

請問1F柱之剪力計算中J端彎矩(Mcpr,j)如何計算得，J端彎矩與I端近似

搜尋文件  

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MIDAS Information Technology Co., Ltd.  Gen 2024 (v1.2) / Design 

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\* cLCBZ = 1.2D + 1.0( 1.0 )( RYD( RS ) + RYD( ES ) ) + 1.0L 

1) Calculate design shear force according to special provisions for seismic design 

$V_{u.org} = 23.196 \text{tonf} \cdot \text{m}^{-2}$  

$\alpha_1 = 1.00$  

$M_{pr,I} = 1,246.870 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,J} = 1,247.254 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,CW,I} = 0.000 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,CCW,I} = 0.000 \text{tonf} \cdot \text{m}^{-2}$  

$k_{1,I} = 1.00$  

Ratio = 0.62 

$M_{pr,CW,J} = 0.000 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,CCW,J} = 0.000 \text{tonf} \cdot \text{m}^{-2}$  

$k_{1,J} = 1.00$  

Ratio = 0.00 

$M_{pr,CW,I} = M_{pr,I} = 1,246.870 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,CCW,I} = M_{pr,I} = 1,246.870 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,CW,I} = M_{pr,I} = 1,247.254 \text{tonf} \cdot \text{m}^{-2}$  

$M_{pr,CCW,I} = M_{pr,I} = 1,247.254 \text{tonf} \cdot \text{m}^{-2}$  

$L_{net} = 3.000 \text{m}$  

$V_{e,CW} = \alpha_1 * \frac{M_{pr,CW,I} + M_{pr,CW,J}}{L_{net}} = 831.375 \text{tonf} \cdot \text{m}^{-2}$  

$V_{e,CCW} = \alpha_1 * \frac{M_{pr,CCW,I} + M_{pr,CCW,J}}{L_{net}} = 831.375 \text{tonf} \cdot \text{m}^{-2}$  

$V_{e1} = \max ( V_{e,CW} , V_{e,CCW} ) = 831.375 \text{tonf} \cdot \text{m}^{-2}$  

$\alpha_2 = 1.000$  

$V_{EQ} = 6.878 \text{tonf} \cdot \text{m}^{-2}$  

$V_{G2} = V_{org} - V_{EQ} = 16.319 \text{tonf} \cdot \text{m}^{-2}$  ( Exclude Seismic Load ) 

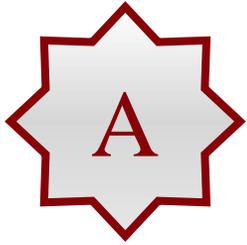
$V_{e2} = V_{G2} + ( \alpha_2 * V_{EQ} ) = 23.196 \text{tonf} \cdot \text{m}^{-2}$  

$V_e = \max ( | V_{e1} | , V_{e2} ) = 831.375 \text{tonf} \cdot \text{m}^{-2}$  ( Special moment frame ) 

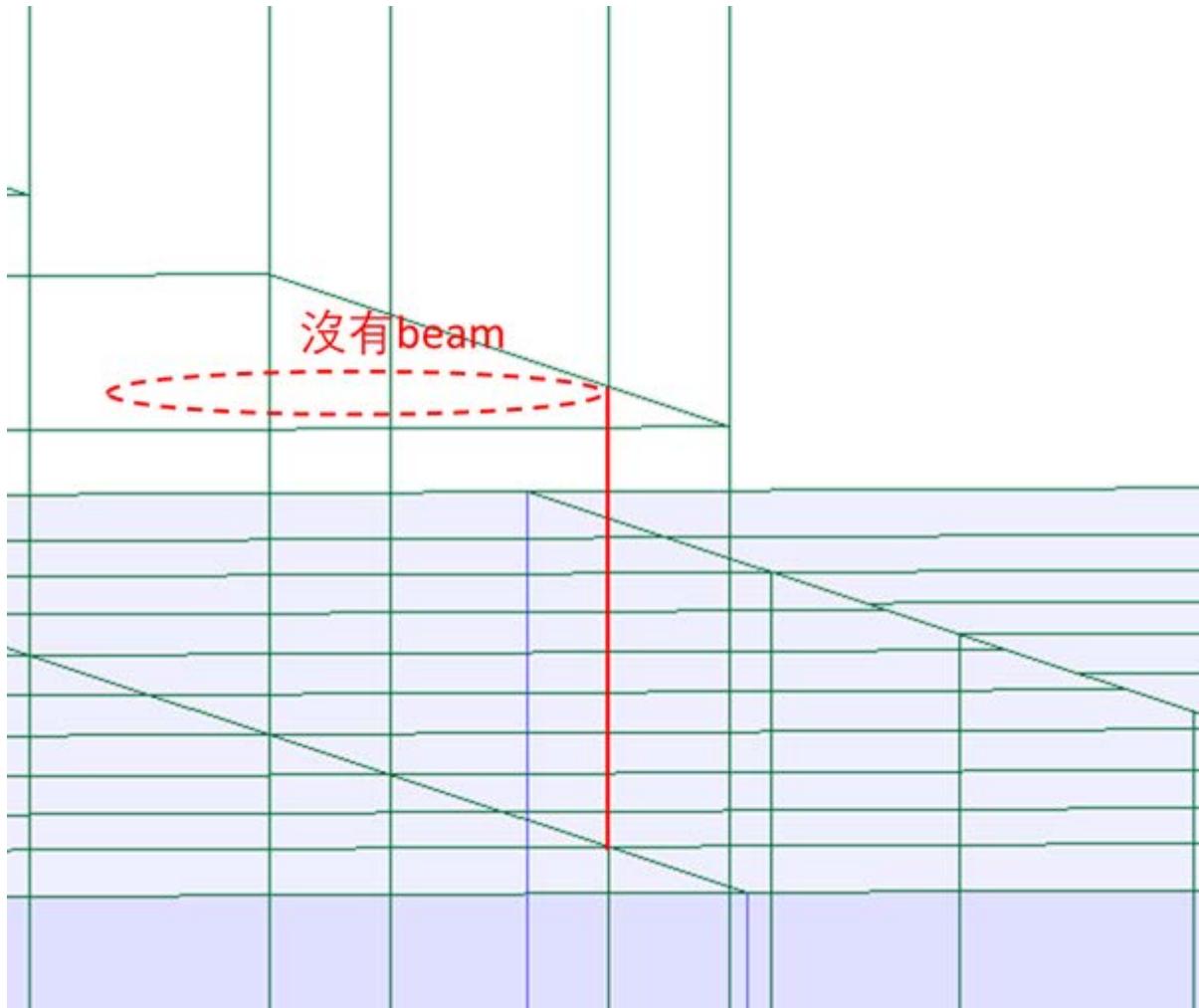
$V_{u,org} = \max ( | V_{u,org} | , | V_e | ) = 831.375 \text{tonf} \cdot \text{m}^{-2}$  

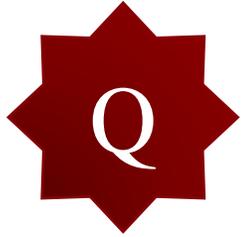
$V_u = V_{u,org} = 831.375 \text{tonf} \cdot \text{m}^{-2}$  ( by  $V_{u,org}$  Sign ) 





因為LCB控制是Rx (x 方向)，您的柱子(Element 8) local axis-z沒有連接Beam，所以Mcp會使用Mcpr。





再請問為何另外模型，卻又相反。  
 Y向有樑(Lnet=2.3m)無樑彎矩  
 Z向無樑(Lnet=3.0m)有樑彎矩

7. Shear Capacity ( y-Dir. ) ( End, 0.00R )

Shear	Memb No.	12354	
	LCB	cLCB15	
	$V_u / \phi V_{n,max}$	1,311.357tonf / 834.300tonf = 1.572	Failure
	$V_u / \phi V_R$	1,311.357tonf / 834.300tonf = 1.572	NG

\* cLCB15 :  $1.2D - 1.0(1.0)(RYD(RS)+RYD(ES)) + 1.0L$

1) Calculate design shear force according to special provisions for seismic design

$V_{org} = 101.089\text{tonf}$

$M_{prj.Colm} = 1,507.467\text{tonf}\cdot\text{m}$

$M_{prj.Colm} = 1,508.653\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CW} = 0.000\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CW} = 0.000\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CCW} = 0.000\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CCW} = 0.000\text{tonf}\cdot\text{m}$

$L_{net} = 2.300\text{m}$

$M_{prj.Calc} = M_{prj.Colm} = 1,507.467\text{tonf}\cdot\text{m}$

$M_{prj.Calc} = M_{prj.Colm} = 1,508.653\text{tonf}\cdot\text{m}$

$\alpha = 1.000$

8. Shear Capacity ( z-Dir. ) ( End, 1.00L )

Shear	Memb No.	12354	
	LCB	cLCB31	
	$V_u / \phi V_{n,max}$	638.774tonf / 755.076tonf = 0.846	OK
	$V_u / \phi V_R$	638.774tonf / 755.076tonf = 0.846	OK

\* cLCB31 :  $0.9D - 1.0(1.0)(RXD(RS)+RXD(ES))$

1) Calculate design shear force according to special provisions for seismic design

$V_{org} = -36.602\text{tonf}$

$M_{prj.Colm} = 1,689.074\text{tonf}\cdot\text{m}$

$M_{prj.Colm} = 1,685.442\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CW} = 0.000\text{tonf}\cdot\text{m}$

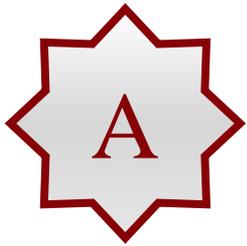
$M_{prj.Beam.CW} = 227.247\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CCW} = 0.000\text{tonf}\cdot\text{m}$

$M_{prj.Beam.CCW} = 178.925\text{tonf}\cdot\text{m}$

$L_{net} = 3.000\text{m}$

$M_{prj.Calc} = M_{prj.Colm} = 1,689.074\text{tonf}\cdot\text{m}$



請看一下情況，我看您的模型Column 的i node 連接Wall Element (圖1) 所以會影響 Beam 跟Column 強柱弱梁ductile design有問題。  
Wall Element 是Shear Wall，如果不是Shear Wall 所以需要改成Plate Element。

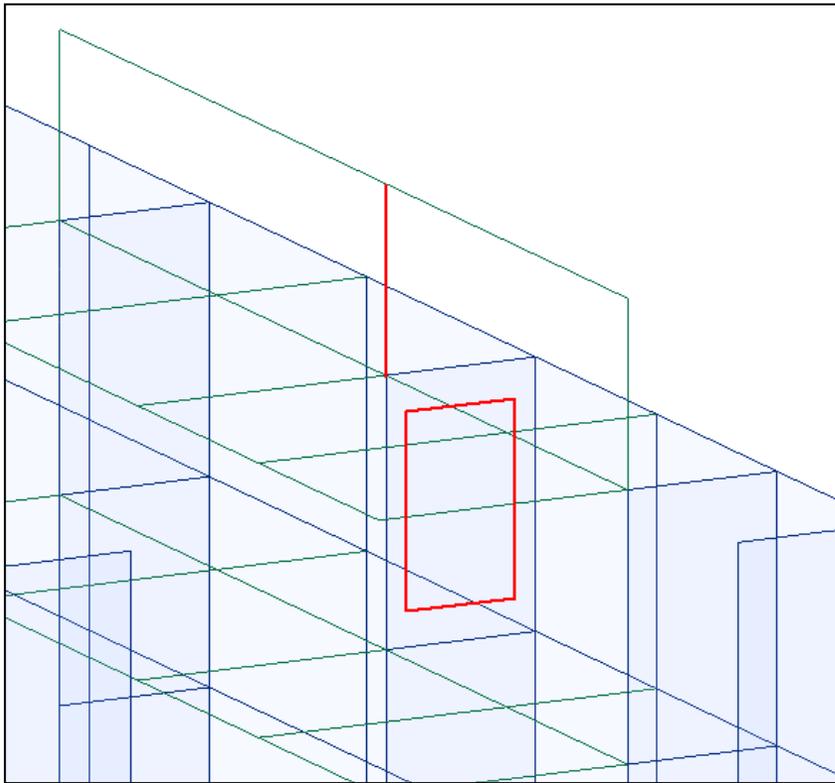


圖 1

改成Plate Element就沒問題了。

**7. Shear Capacity ( y-Dir. ) ( End, 1.00L )**

Shear	LCB	cLCB31
	$V_u / \phi V_{n,max}$	533.647tonf / 733.024tonf = 0.728
	$V_u / \phi V_n$	533.647tonf / 733.024tonf = 0.728

\* cLCB31 :  $0.9D - 1.0(1.0)(RXD(RS)+RXD(ES))$

1) Calculate design shear force according to special provisions for seismic design

Factored shear forces caused by load case type

$V_{u,org} = 7.247\text{tonf}$

$V_{EQ} = -0.600\text{tonf}$

$V_{G2} = V_{u,org} - V_{EQ} = 7.846\text{tonf}$  ( Exclude Seismic Load )

Design Parameter

$\alpha_1 = 1.000$

$\alpha_2 = 1.000$

$L_{net} = 2.300\text{m}$

Probable Moment of column

$M_{cpr,j} = 1,114.147\text{tonf-m}$

$M_{cpr,j} = 1,111.084\text{tonf-m}$

Summation of probable moments of connected girders

$M_{bpr,cw,j} = 0.000\text{tonf-m}$

$M_{bpr,ccw,j} = 0.000\text{tonf-m}$

$M_{bpr,cw,j} = 227.247\text{tonf-m}$

$M_{bpr,ccw,j} = 178.925\text{tonf-m}$

**8. Shear Capacity ( z-Dir. ) ( End, 0.00R )**

Shear	LCB	cLCB15
	$V_u / \phi V_{n,max}$	1,105.858tonf / 842.381tonf = 1.313
	$V_u / \phi V_n$	1,105.858tonf / 842.381tonf = 1.313

\* cLCB15 :  $1.2D - 1.0(1.0)(RYD(RS)+RYD(ES)) + 1.0L$

1) Calculate design shear force according to special provisions for seismic design

Factored shear forces caused by load case type

$V_{u,org} = 44.371\text{tonf}$

$V_{EQ} = 10.242\text{tonf}$

$V_{G2} = V_{u,org} - V_{EQ} = 34.128\text{tonf}$  ( Exclude Seismic Load )

Design Parameter

$\alpha_1 = 1.000$

$\alpha_2 = 1.000$

$L_{net} = 3.000\text{m}$

Probable Moment of column

$M_{cpr,j} = 1,658.170\text{tonf-m}$

$M_{cpr,j} = 1,659.405\text{tonf-m}$

Summation of probable moments of connected girders

$M_{bpr,cw,j} = 0.000\text{tonf-m}$

$M_{bpr,ccw,j} = 0.000\text{tonf-m}$

$M_{bpr,cw,j} = 0.000\text{tonf-m}$

$M_{bpr,ccw,j} = 0.000\text{tonf-m}$