.5	491.0	-4433.5	126.1
491.0	-4383.8	257.8	491.0	-4334.1	389.5	491.0	-4284.4	521.1
491.0	-4234.7	652.8	491.0	-4185.0	784.5	491.0	-4135.3	916.2
491.0	-4085.6	1047.8	491.0	-4556.7	-695.8	491.0	-4556.6	-893.7
491.0	-4556.6	-1091.6	491.0	-4556.6	-1289.5	491.0	-4556.6	-1487.4
491.0	-4556.6	-1685.3	491.0	-4556.6	-1883.2	491.0	-4556.6	-2081.2
491.0	-4556.6	-2279.1	491.0	-4556.6	-2477.0	491.0	-4556.6	-2674.9
491.0	-4556.6	-2872.8	491.0	-4556.6	-3070.7	491.0	-4556.6	-3268.6

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	-4556.6	-3466.5	491.0	-4556.6	-3664.4	491.0	-4556.6	-3862.3
491.0	-4556.6	-4060.2	491.0	-4556.6	-4258.1	491.0	-4556.6	-4456.0
491.0	-4556.6	-4653.9	491.0	-4556.6	-4851.8	491.0	-4556.6	-5049.7
491.0	-4556.6	-5247.6	491.0	-4556.6	-5445.5	491.0	-4492.6	-526.2
491.0	-4428.6	-356.7	491.0	-4364.6	-187.1	491.0	-4300.6	-17.5
491.0	-4236.6	152.1	491.0	-4172.6	321.7	491.0	-4108.6	491.3
491.0	-4044.6	660.8	491.0	-3980.6	830.4	491.0	-3916.6	1000.0
491.0	-3763.8	1275.4	491.0	-3572.9	1275.4	491.0	-3382.0	1275.4
491.0	-3191.1	1275.4	491.0	-3000.3	1275.4	491.0	-2809.4	1275.4
491.0	-2618.5	1275.4	491.0	-2427.6	1275.4	491.0	-2236.7	1275.4
491.0	-2045.9	1275.4	491.0	-1855.0	1275.4	491.0	-1664.1	1275.4
491.0	-1473.2	1275.4	491.0	-1282.4	1275.4	491.0	-1091.5	1275.4
491.0	-900.6	1275.4	491.0	-709.7	1275.4	491.0	-518.9	1275.4
491.0	-328.0	1275.4	491.0	-3768.6	1100.4	491.0	-3576.9	1100.4
491.0	-3385.2	1100.4	491.0	-3193.5	1100.4	491.0	-3001.9	1100.4
491.0	-2810.2	1100.4	491.0	-2618.5	1100.4	491.0	-2426.8	1100.4
491.0	-2235.1	1100.4	491.0	-2043.4	1100.4	491.0	-1851.8	1100.4
491.0	-1660.1	1100.4	491.0	-1468.4	1100.4	491.0	-1276.7	1100.4
491.0	-1085.0	1100.4	491.0	-893.4	1100.4	491.0	-701.7	1100.4
491.0	-510.0	1100.4	491.0	-318.3	1100.4	491.0	-231.5	1355.7
491.0	-231.5	1541.5	491.0	-231.5	1727.3	491.0	-56.6	1352.1
491.0	-56.6	1536.2	491.0	-56.6	1720.2	491.0	-231.5	2000.0
491.0	-231.5	2197.9	491.0	-231.5	2395.8	491.0	-231.5	2593.8
491.0	-231.5	2791.7	491.0	-231.5	2989.6	491.0	-231.5	3187.5
491.0	-231.5	3385.5	491.0	-231.5	3583.4	491.0	-231.5	3781.3
491.0	-231.5	3979.2	491.0	-231.5	4177.1	491.0	-231.5	4375.1
491.0	-231.5	4573.0	491.0	-231.5	4770.9	491.0	-231.5	4968.8
491.0	-231.5	5166.8	491.0	-231.5	5364.7	491.0	-231.5	5562.6
491.0	-231.5	5760.5	491.0	-231.5	5958.5	491.0	-231.5	6156.4
491.0	-231.5	6354.3	491.0	-231.5	6552.2	491.0	-231.5	6750.1
491.0	-231.5	6948.1	491.0	-231.5	7148.0	491.0	-231.5	7343.9
491.0	-56.5	2000.0	491.0	-56.5	2197.7	491.0	-56.5	2395.5
491.0	-56.5	2593.2	491.0	-56.5	2790.9	491.0	-56.5	2988.7
491.0	-56.5	3186.4	491.0	-56.5	3384.1	491.0	-56.5	3581.9
491.0	-56.5	3779.6	491.0	-56.5	3977.4	491.0	-56.5	4175.1
491.0	-56.5	4372.8	491.0	-56.5	4570.6	491.0	-56.5	4768.3
491.0	-56.5	4966.0	491.0	-56.5	5163.8	491.0	-56.5	5361.5
491.0	-56.5	5559.2	491.0	-56.5	5757.0	491.0	-56.5	5954.7
491.0	-56.5	6152.4	491.0	-56.5	6350.2	491.0	-56.5	6547.9
491.0	-56.5	6745.6	491.0	-56.5	6943.4	491.0	-56.5	7141.1
491.0	-56.5	7338.8	491.0	0.0	1975.4	491.0	198.4	1975.4
491.0	396.7	1975.4	491.0	595.1	1975.4	491.0	793.4	1975.4
491.0	991.8	1975.4	491.0	1190.1	1975.4	491.0	1388.5	1975.4
491.0	1586.8	1975.4	491.0	1785.2	1975.4	491.0	1983.6	1975.4
491.0	2181.9	1975.4	491.0	2380.3	1975.4	491.0	2578.6	1975.4
491.0	2777.0	1975.4	491.0	2975.3	1975.4	491.0	3173.7	1975.4
491.0	0.0	1800.4	491.0	198.7	1800.4	491.0	397.3	1800.4
491.0	596.0	1800.4	491.0	794.7	1800.4	491.0	993.4	1800.4
491.0	1192.0	1800.4	491.0	1390.7	1800.4	491.0	1589.4	1800.4
491.0	1788.0	1800.4	491.0	1986.7	1800.4	491.0	2185.4	1800.4
491.0	2384.1	1800.4	491.0	2582.7	1800.4	491.0	2781.4	1800.4
491.0	2980.1	1800.4	491.0	3178.7	1800.4	491.0	0.0	7625.4
491.0	198.9	7625.4	491.0	397.8	7625.4	491.0	596.6	7625.4
491.0	795.5	7625.4	491.0	994.4	7625.4	491.0	1193.3	7625.4
491.0	1392.2	7625.4	491.0	1591.1	7625.4	491.0	1789.9	7625.4
491.0	1988.8	7625.4	491.0	2187.7	7625.4	491.0	2386.6	7625.4
491.0	2585.5	7625.4	491.0	2784.4	7625.4	491.0	2983.2	7625.4
491.0	3182.1	7625.4	491.0	0.0	7450.2	491.0	199.1	7450.2
491.0	398.3	7450.2	491.0	597.4	7450.2	491.0	796.5	7450.2

STRUCTUREPOINT - spColumn v6.50
Licensed to: Dynamic Engineering Consultants Co., Ltd., License ID: 68794-1062303-4-2667D-XXXXX
C:\193018\GCDT\SPcloumn\10-11-63(C2 building)\Shear wall\W8-D12 2-3.col

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	995.7	7450.2	491.0	1194.8	7450.2	491.0	1393.9	7450.2
491.0	1593.1	7450.2	491.0	1792.2	7450.2	491.0	1991.3	7450.2
491.0	2190.5	7450.2	491.0	2389.6	7450.2	491.0	2588.7	7450.2
491.0	2787.9	7450.2	491.0	2987.0	7450.2	491.0	3186.1	7450.2
491.0	3268.5	7393.7	491.0	3268.5	7196.8	491.0	3268.5	7000.0
491.0	3443.5	7000.0	491.0	3443.5	7196.8	491.0	3443.5	7393.7
491.0	3268.5	2000.0	491.0	3268.5	2175.2	491.0	3268.5	2350.3
491.0	3268.5	2525.5	491.0	3443.5	2000.0	491.0	3443.5	2174.2
491.0	3443.5	2348.4	491.0	3443.5	2522.6	491.0	3268.5	1326.8
491.0	3268.5	1465.3	491.0	3268.5	1603.8	491.0	3268.5	1742.4
491.0	3443.5	1326.8	491.0	3443.5	1464.3	491.0	3443.5	1601.9
491.0	3443.5	1739.5	491.0	3515.2	1275.4	491.0	3711.2	1275.4
491.0	3907.2	1275.4	491.0	4103.2	1275.4	491.0	4299.2	1275.4
491.0	4495.2	1275.4	491.0	4691.2	1275.4	491.0	4887.2	1275.4
491.0	5083.2	1275.4	491.0	5279.2	1275.4	491.0	5475.2	1275.4
491.0	5671.2	1275.4	491.0	5867.2	1275.4	491.0	6063.2	1275.4
491.0	3517.5	1100.4	491.0	3713.7	1100.4	491.0	3910.0	1100.4
491.0	4106.3	1100.4	491.0	4302.5	1100.4	491.0	4498.8	1100.4
491.0	4695.1	1100.4	491.0	4891.3	1100.4	491.0	5087.6	1100.4
491.0	5283.9	1100.4	491.0	5480.1	1100.4	491.0	5676.4	1100.4
491.0	5872.7	1100.4	491.0	6068.9	1100.4	491.0	-4471.5	-5524.8
491.0	-4451.9	-5699.8	491.0	-4273.9	-5524.8	491.0	-4254.7	-5699.8
491.0	-4076.4	-5524.8	491.0	-4057.5	-5699.8	491.0	-3878.8	-5524.8
491.0	-3860.3	-5699.8	491.0	-3681.2	-5524.8	491.0	-3663.1	-5699.8
491.0	-3483.6	-5524.8	491.0	-3465.9	-5699.7	491.0	-3286.0	-5524.7
491.0	-3268.7	-5699.7	491.0	-3088.5	-5524.7	491.0	-3071.5	-5699.7
491.0	-2890.9	-5524.7	491.0	-2874.3	-5699.7	491.0	-2693.3	-5524.7
491.0	-2677.1	-5699.7	491.0	-2495.7	-5524.7	491.0	-2479.9	-5699.7
491.0	-2298.2	-5524.7	491.0	-2282.7	-5699.7	491.0	-2100.6	-5524.7
491.0	-2085.4	-5699.7	491.0	-1903.0	-5524.7	491.0	-1888.2	-5699.7
491.0	-1705.4	-5524.7	491.0	-1691.0	-5699.7	491.0	-1507.9	-5524.7
491.0	-1493.8	-5699.7	491.0	-1310.3	-5524.7	491.0	-1296.6	-5699.7
491.0	-1112.7	-5524.7	491.0	-1099.4	-5699.7	491.0	-915.1	-5524.7
491.0	-902.2	-5699.7	491.0	-717.6	-5524.7	491.0	-705.0	-5699.7
491.0	-520.0	-5524.7	491.0	-507.8	-5699.7	491.0	-322.4	-5524.7
491.0	-310.6	-5699.7	491.0	-124.8	-5524.7	491.0	-113.4	-5699.7
491.0	72.7	-5524.7	491.0	83.8	-5699.7	491.0	270.3	-5524.7
491.0	281.0	-5699.7	491.0	467.9	-5524.7	491.0	478.2	-5699.7
491.0	665.5	-5524.7	491.0	675.4	-5699.7	491.0	863.0	-5524.7
491.0	872.6	-5699.7	491.0	1060.6	-5524.7	491.0	1069.9	-5699.7
491.0	1258.2	-5524.7	491.0	1267.1	-5699.6	491.0	1455.8	-5524.6
491.0	1464.3	-5699.6	491.0	1653.3	-5524.6	491.0	1661.5	-5699.6
491.0	1850.9	-5524.6	491.0	1858.7	-5699.6	491.0	2048.5	-5524.6
491.0	2055.9	-5699.6	491.0	2246.1	-5524.6	491.0	2253.1	-5699.6
491.0	2443.6	-5524.6	491.0	2450.3	-5699.6	491.0	2641.2	-5524.6
491.0	2647.5	-5699.6	491.0	2838.8	-5524.6	491.0	2844.7	-5699.6
491.0	3036.4	-5524.6	491.0	3041.9	-5699.6	491.0	3233.9	-5524.6
491.0	3239.1	-5699.6	491.0	3431.5	-5524.6	491.0	3436.3	-5699.6
491.0	3629.1	-5524.6	491.0	3633.5	-5699.6	491.0	3826.7	-5524.6
491.0	3830.7	-5699.6	491.0	4024.2	-5524.6	491.0	4027.9	-5699.6
491.0	4221.8	-5524.6	491.0	4225.1	-5699.6	491.0	4419.4	-5524.6
491.0	4422.4	-5699.6	491.0	4617.0	-5524.6	491.0	4619.6	-5699.6
491.0	4814.5	-5524.6	491.0	4816.8	-5699.6	491.0	5012.1	-5524.6
491.0	5014.0	-5699.6	491.0	5209.7	-5524.6	491.0	5211.2	-5699.6
491.0	5407.3	-5524.6	491.0	5408.4	-5699.6	491.0	5604.8	-5524.6
491.0	5605.6	-5699.6	491.0	5802.4	-5524.6	491.0	5802.8	-5699.6
491.0	6000.0	-5699.5	491.0	6000.0	-5524.6			

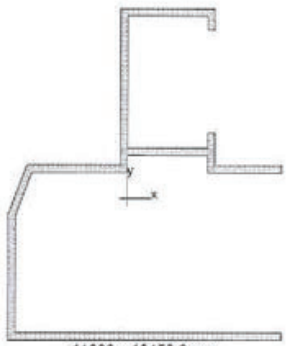
5. Factored Loads and Moments with Corresponding Capacity Ratios

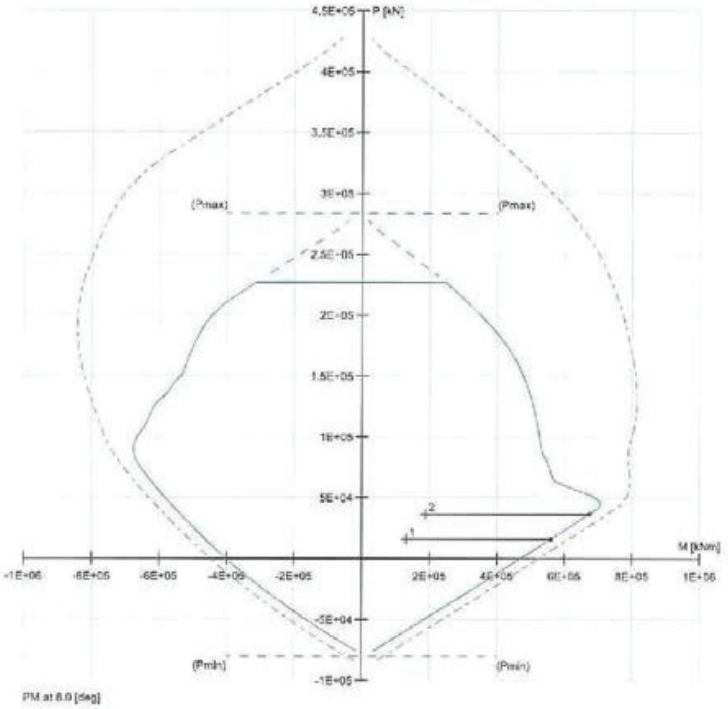
NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kN	kNm	kNm	kN	kNm	kNm	mm			
1	16130.00	130630.00	17370.00	16130.00	555489.19	73864.00	392	0.13437	0.900	0.24
2	36640.00	187070.00	27710.00	36640.00	667664.50	98898.77	1299	0.06632	0.900	0.28
3	37740.00	29390.00	111110.00	37740.00	106613.48	403056.44	5144	0.00378	0.798	0.28

6. Diagrams

6.1. PM at θ=8 [deg]





General Information	
Project	Zone C DD
Column	LW Nos.1&3
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials		
f'_c	35	MPa
E_c	27805.6	MPa
f_y	400	MPa
E_s	200000	MPa

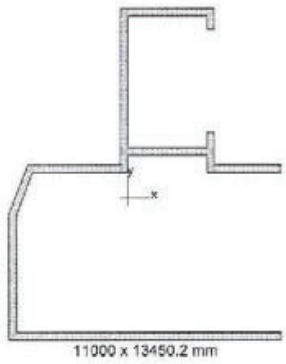
Section		
Type	Irregular	
A_g	1.1895e+007	mm²
I_x	2.41724e+014	mm⁴
I_y	1.17672e+014	mm⁴

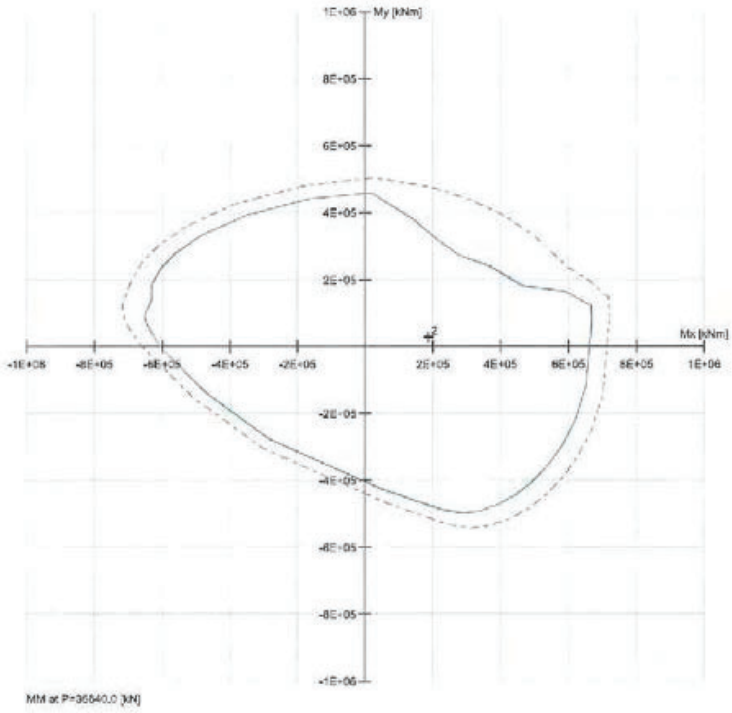
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	223399 mm²
Rho	1.88 %
Min. clear spacing	20 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
2	36640.0	187070.0	27710.0	36640.00	667664.50	98898.77	0.28
1	16130.0	130630.0	17370.0	16130.00	555489.19	73864.00	0.24

Max. Capacity Ratio: 0.28

6.2. MM at P=36640 [kN]





General Information	
Project	Zone C DD
Column	LW Nos.1&3
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	1.1895e+007 mm²
I_x	2.41724e+014 mm⁴
I_y	1.17672e+014 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	—
Cover to	—
Clear cover	—
Bars	—
Confinement type	Tied
Total steel area, A_s	223399 mm²
Rho	1.88 %
Min. clear spacing	20 mm

No.	P_n kN	M_{nx} kNm	M_{ny} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
2	36640.0	187070.0	27710.0	36640.00	667664.50	98898.77	0.28

Max. Capacity Ratio: 0.28

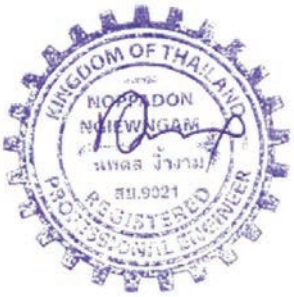
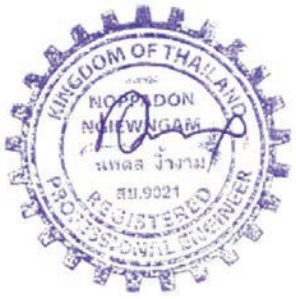


TABLE: Pier Forces W7 (2nd -3rd Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
3 SFL	AComb33-1	-1074	100	248	1277	2833	543
2 SFL	AComb2-1	-3613	47	13	42	54	213
2 SFL	AComb28-1	-2561	497	95	322	1853	7348
2 SFL	AComb30-1	-2436	130	278	1329	3650	534
3 SFL	AComb29-1	-2817	76	273	1468	2779	802
2 SFL	AComb34-1	-2019	81	265	1382	4832	1238
2 SFL	AComb27-1	-3156	426	72	267	2159	9553

TABLE: Pier Forces W7 (4th -Roof Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
R	AComb32-1	-60	32	83	424	168	82
4 SFL	AComb2-1	-2829	33	4	164	123	100
4 SFL	AComb28-1	-1953	432	103	201	1395	4115
4 SFL	AComb29-1	-2391	63	246	1396	1890	576
4 SFL	AComb29-1	-2391	63	246	1396	1890	576
4 SFL	AComb30-1	-2029	110	244	1157	3291	740
4 SFL	AComb27-1	-2467	385	106	38	1706	6380



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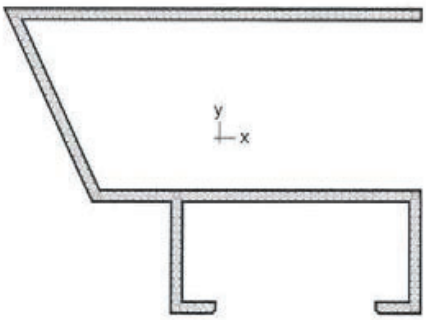

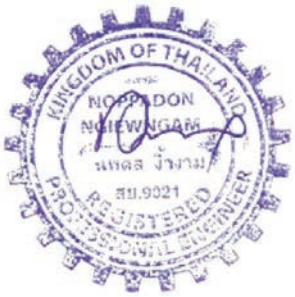

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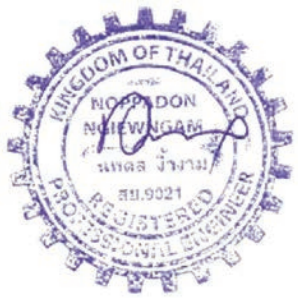


1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63(C2...\W7 2-3.col
Project	Zone C DD
Column	LW No.4
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	9.63673e+006	mm²
I_x	7.38774e+013	mm⁴
I_y	1.09165e+014	mm⁴
r_x	2768.8	mm
r_y	3365.72	mm
X_o	0	mm
Y_o	0.000770386	mm

3.2. Section Figure

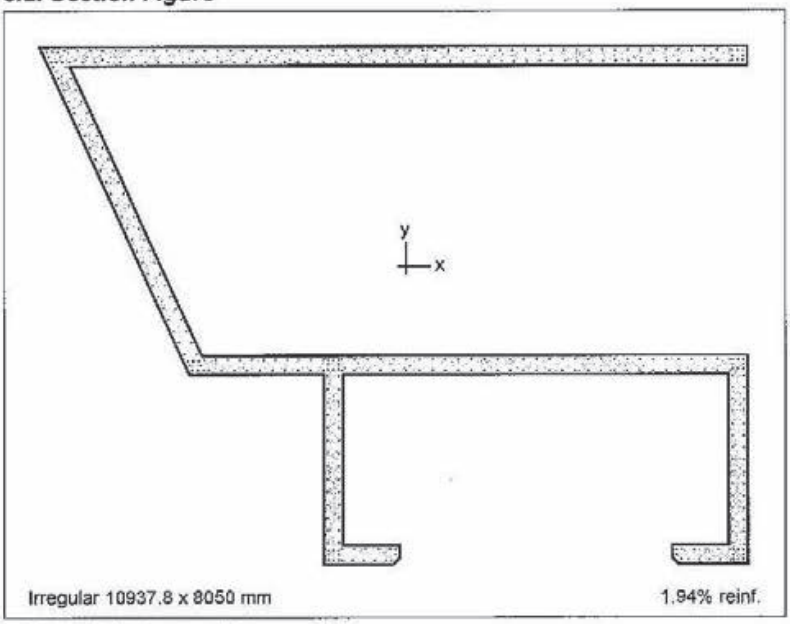


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-5200.7	3112.4	2	5271.9	3112.4	3	5271.9	3412.4
4	-5665.9	3412.4	5	-3356.1	-1687.6	6	-1278.1	-1687.6
7	-1278.1	-4637.6	8	-203.1	-4637.6	9	-103.1	-4537.6
10	-103.1	-4337.6	11	-978.1	-4337.6	12	-978.1	-1687.6
13	4971.9	-1687.6	14	4971.9	-4337.6	15	4096.9	-4337.6
16	4096.9	-4537.6	17	4196.9	-4637.6	18	5271.9	-4637.6
19	5271.9	-1387.6	20	-3162.6	-1387.6			

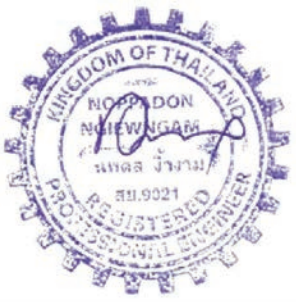
4. Reinforcement

4.1. Bar Set: User-defined

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#20	20.00	314.16						

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#20 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65



4.3. Arrangement

Pattern	Irregular
Bar layout	---

Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	187071 mm²
Rho	1.94 %
Minimum clear spacing	18 mm

4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	4166.9	-4407.6	491.0	4232.6	-4567.4	491.0	4166.9	-4487.3
491.0	5201.9	-4567.1	491.0	5121.9	-4567.1	491.0	5041.9	-4567.1
491.0	5201.9	-4492.1	491.0	5041.9	-4492.1	491.0	5041.9	-4417.1
491.0	5121.9	-4417.1	491.0	5201.9	-4417.1	491.0	5041.9	-1457.6
491.0	5201.9	-1457.6	491.0	5041.9	-1533.0	491.0	5201.9	-1533.0
491.0	5041.9	-1608.0	491.0	5201.9	-1608.0	491.0	5121.9	-1457.5
491.0	5121.9	-1607.9	491.0	-173.1	-4407.6	491.0	-238.8	-4567.4
491.0	-173.1	-4487.3	491.0	-1208.1	-4567.1	491.0	-1128.1	-4567.1
491.0	-1048.1	-4567.1	491.0	-1208.1	-4492.1	491.0	-1048.1	-4492.1
491.0	-1048.1	-4417.1	491.0	-1128.1	-4417.1	491.0	-1208.1	-4417.1
491.0	-1048.1	-1457.6	491.0	-1208.1	-1457.6	491.0	-1048.1	-1533.0
491.0	-1208.1	-1533.0	491.0	-1048.1	-1608.0	491.0	-1208.1	-1608.0
491.0	-1128.1	-1457.5	491.0	-1128.1	-1607.9	491.0	-3135.2	-1617.8
491.0	-3223.0	-1617.8	491.0	-3310.9	-1617.8	491.0	-3346.3	-1539.6
491.0	-3170.6	-1539.6	491.0	-3202.5	-1469.2	491.0	-3290.3	-1469.2
491.0	-3378.2	-1469.2	491.0	5041.9	3342.4	491.0	5201.9	3342.4
491.0	5041.9	3267.0	491.0	5201.9	3267.0	491.0	5041.9	3192.0
491.0	5201.9	3192.0	491.0	5121.9	3342.5	491.0	5121.9	3192.1
491.0	-5381.8	3342.7	491.0	-5345.4	3262.4	491.0	-5469.6	3342.7
491.0	-5557.4	3342.7	491.0	-5521.1	3262.4	491.0	-5484.7	3182.2
491.0	-5396.9	3182.2	491.0	-5309.1	3182.2	491.0	-5274.7	3349.9
491.0	-5077.1	3349.9	491.0	-4879.5	3349.9	491.0	-4681.9	3349.9
491.0	-4484.4	3349.9	491.0	-4286.8	3349.9	491.0	-4089.2	3349.9
491.0	-3891.6	3349.9	491.0	-3694.0	3349.9	491.0	-3496.4	3349.9
491.0	-3298.8	3349.9	491.0	-3101.2	3349.9	491.0	-2903.6	3349.9
491.0	-2706.0	3349.9	491.0	-2508.4	3349.9	491.0	-2310.9	3349.9
491.0	-2113.3	3349.9	491.0	-1915.7	3349.9	491.0	-1718.1	3349.9
491.0	-1520.5	3349.9	491.0	-1322.9	3349.9	491.0	-1125.3	3349.9
491.0	-927.7	3349.9	491.0	-730.1	3349.9	491.0	-532.5	3349.9
491.0	-334.9	3349.9	491.0	-137.4	3349.9	491.0	60.2	3349.9
491.0	257.8	3349.9	491.0	455.4	3349.9	491.0	653.0	3349.9
491.0	850.6	3349.9	491.0	1048.2	3349.9	491.0	1245.8	3349.9
491.0	1443.4	3349.9	491.0	1641.0	3349.9	491.0	1838.5	3349.9
491.0	2036.1	3349.9	491.0	2233.7	3349.9	491.0	2431.3	3349.9
491.0	2628.9	3349.9	491.0	2826.5	3349.9	491.0	3024.1	3349.9
491.0	3221.7	3349.9	491.0	3419.3	3349.9	491.0	3616.9	3349.9
491.0	3814.5	3349.9	491.0	4012.0	3349.9	491.0	4209.6	3349.9
491.0	4407.2	3349.9	491.0	4604.8	3349.9	491.0	4802.4	3349.9
491.0	5000.0	3349.9	491.0	-5201.1	3174.9	491.0	-5005.0	3174.9
491.0	-4808.8	3174.9	491.0	-4612.6	3174.9	491.0	-4416.4	3174.9
491.0	-4220.3	3174.9	491.0	-4024.1	3174.9	491.0	-3827.9	3174.9
491.0	-3631.7	3174.9	491.0	-3435.6	3174.9	491.0	-3239.4	3174.9
491.0	-3043.2	3174.9	491.0	-2847.0	3174.9	491.0	-2650.9	3174.9
491.0	-2454.7	3174.9	491.0	-2258.5	3174.9	491.0	-2062.3	3174.9
491.0	-1866.2	3174.9	491.0	-1670.0	3174.9	491.0	-1473.8	3174.9
491.0	-1277.6	3174.9	491.0	-1081.5	3174.9	491.0	-885.3	3174.9
491.0	-689.1	3174.9	491.0	-492.9	3174.9	491.0	-296.8	3174.9
491.0	-100.6	3174.9	491.0	95.6	3174.9	491.0	291.8	3174.9
491.0	488.0	3174.9	491.0	684.1	3174.9	491.0	880.3	3174.9

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	1076.5	3174.9	491.0	1272.7	3174.9	491.0	1468.8	3174.9
491.0	1665.0	3174.9	491.0	1861.2	3174.9	491.0	2057.4	3174.9
491.0	2253.5	3174.9	491.0	2449.7	3174.9	491.0	2645.9	3174.9
491.0	2842.1	3174.9	491.0	3038.2	3174.9	491.0	3234.4	3174.9
491.0	3430.6	3174.9	491.0	3626.8	3174.9	491.0	3822.9	3174.9
491.0	4019.1	3174.9	491.0	4215.3	3174.9	491.0	4411.5	3174.9
491.0	4607.6	3174.9	491.0	4803.8	3174.9	491.0	5000.0	3174.9
491.0	-5456.0	3100.6	491.0	-5375.1	2921.9	491.0	-5294.2	2743.2
491.0	-5213.2	2564.5	491.0	-5132.3	2385.6	491.0	-5051.4	2207.1
491.0	-4970.4	2028.4	491.0	-4889.5	1849.7	491.0	-4808.6	1671.0
491.0	-4727.6	1492.3	491.0	-4646.7	1313.6	491.0	-4565.8	1134.9
491.0	-4484.8	956.2	491.0	-4403.9	777.5	491.0	-4323.0	598.8
491.0	-4242.1	420.1	491.0	-4161.1	241.4	491.0	-4080.2	62.7
491.0	-3999.3	-116.0	491.0	-3918.3	-294.7	491.0	-3837.4	-473.4
491.0	-3756.5	-652.1	491.0	-3675.5	-830.8	491.0	-3594.6	-1009.5
491.0	-3513.7	-1188.1	491.0	-3432.7	-1366.8	491.0	-5264.2	3101.2
491.0	-5183.2	2922.3	491.0	-5102.1	2743.4	491.0	-5021.1	2564.4
491.0	-4940.1	2385.5	491.0	-4859.0	2206.6	491.0	-4778.0	2027.7
491.0	-4697.0	1848.7	491.0	-4615.9	1669.8	491.0	-4534.9	1490.9
491.0	-4453.8	1312.0	491.0	-4372.8	1133.0	491.0	-4291.8	954.1
491.0	-4210.7	775.2	491.0	-4129.7	596.3	491.0	-4048.7	417.3
491.0	-3967.6	238.4	491.0	-3886.6	59.5	491.0	-3805.6	-119.4
491.0	-3724.5	-298.4	491.0	-3643.5	-477.3	491.0	-3562.5	-656.2
491.0	-3481.4	-835.2	491.0	-3400.4	-1014.1	491.0	-3319.4	-1193.0
491.0	-3238.3	-1371.9	491.0	-3129.0	-1450.1	491.0	-2945.1	-1450.1
491.0	-2761.1	-1450.1	491.0	-2577.1	-1450.1	491.0	-2393.1	-1450.1
491.0	-2209.2	-1450.1	491.0	-2025.2	-1450.1	491.0	-1841.2	-1450.1
491.0	-1657.2	-1450.1	491.0	-1473.3	-1450.1	491.0	-1289.3	-1450.1
491.0	-3052.3	-1625.1	491.0	-2855.8	-1625.1	491.0	-2659.4	-1625.1
491.0	-2462.9	-1625.1	491.0	-2266.5	-1625.1	491.0	-2070.0	-1625.1
491.0	-1873.5	-1625.1	491.0	-1677.1	-1625.1	491.0	-1480.6	-1625.1
491.0	-1284.2	-1625.1	491.0	-1000.0	-1450.1	491.0	-800.0	-1450.1
491.0	-600.0	-1450.1	491.0	-400.0	-1450.1	491.0	-200.0	-1450.1
491.0	0.0	-1450.1	491.0	200.0	-1450.1	491.0	400.0	-1450.1
491.0	600.0	-1450.1	491.0	800.0	-1450.1	491.0	1000.0	-1450.1
491.0	1200.0	-1450.1	491.0	1400.0	-1450.1	491.0	1600.0	-1450.1
491.0	1800.0	-1450.1	491.0	2000.0	-1450.1	491.0	2200.0	-1450.1
491.0	2400.0	-1450.1	491.0	2600.0	-1450.1	491.0	2800.0	-1450.1
491.0	3000.0	-1450.1	491.0	3200.0	-1450.1	491.0	3400.0	-1450.1
491.0	3600.0	-1450.1	491.0	3800.0	-1450.1	491.0	4000.0	-1450.1
491.0	4200.0	-1450.1	491.0	4400.0	-1450.1	491.0	4600.0	-1450.1
491.0	4800.0	-1450.1	491.0	5000.0	-1450.1	491.0	-1000.0	-1625.1
491.0	-800.0	-1625.1	491.0	-600.0	-1625.1	491.0	-400.0	-1625.1
491.0	-200.0	-1625.1	491.0	0.0	-1625.1	491.0	200.0	-1625.1
491.0	400.0	-1625.1	491.0	600.0	-1625.1	491.0	800.0	-1625.1
491.0	1000.0	-1625.1	491.0	1200.0	-1625.1	491.0	1400.0	-1625.1
491.0	1600.0	-1625.1	491.0	1800.0	-1625.1	491.0	2000.0	-1625.1
491.0	2200.0	-1625.1	491.0	2400.0	-1625.1	491.0	2600.0	-1625.1
491.0	2800.0	-1625.1	491.0	3000.0	-1625.1	491.0	3200.0	-1625.1
491.0	3400.0	-1625.1	491.0	3600.0	-1625.1	491.0	3800.0	-1625.1
491.0	4000.0	-1625.1	491.0	4200.0	-1625.1	491.0	4400.0	-1625.1
491.0	4600.0	-1625.1	491.0	4800.0	-1625.1	491.0	5000.0	-1625.1
491.0	-1215.6	-4310.2	491.0	-1215.6	-4110.6	491.0	-1215.6	-3910.9
491.0	-1215.6	-3711.3	491.0	-1215.6	-3511.7	491.0	-1215.6	-3312.0
491.0	-1215.6	-3112.4	491.0	-1215.6	-2912.8	491.0	-1215.6	-2713.1
491.0	-1215.6	-2513.5	491.0	-1215.6	-2313.9	491.0	-1215.6	-2114.3
491.0	-1215.6	-1914.6	491.0	-1215.6	-1715.0	491.0	-1040.6	-4310.2
491.0	-1040.6	-4110.6	491.0	-1040.6	-3911.0	491.0	-1040.6	-3711.4
491.0	-1040.6	-3511.8	491.0	-1040.6	-3312.2	491.0	-1040.6	-3112.6

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	-1040.6	-2913.0	491.0	-1040.6	-2713.4	491.0	-1040.6	-2513.8
491.0	-1040.6	-2314.2	491.0	-1040.6	-2114.5	491.0	-1040.6	-1914.9
491.0	-1040.6	-1715.3	491.0	-1000.0	-4400.1	491.0	-813.6	-4400.1
491.0	-627.2	-4400.1	491.0	-440.8	-4400.1	491.0	-254.4	-4400.1
491.0	-1000.0	-4575.1	491.0	-831.1	-4575.1	491.0	-662.2	-4575.1
491.0	-493.3	-4575.1	491.0	-324.4	-4575.1	491.0	4251.3	-4400.1
491.0	4438.5	-4400.1	491.0	4625.6	-4400.1	491.0	4812.8	-4400.1
491.0	5000.0	-4400.1	491.0	4347.5	-4575.1	491.0	4510.7	-4575.1
491.0	4673.8	-4575.1	491.0	4836.9	-4575.1	491.0	5000.0	-4575.1
491.0	5034.4	-4320.1	491.0	5034.4	-4133.5	491.0	5034.4	-3947.0
491.0	5034.4	-3760.4	491.0	5034.4	-3573.8	491.0	5034.4	-3387.3
491.0	5034.4	-3200.7	491.0	5034.4	-3014.1	491.0	5034.4	-2827.6
491.0	5034.4	-2641.0	491.0	5034.4	-2454.4	491.0	5034.4	-2267.9
491.0	5034.4	-2081.3	491.0	5034.4	-1894.7	491.0	5034.4	-1708.2
491.0	5209.4	-4329.8	491.0	5209.4	-4142.6	491.0	5209.4	-3955.3
491.0	5209.4	-3768.1	491.0	5209.4	-3580.8	491.0	5209.4	-3393.5
491.0	5209.4	-3206.3	491.0	5209.4	-3019.0	491.0	5209.4	-2831.7
491.0	5209.4	-2644.5	491.0	5209.4	-2457.2	491.0	5209.4	-2269.9
491.0	5209.4	-2082.7	491.0	5209.4	-1895.4	491.0	5209.4	-1708.2

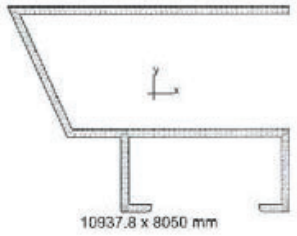
5. Factored Loads and Moments with Corresponding Capacity Ratios

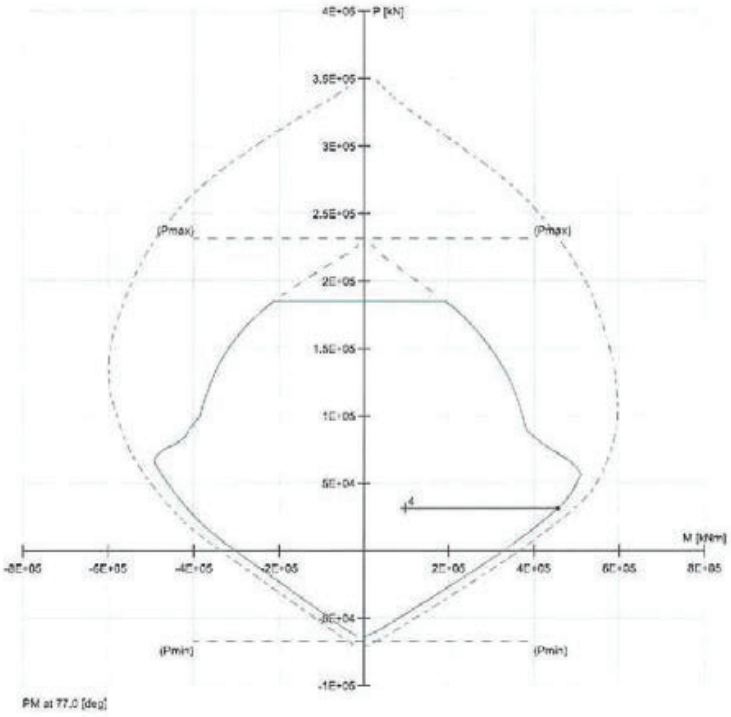
NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kN	kNm	kNm	kN	kNm	kNm	mm			
1	10740.00	28330.00	5430.00	10740.00	255018.59	48879.31	2747	0.00704	0.900	0.11
2	36130.00	540.00	2130.00	36130.00	115971.64	457443.94	3049	0.00838	0.900	0.00
3	20190.00	48320.00	12380.00	20190.00	279198.66	71533.35	2978	0.00615	0.900	0.17
4	31560.00	21590.00	95530.00	31560.00	100618.63	445210.78	2509	0.01003	0.900	0.21

6. Diagrams

6.1. PM at θ=77 [deg]





General Information	
Project	Zone C DD
Column	LW No.4
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

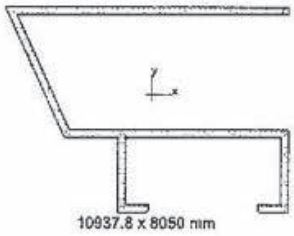
Section	
Type	Irregular
A_g	9.63673e+006 mm²
I_x	7.38774e+013 mm⁴
I_y	1.09165e+014 mm⁴

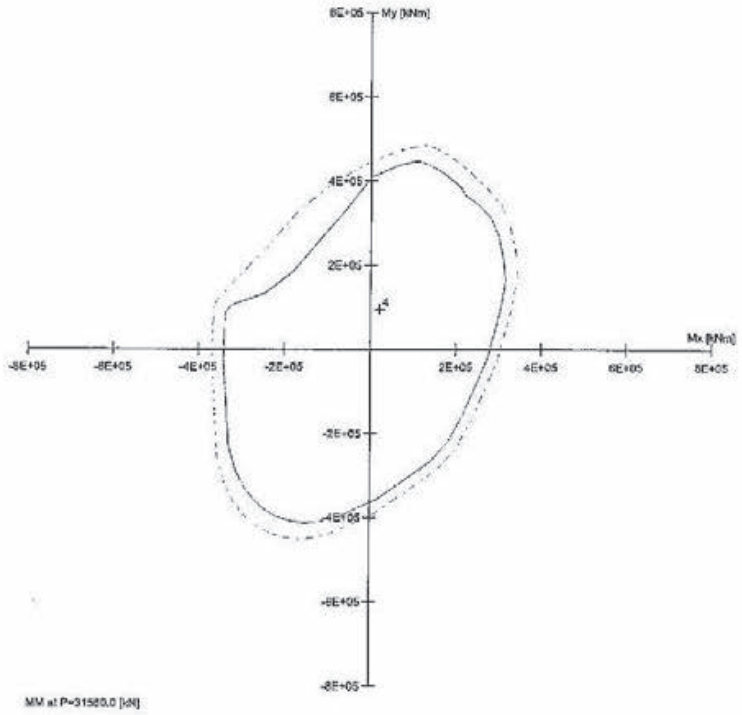
Reinforcement	
Pattern	Irregular
Bar layout	—
Cover to	—
Clear cover	—
Bars	—
Confinement type	Tied
Total steel area, A_s	187071 mm²
Rho	1.94 %
Min. clear spacing	18 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	31560.0	21590.0	95530.0	31560.00	100618.63	445210.78	0.21

Max. Capacity Ratio: 0.21

6.2. MM at P=31560 [kN]





General Information	
Project	Zone C DD
Column	LW No.4
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	9.63673e+006 mm²
I_x	7.38774e+013 mm⁴
I_y	1.09165e+014 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	187071 mm²
Rho	1.94 %
Min. clear spacing	18 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
4	31560.0	21590.0	95530.0	31560.00	100518.63	445210.78	0.21

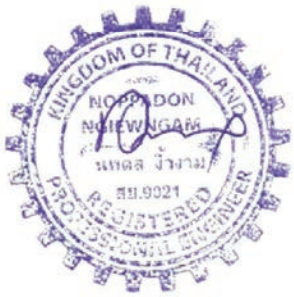
Max. Capacity Ratio: 0.21

TABLE: Pier Forces W8 (2nd -3rd Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
3 SFL	AComb33-1	-1015	93	362	212	3753	1199
2 SFL	AComb2-1	-3341	23	34	171	112	294
2 SFL	AComb27-1	-2455	389	71	459	1146	6595
2 SFL	AComb30-1	-2242	116	444	279	4973	1555
2 SFL	AComb28-1	-2724	357	15	742	1234	6265
2 SFL	AComb34-1	-1925	97	406	68	6788	1680
2 SFL	AComb27-1	-2576	389	71	459	1415	8074

TABLE: Pier Forces W8 (4th -Roof Floor)

Story	Load Case/Combo	P	V2	V3	T	M2	M3
		tonf	tonf	tonf	tonfm	tonfm	tonfm
R	AComb31-1	-60	14	83	128	155	49
4 SFL	AComb2-1	-2599	2	25	67	340	258
4 SFL	AComb32-1	-1217	331	53	564	908	3553
4 SFL	AComb30-1	-1716	96	370	227	2604	1000
4 SFL	AComb28-1	-2054	325	44	568	701	3426
4 SFL	AComb30-1	-1871	96	370	227	4419	1471
4 SFL	AComb27-1	-2011	324	89	442	1441	5398



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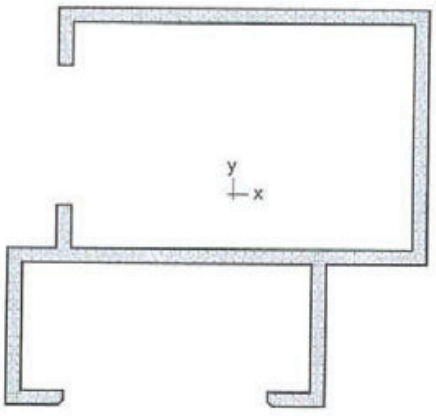

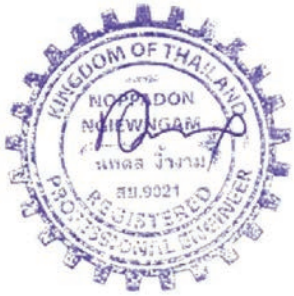

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Figure 1: Column section 4

1. General Information

File Name	C:\193018\GCDT\SPcloumn\10-11-63(C2...\W8 2-3.col
Project	Zone C DD
Column	LW No.5
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_0	0.003	mm/mm
β_1	0.796192	

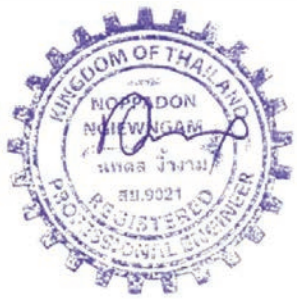
2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	9.09511e+006	mm²
I_x	6.87236e+013	mm⁴
I_y	7.92168e+013	mm⁴
r_x	2748.84	mm
r_y	2951.24	mm
X_o	0.000450352	mm
Y_o	0.000663753	mm



3.2. Section Figure

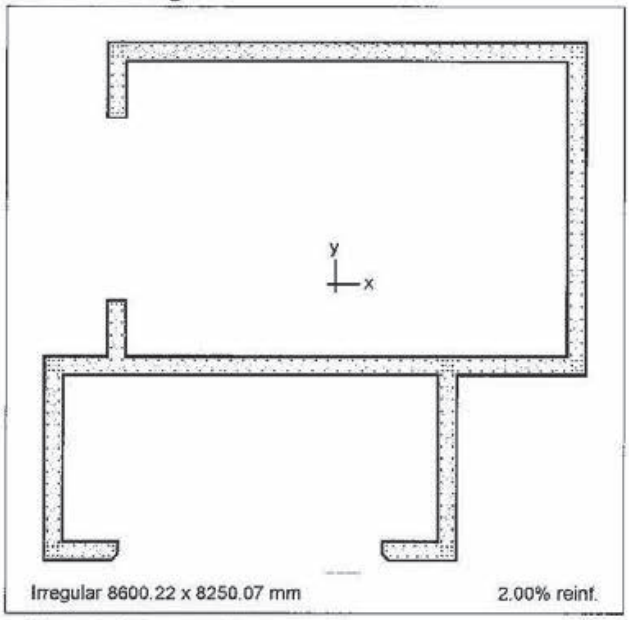


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	3975.8	3843.8	2	-3624.2	3843.8	3	-3624.2	2643.8
4	-3324.2	2643.8	5	-3324.2	3543.8	6	3675.8	3543.8
7	3676.0	-1156.3	8	-3324.2	-1156.3	9	-3324.2	-256.2
10	-3624.2	-256.2	11	-3624.2	-1156.3	12	-4624.2	-1156.3
13	-4624.2	-4406.3	14	-3549.2	-4406.3	15	-3449.2	-4306.3
16	-3449.2	-4106.3	17	-4324.2	-4106.3	18	-4324.2	-1456.3
19	1625.8	-1456.3	20	1625.8	-4106.3	21	750.8	-4106.3
22	750.8	-4306.3	23	850.8	-4406.3	24	1925.8	-4406.3
25	1925.8	-1456.3	26	3976.0	-1456.3			

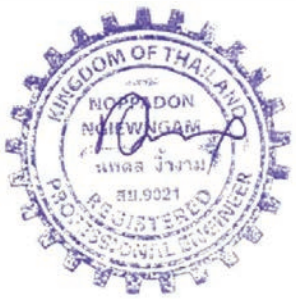
4. Reinforcement

4.1. Bar Set: User-defined

Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#20	20.00	314.16						

4.2. Confinement and Factors

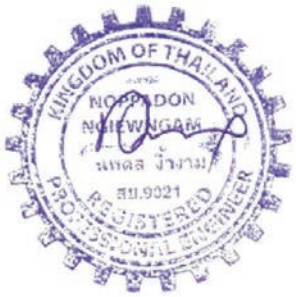
Confinement type	Tied
For #10 bars or less	#20 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.6
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65



4.3. Arrangement

Pattern	Irregular

Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Total steel area, A_s	182161 mm²
Rho	2.00 %
Minimum clear spacing	32 mm



4.4. Bars Provided

Area	X	Y	Area	X	Y	Area	X	Y
mm²	mm	mm	mm²	mm	mm	mm²	mm	mm
491.0	-3519.2	-4176.3	491.0	-3584.9	-4336.1	491.0	-3519.2	-4256.1
491.0	-4554.2	-4335.8	491.0	-4474.2	-4335.9	491.0	-4394.2	-4335.8
491.0	-4554.2	-4260.9	491.0	-4394.2	-4260.9	491.0	-4394.2	-4185.9
491.0	-4474.2	-4185.9	491.0	-4554.2	-4185.9	491.0	-4394.2	-1226.3
491.0	-4554.2	-1226.3	491.0	-4394.2	-1301.7	491.0	-4554.2	-1301.7
491.0	-4394.2	-1376.7	491.0	-4554.2	-1376.7	491.0	-4474.2	-1226.3
491.0	-4474.2	-1376.7	491.0	820.8	-4176.3	491.0	886.5	-4336.1
491.0	820.8	-4256.1	491.0	1855.8	-4335.8	491.0	1775.8	-4335.9
491.0	1695.8	-4335.8	491.0	1855.8	-4260.9	491.0	1695.8	-4260.9
491.0	1695.8	-4185.9	491.0	1775.8	-4185.9	491.0	1855.8	-4185.9
491.0	1695.8	-1226.3	491.0	1855.8	-1226.3	491.0	1695.8	-1301.7
491.0	1855.8	-1301.7	491.0	1695.8	-1376.7	491.0	1855.8	-1376.7
491.0	1775.8	-1226.3	491.0	1775.8	-1376.7	491.0	3746.0	-1226.3
491.0	3906.0	-1226.3	491.0	3746.0	-1301.7	491.0	3906.0	-1301.7
491.0	3746.0	-1376.7	491.0	3906.0	-1376.7	491.0	3826.0	-1226.3
491.0	3826.0	-1376.7	491.0	-3394.2	-326.2	491.0	-3554.2	-326.2
491.0	-3474.2	-326.1	491.0	-3394.2	2713.8	491.0	-3554.2	2713.8
491.0	-3474.2	2713.7	491.0	-3394.4	3773.8	491.0	-3474.4	3773.8
491.0	-3554.4	3773.8	491.0	-3554.5	3693.8	491.0	-3394.3	3693.8
491.0	-3394.2	3613.8	491.0	-3474.2	3613.8	491.0	-3554.2	3613.8
491.0	3746.0	3773.8	491.0	3826.0	3773.8	491.0	3906.0	3773.8
491.0	3906.1	3693.8	491.0	3745.9	3693.8	491.0	3745.8	3613.8
491.0	3825.8	3613.8	491.0	3905.8	3613.8	491.0	-3561.7	3500.0
491.0	-3561.7	3321.6	491.0	-3561.7	3143.1	491.0	-3561.7	2964.7
491.0	-3561.7	2786.3	491.0	-3386.7	3500.0	491.0	-3386.7	3321.6
491.0	-3386.7	3143.1	491.0	-3386.7	2964.7	491.0	-3386.7	2786.3
491.0	-3324.1	3781.3	491.0	-3125.5	3781.3	491.0	-2927.0	3781.3
491.0	-2728.4	3781.3	491.0	-2529.9	3781.3	491.0	-2331.4	3781.3
491.0	-2132.8	3781.3	491.0	-1934.3	3781.3	491.0	-1735.7	3781.3
491.0	-1537.2	3781.3	491.0	-1338.7	3781.3	491.0	-1140.1	3781.3
491.0	-941.6	3781.3	491.0	-743.0	3781.3	491.0	-544.5	3781.3
491.0	-346.0	3781.3	491.0	-147.4	3781.3	491.0	51.1	3781.3
491.0	249.7	3781.3	491.0	448.2	3781.3	491.0	646.7	3781.3
491.0	845.3	3781.3	491.0	1043.8	3781.3	491.0	1242.4	3781.3
491.0	1440.9	3781.3	491.0	1639.4	3781.3	491.0	1838.0	3781.3
491.0	2036.5	3781.3	491.0	2235.1	3781.3	491.0	2433.6	3781.3
491.0	2632.1	3781.3	491.0	2830.7	3781.3	491.0	3029.2	3781.3
491.0	3227.8	3781.3	491.0	3426.3	3781.3	491.0	3624.8	3781.3
491.0	-3307.2	3606.3	491.0	-3109.4	3606.3	491.0	-2911.6	3606.3
491.0	-2713.8	3606.3	491.0	-2516.0	3606.3	491.0	-2318.2	3606.3
491.0	-2120.4	3606.3	491.0	-1922.5	3606.3	491.0	-1724.7	3606.3
491.0	-1526.9	3606.3	491.0	-1329.1	3606.3	491.0	-1131.3	3606.3
491.0	-933.5	3606.3	491.0	-735.7	3606.3	491.0	-537.9	3606.3
491.0	-340.1	3606.3	491.0	-142.3	3606.3	491.0	55.5	3606.3
491.0	253.3	3606.3	491.0	451.1	3606.3	491.0	648.9	3606.3
491.0	846.7	3606.3	491.0	1044.5	3606.3	491.0	1242.3	3606.3
491.0	1440.1	3606.3	491.0	1637.9	3606.3	491.0	1835.7	3606.3

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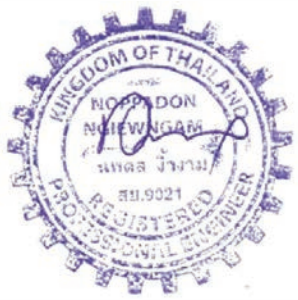
Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	2033.5	3606.3	491.0	2231.3	3606.3	491.0	2429.1	3606.3
491.0	2626.9	3606.3	491.0	2824.7	3606.3	491.0	3022.5	3606.3
491.0	3220.4	3606.3	491.0	3418.2	3606.3	491.0	3616.0	3606.3
491.0	3738.3	3500.0	491.0	3738.3	3307.3	491.0	3738.3	3114.6
491.0	3738.3	2921.9	491.0	3738.3	2729.2	491.0	3738.3	2536.5
491.0	3738.3	2343.8	491.0	3738.3	2151.1	491.0	3738.4	1958.4
491.0	3738.4	1765.7	491.0	3738.4	1573.0	491.0	3738.4	1380.3
491.0	3738.4	1187.6	491.0	3738.4	995.0	491.0	3738.4	802.3
491.0	3738.4	609.6	491.0	3738.4	416.9	491.0	3738.4	224.2
491.0	3738.4	31.5	491.0	3738.5	-161.2	491.0	3738.5	-353.9
491.0	3738.5	-546.6	491.0	3738.5	-739.3	491.0	3738.5	-932.0
491.0	3738.5	-1124.7	491.0	3913.3	3500.0	491.0	3913.3	3307.0
491.0	3913.3	3114.0	491.0	3913.3	2920.9	491.0	3913.3	2727.9
491.0	3913.3	2534.9	491.0	3913.3	2341.9	491.0	3913.3	2148.8
491.0	3913.4	1955.8	491.0	3913.4	1762.8	491.0	3913.4	1569.8
491.0	3913.4	1376.8	491.0	3913.4	1183.7	491.0	3913.4	990.7
491.0	3913.4	797.7	491.0	3913.4	604.7	491.0	3913.4	411.7
491.0	3913.4	218.6	491.0	3913.4	25.6	491.0	3913.4	-167.4
491.0	3913.5	-360.4	491.0	3913.5	-553.5	491.0	3913.5	-746.5
491.0	3913.5	-939.5	491.0	3913.5	-1132.5	491.0	1942.1	-1393.8
491.0	2129.7	-1393.8	491.0	2317.3	-1393.8	491.0	2504.9	-1393.8
491.0	2692.4	-1393.8	491.0	2880.0	-1393.8	491.0	3067.6	-1393.8
491.0	3255.2	-1393.8	491.0	3442.8	-1393.8	491.0	3630.4	-1393.8
491.0	1937.6	-1218.8	491.0	2125.5	-1218.8	491.0	2313.4	-1218.8
491.0	2501.3	-1218.8	491.0	2689.2	-1218.8	491.0	2877.1	-1218.8
491.0	3065.0	-1218.8	491.0	3252.9	-1218.8	491.0	3440.8	-1218.8
491.0	3628.7	-1218.8	491.0	-4325.7	-1218.8	491.0	-4322.1	-1393.8
491.0	-4134.7	-1218.8	491.0	-4124.3	-1393.8	491.0	-3943.7	-1218.8
491.0	-3926.4	-1393.8	491.0	-3752.7	-1218.8	491.0	-3728.6	-1393.8
491.0	-3561.7	-428.5	491.0	-3561.7	-626.1	491.0	-3561.7	-823.6
491.0	-3561.7	-1021.2	491.0	-3561.7	-1218.8	491.0	-3530.8	-1393.8
491.0	-3386.7	-428.5	491.0	-3386.7	-626.1	491.0	-3386.7	-823.6
491.0	-3386.7	-1021.2	491.0	-3386.7	-1218.8	491.0	-3332.9	-1393.8
491.0	-3194.2	-1218.8	491.0	-3135.1	-1393.8	491.0	-3001.7	-1218.8
491.0	-2937.2	-1393.8	491.0	-2809.2	-1218.8	491.0	-2739.4	-1393.8
491.0	-2616.7	-1218.8	491.0	-2541.5	-1393.8	491.0	-2424.2	-1218.8
491.0	-2343.7	-1393.8	491.0	-2231.7	-1218.8	491.0	-2145.8	-1393.8
491.0	-2039.2	-1218.8	491.0	-1948.0	-1393.8	491.0	-1846.7	-1218.8
491.0	-1750.1	-1393.8	491.0	-1654.2	-1218.8	491.0	-1552.3	-1393.8
491.0	-1461.7	-1218.8	491.0	-1354.4	-1393.8	491.0	-1269.2	-1218.8
491.0	-1156.6	-1393.8	491.0	-1076.7	-1218.8	491.0	-958.7	-1393.8
491.0	-864.2	-1218.8	491.0	-760.9	-1393.8	491.0	-691.7	-1218.8
491.0	-563.0	-1393.8	491.0	-499.2	-1218.8	491.0	-365.2	-1393.8
491.0	-306.7	-1218.8	491.0	-167.3	-1393.8	491.0	-114.2	-1218.8
491.0	30.6	-1393.8	491.0	78.3	-1218.8	491.0	228.3	-1393.8
491.0	270.8	-1218.8	491.0	426.2	-1393.8	491.0	463.3	-1218.8
491.0	624.0	-1393.8	491.0	655.8	-1218.8	491.0	821.9	-1393.8
491.0	848.3	-1218.8	491.0	1019.7	-1393.8	491.0	1040.8	-1218.8
491.0	1217.6	-1393.8	491.0	1233.3	-1218.8	491.0	1415.4	-1393.8
491.0	1425.8	-1218.8	491.0	1613.3	-1393.8	491.0	1618.3	-1218.8
491.0	1688.3	-4100.4	491.0	1688.3	-3911.6	491.0	1688.3	-3722.8
491.0	1688.3	-3533.9	491.0	1688.3	-3345.1	491.0	1688.3	-3156.3
491.0	1688.3	-2967.4	491.0	1688.3	-2778.6	491.0	1688.3	-2589.8
491.0	1688.3	-2400.9	491.0	1688.3	-2212.1	491.0	1688.3	-2023.3
491.0	1688.3	-1834.4	491.0	1688.3	-1645.6	491.0	1688.3	-1456.8
491.0	1863.3	-4097.1	491.0	1863.3	-3908.7	491.0	1863.3	-3720.4
491.0	1863.3	-3532.0	491.0	1863.3	-3343.6	491.0	1863.3	-3155.2
491.0	1863.3	-2966.8	491.0	1863.3	-2778.4	491.0	1863.3	-2590.0
491.0	1863.3	-2401.6	491.0	1863.3	-2213.2	491.0	1863.3	-2024.8

Area	X	Y	Area	X	Y	Area	X	Y
mm²	mm	mm	mm²	mm	mm	mm²	mm	mm
491.0	1863.3	-1836.4	491.0	1863.3	-1648.0	491.0	1863.3	-1459.6
491.0	926.6	-4168.8	491.0	1098.3	-4168.8	491.0	1270.0	-4168.8
491.0	1441.6	-4168.8	491.0	1613.3	-4168.8	491.0	1000.0	-4343.8
491.0	1155.1	-4343.8	491.0	1310.2	-4343.8	491.0	1465.3	-4343.8
491.0	1620.4	-4343.8	491.0	-4561.7	-4129.7	491.0	-4561.7	-3941.9
491.0	-4561.7	-3754.0	491.0	-4561.7	-3566.2	491.0	-4561.7	-3378.4
491.0	-4561.7	-3190.5	491.0	-4561.7	-3002.7	491.0	-4561.7	-2814.9
491.0	-4561.7	-2627.0	491.0	-4561.7	-2439.2	491.0	-4561.7	-2251.3
491.0	-4561.7	-2063.5	491.0	-4561.7	-1875.7	491.0	-4561.7	-1687.8
491.0	-4561.7	-1500.0	491.0	-4386.7	-4129.3	491.0	-4386.7	-3941.5
491.0	-4386.7	-3753.7	491.0	-4386.7	-3565.9	491.0	-4386.7	-3378.1
491.0	-4386.7	-3190.3	491.0	-4386.7	-3002.5	491.0	-4386.7	-2814.7
491.0	-4386.7	-2626.9	491.0	-4386.7	-2439.1	491.0	-4386.7	-2251.2
491.0	-4386.7	-2063.4	491.0	-4386.7	-1875.6	491.0	-4386.7	-1687.8
491.0	-4386.7	-1500.0	491.0	-4333.9	-4343.8	491.0	-4167.4	-4343.8
491.0	-4001.0	-4343.8	491.0	-3834.5	-4343.8	491.0	-3668.1	-4343.8
491.0	-4320.6	-4168.8	491.0	-4138.4	-4168.8	491.0	-3956.3	-4168.8
491.0	-3774.1	-4168.8	491.0	-3591.9	-4168.8			

5. Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.

No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u	M_{ux}	M_{uy}	ϕP_n	ϕM_{nx}	ϕM_{ny}	NA Depth	ε_t	ϕ	
	kN	kNm	kNm	kN	kNm	kNm	mm			
1	10150.00	37530.00	11990.00	10150.00	213257.48	68131.01	2904	0.00693	0.900	0.18
2	33410.00	1120.00	2940.00	33410.00	110544.22	290178.69	1996	0.01054	0.900	0.01
3	19250.00	67880.00	16800.00	19250.00	236415.95	58511.92	3087	0.00629	0.900	0.29
4	25760.00	14150.00	80740.00	25760.00	51189.32	292086.78	1873	0.01114	0.900	0.28



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C:\193018\GCDT\SPcloumn\10-11-63(C2 building)\Shear wall\W8 2-3.col

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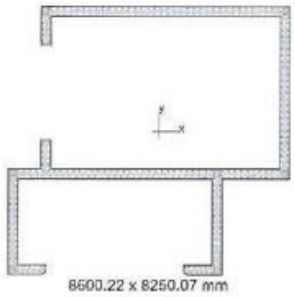
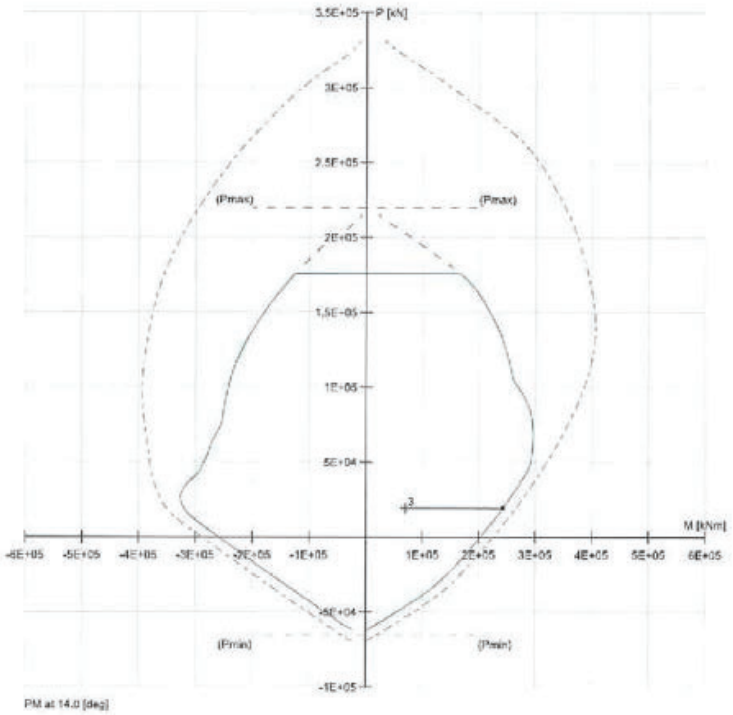
11/26/2020
3.35 PM

6. Diagrams

6.1. PM at θ=14 [deg]





PM at 14.0 [deg]

General Information	
Project	Zone C DD
Column	LW No.5
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

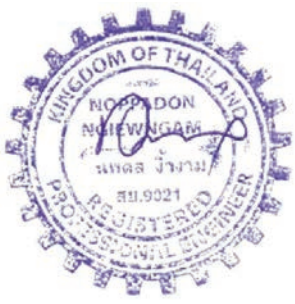
Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	9.09511e+006 mm²
I_x	6.87236e+013 mm⁴
I_y	7.92168e+013 mm⁴

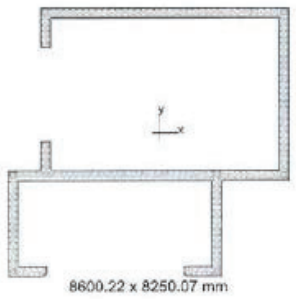
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	182161 mm²
Rho	2.00 %
Min. clear spacing	32 mm

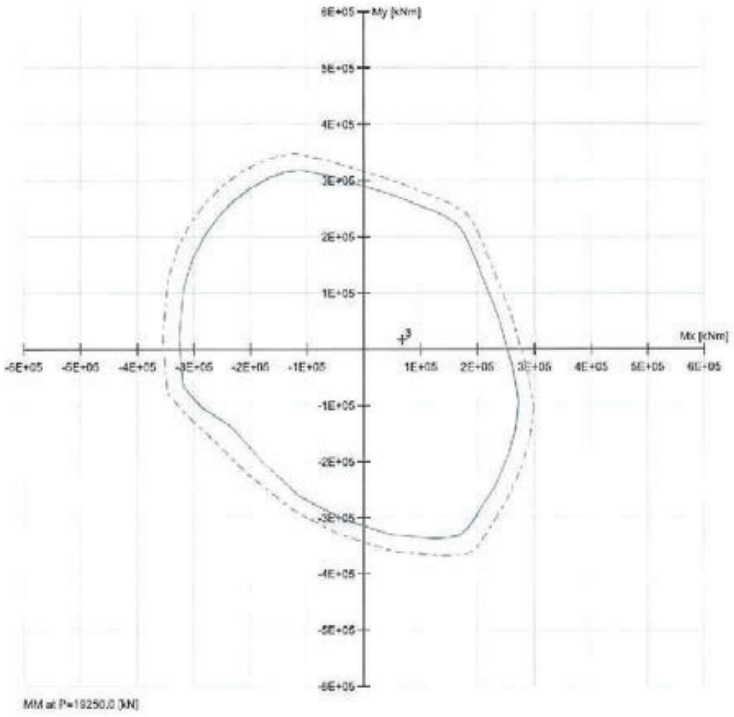
No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
3	19250.0	67880.0	16800.0	19250.00	236415.95	58511.92	0.29

Max. Capacity Ratio: 0.29



6.2. MM at P=19250 [kN]





General Information	
Project	Zone C DD
Column	LW No.5
Engineer	PTR
Code	ACI 318-14
Bar Set	User-defined
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Moment capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section	
Type	Irregular
A_g	9.09511e+006 mm²
I_x	6.87236e+013 mm⁴
I_y	7.92168e+013 mm⁴

Reinforcement	
Pattern	Irregular
Bar layout	—
Cover to	—
Clear cover	—
Bars	—
Confinement type	Tied
Total steel area, A_s	182161 mm²
Rho	2.00 %
Min. clear spacing	32 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
3	19250.0	67880.0	16800.0	19250.00	236415.95	58511.92	0.29

Max. Capacity Ratio: 0.29

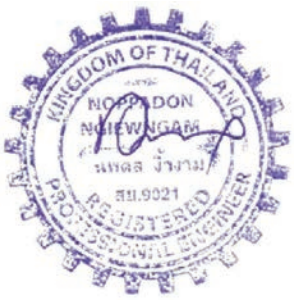


TABLE: Column Forces C58A (2nd to 3rd Floor)

Story	Column	Load Case/Combo	P	V2	V3	M2	M3
			tonf	tonf	tonf	tonfm	tonfm
3 SFL	C58	Area Method	-100	0	0	0	0
3 SFL	C58	Area Method	-40	0	0	0	0
3 SFL	C58	Area Method	-30	0	0	0	0
3 SFL	C58	Area Method	-70	0	0	0	0

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Computer program for the Strength Design of Reinforced Concrete Sections
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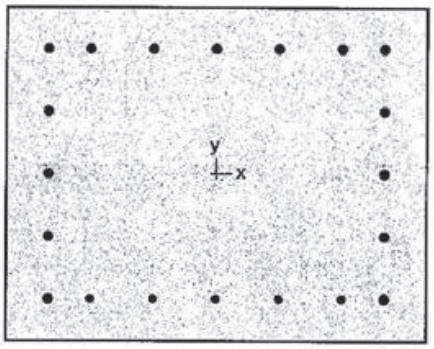

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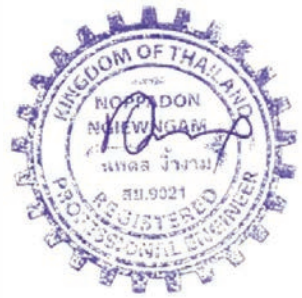
List of Figures

Figure 1: Column section 4

1. General Information

File Name	D:\004-My Works\A001-For DEC Off...\c58A-roof.col
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity



2. Material Properties

2.1. Concrete

Type	Standard	
f'_c	35	MPa
E_c	27805.6	MPa
f_c	29.75	MPa
ε_u	0.003	mm/mm
β_1	0.796192	

2.2. Steel

Type	Standard	
f_y	400	MPa
E_s	200000	MPa
ε_{yt}	0.002	mm/mm

3. Section

3.1. Shape and Properties

Type	Irregular	
A_g	800000	mm²
I_x	4.26667e+010	mm⁴
I_y	6.66667e+010	mm⁴
r_x	230.94	mm
r_y	288.675	mm
X_o	0	mm
Y_o	0	mm

3.2. Section Figure

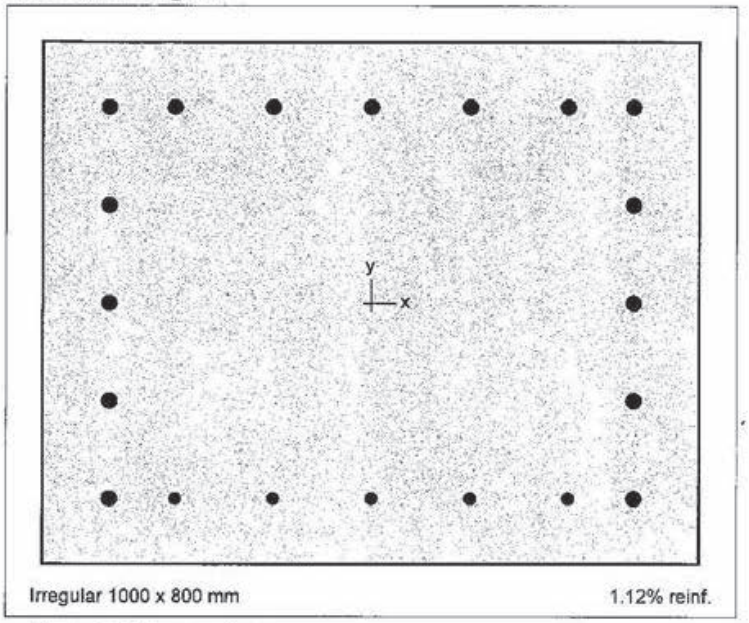


Figure 1: Column section

3.3. Exterior Points

Points	X	Y	Points	X	Y	Points	X	Y
	mm	mm		mm	mm		mm	mm
1	-500.0	-400.0	2	500.0	-400.0	3	500.0	400.0
4	-500.0	400.0						

4. Reinforcement

4.1. Bar Set: ASTM A615

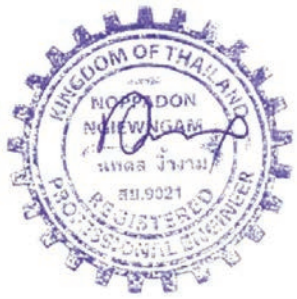
Bar	Diameter	Area	Bar	Diameter	Area	Bar	Diameter	Area
	mm	mm²		mm	mm²		mm	mm²
#3	9.53	70.97	#4	12.70	129.03	#5	15.88	200.00
#6	19.05	283.87	#7	22.23	387.10	#8	25.40	509.68
#9	28.65	645.16	#10	32.26	819.35	#11	35.81	1006.45
#14	43.00	1451.61	#18	57.33	2580.64			

4.2. Confinement and Factors

Confinement type	Tied
For #10 bars or less	#3 ties
For larger bars	#4 ties
Capacity Reduction Factors	
Axial compression, (a)	0.8
Tension controlled ϕ, (b)	0.9
Compression controlled ϕ, (c)	0.65

4.3. Arrangement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---



Bars	---
Total steel area, A_s	8935 mm²
Rho	1.12 %
Minimum clear spacing	75 mm

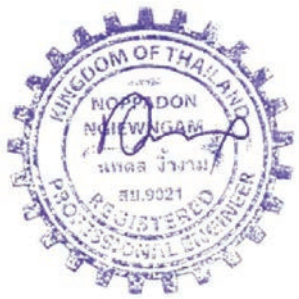
4.4. Bars Provided

Area mm²	X mm	Y mm	Area mm²	X mm	Y mm	Area mm²	X mm	Y mm
491.0	-400.0	-300.0	491.0	-400.0	-150.0	491.0	-400.0	0.0
491.0	-400.0	150.0	491.0	-400.0	300.0	491.0	400.0	-300.0
491.0	400.0	-150.0	491.0	400.0	0.0	491.0	400.0	150.0
491.0	400.0	300.0	491.0	-300.0	300.0	491.0	-150.0	300.0
491.0	0.0	300.0	491.0	150.0	300.0	491.0	300.0	300.0
314.0	-300.0	-300.0	314.0	-150.0	-300.0	314.0	0.0	-300.0
314.0	150.0	-300.0	314.0	300.0	-300.0			

5. Factored Loads and Moments with Corresponding Capacity Ratios

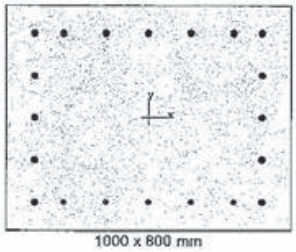
NOTE: Calculations are based on "Critical Capacity" Method.

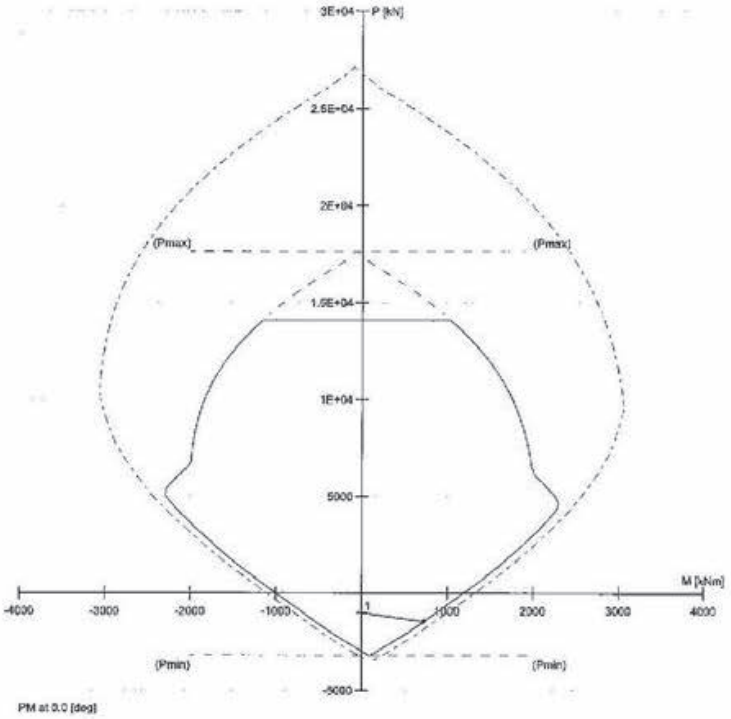
No.	Demand			Capacity			Parameters at Capacity			Capacity Ratio
	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	NA Depth mm	ε_t	ϕ	
1	-1000.00	0.00	0.00	-1455.07	730.57	0.00	69	0.02751	0.900	0.65



6. Diagrams

6.1. PM at θ=0 [deg]





General Information	
Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials	
f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

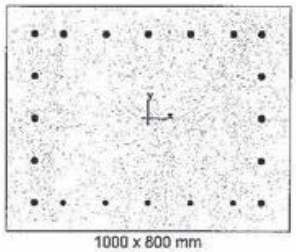
Section	
Type	Irregular
A_g	800000 mm²
I_x	4.26667e+010 mm⁴
I_y	6.66667e+010 mm⁴

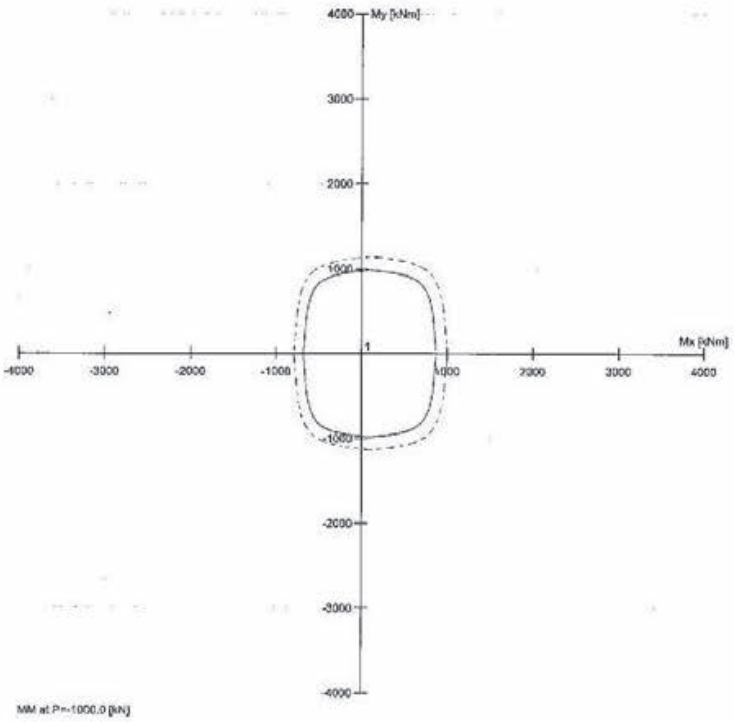
Reinforcement	
Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_{st}	8935 mm²
Rho	1.12 %
Min. clear spacing	75 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
1	-1000.0	0.0	0.0	-1455.07	730.57	0.00	0.65

Max. Capacity Ratio: 0.65

6.2. MM at P=-1000 [kN]





General Information

Project	---
Column	---
Engineer	---
Code	ACI 318-14
Bar Set	ASTM A615
Units	Metric
Run Option	Investigation
Run Axis	Biaxial
Slenderness	Not Considered
Column Type	Structural
Capacity Method	Critical capacity

Materials

f'_c	35 MPa
E_c	27805.6 MPa
f_y	400 MPa
E_s	200000 MPa

Section

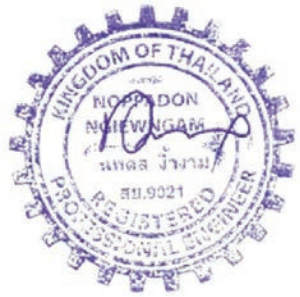
Type	Irregular
A_g	800000 mm²
I_x	4.26667e+010 mm⁴
I_y	6.66667e+010 mm⁴

Reinforcement

Pattern	Irregular
Bar layout	---
Cover to	---
Clear cover	---
Bars	---
Confinement type	Tied
Total steel area, A_s	8935 mm²
Rho	1.12 %
Min. clear spacing	75 mm

No.	P_u kN	M_{ux} kNm	M_{uy} kNm	ϕP_n kN	ϕM_{nx} kNm	ϕM_{ny} kNm	Capacity Ratio
1	-1000.0	0.0	0.0	-1455.07	730.57	0.00	0.65

Max. Capacity Ratio: 0.65



3. รายการคำนวณพื้นคอนกรีตอัดแรงภายหลัง

```
-------------------------------------------------------------------------------
|                            ADAPT CORPORATION                                |
|                     STRUCTURAL CONCRETE SOFTWARE SYSTEM                     |
|          1733 Woodside Road, Suite 220, Redwood City, California 94061      |
-------------------------------------------------------------------------------
|                  ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN               |
|                  Version 7.10  AMERICAN (ACI 318-99/UBC-77)                 |
|              ADAPT CORPORATION - Structural Concrete Software System        |
|          1733 Woodside Road, Suite 220, Redwood City, California 94061      |
|                    Phone: (650)306-2400,  Fax: (650)364-4678                |
|        Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com     |
-------------------------------------------------------------------------------
DATE AND TIME OF PROGRAM EXECUTION:                Mar 8,2021   At Time: 11:31
PROJECT FILE:                                              FL.3 GL. DJ (D4-D7)

P R O J E C T   T I T L E:
GCDT
FL.2 GL. D6 (DH-DL)

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
===============================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............    350.00 Kg/cm^2
                     for COLUMNS .................    350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............    275.00 T/cm^2
                      for COLUMNS .................    238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....      2.00
CONCRETE WEIGHT ..................................     NORMAL

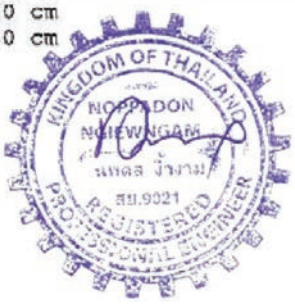
SELF WEIGHT ......................................   2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top  ..........................................      1.590
At Bottom  .......................................      1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................       .450

REINFORCEMENT:
YIELD Strength ...................................      4.00 T/cm^2
Minimum Cover at TOP .............................      2.00 cm
Minimum Cover at BOTTOM ..........................      2.00 cm

POST-TENSIONING:
SYSTEM ...........................................      BONDED
Ultimate strength of strand ......................     19.00 T/cm^2
Average effective stress in strand (final) .......     10.90 T/cm^2
Strand area.......................................      .990 cm^2
Min CGS of tendon from TOP........................      4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..     5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..     5.60 cm
```



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Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ..(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

SPAN	FORM	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
C	1	3.50	900.00	25.00					.00	.50	.50
1	1	9.00	900.00	25.00					.00	.50	.50
C	1	3.50	900.00	25.00					.00	.50	.50

LEGEND:
1 - SPAN
C = Cantilever

3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 - Top surface to reference line

2.1.5 - D R O P C A P A N D D R O P P A N E L D A T A

JOINT	CAPT cm	CAPB cm	CAPDL cm	CAPDR cm	DROPTL cm	DROPTR cm	DROPB cm	DROPL cm	DROPR cm
1	.00	.00	.00	.00	55.00	55.00	300.00	150.00	450.00
2	.00	.00	.00	.00	55.00	55.00	300.00	450.00	150.00

LEGEND:
DROP CAP DIMENSIONS:
CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint

DROP PANEL DIMENSIONS:
DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint

2.2 - S U P P O R T W I D T H A N D C O L U M N D A T A

JOINT	SUPPORT WIDTH cm	<------ LOWER COLUMN ------> LENGTH m	B(DIA) cm	D cm	CBC*	<------ UPPER COLUMN ------> LENGTH m	B(DIA) cm	D cm	CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

3 - I N P U T A P P L I E D L O A D I N G

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<----------TYPE---------->
U = UNIFORM P = PARTIAL UNIFORM
C = CONCENTRATED M = APPLIED MOMENT
Li= LINE LOAD

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	(From ... m	To) m	(M or C ...At) T-m or T ...m)		Total on Trib T/m
CANT	L	U	.500	.00	1.50			4.500
CANT	L	U	.500	1.50	3.50			4.500
CANT	D	U	.300	.00	1.50			2.700
CANT	D	U	.300	1.50	3.50			2.700
CANT	SW	P		.00	1.50			7.560
CANT	SW	P		1.50	3.50			5.400
1	L	U	.500	.00	4.50			4.500
1	L	U	.500	4.50	9.00			4.500
1	D	U	.300	.00	4.50			2.700
1	D	U	.300	4.50	9.00			2.700
1	D	C				100.00	4.50	
1	SW	P		.00	4.50			7.560
1	SW	P		4.50	9.00			7.560

```
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-------------------------------------------------------------------------------

CANT    L       U          .500       .00      1.50                        4.500
CANT    L       U          .500      1.50      3.50                        4.500
CANT    D       U          .300       .00      1.50                        2.700
CANT    D       U          .300      1.50      3.50                        2.700
CANT    SW      P                     .00      1.50                        7.560
CANT    SW      P                    1.50      3.50                        5.400

 NOTE: LIVE LOADING is SKIPPED with a skip factor of      .75


3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING
===============================================================================

                    UNIFORM
                    ( T/m^2),     ( CON. or PART. )     ( M O M E N T )
SPAN  CLASS  TYPE   LINE( T/m)    (  T@m or  m-m  )     (  T-m  @  m  )
-1----2------3---------4-----------5-------6-----------7-------8-----------
CANT    L       U          .500
CANT    D       U          .300
 1      L       U          .500
 1      D       U          .300
 1      D       C                     100.00    4.50
CANT    L       U          .500
CANT    D       U          .300
 NOTE: SELFWEIGHT INCLUSION REQUIRED
       LIVE LOADING is SKIPPED with a skip factor of      .75


4 - C A L C U L A T E D   S E C T I O N   P R O P E R T I E S
===============================================================================

4.2 - Computed Section Properties for Segments of Nonprismatic Spans
-------------------------------------------------------------------------------
Section properties are listed for all segments of each span
A= cross-sectional geometry       Yt= centroidal distance to top fiber
I= gross moment of inertia        Yb= centroidal distance to bottom fiber

SPAN          AREA              I               Yb              Yt
(SEGMENT)     cm^2              cm^4            cm              cm
--------------2----------------3---------------4-------------5-----
CANTILEVER LEFT
  1          31500.00         .6708E+07        34.64           20.36
  2          22500.00         .1172E+07        12.50           12.50
SPAN  1
  1          31500.00         .6708E+07        34.64           20.36
  2          31500.00         .6708E+07        34.64           20.36
CANTILEVER RIGHT
  1          31500.00         .6708E+07        34.64           20.36
  2          22500.00         .1172E+07        12.50           12.50
```

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5 - DEAD LOAD MOMENTS, SHEARS & REACTIONS

SPAN	< 5.1 SPAN MOMENTS (Tm) > M(l)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (T) > SH(l)	SH(r)
CANT	-----	-----	-52.04	-----	31.59
1	-145.53	183.35	-145.54	-96.17	96.17
CANT	-52.04	-----	-----	-31.59	-----

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) -> Lower columns	Upper columns
1	127.76	-52.05	-41.43
2	127.76	52.05	41.44

6 - LIVE LOAD MOMENTS, SHEARS & REACTIONS

<-- 6.1 LIVE LOAD SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

SPAN	<----- left* -----> max	min	<--- midspan ---> max	min	<---- right* -----> max	min	<--SHEAR FORCE--> left	right
CL	-----		-----		-27.56		-----	15.75
1	-29.58	-5.77	17.76	-5.77	-29.58	-5.77	-20.25	20.25
CR	-27.56		-----		-----		-15.75	-----

Note:
* = Centerline moments

JOINT	<- 6.2 REACTIONS (T) -> max	min	<-------- 6.3 COLUMN MOMENTS (Tm) --------> <--- LOWER COLUMN ---> max	min	<--- UPPER COLUMN ---> max	min
1	37.65	15.75	11.06	-12.18	8.81	-9.70
2	37.65	15.75	12.19	-11.06	9.70	-8.81

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

7 - MOMENTS REDUCED TO FACE-OF-SUPPORT

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7.1 R E D U C E D DEAD LOAD MOMENTS (Tm)

SPAN	<- left* ->	<- midspan ->	<- right* ->
CANT	-----	-----	-38.06
1	-100.50	183.40	-100.60
CANT	-38.06	-----	-----

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
CL	-----		-----		-20.52	
1	-20.37	-5.77	17.76	-5.77	-20.38	-5.77
CR	-20.52		-----		-----	

Note:
* = face-of-support

8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
CL	-----		-----		-58.58	
1	-120.87	-106.27	201.16	177.63	-120.98	-106.37
CR	-58.58		-----		-----	

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

9.1 PROFILE TYPES AND PARAMETERS
LEGEND:
For Span:
1 = reversed parabola

2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 TENDON PROFILE

	TYPE	X1/L	X2/L	X3/L	A/L
CANT	1			.000	
1	1	.100	.500	.100	.000
CANT	1	.100			

9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Tendon editing mode selected: TENDON SELECTION

	<-------- SELECTED VALUES -------->				<--- CALCULATED VALUES --->		
SPAN	FORCE (T/-)	<- DISTANCE OF CGS (cm) -> Left	Center	Right	P/A (Kg/cm^2)	Wbal (T/-)	Wbal (%DL)
CANT	500.319	-12.50		-4.20	22.61	6.894	76
1	540.210	-4.20	-49.40	-4.20	17.15	24.116	113
CANT	499.577	-4.20		-12.50	22.71	6.923	77

Approximate weight of strand ..... 555.0 Kg

9.35 - TENDON SELECTION DATA:

TYPE	SEL.	FORCE (T)	<---------- TENDON EXTENTS ----------> <C>	<1>	<C>
B	21	12.32	<==========		
C	21	12.30	==========>		

9.5 REQUIRED MINIMUM POST-TENSIONING FORCES (T)

	<- BASED ON STRESS CONDITIONS ->			<- BASED ON MINIMUM P/A ->		
SPAN	LEFT*	CENTER	RIGHT*	LEFT	CENTER	RIGHT
CANT	-----	-----	.00	-----	-----	315.00
1	70.71	518.63	70.91	315.00	315.00	315.00
CANT	.00	-----	-----	315.00	-----	-----

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Note:
* = face-of-support

9.6 S E R V I C E S T R E S S E S (Kg/cm^2) (tension shown positive)

	L E F T *				R I G H T *			
	TOP		BOTTOM		TOP		BOTTOM	
	max-T	max-C	max-T	max-C	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----	-----	-26.00	1.13	-9.47
1	-----	-18.86	-----	-20.15	-----	-18.90	-----	-20.07
CR	-----	-26.07	1.32	-9.28	-----	-----	-----	-----

Note:
* = face-of-support

	C E N T E R			
	TOP		BOTTOM	
	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----
1	-----	-42.89	26.66	-----
CR	-----	-----	-----	-----

9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

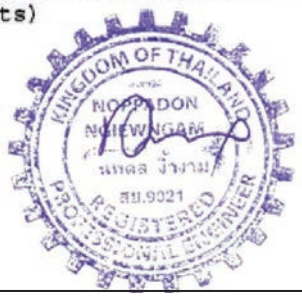
	<-- S P A N M O M E N T S (Tm) -->			<-- SPAN SHEARS (T) -->	
SPAN	left*	midspan	right*	SH(l)	SH(r)
CANT	-----	-----	71.16	-----	18.48
1	113.90	-116.30	114.10	-.03	-.03
CANT	71.47	-----	-----	18.72	-----

Note:
* = face-of-support

	<--REACTIONS (T)-->	<-- COLUMN MOMENTS (Tm) -->	
joint		Lower columns	Upper columns
1	.031	26.820	21.350
2	-.031	-26.690	-21.250

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)



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10.1   FACTORED DESIGN MOMENTS  (Tm)

      <------ left* ------>   <---- midspan  ---->   <----- right* ----->
SPAN       max         min         max         min         max         min
-1---------2----------3----------4----------5----------6----------7-----
CL        -----                    -----                  -88.17
 1      -127.17     -102.35      334.93      294.94      -127.54     -102.70
CR       -88.17                    -----                   -----

Note:
   * = face-of-support


10.2   SECONDARY MOMENTS  (Tm)
SPAN    <-- left* -->   <- midspan  ->   <-- right* -->
-1----------2----------------3----------------4--------
 1          48.16             48.06             47.95

Note:
   * = face-of-support


     10.3 FACTORED REACTIONS      10.4 FACTORED COLUMN MOMENTS  (Tm)
              (T)              <-- LOWER column -->   <-- UPPER column -->
JOINT      max         min         max         min         max         min
-1---------2----------3----------4----------5----------6----------7-----
 1       242.96      205.73      -27.25      -66.76      -21.68      -53.14
 2       242.89      205.66       66.90       27.38       53.26       21.79


11 - M I L D   S T E E L
===============================================================================

Support cut-off length for minimum steel(length/span) ...     .17
Span    cut-off length for minimum steel(length/span) ...     .33
Top     bar extension beyond where required .............  30.00 cm
Bottom  bar extension beyond where required .............  30.00 cm


      REINFORCEMENT based on    NO   REDISTRIBUTION  of factored moments
-------------------------------------------------------------------------------
11.1     TOTAL WEIGHT OF REBAR =         3.4  Kg     AVERAGE =        .0 Kg/m^2
         TOTAL AREA COVERED    =      144.00   m^2


11.2.1     S T E E L    A T    M I D - S P A N
                  T O P                                  B O T T O M
          As    DIFFERENT REBAR CRITERIA        As    DIFFERENT REBAR CRITERIA
 SPAN  (cm^2) <---ULT-----TENS-------->      (cm^2) <---ULT-----TENS-------->
--1------2----------3-------4-------5-----------6----------7-------8-------9----
```

```
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  1       .0  (     .0       .0       .0)        1.3  (    1.3       .0       .0)


11.3.1     S T E E L    A T    S U P P O R T S
                      T O P                                  B O T T O M
           As     DIFFERENT REBAR CRITERIA          As     DIFFERENT REBAR CRITERIA
JOINT    (cm^2) <---ULT-----MIN--------->        (cm^2) <---ULT-----MIN--------->
--1------2---------3-------4-------5-----------6---------7-------8-------9----
  1       .0  (     .0       .0       .0)         .0  (     .0       .0       .0)
  2       .0  (     .0       .0       .0)         .0  (     .0       .0       .0)

11.2.2 & 11.3.2      LISTING OF THE ENTIRE PROVIDED REBAR

-------------------------------------------------------
SPAN  ID LOCATION  NUM  BAR  LENGTH [cm] AREA [cm^2]
--1----2-----3------4----5-------6---------7----------
  1    1     B      1 #  6 x     60        2.8
-------------------------------------------------------
Notes:
Bar location - T = Top, B = Bottom.
NUM - Number of bars.
For two-way systems a minimum of 4 bars is specified over the supports.
Refer to tables 11.5.1,11.5.2 and PTsum graphical display for
positioning of bars.

11.5.1     ARRANGEMENT OF TOP BARS

-------|----------- TOP STEEL -----------------|
SPAN   | ID   LOCATION | NUM  BAR   LENGTH [cm]|
--1----|--2------3-----|---4----5------6-------|


11.5.2     ARRANGEMENT OF BOTTOM BARS

-------|-------- BOTTOM STEEL ----------------|
SPAN   | ID   LOCATION | NUM  BAR   LENGTH [cm]|
--1----|--2------3-----|---4----5------6-------|
  1    |  1   CENTER   |    1 #  6 x     60    |
-------|---------------|-----------------------|

12 -  P U N C H I N G   S H E A R   C H E C K
===============================================================================

LEGEND:
CONDITION... 1 = INTERIOR COLUMN
             2 = END COLUMN
             3 = CORNER COLUMN
             4 = EDGE COLUMN (PARALLEL TO SPAN)
             5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
                 PERFORM SHEAR CHECK MANUALLY
             6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR
```

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CASE..... 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)

JNT	COND.	FACTORED ACTIONS shear T	moment T-m	<- PUNCHING SHEAR STRESSES IN Kg/cm^2 -> due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
1	1	242.96	119.90	8.01	3.12	11.14	20.73	.54	1
2	1	242.89	120.16	8.01	3.13	11.14	20.75	.54	1

PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

SPAN	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
	<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....>				
CANL	-.2	-.3	-.9(395)	.1(5799)	-.8(424)
1	.5	.1	.4(2039)	.0(19653)	.5(1848)
CANR	-.2	-.3	-.9(393)	.1(5799)	-.8(421)

```
                        ADAPT CORPORATION
                    STRUCTURAL CONCRETE SOFTWARE SYSTEM
          1733 Woodside Road, Suite 220, Redwood City, California 94061

                ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN
                Version 7.10  AMERICAN (ACI 318-99/UBC-77)
             ADAPT CORPORATION - Structural Concrete Software System
          1733 Woodside Road, Suite 220, Redwood City, California 94061
                   Phone: (650)306-2400,  Fax: (650)364-4678
        Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com

DATE AND TIME OF PROGRAM EXECUTION:              Mar 8,2021   At Time: 10:48
PROJECT FILE:                                          FL.3 GL. D7 (DH-DL)

P R O J E C T   T I T L E:
GCDT
FL.3 GL. D7 (DH-DL)

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
=============================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............    350.00 Kg/cm^2
                     for COLUMNS .................    350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............    275.00 T/cm^2
                      for COLUMNS .................    238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....      2.00
CONCRETE WEIGHT ..................................     NORMAL

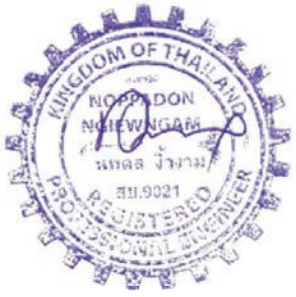
SELF WEIGHT ......................................   2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top  ..........................................      1.590
At Bottom  .......................................      1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................       .450

REINFORCEMENT:
YIELD Strength ...................................      4.00 T/cm^2
Minimum Cover at TOP .............................      2.00 cm
Minimum Cover at BOTTOM ..........................      2.00 cm

POST-TENSIONING:
SYSTEM ...........................................      BONDED
Ultimate strength of strand ......................     19.00 T/cm^2
Average effective stress in strand (final) .......     10.80 T/cm^2
Strand area.......................................       .990 cm^2
Min CGS of tendon from TOP........................      4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..     5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..     5.60 cm
```



Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ....(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

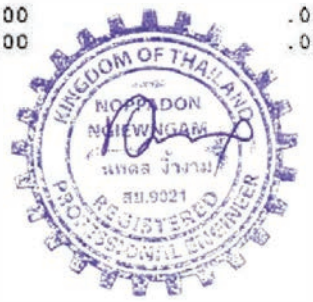
SPAN	TYPE	FORM	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
C	U	2	5.00	490.00	55.00	900.00	25.00			.00	.50	.50
1	U	2	9.00	490.00	55.00	900.00	25.00			.00	.50	.50
2	N	1	9.00	900.00	25.00					.00	.50	.50
C	U	1	2.00	900.00	25.00					.00	.50	.50

LEGEND:
1 - SPAN
C = Cantilever
2 - TYPE
U = Uniform; prismatic
N = Nonuniform section
3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 - Top surface to reference line

2.1.2 DETAILED DATA FOR NONUNIFORM SPANS

The following are geometry of nonuniform spans and/or cantilevers.
Left distance is from left support centerline to start of a span segment.

SEG	FORM	LEFT DISTANCE m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
SPAN: 2											
1	2	.00	490.00	55.00	900.00	25.00			.00	.50	.50
2	2	.48	490.00	55.00	900.00	25.00			.00	.50	.50



3	1	1.50	900.00	25.00		.00	.50	.50
4	1	8.52	900.00	25.00		.00	.50	.50

2.2 - S U P P O R T W I D T H A N D C O L U M N D A T A

JOINT	SUPPORT WIDTH cm	<------ LOWER COLUMN ------> LENGTH m	B(DIA) cm	D cm	CBC*	<------ UPPER COLUMN ------> LENGTH m	B(DIA) cm	D cm	CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
3	75.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

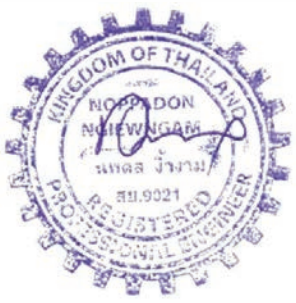
3 - I N P U T A P P L I E D L O A D I N G

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<--------------TYPE------------------>
U = UNIFORM P = PARTIAL UNIFORM
C = CONCENTRATED M = APPLIED MOMENT
Li= LINE LOAD

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	(From ... m	To) m)	(M or C ...At) (T-m or T ...m)		Total on Trib T/m
CANT	L	U	.500	.00	5.00			4.500
CANT	D	U	.300	.00	5.00			2.700
CANT	D	C				.00	1.09	
CANT	D	C				.00	2.36	
CANT	D	C				.00	2.94	
CANT	D	C				36.40	4.21	
CANT	SW	U		.00	5.00			8.928
1	L	U	.500	.00	9.00			4.500
1	D	U	.300	.00	9.00			2.700
1	SW	U		.00	9.00			8.928



2	L	U	.500	.00	.48	4.500
2	L	U	.500	.48	1.50	4.500
2	L	U	.500	1.50	8.52	4.500
2	L	U	.500	8.52	9.00	4.500
2	D	U	.300	.00	.48	2.700
2	D	U	.300	.48	1.50	2.700
2	D	U	.300	1.50	8.52	2.700
2	D	U	.300	8.52	9.00	2.700
2	SW	P		.00	.48	8.928
2	SW	P		.48	1.50	8.928
2	SW	P		1.50	8.52	5.400
2	SW	P		8.52	9.00	5.400
CANT	SW	U		.00	2.00	5.400

NOTE: LIVE LOADING is SKIPPED with a skip factor of .75

3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING

SPAN	CLASS	TYPE	UNIFORM (T/m^2), LINE(T/m)	(CON. or PART.) (T@m or m-m)		(M O M E N T) (T-m @ m)
CANT	L	U	.500			
CANT	D	U	.300			
CANT	D	C		.00	1.09	
CANT	D	C		.00	2.36	
CANT	D	C		.00	2.94	
CANT	D	C		36.40	4.21	
1	L	U	.500			
1	D	U	.300			
2	L	U	.500			
2	D	U	.300			

NOTE: SELFWEIGHT INCLUSION REQUIRED
LIVE LOADING is SKIPPED with a skip factor of .75

4 - C A L C U L A T E D S E C T I O N P R O P E R T I E S

4.1 For Uniform Spans and Cantilevers only

SPAN	AREA cm^2	I cm^4	Yb cm	Yt cm
CANT.	37200.00	.8998E+07	31.63	23.37
1	37200.00	.8998E+07	31.63	23.37

2	---	---	---	---
CANT.	22500.00	.1172E+07	12.50	12.50

Note:
--- = Span/Cantilever is Nonuniform, see block 4.2

4.2 - Computed Section Properties for Segments of Nonprismatic Spans

Section properties are listed for all segments of each span
A= cross-sectional geometry Yt= centroidal distance to top fiber
I= gross moment of inertia Yb= centroidal distance to bottom fiber

SPAN (SEGMENT)	AREA cm^2	I cm^4	Yb cm	Yt cm
SPAN 2				
1	37200.00	.8998E+07	31.63	23.37
2	37200.00	.8998E+07	31.63	23.37
3	22500.00	.1172E+07	12.50	12.50
4	22500.00	.1172E+07	12.50	12.50

5 - D E A D L O A D M O M E N T S, S H E A R S & R E A C T I O N S

	< 5.1 SPAN MOMENTS (Tm) >			< 5.2 SPAN SHEARS (T) >	
SPAN	M(l)*	Midspan	M(r)*	SH(l)	SH(r)
CANT	-----	-----	-298.59	-----	94.54
1	-166.23	7.56	-54.12	-64.78	39.87
2	-88.37	23.18	-33.26	-47.42	30.77
CANT	-10.80	-----	-----	-10.80	-----

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) ->	
		Lower columns	Upper columns
1	159.32	73.69	58.67
2	87.29	-19.07	-15.18
3	41.57	11.59	10.87

6 - L I V E L O A D M O M E N T S, S H E A R S & R E A C T I O N S

<-- 6.1 L I V E L O A D SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

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SPAN	<-- left* --> max	min	<-- midspan --> max	min	<-- right* --> max	min	<--SHEAR FORCE--> left	right
CL	-----		-----		-56.25		-----	22.50
1	-38.39	-11.62	17.36	-10.59	-34.69	-8.57	-20.66	19.84
2	-49.32	-1.59	12.87	-.78	-16.06	.82	-23.94	16.56
CR	.00		-----		-----		.00	-----

Note:
* = Centerline moments

JOINT	<- 6.2 REACTIONS (T) -> max	min	<-------- 6.3 COLUMN MOMENTS (Tm) --------> <--- LOWER COLUMN ---> max	min	<--- UPPER COLUMN ---> max	min
1	45.96	17.27	21.95	-12.01	17.48	-9.56
2	47.34	17.36	11.19	-19.33	8.91	-15.39
3	17.03	-.47	8.85	-.57	8.31	-.53

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

7 - M O M E N T S REDUCED TO FACE-OF-SUPPORT

7.1 R E D U C E D DEAD LOAD MOMENTS (Tm)

SPAN	<- left* ->	<- midspan ->	<- right* ->
CANT	-----	-----	-254.60
1	-136.50	7.56	-36.32
2	-66.95	23.18	-22.29
CANT	-7.13	-----	-----

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

SPAN	<----- left* -----> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-45.97	
1	-28.99	-5.80	17.36	-10.59	-25.69	-5.95
2	-38.34	-1.48	12.87	-.78	-10.17	.69
CR	.00		-----		-----	

Note:
* = face-of-support

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8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
CL	-----		-----		-300.57	
1	-165.49	-142.30	24.92	-3.03	-62.01	-42.28
2	-105.29	-68.43	36.05	22.40	-32.46	-21.60
CR	-7.13		-----		-----	

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

9.1 PROFILE TYPES AND PARAMETERS
LEGEND:
For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 TENDON PROFILE

	TYPE	X1/L	X2/L	X3/L	A/L
CANT	1			.000	
1	1	.100	.500	.100	.000
2	1	.100	.500	.100	.000
CANT	1	.100			

9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE
Tendon editing mode selected: TENDON SELECTION

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SPAN	FORCE (T/-)	DISTANCE OF CGS (cm) Left	Center	Right	P/A (Kg/cm^2)	Wbal (T/-)	Wbal (%DL)
CANT	1055.907	-23.37		-4.20	29.32	16.723	88
1	1083.373	-4.20	-10.40	-4.20	29.12	6.634	57
2	280.082	-4.20	-19.40	-10.50	12.45	3.333	38
CANT	270.396	-10.50		-12.50	12.16	2.735	51

Approximate weight of strand ..... 1247.0 Kg

9.35 - TENDON SELECTION DATA:

TYPE	SEL. FORCE (T)	TENDON EXTENTS <C>	<1>	<2>	<C>
A	21	12.75	|=========>		
B	63	12.95	<=========|		

9.5 R E Q U I R E D MINIMUM P O S T - T E N S I O N I N G FORCES (T)

SPAN	BASED ON STRESS CONDITIONS LEFT*	CENTER	RIGHT*	BASED ON MINIMUM P/A LEFT	CENTER	RIGHT
CANT	-----	-----	710.65	-----	-----	372.00
1	279.30	.00	.00	372.00	372.00	372.00
2	.00	63.65	38.54	372.00	225.00	225.00
CANT	.00	-----	-----	225.00	-----	-----

Note:
* = face-of-support

9.6 S E R V I C E S T R E S S E S (Kg/cm^2)(tension shown positive)

	LEFT * TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C	RIGHT * TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C
CL	-----	-----	-----	-----	4.67	-7.27	-----	-74.63
1	-----	-15.07	-----	-57.25	-----	-36.27	-----	-25.42
2	-----	-55.76	8.03	-4.93	-----	-11.60	-----	-24.33
CR	-----	-8.31	-----	-15.82	-----	-----	-----	-----

Note:
* = face-of-support

CENTER
TOP BOTTOM

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	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----
1	-----	-39.49	-----	-24.92
2	-----	-25.02	.12	-14.44
CR	-----	-----	-----	-----

9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

SPAN	<-- S P A N M O M E N T S (Tm) --> left*	midspan	right*	<-- SPAN SHEARS (T) --> SH(l)	SH(r)
CANT	-----	-----	170.80	-----	75.59
1	86.63	15.00	71.36	1.74	1.74
2	172.80	-24.27	21.06	-5.16	-5.16
CANT	3.61	-----	-----	4.40	-----

Note:
* = face-of-support

joint	<--REACTIONS (T)-->	<-- COLUMN MOMENTS (Tm) --> Lower columns	Upper columns
1	-1.739	-65.500	-52.150
2	6.895	57.420	45.710
3	-5.156	-9.557	-8.967

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

10.1 FACTORED DESIGN MOMENTS (Tm)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-434.59	
1	-358.78	-319.35	-84.24	-131.76	-224.82	-191.27
2	-184.32	-121.66	49.66	26.45	-31.91	-13.44
CR	-9.98		-----		-----	

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (Tm)
SPAN <-- left* --> <- midspan -> <-- right* -->

-1-	-2-	-3-	-4-
1	-118.40	-124.30	-130.30
2	-25.41	-4.68	16.59

Note:
* = face-of-support

10.3 FACTORED REACTIONS (T) — 10.4 FACTORED COLUMN MOMENTS (Tm)

JOINT	reaction max	reaction min	<-- LOWER column --> max	<-- LOWER column --> min	<-- UPPER column --> max	<-- UPPER column --> min
1	299.41	250.64	74.98	17.25	59.70	13.73
2	209.58	158.61	49.74	-2.14	39.60	-1.71
3	81.99	52.24	21.72	5.71	20.37	5.35

11 - M I L D S T E E L

Support cut-off length for minimum steel(length/span) ... .17
Span cut-off length for minimum steel(length/span) ... .33
Top bar extension beyond where required ..... 30.00 cm
Bottom bar extension beyond where required ..... 30.00 cm

REINFORCEMENT based on NO REDISTRIBUTION of factored moments

11.1 TOTAL WEIGHT OF REBAR = 16.3 Kg AVERAGE = .1 Kg/m^2
TOTAL AREA COVERED = 225.00 m^2

11.2.1 S T E E L A T M I D - S P A N

	TOP					BOTTOM				
SPAN	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS--->			As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS--->		
-1-	-2-	-3-	-4-	-5-		-6-	-7-	-8-	-9-	
1	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
2	13.4	(13.4	.0	.0)		.0	(.0	.0	.0)	

11.3.1 S T E E L A T S U P P O R T S

	TOP					BOTTOM				
JOINT	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	MIN--->			As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	MIN--->		
-1-	-2-	-3-	-4-	-5-		-6-	-7-	-8-	-9-	
1	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
2	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
3	.0	(.0	.0	.0)		.0	(.0	.0	.0)	

11.2.2 & 11.3.2 LISTING OF THE ENTIRE PROVIDED REBAR

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SPAN ID LOCATION NUM BAR LENGTH [cm] AREA [cm^2]
--1----2------3-------4----5-------6----------7----------

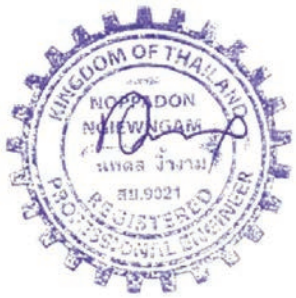
Notes:
Bar location - T = Top, B = Bottom.
NUM - Number of bars.
For two-way systems a minimum of 4 bars is specified over the supports.
Refer to tables 11.5.1,11.5.2 and PTsum graphical display for positioning of bars.

11.5.1 ARRANGEMENT OF TOP BARS

SPAN	TOP STEEL ID	LOCATION	NUM	BAR	LENGTH [cm]
--1--	--2--	--3--	--4--	--5--	--6--

11.5.2 ARRANGEMENT OF BOTTOM BARS

SPAN	BOTTOM STEEL ID	LOCATION	NUM	BAR	LENGTH [cm]
--1--	--2--	--3--	--4--	--5--	--6--



12 - P U N C H I N G S H E A R C H E C K

LEGEND:
CONDITION... 1 = INTERIOR COLUMN
2 = END COLUMN
3 = CORNER COLUMN
4 = EDGE COLUMN (PARALLEL TO SPAN)
5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
PERFORM SHEAR CHECK MANUALLY
6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

CASE..... 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)

JNT	COND.	FACTORED ACTIONS shear T	moment T-m	<- PUNCHING SHEAR STRESSES IN Kg/cm^2 -> due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
-1--	--2--	------3------	------4------	-------5-------	-------6-------	------7-------	-------8-------	------9-----	---10-
1	1	299.41	134.69	9.88	3.51	13.39	22.44	.60	1
2	1	209.58	89.34	6.91	2.33	9.24	22.39	.41	1
3	1	81.99	42.09	8.15	4.24	12.40	17.46	.71	1

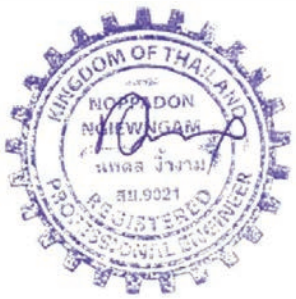
PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

	<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....>				
SPAN	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
-1-------2--------3-----------4---------------5---------------6------					
CANL	1.3	.3	1.0(478)	.2(2368)	1.3(398)
1	-.1	.0	.1(6436)	.0(*****)	.1(6808)
2	.4	-.1	-.3(3052)	.2(4039)	-.1(12500)
CANR	-.1	.0	-.1(1791)	-.1(1805)	-.2(899)



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                        ADAPT CORPORATION
                    STRUCTURAL CONCRETE SOFTWARE SYSTEM
          1733 Woodside Road, Suite 220, Redwood City, California 94061

                ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN
                Version 7.10  AMERICAN (ACI 318-99/UBC-77)
             ADAPT CORPORATION - Structural Concrete Software System
          1733 Woodside Road, Suite 220, Redwood City, California 94061
                  Phone: (650)306-2400,  Fax: (650)364-4678
        Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com

DATE AND TIME OF PROGRAM EXECUTION:                 Mar 8,2021   At Time: 9:50
PROJECT FILE:                                              FL.2 GL. D6 (DH-DL)


P R O J E C T   T I T L E:
GCDT
FL.2 GL. D6 (DH-DL)

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
=============================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............    350.00 Kg/cm^2
                     for COLUMNS .................    350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............    275.00 T/cm^2
                      for COLUMNS .................    238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....      2.00
CONCRETE WEIGHT ..................................     NORMAL

SELF WEIGHT ......................................   2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top  ..........................................      1.590
At Bottom  .......................................      1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................       .450

REINFORCEMENT:
YIELD Strength ...................................      4.00 T/cm^2
Minimum Cover at TOP .............................      2.00 cm
Minimum Cover at BOTTOM ..........................      2.00 cm

POST-TENSIONING:
SYSTEM ...........................................      BONDED
Ultimate strength of strand ......................     19.00 T/cm^2
Average effective stress in strand (final) .......     10.80 T/cm^2
Strand area.......................................       .990 cm^2
Min CGS of tendon from TOP........................      4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..     5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..     5.60 cm
```

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Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ....(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

SPAN	FORM	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
C	1	5.00	450.00	25.00					.00	.50	.50
1	1	9.00	450.00	25.00					.00	.50	.50
C	1	2.00	450.00	25.00					.00	.50	.50

LEGEND:
1 - SPAN
C = Cantilever

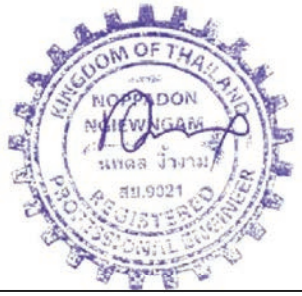
3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 - Top surface to reference line

2.1.5 - D R O P C A P A N D D R O P P A N E L D A T A

JOINT	CAPT cm	CAPB cm	CAPDL cm	CAPDR cm	DROPTL cm	DROPTR cm	DROPB cm	DROPL cm	DROPR cm
1	.00	.00	.00	.00	55.00	55.00	290.00	500.00	450.00
2	.00	.00	.00	.00	55.00	55.00	290.00	450.00	150.00

LEGEND:
DROP CAP DIMENSIONS:
CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint

DROP PANEL DIMENSIONS:
DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint



2.2 - S U P P O R T W I D T H A N D C O L U M N D A T A

JOINT	SUPPORT WIDTH cm	LOWER COLUMN LENGTH m	LOWER COLUMN B(DIA) cm	LOWER COLUMN D cm	LOWER COLUMN CBC*	UPPER COLUMN LENGTH m	UPPER COLUMN B(DIA) cm	UPPER COLUMN D cm	UPPER COLUMN CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

3 - I N P U T A P P L I E D L O A D I N G

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<----------TYPE---------->
U = UNIFORM P = PARTIAL UNIFORM
C = CONCENTRATED M = APPLIED MOMENT
Li= LINE LOAD

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	From ... m	To m	M or C ...At) T-m or T	...m)	Total on Trib T/m
CANT	L	U	.500	.00	5.00			2.250
CANT	D	U	.300	.00	5.00			1.350
CANT	D	C				7.50	1.09	
CANT	D	C				7.50	2.36	
CANT	D	C				8.20	2.94	
CANT	D	C				8.20	4.21	
CANT	SW	P		.00	5.00			4.788
1	L	U	.500	.00	4.50			2.250
1	L	U	.500	4.50	9.00			2.250
1	D	U	.300	.00	4.50			1.350
1	D	U	.300	4.50	9.00			1.350
1	SW	P		.00	4.50			4.788
1	SW	P		4.50	9.00			4.788

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CANT	L	U	.500	.00	1.50	2.250
CANT	L	U	.500	1.50	2.00	2.250
CANT	D	U	.300	.00	1.50	1.350
CANT	D	U	.300	1.50	2.00	1.350
CANT	SW	P		.00	1.50	4.788
CANT	SW	P		1.50	2.00	2.700

NOTE: LIVE LOADING is SKIPPED with a skip factor of .75

3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING

SPAN	CLASS	TYPE	UNIFORM (T/m^2), LINE(T/m)	(CON. or PART.) (T@m or m-m)		(M O M E N T) (T-m @ m)
CANT	L	U	.500			
CANT	D	U	.300			
CANT	D	C		7.50	1.09	
CANT	D	C		7.50	2.36	
CANT	D	C		8.20	2.94	
CANT	D	C		8.20	4.21	
1	L	U	.500			
1	D	U	.300			
CANT	L	U	.500			
CANT	D	U	.300			

NOTE: SELFWEIGHT INCLUSION REQUIRED
LIVE LOADING is SKIPPED with a skip factor of .75

4 - C A L C U L A T E D S E C T I O N P R O P E R T I E S

4.2 - Computed Section Properties for Segments of Nonprismatic Spans

Section properties are listed for all segments of each span
A= cross-sectional geometry Yt= centroidal distance to top fiber
I= gross moment of inertia Yb= centroidal distance to bottom fiber

SPAN (SEGMENT)	AREA cm^2	I cm^4	Yb cm	Yt cm
CANTILEVER LEFT				
1	19950.00	.4949E+07	30.51	24.49
SPAN 1				
1	19950.00	.4949E+07	30.51	24.49
2	19950.00	.4949E+07	30.51	24.49
CANTILEVER RIGHT				
1	19950.00	.4949E+07	30.51	24.49

2	11250.00	.5859E+06	12.50	12.50

5 - DEAD LOAD MOMENTS, SHEARS & REACTIONS

SPAN	< 5.1 SPAN MOMENTS (Tm) > M(l)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (T) > SH(l)	SH(r)
CANT	-----	-----	-161.23	-----	62.09
1	-64.22	16.40	-27.27	-31.73	23.52
CANT	-10.45	-----	-----	-11.23	-----

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) -> Lower columns	Upper columns
1	93.82	54.01	43.00
2	34.75	9.37	7.46

6 - LIVE LOAD MOMENTS, SHEARS & REACTIONS

<-- 6.1 LIVE LOAD SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

SPAN	<----- left* -----> max	min	<--- midspan ---> max	min	<---- right* -----> max	min	<--SHEAR FORCE--> left	right
CL	-----		-----		-28.13		-----	11.25
1	-18.18	-3.37	7.49	-1.19	-12.39	.98	-10.77	9.48
CR	-4.50		-----		-----		-4.50	-----

Note:
* = Centerline moments

JOINT	<- 6.2 REACTIONS (T) -> max	min	<-------- 6.3 COLUMN MOMENTS (Tm) --------> <--- LOWER COLUMN ---> max	min	<--- UPPER COLUMN ---> max	min
1	22.14	10.00	13.16	-7.63	10.48	-6.07
2	14.75	3.86	7.63	-3.23	6.07	-2.57

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

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7 - M O M E N T S REDUCED TO FACE-OF-SUPPORT

7.1 R E D U C E D DEAD LOAD MOMENTS (Tm)

SPAN	<- left* ->	<- midspan ->	<- right* ->
CANT	-----	-----	-132.10
1	-49.69	16.40	-16.69
CANT	-5.76	-----	-----

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-22.98	
1	-13.27	-3.13	7.49	-1.19	-8.10	.74
CR	-2.60		-----		-----	

Note:
* = face-of-support

8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-155.08	
1	-62.96	-52.82	23.89	15.21	-24.79	-15.95
CR	-8.36		-----		-----	

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

9.1 PROFILE TYPES AND PARAMETERS
LEGEND:

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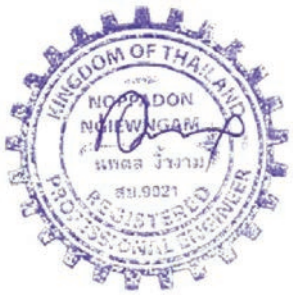
For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 TENDON PROFILE

TYPE	X1/L	X2/L	X3/L	A/L
CANT 1			.000	
1	.100	.500	.100	.000
CANT 1	.100			



9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Tendon editing mode selected: TENDON SELECTION

SPAN	SELECTED VALUES FORCE (T/-)	DISTANCE OF CGS (cm) Left	Center	Right	CALCULATED VALUES P/A (Kg/cm^2)	Wbal (T/-)	Wbal (%DL)
CANT	473.814	-22.65		-4.20	24.64	7.255	58
1	482.423	-4.20	-15.40	-7.00	24.18	4.669	76
CANT	452.489	-7.00		-12.50	22.92	12.576	224

Approximate weight of strand ..... 488.9 Kg

9.35 - TENDON SELECTION DATA:

TYPE	SEL. FORCE (T)	TENDON EXTENTS <C>	<1>	<C>
B	37 12.83	<==========		

9.5 REQUIRED MINIMUM POST-TENSIONING FORCES (T)

SPAN	BASED ON STRESS CONDITIONS LEFT*	CENTER	RIGHT*	BASED ON MINIMUM P/A LEFT	CENTER	RIGHT
CANT	-----	-----	348.82	-----	-----	199.50
1	16.46	.00	.00	199.50	199.50	199.50
CANT	.00	-----	-----	199.50	-----	-----

Note:
* = face-of-support

9.6 S E R V I C E S T R E S S E S (Kg/cm^2) (tension shown positive)

	L E F T *				R I G H T *			
	TOP		BOTTOM		TOP		BOTTOM	
	max-T	max-C	max-T	max-C	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----	11.20	-.17	-----	-68.71
1	-----	-16.09	-----	-41.53	-----	-29.95	-----	-21.14
CR	-----	-54.13	16.56	-----	-----	-----	-----	-----

Note:
* = face-of-support

	C E N T E R			
	TOP		BOTTOM	
	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----
1	-----	-29.43	-----	-23.00
CR	-----	-----	-----	-----

9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

	<-- S P A N M O M E N T S (Tm) -->			<-- SPAN SHEARS (T) -->	
SPAN	left*	midspan	right*	SH(l)	SH(r)
CANT	-----	-----	83.18	-----	32.79
1	35.56	-13.29	28.77	-.96	-.96
CANT	69.37	-----	-----	19.11	-----

Note:
* = face-of-support

	<--REACTIONS (T)-->	<-- COLUMN MOMENTS (Tm) -->	
joint		Lower columns	Upper columns
1	.956	-34.210	-27.240
2	-.956	26.210	20.860

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

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Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

10.1 FACTORED DESIGN MOMENTS (Tm)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
CL	-----		-----		-224.01	
1	-152.80	-135.57	-18.56	-33.32	-84.98	-69.94
CR	-12.49		-----		-----	

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (Tm)

SPAN	<-- left* -->	<- midspan ->	<-- right* -->
1	-60.68	-54.26	-47.84

Note:
* = face-of-support

10.3 FACTORED REACTIONS (T) 10.4 FACTORED COLUMN MOMENTS (Tm)

			<-- LOWER column -->		<-- UPPER column -->	
JOINT	max	min	max	min	max	min
1	169.94	149.30	63.78	28.44	50.78	22.64
2	72.77	54.25	52.28	33.83	41.62	26.93

11 - M I L D S T E E L

Support cut-off length for minimum steel(length/span) ... .17
Span cut-off length for minimum steel(length/span) ... .33
Top bar extension beyond where required ..... 30.00 cm
Bottom bar extension beyond where required ..... 30.00 cm

RREINFORCEMENT based on NO REDISTRIBUTION of factored moments

11.1 TOTAL WEIGHT OF REBAR = .0 Kg AVERAGE = .0 Kg/m^2
TOTAL AREA COVERED = 72.00 m^2

11.2.1 S T E E L A T M I D - S P A N

	TOP		BOTTOM	
SPAN	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----TENS--------->	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----TENS--------->

```
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 ------------------------------------------------------------------------------

 --1------2---------3-------4-------5-----------6---------7-------8-------9----
   1       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)


 11.3.1     S T E E L    A T    S U P P O R T S
                      T O P                                  B O T T O M
          As     DIFFERENT REBAR CRITERIA          As     DIFFERENT REBAR CRITERIA
 JOINT  (cm^2) <---ULT-----MIN--------->        (cm^2) <---ULT-----MIN--------->
 --1------2---------3-------4-------5-----------6---------7-------8-------9----
   1       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)
   2       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)

 11.2.2 & 11.3.2      LISTING OF THE ENTIRE PROVIDED REBAR

 ------------------------------------------------------
 SPAN  ID LOCATION  NUM  BAR  LENGTH [cm] AREA [cm^2]
 --1----2-----3------4----5-------6---------7----------
 Notes:
 Bar location - T = Top, B = Bottom.
 NUM - Number of bars.
 For two-way systems a minimum of 4 bars is specified over the supports.
 Refer to tables 11.5.1,11.5.2 and PTsum graphical display for
 positioning of bars.


 11.5.1      ARRANGEMENT OF TOP BARS

 -------|----------- TOP STEEL -----------------|
 SPAN    | ID   LOCATION | NUM  BAR   LENGTH [cm]|
 --1----|--2------3-----|---4----5------6-------|


 11.5.2      ARRANGEMENT OF BOTTOM BARS

 -------|-------- BOTTOM STEEL -----------------|
 SPAN    | ID   LOCATION | NUM  BAR   LENGTH [cm]|
 --1----|--2------3-----|---4----5------6-------|


 12 -  P U N C H I N G   S H E A R   C H E C K
 ==============================================================================

 LEGEND:
 CONDITION... 1 = INTERIOR COLUMN
              2 = END COLUMN
              3 = CORNER COLUMN
              4 = EDGE COLUMN (PARALLEL TO SPAN)
              5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
                  PERFORM SHEAR CHECK MANUALLY
              6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

 CASE........ 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
              2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)
```

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JNT	COND.	FACTORED ACTIONS shear T	moment T-m	<- PUNCHING SHEAR STRESSES IN Kg/cm^2 -> due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
1	1	169.94	114.55	5.61	2.99	8.59	21.24	.40	1
2	1	72.77	93.90	2.40	2.45	4.85	21.13	.23	1

PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

SPAN	<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....> DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
CANL	.9	.3	.9(571)	.1(3380)	1.0(489)
1	.0	.0	.1(12558)	.0(33014)	.1(9097)
CANR	.0	-.1	-.4(446)	.0(97483)	-.5(444)

```
                        ADAPT CORPORATION
                   STRUCTURAL CONCRETE SOFTWARE SYSTEM
       1733 Woodside Road, Suite 220, Redwood City, California 94061

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DATE AND TIME OF PROGRAM EXECUTION:              Mar 8,2021   At Time: 9:45
PROJECT FILE:                                          FL.2 GL. D5 (DH-DL)


P R O J E C T   T I T L E:
GCDT
FL.2 GL. D5 (DH-DL)

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
=============================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............    350.00 Kg/cm^2
                     for COLUMNS .................    350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............    275.00 T/cm^2
                      for COLUMNS .................    238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....      2.00
CONCRETE WEIGHT ..................................     NORMAL

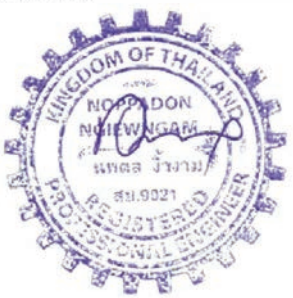
SELF WEIGHT ......................................   2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top  ..........................................      1.590
At Bottom  .......................................      1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................       .450

REINFORCEMENT:
YIELD Strength ...................................      4.00 T/cm^2
Minimum Cover at TOP .............................      2.00 cm
Minimum Cover at BOTTOM ..........................      2.00 cm

POST-TENSIONING:
SYSTEM ...........................................     BONDED
Ultimate strength of strand ......................     19.00 T/cm^2
Average effective stress in strand (final) .......     10.80 T/cm^2
Strand area.......................................      .990 cm^2
Min CGS of tendon from TOP........................      4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..     5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..     5.60 cm
```



Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ....(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES --

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

SPAN	FORM	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
C	1	5.00	100.00	25.00					.00	4.50	.50
1	1	9.00	100.00	25.00					.00	4.50	4.50
2	1	9.00	100.00	25.00					.00	4.50	4.50
3	1	9.00	100.00	25.00					.00	4.50	4.50
C	1	3.50	100.00	25.00					.00	4.50	4.50

LEGEND:
1 - SPAN
C = Cantilever

3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle

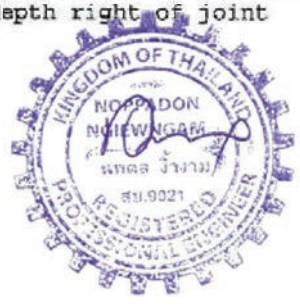
11 - Top surface to reference line

2.1.5 - D R O P C A P A N D D R O P P A N E L D A T A

JOINT	CAPT cm	CAPB cm	CAPDL cm	CAPDR cm	DROPTL cm	DROPTR cm	DROPB cm	DROPL cm	DROPR cm
1	.00	.00	.00	.00	55.00	55.00	400.00	500.00	450.00
2	.00	.00	.00	.00	55.00	55.00	400.00	450.00	150.00
3	.00	.00	.00	.00	35.00	35.00	300.00	150.00	150.00
4	.00	.00	.00	.00	35.00	35.00	300.00	150.00	150.00

LEGEND:
DROP CAP DIMENSIONS:
CAPT = Total depth of cap
CAPB = Transverse Width

DROP PANEL DIMENSIONS:
DROPTL = Total depth left of joint
DROPTR = Total depth right of joint



CAPDL = Extension left of joint
CAPDR = Extension right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint

2.2 - S U P P O R T W I D T H A N D C O L U M N D A T A

JOINT	SUPPORT WIDTH cm	LOWER COLUMN LENGTH m	LOWER COLUMN B(DIA) cm	LOWER COLUMN D cm	LOWER COLUMN CBC*	UPPER COLUMN LENGTH m	UPPER COLUMN B(DIA) cm	UPPER COLUMN D cm	UPPER COLUMN CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
3	75.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
4	75.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

3 - I N P U T A P P L I E D L O A D I N G

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<----------TYPE---------->
U = UNIFORM
C = CONCENTRATED
Li= LINE LOAD
P = PARTIAL UNIFORM
M = APPLIED MOMENT

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	From ... m	To m	M or C ...At) T-m or T	...m)	Total on Trib T/m
CANT	L	U	.500	.00	5.00			2.500
CANT	D	U	.300	.00	5.00			1.500
CANT	D	C				7.50	1.09	
CANT	D	C				7.50	2.36	
CANT	D	C				8.20	2.94	
CANT	D	C				8.20	4.21	
CANT	SW	P		.00	5.00			5.880
1	L	U	.500	.00	4.50			4.500

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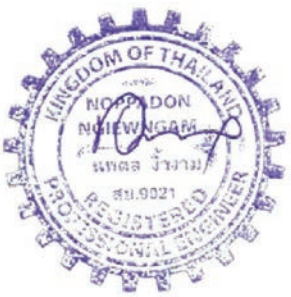
1	L	U	.500	4.50	9.00	4.500
1	D	U	.300	.00	4.50	2.700
1	D	U	.300	4.50	9.00	2.700
1	SW	P		.00	4.50	8.280
1	SW	P		4.50	9.00	8.280
2	L	U	.500	.00	1.50	4.500
2	L	U	.500	1.50	7.50	4.500
2	L	U	.500	7.50	9.00	4.500
2	D	U	.300	.00	1.50	2.700
2	D	U	.300	1.50	7.50	2.700
2	D	U	.300	7.50	9.00	2.700
2	SW	P		.00	1.50	8.280
2	SW	P		1.50	7.50	5.400
2	SW	P		7.50	9.00	6.120
3	L	U	.500	.00	1.50	4.500
3	L	U	.500	1.50	7.50	4.500
3	L	U	.500	7.50	9.00	4.500
3	D	U	.300	.00	1.50	2.700
3	D	U	.300	1.50	7.50	2.700
3	D	U	.300	7.50	9.00	2.700
3	SW	P		.00	1.50	6.120
3	SW	P		1.50	7.50	5.400
3	SW	P		7.50	9.00	6.120
CANT	L	U	.500	.00	1.50	4.500
CANT	L	U	.500	1.50	3.50	4.500
CANT	D	U	.300	.00	1.50	2.700
CANT	D	U	.300	1.50	3.50	2.700
CANT	SW	P		.00	1.50	6.120
CANT	SW	P		1.50	3.50	5.400

NOTE: LIVE LOADING is SKIPPED with a skip factor of .75

3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING

SPAN	CLASS	TYPE	UNIFORM (T/m^2), LINE(T/m)	(CON. or PART.) (T@m or m-m)		(M O M E N T) (T-m @ m)
CANT	L	U	.500			
CANT	D	U	.300			
CANT	D	C		7.50	1.09	
CANT	D	C		7.50	2.36	
CANT	D	C		8.20	2.94	
CANT	D	C		8.20	4.21	
1	L	U	.500			
1	D	U	.300			
2	L	U	.500			



2	D	U	.300
3	L	U	.500
3	D	U	.300
CANT	L	U	.500
CANT	D	U	.300

NOTE: SELFWEIGHT INCLUSION REQUIRED
LIVE LOADING is SKIPPED with a skip factor of .75

4 - C A L C U L A T E D S E C T I O N P R O P E R T I E S

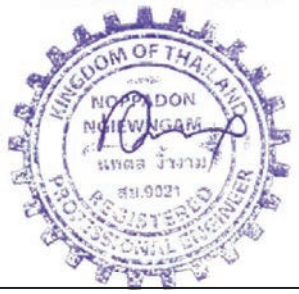
4.2 - Computed Section Properties for Segments of Nonprismatic Spans

Section properties are listed for all segments of each span
A= cross-sectional geometry Yt= centroidal distance to top fiber
I= gross moment of inertia Yb= centroidal distance to bottom fiber

SPAN (SEGMENT)	AREA cm^2	I cm^4	Yb cm	Yt cm
CANTILEVER LEFT				
1	24500.00	.6181E+07	29.03	25.97
SPAN 1				
1	34500.00	.7990E+07	32.93	22.07
2	34500.00	.7990E+07	32.93	22.07
SPAN 2				
1	34500.00	.7990E+07	32.93	22.07
2	22500.00	.1172E+07	12.50	12.50
3	25500.00	.2008E+07	20.44	14.56
SPAN 3				
1	25500.00	.2008E+07	20.44	14.56
2	22500.00	.1172E+07	12.50	12.50
3	25500.00	.2008E+07	20.44	14.56
CANTILEVER RIGHT				
1	25500.00	.2008E+07	20.44	14.56
2	22500.00	.1172E+07	12.50	12.50

5 - D E A D L O A D M O M E N T S, S H E A R S & R E A C T I O N S

SPAN	< 5.1 SPAN MOMENTS (Tm) > M(1)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (T) > SH(l)	SH(r)
CANT	-----	-----	-176.76	-----	68.30
1	-103.12	27.01	-65.20	-53.62	45.20
2	-75.17	19.13	-54.65	-42.78	35.52
3	-59.64	24.00	-58.01	-37.71	37.35



CANT	-50.42	-----	-----	-29.43	-----

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) ->	
		Lower columns	Upper columns
1	121.92	41.00	32.64
2	87.98	-5.55	-4.42
3	73.23	-2.64	-2.35
4	66.78	4.02	3.57

6 - L I V E L O A D M O M E N T S, S H E A R S & R E A C T I O N S

<-- 6.1 L I V E L O A D SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

	<----- left* ----->		<--- midspan --->		<----- right* ----->		<--SHEAR FORCE-->	
SPAN	max	min	max	min	max	min	left	right
CL	-----		-----		-31.25		-----	12.50
1	-29.12	-1.77	16.77	-6.36	-34.25	-8.90	-19.68	20.82
2	-40.02	1.79	10.58	-1.89	-29.95	-4.94	-21.37	19.13
3	-32.72	-1.65	13.30	-2.75	-31.81	-3.47	-20.35	20.15
CR	-27.56		-----		-----		-15.75	-----

Note:
* = Centerline moments

<- 6.2 REACTIONS (T) -> <-------- 6.3 COLUMN MOMENTS (Tm) -------->

			<--- LOWER COLUMN --->		<--- UPPER COLUMN --->	
JOINT	max	min	max	min	max	min
1	34.83	11.14	16.09	-14.90	12.81	-11.86
2	45.65	17.19	12.26	-15.47	9.76	-12.32
3	41.56	17.08	10.50	-11.96	9.34	-10.65
4	36.90	15.97	14.38	-12.13	12.80	-10.80

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

7 - M O M E N T S REDUCED TO FACE-OF-SUPPORT

7.1 R E D U C E D DEAD LOAD MOMENTS (Tm)

SPAN	<- left* ->	<- midspan ->	<- right* ->
1			

CANT	-----	-----	-144.80
1	-78.64	27.01	-44.77
2	-55.90	19.13	-41.95
3	-46.12	24.00	-44.62
CANT	-40.01	-----	-----

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
-1	----------2	----------3	----------4	----------5	----------6	----------7
CL	-----		-----		-25.54	
1	-20.19	-2.26	16.77	-6.36	-24.78	-6.51
2	-30.28	1.40	10.58	-1.89	-23.09	-4.66
3	-25.41	-1.73	13.30	-2.75	-24.57	-3.41
CR	-21.97		-----		-----	

Note:
* = face-of-support

8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
-1	----------2	----------3	----------4	----------5	----------6	----------7
CL	-----		-----		-170.34	
1	-98.83	-80.90	43.78	20.65	-69.55	-51.28
2	-86.18	-54.50	29.71	17.24	-65.04	-46.61
3	-71.53	-47.85	37.30	21.25	-69.19	-48.03
CR	-61.98		-----		-----	

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

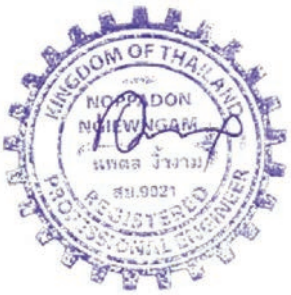
9.1 PROFILE TYPES AND PARAMETERS
LEGEND:
For Span:

1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 TENDON PROFILE

	TYPE	X1/L	X2/L	X3/L	A/L
CANT	1			.000	
1	1	.100	.500	.100	.000
2	1	.100	.500	.100	.000
3	1	.100	.500	.100	.000
CANT	1	.100			



9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Tendon editing mode selected: TENDON SELECTION

SPAN	SELECTED VALUES FORCE (T/-)	DISTANCE OF CGS (cm) Left	Center	Right	CALCULATED VALUES P/A (Kg/cm^2)	Wbal (T/-)	Wbal (%DL)
CANT	450.475	-25.97		-4.20	19.01	8.112	59
1	460.118	-4.20	-13.40	-4.20	13.34	4.181	38
2	427.729	-4.20	-19.40	-4.20	19.01	6.421	74
3	460.336	-4.20	-19.40	-4.20	20.46	6.911	83
CANT	443.423	-4.20		-12.50	20.15	6.145	73

Approximate weight of strand ..... 992.9 Kg

9.35 - TENDON SELECTION DATA:

TYPE	SEL.	FORCE (T)	TENDON EXTENTS <C>	<1>	<2>	<3>	<C>
A	35	12.88	<==========				==========>

9.5 REQUIRED MINIMUM POST-TENSIONING FORCES (T)

SPAN	BASED ON STRESS CONDITIONS LEFT*	CENTER	RIGHT*	BASED ON MINIMUM P/A LEFT	CENTER	RIGHT

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CANT	-----	-----	360.62	-----	-----	245.00
1	.00	.00	.00	345.00	345.00	345.00
2	.00	19.07	144.02	345.00	225.00	255.00
3	179.78	95.72	171.85	255.00	225.00	255.00
CANT	147.83	-----	-----	255.00	-----	-----

Note:
* = face-of-support

9.6 S E R V I C E S T R E S S E S (Kg/cm^2)(tension shown positive)

	L E F T *				R I G H T *			
	TOP		BOTTOM		TOP		BOTTOM	
	max-T	max-C	max-T	max-C	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----	17.84	-----	-----	-59.99
1	4.94	-.01	-----	-41.57	-----	-10.89	-----	-23.92
2	-----	-21.26	-----	-13.12	-----	-19.38	-----	-32.83
3	-----	-20.31	-----	-37.73	-----	-19.06	-----	-37.52
CR	-----	-16.95	-----	-40.71	-----	-----	-----	-----

Note:
* = face-of-support

C E N T E R

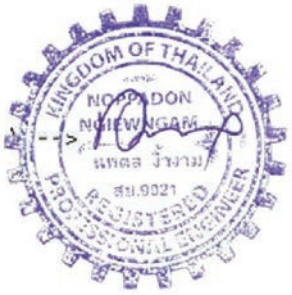
	TOP		BOTTOM	
	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----
1	-----	-24.57	3.43	-6.11
2	-----	-26.10	-----	-25.22
3	-----	-32.44	-----	-25.60
CR	-----	-----	-----	-----

9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

	<-- S P A N M O M E N T S (Tm) -->			<-- SPAN SHEARS (T) -->	
SPAN	left*	midspan	right*	SH(l)	SH(r)
CANT	-----	-----	82.87	-----	36.67
1	31.27	-3.11	43.30	-1.61	-1.61
2	85.29	-23.06	49.66	.44	.44
3	51.69	-26.07	49.80	.25	.25
CANT	39.22	-----	-----	18.44	-----

Note:
* = face-of-support

<--REACTIONS (T)--> <-- COLUMN MOMENTS (Tm)-->



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-joint-	2	Lower columns	Upper columns
1	1.614	-27.970	-22.270
2	-2.053	23.890	19.020
3	.194	1.240	1.103
4	.246	-3.037	-2.702

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

10.1 FACTORED DESIGN MOMENTS (Tm)

	<----- left* ------>		<---- midspan ---->		<----- right* ----->	
SPAN	max	min	max	min	max	min
CL	-----		-----		-246.14	
1	-193.89	-163.40	23.35	-15.98	-141.29	-110.24
2	-122.63	-68.77	51.17	29.98	-92.31	-60.98
3	-99.90	-59.65	63.05	35.77	-98.41	-62.44
CR	-93.36		-----		-----	

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (Tm)

SPAN	<-- left* -->	<- midspan ->	<-- right* -->
1	-49.47	-42.98	-36.49
2	7.11	6.40	5.68
3	7.86	6.85	5.83

Note:
* = face-of-support

	10.3 FACTORED REACTIONS (T)		10.4 FACTORED COLUMN MOMENTS (Tm)			
			<-- LOWER column -->		<-- UPPER column -->	
JOINT	max	min	max	min	max	min
1	231.49	191.21	56.78	4.10	45.20	3.26
2	198.72	150.34	36.97	-10.17	29.43	-8.11
3	173.37	131.75	15.40	-22.79	13.69	-20.29
4	156.47	120.89	27.03	-18.04	24.06	-16.06

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11 - M I L D S T E E L

Support cut-off length for minimum steel(length/span) ... .17
Span cut-off length for minimum steel(length/span) ... .33
Top bar extension beyond where required ..... 30.00 cm
Bottom bar extension beyond where required ..... 30.00 cm

REINFORCEMENT based on NO REDISTRIBUTION of factored moments

11.1 TOTAL WEIGHT OF REBAR = .0 Kg AVERAGE = .0 Kg/m^2
TOTAL AREA COVERED = 299.50 m^2

11.2.1 S T E E L A T M I D - S P A N

	TOP					BOTTOM				
SPAN	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS			As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS		
1	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
2	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
3	.0	(.0	.0	.0)		.0	(.0	.0	.0)	

11.3.1 S T E E L A T S U P P O R T S

	TOP					BOTTOM				
JOINT	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	MIN			As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	MIN		
1	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
2	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
3	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
4	.0	(.0	.0	.0)		.0	(.0	.0	.0)	

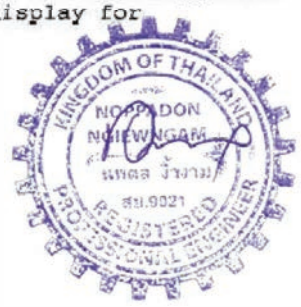
11.2.2 & 11.3.2 LISTING OF THE ENTIRE PROVIDED REBAR

SPAN	ID	LOCATION	NUM	BAR	LENGTH [cm]	AREA [cm^2]

Notes:
Bar location - T = Top, B = Bottom.
NUM - Number of bars.
For two-way systems a minimum of 4 bars is specified over the supports.
Refer to tables 11.5.1,11.5.2 and PTsum graphical display for positioning of bars.

11.5.1 ARRANGEMENT OF TOP BARS

SPAN	TOP STEEL ID	LOCATION	NUM	BAR	LENGTH [cm]



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11.5.2 ARRANGEMENT OF BOTTOM BARS

SPAN	ID	LOCATION	NUM	BAR	LENGTH [cm]
		BOTTOM STEEL			

12 - P U N C H I N G S H E A R C H E C K

LEGEND:
CONDITION... 1 = INTERIOR COLUMN
2 = END COLUMN
3 = CORNER COLUMN
4 = EDGE COLUMN (PARALLEL TO SPAN)
5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
PERFORM SHEAR CHECK MANUALLY
6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

CASE..... 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)

JNT	COND.	FACTORED ACTIONS shear T	moment T-m	<- PUNCHING SHEAR STRESSES IN Kg/cm^2 -> due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
1	4	231.49	101.99	11.52	3.82	15.34	17.13	.90	1
2	1	198.72	66.40	6.56	1.73	8.29	19.81	.42	1
3	1	173.37	43.08	10.83	2.50	13.33	20.18	.66	1
4	1	156.47	51.09	9.77	2.96	12.73	20.18	.63	1

PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

SPAN	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
	<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....>				

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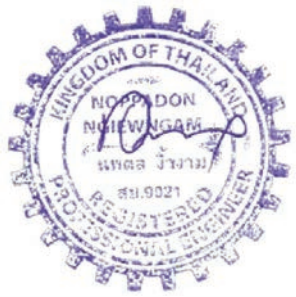
CANL	.8	.3	.9(530)	.1(4055)	1.1(469)
1	.0	.1	.2(3894)	.0(32451)	.3(3476)
2	.3	-.1	-.2(4200)	.1(6470)	-.1(11976)
3	.4	.0	.1(17821)	.2(4515)	.2(3602)
CANR	.2	-.2	-.7(485)	.1(2793)	-.6(587)



```
|                          ADAPT CORPORATION                              |
|                  STRUCTURAL CONCRETE SOFTWARE SYSTEM                    |
|       1733 Woodside Road, Suite 220, Redwood City, California 94061     |
|                ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN             |
|                Version 7.10  AMERICAN (ACI 318-99/UBC-77)               |
|           ADAPT CORPORATION - Structural Concrete Software System       |
|       1733 Woodside Road, Suite 220, Redwood City, California 94061     |
|                Phone: (650)306-2400,  Fax: (650)364-4678                |
|     Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com    |
DATE AND TIME OF PROGRAM EXECUTION:              Jan 26,2021   At Time: 9:35
PROJECT FILE:                                                   FL.6 GL. DH

P R O J E C T   T I T L E:
GCDT
FL.7 GL .DH

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
=========================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............    350.00 Kg/cm^2
                     for COLUMNS .................    350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............    275.00 T/cm^2
                      for COLUMNS .................    238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....      2.00
CONCRETE WEIGHT ..................................     NORMAL

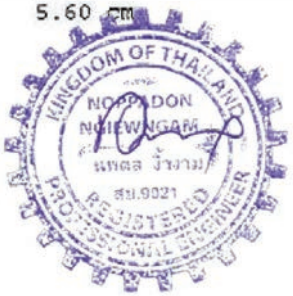
SELF WEIGHT ......................................   2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top  ..........................................      1.590
At Bottom  .......................................      1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................       .450

REINFORCEMENT:
YIELD Strength ...................................      4.00 T/cm^2
Minimum Cover at TOP .............................      2.00 cm
Minimum Cover at BOTTOM ..........................      2.00 cm

POST-TENSIONING:
SYSTEM ...........................................      BONDED
Ultimate strength of strand ......................     19.00 T/cm^2
Average effective stress in strand (final) .......     10.80 T/cm^2
Strand area.......................................       .990 cm^2
Min CGS of tendon from TOP........................      4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..     5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..     5.60 cm
```



Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ....(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

SPAN	FOR M	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
1	1	18.00	100.00	25.00					.00	4.50	4.50
2	1	9.00	100.00	25.00					.00	4.50	4.50
3	1	9.00	100.00	25.00					.00	4.50	4.50
4	1	9.00	100.00	25.00					.00	4.50	4.50
5	1	4.50	100.00	25.00					.00	4.50	4.50

LEGEND:
1 - SPAN
C = Cantilever

3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 - Top surface to reference line

2.1.5 - D R O P C A P A N D D R O P P A N E L D A T A

JOINT	CAPT cm	CAPB cm	CAPDL cm	CAPDR cm	DROPTL cm	DROPTR cm	DROPB cm	DROPL cm	DROPR cm
1	.00	.00	.00	.00	.00	65.00	400.00	.00	900.00
2	.00	.00	.00	.00	65.00	65.00	400.00	900.00	150.00
3	.00	.00	.00	.00	35.00	35.00	400.00	150.00	150.00
4	.00	.00	.00	.00	35.00	35.00	400.00	150.00	150.00
5	.00	.00	.00	.00	35.00	35.00	400.00	150.00	150.00
6	.00	.00	.00	.00	35.00	.00	400.00	150.00	.00

LEGEND:
DROP CAP DIMENSIONS:
DROP PANEL DIMENSIONS:

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CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint

DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint

2.2 - S U P P O R T W I D T H A N D C O L U M N D A T A

JOINT	SUPPORT WIDTH cm	<------ LOWER COLUMN ------> LENGTH m	B(DIA) cm	D cm	CBC*	<------ UPPER COLUMN ------> LENGTH m	B(DIA) cm	D cm	CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
3	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
4	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
5	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
6	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

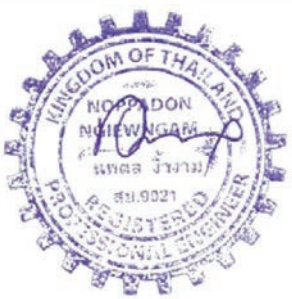
3 - I N P U T A P P L I E D L O A D I N G

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<----------TYPE---------->
U = UNIFORM
C = CONCENTRATED
Li= LINE LOAD
P = PARTIAL UNIFORM
M = APPLIED MOMENT

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	(From ... m	To) m	(M or C ...At) T-m or T ...m)	Total on Trib T/m
1	L	U	.500	.00	9.00		4.500
1	L	U	.500	9.00	18.00		4.500
1	D	U	.300	.00	9.00		2.700
1	D	U	.300	9.00	18.00		2.700



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1	SW	P		.00	9.00	9.240
1	SW	P		9.00	18.00	9.240
2	L	U	.500	.00	1.50	4.500
2	L	U	.500	1.50	7.50	4.500
2	L	U	.500	7.50	9.00	4.500
2	D	U	.300	.00	1.50	2.700
2	D	U	.300	1.50	7.50	2.700
2	D	U	.300	7.50	9.00	2.700
2	SW	P		.00	1.50	9.240
2	SW	P		1.50	7.50	5.400
2	SW	P		7.50	9.00	6.360
3	L	U	.500	.00	1.50	4.500
3	L	U	.500	1.50	7.50	4.500
3	L	U	.500	7.50	9.00	4.500
3	D	U	.300	.00	1.50	2.700
3	D	U	.300	1.50	7.50	2.700
3	D	U	.300	7.50	9.00	2.700
3	SW	P		.00	1.50	6.360
3	SW	P		1.50	7.50	5.400
3	SW	P		7.50	9.00	6.360
4	L	U	.500	.00	1.50	4.500
4	L	U	.500	1.50	7.50	4.500
4	L	U	.500	7.50	9.00	4.500
4	D	U	.300	.00	1.50	2.700
4	D	U	.300	1.50	7.50	2.700
4	D	U	.300	7.50	9.00	2.700
4	SW	P		.00	1.50	6.360
4	SW	P		1.50	7.50	5.400
4	SW	P		7.50	9.00	6.360
5	L	U	.500	.00	1.50	4.500
5	L	U	.500	1.50	3.00	4.500
5	L	U	.500	3.00	4.50	4.500
5	D	U	.300	.00	1.50	2.700
5	D	U	.300	1.50	3.00	2.700
5	D	U	.300	3.00	4.50	2.700
5	SW	P		.00	1.50	6.360
5	SW	P		1.50	3.00	5.400
5	SW	P		3.00	4.50	6.360

NOTE: LIVE LOADING is SKIPPED with a skip factor of .75

3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING

UNIFORM
(T/m^2), (CON. or PART.) (M O M E N T)

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SPAN	CLASS	TYPE	LINE	(T/m)	(T@m or m-m)	(T-m @ m)
1	L	U		.500		
1	D	U		.300		
2	L	U		.500		
2	D	U		.300		
3	L	U		.500		
3	D	U		.300		
4	L	U		.500		
4	D	U		.300		
5	L	U		.500		
5	D	U		.300		

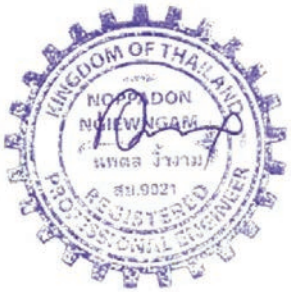
NOTE: SELFWEIGHT INCLUSION REQUIRED
LIVE LOADING is SKIPPED with a skip factor of .75

4 - C A L C U L A T E D S E C T I O N P R O P E R T I E S

4.2 - Computed Section Properties for Segments of Nonprismatic Spans

Section properties are listed for all segments of each span
A= cross-sectional geometry Yt= centroidal distance to top fiber
I= gross moment of inertia Yb= centroidal distance to bottom fiber

SPAN (SEGMENT)	AREA cm^2	I cm^4	Yb cm	Yt cm
SPAN 1				
1	38500.00	.1318E+08	38.99	26.01
2	38500.00	.1318E+08	38.99	26.01
SPAN 2				
1	38500.00	.1318E+08	38.99	26.01
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14
SPAN 3				
1	26500.00	.2245E+07	19.86	15.14
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14
SPAN 4				
1	26500.00	.2245E+07	19.86	15.14
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14
SPAN 5				
1	26500.00	.2245E+07	19.86	15.14
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14



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5 - DEAD LOAD MOMENTS, SHEARS & REACTIONS

SPAN	< 5.1 SPAN MOMENTS (Tm) > M(l)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (T) > SH(l)	SH(r)
1	-247.87	212.80	-293.66	-104.92	110.00
2	-109.90	10.02	-39.47	-49.68	30.42
3	-55.08	23.31	-64.47	-36.85	38.93
4	-64.89	23.63	-54.03	-39.10	36.68
5	-35.14	2.40	-3.23	-26.76	12.57

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) -> Lower columns	Upper columns
1	104.92	-141.59	-106.28
2	159.68	104.97	78.80
3	67.27	-8.26	-7.35
4	78.03	-.22	-.20
5	63.44	10.00	8.89
6	12.57	1.71	1.52

6 - LIVE LOAD MOMENTS, SHEARS & REACTIONS

<-- 6.1 LIVE LOAD SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

SPAN	<----- left* -----> max	min	<--- midspan ---> max	min	<----- right* -----> max	min	<--SHEAR FORCE--> left	right
1	-92.19	3.72	79.15	-3.19	-114.00	-8.59	-39.29	41.71
2	-52.34	-6.12	9.16	-3.78	-24.43	-.62	-23.35	17.15
3	-30.83	-.54	12.71	-2.55	-34.88	-3.36	-19.80	20.70
4	-35.40	-4.54	13.11	-1.78	-29.50	1.72	-20.91	19.59
5	-18.94	-3.87	4.17	-3.29	-4.88	3.75	-14.00	7.82

Note:
* = Centerline moments

JOINT	<- 6.2 REACTIONS (T) -> max	min	<-------- 6.3 COLUMN MOMENTS (Tm) --------> <--- LOWER COLUMN ---> max	min	<--- UPPER COLUMN ---> max	min
1	40.31	-1.02	2.83	-55.49	2.12	-41.66
2	66.80	21.72	52.77	-17.55	39.61	-13.17
3	40.97	15.06	10.69	-14.07	9.51	-12.52

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4	43.07	20.05	12.36	-12.63	11.00	-11.24
5	35.40	8.79	9.53	-3.94	8.48	-3.51
6	10.43	-4.17	3.45	-2.65	3.07	-2.36

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

7 - M O M E N T S REDUCED TO FACE-OF-SUPPORT

7.1 R E D U C E D DEAD LOAD MOMENTS (Tm)

SPAN	<- left* ->	<- midspan ->	<- right* ->
1	-198.90	212.80	-242.20
2	-87.43	10.02	-25.91
3	-38.44	23.31	-46.83
4	-47.17	23.63	-37.47
5	-23.34	2.40	1.76

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

	<----- left* ------>		<---- midspan ---->		<------ right* ----->	
SPAN	max	min	max	min	max	min
1	-73.85	3.35	79.15	-3.19	-94.50	-8.27
2	-41.65	-5.87	9.16	-3.78	-17.45	-.94
3	-21.84	-.72	12.71	-2.55	-25.46	-3.28
4	-25.89	-4.23	13.11	-1.78	-20.62	1.34
5	-12.73	-.72	4.17	-3.29	-1.52	2.25

Note:
* = face-of-support

8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

	<----- left* ------>		<---- midspan ---->		<------ right* ----->	
SPAN	max	min	max	min	max	min
1	-272.75	-195.55	291.95	209.61	-336.70	-250.47
2	-129.08	-93.30	19.18	6.24	-43.36	-26.85
3	-60.28	-39.16	36.02	20.76	-72.29	-50.10

4	-73.06	-51.40	36.74	21.85	-58.09	-36.13
5	-36.07	-24.06	6.56	-.89	.25	4.01

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

9.1 PROFILE TYPES AND PARAMETERS

LEGEND:

For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 TENDON PROFILE

	TYPE	X1/L	X2/L	X3/L	A/L
1	1	.100	.500	.100	.000
2	1	.100	.500	.100	.000
3	1	.100	.500	.100	.000
4	1	.100	.500	.100	.000
5	1	.100	.500	.100	.000

9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Tendon editing mode selected: TENDON SELECTION

	<-------- SELECTED VALUES -------->				<--- CALCULATED VALUES --->		
SPAN	FORCE (T/-)	<- DISTANCE OF CGS (cm) -> Left	Center	Right	P/A (Kg/cm^2)	Wbal (T/-)	Wbal (%DL)
1	970.161	-26.01	-59.40	-4.20	25.20	10.611	89
2	277.774	-4.20	-19.40	-4.20	12.35	4.170	47
3	284.553	-4.20	-18.00	-4.20	12.65	3.878	46
4	304.232	-4.20	-18.00	-4.20	13.52	4.147	49
5	295.443	-4.20	-11.50	-12.50	13.13	3.677	42

Approximate weight of strand ..... 1711.1 Kg

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9.35 - TENDON SELECTION DATA:

TYPE	SEL.	FORCE (T)	TENDON EXTENTS <1>	<2>	<3>	<4>	<5>
A	23	12.77	<==========				==========>
B	50	12.99	<=====				

9.5 R E Q U I R E D MINIMUM P O S T - T E N S I O N I N G FORCES (T)

SPAN	<- BASED ON STRESS CONDITIONS -> LEFT*	CENTER	RIGHT*	<- BASED ON MINIMUM P/A -> LEFT	CENTER	RIGHT
1	385.46	647.94	429.45	385.00	385.00	385.00
2	.00	.00	.00	385.00	225.00	265.00
3	98.17	88.82	169.09	265.00	225.00	265.00
4	171.22	93.67	92.44	265.00	225.00	265.00
5	.00	.00	.00	265.00	265.00	265.00

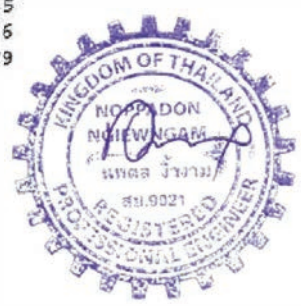
Note:
* = face-of-support

9.6 S E R V I C E S T R E S S E S (Kg/cm^2) (tension shown positive)

	L E F T * TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C	R I G H T * TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C
1	-----	-18.99	-----	-54.23	-----	-28.29	-----	-42.15
2	-----	-44.87	8.92	-1.66	-----	-12.39	-----	-22.05
3	9.82	-4.42	-----	-37.09	16.05	-----	-----	-46.43
4	15.29	-----	-----	-46.04	7.80	-7.01	-----	-37.06
5	1.57	-6.52	-----	-28.31	-----	-17.13	-----	-6.46

Note:
* = face-of-support

	C E N T E R TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C
1	-----	-43.07	1.59	-22.77
2	-----	-16.54	-----	-21.96
3	-----	-35.92	10.63	-5.65
4	-----	-35.56	8.52	-7.36
5	-----	-15.42	-----	-18.79



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9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

SPAN	<-- S P A N left*	M O M E N T S (Tm midspan	) --> right*	<-- SPAN SHEARS (T) --> SH(l)	SH(r)
1	170.40	-201.40	274.10	6.15	6.15
2	202.40	-15.26	30.02	.58	.58
3	30.19	-14.20	32.21	.00	.00
4	33.71	-16.08	29.31	.58	.58
5	16.90	-4.42	4.97	-2.18	-2.18

Note:
* = face-of-support

joint	<--REACTIONS (T)-->	<-- COLUMN MOMENTS (Tm) --> Lower columns	Upper columns
1	-6.152	101.700	76.310
2	5.569	-39.760	-29.850
3	.580	.129	.115
4	-.575	.965	.859
5	2.761	-5.512	-4.904
6	-2.182	1.302	1.159

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

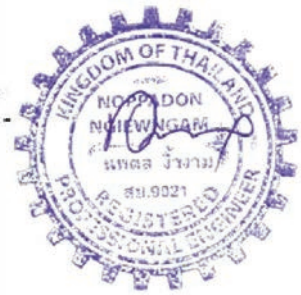
10.1 FACTORED DESIGN MOMENTS (Tm)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
1	-228.90	-97.67	555.12	415.14	-429.50	-282.91
2	-195.82	-135.00	24.65	2.65	-73.24	-45.17
3	-97.81	-61.90	51.33	25.39	-107.79	-70.08
4	-106.98	-70.16	56.11	30.79	-89.10	-51.76
5	-65.55	-45.13	3.07	-9.60	-3.62	2.79

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (Tm)

SPAN	<-- left* -->	<- midspan ->	<-- right* -->
1	175.10	122.60	70.23
2	-2.62	-4.96	-7.30



3	-6.87	-2.91	1.05
4	3.07	.74	-1.59
5	-11.23	-7.37	-3.51

Note:
* = face-of-support

10.3 FACTORED REACTIONS (T) — 10.4 FACTORED COLUMN MOMENTS (Tm)

JOINT	10.3 max	10.3 min	10.4 <-- LOWER column --> max	10.4 <-- LOWER column --> min	10.4 <-- UPPER column --> max	10.4 <-- UPPER column --> min
1	209.24	138.97	-91.73	-190.87	-68.90	-143.33
2	342.71	266.07	196.95	77.40	147.81	58.08
3	164.41	120.36	6.74	-35.35	5.99	-31.46
4	181.89	142.75	21.67	-20.81	19.28	-18.52
5	151.76	106.52	24.69	1.78	21.97	1.59
6	33.15	8.32	9.55	-.81	8.50	-.72

11 - M I L D S T E E L

Support cut-off length for minimum steel(length/span) ... .17
Span cut-off length for minimum steel(length/span) ... .33
Top bar extension beyond where required ..... 30.00 cm
Bottom bar extension beyond where required ..... 30.00 cm

REINFORCEMENT based on NO REDISTRIBUTION of factored moments

11.1 TOTAL WEIGHT OF REBAR = 72.6 Kg AVERAGE = .2 Kg/m^2
TOTAL AREA COVERED = 445.50 m^2

11.2.1 S T E E L A T M I D - S P A N

	TOP					BOTTOM				
SPAN	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS			As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS		
1	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
2	36.7	(36.7	.0	.0)		.0	(.0	.0	.0)	
3	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
4	.0	(.0	.0	.0)		.0	(.0	.0	.0)	
5	.0	(.0	.0	.0)		.0	(.0	.0	.0)	

11.3.1 S T E E L A T S U P P O R T S

	TOP			BOTTOM		
JOINT	As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----MIN--------->		As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----MIN--------->	

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--1--	------2------	---3---	---4---	---5---	------6------	---7---	---8---	---9---
1	.0 (	.0	.0	.0)	.0 (	.0	.0	.0)
2	.0 (	.0	.0	.0)	.0 (	.0	.0	.0)
3	.0 (	.0	.0	.0)	.0 (	.0	.0	.0)
4	3.6 (	3.6	.0	.0)	.0 (	.0	.0	.0)
5	.0 (	.0	.0	.0)	.0 (	.0	.0	.0)
6	.0 (	.0	.0	.0)	.0 (	.0	.0	.0)

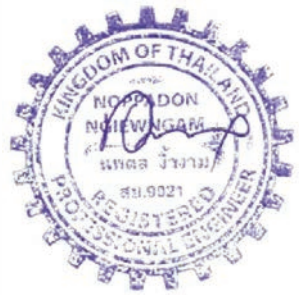
11.2.2 & 11.3.2 LISTING OF THE ENTIRE PROVIDED REBAR

SPAN	ID	LOCATION	NUM	BAR	LENGTH [cm]	AREA [cm^2]
3	1	T	4 #	5 x	60	8.0

Notes:
Bar location - T = Top, B = Bottom.
NUM - Number of bars.
For two-way systems a minimum of 4 bars is specified over the supports.
Refer to tables 11.5.1,11.5.2 and PTsum graphical display for
positioning of bars.

11.5.1 ARRANGEMENT OF TOP BARS

SPAN	TOP STEEL ID	LOCATION	NUM	BAR	LENGTH [cm]
3	1	RIGHT	4 #	5 x	60
4	1	LEFT	2 #	5 x	30



11.5.2 ARRANGEMENT OF BOTTOM BARS

SPAN	BOTTOM STEEL ID	LOCATION	NUM	BAR	LENGTH [cm]

12 - P U N C H I N G S H E A R C H E C K

LEGEND:
CONDITION... 1 = INTERIOR COLUMN
2 = END COLUMN
3 = CORNER COLUMN
4 = EDGE COLUMN (PARALLEL TO SPAN)
5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
PERFORM SHEAR CHECK MANUALLY
6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

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CASE..... 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)

JNT	COND.	FACTORED ACTIONS shear T	moment T-m	<- PUNCHING SHEAR STRESSES IN Kg/cm^2 -> due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
1	2	209.24	334.20	8.29	7.40	15.70	17.13	.92	1
2	1	342.71	344.76	8.86	6.55	15.41	21.39	.72	1
3	1	164.41	66.81	10.27	3.87	14.14	18.18	.78	1
4	1	181.89	40.95	11.36	2.37	13.73	18.41	.75	1
5	1	151.76	46.66	9.48	2.70	12.18	18.41	.66	1
6	2	33.15	18.05	3.01	.75	3.76	17.13	.22	1

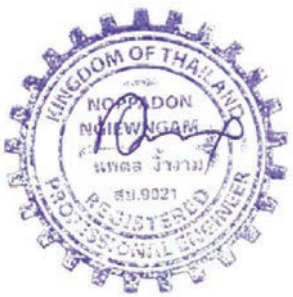
PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

SPAN	<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....> DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
1	1.5	.3	.8(2393)	.5(3323)	1.3(1391)
2	-.1	-.3	-.9(966)	.1(13746)	-.9(1039)
3	.3	.2	.5(1935)	.2(4854)	.7(1383)
4	.4	.1	.4(2505)	.2(4563)	.6(1617)
5	.0	.0	.0(19286)	.0(56447)	.0(14375)



```
|                          ADAPT CORPORATION                              |
|                  STRUCTURAL CONCRETE SOFTWARE SYSTEM                    |
|       1733 Woodside Road, Suite 220, Redwood City, California 94061     |
|                ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN             |
|                Version 7.10  AMERICAN (ACI 318-99/UBC-77)               |
|           ADAPT CORPORATION - Structural Concrete Software System       |
|      ** 1733 Woodside Road, Suite 220, Redwood City, California 94061   |
|                  Phone: (650)306-2400,  Fax: (650)364-4678              |
|      Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com   |
DATE AND TIME OF PROGRAM EXECUTION:              Jan 26,2021   At Time: 9:31
PROJECT FILE:                                                   FL.7 GL. DH

P R O J E C T   T I T L E:
GCDT
FL.7 GL .DH

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
=============================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............    350.00 Kg/cm^2
                     for COLUMNS .................    350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............    275.00 T/cm^2
                      for COLUMNS .................    238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....      2.00
CONCRETE WEIGHT ..................................     NORMAL

SELF WEIGHT ......................................   2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top  ..........................................      1.590
At Bottom  .......................................      1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................       .450

REINFORCEMENT:
YIELD Strength ...................................      4.00 T/cm^2
Minimum Cover at TOP .............................      2.00 cm
Minimum Cover at BOTTOM ..........................      2.00 cm

POST-TENSIONING:
SYSTEM ...........................................      BONDED
Ultimate strength of strand ......................     19.00 T/cm^2
Average effective stress in strand (final) .......     10.80 T/cm^2
Strand area.......................................       .990 cm^2
Min CGS of tendon from TOP........................      4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..     5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..     5.60 cm
```

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Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ....(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

SPAN	FORM	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
1	1	18.00	100.00	25.00					.00	4.50	4.50
2	1	9.00	100.00	25.00					.00	4.50	4.50
3	1	9.00	100.00	25.00					.00	4.50	4.50
4	1	9.00	100.00	25.00					.00	4.50	4.50
5	1	4.50	100.00	25.00					.00	4.50	4.50

LEGEND:
1 - SPAN
C = Cantilever

3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 - Top surface to reference line

2.1.5 - D R O P C A P A N D D R O P P A N E L D A T A

JOINT	CAPT cm	CAPB cm	CAPDL cm	CAPDR cm	DROPTL cm	DROPTR cm	DROPB cm	DROPL cm	DROPR cm
1	.00	.00	.00	.00	.00	65.00	400.00	.00	900.00
2	.00	.00	.00	.00	65.00	65.00	400.00	900.00	150.00
3	.00	.00	.00	.00	35.00	35.00	400.00	150.00	150.00
4	.00	.00	.00	.00	35.00	35.00	400.00	150.00	150.00
5	.00	.00	.00	.00	35.00	35.00	400.00	150.00	150.00
6	.00	.00	.00	.00	35.00	.00	400.00	150.00	.00

LEGEND:
DROP CAP DIMENSIONS: DROP PANEL DIMENSIONS:

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CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint

DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint

2.2 - SUPPORT WIDTH AND COLUMN DATA

JOINT	SUPPORT WIDTH cm	LOWER COLUMN LENGTH m	LOWER COLUMN B(DIA) cm	LOWER COLUMN D cm	LOWER COLUMN CBC*	UPPER COLUMN LENGTH m	UPPER COLUMN B(DIA) cm	UPPER COLUMN D cm	UPPER COLUMN CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
3	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
4	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
5	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
6	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

3 - INPUT APPLIED LOADING

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<----------TYPE---------->
U = UNIFORM
C = CONCENTRATED
Li= LINE LOAD
P = PARTIAL UNIFORM
M = APPLIED MOMENT

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	From ... m	To m	M or C ...At) T-m or T ...m	Total on Trib T/m
1	L	U	.500	.00	9.00		4.500
1	L	U	.500	9.00	18.00		4.500
1	D	U	.300	.00	9.00		2.700
1	D	U	.300	9.00	18.00		2.700

1	SW	P		.00	9.00	9.240
1	SW	P		9.00	18.00	9.240
2	L	U	.500	.00	1.50	4.500
2	L	U	.500	1.50	7.50	4.500
2	L	U	.500	7.50	9.00	4.500
2	D	U	.300	.00	1.50	2.700
2	D	U	.300	1.50	7.50	2.700
2	D	U	.300	7.50	9.00	2.700
2	SW	P		.00	1.50	9.240
2	SW	P		1.50	7.50	5.400
2	SW	P		7.50	9.00	6.360
3	L	U	.500	.00	1.50	4.500
3	L	U	.500	1.50	7.50	4.500
3	L	U	.500	7.50	9.00	4.500
3	D	U	.300	.00	1.50	2.700
3	D	U	.300	1.50	7.50	2.700
3	D	U	.300	7.50	9.00	2.700
3	SW	P		.00	1.50	6.360
3	SW	P		1.50	7.50	5.400
3	SW	P		7.50	9.00	6.360
4	L	U	.500	.00	1.50	4.500
4	L	U	.500	1.50	7.50	4.500
4	L	U	.500	7.50	9.00	4.500
4	D	U	.300	.00	1.50	2.700
4	D	U	.300	1.50	7.50	2.700
4	D	U	.300	7.50	9.00	2.700
4	SW	P		.00	1.50	6.360
4	SW	P		1.50	7.50	5.400
4	SW	P		7.50	9.00	6.360
5	L	U	.500	.00	1.50	4.500
5	L	U	.500	1.50	3.00	4.500
5	L	U	.500	3.00	4.50	4.500
5	D	U	.300	.00	1.50	2.700
5	D	U	.300	1.50	3.00	2.700
5	D	U	.300	3.00	4.50	2.700
5	SW	P		.00	1.50	6.360
5	SW	P		1.50	3.00	5.400
5	SW	P		3.00	4.50	6.360

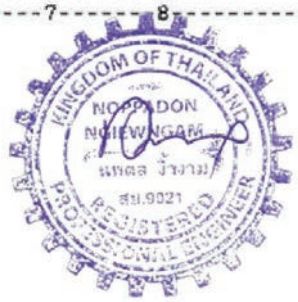
NOTE: LIVE LOADING is SKIPPED with a skip factor of .75

3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING

	UNIFORM (T/m^2),	(CON. or PART.)	(M O M E N T)

SPAN	CLASS	TYPE	LINE(T/m)	(T@m or m-m)	(T-m @ m)
1	L	U	.500		
1	D	U	.300		
2	L	U	.500		
2	D	U	.300		
3	L	U	.500		
3	D	U	.300		
4	L	U	.500		
4	D	U	.300		
5	L	U	.500		
5	D	U	.300		

NOTE: SELFWEIGHT INCLUSION REQUIRED
LIVE LOADING is SKIPPED with a skip factor of .75



4 - CALCULATED SECTION PROPERTIES

4.2 - Computed Section Properties for Segments of Nonprismatic Spans

Section properties are listed for all segments of each span
A= cross-sectional geometry Yt= centroidal distance to top fiber
I= gross moment of inertia Yb= centroidal distance to bottom fiber

SPAN (SEGMENT)	AREA cm^2	I cm^4	Yb cm	Yt cm
SPAN 1				
1	38500.00	.1318E+08	38.99	26.01
2	38500.00	.1318E+08	38.99	26.01
SPAN 2				
1	38500.00	.1318E+08	38.99	26.01
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14
SPAN 3				
1	26500.00	.2245E+07	19.86	15.14
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14
SPAN 4				
1	26500.00	.2245E+07	19.86	15.14
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14
SPAN 5				
1	26500.00	.2245E+07	19.86	15.14
2	22500.00	.1172E+07	12.50	12.50
3	26500.00	.2245E+07	19.86	15.14

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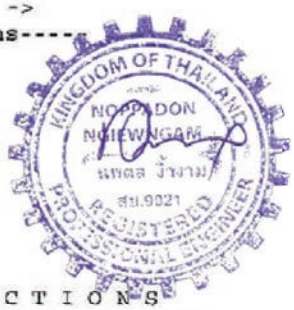
5 - DEAD LOAD MOMENTS, SHEARS & REACTIONS

SPAN	< 5.1 SPAN MOMENTS (Tm) > M(1)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (T) > SH(1)	SH(r)
1	-247.87	212.80	-293.66	-104.92	110.00
2	-109.90	10.02	-39.47	-49.68	30.42
3	-55.08	23.31	-64.47	-36.85	38.93
4	-64.89	23.63	-54.03	-39.10	36.68
5	-35.14	2.40	-3.23	-26.76	12.57

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) -> Lower columns	Upper columns
1	104.92	-141.59	-106.28
2	159.68	104.97	78.80
3	67.27	-8.26	-7.35
4	78.03	-.22	-.20
5	63.44	10.00	8.89
6	12.57	1.71	1.52



6 - LIVE LOAD MOMENTS, SHEARS & REACTIONS

<-- 6.1 LIVE LOAD SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

SPAN	<----- left* -----> max	min	<--- midspan ---> max	min	<---- right* -----> max	min	<--SHEAR FORCE--> left	right
1	-92.19	3.72	79.15	-3.19	-114.00	-8.59	-39.29	41.71
2	-52.34	-6.12	9.16	-3.78	-24.43	-.62	-23.35	17.15
3	-30.83	-.54	12.71	-2.55	-34.88	-3.36	-19.80	20.70
4	-35.40	-4.54	13.11	-1.78	-29.50	1.72	-20.91	19.59
5	-18.94	-3.87	4.17	-3.29	-4.88	3.75	-14.00	7.82

Note:
* = Centerline moments

JOINT	<- 6.2 REACTIONS (T) -> max	min	<--------- 6.3 COLUMN MOMENTS (Tm) --------> <--- LOWER COLUMN ---> max	min	<--- UPPER COLUMN ---> max	min
1	40.31	-1.02	2.83	-55.49	2.12	-41.66
2	66.80	21.72	52.77	-17.55	39.61	-13.17
3	40.97	15.06	10.69	-14.07	9.51	-12.52

4	43.07	20.05	12.36	-12.63	11.00	-11.24
5	35.40	8.79	9.53	-3.94	8.48	-3.51
6	10.43	-4.17	3.45	-2.65	3.07	-2.36

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

7 - M O M E N T S REDUCED TO FACE-OF-SUPPORT

7.1 R E D U C E D DEAD LOAD MOMENTS (Tm)

SPAN	<- left* ->	<- midspan ->	<- right* ->
1	-198.90	212.80	-242.20
2	-87.43	10.02	-25.91
3	-38.44	23.31	-46.83
4	-47.17	23.63	-37.47
5	-23.34	2.40	1.76

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
1	-73.85	3.35	79.15	-3.19	-94.50	-8.27
2	-41.65	-5.87	9.16	-3.78	-17.45	-.94
3	-21.84	-.72	12.71	-2.55	-25.46	-3.28
4	-25.89	-4.23	13.11	-1.78	-20.62	1.34
5	-12.73	-.72	4.17	-3.29	-1.52	2.25

Note:
* = face-of-support

8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
1	-272.75	-195.55	291.95	209.61	-336.70	-250.47
2	-129.08	-93.30	19.18	6.24	-43.36	-26.85
3	-60.28	-39.16	36.02	20.76	-72.29	-50.10

4	-73.06	-51.40	36.74	21.85	-58.09	-36.13
5	-36.07	-24.06	6.56	-.89	.25	4.01

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

9.1 PROFILE TYPES AND PARAMETERS

LEGEND:
For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

9.2 TENDON PROFILE

	TYPE	X1/L	X2/L	X3/L	A/L
1	1	.100	.500	.100	.000
2	1	.100	.500	.100	.000
3	1	.100	.500	.100	.000
4	1	.100	.500	.100	.000
5	1	.100	.500	.100	.000

9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Tendon editing mode selected: TENDON SELECTION

	<-------- SELECTED VALUES -------->			<--- CALCULATED VALUES --->		
SPAN	FORCE (T/-)	<- DISTANCE OF CGS (cm) -> Left	Center	Right	P/A (Kg/cm^2)	Wbal (T/-)	Wbal (%DL)
1	994.226	-26.01	-59.40	-4.20	25.82	10.874	91
2	304.921	-4.20	-14.40	-4.20	13.55	3.072	35
3	308.582	-4.20	-18.00	-4.20	13.71	4.206	50
4	330.015	-4.20	-18.00	-4.20	14.67	4.498	53
5	320.714	-4.20	-11.50	-12.50	14.25	3.991	46

Approximate weight of strand ..... 1789.6 Kg

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9.35 - TENDON SELECTION DATA:

TYPE	SEL. FORCE (T)	TENDON EXTENTS <1>	<2>	<3>	<4>	<5>
A	25	12.77	<==========			
B	50	12.97	<=====			

9.5 R E Q U I R E D MINIMUM P O S T - T E N S I O N I N G FORCES (T)

SPAN	<- BASED ON STRESS CONDITIONS -> LEFT*	CENTER	RIGHT*	<- BASED ON MINIMUM P/A -> LEFT	CENTER	RIGHT
1	384.21	645.26	433.79	385.00	385.00	385.00
2	.00	.00	.00	385.00	225.00	265.00
3	101.64	87.39	166.64	265.00	225.00	265.00
4	170.48	93.84	92.61	265.00	225.00	265.00
5	.00	.00	.00	265.00	265.00	265.00

Note:
* = face-of-support

9.6 S E R V I C E S T R E S S E S (Kg/cm^2) (tension shown positive)

	L E F T * TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C	R I G H T * TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C
1	-----	-20.71	-----	-53.24	-----	-29.51	-----	-41.85
2	-----	-44.20	6.36	-4.22	.45	-10.68	-----	-26.68
3	8.34	-5.91	-----	-37.20	12.78	-2.19	-----	-44.27
4	12.27	-2.34	-----	-44.27	5.21	-9.60	-----	-35.93
5	-----	-8.43	-----	-28.05	-----	-18.37	-----	-7.02

Note:
* = face-of-support

	C E N T E R TOP max-T	TOP max-C	BOTTOM max-T	BOTTOM max-C
1	-----	-42.46	-----	-25.23
2	-----	-21.46	-----	-19.44
3	-----	-35.22	7.79	-8.49
4	-----	-35.32	5.98	-9.90
5	-----	-16.13	-----	-20.33

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9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

SPAN	<-- S P A N M O M E N T S (Tm) --> left*	midspan	right*	<-- SPAN SHEARS (T) --> SH(l)	SH(r)
1	175.90	-207.60	277.20	6.60	6.60
2	195.80	-11.77	25.95	-.05	-.05
3	31.07	-15.86	35.70	-.30	-.30
4	36.79	-17.38	31.70	.67	.67
5	18.29	-4.81	5.40	-2.38	-2.38

Note:
* = face-of-support

joint	<--REACTIONS (T)-->	<-- COLUMN MOMENTS (Tm) --> Lower columns	Upper columns
1	-6.599	105.000	78.810
2	6.644	-45.790	-34.370
3	.253	2.953	2.628
4	-.964	.693	.617
5	3.049	-5.944	-5.290
6	-2.383	1.404	1.250

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

10.1 FACTORED DESIGN MOMENTS (Tm)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
1	-223.40	-92.17	556.90	416.92	-431.53	-284.94
2	-208.32	-147.50	14.68	-7.32	-80.69	-52.62
3	-99.47	-63.56	50.94	25.00	-106.93	-69.21
4	-106.52	-69.70	56.22	30.90	-89.34	-52.00
5	-66.55	-46.13	2.42	-10.25	-3.91	2.50

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (Tm)

SPAN	<-- left* -->	<- midspan ->	<-- right* -->
1	180.60	124.40	68.20
2	-15.11	-14.93	-14.75

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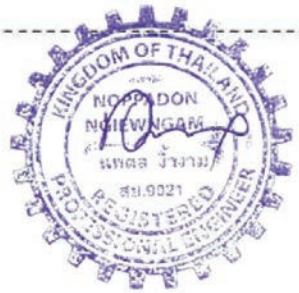
3	-8.52	-3.30	1.92
4	3.53	.85	-1.82
5	-12.23	-8.02	-3.80

Note:
* = face-of-support



JOINT	10.3 FACTORED REACTIONS (T) max	min	10.4 FACTORED COLUMN MOMENTS (Tm) <-- LOWER column --> max	min	<-- UPPER column --> max	min
1	208.79	138.52	-88.43	-187.57	-66.40	-140.83
2	343.78	267.15	190.92	71.38	143.29	53.56
3	164.08	120.03	9.56	-32.53	8.51	-28.95
4	181.50	142.36	21.40	-21.09	19.04	-18.76
5	152.04	106.80	24.26	1.35	21.58	1.20
6	32.95	8.12	9.65	-.71	8.59	-.63

11 - M I L D S T E E L

Support cut-off length for minimum steel(length/span) ... .17
Span cut-off length for minimum steel(length/span) ... .33
Top bar extension beyond where required ..... 30.00 cm
Bottom bar extension beyond where required ..... 30.00 cm

REINFORCEMENT based on NO REDISTRIBUTION of factored moments

11.1 TOTAL WEIGHT OF REBAR = 45.5 Kg AVERAGE = .1 Kg/m^2
TOTAL AREA COVERED = 445.50 m^2

11.2.1 S T E E L A T M I D - S P A N

SPAN	TOP As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----TENS--------->			BOTTOM As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----TENS--------->		
1	.0	(.0	.0	.0)	.0	(.0	.0	.0)
2	28.7	(28.7	.0	.0)	.0	(.0	.0	.0)
3	.0	(.0	.0	.0)	.0	(.0	.0	.0)
4	.0	(.0	.0	.0)	.0	(.0	.0	.0)
5	.0	(.0	.0	.0)	.0	(.0	.0	.0)

11.3.1 S T E E L A T S U P P O R T S

JOINT	TOP As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----MIN--------->	BOTTOM As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT-----MIN--------->

```
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-------------------------------------------------------------------------------

--1------2---------3-------4-------5-----------6---------7-------8-------9----
  1       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)
  2       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)
  3       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)
  4       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)
  5       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)
  6       .0  (     .0      .0       .0)         .0  (     .0      .0       .0)

11.2.2 & 11.3.2      LISTING OF THE ENTIRE PROVIDED REBAR

-------------------------------------------------------
SPAN  ID LOCATION  NUM  BAR  LENGTH [cm] AREA [cm^2]
--1----2-----3------4----5-------6---------7----------
Notes:
Bar location - T = Top, B = Bottom.
NUM - Number of bars.
For two-way systems a minimum of 4 bars is specified over the supports.
Refer to tables 11.5.1,11.5.2 and PTsum graphical display for
positioning of bars.


11.5.1      ARRANGEMENT OF TOP BARS

-------|----------- TOP STEEL -----------------|
SPAN   | ID   LOCATION | NUM  BAR   LENGTH [cm]|
--1----|--2------3-----|---4----5------6-------|


11.5.2      ARRANGEMENT OF BOTTOM BARS

-------|-------- BOTTOM STEEL -----------------|
SPAN   | ID   LOCATION | NUM  BAR   LENGTH [cm]|
--1----|--2------3-----|---4----5------6-------|


12 -  P U N C H I N G   S H E A R   C H E C K
===============================================================================

LEGEND:
CONDITION... 1 = INTERIOR COLUMN
             2 = END COLUMN
             3 = CORNER COLUMN
             4 = EDGE COLUMN (PARALLEL TO SPAN)
             5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
                 PERFORM SHEAR CHECK MANUALLY
             6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

CASE........ 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
             2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)


          FACTORED ACTIONS   <- PUNCHING SHEAR STRESSES IN Kg/cm^2 ->
           shear   moment    due to   due to              allow-   STRESS
```

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JNT	COND.	T	T-m	shear	moment	TOTAL	able	RATIO	CASE
1	2	208.79	328.40	8.28	7.24	15.51	17.13	.91	1
2	1	343.78	334.21	8.89	6.35	15.24	21.55	.71	1
3	1	164.08	61.48	10.25	3.56	13.81	18.46	.75	1
4	1	181.50	40.44	11.34	2.34	13.68	18.70	.73	1
5	1	152.05	45.84	9.50	2.66	12.15	18.70	.65	1
6	2	32.95	18.24	2.99	.77	3.76	17.13	.22	1

PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....>

SPAN	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
1	1.5	.2	.6(2822)	.5(3323)	1.2(1526)
2	-.1	-.3	-.8(1155)	.1(13746)	-.7(1261)
3	.3	.1	.4(2337)	.2(4854)	.6(1577)
4	.4	.1	.3(2677)	.2(4563)	.5(1687)
5	.0	.0	.0(20545)	.0(56447)	.0(15063)

```
------------------------------------------------------------------------------
|                              ADAPT CORPORATION                             |
|                      STRUCTURAL CONCRETE SOFTWARE SYSTEM                   |
|           1733 Woodside Road, Suite 220, Redwood City, California 94061    |
------------------------------------------------------------------------------
|                  ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN              |
|                  Version 7.10  AMERICAN (ACI 318-99/UBC-77)                |
|              ADAPT CORPORATION - Structural Concrete Software System       |
|           1733 Woodside Road, Suite 220, Redwood City, California 94061    |
|                    Phone: (650)306-2400,  Fax: (650)364-4678               |
|         Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com   |
------------------------------------------------------------------------------
DATE AND TIME OF PROGRAM EXECUTION:                Jan 26,2021   At Time: 9:40
PROJECT FILE:                                              FL.2 GL. D6 (DF-DD)


P R O J E C T   T I T L E:
GCDT
FL.2 GL. D6 (DF-DD)

1 - USER SPECIFIED   G E N E R A L   D E S I G N   P A R A M E T E R S
==============================================================================

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .............     350.00 Kg/cm^2
                     for COLUMNS .................     350.00 Kg/cm^2

MODULUS OF ELASTICITY for BEAMS/SLABS .............     275.00 T/cm^2
                      for COLUMNS .................     238.00 T/cm^2

CREEP factor for deflections for BEAMS/SLABS .....       2.00
CONCRETE WEIGHT ..................................      NORMAL

SELF WEIGHT ......................................    2400.00 Kg/m^3


TENSION STRESS limits (multiple of (f'c)1/2)
At Top ...........................................       1.590
At Bottom ........................................       1.590

COMPRESSION STRESS limits (multiple of (f'c))
At all locations .................................        .450

REINFORCEMENT:
YIELD Strength ...................................       4.00 T/cm^2
Minimum Cover at TOP .............................       2.00 cm
Minimum Cover at BOTTOM ..........................       2.00 cm

POST-TENSIONING:
SYSTEM ...........................................      BONDED
Ultimate strength of strand ......................      19.00 T/cm^2
Average effective stress in strand (final) .......      10.80 T/cm^2
Strand area.......................................       .990 cm^2
Min CGS of tendon from TOP........................       4.20 cm
Min CGS of tendon from BOTTOM for INTERIOR spans..      5.60 cm
Min CGS of tendon from BOTTOM for EXTERIOR spans..      5.60 cm
```

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Min average precompression ..... 10.00 Kg/cm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths..... (see section 9)

ANALYSIS OPTIONS USED:
Structural system ....(using EQUIVALENT FRAME).... TWO-WAY
Moments REDUCED to face of support ..... YES

2 - I N P U T G E O M E T R Y

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

SPAN	FORM	LENGTH m	WIDTH cm	DEPTH cm	TOP FLANGE width cm	TOP FLANGE thick. cm	BOTTOM/MIDDLE FLANGE width cm	BOTTOM/MIDDLE FLANGE thick. cm	REF HEIGHT cm	MULTIPLIER left	MULTIPLIER right
C	1	2.00	100.00	25.00					.00	4.50	4.50
1	1	9.00	100.00	25.00					.00	4.50	4.50
C	1	3.50	100.00	25.00					.00	4.50	4.50

LEGEND:
1 - SPAN
C = Cantilever

3 - FORM
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 - Top surface to reference line

2.1.5 - D R O P C A P A N D D R O P P A N E L D A T A

JOINT	CAPT cm	CAPB cm	CAPDL cm	CAPDR cm	DROPTL cm	DROPTR cm	DROPB cm	DROPL cm	DROPR cm
1	.00	.00	.00	.00	35.00	35.00	300.00	150.00	450.00
2	.00	.00	.00	.00	35.00	35.00	300.00	450.00	150.00

LEGEND:
DROP CAP DIMENSIONS:
CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint

DROP PANEL DIMENSIONS:
DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint

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2.2 - S U P P O R T W I D T H A N D C O L U M N D A T A

JOINT	SUPPORT WIDTH cm	LOWER COLUMN LENGTH m	LOWER COLUMN B(DIA) cm	LOWER COLUMN D cm	LOWER COLUMN CBC*	UPPER COLUMN LENGTH m	UPPER COLUMN B(DIA) cm	UPPER COLUMN D cm	UPPER COLUMN CBC*
1	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)
2	96.00	3.50	96.00	96.00	(1)	3.50	96.00	96.00	(1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ...(STANDARD) ..... = 1
Hinged at near end, fixed at far end ..... = 2
Fixed at near end, hinged at far end ..... = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

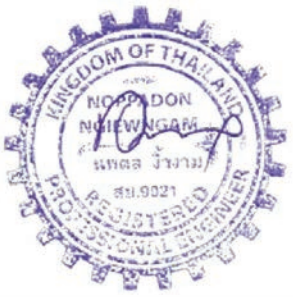
3 - I N P U T A P P L I E D L O A D I N G

<---CLASS--->
D = DEAD LOAD
L = LIVE LOAD

<----------TYPE---------->
U = UNIFORM P = PARTIAL UNIFORM
C = CONCENTRATED M = APPLIED MOMENT
Li= LINE LOAD

SW= SELF WEIGHT Computed from geometry input and treated as dead loading
Unit selfweight W = 2400.0 Kg/m^3

SPAN	CLASS	TYPE	Intensity T/m^2	From ... m	To m	M or C ...At) T-m or T ...m	Total on Trib T/m
CANT	L	U	.500	.00	1.50		4.500
CANT	L	U	.500	1.50	2.00		4.500
CANT	D	U	.300	.00	1.50		2.700
CANT	D	U	.300	1.50	2.00		2.700
CANT	SW	P		.00	1.50		6.120
CANT	SW	P		1.50	2.00		5.400
1	L	U	.500	.00	4.50		4.500
1	L	U	.500	4.50	9.00		4.500
1	D	U	.300	.00	4.50		2.700
1	D	U	.300	4.50	9.00		2.700
1	SW	P		.00	4.50		6.120
1	SW	P		4.50	9.00		6.120
CANT	L	U	.500	.00	1.50		4.500



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 ------------------------------------------------------------------------------

 CANT    L       U          .500      1.50      3.50                        4.500
 CANT    D       U          .300       .00      1.50                        2.700
 CANT    D       U          .300      1.50      3.50                        2.700
 CANT    SW      P                     .00      1.50                        6.120
 CANT    SW      P                    1.50      3.50          --            5.400

  NOTE: LIVE LOADING is SKIPPED with a skip factor of      .75


3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING
==============================================================================

                     UNIFORM
                     ( T/m^2),     ( CON. or PART. )     ( M O M E N T )
SPAN  CLASS  TYPE    LINE( T/m)    (  T@m or  m-m  )     (  T-m  @  m  )
-1-----2------3---------4------------5-------6-----------7-------8-----------
CANT    L       U          .500
CANT    D       U          .300
 1      L       U          .500
 1      D       U          .300
CANT    L       U          .500
CANT    D       U          .300
  NOTE: SELFWEIGHT INCLUSION REQUIRED
        LIVE LOADING is SKIPPED with a skip factor of      .75


4 - C A L C U L A T E D   S E C T I O N   P R O P E R T I E S
==============================================================================

4.2 - Computed Section Properties for Segments of Nonprismatic Spans
------------------------------------------------------------------------------
Section properties are listed for all segments of each span
A= cross-sectional geometry       Yt= centroidal distance to top fiber
I= gross moment of inertia        Yb= centroidal distance to bottom fiber

SPAN          AREA              I               Yb             Yt
(SEGMENT)     cm^2              cm^4            cm             cm
--------------2-----------------3---------------4--------------5-----
CANTILEVER LEFT
  1          25500.00         .2008E+07         20.44          14.56
  2          22500.00         .1172E+07         12.50          12.50
SPAN  1
  1          25500.00         .2008E+07         20.44          14.56
  2          25500.00         .2008E+07         20.44          14.56
CANTILEVER RIGHT
  1          25500.00         .2008E+07         20.44          14.56
  2          22500.00         .1172E+07         12.50          12.50
```

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5 - DEAD LOAD MOMENTS, SHEARS & REACTIONS

SPAN	< 5.1 SPAN MOMENTS (Tm) > M(1)*	Midspan	M(r)*	< 5.2 SPAN SHEARS (T) > SH(1)	SH(r)
CANT	-----	-----	-17.01	-----	17.28
1	-47.09	35.01	-61.50	-38.09	41.29
CANT	-50.42	-----	-----	-29.43	-----

Note:
* = Centerline moments

JOINT	< 5.3 REACTIONS (T) >	<- 5.4 COLUMN MOMENTS (Tm) -> Lower columns	Upper columns
1	55.37	-15.92	-14.16
2	70.72	5.86	5.21

6 - LIVE LOAD MOMENTS, SHEARS & REACTIONS

<-- 6.1 LIVE LOAD SPAN MOMENTS (Tm) and SHEAR FORCES (T) -->

SPAN	<----- left* -----> max	min	<--- midspan ---> max	min	<---- right* -----> max	min	<--SHEAR FORCE--> left	right
CL	-----		-----		-9.00		-----	9.00
1	-23.92	.24	17.64	-2.77	-31.92	-5.77	-19.36	21.14
CR	-27.56		-----		-----		-15.75	-----

Note:
* = Centerline moments

<- 6.2 REACTIONS (T) -> <-------- 6.3 COLUMN MOMENTS (Tm) -------->

JOINT	max	min	<--- LOWER COLUMN ---> max	min	<--- UPPER COLUMN ---> max	min
1	29.68	8.11	4.93	-12.82	4.39	-11.41
2	37.32	16.64	12.82	-10.51	11.41	-9.36

Note: Block 6.1 through 6.3 values are maxima of all skipped loading cases

7 - MOMENTS REDUCED TO FACE-OF-SUPPORT

7.1 REDUCED DEAD LOAD MOMENTS (Tm)

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SPAN	<- left* ->	<- midspan ->	<- right* ->
CANT	-----	-----	-9.73
1	-29.83	35.01	-42.69
CANT	-37.31	-----	-----

Note:
* = face-of-support

7.2 R E D U C E D LIVE LOAD MOMENTS (Tm)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-5.20	
1	-15.14	-.08	17.64	-2.77	-22.29	-5.45
CR	-20.52		-----		-----	

Note:
* = face-of-support

8 - SUM OF DEAD AND LIVE MOMENTS (Tm)

Maxima of dead load and live load span moments combined
for serviceability checks (1.00DL + 1.00LL)

SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-14.93	
1	-44.97	-29.91	52.65	32.24	-64.98	-48.14
CR	-57.83		-----		-----	

Note:
* = face-of-support

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES

9.1 PROFILE TYPES AND PARAMETERS
LEGEND:
For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

```
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 ------------------------------------------------------------------------------


  For Cantilever:
        1 = simple parabola
        2 = partial parabola
        3 = harped tendon


9.2      T E N D O N          P R O F I L E
         TYPE      X1/L       X2/L       X3/L       A/L
----------1--------2----------3----------4----------5-------
CANT      1                                .000
 1        1        .100       .500       .100       .000
CANT      1        .000


9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE
==============================================================================
Tendon editing mode selected: TENDON SELECTION

          <--------  SELECTED VALUES  -------->   <--- CALCULATED VALUES --->
            FORCE   <- DISTANCE OF CGS (cm) ->       P/A         Wbal      Wbal
 SPAN       (T/-)     Left     Center    Right     (Kg/cm^2)    (T/-)     (%DL)
--1----------2---------3--------4--------5-----------6----------7--------8--
 CANT     303.394    -12.50              -4.20        12.06     12.760      148
  1       325.415     -4.20    -29.40     -4.20        12.76      8.099       92
 CANT     309.389     -4.20              -12.50        14.12      4.306       51


Approximate weight of strand .....................        301.2 Kg


9.35 - TENDON SELECTION DATA:

TYPE  SEL. FORCE  <------------------ TENDON EXTENTS ----------------------->
           (T )     <C>   <1>   <C>
--1----2-----3---|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
  B   14   12.57    <==============|
  C   11   12.86    |==============>


9.5   R E Q U I R E D  MINIMUM  P O S T - T E N S I O N I N G  FORCES (T   )

          <- BASED ON STRESS CONDITIONS ->      <-  BASED ON MINIMUM P/A  ->
 SPAN        LEFT*     CENTER      RIGHT*          LEFT     CENTER      RIGHT
--1----------2----------3----------4---------------5---------6---------7----
 CANT        -----       -----        .00          -----      -----    255.00
  1         21.58      155.83     130.61         255.00     255.00    255.00
 CANT      122.27       -----      -----         255.00      -----     -----

Note:
   * = face-of-support
```

9.6 S E R V I C E S T R E S S E S (Kg/cm^2)(tension shown positive)

	L E F T *				R I G H T *			
	TOP		BOTTOM		TOP		BOTTOM	
	max-T	max-C	max-T	max-C	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----	-----	-20.16	-----	-5.68
1	-----	-20.04	-----	-16.81	4.66	-7.55	-----	-36.63
CR	10.72	-4.16	-----	-44.57	-----	-----	-----	-----

Note:
* = face-of-support

	C E N T E R			
	TOP		BOTTOM	
	max-T	max-C	max-T	max-C
CL	-----	-----	-----	-----
1	-----	-24.54	3.77	-17.01
CR	-----	-----	-----	-----

9.7 POST-TENSIONING B A L A N C E D M O M E N T S, SHEARS & REACTIONS

	<-- S P A N M O M E N T S (Tm) -->			<-- SPAN SHEARS (T) -->	
SPAN	left*	midspan	right*	SH(l)	SH(r)
CANT	-----	-----	21.07	-----	19.39
1	40.56	-36.41	41.30	-.09	-.09
CANT	26.12	-----	-----	12.43	-----

Note:
* = face-of-support

	<--REACTIONS (T)-->	<-- COLUMN MOMENTS (Tm) -->	
joint	2	Lower columns	Upper columns
1	.091	7.843	6.979
2	-.091	-7.262	-6.462

10 - F A C T O R E D M O M E N T S & R E A C T I O N S

Calculated as (1.40D + 1.70L + 1.00 secondary moment effects)

10.1 FACTORED DESIGN MOMENTS (Tm)

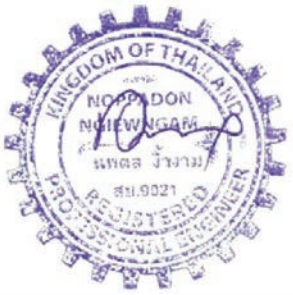
SPAN	<----- left* ------> max	min	<---- midspan ----> max	min	<----- right* -----> max	min
CL	-----		-----		-22.46	--
1	-52.74	-27.15	93.28	58.58	-83.88	-55.25
CR	-87.12		-----		-----	

Note:
* = face-of-support

10.2 SECONDARY MOMENTS (Tm)

SPAN	<-- left* -->	<- midspan ->	<-- right* -->
1	14.76	14.27	13.78

Note:
* = face-of-support



JOINT	10.3 FACTORED REACTIONS (T) max	min	10.4 FACTORED COLUMN MOMENTS (Tm) <-- LOWER column --> max	min	<-- UPPER column --> max	min
1	128.07	91.40	-6.07	-36.24	-5.39	-32.24
2	162.36	127.20	22.74	-16.92	20.24	-15.07

11 - M I L D S T E E L

Support cut-off length for minimum steel(length/span) ... .17
Span cut-off length for minimum steel(length/span) ... .33
Top bar extension beyond where required ..... 30.00 cm
Bottom bar extension beyond where required ..... 30.00 cm

REINFORCEMENT based on NO REDISTRIBUTION of factored moments

11.1 TOTAL WEIGHT OF REBAR = .0 Kg AVERAGE = .0 Kg/m^2
TOTAL AREA COVERED = 130.50 m^2

11.2.1 S T E E L A T M I D - S P A N

SPAN	TOP As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS--------->			BOTTOM As (cm^2)	DIFFERENT REBAR CRITERIA <---ULT	TENS--------->		
1	.0	(.0	.0	.0)		.0	(.0	.0	.0)	

```
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11.3.1     S T E E L    A T    S U P P O R T S
                      T O P                                   B O T T O M
          As     DIFFERENT REBAR CRITERIA          As     DIFFERENT REBAR CRITERIA
JOINT   (cm^2) <---ULT-----MIN--------->        (cm^2) <---ULT-----MIN--------->
--1------2---------3-------4-------5-----------6---------7-------8-------9----
  1        .0  (     .0       .0       .0)          .0  (     .0       .0       .0)
  2        .0  (     .0       .0       .0)          .0  (     .0       .0       .0)

11.2.2 & 11.3.2     LISTING OF THE ENTIRE PROVIDED REBAR

-----------------------------------------------------
SPAN  ID LOCATION  NUM  BAR  LENGTH [cm] AREA [cm^2]
--1----2-----3------4----5--------6----------7-----------
Notes:
Bar location - T = Top, B = Bottom.
NUM - Number of bars.
For two-way systems a minimum of 4 bars is specified over the supports.
Refer to tables 11.5.1,11.5.2 and PTsum graphical display for
positioning of bars.

11.5.1     ARRANGEMENT OF TOP BARS

-------|----------- TOP STEEL -----------------|
SPAN   | ID   LOCATION | NUM  BAR   LENGTH [cm]|
--1----|--2------3-----|---4----5------6-------|

11.5.2     ARRANGEMENT OF BOTTOM BARS

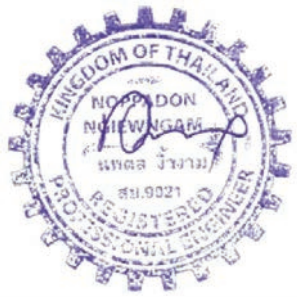
-------|-------- BOTTOM STEEL -----------------|
SPAN   | ID   LOCATION | NUM  BAR   LENGTH [cm]|
--1----|--2------3-----|---4----5------6-------|

12 -  P U N C H I N G    S H E A R    C H E C K
===============================================================================

LEGEND:
CONDITION... 1 = INTERIOR COLUMN
             2 = END COLUMN
             3 = CORNER COLUMN
             4 = EDGE COLUMN (PARALLEL TO SPAN)
             5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
                 PERFORM SHEAR CHECK MANUALLY
             6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

CASE........ 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
             2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)


          FACTORED ACTIONS    <- PUNCHING SHEAR STRESSES IN Kg/cm^2 ->
```



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JNT	COND.	shear T	moment T-m	due to shear	due to moment	TOTAL	allow-able	STRESS RATIO	CASE
1	1	128.07	68.48	8.00	3.97	11.97	18.21	.66	1
2	1	162.36	42.97	10.14	2.49	12.63	18.56	.68	1

PUNCHING SHEAR CHECK SATISFACTORY
NO ADDITIONAL REBAR OR CHANGE IN SECTION IS NECESSARY

13 - MAXIMUM S P A N D E F L E C T I O N S

Concrete`s modulus of elasticity ..... Ec = 275.00 T/cm^2
Creep factor ..... K = 2.00
Ieffective/Igross...(due to cracking)..... K = 1.00

Where stresses exceed 1.616(fc`)^1/2 cracking of section is allowed for.
Values in parentheses are (span/max deflection) ratios

<.....DEFLECTION ARE ALL IN cm , DOWNWARD POSITIVE.....>

SPAN	DL	DL+PT	DL+PT+CREEP	LL	DL+PT+LL+CREEP
CANL	-.1	-.2	-.5(422)	.0(4525)	-.5(386)
1	.4	.0	.1(10556)	.2(4920)	.3(3356)
CANR	.2	.1	.2(2171)	.1(2597)	.3(1182)

4. รายการคำนวณพื้นค.ส.ล.

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

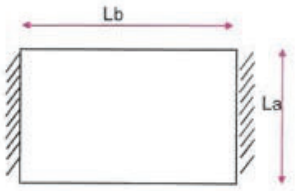
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **S1**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab S1**

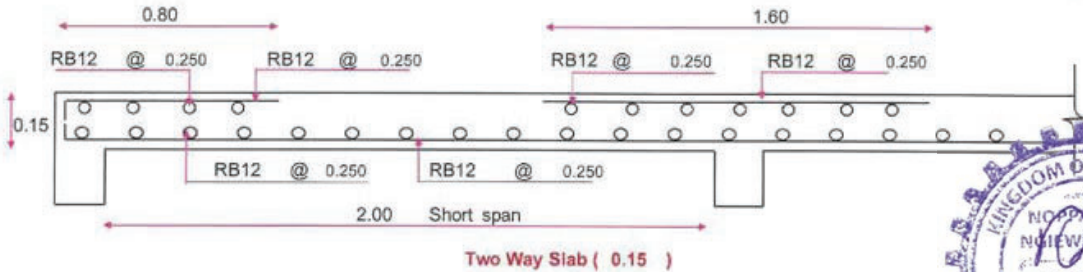
Concrete Strength	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy RD	=	2400	[kg/cm^2]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	200	[cm]
	Lb	=	400	[cm]
Live Load	LL	=	500	[kg/m^2]
Finishing Load	FL	=	120	[kg/m^2]
2 Siden Continuous	RB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	2.00	OK
La/Lb	m	=	0.500	
Shear	Vu S	=	1015	[kg/m]
	Vu L	=	1395	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L	<	Vc all	OK



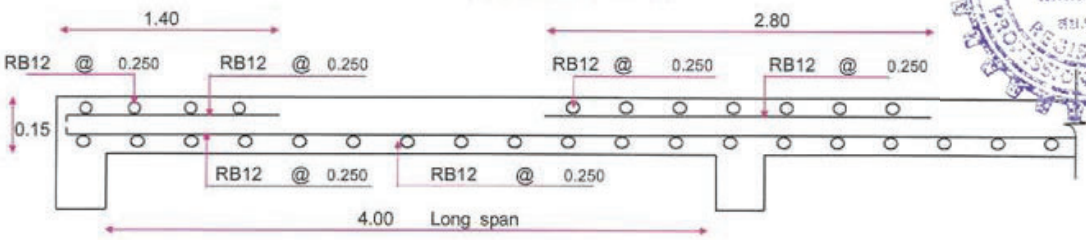
Check h	ρ	=	0.0712
	ρ_{max}	=	0.0534
Required d		=	2.4 [cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$		
	d	<	11.5 OK
	a try	=	0 [cm]
	As	=	2.124 [cm^2]
	a	=	0.1713 [cm]
Moment arm	j d	=	11.41 [cm]

Mu / ϕ fy j d

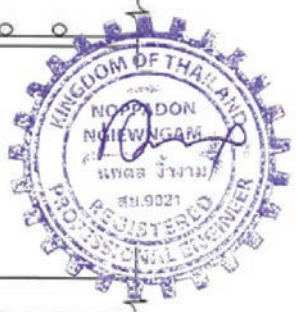
Short Span									
Moment	- Continue	Mu	= 524 [kg-m]	As1	= 2.12	[cm^2]	2 RB12	@ 0.250	OK
	- Not Cont.	Mu	= 274 [kg-m]	As2	= 1.11	[cm^2]	1 RB12	@ 0.250	OK
	+ Middle	Mu m	= 414 [kg-m]	As3	= 1.68	[cm^2]	2 RB12	@ 0.250	OK
Long Span									
Moment	- Continue	Mu	= 298 [kg-m]	As4	= 1.21	[cm^2]	2 RB12	@ 0.250	OK
	- Not Cont.	Mu	= 152 [kg-m]	As5	= 0.62	[cm^2]	1 RB12	@ 0.250	OK
	+ Middle	Mu m	= 225 [kg-m]	As6	= 0.91	[cm^2]	1 RB12	@ 0.250	OK
	Minimum Steel			As min	= 3.75	[cm^2]	4 RB12	@ 0.250	OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Ultimate Stress Design method

One way Slab Design-Case : Cantilever — Slab Code: S0

Project: สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม) — File Name: Slab S0

Engineer: Noppadon N. — Date: 7 Dec 20

Name No. S0 — Short Span ; S = 2.00 m. — Long Span ; L = 9.00 m. — Ratio ; m=S/L= 0.222 < 0.50 O.K.

Factor for Dead Load = 1.4 — Factor for Live Load = 1.7

support : beam size (bxh)

short : 0.30 x 0.60 m. — S_n = 1.70 m.

Long : 0.30 x 0.60 m. — L_n = 8.70 m.

use: fy = 4000 ksc.

minimum slab thickness = Sn/10 = 17.00 cm. — select **t** = 20 cm.

LOADS :

Self weight of slab	20 cm.	1.4x	480	= 672.00	kg/m².
Floor finish + others		1.4x	120	= 168.00	kg/m².
Live Loads		1.7x	400	= 680.00	kg/m².
Σ w_u				= **1,520.00**	kg/m².
$w_u S_n^2$				= 4,392.80	kg-m./1 m.

Materials Properties :

fy = 4000 ksc. — fc' = 350 ksc.

β_1 = 0.85-0.05(fc'-280)/70= 0.800 **use :** 0.800

ρ_b = $0.85\beta_1(fc'/fy)(6120/(6120+fy))$ = 0.0360

ρ = $0.50\rho_b$ = 0.0180

R_u = $\rho f_y(1-0.59\rho f_y/f_c')$ = 63.234 ksc.

	factor (C)
M - exterior face of **exterior beam** = (1/2)wS_n^2	-0.500

List	at exterior beam
Factor (C)	-0.500
M_u = Cw_uS^2 , kg-m./ 1 m.	-2,196.40
Main Reinf. :	
$d_{req.}$ = sqrt[$M_u/\phi R_u b$] ,cm.	6.21
d = thickness - covering - (dia/2) ,cm.	16.50
R_u = $M_u/\phi bd^2$ ksc.	8.964
$\rho_{req.}$ = 0.85(fc'/fy){1-sqrt[1-(2Ru/0.85fc')]}	0.0023
As = ρbd , cm^2/1 m.	3.755
As.req /1 m. (As,min = 3.600 cm^2./1 m.)	3.755
As,req. = (As,req./1 m.)x(span) , cm^2.	32.669
Reinforced dia.= 12 mm., required (amount)	28.886
max. spacing , m.	0.301
uses : spacing , m.	0.200
amount of reinf. =	44
Distribution Reinf. :	
As.req /1 m. (As,min = 3.600 cm^2./1 m.)	
As,req. = (As,req./1 m.)x(span) , cm^2.	6.120
Reinforced dia.= 12 mm., required (amount)	5.411
max. spacing , m.	0.314
uses : spacing , m.	0.250
amount of reinf. =	7

Shear at d from first interior beam edge :

$v_u@d$ = w_uS_n - w_ud = 1,235.00 kg/m.

check Shear :

ϕv_c = $\phi 0.53\sqrt{f_c'}100d$ = 13,906.34 kg/m. > $v_u@d$ O.K

Loads transfer to beam :

Long beam = [w_uS] = 1,520.00 kg/m. (DL. = 840.00 kg/m.)

(LL. = 680.00 kg/m.)

Torsion = $w_uS[S/2]$ = 3,040.00 kg-m./ 1 m.

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

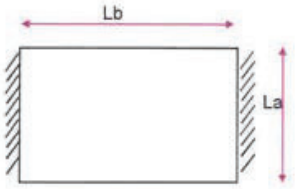
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **SX**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab SX**

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	20	[cm]
Clear span Length	La	=	330	[cm]
	Lb	=	250	[cm]
Live Load	LL	=	750	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
0 Siden Continuous	DB12			
	DL	=	480	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	16.5	[cm]
	Wu	=	2115	[kg/ m]
	Lb/La	=	0.76	OK
La/Lb	m	=	1.320	
Shear	Vu S	=	2327	[kg/m]
	Vu L	=	1463	[kg/m]
	Vc all	=	10957	[kg/m]
	Vu L	<	Vc all	OK



Check h	ρ	=	0.0360
	ρmax	=	0.0270
Required d		=	3.8 [cm]

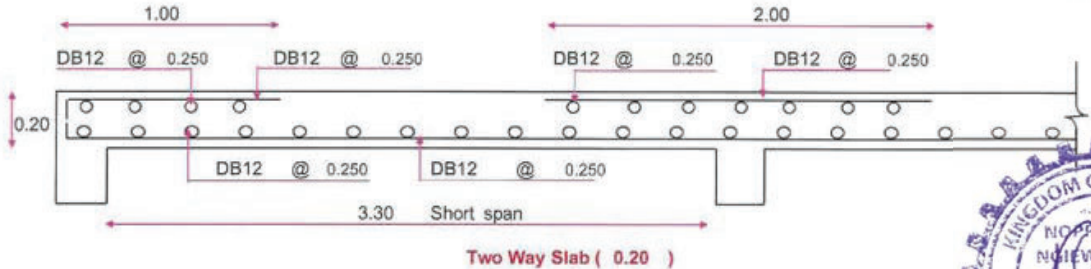
(Mu / ϕ ρ fy db² (1-0.59 ρ fy / fc'))^0.5

d < 16.5 OK

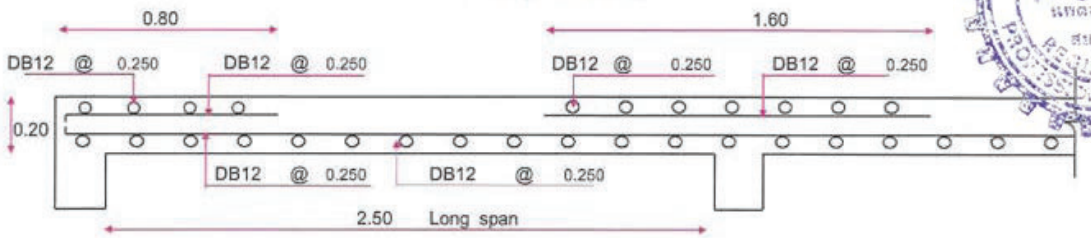
a try	=	0	[cm]
As	=	1.954	[cm²]
a	=	0.2628	[cm]
Moment arm j d	=	16.37	[cm]

Mu / ϕ fy j d

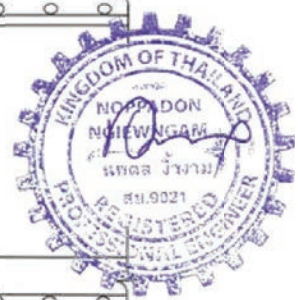
Short Span								
Moment - Continue	Mu	=	0 [kg-m]	As1 =	0.00	[cm²]	1 DB12 @ 0.250	OK
- Not Cont.	Mu	=	0 [kg-m]	As2 =	0.00	[cm²]	1 DB12 @ 0.250	OK
+ Middle	Mu m	=	0 [kg-m]	As3 =	0.00	[cm²]	1 DB12 @ 0.250	OK
Long Span								
Moment - Continue	Mu	=	0 [kg-m]	As4 =	0.00	[cm²]	1 DB12 @ 0.250	OK
- Not Cont.	Mu	=	760 [kg-m]	As5 =	1.29	[cm²]	2 DB12 @ 0.250	OK
+ Middle	Mu m	=	1152 [kg-m]	As6 =	1.95	[cm²]	2 DB12 @ 0.250	OK
Minimum Steel				As min =	4.00	[cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.20)



Two Way Slab (0.20)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

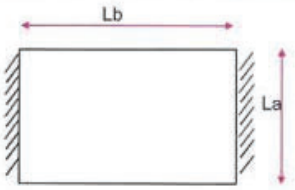
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 6S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 6S1

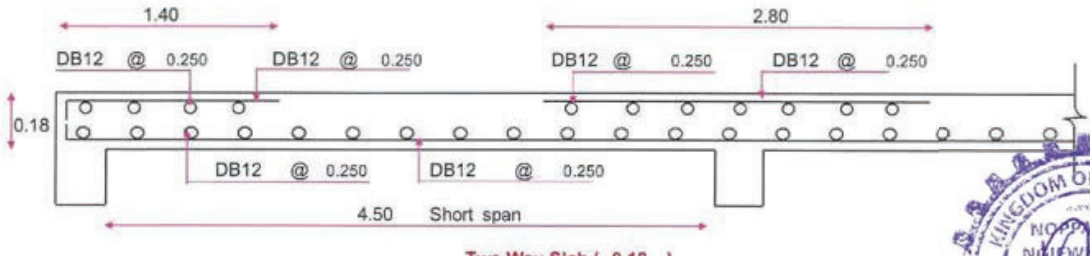
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	500	[kg/m²]
2 Siden Continuous / DB12	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	2138	[kg/ m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	3207	[kg/m]
	Vu L	=	3207	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L	<	Vc all	OK



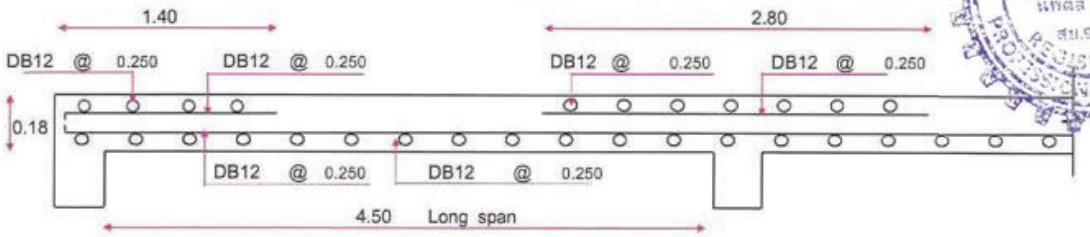
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	5.2	[cm]
	(Mu / ϕ ρ fy db² (1-0.59 ρ fy / fc'))^0.5			
	d	<	14.0	OK
	a try	=	1	[cm]
	As	=	4.298	[cm²]
	a	=	0.5779	[cm]
Moment arm	j d	=	13.71	[cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	0	[kg-m]	As1 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	1082	[kg-m]	As2 =	2.19	[cm²]	2 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1602	[kg-m]	As3 =	3.25	[cm²]	3 DB12 @ 0.250	OK
Long Span										
Moment	- Continue	Mu	=	2121	[kg-m]	As4 =	4.30	[cm²]	4 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	1082	[kg-m]	As5 =	2.19	[cm²]	2 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1602	[kg-m]	As6 =	3.25	[cm²]	3 DB12 @ 0.250	OK
	Minimum Steel					As min =	3.50	[cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 6S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

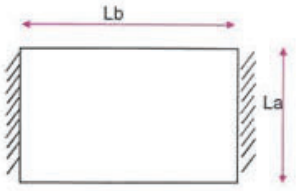
Filename : Slab 6S1

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	500	[kg/m²]

3 Siden Continuous | DB12

	DL	=	420	[kg/m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	2138	[kg/m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	3207	[kg/m]
	Vu L	=	3207	[kg/m]
	Vc all	=	8849	[kg/m]

Vu L < Vc all OK



Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	4.7 [cm]

$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$

d < 14.0 OK

a try		=	1 [cm]
As		=	3.596 [cm²]
a		=	0.4835 [cm]
Moment arm	j d	=	13.76 [cm]

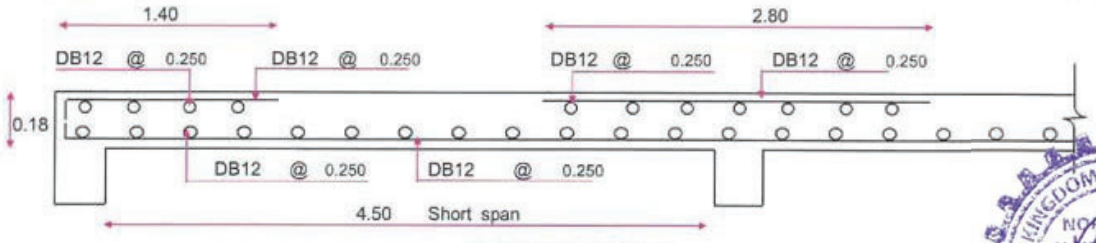
Short Span

Mu / ϕ fy j d

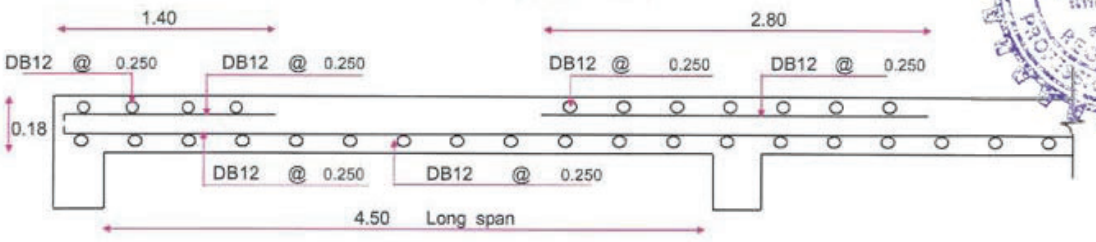
Moment	- Continue	Mu	=	0 [kg-m]	As1	=	0.00 [cm²]	1 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	909 [kg-m]	As2	=	1.84 [cm²]	2 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1342 [kg-m]	As3	=	2.71 [cm²]	3 DB12 @ 0.250	OK

Long Span

Moment	- Continue	Mu	=	1775 [kg-m]	As4	=	3.58 [cm²]	4 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	909 [kg-m]	As5	=	1.84 [cm²]	2 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1342 [kg-m]	As6	=	2.71 [cm²]	3 DB12 @ 0.250	OK
	Minimum Steel				As min	=	3.50 [cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

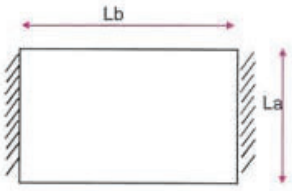
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 6S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 6S1

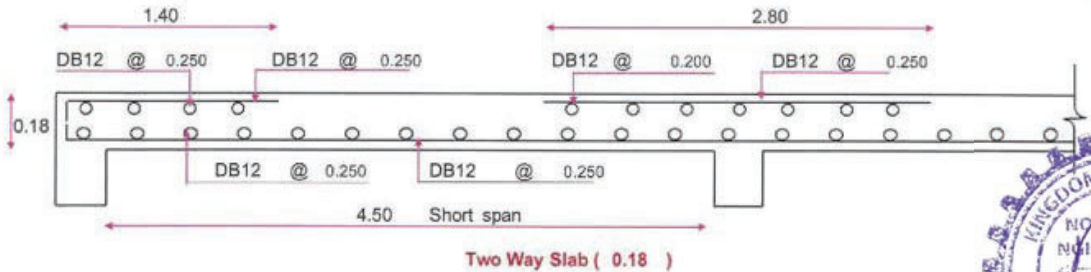
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	500	[kg/m²]
1 Siden Continuous / DB12	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	2138	[kg/ m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	3207	[kg/m]
	Vu L	=	3207	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L < Vc all			OK



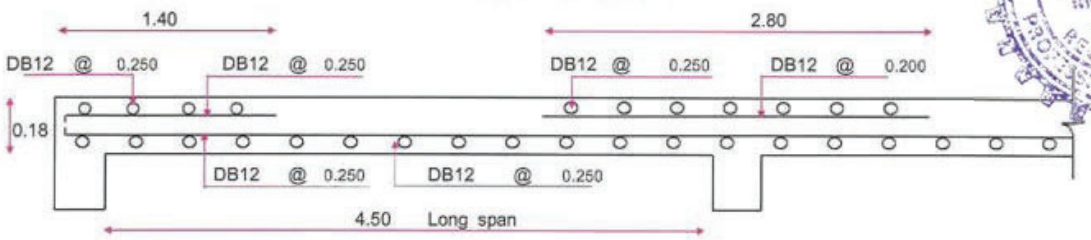
Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	5.6 [cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$		
	d	<	14.0 OK
	a try	=	1 [cm]
	As	=	5.087 [cm^2]
	a	=	0.6840 [cm]
Moment arm	j d	=	13.66 [cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	0 [kg-m]	As1	=	0.00 [cm^2]	1 DB12	@	0.250 OK
	- Not Cont.	Mu	=	1256 [kg-m]	As2	=	2.55 [cm^2]	3 DB12	@	0.250 OK
	+ Middle	Mu m	=	1905 [kg-m]	As3	=	3.87 [cm^2]	4 DB12	@	0.250 OK
Long Span										
Moment	- Continue	Mu	=	2511 [kg-m]	As4	=	5.11 [cm^2]	5 DB12	@	0.200 OK
	- Not Cont.	Mu	=	1256 [kg-m]	As5	=	2.55 [cm^2]	3 DB12	@	0.250 OK
	+ Middle	Mu m	=	1905 [kg-m]	As6	=	3.87 [cm^2]	4 DB12	@	0.250 OK
	Minimum Steel				As min	=	3.50 [cm^2]	4 DB12	@	0.250 OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

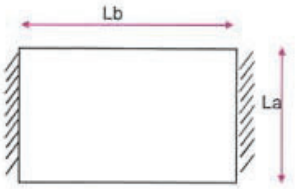
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 6S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 6S1

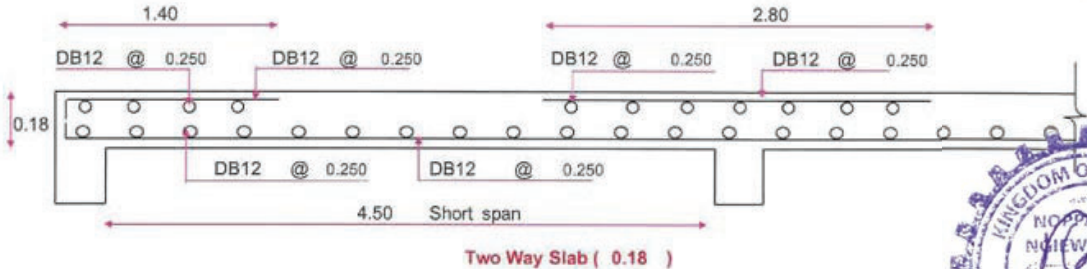
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	500	[kg/m²]
4 Siden Continuous	DB12			
	DL	=	420	[kg/m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	2138	[kg/m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	3207	[kg/m]
	Vu L	=	3207	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L < Vc all			OK



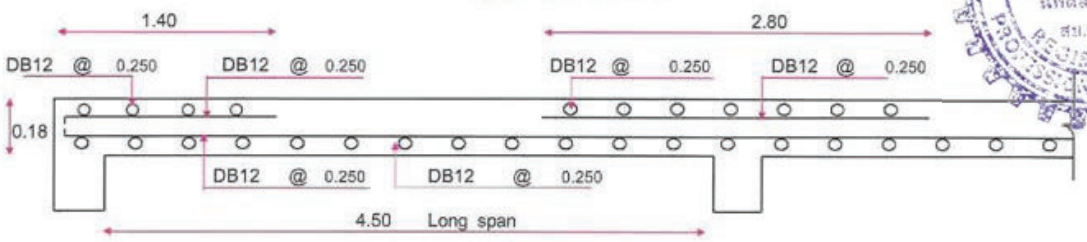
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	4.6	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	14.0	OK
	a try	=	1	[cm]
	As	=	3.333	[cm²]
	a	=	0.4481	[cm]
Moment arm	j d	=	13.78	[cm]

$Mu / \phi fy j d$

Short Span										
Moment	- Continue	Mu	=	1429	[kg-m]	As1 =	2.88	[cm²]	3 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	0	[kg-m]	As2 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1082	[kg-m]	As3 =	2.18	[cm²]	2 DB12 @ 0.250	OK
Long Span										
Moment	- Continue	Mu	=	1645	[kg-m]	As4 =	3.32	[cm²]	3 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	0	[kg-m]	As5 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1082	[kg-m]	As6 =	2.18	[cm²]	2 DB12 @ 0.250	OK
	Minimum Steel					As min =	3.50	[cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

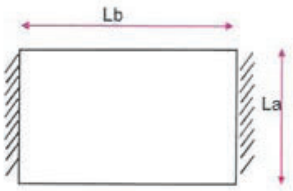
Job No. : 193018
Rev. : 1
Initial : Noppadon N.
Date : 7-ธ.ค.-20

Slab No. 6S2

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

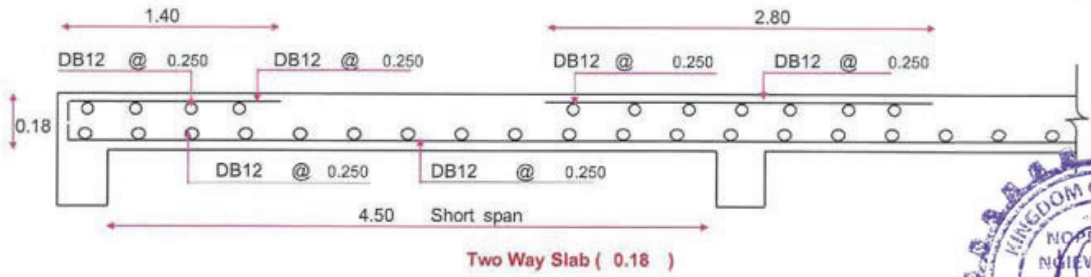
Filename : Slab 6S2

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	900	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	500	[kg/m²]
0 Siden Continuous / DB12	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	2138	[kg/ m]
	Lb/La	=	2.00	OK
La/Lb	m	=	0.500	
Shear	Vu S	=	3207	[kg/m]
	Vu L	=	4410	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L < Vc all			OK

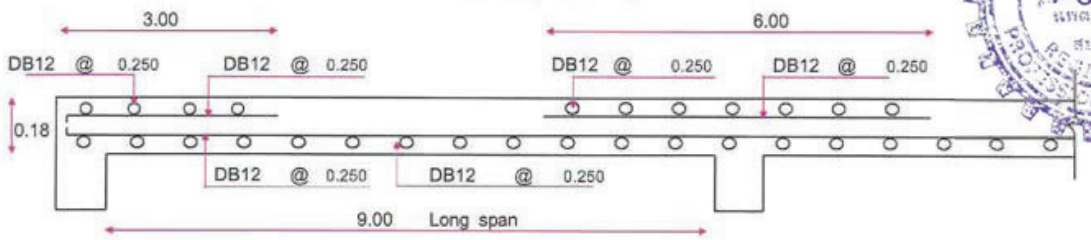


Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	5.2 [cm]
	(Mu / ϕ ρ fy db² (1-0.59 ρ fy / fc'))^0.5		
	d	<	14.0 OK
	a try	=	1 [cm]
	As	=	4.388 [cm²]
	a	=	0.5899 [cm]
Moment arm	j d	=	13.71 [cm]

					Mu / ϕ fy j d						
Short Span											
Moment	- Continue	Mu	=	0 [kg-m]	As1	=	0.00	[cm²]	1 DB12	@	0.250 OK
	- Not Cont.	Mu	=	0 [kg-m]	As2	=	0.00	[cm²]	1 DB12	@	0.250 OK
	+ Middle	Mu m	=	0 [kg-m]	As3	=	0.00	[cm²]	1 DB12	@	0.250 OK
Long Span											
Moment	- Continue	Mu	=	0 [kg-m]	As4	=	0.00	[cm²]	1 DB12	@	0.250 OK
	- Not Cont.	Mu	=	1429 [kg-m]	As5	=	2.90	[cm²]	3 DB12	@	0.250 OK
	+ Middle	Mu m	=	2165 [kg-m]	As6	=	4.39	[cm²]	4 DB12	@	0.250 OK
	Minimum Steel				As min	=	3.50	[cm²]	4 DB12	@	0.250 OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

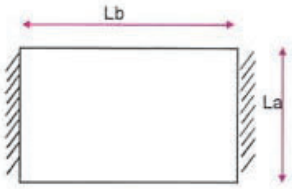
Job No.	:	193018	
Rev.	:	1	
Initial	:	Noppadon N.	
Date	:	17-มี.ค.-21	

Slab No. 6S3

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 6S3

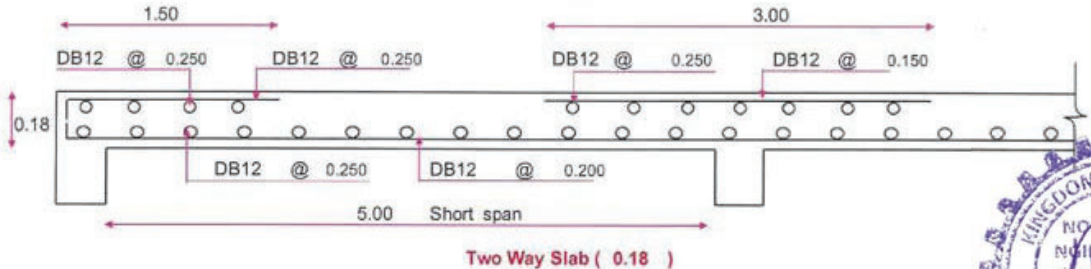
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	500	[cm]
	Lb	=	900	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous	DB12			
	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	1438	[kg/ m]
	Lb/La	=	1.80	OK
La/Lb	m	=	0.556	
Shear	Vu S	=	2397	[kg/m]
	Vu L	=	3225	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L < Vc all			OK



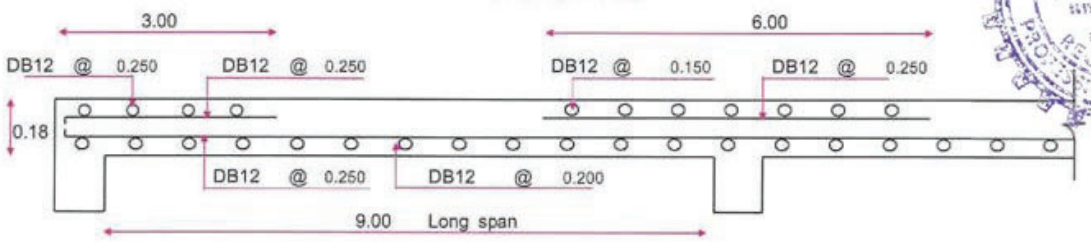
Check h	ρ	=	0.0360
	ρmax	=	0.0270
Required d		=	6.1 [cm]
	(Mu / ϕ ρ fy db² (1-0.59 ρ fy / fc'))^0.5		
	d	<	14.0 OK
	a try	=	1 [cm]
	As	=	6.023 [cm²]
	a	=	0.8098 [cm]
Moment arm	j d	=	13.60 [cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	2948	[kg-m]	As1 =	6.02	[cm²]	6 DB12 @ 0.150	OK
	- Not Cont.	Mu	=	1546	[kg-m]	As2 =	3.16	[cm²]	3 DB12 @ 0.250	OK
	+ Middle	Mu m	=	2337	[kg-m]	As3 =	4.77	[cm²]	5 DB12 @ 0.200	OK
Long Span										
Moment	- Continue	Mu	=	1762	[kg-m]	As4 =	3.60	[cm²]	4 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	899	[kg-m]	As5 =	1.84	[cm²]	2 DB12 @ 0.250	OK
	+ Middle	Mu m	=	1330	[kg-m]	As6 =	2.72	[cm²]	3 DB12 @ 0.250	OK
	Minimum Steel					As min =	3.50	[cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.18)



Two Way Slab (0.18)

GCDT
GOVERNMENT CENTER DESIGN TEAM

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

ONE WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

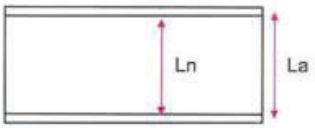
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	17-มี.ค.-21

Slab No. 6S4

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

filename : Slab 6S4

Concrete Strength	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy RD	=	2400	[kg/cm^2]
Clear span Length	Ln	=	300	[cm]
Load	LL	=	500	[kg/ m]
Type of Slab			Simple Support	
Reinforcement Steel			DB12	



	DL		360	[kg/ m]
Thick of Slab	h	=	15	[cm]
	Wu	=	1354	[kg/ m]
Moment	- Mu L	=	508	[kg-m]
	+ Mu m	=	1523	[kg-m]
	- Mu R	=	508	[kg-m]
Shear	Vur	=	2336	[kg]
	K	=	0.80	
	ϕ	=	0.90	
	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
	d	=	4.4	[cm] ($Mu / \phi \rho fy db^2 (1-0.59 r fy / fc'))^{0.5}$
	d	<	h	OK

$Mu = 1/24\ Wu\ Ln^2$ $Mu = 1/9\ Wu\ Ln^2$

$Mu = 1/14\ Wu\ Ln^2$ $Mu = 1/14\ Wu\ Ln^2$

$Vur = 1.15 Wu\ Ln / 2$

Sheck Shear

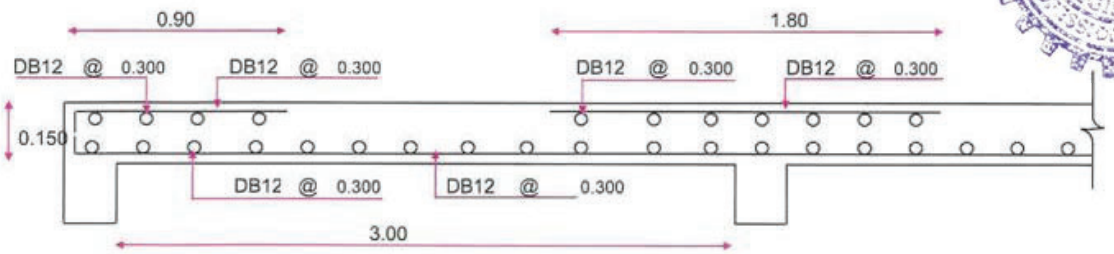
Existing Unit Shear	vc	=	2.03	[kg/cm^2]
Allowable Shear unit	v all	=	8.92	[kg/cm^2]
	vc	<	v all	OK
	a try	=	0.5056	[cm]
	As	=	3.762	[cm^2]
	a	=	0.5058	[cm]
	j d	=	14.75	[cm]

Check Deflection

Δ all	=	0.83	[cm]	
ρ fy	=	8.0	< 35	[kg/cm^2]
I	=	28125	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	0.18	[cm] <	Δ all OK
LL = 40 % and Cofficient =		2		
Δ Time	=	0.15	<	Δ all OK

Moment arm $Mu / \phi fy j$

- Mu L	As L	=	0.96	[cm^2]	1 DB12	@ 0.300	OK
+ Mu m	As m	=	2.87	[cm^2]	3 DB12	@ 0.300	OK
- Mu R	As R	=	0.96	[cm^2]	1 DB12	@ 0.300	OK
Minmum Steel	As min	=	3.00	[cm^2]	3 DB12	@ 0.300	OK



One Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

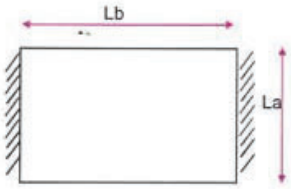
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	17-มี.ค.-21

Slab No. 6S4

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 6S4

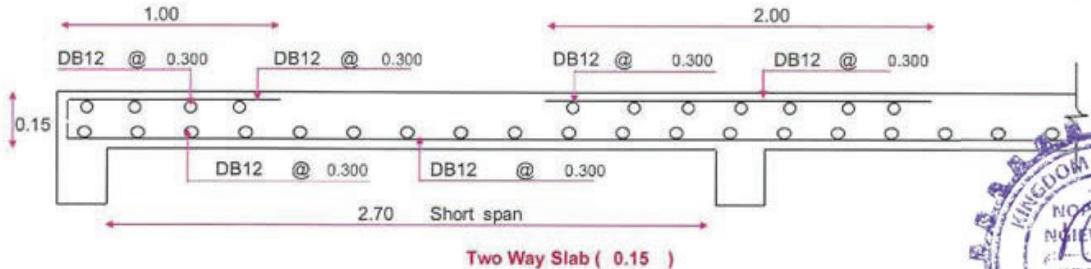
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	270	[cm]
	Lb	=	400	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous / DB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1354	[kg/ m]
	Lb/La	=	1.48	OK
La/Lb	m	=	0.675	
Shear	Vu S	=	1219	[kg/m]
	Vu L	=	1550	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



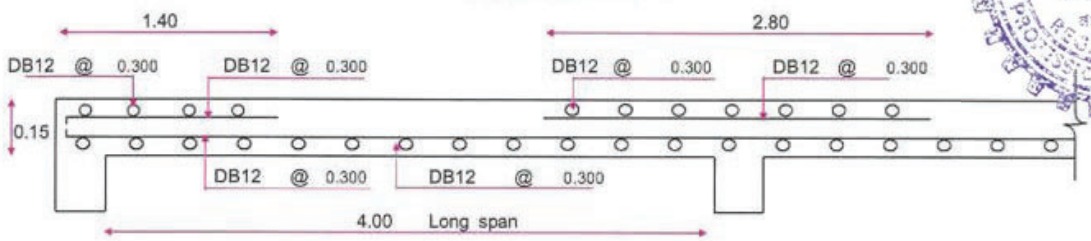
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	3.0	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	11.5	OK
	a try	=	0	[cm]
	As	=	1.710	[cm²]
	a	=	0.2299	[cm]
Moment arm	j d	=	11.39	[cm]

$Mu / \phi fy j d$

Short Span										
Moment	- Continue	Mu	=	701	[kg-m]	As1 =	1.71	[cm²]	2 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	365	[kg-m]	As2 =	0.89	[cm²]	1 DB12 @ 0.300	OK
	+ Middle	Mu m	=	549	[kg-m]	As3 =	1.34	[cm²]	2 DB12 @ 0.300	OK
Long Span										
Moment	- Continue	Mu	=	484	[kg-m]	As4 =	1.18	[cm²]	2 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	247	[kg-m]	As5 =	0.60	[cm²]	1 DB12 @ 0.300	OK
	+ Middle	Mu m	=	365	[kg-m]	As6 =	0.89	[cm²]	1 DB12 @ 0.300	OK
	Minimum Steel					As min =	3.00	[cm²]	3 DB12 @ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

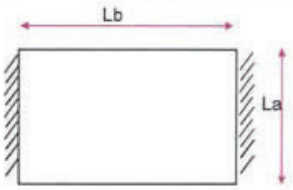
Slab No. 8S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S1

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	2283	[kg/m]
	Vu L	=	2283	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L	<	Vc all	OK

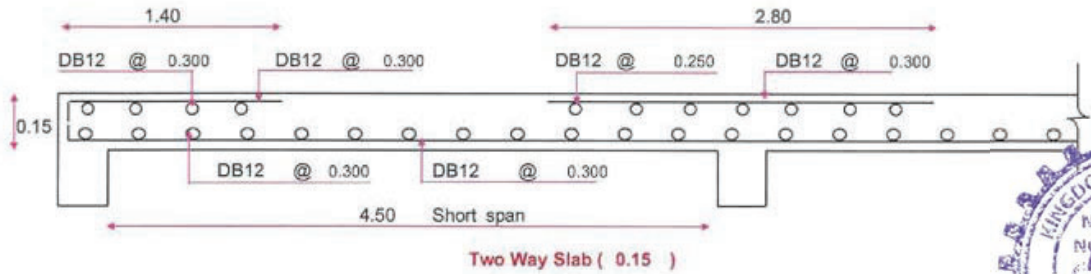
1 Siden Continuous | DB12



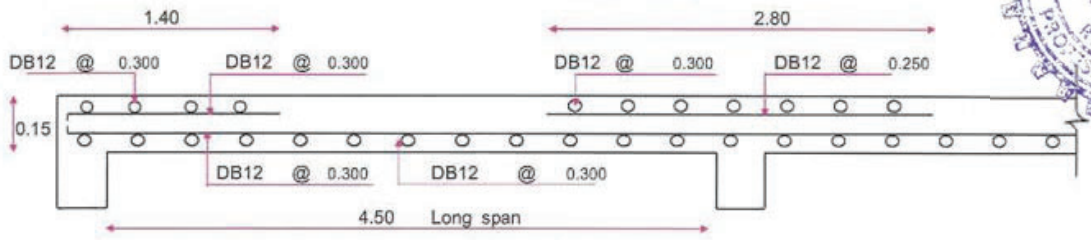
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	4.7	[cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$			
	d	<	11.5	OK
	a try	=	1	[cm]
	As	=	4.433	[cm^2]
	a	=	0.5960	[cm]
Moment arm	j d	=	11.20	[cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	0	[kg-m]	As1 =	0.00	[cm^2]	1 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	894	[kg-m]	As2 =	2.22	[cm^2]	2 DB12 @ 0.300	OK
	+ Middle	Mu m	=	1356	[kg-m]	As3 =	3.36	[cm^2]	3 DB12 @ 0.300	OK
Long Span										
Moment	- Continue	Mu	=	1788	[kg-m]	As4 =	4.43	[cm^2]	4 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	894	[kg-m]	As5 =	2.22	[cm^2]	2 DB12 @ 0.300	OK
	+ Middle	Mu m	=	1356	[kg-m]	As6 =	3.36	[cm^2]	3 DB12 @ 0.300	OK
	Minimum Steel					As min =	3.00	[cm^2]	3 DB12 @ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

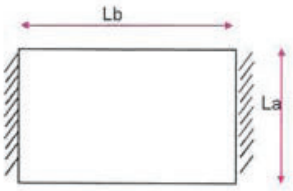
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

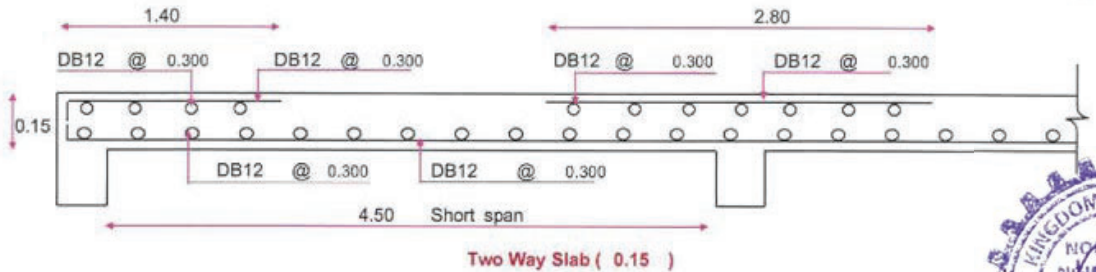
Filename : Slab 8S1

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
3 Siden Continuous	DB12			
	DL	=	360	[kg/m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	2283	[kg/m]
	Vu L	=	2283	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK

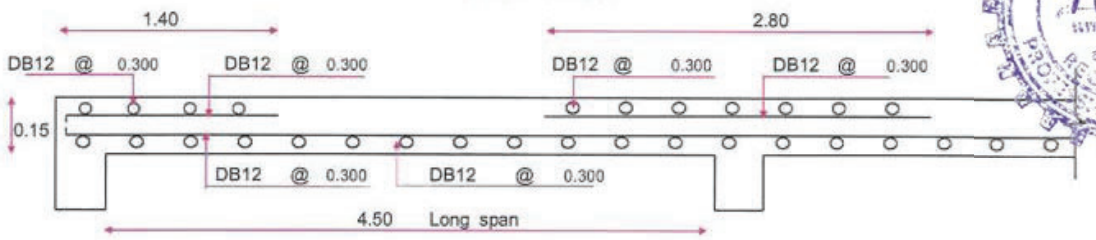


Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	4.0 [cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$		
	d	<	11.5 OK
	a try	=	1 [cm]
	As	=	3.133 [cm²]
	a	=	0.4213 [cm]
Moment arm	j d	=	11.29 [cm]

					Mu / ϕ fy j d					
Short Span										
Moment	- Continue	Mu	=	0 [kg-m]	As1 =	0.00	[cm²]	1 DB12	@	0.300 OK
	- Not Cont.	Mu	=	647 [kg-m]	As2 =	1.59	[cm²]	2 DB12	@	0.300 OK
	+ Middle	Mu m	=	955 [kg-m]	As3 =	2.35	[cm²]	3 DB12	@	0.300 OK
Long Span										
Moment	- Continue	Mu	=	1264 [kg-m]	As4 =	3.11	[cm²]	3 DB12	@	0.300 OK
	- Not Cont.	Mu	=	647 [kg-m]	As5 =	1.59	[cm²]	2 DB12	@	0.300 OK
	+ Middle	Mu m	=	955 [kg-m]	As6 =	2.35	[cm²]	3 DB12	@	0.300 OK
	Minimum Steel				As min =	3.00	[cm²]	3 DB12	@	0.300 OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

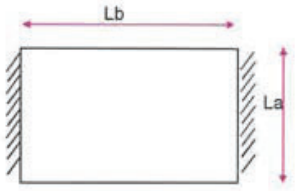
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S1

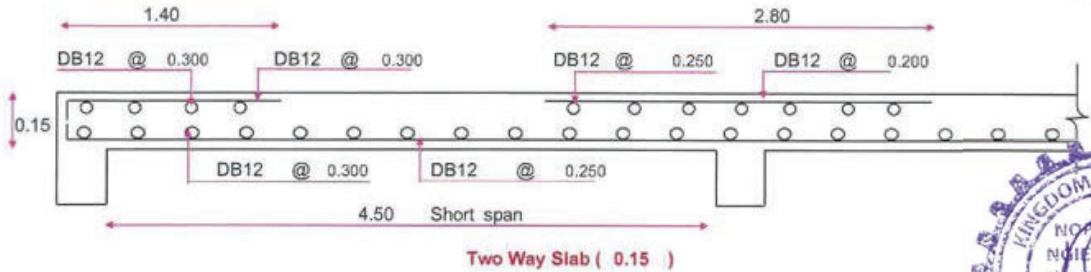
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	600	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
2 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	1.33	OK
La/Lb	m	=	0.750	
Shear	Vu S	=	2283	[kg/m]
	Vu L	=	2782	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L	<	Vc all	OK



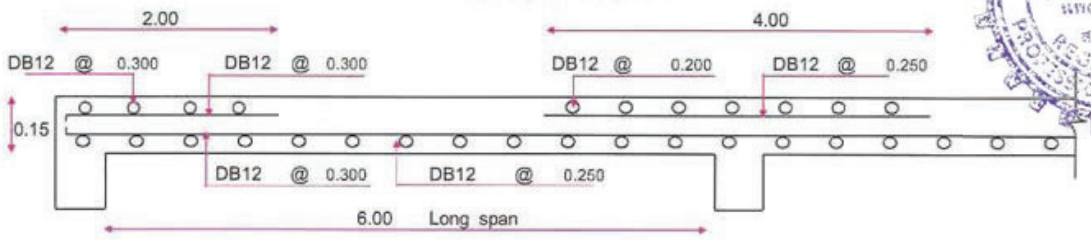
Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	5.1 [cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$		
	d	<	11.5 OK
	a try	=	1 [cm]
	As	=	5.090 [cm^2]
	a	=	0.6843 [cm]
Moment arm	j d	=	11.16 [cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	= 2044	[kg-m]	As1 =	5.09	[cm^2]	5 DB12	@ 0.200	OK
	- Not Cont.	Mu	= 1068	[kg-m]	As2 =	2.66	[cm^2]	3 DB12	@ 0.300	OK
	+ Middle	Mu m	= 1603	[kg-m]	As3 =	3.99	[cm^2]	4 DB12	@ 0.250	OK
Long Span										
Moment	- Continue	Mu	= 1510	[kg-m]	As4 =	3.76	[cm^2]	4 DB12	@ 0.250	OK
	- Not Cont.	Mu	= 771	[kg-m]	As5 =	1.92	[cm^2]	2 DB12	@ 0.300	OK
	+ Middle	Mu m	= 1140	[kg-m]	As6 =	2.84	[cm^2]	3 DB12	@ 0.300	OK
	Minimum Steel				As min =	3.00	[cm^2]	3 DB12	@ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

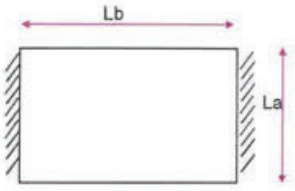
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S2

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S2

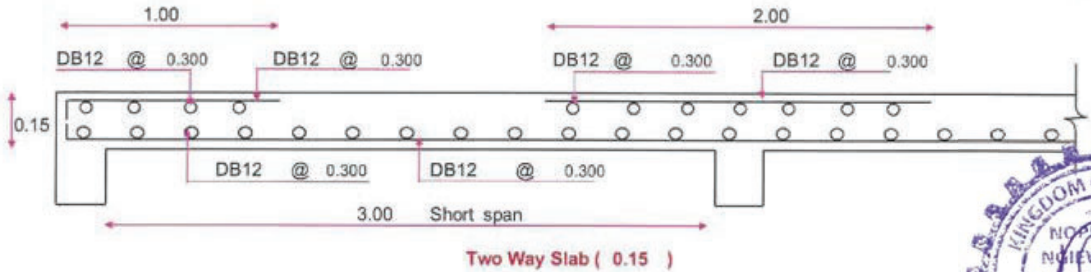
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	300	[cm]
	Lb	=	600	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
2 Siden Continuous / DB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	2.00	OK
La/Lb	m	=	0.500	
Shear	Vu S	=	1522	[kg/m]
	Vu L	=	2093	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



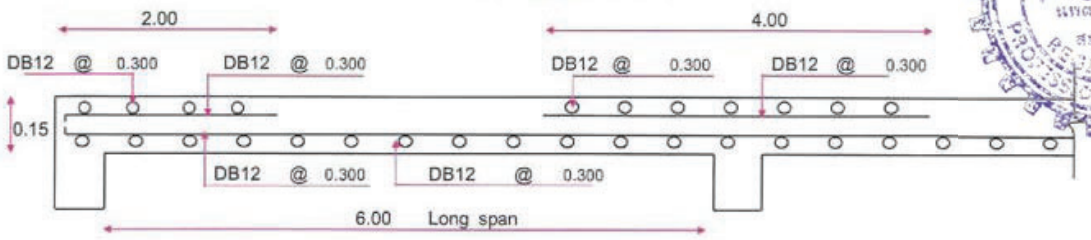
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	3.9	[cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$			
	d	<	11.5	OK
	a try	=	0	[cm]
	As	=	2.894	[cm^2]
	a	=	0.3892	[cm]
Moment arm	j d	=	11.31	[cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	1178	[kg-m]	As1 =	2.89	[cm^2]	3 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	616	[kg-m]	As2 =	1.51	[cm^2]	2 DB12 @ 0.300	OK
	+ Middle	Mu m	=	931	[kg-m]	As3 =	2.29	[cm^2]	3 DB12 @ 0.300	OK
Long Span										
Moment	- Continue	Mu	=	671	[kg-m]	As4 =	1.65	[cm^2]	2 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	342	[kg-m]	As5 =	0.84	[cm^2]	1 DB12 @ 0.300	OK
	+ Middle	Mu m	=	507	[kg-m]	As6 =	1.25	[cm^2]	2 DB12 @ 0.300	OK
	Minimum Steel					As min =	3.00	[cm^2]	3 DB12 @ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

ONE WAY SLAB

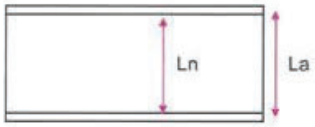
(Ultimate Stress Design method in accordance with ACI 318-89)

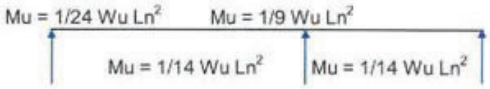
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S2

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม) filename : Slab 8S2

Concrete Strength	fc' =	350	[kg/cm^2]	
	fy SD =	4000	[kg/cm^2]	
	fy RD =	2400	[kg/cm^2]	
Clear span Length	Ln =	300	[cm]	
Load	LL =	500	[kg/ m]	
Type of Slab		Simple Support		
Reinforcement Steel		DB12		
	DL	360	[kg/ m]	
Thick of Slab	h =	15	[cm]	
	Wu =	1354	[kg/ m]	
Moment	- Mu L =	508	[kg-m]	
	+ Mu m =	1523	[kg-m]	
	- Mu R =	508	[kg-m]	
Shear	Vur =	2336	[kg]	
	K =	0.80		
	ϕ =	0.90		
	ρ =	0.0360		
	ρ_{max} =	0.0270		
	d =	4.4	[cm] (Mu / $\phi\rho$ fy db^2 (1-0.59 r fy / fc'))$^{0.5}$	
	d < h	OK		





Vur =1.15Wu Ln /2

Sheck Shear

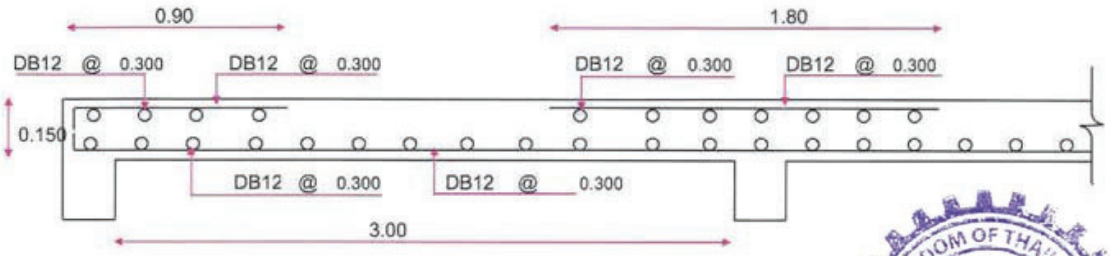
Existing Unit Shear	vc =	2.03	[kg/cm^2]
Allowable Shear unit	v all =	8.92	[kg/cm^2]
	vc < v all	OK	
	a try =	0.5056	[cm]
	As =	3.762	[cm^2]
	a =	0.5058	[cm]
	j d =	14.75	[cm]

Check Deflection

Δ all =	0.83	[cm]	
ρ fy =	8.0 <	35	[kg/cm^2]
I =	28125	[cm^4]	
Ec =	284116	[kg/cm^2]	
Δ =	0.18 [cm] <	Δ all	OK
LL = 40 % and Cofficient =	2		
Δ Time =	0.15 <	Δ all	OK

Moment arm Mu / ϕ fy j

- Mu L	As L =	0.96	[cm^2]	1 DB12	@	0.300	OK
+ Mu m	As m =	2.87	[cm^2]	3 DB12	@	0.300	OK
- Mu R	As R =	0.96	[cm^2]	1 DB12	@	0.300	OK
Minmum Steel	As min =	3.00	[cm^2]	3 DB12	@	0.300	OK



One Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

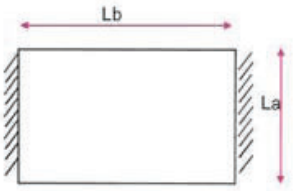
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S3

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S3

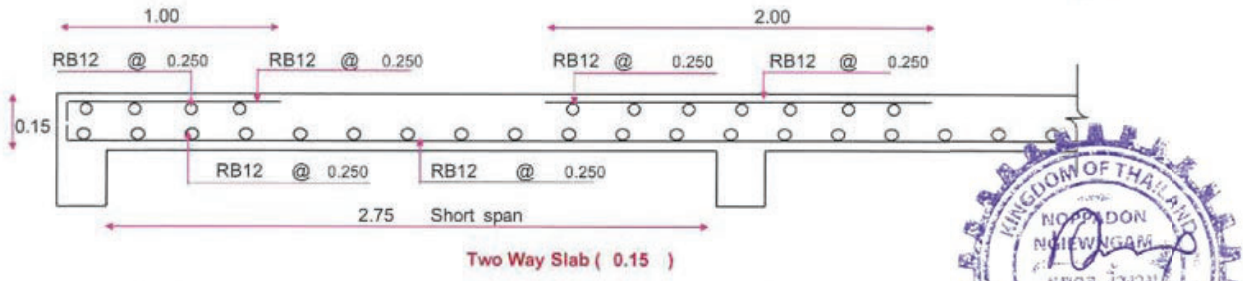
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	275	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
1 Siden Continuous / RB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	1.64	OK
La/Lb	m	=	0.611	
Shear	Vu S	=	1395	[kg/m]
	Vu L	=	1832	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



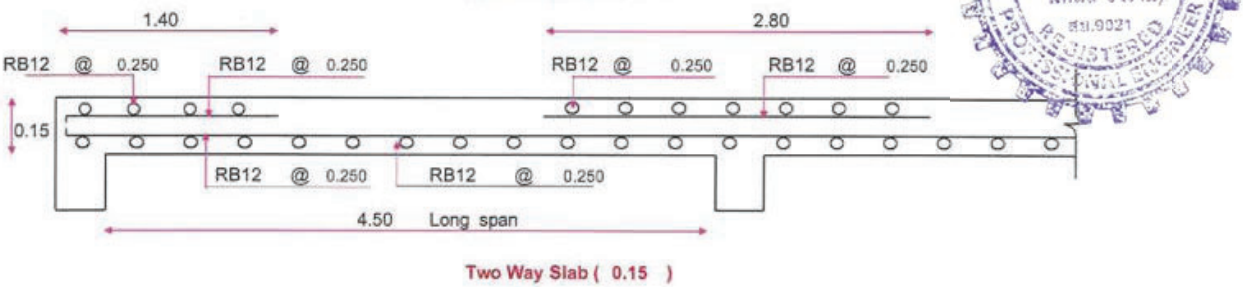
Check h	ρ	=	0.0712
	ρ_{max}	=	0.0534
Required d		=	3.3 [cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$		
	d	<	11.5 OK
	a try	=	0 [cm]
	As	=	4.106 [cm²]
	a	=	0.3312 [cm]
Moment arm	j d	=	11.33 [cm]

Mu / ϕ fy j d

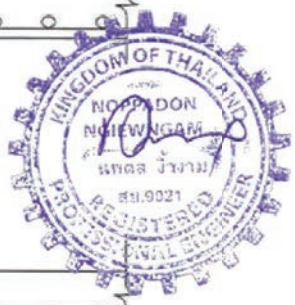
Short Span									
Moment	- Continue	Mu	= 1005 [kg-m]	As1	= 4.11	[cm²]	4 RB12	@ 0.250	OK
	- Not Cont.	Mu	= 518 [kg-m]	As2	= 2.12	[cm²]	2 RB12	@ 0.250	OK
	+ Middle	Mu m	= 783 [kg-m]	As3	= 3.20	[cm²]	3 RB12	@ 0.250	OK
Long Span									
Moment	- Continue	Mu	= 668 [kg-m]	As4	= 2.73	[cm²]	3 RB12	@ 0.250	OK
	- Not Cont.	Mu	= 334 [kg-m]	As5	= 1.36	[cm²]	2 RB12	@ 0.250	OK
	+ Middle	Mu m	= 506 [kg-m]	As6	= 2.07	[cm²]	2 RB12	@ 0.250	OK
	Minimum Steel			As min	= 3.75	[cm²]	4 RB12	@ 0.250	OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

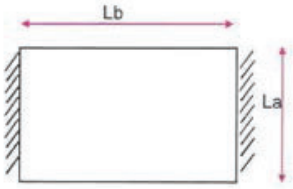
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S3

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S3

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	275	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
2 Siden Continuous / RB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	1.64	OK
La/Lb	m	=	0.611	
Shear	Vu S	=	1395	[kg/m]
	Vu L	=	1832	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



Check h ρ = 0.0712 ; ρ_{max} = 0.0534

Required d = 3.1 [cm]

(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$

d < 11.5 OK

a try = 0 [cm]

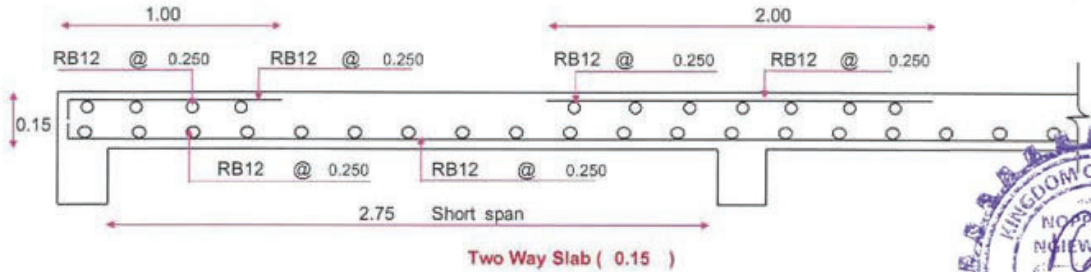
As = 3.550 [cm^2]

a = 0.2864 [cm]

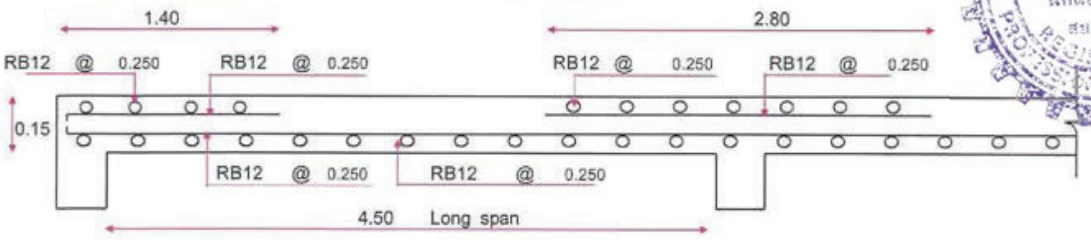
Moment arm j d = 11.36 [cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	= 871	[kg-m]	As1 = 3.55	[cm^2]	4 RB12	@	0.250	OK
	- Not Cont.	Mu	= 449	[kg-m]	As2 = 1.83	[cm^2]	2 RB12	@	0.250	OK
	+ Middle	Mu m	= 679	[kg-m]	As3 = 2.77	[cm^2]	3 RB12	@	0.250	OK
Long Span										
Moment	- Continue	Mu	= 564	[kg-m]	As4 = 2.30	[cm^2]	3 RB12	@	0.250	OK
	- Not Cont.	Mu	= 288	[kg-m]	As5 = 1.17	[cm^2]	2 RB12	@	0.250	OK
	+ Middle	Mu m	= 426	[kg-m]	As6 = 1.74	[cm^2]	2 RB12	@	0.250	OK
	Minimum Steel				As min = 3.75	[cm^2]	4 RB12	@	0.250	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

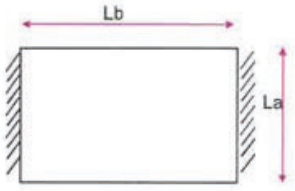
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S3

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S3

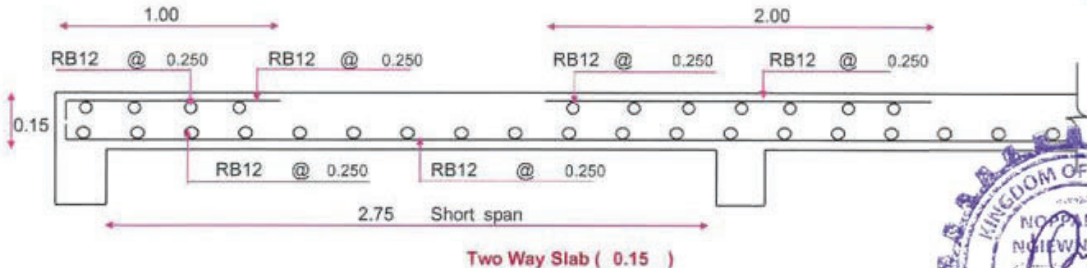
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	275	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
3 Siden Continuous / RB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	1.64	OK
La/Lb	m	=	0.611	
Shear	Vu S	=	1395	[kg/m]
	Vu L	=	1832	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



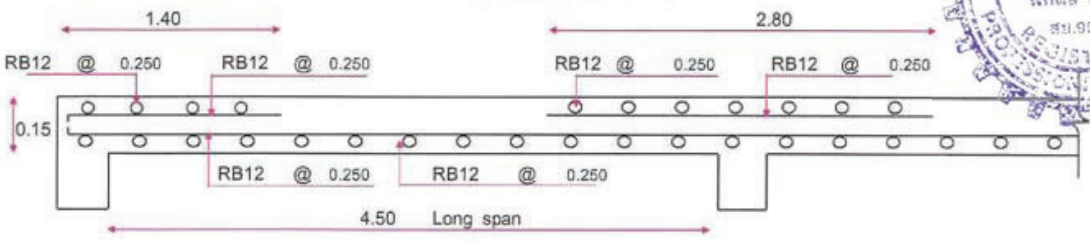
Check h	ρ	=	0.0712
	ρ_{max}	=	0.0534
Required	d	=	2.9 [cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$		
	d	<	11.5 OK
	a try	=	0 [cm]
	As	=	3.123 [cm^2]
	a	=	0.2520 [cm]
Moment arm	j d	=	11.37 [cm]

Mu / ϕ fy j d

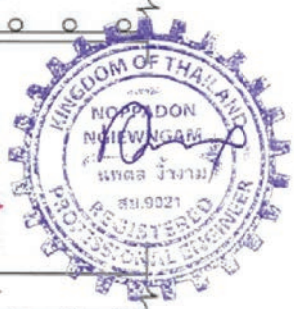
Short Span										
Moment	- Continue	Mu	=	767 [kg-m]	As1	=	3.12 [cm^2]	3 RB12	@	0.250 OK
	- Not Cont.	Mu	=	403 [kg-m]	As2	=	1.64 [cm^2]	2 RB12	@	0.250 OK
	+ Middle	Mu m	=	599 [kg-m]	As3	=	2.44 [cm^2]	3 RB12	@	0.250 OK
Long Span										
Moment	- Continue	Mu	=	472 [kg-m]	As4	=	1.92 [cm^2]	2 RB12	@	0.250 OK
	- Not Cont.	Mu	=	242 [kg-m]	As5	=	0.98 [cm^2]	1 RB12	@	0.250 OK
	+ Middle	Mu m	=	357 [kg-m]	As6	=	1.45 [cm^2]	2 RB12	@	0.250 OK
	Minimum Steel				As min	=	3.75 [cm^2]	4 RB12	@	0.250 OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

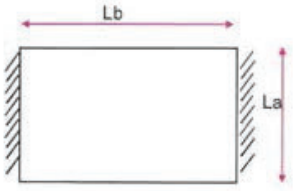
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S4

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S4

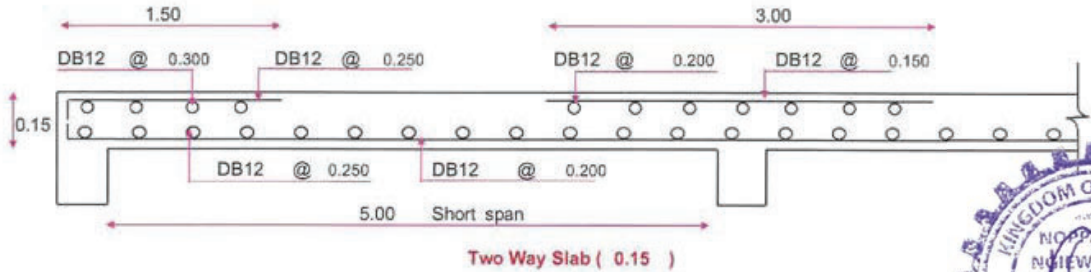
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	500	[cm]
	Lb	=	600	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	120	[kg/m²]
1 Siden Continuous / DB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1522	[kg/ m]
	Lb/La	=	1.20	OK
La/Lb	m	=	0.833	
Shear	Vu S	=	2537	[kg/m]
	Vu L	=	2924	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



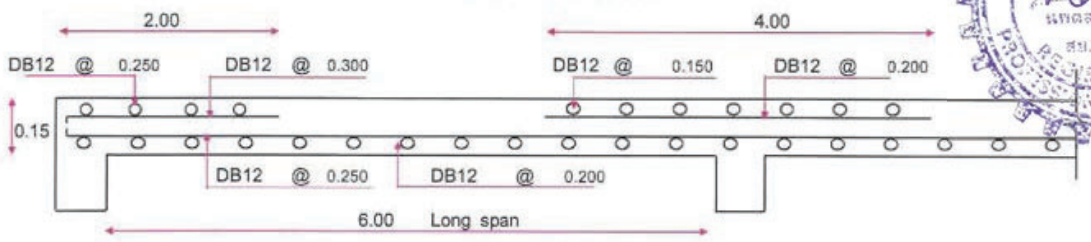
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	5.7	[cm]
	(Mu / ϕ ρ fy db² (1-0.59 ρ fy / fc'))^0.5			
	d	<	11.5	OK
	a try	=	0	[cm]
	As	=	6.391	[cm²]
	a	=	0.8592	[cm]
Moment arm	j d	=	11.07	[cm]

Mu / ϕ fy j d

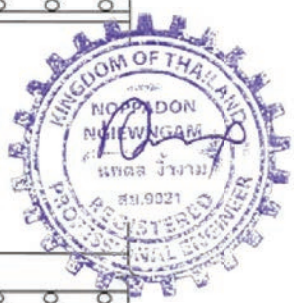
Short Span										
Moment	- Continue	Mu	= 2613	[kg-m]	As1 =	6.56	[cm²]	6 DB12	@ 0.150	OK
	- Not Cont.	Mu	= 1357	[kg-m]	As2 =	3.41	[cm²]	4 DB12	@ 0.250	OK
	+ Middle	Mu m	= 2055	[kg-m]	As3 =	5.16	[cm²]	5 DB12	@ 0.200	OK
Long Span										
Moment	- Continue	Mu	= 2207	[kg-m]	As4 =	5.54	[cm²]	5 DB12	@ 0.200	OK
	- Not Cont.	Mu	= 1103	[kg-m]	As5 =	2.77	[cm²]	3 DB12	@ 0.300	OK
	+ Middle	Mu m	= 1674	[kg-m]	As6 =	4.20	[cm²]	4 DB12	@ 0.250	OK
	Minimum Steel				As min =	3.00	[cm²]	3 DB12	@ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

ONE WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

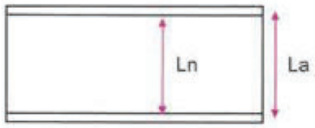
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S4

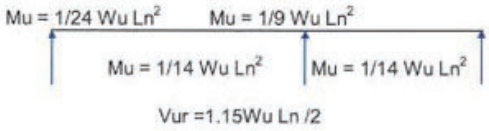
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

filename : Slab 8S4

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Clear span Length	Ln	=	400	[cm]
Load	LL	=	500	[kg/ m]
Type of Slab			Simple Support	
Reinforcement Steel			DB12	



	DL		360	[kg/ m]
Thick of Slab	h	=	15	[cm]
	Wu	=	1354	[kg/ m]
Moment	- Mu L	=	903	[kg-m]
	+ Mu m	=	2708	[kg-m]
	- Mu R	=	903	[kg-m]
Shear	Vur	=	3114	[kg]
	K	=	0.80	
	ϕ	=	0.90	
	ρ	=	0.0360	
	ρmax	=	0.0270	
	d	=	5.8	[cm] (Mu / ϕ ρ fy db² (1-0.59 r fy / fc'))^0.5
		d < h		OK



Vur =1.15Wu Ln /2

Sheck Shear

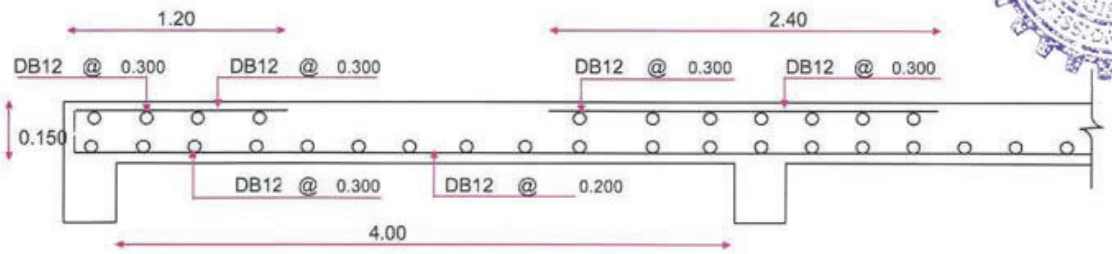
Existing Unit Shear	vc	=	2.71	[kg/cm²]
Allowable Shear unit	v all	=	8.92	[kg/cm²]
	vc	< v all		OK
	a try	=	0.9159	[cm]
	As	=	6.812	[cm²]
	a	=	0.9159	[cm]
	j d	=	14.54	[cm]

Check Deflection

Δ all	=	1.11	[cm]	
ρ fy	=	13.8 <	35	[kg/cm²]
I	=	28125	[cm⁴]	
Ec	=	284116	[kg/cm²]	
Δ	=	0.56 [cm] <	Δ all	OK
LL = 40 % and Cofficient =		2		
Δ Time	=	0.47 <	Δ all	OK

Moment arm Mu / ϕ fy j

- Mu L	As L	=	1.72	[cm²]	2 DB12 @ 0.300		OK
+ Mu m	As m	=	5.17	[cm²]	5 DB12 @ 0.200		OK
- Mu R	As R	=	1.72	[cm²]	2 DB12 @ 0.300		OK
Minmum Steel	As min	=	3.00	[cm²]	3 DB12 @ 0.300		OK



One Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

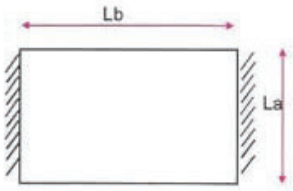
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **8S5**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab 8S5**

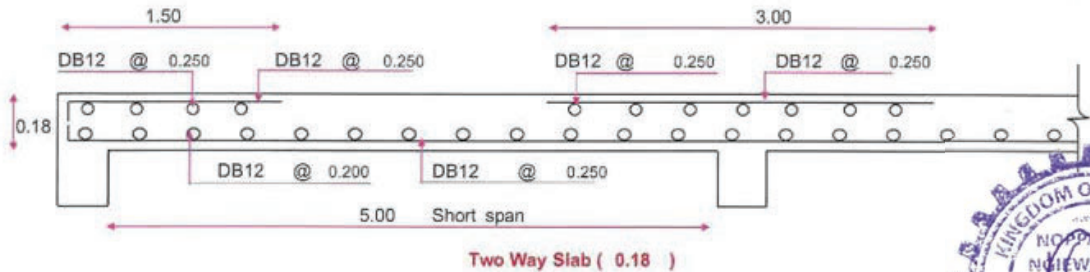
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	500	[cm]
	Lb	=	700	[cm]
Live Load	LL	=	750	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
0 Siden Continuous	DB12			
	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	1863	[kg/ m]
	Lb/La	=	1.40	OK
La/Lb	m	=	0.714	
Shear	Vu S	=	3105	[kg/m]
	Vu L	=	3865	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L	<	Vc all	OK



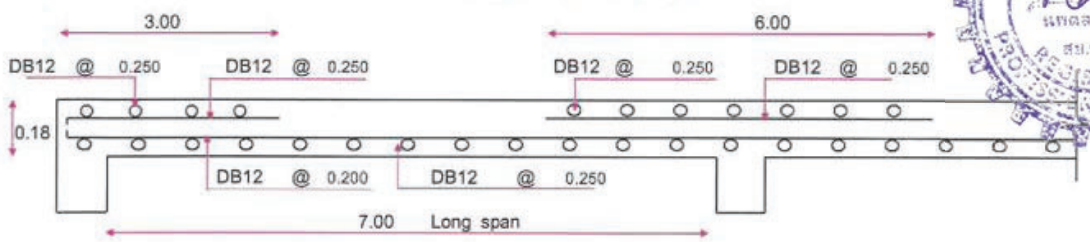
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	5.4	[cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$			
	d	<	14.0	OK
	a try	=	1	[cm]
	As	=	4.771	[cm²]
	a	=	0.6414	[cm]
Moment arm	j d	=	13.68	[cm]

Mu / ϕ fy j d

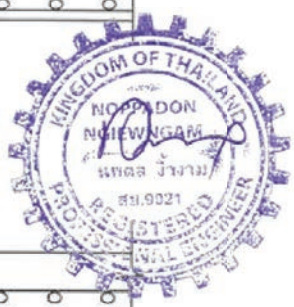
Short Span										
Moment	- Continue	Mu	=	0	[kg-m]	As1 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	0	[kg-m]	As2 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	+ Middle	Mu m	=	0	[kg-m]	As3 =	0.00	[cm²]	1 DB12 @ 0.250	OK
Long Span										
Moment	- Continue	Mu	=	0	[kg-m]	As4 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	1537	[kg-m]	As5 =	3.12	[cm²]	3 DB12 @ 0.250	OK
	+ Middle	Mu m	=	2329	[kg-m]	As6 =	4.73	[cm²]	5 DB12 @ 0.200	OK
	Minimum Steel					As min =	3.50	[cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.18)



Two Way Slab (0.18)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

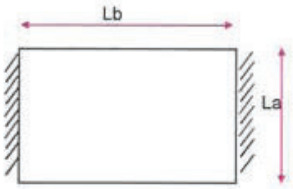
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S5

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

Filename : Slab 8S5

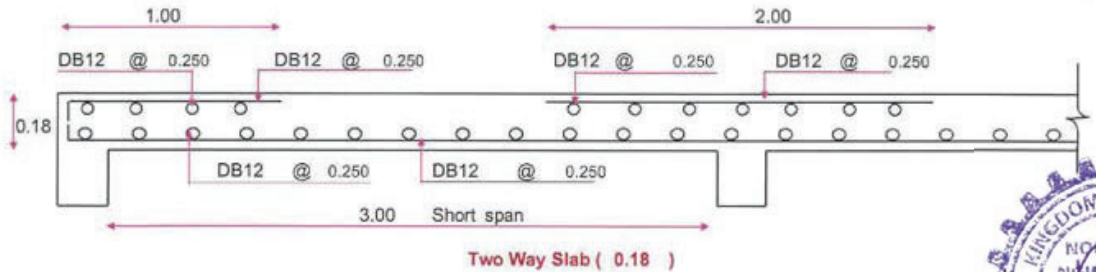
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	300	[cm]
	Lb	=	600	[cm]
Live Load	LL	=	750	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
0 Siden Continuous	DB12			
	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	1863	[kg/ m]
	Lb/La	=	2.00	OK
La/Lb	m	=	0.500	
Shear	Vu S	=	1863	[kg/m]
	Vu L	=	2562	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L	<	Vc all	OK



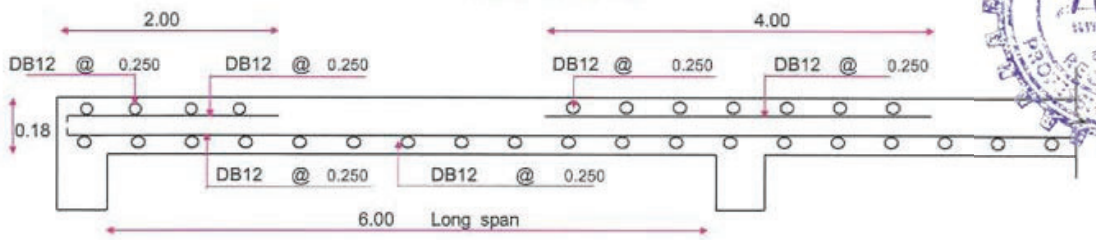
Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	3.2 [cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$		
	d	<	14.0 OK
	a try	=	1 [cm]
	As	=	1.717 [cm^2]
	a	=	0.2309 [cm]
Moment arm	j d	=	13.88 [cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	0 [kg-m]	As1	=	0.00 [cm^2]	1 DB12	@	0.250 OK
	- Not Cont.	Mu	=	0 [kg-m]	As2	=	0.00 [cm^2]	1 DB12	@	0.250 OK
	+ Middle	Mu m	=	0 [kg-m]	As3	=	0.00 [cm^2]	1 DB12	@	0.250 OK
Long Span										
Moment	- Continue	Mu	=	0 [kg-m]	As4	=	0.00 [cm^2]	1 DB12	@	0.250 OK
	- Not Cont.	Mu	=	553 [kg-m]	As5	=	1.11 [cm^2]	1 DB12	@	0.250 OK
	+ Middle	Mu m	=	838 [kg-m]	As6	=	1.68 [cm^2]	2 DB12	@	0.250 OK
	Minimum Steel				As min	=	3.50 [cm^2]	4 DB12	@	0.250 OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

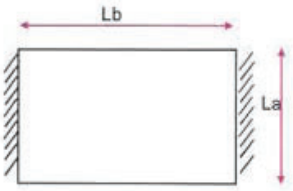
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. 8S5

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

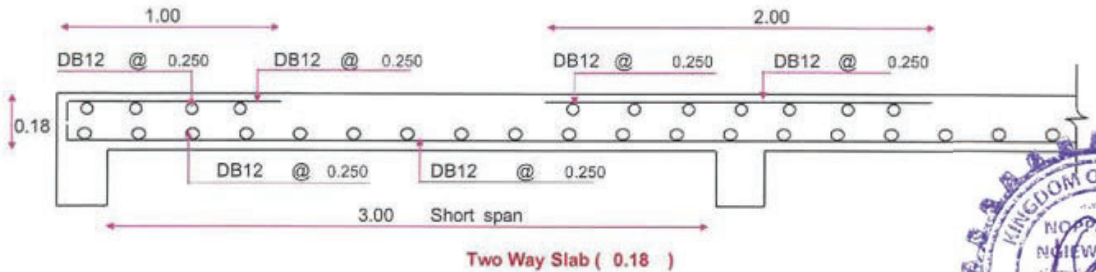
Filename : Slab 8S5

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	17.5	[cm]
Clear span Length	La	=	300	[cm]
	Lb	=	500	[cm]
Live Load	LL	=	750	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
0 Siden Continuous / DB12	DL	=	420	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	14.0	[cm]
	Wu	=	1863	[kg/ m]
	Lb/La	=	1.67	OK
La/Lb	m	=	0.600	
Shear	Vu S	=	1863	[kg/m]
	Vu L	=	2459	[kg/m]
	Vc all	=	8849	[kg/m]
	Vu L < Vc all			OK

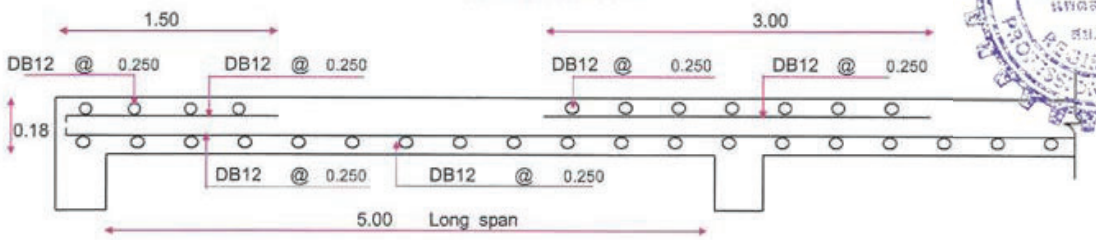


Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	3.2	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	14.0	OK
	a try	=	1	[cm]
	As	=	1.717	[cm²]
	a	=	0.2309	[cm]
Moment arm	j d	=	13.88	[cm]

Short Span					Mu / ϕ fy j d					
Moment	- Continue	Mu	=	0	[kg-m]	As1 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	0	[kg-m]	As2 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	+ Middle	Mu m	=	0	[kg-m]	As3 =	0.00	[cm²]	1 DB12 @ 0.250	OK
Long Span										
Moment	- Continue	Mu	=	0	[kg-m]	As4 =	0.00	[cm²]	1 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	553	[kg-m]	As5 =	1.11	[cm²]	1 DB12 @ 0.250	OK
	+ Middle	Mu m	=	838	[kg-m]	As6 =	1.68	[cm²]	2 DB12 @ 0.250	OK
	Minimum Steel					As min =	3.50	[cm²]	4 DB12 @ 0.250	OK



Two Way Slab (0.18)



Two Way Slab (0.18)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

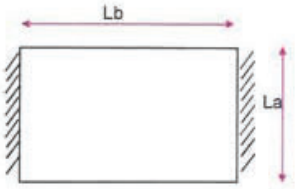
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS1**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab RS1**

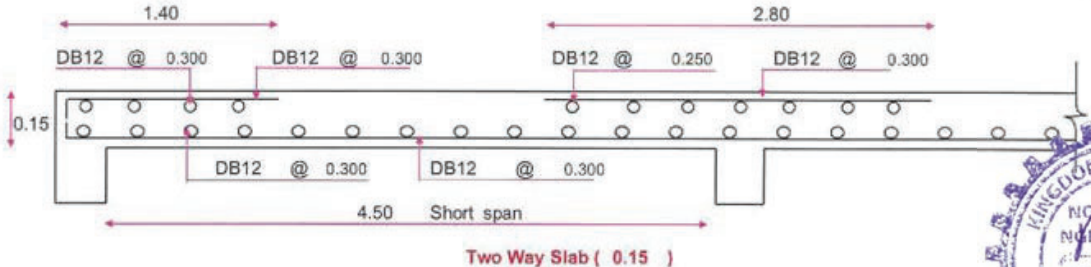
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	100	[kg/m²]
2 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1494	[kg/ m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	2241	[kg/m]
	Vu L	=	2241	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L	<	Vc all	OK



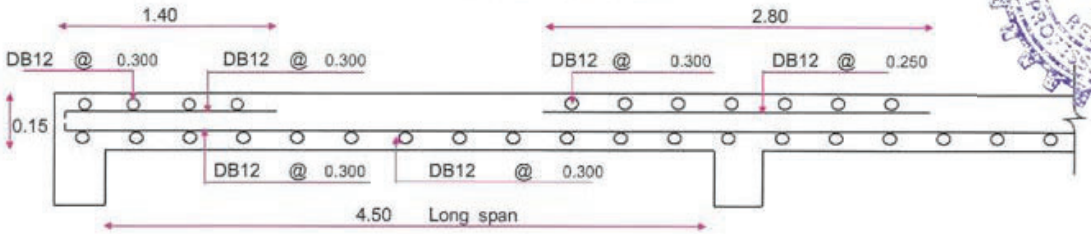
Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	4.3 [cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$		
	d	<	11.5 OK
a try		=	1 [cm]
As		=	3.661 [cm²]
a		=	0.4922 [cm]
Moment arm	j d	=	11.25 [cm]

Mu / ϕ fy j d

Short Span										
Moment	- Continue	Mu	=	0 [kg-m]	As1	=	0.00 [cm²]	1 DB12	@	0.300 OK
	- Not Cont.	Mu	=	756 [kg-m]	As2	=	1.87 [cm²]	2 DB12	@	0.300 OK
	+ Middle	Mu m	=	1119 [kg-m]	As3	=	2.76 [cm²]	3 DB12	@	0.300 OK
Long Span										
Moment	- Continue	Mu	=	1482 [kg-m]	As4	=	3.66 [cm²]	4 DB12	@	0.250 OK
	- Not Cont.	Mu	=	756 [kg-m]	As5	=	1.87 [cm²]	2 DB12	@	0.300 OK
	+ Middle	Mu m	=	1119 [kg-m]	As6	=	2.76 [cm²]	3 DB12	@	0.300 OK
	Minimum Steel				As min	=	3.00 [cm²]	3 DB12	@	0.300 OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

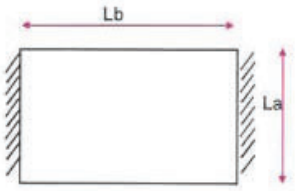
(Method 2)

Job No.	:	193018	
Rev.	:	1	
Initial	:	Noppadon N.	
Date	:	7-ธ.ค.-20	
Filename	:	Slab RS1	

Slab No. **RS1**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	450	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	100	[kg/m²]
3 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1494	[kg/ m]
	Lb/La	=	1.00	OK
La/Lb	m	=	1.000	
Shear	Vu S	=	2241	[kg/m]
	Vu L	=	2241	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



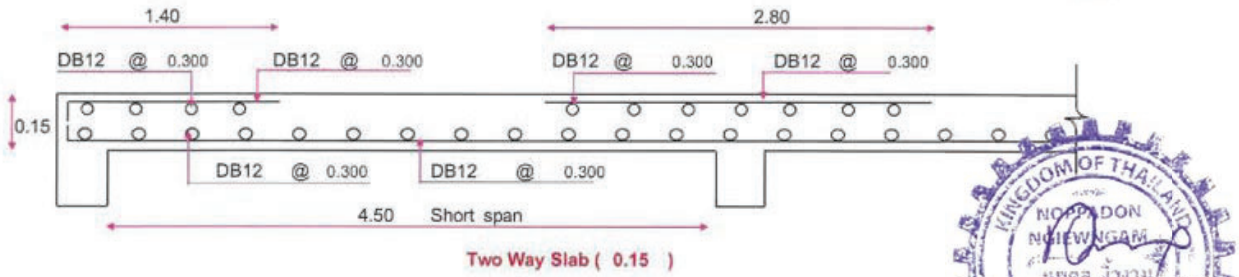
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	4.0	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	11.5	OK
	a try	=	1	[cm]
	As	=	3.063	[cm²]
	a	=	0.4118	[cm]
Moment arm	j d	=	11.29	[cm]

Short Span

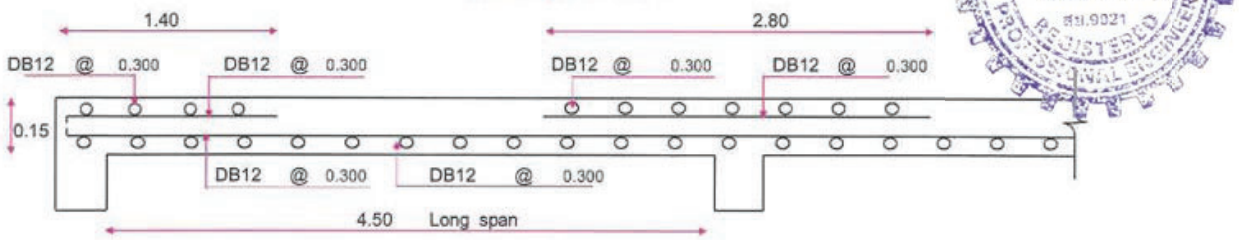
Moment	- Continue	Mu	=	0	[kg-m]	As1 =	0.00	[cm²]	1 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	635	[kg-m]	As2 =	1.56	[cm²]	2 DB12 @ 0.300	OK
	+ Middle	Mu m	=	938	[kg-m]	As3 =	2.31	[cm²]	3 DB12 @ 0.300	OK

Long Span

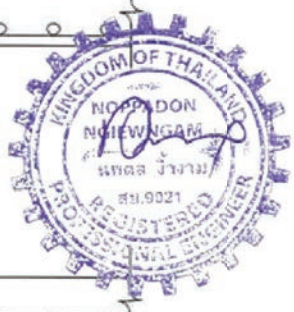
Moment	- Continue	Mu	=	1240	[kg-m]	As4 =	3.05	[cm²]	3 DB12 @ 0.300	OK
	- Not Cont.	Mu	=	635	[kg-m]	As5 =	1.56	[cm²]	2 DB12 @ 0.300	OK
	+ Middle	Mu m	=	938	[kg-m]	As6 =	2.31	[cm²]	3 DB12 @ 0.300	OK
	Minimum Steel					As min =	3.00	[cm²]	3 DB12 @ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

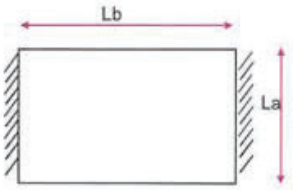
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS1**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

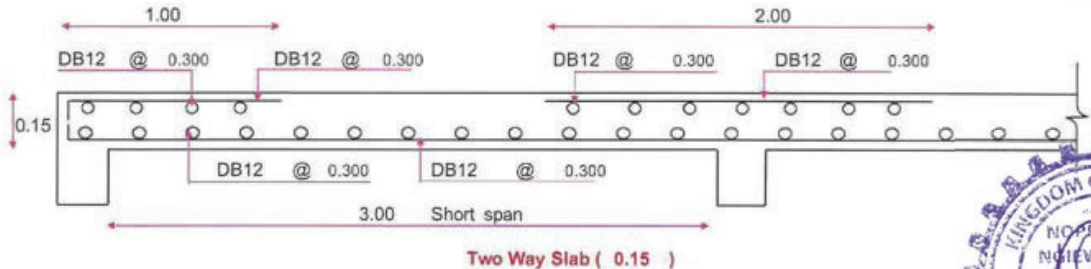
Filename : **Slab RS1**

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	300	[cm]
	Lb	=	450	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	100	[kg/m²]
3 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1494	[kg/ m]
	Lb/La	=	1.50	OK
La/Lb	m	=	0.667	
Shear	Vu S	=	1494	[kg/m]
	Vu L	=	1909	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK

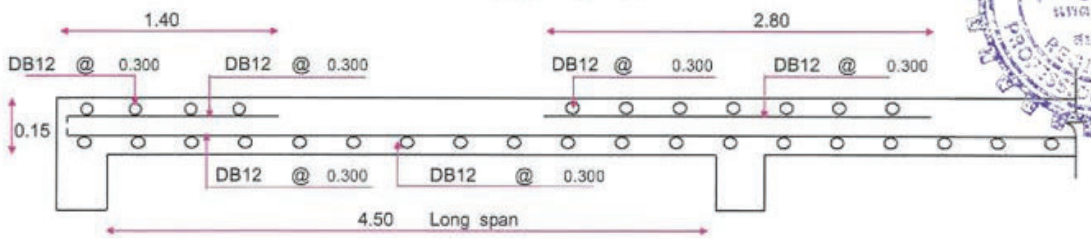


Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	3.2	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	11.5	OK
	a try	=	1	[cm]
	As	=	2.059	[cm²]
	a	=	0.2768	[cm]
Moment arm	j d	=	11.36	[cm]

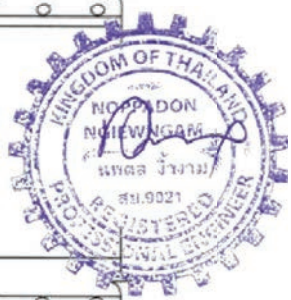
					$Mu / \phi fy jd$						
Short Span											
Moment	- Continue	Mu	=	834	[kg-m]	As1 =	2.04	[cm²]	2 DB12	@ 0.300	OK
	- Not Cont.	Mu	=	435	[kg-m]	As2 =	1.06	[cm²]	1 DB12	@ 0.300	OK
	+ Middle	Mu m	=	654	[kg-m]	As3 =	1.60	[cm²]	2 DB12	@ 0.300	OK
Long Span											
Moment	- Continue	Mu	=	551	[kg-m]	As4 =	1.35	[cm²]	2 DB12	@ 0.300	OK
	- Not Cont.	Mu	=	282	[kg-m]	As5 =	0.69	[cm²]	1 DB12	@ 0.300	OK
	+ Middle	Mu m	=	417	[kg-m]	As6 =	1.02	[cm²]	1 DB12	@ 0.300	OK
	Minimum Steel					As min =	3.00	[cm²]	3 DB12	@ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

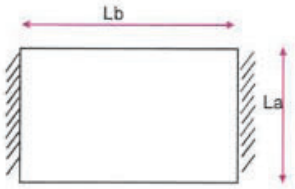
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS2**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab RS2**

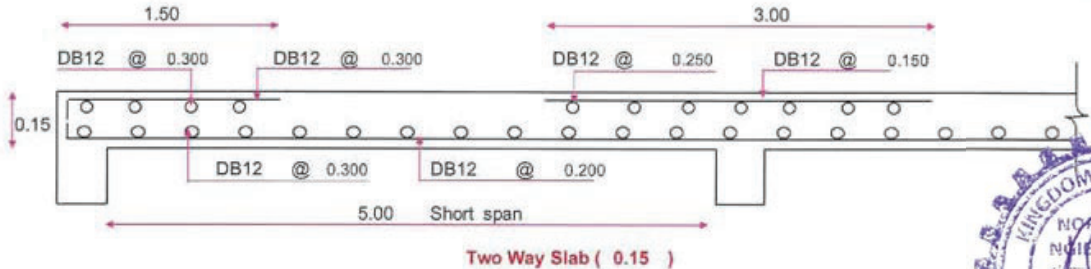
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	500	[cm]
	Lb	=	700	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1354	[kg/ m]
	Lb/La	=	1.40	OK
La/Lb	m	=	0.714	
Shear	Vu S	=	2257	[kg/m]
	Vu L	=	2809	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



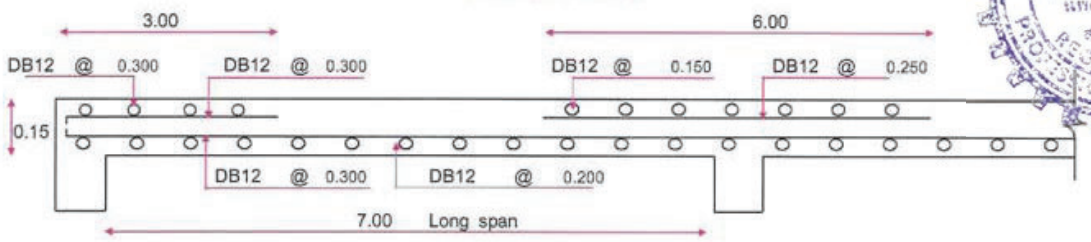
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	5.4	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	11.5	OK
	a try	=	1	[cm]
	As	=	5.784	[cm²]
	a	=	0.7777	[cm]
Moment arm	j d	=	11.11	[cm]

$Mu / \phi fy jd$

Short Span										
Moment	- Continue	Mu	=	2324	[kg-m]	As1 =	5.81	[cm²]	6 DB12 @ 0.150	OK
	- Not Cont.	Mu	=	1219	[kg-m]	As2 =	3.05	[cm²]	3 DB12 @ 0.300	OK
	+ Middle	Mu m	=	1828	[kg-m]	As3 =	4.57	[cm²]	5 DB12 @ 0.200	OK
Long Span										
Moment	- Continue	Mu	=	1659	[kg-m]	As4 =	4.15	[cm²]	4 DB12 @ 0.250	OK
	- Not Cont.	Mu	=	846	[kg-m]	As5 =	2.12	[cm²]	2 DB12 @ 0.300	OK
	+ Middle	Mu m	=	1252	[kg-m]	As6 =	3.13	[cm²]	3 DB12 @ 0.300	OK
	Minimum Steel					As min =	3.00	[cm²]	3 DB12 @ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

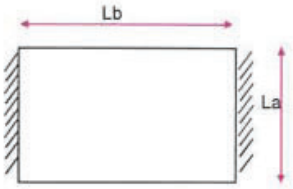
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS2**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab RS2**

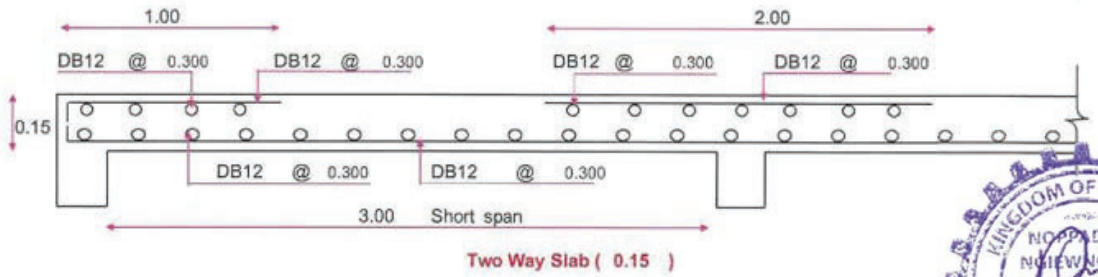
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	300	[cm]
	Lb	=	600	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous / DB12	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1354	[kg/ m]
	Lb/La	=	2.00	OK
La/Lb	m	=	0.500	
Shear	Vu S	=	1354	[kg/m]
	Vu L	=	1862	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



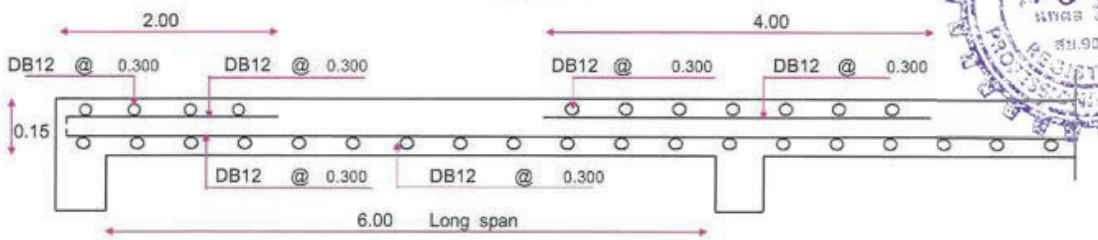
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	3.6	[cm]
	$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$			
	d	<	11.5	OK
	a try	=	1	[cm]
	As	=	2.608	[cm²]
	a	=	0.3507	[cm]
Moment arm	j d	=	11.32	[cm]

$Mu / \phi fy j d$

Short Span									
Moment	- Continue	Mu	= 1048	[kg-m]	As1 = 2.57	[cm²]	3 DB12	@ 0.300	OK
	- Not Cont.	Mu	= 548	[kg-m]	As2 = 1.35	[cm²]	2 DB12	@ 0.300	OK
	+ Middle	Mu m	= 829	[kg-m]	As3 = 2.03	[cm²]	2 DB12	@ 0.300	OK
Long Span									
Moment	- Continue	Mu	= 597	[kg-m]	As4 = 1.46	[cm²]	2 DB12	@ 0.300	OK
	- Not Cont.	Mu	= 305	[kg-m]	As5 = 0.75	[cm²]	1 DB12	@ 0.300	OK
	+ Middle	Mu m	= 451	[kg-m]	As6 = 1.11	[cm²]	1 DB12	@ 0.300	OK
	Minimum Steel				As min = 3.00	[cm²]	3 DB12	@ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

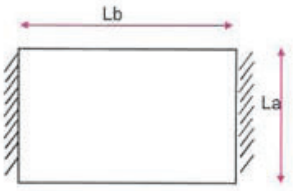
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS2**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab RS2**

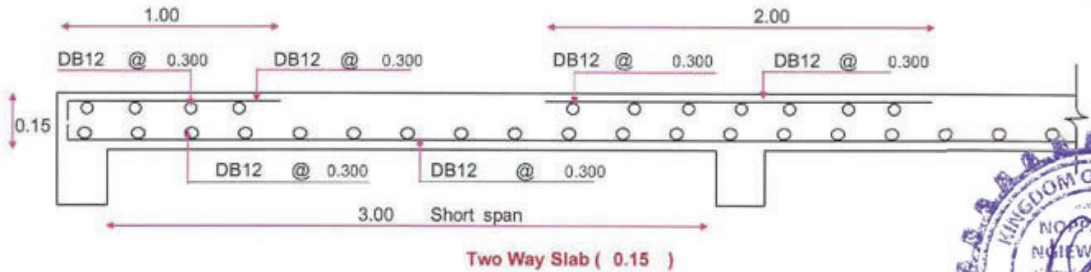
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	300	[cm]
	Lb	=	500	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1354	[kg/ m]
	Lb/La	=	1.67	OK
La/Lb	m	=	0.600	
Shear	Vu S	=	1354	[kg/m]
	Vu L	=	1787	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



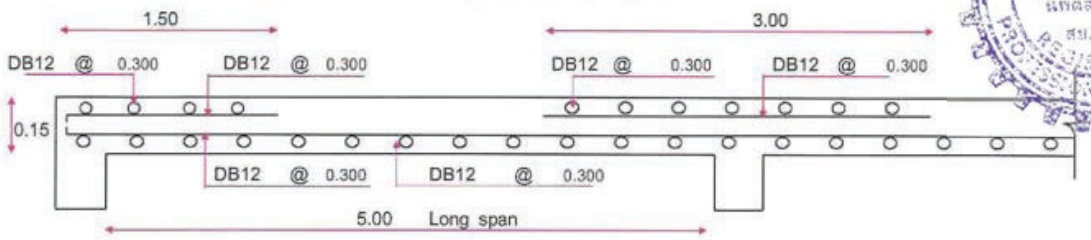
Check h	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
Required d		=	3.4	[cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$			
	d	<	11.5	OK
	a try	=	1	[cm]
	As	=	2.295	[cm²]
	a	=	0.3085	[cm]
Moment arm	j d	=	11.35	[cm]

Mu / ϕ fy j d

Short Span									
Moment	- Continue	Mu	=	922	[kg-m]	As1 =	2.26	[cm²]	2 DB12 @ 0.300 OK
	- Not Cont.	Mu	=	475	[kg-m]	As2 =	1.16	[cm²]	2 DB12 @ 0.300 OK
	+ Middle	Mu m	=	719	[kg-m]	As3 =	1.76	[cm²]	2 DB12 @ 0.300 OK
Long Span									
Moment	- Continue	Mu	=	597	[kg-m]	As4 =	1.46	[cm²]	2 DB12 @ 0.300 OK
	- Not Cont.	Mu	=	305	[kg-m]	As5 =	0.75	[cm²]	1 DB12 @ 0.300 OK
	+ Middle	Mu m	=	451	[kg-m]	As6 =	1.10	[cm²]	1 DB12 @ 0.300 OK
	Minimum Steel					As min =	3.00	[cm²]	3 DB12 @ 0.300 OK



Two Way Slab (0.15)



Two Way Slab (0.15)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

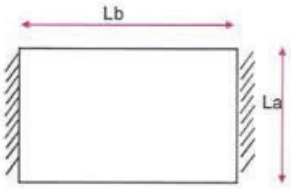
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS2**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab RS2**

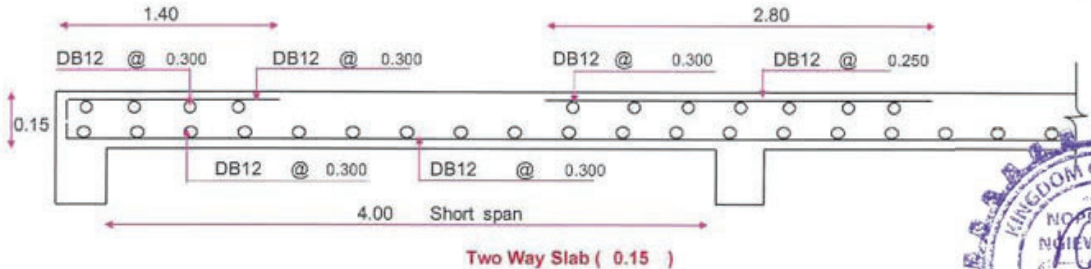
Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	400	[cm]
	Lb	=	600	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous	DB12			
	DL	=	360	[kg/ m]
	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1354	[kg/ m]
	Lb/La	=	1.50	OK
La/Lb	m	=	0.667	
Shear	Vu S	=	1805	[kg/m]
	Vu L	=	2307	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L	<	Vc all	OK



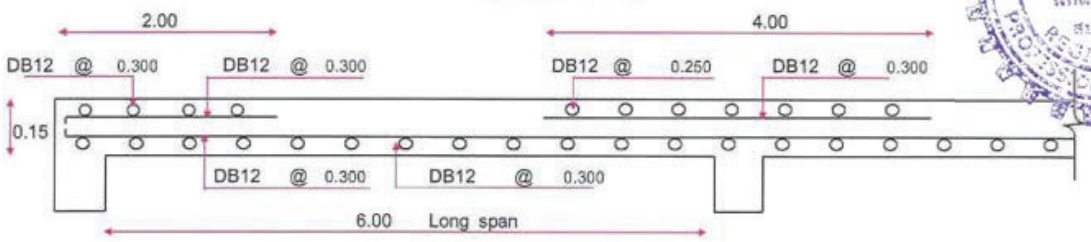
Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	4.4 [cm]
	(Mu / ϕ ρ fy db^2 (1-0.59 ρ fy / fc'))$^{0.5}$		
	d	<	11.5 OK
	a try	=	1 [cm]
	As	=	3.828 [cm^2]
	a	=	0.5147 [cm]
Moment arm	j d	=	11.24 [cm]

Mu / ϕ fy j d

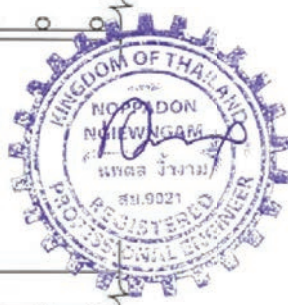
Short Span										
Moment	- Continue	Mu	=	1538 [kg-m]	As1	=	3.80 [cm^2]	4 DB12	@	0.250 OK
	- Not Cont.	Mu	=	802 [kg-m]	As2	=	1.98 [cm^2]	2 DB12	@	0.300 OK
	+ Middle	Mu m	=	1206 [kg-m]	As3	=	2.98 [cm^2]	3 DB12	@	0.300 OK
Long Span										
Moment	- Continue	Mu	=	1062 [kg-m]	As4	=	2.62 [cm^2]	3 DB12	@	0.300 OK
	- Not Cont.	Mu	=	542 [kg-m]	As5	=	1.34 [cm^2]	2 DB12	@	0.300 OK
	+ Middle	Mu m	=	802 [kg-m]	As6	=	1.98 [cm^2]	2 DB12	@	0.300 OK
	Minimum Steel				As min	=	3.00 [cm^2]	3 DB12	@	0.300 OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

TWO WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

(Method 2)

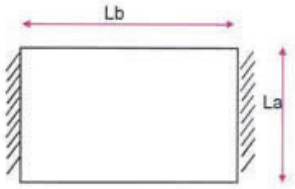
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS2**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

Filename : **Slab RS2**

Concrete Strength	fc'	=	350	[kg/cm²]
	fy SD	=	4000	[kg/cm²]
	fy RD	=	2400	[kg/cm²]
Thick of Slab	h	=	15	[cm]
Clear span Length	La	=	400	[cm]
	Lb	=	500	[cm]
Live Load	LL	=	500	[kg/m²]
Finishing Load	FL	=	0	[kg/m²]
2 Siden Continuous	DL	=	360	[kg/ m]
DB12	k	=	0.80	
	ϕ bend	=	0.90	
	ϕ Shear	=	0.85	
Cover (Short Side)	Cc	=	3.5	[cm]
Depth of Slab	d	=	11.5	[cm]
	Wu	=	1354	[kg/ m]
	Lb/La	=	1.25	OK
La/Lb	m	=	0.800	
Shear	Vu S	=	1805	[kg/m]
	Vu L	=	2130	[kg/m]
	Vc all	=	6742	[kg/m]
	Vu L < Vc all			OK



Check h	ρ	=	0.0360
	ρ_{max}	=	0.0270
Required d		=	4.1 [cm]

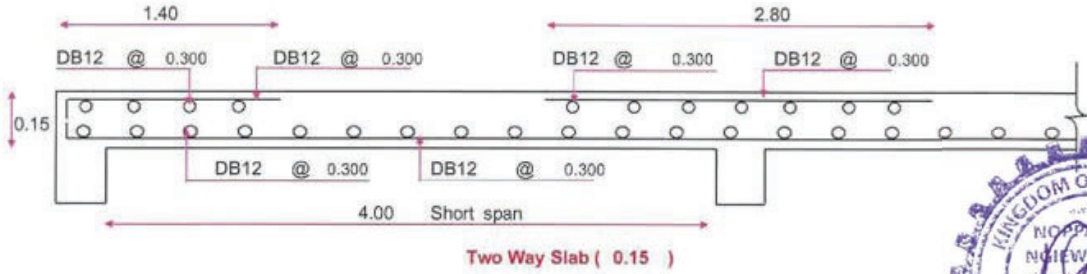
$(Mu / \phi \rho fy db^2 (1-0.59 \rho fy / fc'))^{0.5}$

d < 11.5 OK

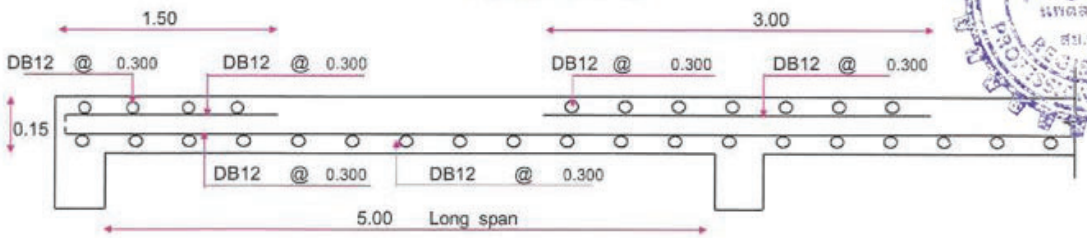
a try		=	1 [cm]
As		=	3.325 [cm²]
a		=	0.4470 [cm]
Moment arm	j d	=	11.28 [cm]

$Mu / \phi fy j d$

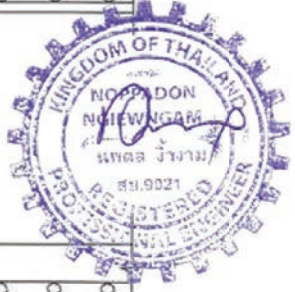
Short Span									
Moment	- Continue	Mu	= 1336	[kg-m]	As1 = 3.29	[cm²]	3 DB12	@ 0.300	OK
	- Not Cont.	Mu	= 693	[kg-m]	As2 = 1.71	[cm²]	2 DB12	@ 0.300	OK
	+ Middle	Mu m	= 1040	[kg-m]	As3 = 2.56	[cm²]	3 DB12	@ 0.300	OK
Long Span									
Moment	- Continue	Mu	= 1062	[kg-m]	As4 = 2.61	[cm²]	3 DB12	@ 0.300	OK
	- Not Cont.	Mu	= 542	[kg-m]	As5 = 1.33	[cm²]	2 DB12	@ 0.300	OK
	+ Middle	Mu m	= 802	[kg-m]	As6 = 1.97	[cm²]	2 DB12	@ 0.300	OK
	Minimum Steel				As min = 3.00	[cm²]	3 DB12	@ 0.300	OK



Two Way Slab (0.15)



Two Way Slab (0.15)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

ONE WAY SLAB

(Ultimate Stress Design method in accordance with ACI 318-89)

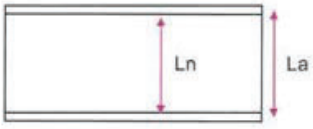
Job No.	:	193018
Rev.	:	1
Initial	:	Noppadon N.
Date	:	7-ธ.ค.-20

Slab No. **RS2**

Project : **สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)**

filename : **Slab RS2**

Concrete Strength	fc' =	350	[kg/cm^2]	
	fy SD =	4000	[kg/cm^2]	
	fy RD =	2400	[kg/cm^2]	
Clear span Length	Ln =	400	[cm]	
Load	LL =	500	[kg/ m]	
Type of Slab		Simple Support		
Reinforcement Steel		DB12		



	DL	360	[kg/ m]
Thick of Slab	h =	15	[cm]
	Wu =	1354	[kg/ m]
Moment	- Mu L =	903	[kg-m]
	+ Mu m =	2708	[kg-m]
	- Mu R =	903	[kg-m]
Shear	Vur =	3114	[kg]
	K =	0.80	
	ϕ =	0.90	
	ρ =	0.0360	
	ρ_{max} =	0.0270	
	d =	5.8	[cm] (Mu / $\phi \rho$ fy db^2 (1-0.59 r fy / fc'))$^{0.5}$
	d < h	OK	

$Mu = 1/24\ Wu\ Ln^2$ $Mu = 1/9\ Wu\ Ln^2$

$Mu = 1/14\ Wu\ Ln^2$ $Mu = 1/14\ Wu\ Ln^2$

$Vur = 1.15 Wu\ Ln / 2$

Sheck Shear

Existing Unit Shear	vc =	2.71	[kg/cm^2]
Allowable Shear unit	v all =	8.92	[kg/cm^2]
	vc < v all	OK	
	a try =	0.9159	[cm]
	As =	6.812	[cm^2]
	a =	0.9159	[cm]
	j d =	14.54	[cm]

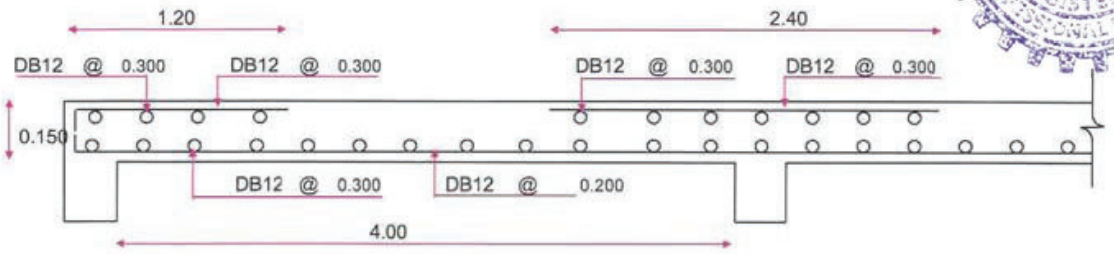
Check Deflection

Δ all =	1.11	[cm]		
ρ fy =	13.8	<	35	[kg/cm^2]
I =	28125	[cm^4]		
Ec =	284116	[kg/cm^2]		
Δ =	0.56	[cm] <	Δ all	OK
LL = 40 % and Cofficient =	2			
Δ Time =	0.47	<	Δ all	OK

Moment arm

Mu / ϕ fy j

- Mu L	As L =	1.72	[cm^2]	2 DB12	@	0.300	OK
+ Mu m	As m =	5.17	[cm^2]	5 DB12	@	0.200	OK
- Mu R	As R =	1.72	[cm^2]	2 DB12	@	0.300	OK
Minmum Steel	As min =	3.00	[cm^2]	3 DB12	@	0.300	OK



One Way Slab (0.15)

5. รายการคำนวณคาน

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam SWB1		

Beam No. SWB1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{+pos.}$	Mu$^+$	=	47185	[kg-m]
$M_{-neg.}$	Mu$^-$	=	30670	[kg-m]
$V_{max.}$	Vu	=	27962	[kg]
	DL	=	2000	[kg/m]
	LL	=	2000	[kg/m]
Length	Ln	=	675	[cm]
width	b	=	30	[cm]
depth	t	=	100	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	95	[cm]
a try	=	6.3985	[cm]
As	=	14.278	[cm^2]
a	=	**6.3989**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	76.897	[cm^2]
a	=	34.464	[cm]
Mu	=	215285	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	14.278	[cm^2]
No. of steel	=	3	
As	=	14.73	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	95	[cm]
a try	=	8.7110	[cm]
As	=	9.399	[cm^2]
a	=	**4.2124**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	100.927	[cm^2]
a	=	45.234	[cm]
Mu	=	262996	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	9.399	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]

Section OK

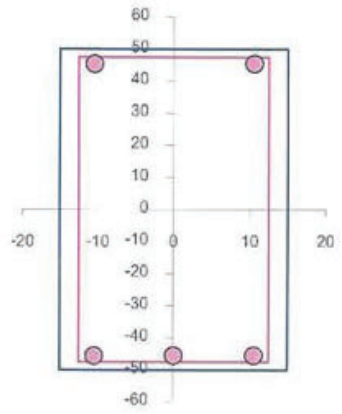
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	25433	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	76.6	[cm]	
S	=	47.500	[cm]	
Used Stirrup	1	DB12 @ 0.400		OK

Check Deflection

Δ all	=	1.88	[cm]		
ρ fy	=	21	<	35	[kg/cm^2]
Y'	=	22.3			
I	=	2143438	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.18 [cm]	<	Δ all	OK
Δ Time	=	0.06	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. SWB1	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam SWB1



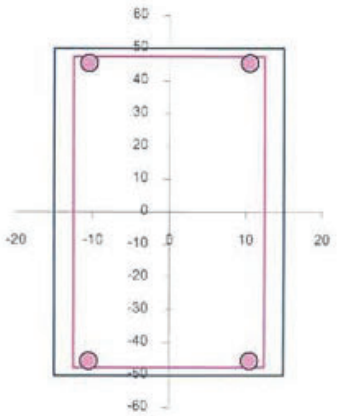
2 DB25

1 DB12 @ 0.400

3 DB25

Positive Section

SWB1 (30 X 100)



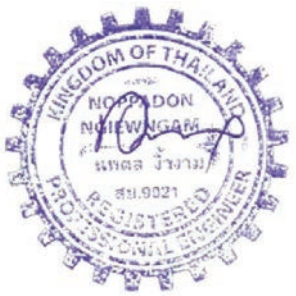
2 DB25

1 DB12 @ 0.400

2 DB25

Negative Section

SWB1 (30 X 100)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

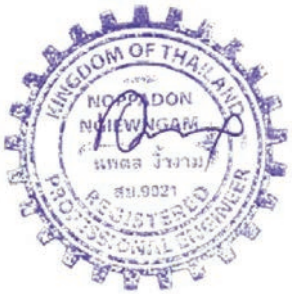
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	15-ม.ค.-21
File	Beam B1

Beam No. B1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	153229	[kg-m]
$M_{neg.}$	Mu$^-$	=	174083	[kg-m]
$V_{max.}$	Vu	=	110000	[kg]
	DL	=	5600	[kg/m]
	LL	=	3800	[kg/m]
Length	Ln	=	1100	[cm]
width	b	=	200	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d''	=	5	[cm]



For Positive Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	60	[cm]
a try	=	4.9749	[cm]
As	=	74.008	[cm^2]
a	=	**4.9753**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	323.777	[cm^2]
a	=	21.767	[cm]
Mu	=	572504	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	74.008	[cm^2]
No. of steel	=	**16**	
As	=	78.54	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	60	[cm]
a try	=	8.7110	[cm]
As	=	84.080	[cm^2]
a	=	**4.9753**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	424.958	[cm^2]
a	=	28.569	[cm]
Mu	=	699381	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	84.080	[cm^2]
No. of steel	=	**18**	
As	=	88.36	[cm^2]

Section OK

Stirrup Design

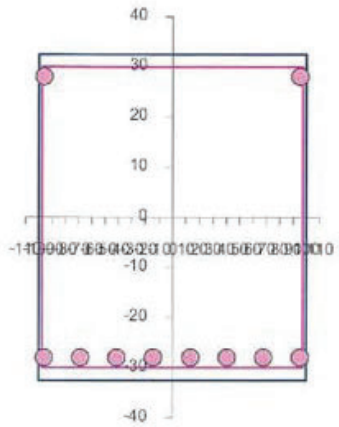
0.53 ϕ (fc')$^{0.5}$ bd	=	107086	[kg]
L=Ln/2(Vu - ϕ Vc)/Vu	=	94.9	[cm]
S	=	30.000	[cm]

Used Stirrup RB 12 @ 0.300 OK

Check Deflection

Δ all	=	3.06	[cm]		
ρ fy	=	26	<	35	[kg/cm^2]
Y'	=	15.6			
I	=	3600000	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	1.75	[cm] <	Δ all	OK
Δ Time	=	2.65	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B1	Date	15-ม.ค.-21
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B1



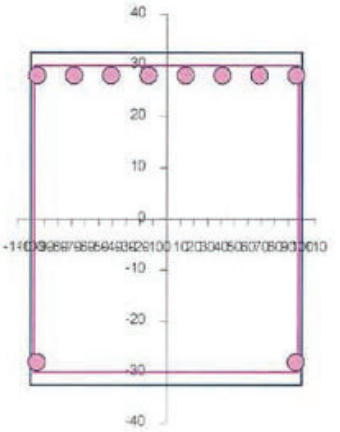
2 DB 25

Stirrup RB 12 @ 0.300

16 DB 25

Positive Section

B1 (200 X 65)



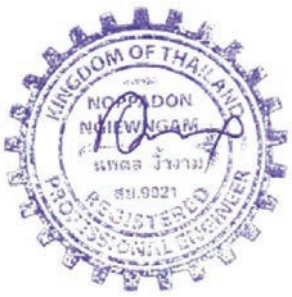
18 DB 25

Stirrup RB 12 @ 0.300

2 DB 25

Negative Section

B1 (200 X 65)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

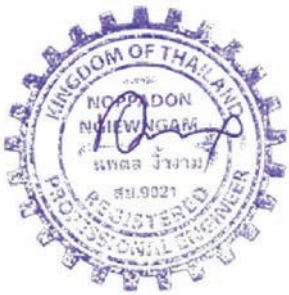
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	07-ธ.ค.-20		
File	Beam B2		

Beam No. B2

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	34398	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	33000	[kg]
	DL	=	1000	[kg/m]
	LL	=	1000	[kg/m]
Length	Ln	=	700	[cm]
width	b	=	60	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB 20
Lower Steel	DB 20
Stirrup Steel	RB 12

d	=	60	[cm]
a try	=	3.6813	[cm]
As	=	16.429	[cm^2]
a	=	**3.6816**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	97.133	[cm^2]
a	=	21.767	[cm]
Mu	=	171751	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]
Σ As	=	16.429	[cm^2]
No. of steel	=	6	
As	=	18.85	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 20
Lower Steel	DB 20
Stirrup Steel	RB 12

d	=	60	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**3.6816**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	127.487	[cm^2]
a	=	28.569	[cm]
Mu	=	209814	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]

Section OK

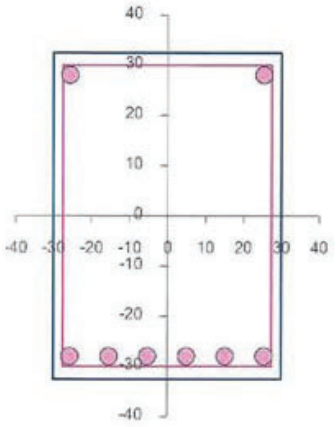
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	32126	[kg]
L=Ln/2(Vu - ϕ Vc)/Vu	=	60.4	[cm]
S	=	30.000	[cm]
Used Stirrup		RB 12 @ 0.300	OK

Check Deflection

Δ all	=	1.94 [cm]		
ρ fy	=	21	<	35 [kg/cm^2]
Y'	=	14.2		
I	=	1080000 [cm^4]		
Ec	=	284116 [kg/cm^2]		
Δ	=	0.20 [cm]	<	Δ all OK
Δ Time	=	0.29	<	Δ all OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B2	Date	07-ธ.ค.-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B2



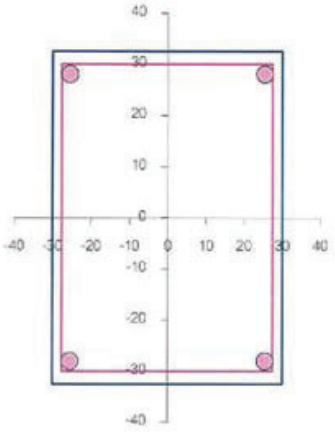
2 DB 20

Stirrup RB 12
@ 0.300

6 DB 20

Positive Section

B2 (60 X 65)



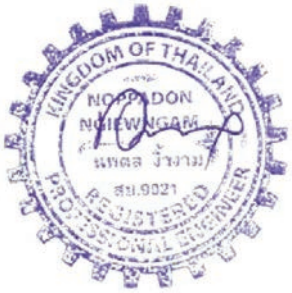
2 DB 20

Stirrup RB 12
@ 0.300

2 DB 20

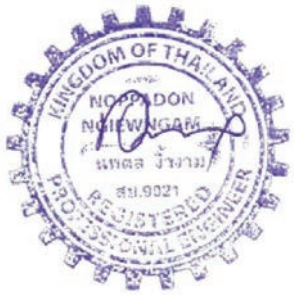
Negative Section

B2 (60 X 65)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B3	Date	07-ธ.ค.-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B3

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	48091	[kg-m]
$M_{neg.}$	Mu$^-$	=	85496	[kg-m]
$V_{max.}$	Vu	=	165000	[kg]
	DL	=	2800	[kg/m]
	LL	=	2000	[kg/m]
Length	Ln	=	900	[cm]
width	b	=	300	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	60	[cm]
a try	=	1.0063	[cm]
As	=	22.453	[cm^2]
a	=	**1.0063**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	485.666	[cm^2]
a	=	21.767	[cm]
Mu	=	858755	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	22.453	[cm^2]
No. of steel	=	**5**	
As	=	24.54	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	60	[cm]
a try	=	8.7110	[cm]
As	=	39.916	[cm^2]
a	=	**1.0063**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	637.436	[cm^2]
a	=	28.569	[cm]
Mu	=	1049071	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	39.916	[cm^2]
No. of steel	=	**9**	
As	=	44.18	[cm^2]

Section OK

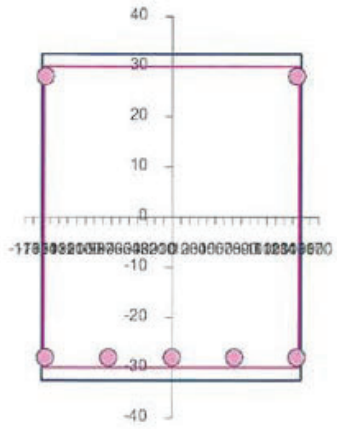
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	160629	[kg]
L=Ln/2(Vu - ϕ Vc)/Vu	=	77.6	[cm]
S	=	30.000	[cm]
Used Stirrup		**RB 12 @ 0.300**	**OK**

Check Deflection

Δ all	=	2.50 [cm]		
ρ fy	=	5 <	35	[kg/cm^2]
Y'	=	7.74		
I	=	5400000	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	0.27 [cm] <	Δ all	**OK**
Δ Time	=	0.4 <	Δ all	**OK**

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B3	Date	07-ธ.ค.-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B3



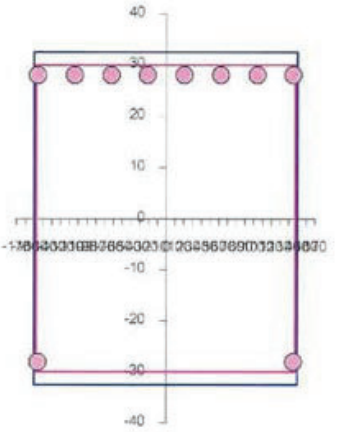
2 DB 25

Stirrup RB 12 @ 0.300

5 DB 25

Positive Section

B3 (300 X 65)



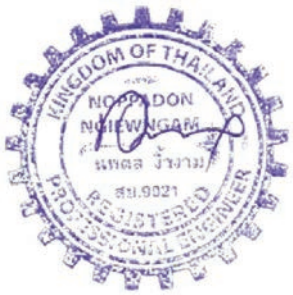
9 DB 25

Stirrup RB 12 @ 0.300

2 DB 25

Negative Section

B3 (300 X 65)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

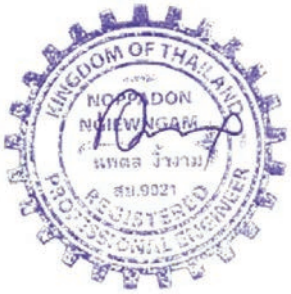
Design Reinforced Concrete Beam
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam B4

Beam No. B4

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
M_{+ve}	Mu$^+$	=	208831	[kg-m]
$M_{-neg.}$	Mu$^-$	=	265938	[kg-m]
$V_{max.}$	Vu	=	159565	[kg]
	DL	=	4200	[kg/m]
	LL	=	2800	[kg/m]
Length	Ln	=	1200	[cm]
width	b	=	200	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	60	[cm]
a try	=	6.8956	[cm]
As	=	102.575	[cm^2]
a	=	**6.8958**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	323.777	[cm^2]
a	=	21.767	[cm]
Mu	=	572504	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	102.575	[cm^2]
No. of steel	=	**17**	
As	=	104.68	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	60	[cm]
a try	=	8.7110	[cm]
As	=	132.757	[cm^2]
a	=	**8.9248**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	424.958	[cm^2]
a	=	28.569	[cm]
Mu	=	699381	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	132.757	[cm^2]
No. of steel	=	**22**	
As	=	135.47	[cm^2]

Section OK

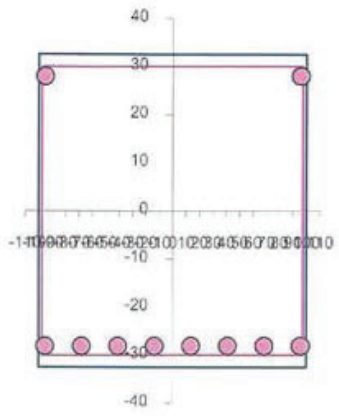
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	107086	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	257.7	[cm]	
S	=	15.827	[cm]	
Used Stirrup	3	DB12 @ 0.150		OK

Check Deflection

Δ all	=	3.33 [cm]		
ρ fy	=	35	< 35	[kg/cm^2]
Y'	=	17.6		
I	=	3600000	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	1.85 [cm]	< Δ all	OK
Δ Time	=	2.81	< Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B4	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B4



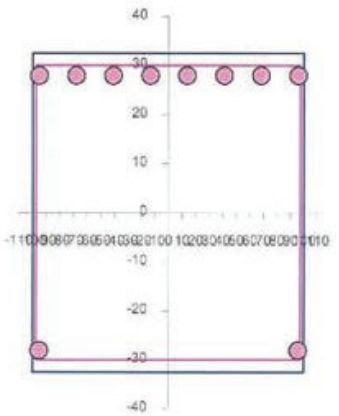
2 DB28

3 DB12 @ 0.150

17 DB28

Positive Section

B4 (200 X 65)



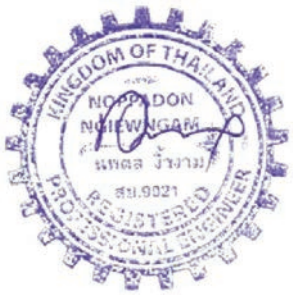
22 DB28

3 DB12 @ 0.150

2 DB28

Negative Section

B4 (200 X 65)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

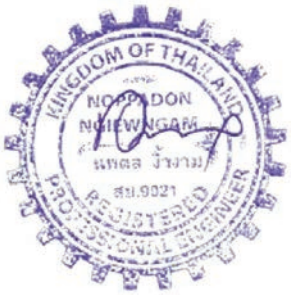
Design Reinforced Concrete Beam
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam B5

Beam No. B5

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	8599	[kg-m]
$M_{-neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	22000	[kg]
	DL	=	1000	[kg/m]
	LL	=	1000	[kg/m]
Length	Ln	=	400	[cm]
width	b	=	40	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB20
Lower Steel	DB20
Stirrup Steel	RB9

	d	=	60	[cm]
	a try	=	1.3532	[cm]
	As	=	4.026	[cm^2]
	a	=	**1.3534**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	0.80	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
	As	=	64.755	[cm^2]
	a	=	21.767	[cm]
	Mu	=	114501	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	2	
	As	=	6.28	[cm^2]
	Σ As	=	4.026	[cm^2]
No. of steel		=	2	
	As	=	6.28	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB20
Lower Steel	DB20
Stirrup Steel	RB9

	d	=	60	[cm]
	a try	=	8.7110	[cm]
	As	=	0.000	[cm^2]
	a	=	**0.0000**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	1.05	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0472	
	ρ_{max}	=	0.0354	
	As	=	84.992	[cm^2]
	a	=	28.569	[cm]
	Mu	=	139876	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	2	
	As	=	6.28	[cm^2]
	Σ As	=	0.000	[cm^2]
No. of steel		=	2	
	As	=	6.28	[cm^2]

Section OK

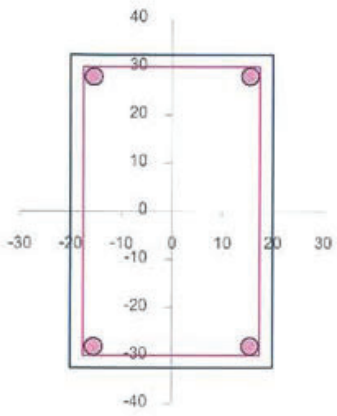
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	21417	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	34.5	[cm]	
S	=	30.000	[cm]	
Used Stirrup	1	RB9 @ 0.300		OK

Check Deflection

Δ all	=	1.11	[cm]		
ρ fy	=	10	<	35	[kg/cm^2]
Y'	=	10.4			
I	=	720000	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.03 [cm]	<	Δ all	OK
Δ Time	=	0.02	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B5	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B5



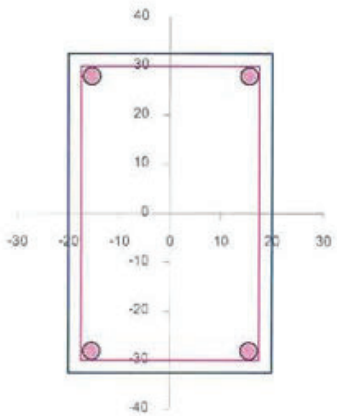
2 DB20

1 RB9 @ 0.300

2 DB20

Positive Section

B5 (40 X 65)



2 DB20

1 RB9 @ 0.300

2 DB20

Negative Section

B5 (40 X 65)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

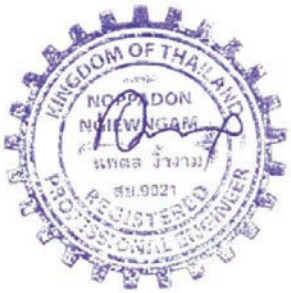
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam B6

Beam No. B6

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	158724	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	80902	[kg]
	DL	=	6000	[kg/m]
	LL	=	4000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	40	[cm]
depth	t	=	80	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	75	[cm]
a try	=	23.4155	[cm]
As	=	69.661	[cm^2]
a	=	**23.4155**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	80.944	[cm^2]
a	=	27.208	[cm]
Mu	=	178907	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	69.661	[cm^2]
No. of steel	=	**12**	
As	=	73.89	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	75	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	106.239	[cm^2]
a	=	35.711	[cm]
Mu	=	218556	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	26772	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	287.5	[cm]	
S	=	12.787	[cm]	
Used Stirrup	**2**	**DB12 @ 0.125**		**OK**

Check Deflection

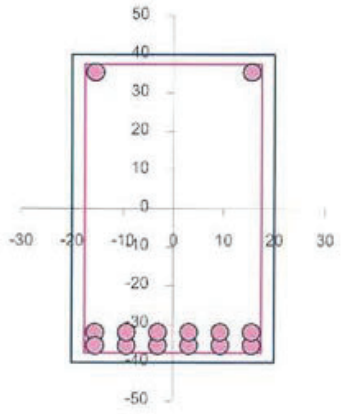
Δ all	=	2.22	[cm]		
ρ fy	=	99	>	35	[kg/cm^2]
Y'	=	33			
I	=	1391555	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	1.35 [cm]	<	Δ all	**OK**
Δ Time	=	2.05	<	Δ all	**OK**

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Beam No.	B6	Date	7-Dec-20
Project :	สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B6



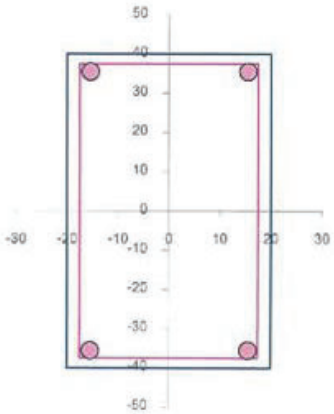
2 DB28

2 DB12 @ 0.125

12 DB28

Positive Section

B6 (40 X 80)



2 DB28

2 DB12 @ 0.125

2 DB28

Negative Section

B6 (40 X 80)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	15-Jan-21
File	Beam B7

Beam No. B7

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	177565	[kg-m]
$M_{-neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	90149.3	[kg]
	DL	=	4500	[kg/m]
	LL	=	3000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	70	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

	d	=	65	[cm]
	a try	=	0.0000	[cm]
	As	=	0.000	[cm^2]
	a	=	**0.0000**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	0.80	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
	As	=	87.690	[cm^2]
	a	=	23.580	[cm]
	Mu	=	167974	[kg-m]
	M1	=	9590	[kg-m]
	As'	=	4.4400	[cm^2]
No. of steel		=	**2**	
	As	=	12.32	[cm^2]
	Σ As	=	92.130	[cm^2]
No. of steel		=	**15**	
	As	=	92.36	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

	d	=	65	[cm]
	a try	=	8.7110	[cm]
	As	=	0.000	[cm^2]
	a	=	**0.0000**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	1.05	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0472	
	ρ_{max}	=	0.0354	
	As	=	115.093	[cm^2]
	a	=	30.949	[cm]
	Mu	=	205200	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	**2**	
	As	=	12.32	[cm^2]
	Σ As	=	0.000	[cm^2]
No. of steel		=	**2**	
	As	=	12.32	[cm^2]

Section OK

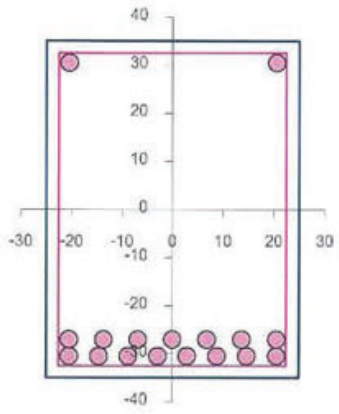
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	29003	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	290.6	[cm]	
S	=	14.715	[cm]	
Used Stirrup	3	DB12 @ 0.125		OK

Check Deflection

Δ all	=	2.22 [cm]			
ρ fy	=	114	>	35	[kg/cm^2]
Y'	=	30.1			
I	=	1242003	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	1.13 [cm]	<	Δ all	OK
Δ Time	=	1.72	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B7	Date	15-Jan-21
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B7



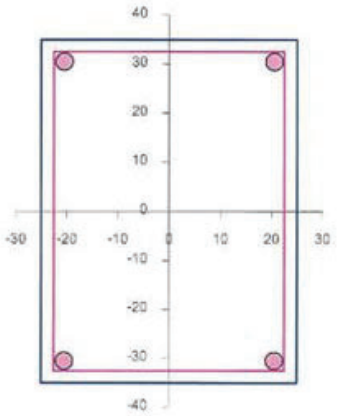
2 DB28

3 DB12 @ 0.125

15 DB28

Positive Section

B7 (50 X 70)



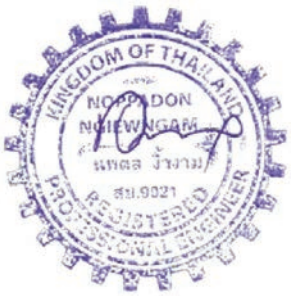
2 DB28

3 DB12 @ 0.125

2 DB28

Negative Section

B7 (50 X 70)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

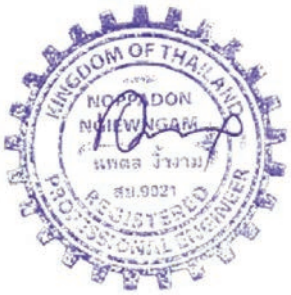
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam B8		

Beam No. B8

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	9861	[kg-m]
$M_{-neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	13000	[kg]
	DL	=	1000	[kg/m]
	LL	=	1000	[kg/m]
Length	Ln	=	400	[cm]
width	b	=	25	[cm]
depth	t	=	60	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB20
Lower Steel	DB20
Stirrup Steel	RB9

d	=	55	[cm]
a try	=	2.7465	[cm]
As	=	5.108	[cm^2]
a	=	**2.7471**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	37.099	[cm^2]
a	=	19.953	[cm]
Mu	=	60133	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]
Σ As	=	5.108	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB20
Lower Steel	DB20
Stirrup Steel	RB9

d	=	55	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	48.693	[cm^2]
a	=	26.188	[cm]
Mu	=	73459	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]

Section OK

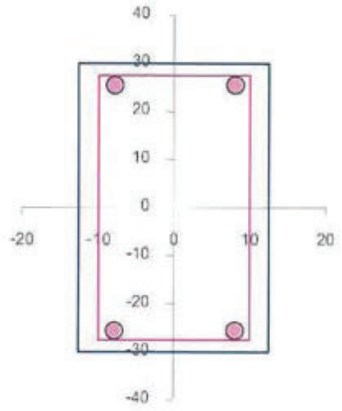
Stirrup Design

$0.53\,\phi\,(fc')^{0.5}\,bd$	=	12270	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	39.5	[cm]	
S	=	27.500	[cm]	
Used Stirrup	1	RB9 @ 0.250		OK

Check Deflection

Δ all	=	1.11 [cm]			
ρ fy	=	18	<	35	[kg/cm^2]
Y'	=	12.3			
I	=	346615	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.07 [cm]	<	Δ all	OK
Δ Time	=	0.04	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B8	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B8



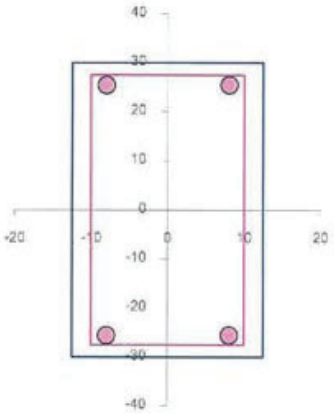
2 DB20

1 RB9 @ 0.250

2 DB20

Positive Section

B8 (25 X 60)



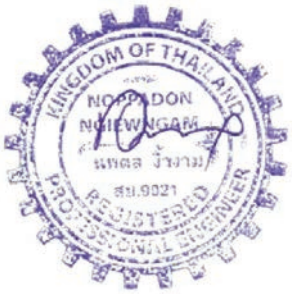
2 DB20

1 RB9 @ 0.250

2 DB20

Negative Section

B8 (25 X 60)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam B9

Beam No. B9

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	2000	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	50000	[kg]
	DL	=	1000	[kg/m]
	LL	=	1000	[kg/m]
Length	Ln	=	150	[cm]
width	b	=	100	[cm]
depth	t	=	60	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB20
Lower Steel	DB20
Stirrup Steel	RB9

	d	=	55	[cm]
	a try	=	0.1360	[cm]
	As	=	1.011	[cm^2]
	a	=	**0.1360**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	0.80	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
	As	=	148.398	[cm^2]
	a	=	19.953	[cm]
	Mu	=	240531	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]
	Σ As	=	1.011	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB20
Lower Steel	DB20
Stirrup Steel	RB9

	d	=	55	[cm]
	a try	=	8.7110	[cm]
	As	=	0.000	[cm^2]
	a	=	**0.0000**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	1.05	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0472	
	ρ_{max}	=	0.0354	
	As	=	194.772	[cm^2]
	a	=	26.188	[cm]
	Mu	=	293837	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]
	Σ As	=	0.000	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]

Section OK

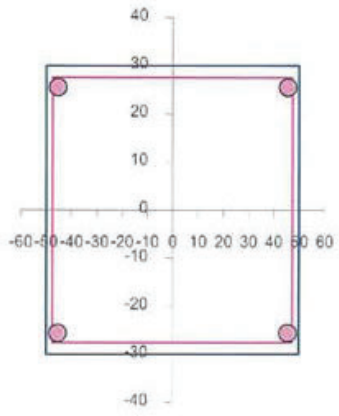
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	49081	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	12.4	[cm]	
S	=	27.500	[cm]	
Used Stirrup	1	RB9 @ 0.250		OK

Check Deflection

Δ all	=	0.42	[cm]		
ρ fy	=	5	< 35	[kg/cm^2]	
Y'	=	6.53			
I	=	1386458	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.00	[cm] <	Δ all	OK
Δ Time	=	0	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B9	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B9



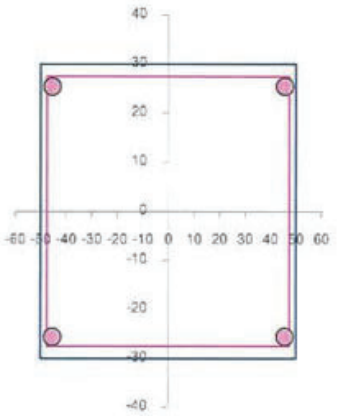
2 DB20

1 RB9
@ 0.250

2 DB20

Positive Section

B9 (100 X 60)



2 DB20

1 RB9
@ 0.250

2 DB20

Negative Section

B9 (100 X 60)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam B10

Beam No. B10

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{+pos.}$	Mu$^+$	=	47796	[kg-m]
$M_{-neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	38422	[kg]
	DL	=	2500	[kg/m]
	LL	=	1700	[kg/m]
Length	Ln	=	520	[cm]
width	b	=	30	[cm]
depth	t	=	60	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

	d	=	55	[cm]
	a try	=	12.1635	[cm]
	As	=	27.141	[cm^2]
	a	=	**12.1638**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	0.80	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
	As	=	44.519	[cm^2]
	a	=	19.953	[cm]
	Mu	=	72159	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	2	
	As	=	9.82	[cm^2]
	Σ As	=	27.141	[cm^2]
No. of steel		=	6	
	As	=	29.45	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

	d	=	55	[cm]
	a try	=	8.7110	[cm]
	As	=	0.000	[cm^2]
	a	=	**0.0000**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	1.05	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0472	
	ρ_{max}	=	0.0354	
	As	=	58.432	[cm^2]
	a	=	26.188	[cm]
	Mu	=	88151	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	2	
	As	=	9.82	[cm^2]
	Σ As	=	0.000	[cm^2]
No. of steel		=	2	
	As	=	9.82	[cm^2]

Section OK

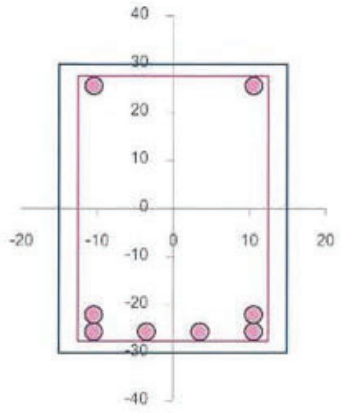
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd		=	14724	[kg]
L=Ln/2(Vu - ϕ Vc)/Vu		=	175.3	[cm]
	S	=	21.419	[cm]
Used Stirrup	2	DB12 @	0.200	OK

Check Deflection

Δ all	=	1.44 [cm]		
ρ fy	=	71	> 35	[kg/cm^2]
Y'	=	21.5		
I	=	330754	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	0.43 [cm]	< Δ all	OK
Δ Time	=	0.64	< Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B10	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B10



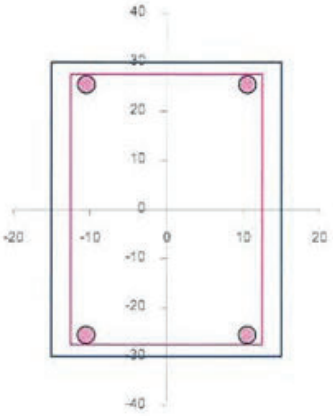
2 DB25

2 DB12 @ 0.200

6 DB25

Positive Section

B10 (30 X 60)



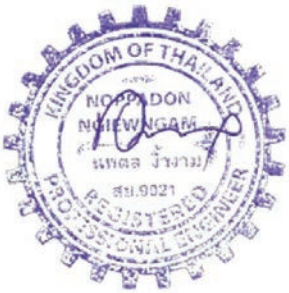
2 DB25

2 DB12 @ 0.200

2 DB25

Negative Section

B10 (30 X 60)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

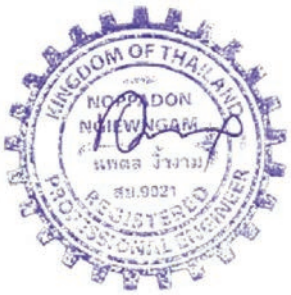
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam B11		

Beam No. B11

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	167078	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	85160	[kg]
	DL	=	5000	[kg/m]
	LL	=	3000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	60	[cm]
a try	=	0.0000	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	80.944	[cm^2]
a	=	21.767	[cm]
Mu	=	143126	[kg-m]
M1	=	23952	[kg-m]
As'	=	12.0970	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	93.041	[cm^2]
No. of steel	=	**16**	
As	=	98.52	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	60	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	106.239	[cm^2]
a	=	28.569	[cm]
Mu	=	174845	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

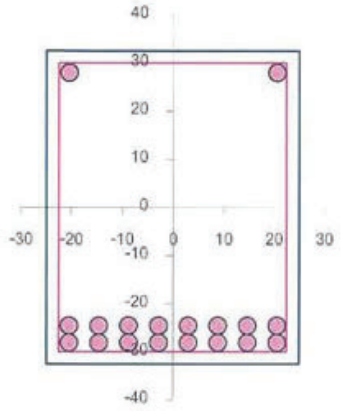
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	26772	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	293.1	[cm]	
S	=	14.225	[cm]	
Used Stirrup	3	DB12 @ 0.125		OK

Check Deflection

Δ all	=	2.22 [cm]		
ρ fy	=	131	> 35	[kg/cm^2]
Y'	=	29.2		
I	=	1069171	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	1.40 [cm]	< Δ all	OK
Δ Time	=	2.18	< Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B11	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B11



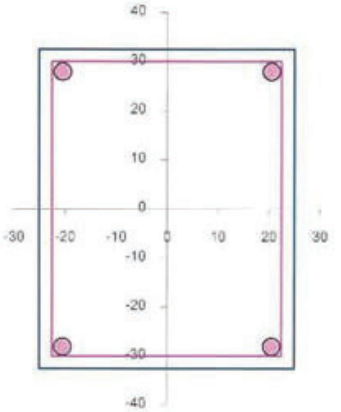
2 DB28

3 DB12 @ 0.125

16 DB28

Positive Section

B11 (50 X 65)



2 DB28

3 DB12 @ 0.125

2 DB28

Negative Section

B11 (50 X 65)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam B12		

Beam No. B12

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{+pos.}$	Mu$^+$	=	74720	[kg-m]
$M_{-neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	37360	[kg]
	DL	=	2500	[kg/m]
	LL	=	1700	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	65	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	60	[cm]
a try	=	10.1628	[cm]
As	=	37.793	[cm^2]
a	=	**10.1629**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	80.944	[cm^2]
a	=	21.767	[cm]
Mu	=	143126	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	37.793	[cm^2]
No. of steel	=	8	
As	=	39.27	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	60	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	106.239	[cm^2]
a	=	28.569	[cm]
Mu	=	174845	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]

Section OK

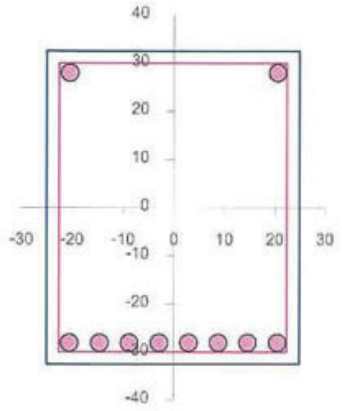
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	26772	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	156.4	[cm]	
S	=	26.148	[cm]	
Used Stirrup	1	DB12 @ 0.250		OK

Check Deflection

Δ all	=	2.22 [cm]			
ρ fy	=	52	>	35	[kg/cm^2]
Y'	=	20.8			
I	=	572387	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	1.38 [cm]	<	Δ all	OK
Δ Time	=	2.09	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B12	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B12



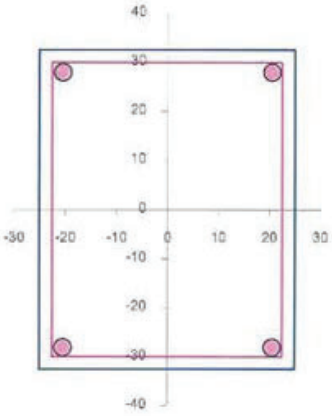
2 DB25

1 DB12 @ 0.250

8 DB25

Positive Section

B12 (50 X 65)



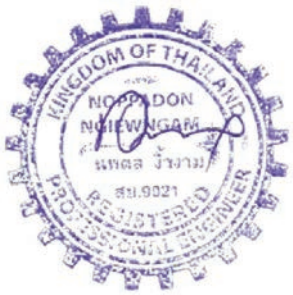
2 DB25

1 DB12 @ 0.250

2 DB25

Negative Section

B12 (50 X 65)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam B13		

Beam No. B13

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	211360	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	86680	[kg]
	DL	=	6000	[kg/m]
	LL	=	4000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	90	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	85	[cm]
a try	=	21.2235	[cm]
As	=	78.925	[cm^2]
a	=	**21.2236**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	114.671	[cm^2]
a	=	30.836	[cm]
Mu	=	287246	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	78.925	[cm^2]
No. of steel	=	**13**	
As	=	80.05	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	85	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	150.506	[cm^2]
a	=	40.472	[cm]
Mu	=	350904	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

Stirrup Design

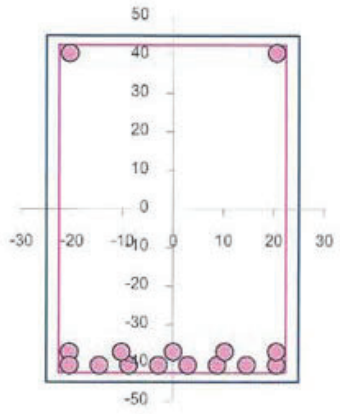
0.53 ϕ $(fc')^{0.5}$ bd	=	37926	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	251.2	[cm]	
S	=	16.090	[cm]	
Used Stirrup	**2**	**DB12**	**@**	**0.150**

OK

Check Deflection

Δ all	=	2.22 [cm]		
p fy	=	75	>	35 [kg/cm^2]
Y'	=	33.9		
I	=	2112450	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	0.89 [cm]	<	Δ all **OK**
Δ Time	=	1.35	<	Δ all **OK**

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B13	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B13



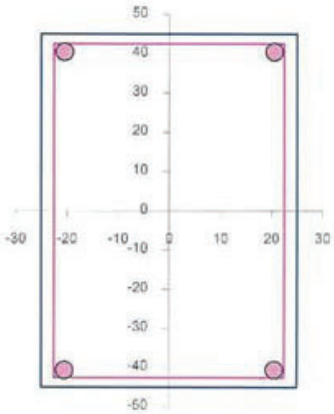
2 DB28

2 DB12 @ 0.150

13 DB28

Positive Section

B13 (50 X 90)



2 DB28

2 DB12 @ 0.150

2 DB28

Negative Section

B13 (50 X 90)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam B14		

Beam No. B14

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	317832	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	95266	[kg]
	DL	=	6000	[kg/m]
	LL	=	4000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	90	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	85	[cm]
a try	=	0.0000	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	114.671	[cm^2]
a	=	30.836	[cm]
Mu	=	287246	[kg-m]
M1	=	30586	[kg-m]
As'	=	10.6202	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	125.291	[cm^2]
No. of steel	=	**21**	
As	=	129.31	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	85	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	150.506	[cm^2]
a	=	40.472	[cm]
Mu	=	350904	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

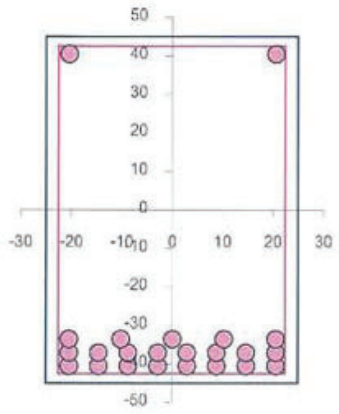
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	37926	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	264.6	[cm]	
S	=	13.681	[cm]	
Used Stirrup	**2**	**DB12 @ 0.125**		OK

Check Deflection

Δ all	=	2.22 [cm]			
ρ fy	=	122	>	35	[kg/cm^2]
Y'	=	40.3			
I	=	2899423	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.65 [cm]	<	Δ all	OK
Δ Time	=	0.98	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B14	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B14



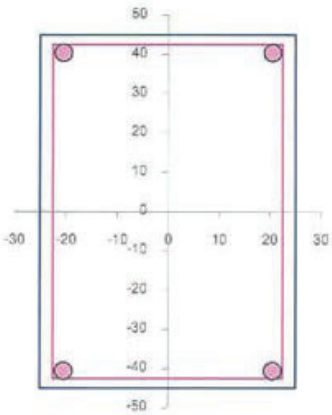
2 DB28

2 DB12 @ 0.125

21 DB28

Positive Section

B14 (50 X 90)



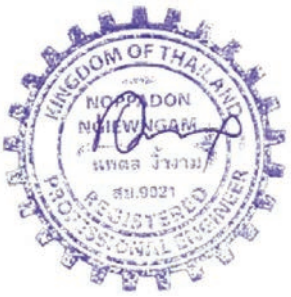
2 DB28

2 DB12 @ 0.125

2 DB28

Negative Section

B14 (50 X 90)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

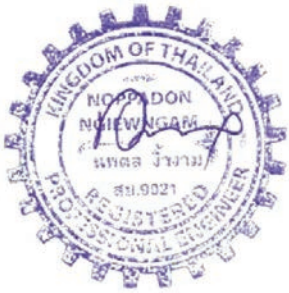
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	15-Jan-21		
File	Beam B15		

Beam No. B15

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	74672	[kg-m]
$M_{neg.}$	Mu$^-$	=	124531	[kg-m]
$V_{max.}$	Vu	=	63959	[kg]
	DL	=	2000	[kg/m]
	LL	=	3000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	40	[cm]
depth	t	=	70	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	65	[cm]
a try	=	11.7969	[cm]
As	=	35.096	[cm^2]
a	=	**11.7969**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	70.152	[cm^2]
a	=	23.580	[cm]
Mu	=	134379	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	35.096	[cm^2]
No. of steel	=	**8**	
As	=	39.27	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	65	[cm]
a try	=	8.7110	[cm]
As	=	57.040	[cm^2]
a	=	**19.1733**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	92.074	[cm^2]
a	=	30.949	[cm]
Mu	=	164160	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	57.040	[cm^2]
No. of steel	=	**12**	
As	=	58.90	[cm^2]

Section OK

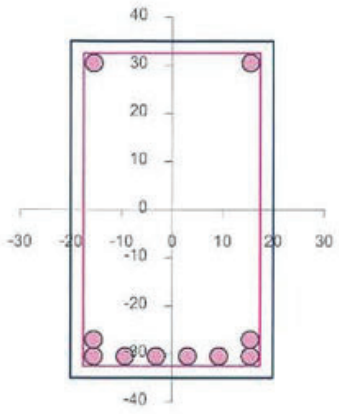
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	23202	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	276.7	[cm]	
S	=	14.718	[cm]	
Used Stirrup	2	DB12 @ 0.125		OK

Check Deflection

Δ all	=	2.22 [cm]		
ρ fy	=	60	> 35	[kg/cm^2]
Y'	=	23.8		
I	=	646358	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	1.45 [cm]	< Δ all	OK
Δ Time	=	1.86	< Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B15	Date	15-Jan-21
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B15



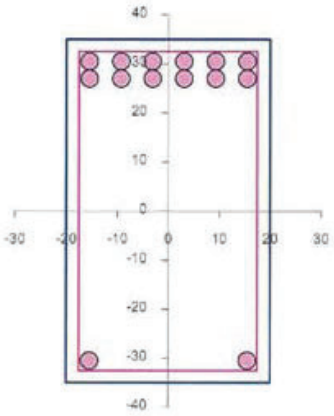
2 DB25

2 DB12 @ 0.125

8 DB25

Positive Section

B15 (40 X 70)



12 DB25

2 DB12 @ 0.125

2 DB25

Negative Section

B15 (40 X 70)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

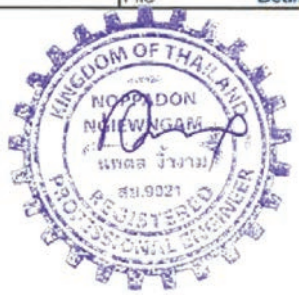
Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam B16

Beam No. B16

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)



	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	25312	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	22500	[kg]
	DL	=	2000	[kg/m]
	LL	=	1000	[kg/m]
Length	Ln	=	450	[cm]
width	b	=	30	[cm]
depth	t	=	60	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel DB20

Lower Steel DB20

Stirrup Steel RB9

	d	=	55	[cm]
	a try	=	6.0637	[cm]
	As	=	13.530	[cm^2]
	a	=	**6.0637**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	0.80	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0360	
	ρ_{max}	=	0.0270	
	As	=	44.519	[cm^2]
	a	=	19.953	[cm]
	Mu	=	72159	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]
	Σ As	=	13.530	[cm^2]
No. of steel		=	**5**	
	As	=	15.71	[cm^2]

Section OK

For Negative Moment :

Upper Steel DB20

Lower Steel DB20

Stirrup Steel RB9

	d	=	55	[cm]
	a try	=	8.7110	[cm]
	As	=	0.000	[cm^2]
	a	=	**0.0000**	[cm]
	ϕ Bend	=	0.90	
	ϕ Shea	=	0.85	
	k1	=	1.05	
	Es	=	2040000	[kg/cm^2]
	Ec	=	284553	[kg/cm^2]
	n	=	7	
	ρ	=	0.0472	
	ρ_{max}	=	0.0354	
	As	=	58.432	[cm^2]
	a	=	26.188	[cm]
	Mu	=	88151	[kg-m]
	M1	=	0	[kg-m]
	As'	=	0.0000	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]
	Σ As	=	0.000	[cm^2]
No. of steel		=	**2**	
	As	=	6.28	[cm^2]

Section OK

Stirrup Design

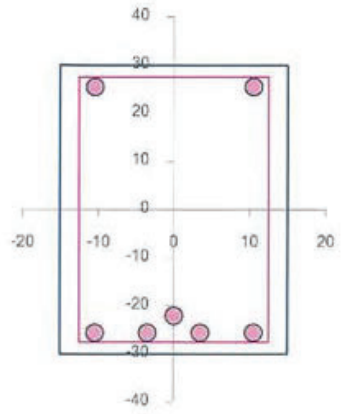
0.53 ϕ (fc')$^{0.5}$ bd	=	14724	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	99.8	[cm]	
S	=	18.360	[cm]	
Used Stirrup	**1**	**RB9**	**@**	**0.175**

OK

Check Deflection

Δ all	=	1.25	[cm]		
ρ fy	=	38	>	35	[kg/cm^2]
Y'	=	16.7			
I	=	207866	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.27 [cm]	<	Δ all	OK
Δ Time	=	0.43	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B16	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B16



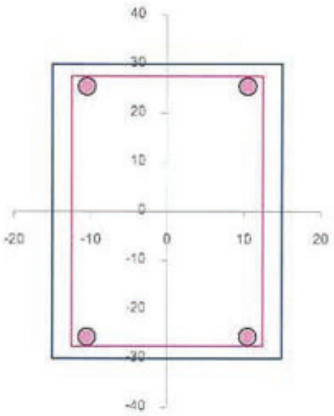
2 DB20

1 RB9 @ 0.175

5 DB20

Positive Section

B16 (30 X 60)



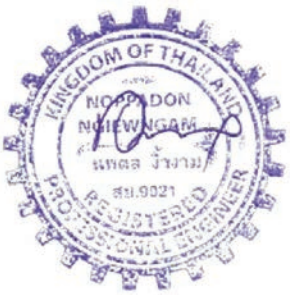
2 DB20

1 RB9 @ 0.175

2 DB20

Negative Section

B16 (30 X 60)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

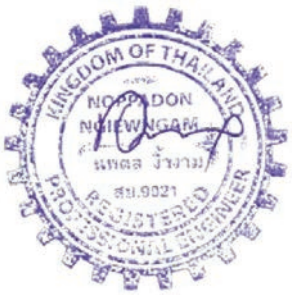
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam 3B8		

Beam No. 3B8

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	10351	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	13000	[kg]
	DL	=	1000	[kg/m]
	LL	=	1000	[kg/m]
Length	Ln	=	400	[cm]
width	b	=	25	[cm]
depth	t	=	60	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel: DB20
Lower Steel: DB20
Stirrup Steel: RB9

d	=	55	[cm]
a try	=	2.7465	[cm]
As	=	5.362	[cm^2]
a	=	**2.8836**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	37.099	[cm^2]
a	=	19.953	[cm]
Mu	=	60133	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]
Σ As	=	5.362	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]

Section OK

For Negative Moment :

Upper Steel: DB20
Lower Steel: DB20
Stirrup Steel: RB9

d	=	55	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	48.693	[cm^2]
a	=	26.188	[cm]
Mu	=	73459	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	6.28	[cm^2]

Section OK

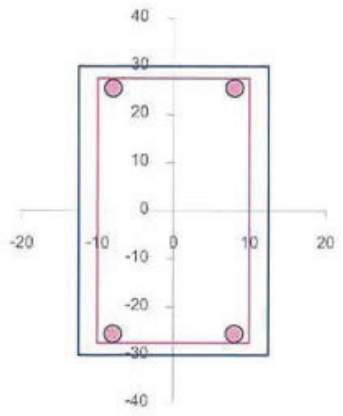
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	12270	[kg]		
L=Ln/2(Vu - ϕ Vc)/Vu	=	39.5	[cm]		
S	=	27.500	[cm]		
Used Stirrup	1	RB9	@	0.250	OK

Check Deflection

Δ all	=	1.11	[cm]		
ρ fy	=	18	<	35	[kg/cm^2]
Y'	=	12.3			
I	=	346615	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.07 [cm]	<	Δ all	OK
Δ Time	=	0.04	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 3B8	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 3B8



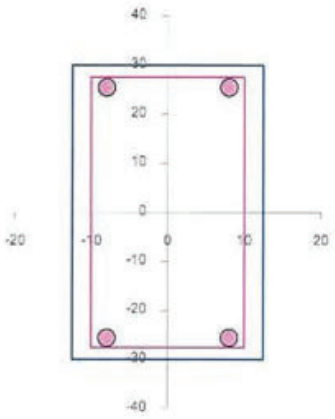
2 DB20

1 RB9 @ 0.250

2 DB20

Positive Section

3B8 (25 X 60)



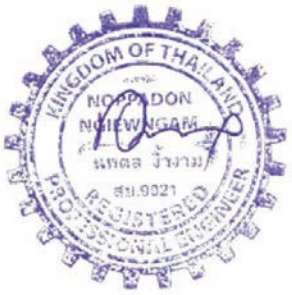
2 DB20

1 RB9 @ 0.250

2 DB20

Negative Section

3B8 (25 X 60)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

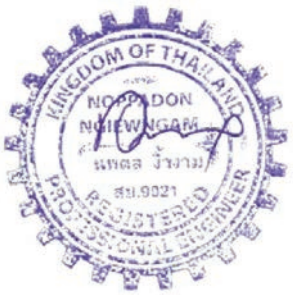
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018
Rev.	1
Initial	Noppadon N.
Date	7-Dec-20
File	Beam 3B10

Beam No. 3B10

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	47796	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	38422	[kg]
	DL	=	2500	[kg/m]
	LL	=	1700	[kg/m]
Length	Ln	=	520	[cm]
width	b	=	30	[cm]
depth	t	=	60	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	55	[cm]
a try	=	12.1635	[cm]
As	=	27.141	[cm^2]
a	=	**12.1638**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	44.519	[cm^2]
a	=	19.953	[cm]
Mu	=	72159	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	27.141	[cm^2]
No. of steel	=	**6**	
As	=	29.45	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	55	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	58.432	[cm^2]
a	=	26.188	[cm]
Mu	=	88151	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]

Section OK

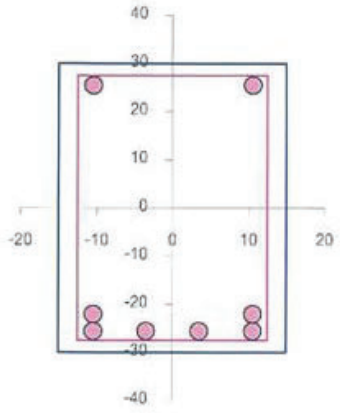
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	14724	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	175.3	[cm]	
S	=	21.419	[cm]	
Used Stirrup	2	DB12 @ 0.200		OK

Check Deflection

Δ all	=	1.44 [cm]		
ρ fy	=	71	> 35	[kg/cm^2]
Y'	=	21.5		
I	=	330754	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	0.43 [cm]	< Δ all	OK
Δ Time	=	0.64	< Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 3B10	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 3B10



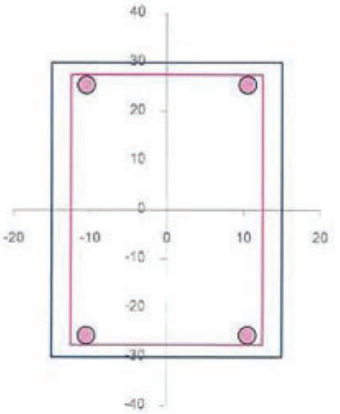
2 DB25

2 DB12 @ 0.200

6 DB25

Positive Section

3B10 (30 X 60)



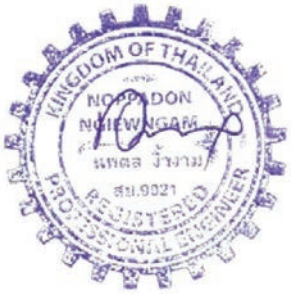
2 DB25

2 DB12 @ 0.200

2 DB25

Negative Section

3B10 (30 X 60)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

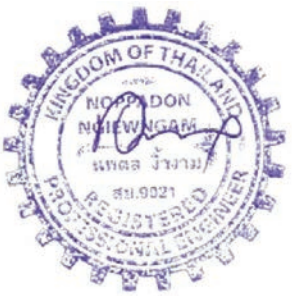
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam 3B11		

Beam No. 3B11

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	197684	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	98586	[kg]
	DL	=	5000	[kg/m]
	LL	=	3000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	80	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	75	[cm]
a try	=	23.3112	[cm]
As	=	86.688	[cm^2]
a	=	**23.3112**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	101.180	[cm^2]
a	=	27.208	[cm]
Mu	=	223634	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	86.688	[cm^2]
No. of steel	=	**15**	
As	=	92.36	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	75	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	132.799	[cm^2]
a	=	35.711	[cm]
Mu	=	273196	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

Stirrup Design

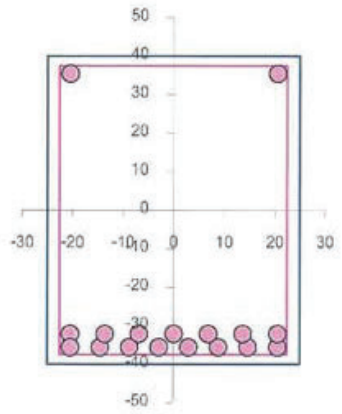
0.53 ϕ $(fc')^{0.5}$ bd	=	33464	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	284.6	[cm]	
S	=	15.943	[cm]	
Used Stirrup	3	DB12 @ 0.150		OK

Check Deflection

Δ all	=	2.22	[cm]		
ρ fy	=	99	>	35	[kg/cm^2]
Y'	=	33			
I	=	1739444	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.86 [cm]	<	Δ all	OK
Δ Time	=	1.34	<	Δ all	OK

GCDT
GOVERNMENT CENTER DESIGN TEAM

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 3B11	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 3B11



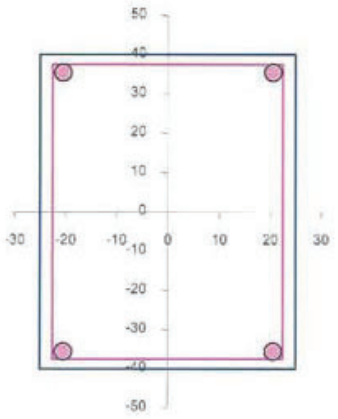
2 DB28

3 DB12 @ 0.150

15 DB28

Positive Section

3B11 (50 X 80)



2 DB28

3 DB12 @ 0.150

2 DB28

Negative Section

3B11 (50 X 80)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

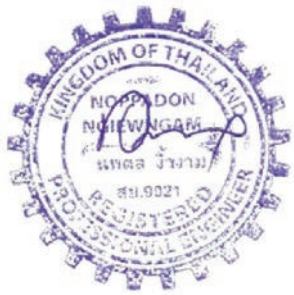
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam 3B12		

Beam No. 3B12

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	155411	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	75930	[kg]
	DL	=	2500	[kg/m]
	LL	=	1700	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	50	[cm]
depth	t	=	70	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	65	[cm]
a try	=	21.3730	[cm]
As	=	79.482	[cm^2]
a	=	**21.3734**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	87.690	[cm^2]
a	=	23.580	[cm]
Mu	=	167974	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	79.482	[cm^2]
No. of steel	=	**13**	
As	=	80.05	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	65	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	115.093	[cm^2]
a	=	30.949	[cm]
Mu	=	205200	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

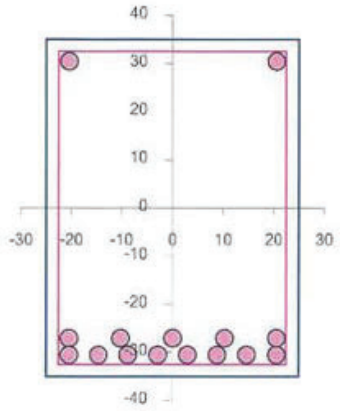
Stirrup Design

$0.53\,\phi\,(fc')^{0.5}\,bd$	=	29003	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	270.1	[cm]	
S	=	12.783	[cm]	
Used Stirrup	**2**	**DB12 @ 0.125**		OK

Check Deflection

Δ all	=	2.22 [cm]			
ρ fy	=	99	>	35	[kg/cm^2]
Y'	=	28.6			
I	=	1132314	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.70 [cm]	<	Δ all	OK
Δ Time	=	1.05	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 3B12	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 3B12



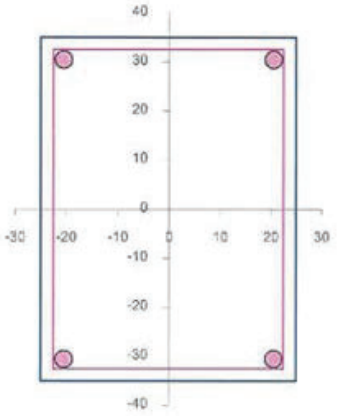
2 DB28

2 DB12 @ 0.125

13 DB28

Positive Section

3B12 (50 X 70)



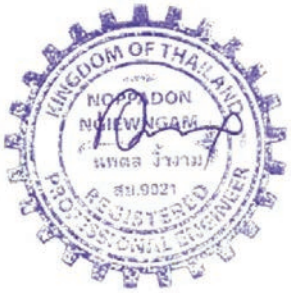
2 DB28

2 DB12 @ 0.125

2 DB28

Negative Section

3B12 (50 X 70)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

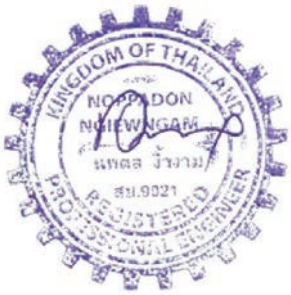
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam 3B14		

Beam No. 3B14

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	387250	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	179800	[kg]
	DL	=	2500	[kg/m]
	LL	=	1700	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	100	[cm]
depth	t	=	100	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel: DB28
Lower Steel: DB28
Stirrup Steel: DB12

d	=	95	[cm]
a try	=	16.6905	[cm]
As	=	124.136	[cm^2]
a	=	**16.6905**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
$ρ_{max}$	=	0.0270	
As	=	256.324	[cm^2]
a	=	34.464	[cm]
Mu	=	717617	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	124.136	[cm^2]
No. of steel	=	**21**	
As	=	129.31	[cm^2]

Section OK

For Negative Moment :

Upper Steel: DB28
Lower Steel: DB28
Stirrup Steel: DB12

d	=	95	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**0.0000**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
$ρ_{max}$	=	0.0354	
As	=	336.425	[cm^2]
a	=	45.234	[cm]
Mu	=	876654	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]

Section OK

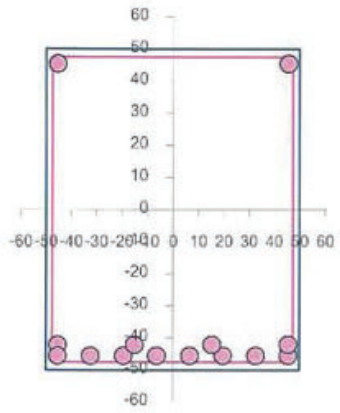
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	84777	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	239.7	[cm]	
S	=	13.840	[cm]	
Used Stirrup	3	DB12 @ 0.125		OK

Check Deflection

Δ all	=	2.22 [cm]			
ρ fy	=	54	>	35	[kg/cm^2]
Y'	=	33.4			
I	=	4676658	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.17 [cm]	<	Δ all	OK
Δ Time	=	0.26	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.		
Design Reinforced Concrete Beam	Job No.	193018
(Ultimate Stress Design method in accordance with ACI 318-89)	Rev.	1
	Initial	Noppadon N.
Beam No. 3B14	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 3B14



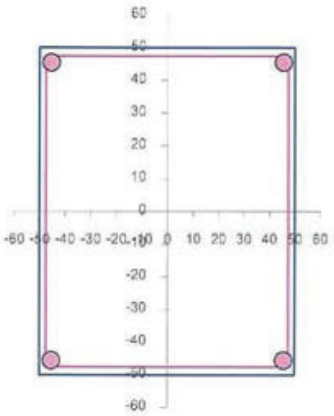
2 DB28

3 DB12 @ 0.125

21 DB28

Positive Section

3B14 (100 X 100)



2 DB28

3 DB12 @ 0.125

2 DB28

Negative Section

3B14 (100 X 100)

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

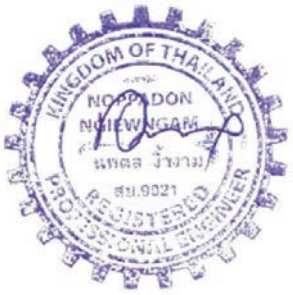
(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam 3B14		

Beam No. 3B15

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	168948	[kg-m]
$M_{-neg.}$	Mu$^-$	=	247896	[kg-m]
$V_{max.}$	Vu	=	132435	[kg]
	DL	=	2500	[kg/m]
	LL	=	1700	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	100	[cm]
depth	t	=	100	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	95	[cm]
a try	=	6.8920	[cm]
As	=	51.259	[cm^2]
a	=	**6.8920**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
$ρ_{max}$	=	0.0270	
As	=	256.324	[cm^2]
a	=	34.464	[cm]
Mu	=	717617	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	51.259	[cm^2]
No. of steel	=	**9**	
As	=	55.42	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB28
Lower Steel	DB28
Stirrup Steel	DB12

d	=	95	[cm]
a try	=	8.7110	[cm]
As	=	75.967	[cm^2]
a	=	**10.2141**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
$ρ_{max}$	=	0.0354	
As	=	336.425	[cm^2]
a	=	45.234	[cm]
Mu	=	876654	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	12.32	[cm^2]
Σ As	=	75.967	[cm^2]
No. of steel	=	**13**	
As	=	80.05	[cm^2]

Section OK

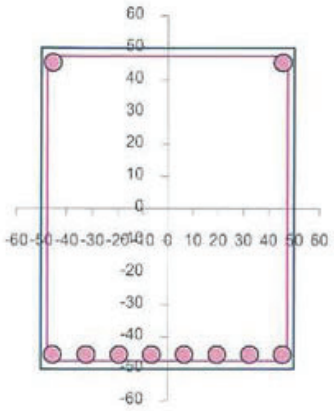
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	84777	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	182.4	[cm]	
S	=	27.594	[cm]	
Used Stirrup	3	DB12 @ 0.250		OK

Check Deflection

Δ all	=	2.22 [cm]		
ρ fy	=	23	< 35	[kg/cm^2]
Y'	=	23.5		
I	=	7144792	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	0.11 [cm]	<	Δ all OK
Δ Time	=	0.17	<	Δ all OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 3B15	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 3B14



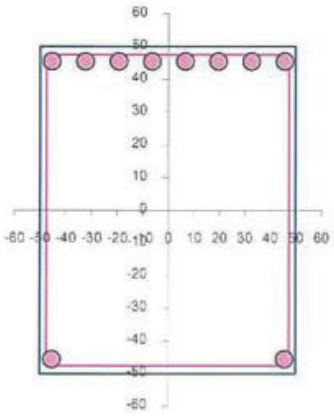
2 DB28

3 DB12 @ 0.250

9 DB28

Positive Section

3B15 (100 X 100)



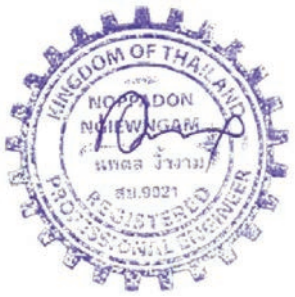
13 DB28

3 DB12 @ 0.250

2 DB28

Negative Section

3B15 (100 X 100)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. B0	Date	07-ธ.ค.-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B0

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{+pos.}$	Mu$^+$	=	78390	[kg-m]
$M_{-neg.}$	Mu$^-$	=	93957	[kg-m]
$V_{max.}$	Vu	=	90000	[kg]
	DL	=	5600	[kg/m]
	LL	=	3800	[kg/m]
Length	Ln	=	900	[cm]
width	b	=	100	[cm]
depth	t	=	100	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	95	[cm]
a try	=	3.1335	[cm]
As	=	23.305	[cm^2]
a	=	**3.1335**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	256.324	[cm^2]
a	=	34.464	[cm]
Mu	=	717617	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	23.305	[cm^2]
No. of steel	=	**5**	
As	=	24.54	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	95	[cm]
a try	=	8.7110	[cm]
As	=	27.933	[cm^2]
a	=	**3.1335**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	336.425	[cm^2]
a	=	45.234	[cm]
Mu	=	876654	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	27.933	[cm^2]
No. of steel	=	**6**	
As	=	29.45	[cm^2]

Section OK

Stirrup Design

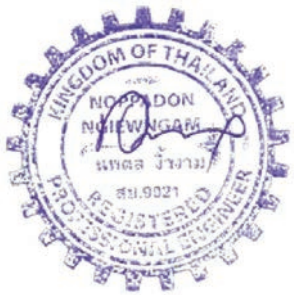
0.53 ϕ (fc')$^{0.5}$ bd	=	84777	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	89.7	[cm]	
S	=	47.500	[cm]	
Used Stirrup	RB 12	@	0.400	OK

Check Deflection

Δ all	=	2.50 [cm]			
ρ fy	=	10	<	35	[kg/cm^2]
Y'	=	16.4			
I	=	7144792	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.40 [cm]	<	Δ all	OK
Δ Time	=	0.6	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.		Job No.	193018
Design Reinforced Concrete Beam		Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)		Initial	Noppadon N.
Beam No.	B0A	Date	07-ธ.ค.-20
Project :	สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam B0A

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	140241	[kg-m]
$M_{-neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	80000	[kg]
	DL	=	5600	[kg/m]
	LL	=	3800	[kg/m]
Length	Ln	=	900	[cm]
width	b	=	80	[cm]
depth	t	=	100	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	95	[cm]
a try	=	7.1617	[cm]
As	=	42.612	[cm^2]
a	=	**7.1617**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	205.059	[cm^2]
a	=	34.464	[cm]
Mu	=	574094	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	42.612	[cm^2]
No. of steel	=	**9**	
As	=	44.18	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	95	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**7.1617**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	269.140	[cm^2]
a	=	45.234	[cm]
Mu	=	701323	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]

Section OK

Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	67821		[kg]
L=Ln/2(Vu - ϕ Vc)/Vu	=	125.7		[cm]
S	=	35.994		[cm]
Used Stirrup		**RB 12**	@	**0.350**

OK

Check Deflection

Δ all	=	2.50	[cm]		
ρ fy	=	23	<	35	[kg/cm^2]
Y'	=	23.5			
I	=	5715833	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.49	[cm] <	Δ all	OK
Δ Time	=	0.75	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam BG-0		

Beam No. BG-0

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	247420	[kg-m]
$M_{-neg.}$	Mu$^-$	=	401388	[kg-m]
$V_{max.}$	Vu	=	270000	[kg]
	DL	=	5800	[kg/m]
	LL	=	4000	[kg/m]
Length	Ln	=	900	[cm]
width	b	=	100	[cm]
depth	t	=	300	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel: DB25
Lower Steel: DB25
Stirrup Steel: DB12

d	=	295	[cm]
a try	=	3.1493	[cm]
As	=	23.423	[cm^2]
a	=	**3.1493**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
$ρ_{max}$	=	0.0270	
As	=	795.952	[cm^2]
a	=	107.019	[cm]
Mu	=	6919739	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	23.423	[cm^2]
No. of steel	=	**5**	
As	=	24.54	[cm^2]

Section OK

For Negative Moment :

Upper Steel: DB25
Lower Steel: DB25
Stirrup Steel: DB12

d	=	295	[cm]
a try	=	8.7110	[cm]
As	=	38.362	[cm^2]
a	=	**5.1579**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
$ρ_{max}$	=	0.0354	
As	=	1044.687	[cm^2]
a	=	140.462	[cm]
Mu	=	8453276	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	38.362	[cm^2]
No. of steel	=	**8**	
As	=	39.27	[cm^2]

Section OK

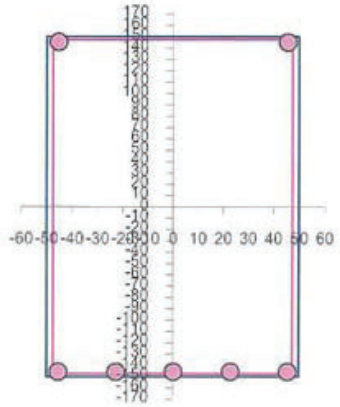
Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	263254	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	77.1	[cm]	
S	=	147.500	[cm]	
Used Stirrup	1	DB12 @ 0.400		OK

Check Deflection

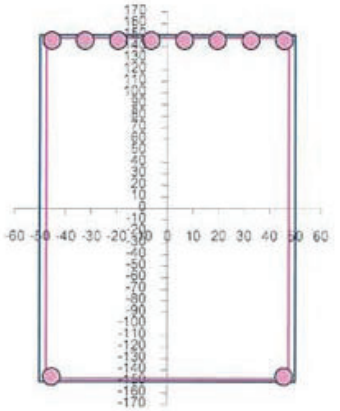
Δ all	=	2.50 [cm]			
ρ fy	=	3	<	35	[kg/cm^2]
Y'	=	30.2			
I	=	213936458 [cm^4]			
Ec	=	284116 [kg/cm^2]			
Δ	=	0.01 [cm]	<	Δ all	OK
Δ Time	=	0.02	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. BG-0	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam BG-0



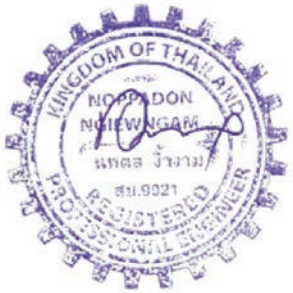
Positive Section

BG-0 (100 X 300)



Negative Section

BG-0 (100 X 300)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Date	7-Dec-20		
File	Beam BG-1		

Beam No. BG-1

Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	55688	[kg-m]
$M_{neg.}$	Mu$^-$	=	99000	[kg-m]
$V_{max.}$	Vu	=	85000	[kg]
	DL	=	5800	[kg/m]
	LL	=	4000	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	100	[cm]
depth	t	=	100	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

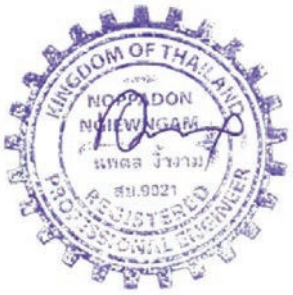
Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

d	=	95	[cm]
a try	=	2.2147	[cm]
As	=	16.475	[cm^2]
a	=	**2.2151**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	256.324	[cm^2]
a	=	34.464	[cm]
Mu	=	717617	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	16.475	[cm^2]
No. of steel	=	**4**	
As	=	19.63	[cm^2]

Section OK

Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	84777	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	60.9	[cm]	
S	=	47.500	[cm]	
Used Stirrup	1	DB12 @ 0.400		OK



For Negative Moment :

Upper Steel	DB25
Lower Steel	DB25
Stirrup Steel	DB12

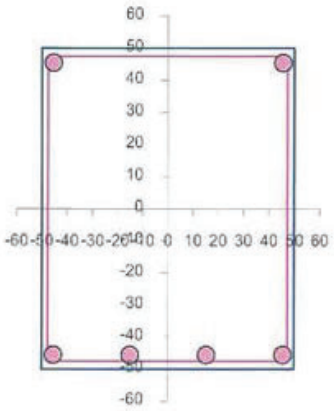
d	=	95	[cm]
a try	=	8.7110	[cm]
As	=	30.338	[cm^2]
a	=	**4.0791**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	336.425	[cm^2]
a	=	45.234	[cm]
Mu	=	876654	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	**2**	
As	=	9.82	[cm^2]
Σ As	=	30.338	[cm^2]
No. of steel	=	**7**	
As	=	34.36	[cm^2]

Section OK

Check Deflection

Δ all	=	2.22 [cm]			
ρ fy	=	8	<	35	[kg/cm^2]
Y'	=	14.8			
I	=	7144792	[cm^4]		
Ec	=	284116	[kg/cm^2]		
Δ	=	0.26 [cm]	<	Δ all	OK
Δ Time	=	0.16	<	Δ all	OK

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. BG-1	Date	7-Dec-20
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam BG-1



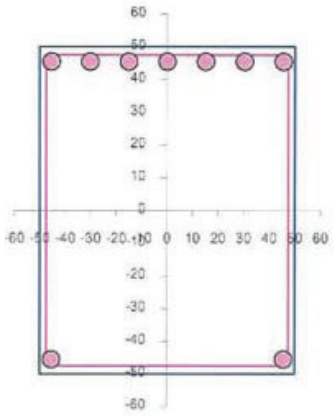
2 DB25

1 DB12 @ 0.400

4 DB25

Positive Section

BG-1 (100 X 100)



7 DB25

1 DB12 @ 0.400

2 DB25

Negative Section

BG-1 (100 X 100)

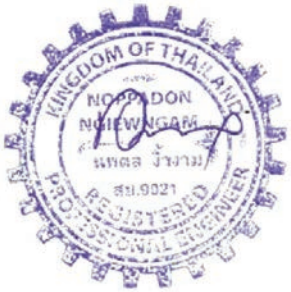
DYNAMIC ENGINEERING CONSULTANT CO., LTD.

Design Reinforced Concrete Beam

(Ultimate Stress Design method in accordance with ACI 318-89)

Job No.	193018		
Rev.	1		
Initial	Noppadon N.		
Beam No.	6B1	Date	17-มี.ค.-21
Project :	สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 6B1

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu$^+$	=	67854	[kg-m]
$M_{-neg.}$	Mu$^-$	=	51387	[kg-m]
$V_{max.}$	Vu	=	35560	[kg]
	DL	=	5600	[kg/m]
	LL	=	3800	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	40	[cm]
depth	t	=	70	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]



For Positive Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	65	[cm]
a try	=	3.8693	[cm]
As	=	29.887	[cm^2]
a	=	**10.0460**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	70.152	[cm^2]
a	=	23.580	[cm]
Mu	=	134379	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	29.887	[cm^2]
No. of steel	=	7	
As	=	34.36	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	65	[cm]
a try	=	8.7110	[cm]
As	=	22.634	[cm^2]
a	=	**10.0460**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	92.074	[cm^2]
a	=	30.949	[cm]
Mu	=	164160	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	22.634	[cm^2]
No. of steel	=	5	
As	=	24.54	[cm^2]

Section OK

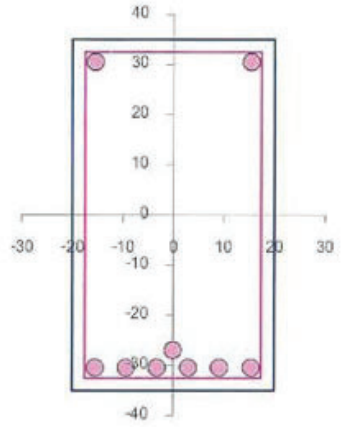
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	23202	[kg]
L=Ln/2(Vu - ϕ Vc)/Vu	=	178.2	[cm]
S	=	24.270	[cm]
Used Stirrup		**RB 12 @ 0.225**	OK

Check Deflection

Δ all	=	2.22 [cm]		
ρ fy	=	53	> 35	[kg/cm^2]
Y"	=	17.8		
I	=	610682	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	2.89 [cm]	> Δ all	NO
Δ Time	=	4.38	> Δ all	NO

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 6B1	Date	17-มี.ค.-21
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 6B1



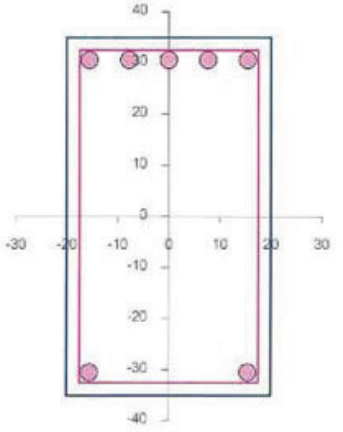
2 DB 25

Stirrup RB 12 @ 0.225

7 DB 25

Positive Section

6B1 (40 X 70)



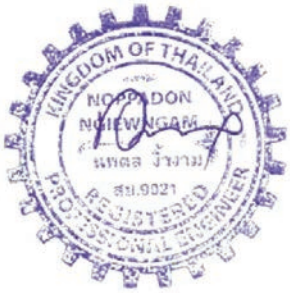
5 DB 25

Stirrup RB 12 @ 0.225

2 DB 25

Negative Section

6B1 (40 X 70)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 6B2	Date	17-มี.ค.-21
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 6B2

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{pos.}$	Mu$^+$	=	63008	[kg-m]
$M_{neg.}$	Mu$^-$	=	0	[kg-m]
$V_{max.}$	Vu	=	34504	[kg]
	DL	=	5600	[kg/m]
	LL	=	3800	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	40	[cm]
depth	t	=	70	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	65	[cm]
a try	=	9.7874	[cm]
As	=	29.119	[cm^2]
a	=	**9.7878**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
ρ_{max}	=	0.0270	
As	=	70.152	[cm^2]
a	=	23.580	[cm]
Mu	=	134379	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	29.119	[cm^2]
No. of steel	=	6	
As	=	29.45	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 25
Lower Steel	DB 25
Stirrup Steel	RB 12

d	=	65	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**9.7878**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
ρ_{max}	=	0.0354	
As	=	92.074	[cm^2]
a	=	30.949	[cm]
Mu	=	164160	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	2	
As	=	9.82	[cm^2]

Section OK

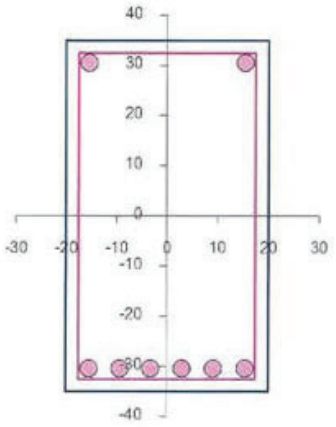
Stirrup Design

0.53 ϕ (fc')$^{0.5}$ bd	=	23202	[kg]
L=Ln/2(Vu - ϕ Vc)/Vu	=	171.4	[cm]
S	=	26.538	[cm]
Used Stirrup		RB 12 @ 0.250	OK

Check Deflection

Δ all	=	2.22 [cm]		
ρ fy	=	45	> 35	[kg/cm^2]
Y'	=	21.2		
I	=	522559	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	3.38 [cm]	> Δ all	NO
Δ Time	=	5.12	> Δ all	NO

DYNAMIC ENGINEERING CONSULTANT CO., LTD.	Job No.	193018
Design Reinforced Concrete Beam	Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)	Initial	Noppadon N.
Beam No. 6B2	Date	17-มี.ค.-21
Project : สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 6B2



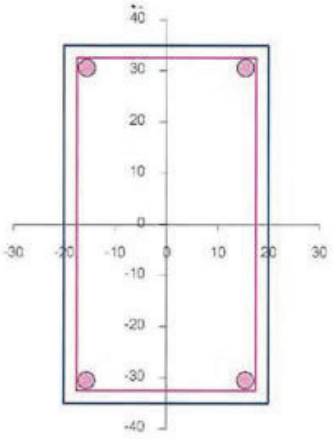
2 DB 25

Stirrup RB 12 @ 0.250

6 DB 25

Positive Section

6B2 (40 X 70)



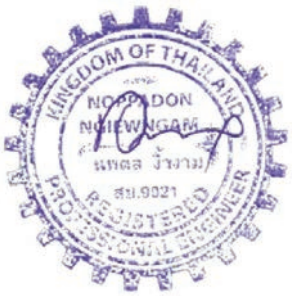
2 DB 25

Stirrup RB 12 @ 0.250

2 DB 25

Negative Section

6B2 (40 X 70)



DYNAMIC ENGINEERING CONSULTANT CO., LTD.		Job No.	193018
Design Reinforced Concrete Beam		Rev.	1
(Ultimate Stress Design method in accordance with ACI 318-89)		Initial	Noppadon N.
Beam No.	6B3	Date	17-มี.ค.-21
Project :	สัญญาก่อสร้างอาคารสนับสนุน (ส่วนศูนย์ประชุม)	File	Beam 6B3

	fc'	=	350	[kg/cm^2]
	fy SD	=	4000	[kg/cm^2]
	fy SR	=	2400	[kg/cm^2]
$M_{-pos.}$	Mu^+	=	5406	[kg-m]
$M_{-neg.}$	Mu^-	=	0	[kg-m]
$V_{max.}$	Vu	=	11000	[kg]
	DL	=	5600	[kg/m]
	LL	=	3800	[kg/m]
Length	Ln	=	800	[cm]
width	b	=	25	[cm]
depth	t	=	50	[cm]
	d'	=	5	[cm]
siding	d"	=	5	[cm]

For Positive Moment :

Upper Steel	DB 20
Lower Steel	DB 20
Stirrup Steel	RB 9

d	=	45	[cm]
a try	=	1.8320	[cm]
As	=	3.406	[cm^2]
a	=	**1.8320**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	0.80	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0360	
$ρ_{max}$	=	0.0270	
As	=	30.354	[cm^2]
a	=	16.325	[cm]
Mu	=	40254	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]
Σ As	=	3.406	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]

Section OK

For Negative Moment :

Upper Steel	DB 20
Lower Steel	DB 20
Stirrup Steel	RB 9

d	=	45	[cm]
a try	=	8.7110	[cm]
As	=	0.000	[cm^2]
a	=	**1.8320**	[cm]
ϕ Bend	=	0.90	
ϕ Shea	=	0.85	
k1	=	1.05	
Es	=	2040000	[kg/cm^2]
Ec	=	284553	[kg/cm^2]
n	=	7	
ρ	=	0.0472	
$ρ_{max}$	=	0.0354	
As	=	39.840	[cm^2]
a	=	21.426	[cm]
Mu	=	49175	[kg-m]
M1	=	0	[kg-m]
As'	=	0.0000	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]
Σ As	=	0.000	[cm^2]
No. of steel	=	2	
As	=	6.28	[cm^2]

Section OK

Stirrup Design

0.53 ϕ $(fc')^{0.5}$ bd	=	10039	[kg]	
L=Ln/2(Vu - ϕ Vc)/Vu	=	89.7	[cm]	
S	=	22.500	[cm]	
Used Stirrup		RB 9 @ 0.225		OK

Check Deflection

Δ all	=	2.22 [cm]		
ρ fy	=	22	< 35	[kg/cm^2]
Y'	=	10.9		
I	=	189844	[cm^4]	
Ec	=	284116	[kg/cm^2]	
Δ	=	9.29 [cm]	>	Δ all NO
Δ Time	=	5.63	>	Δ all NO