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带多块混凝土板的防屈曲钢板剪力墙 边缘柱内力需求计算分析

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摘要:过小的边缘构件截面易出现钢板剪力墙的内拉现象,难以充分发挥防屈曲钢板剪力墙的抗震性能,边缘构件截面设计与其内力需求相关,分析边缘柱的内力需求显得尤为必要。基于提出的带多块混凝土板的防屈曲钢板剪力墙,本文理论推导了带多块混凝土板的防屈曲钢板剪力墙边缘柱内力的解析计算表达式,结合已开展的带多块水平拼装的混凝土板的防屈曲钢板剪力墙试验研究,建立并验证了其数值模型,进一步对比了数值分析和解析计算得到的边缘柱的内力分布情况。研究表明,带多块混凝土板的防屈曲钢板剪力墙的内藏钢板划分为约束区域和非约束区域。边缘柱的轴力、剪力及弯矩分布的解析计算结果与数值分析结果较为接近,边缘柱的内力计算表达式正确。研究成果可为该类钢板剪力墙的设计提供参考。

关键词:防屈曲钢板剪力墙;多块混凝土板;边缘柱;内力需求;解析计算

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1 引言

在水平荷载作用下,钢板剪力墙易发生屈曲而以拉力带形式提供承载能力及抗侧刚度,其滞回曲线存在捏缩特性^[1]。为此,学者提出在钢板两侧拼装约束盖板,以防止内藏钢板发生面外屈曲,即形成了防屈曲钢板剪力墙。

针对防屈曲钢板剪力墙已开展大量研究。范重等^[2]研究了螺栓与混凝土盖板的简化计算方法,建立了钢板与盖板相互作用计算模型。马张永等^[3]针对半刚性框架-密肋防屈曲钢板剪力墙开展了振动台试验研究及数值分析。田炜烽等^[4]推导了考虑墙板抗压能力的钢板剪力墙的刚度、承载力和屈服位移的计算式,并提出了一种通用拉杆模型。谭继可等^[5]针对方钢管混凝土框架-冷弯薄壁型钢屈曲约束钢板剪力墙开展了抗震性能试验研究,分析了破坏形态、荷载-位移滞回曲线、耗能能力等抗震性能。周绪红等^[6]研究了不同冷弯薄壁型钢屈曲约束形式的钢板剪力墙的往复剪切性能。

侯健等^[7,8]针对防屈曲钢板剪力墙开展了平面外工作机理与设计方法研究,给出了面外相互作用荷载的理论计算公式。综上所述可以发现,传统的防屈曲钢板剪力墙主要在内藏钢板两侧外挂单块约束盖板,其尺寸较大,制作运输易产生裂缝,对运输及吊装设备要求很高。为此,文献^[9,10]提出了多块混凝土板拼装的防屈曲钢板剪力墙,对其开展试验研究及有限元分析。随后,钟恒等^[11]亦针对设置分块盖板的防屈曲钢板剪力墙开展了抗震性能试验研究及数值分析,结果表明,设置分块盖板的防屈曲钢板剪力墙具有良好的滞回性能,相比盖板不分块的防屈曲钢板剪力墙,设置分块盖板的剪力墙承载力和耗能能力略有降低,但墙体的延性得到提高。范重等^[12]采用 ABAQUS 软件对分块盖板屈曲约束钢板剪力墙的受力机理进行了研究,并建立了简化计算模型。

过去的研究发现,过小的边缘构件截面不足以抵抗内藏钢板形成的屈服荷载,试件出现了内拉变形现象^[13]。钢板剪力墙规范^[14]给出了非加劲钢板

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剪力墙边缘框架柱的最小截面惯性矩,以指导确定边缘柱的截面尺寸。范重等^[15]研究了边框刚度对屈曲约束钢板剪力墙性能的影响,但未给出量化的边框内力的计算公式。可见,现有研究鲜有针对带分块约束盖板的防屈曲钢板剪力墙的边缘框架构件内力需求的分析。因此,针对带多块混凝土板的防屈曲钢板剪力墙,研究内藏钢板对边缘框架构件附加的内力显得尤为必要。

为此,本文采用理论分析与数值模拟方法,推导了带多块混凝土板的防屈曲钢板剪力墙边缘柱内力的解析计算表达式,结合有限元软件开展数值分析,将解析计算结果与数值分析结果进行对比,有效验证了给出的解析计算表达式的正确性。研究成果可为该类钢板剪力墙的设计提供参考。

2 带多块混凝土板的防屈曲钢板剪力墙(MBRSPSW)介绍

图1(a)为带多块混凝土板拼装的防屈曲钢板剪力墙系统(MBRSPSW),主要由水平边缘构件(边缘框架梁)、竖向边缘构件(边缘框架柱)、内藏钢板、混凝土板和螺栓组成。内藏钢板通过鱼尾板采用焊接或栓接方式与边缘构件连接,内藏钢板通过螺栓在其两侧外挂多块混凝土板。混凝土板之间以及混凝土板与边缘构件预留缝隙,以防止其相互挤压造成混凝土压碎。外挂混凝土板不承担主框架的水平和竖向荷载,仅用于约束内藏钢板面外变形,防止内藏钢板屈曲失稳。根据混凝土板的拼装方式,可以分成带多块水平拼装的混凝土板的防屈曲钢板剪力墙(H-MBRSPSW)和带多块竖向拼装的混凝土板的防屈曲钢板剪力墙(V-MBRSPSW)。韩启浩等^[9]针对带多块混凝土板的防屈曲钢板剪力墙开展了拟静力加载试验研究,分析了其抗震性能,如图1(b)所示。

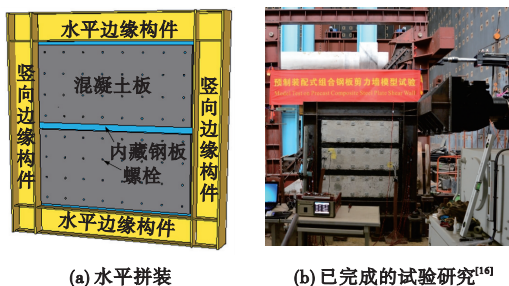


图1 带多块混凝土板的防屈曲钢板剪力墙(MBRSPSW)
Fig.1 Proposed buckling-restrained steel plate shear wall with multi-concrete panels(MBRSPSW)

3 MBRSPSW 边缘框架柱内力的解析计算表达式

内藏钢板的受力变形行为会对边缘框架柱产

生附加内力,为定量分析这种不利影响,采用理论方法推导内藏钢板附加给边缘框架柱内力的解析计算表达式。以下分析假定边缘框架梁柱为铰接连接,忽略边缘构件对抗侧力的贡献。

3.1 内藏钢板的应力状态分析

图2给出了带两块水平拼装的混凝土板的防屈曲钢板剪力墙尺寸构造,其中,混凝土板与边缘构件的缝隙为 Δ ,混凝土板之间的缝隙为 δ 。

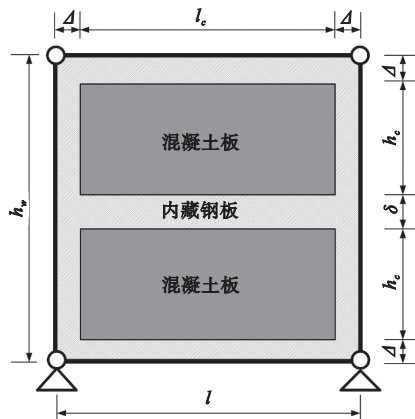


图2 带多块混凝土板的防屈曲钢板剪力墙尺寸构造
Fig.2 Dimensions and details of MBRSPSW

在侧向荷载作用下,根据其约束状态,内藏钢板可以划分为两类区域,即带混凝土板(约束盖板)的约束区域和不带混凝土板的非约束区域,其中,约束区域为纯剪切应力状态,非约束区域为斜拉应力状态。内藏钢板的应力状态如图3所示。根据薄板弹性理论^[16],屈曲剪切应力(极限剪切应力) τ_{cr} 可由式(1)确定, k 为弹性屈曲系数。针对非约束区域的内藏钢板,周围具有混凝土盖板,可以近似视为四边简支的钢板,其屈曲剪切应力 τ_{cr} 可能低于屈服剪切应力 τ_s ,从而发生面外屈曲行为。

$$\tau_{cr} = \frac{k\pi^2 E_s}{12(1-\mu^2)} \left(\frac{t}{b}\right)^2 \quad (1)$$

式中 E_s 为钢板弹性模量, μ 为泊松比, t 为钢板厚度, b 为钢板的宽度。

3.1.1 约束区域 I

混凝土板约束区域 I 的内藏钢板处于纯剪切应力状态,由平面应力状态的莫尔圆理论,沿着斜柱方向的剪应力为 τ_x ,与之正交方向的正应力为 σ_x ,分别由式(2,3)确定。

$$\tau_x = \tau_s \cos(2\theta_r), \quad \sigma_x = \tau_s \sin(2\theta_r) \quad (2,3)$$

式中 θ_r 为主应力夹角。

3.1.2 非约束区域 II&III

根据 von Mises 屈服准则^[17],引起钢板屈服的对角拉伸应力为 σ_{r3} (实际为第一主应力),可根据

式(4)求得。其中,对混凝土板之间的缝隙,即区域II,表示为 ${}_g\sigma_{ty}$;对 δ 缝隙,表示为 ${}_b\sigma_{ty}$ 。

$$\sigma_{ty} = -1.5\tau_{cr}\sin 2\alpha + \sqrt{\sigma_y^2 + \tau_{cr}^2[(1.5\sin 2\alpha)^2 - 3]} \quad (4)$$

式中 σ_y 为单轴屈服应力, α 为拉力场倾角,可根据规范^[14]公式求得,对 Δ 缝隙, $\alpha_b = \tan^{-1}(\Delta/l)$;对 δ 缝隙, $\alpha_g = \tan^{-1}(\delta/l)$ 。

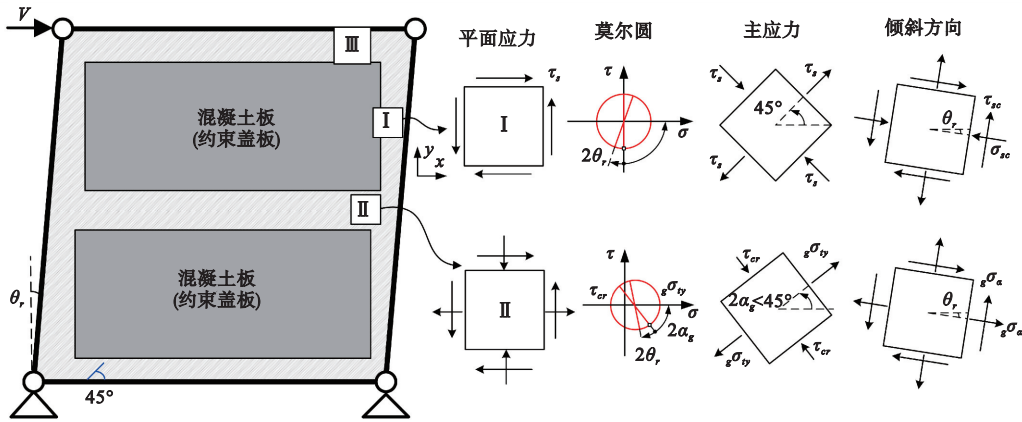


图3 侧向荷载作用下内藏钢板的应力状态
Fig. 3 Stress states of inner steel plate under lateral loading

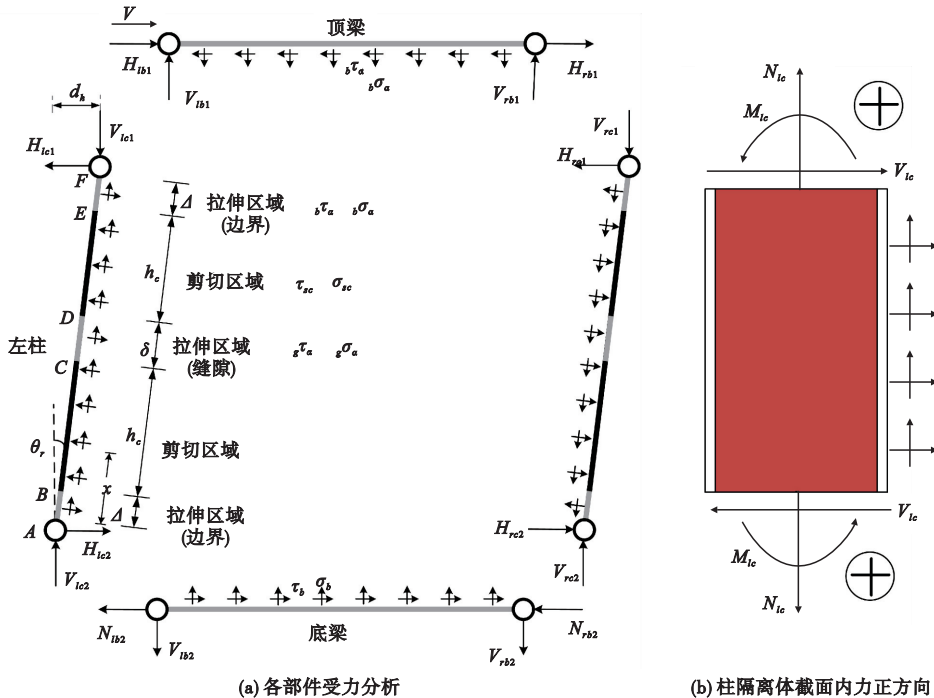
非约束区域II的内藏钢板在加载后期会形成拉力场,在混凝土板之间的缝隙区域II,假设沿着斜柱方向的剪应力为 ${}_g\tau_\alpha$,与之正交方向的正应力为 ${}_g\sigma_\alpha$,可根据斜截面应力计算公式确定。类似地,在混凝土板与周边构件缝隙区域,即图3中的区域III,可得到相应的斜截面剪应力 ${}_b\tau_\alpha$ 和正应力 ${}_b\sigma_\alpha$ 的计算表达式。

3.2 边缘框架柱内力计算表达式

边缘框架柱内力与内藏钢板相关,根据已有试

验研究及数值分析成果,并结合内藏钢板的应力分布状态及构造,柱内力可以分成5区段进行计算,3个与钢板拉力带对应的区段(拉伸区域)和2个钢板纯剪切应力状态对应的区段,如图4(a)所示,截面内力的正方向如图4(b)所示。

对左柱顶部和顶梁左端交点进行受力分析可知,根据力矩平衡原理,对顶梁右端求力矩可得顶梁左端竖向力 V_{lb1} 的计算表达式,具体见式(5)。左柱顶水平剪力 H_{lc1} 可以通过力矩平衡求



(a) 各部件受力分析
(b) 柱隔离体截面内力正方向
图4 带多块混凝土板的防屈曲钢板剪力墙各部件受力分析
Fig. 4 Mechanical analysis of members of MBRSPSW

得,即对图中左柱底部点 A 求矩,即可确定左柱顶水平力 H_{lc1} 的计算表达式,具体见式(6)

$$V_{lc1} = V_{lb1} = {}_b\sigma_a t l / 2 \quad (5)$$

$$\left\{ \begin{aligned} H_{lc1} &= V_{lc1} \tan\theta_r + \frac{{}_b\sigma_a t \Delta \left(h_w - \frac{\Delta}{2} \right)}{h_w \cos\theta_r} + \\ &\frac{{}_b\sigma_a t \Delta^2}{2h_w \cos\theta_r} + \frac{{}_g\sigma_a t \delta \left(\Delta + h_c + \frac{\delta}{2} \right)}{h_w \cos\theta_r} - \\ &\frac{\sigma_x t h_c \left(h_w - \Delta - \frac{h_c}{2} \right)}{h_w \cos\theta_r} - \frac{\sigma_x t h_c \left(\Delta + \frac{h_c}{2} \right)}{h_w \cos\theta_r} \end{aligned} \right. \quad (6)$$

3.2.1 轴力计算表达式

取左柱计算截面与柱顶端为隔离体,通过静力平衡,采用分段函数表示方法,左柱轴力 N_{lc} 计算见式(7),框架柱倾角 $\theta_a = \sin^{-1}(d_h/h_w)$ 。为便于基于柱高度绘制内力图形,假定 x 为计算截面至柱底点 A 的距离(柱轴方向)。

当 $0 < x \leq \Delta$ (AB段)时,

$$N_{lc}(x) = {}_b\tau_a \Delta t + 2\tau_x h_c t + {}_g\tau_a \delta t + {}_b\tau_a (\Delta - x)t - V_{lc1} \cos\theta_r - H_{lc1} \sin\theta_r \quad (7a)$$

当 $\Delta \leq x < \Delta + h_c$ (BC段)时,

$$N_{lc}(x) = {}_b\tau_a \Delta t + \tau_x h_c t + {}_g\tau_a \delta t + \tau_x (\Delta + h_c - x)t - V_{lc1} \cos\theta_r - H_{lc1} \sin\theta_r \quad (7b)$$

当 $\Delta + h_c \leq x < \Delta + h_c + \delta$ (CD段)时,

$$N_{lc}(x) = {}_b\tau_a \Delta t + \tau_x h_c t + {}_g\tau_a (\Delta + h_c + \delta - x)t - V_{lc1} \cos\theta_r - H_{lc1} \sin\theta_r \quad (7c)$$

当 $\Delta + h_c + \delta \leq x < \Delta + 2h_c + \delta$ (DE段)时,

$$N_{lc}(x) = {}_b\tau_a \Delta t + \tau_x (\Delta + 2h_c + \delta - x)t - V_{lc1} \cos\theta_r - H_{lc1} \sin\theta_r \quad (7d)$$

当 $\Delta + 2h_c + \delta \leq x < h_w$ (EF段)时,

$$N_{lc}(x) = {}_b\tau_a (h_w - x)t - V_{lc1} \cos\theta_r - H_{lc1} \sin\theta_r \quad (7e)$$

3.2.2 剪力计算表达式

通过静力平衡,采用分段函数表示方法,可得左柱剪力 V_{lc} 计算表达式,具体见式(8)。

当 $0 < x \leq \Delta$ (AB段)时,

$$V_{lc}(x) = V_{lc1} \cos\theta_r - H_{lc1} \sin\theta_r + {}_b\sigma_a \Delta t - 2\sigma_x h_c t + {}_g\sigma_a \delta t + {}_b\sigma_a (\Delta - x)t \quad (8a)$$

当 $\Delta \leq x < \Delta + h_c$ (BC段)时,

$$V_{lc}(x) = V_{lc1} \sin\theta_r - H_{lc1} \cos\theta_r + {}_b\sigma_a \Delta t - \sigma_x h_c t + {}_g\sigma_a \delta t - \sigma_x (\Delta + h_c - x)t \quad (8b)$$

当 $\Delta + h_c \leq x < \Delta + h_c + \delta$ (CD段)时,

$$V_{lc}(x) = V_{lc1} \sin\theta_r - H_{lc1} \cos\theta_r + {}_b\sigma_a \Delta t - \sigma_x h_c t + {}_g\sigma_a (\Delta + h_c + \delta - x)t \quad (8c)$$

当 $\Delta + h_c + \delta \leq x < \Delta + 2h_c + \delta$ (DE段)时,

$$V_{lc}(x) = V_{lc1} \sin\theta_r - H_{lc1} \cos\theta_r + {}_b\sigma_a \Delta t - \sigma_x (\Delta + 2h_c + \delta - x)t \quad (8d)$$

当 $\Delta + 2h_c + \delta \leq x < h_w$ (EF段)时,

$$V_{lc}(x) = V_{lc1} \sin\theta_r - H_{lc1} \cos\theta_r + {}_b\sigma_a (h_w - x)t \quad (8e)$$

3.2.3 弯矩计算表达式

通过力矩平衡,同样采用分段函数表示方法,可得左柱弯矩 M_{lc} 计算表达式,具体见式(9)。

当 $0 < x \leq \Delta$ (AB段)时,

$$\left\{ \begin{aligned} M_{lc}(x) &= V_{lc1} (h_w - x) \sin\theta_r - H_{lc1} (h_w - x) \cos\theta_r + \\ &{}_b\sigma_a t \Delta \left(h_w - x - \frac{\Delta}{2} \right) - \sigma_x t h_c \left(h_w - x - \Delta - \frac{h_c}{2} \right) + \\ &{}_g\sigma_a t \delta \left(h_w - x - \Delta - h_c - \frac{\delta}{2} \right) - \\ &\sigma_x t h_c \left(h_w - x - \Delta - \delta - \frac{3h_c}{2} \right) + \frac{{}_b\sigma_a t (\Delta - x)^2}{2} \end{aligned} \right. \quad (9a)$$

当 $\Delta \leq x < \Delta + h_c$ (BC段)时,

$$\left\{ \begin{aligned} M_{lc}(x) &= V_{lc1} (h_w - x) \sin\theta_r - H_{lc1} (h_w - x) \cos\theta_r + \\ &{}_b\sigma_a t \Delta \left(h_w - x - \frac{\Delta}{2} \right) - \sigma_x t h_c \left(h_w - x - \Delta - \frac{h_c}{2} \right) + \\ &{}_g\sigma_a t \delta \left(h_w - x - \Delta - h_c - \frac{\delta}{2} \right) - \sigma_x t \frac{(\Delta + h_c - x)^2}{2} \end{aligned} \right. \quad (9b)$$

当 $\Delta + h_c \leq x < \Delta + h_c + \delta$ (CD段)时,

$$\left\{ \begin{aligned} M_{lc}(x) &= V_{lc1} (h_w - x) \sin\theta_r - H_{lc1} (h_w - x) \cos\theta_r + \\ &{}_b\sigma_a t \Delta \left(h_w - x - \frac{\Delta}{2} \right) - \sigma_x t h_c \left(h_w - x - \Delta - \frac{h_c}{2} \right) + \\ &{}_g\sigma_a t \frac{(\Delta + h_c + \delta - x)^2}{2} \end{aligned} \right. \quad (9c)$$

当 $\Delta + h_c + \delta \leq x < \Delta + 2h_c + \delta$ (DE段)时,

$$\left\{ \begin{aligned} M_{lc}(x) &= V_{lc1} (h_w - x) \sin\theta_r - H_{lc1} (h_w - x) \cos\theta_r + \\ &{}_b\sigma_a t \Delta \left(h_w - x - \frac{\Delta}{2} \right) - \sigma_x t \frac{(h_w - x - \Delta)^2}{2} \end{aligned} \right. \quad (9d)$$

当 $\Delta + 2h_c + \delta \leq x < h_w$ (EF段)时,

$$\left\{ \begin{aligned} M_{lc}(x) &= V_{lc1} (h_w - x) \sin\theta_r - \\ &H_{lc1} (h_w - x) \cos\theta_r + {}_b\sigma_a t \frac{(h_w - x)^2}{2} \end{aligned} \right. \quad (9e)$$

4 MBRSPSW 数值分析验证

选取已经完成的试验研究,采用 ABAQUS 有限元软件建立相应的数值模型,通过将数值分析结果与试验结果进行对比,验证数值模型的正确性。进一步,提取边缘柱数值分析结果,并与解析计算结果进行对比,以验证解析计算表达式的正确性。

4.1 已有文献试验介绍

为验证本文数值建模方法的正确性,选取钟恒等^[11]完成的分块盖板屈曲约束钢板剪力墙进行对比,试件编号为 BRSPSW-H。如图 5 所示,边缘框架梁柱构件为铰接连接,内藏钢板为试件的唯一抗侧力构件,厚度为 1.8 mm,内藏钢板两侧均外挂两块水平拼装的约束钢盖板,盖板厚度为 30 mm,其余设计参数参见文献^[11]。

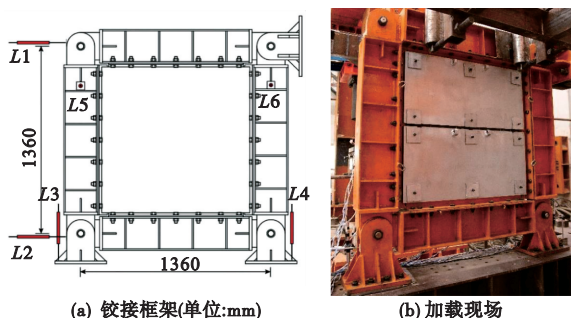


图 5 分块盖板屈曲约束钢板剪力墙试验研究^[11]

Fig. 5 Experimental research of buckling-restrained steel plate shear walls with partitioned restraining panels

4.2 数值模型的建立

4.2.1 材料本构模型

试验模型中,边缘钢框架采用铰接连接,边缘框架和约束盖板在加载过程中受力较小,其本构关系对数值分析结果影响较小,属于次要构件,采用双线性随动硬化模型来模拟钢材的力学行为,如图 6(a)所示,其中屈服强度设置为 345 MPa,弹性模量为 2.06×10^5 N/mm²,硬化模量为弹性模量的 1%。内藏钢板为模型的主要受力部件,其本构参数对结果影响较大,内藏钢板材料本构参数可根

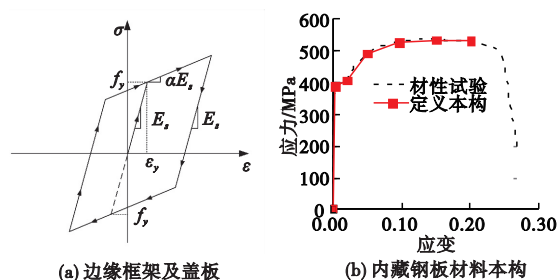


图 6 钢材材料本构模型

Fig. 6 Material constitutive model of steel

据试验研究中的材料试验结果确定,采用多折线等向强化本构模型进行模拟,如图 6(b)所示。

4.2.2 单元类型和边界条件

如图 7 所示,内藏钢板采用四节点减缩积分单元(S4R)进行模拟,边缘框架构件采用三维梁单元(B31)进行模拟,约束钢盖板采用八节点减缩积分实体单元(C3D8R)进行模拟。试件中,约束钢盖板与内藏钢板通过螺栓进行连接,由于螺栓的应力与变形行为不是本文的研究重点,采用带连接(LINK)行为的连接器单元(CONN3D2)模拟螺栓连接,可以反映两节点铰接连接及间距不变的特性。内藏钢板与边缘构件通过绑定(tie)连接实现两者的焊接效果。钢盖板与内藏钢板接触面设置接触行为,剪切方向为无摩擦接触,法方向为硬接触(hard contact)。

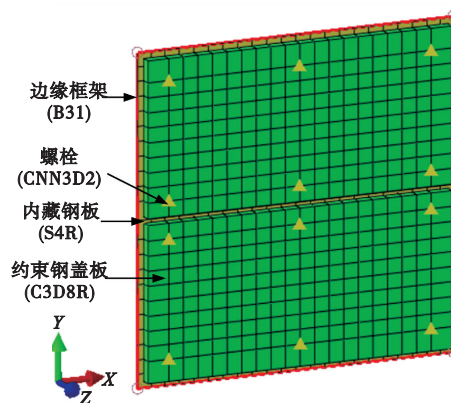


图 7 模型单元类型

Fig. 7 Element types of models

根据试验条件设定模型边界条件,边缘梁与柱为铰接连接,约束顶梁的面外自由度,以模拟构件的平面内加载行为。底部梁柱铰接点为固定铰支座,放松绕垂直于纸面轴的转动自由度(RZ),其余自由度受约束。通过屈曲分析来考虑钢板的初始缺陷,根据文献可知,试验中钢板的面外变形为 2 mm,即取缺陷幅值为 2 mm。

4.3 数值模型验证

图 8 给出了试验和数值分析得到的内藏钢板平面外变形图,可以看出,拉力带分布均匀、细密,数值分析得到的内藏钢板的面外屈曲变形模式与试验结果吻合良好。数值分析得到的侧向荷载-位移角关系与试验结果的对比如图 9 所示,可以发现,数值分析得到的滞回曲线和骨架曲线与试验结果吻合较好,数值分析和试验得到的承载能力和加载/卸载刚度较为接近,说明建立的数值模型可以较好地模拟带多块约束盖板的防屈曲钢板剪力墙

的力学行为。其中,数值分析和试验得到的最大侧向荷载分别为 610.0 kN(-623.7 kN,负向加载)和 608.3 kN(-601.5 kN),正负向最大误差为 3.69%;正向加载下,数值分析和试验得到的弹性抗侧刚度(即第一级加载时的侧向荷载与侧向位移之比)分别为 112.6 kN/mm 和 112.7 kN/mm,误差很小。以上分析有效验证了数值模型的合理性和正确性,可用于进一步分析各部件的内力分布情况。

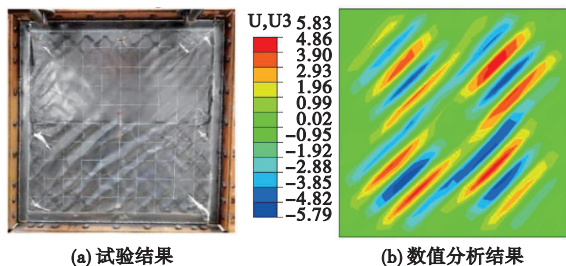


图 8 内藏钢板平面外变形对比
Fig. 8 Comparisons of out-of-plane deformations of infill steel plate

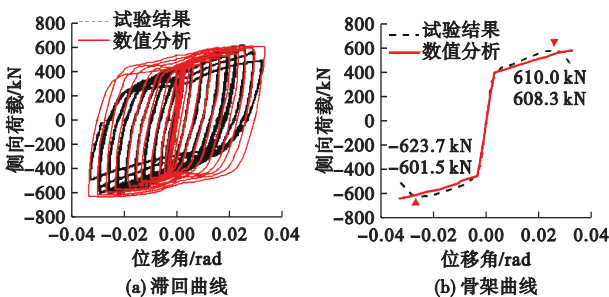


图 9 侧向荷载-位移角关系对比
Fig. 9 Comparisons of lateral force versus drift ratio relationships

4.4 边缘框架柱内力数值分析与解析计算结果对比

根据试验模型的尺寸参数及材料试验结果,将相关数据代入第 3.2 节,可得边缘框架柱的内力具体计算表达式。当模型加载至位移角为 1/250 rad、1/100 rad 和 1/50 rad 时,左边缘框架柱的内力数值分析结果和解析计算结果如图 10 所示。

由图 10(a)可见,当模型加载至位移角为 1/250 rad 和 1/50 rad 时,边缘框架柱轴力的解析计算和数值分析结果吻合较好,说明给出的轴力计算表达式可以反映框架柱的轴力分布情况。由图 10(b)可以发现,边缘框架柱剪力分布的解析计算和数值分析总体吻合较好,但在较小位移角下,如图中位移角为 1/250 rad 时,两者存在较大差别,主要原因是在较小加载位移角下,内藏钢板的应力状态与假定不完全相符,如加载前期,非约束区域

II&III 对应的钢板并不一定会发生面外屈曲,内藏钢板形成的正应力影响框架柱剪力分布,最终造成剪力计算有差别。但在较大位移角下,框架柱剪力的解析计算和数值分析结果较为接近,说明给出的剪力计算表达式可以反映框架柱的剪力分布情况。由图 10(c)可知,边缘框架柱弯矩分布的解析计算和数值分析吻合较好。如当加载至 1/250 rad、1/100 rad 和 1/50 rad 时,由解析计算得到的柱中部弯矩值分别为 11490 kN·mm, 22435 kN·mm, 24977 kN·mm, 数值分析得到的柱中部弯矩值分别为 10617 kN·mm, 21718 kN·mm, 24505 kN·mm, 两者最大误差为 8.2%。综上,边缘框架柱的轴力、剪力及弯矩的解析计算结果与数值分析结果较为接近,证明给出的内力计算表达式的正确性。

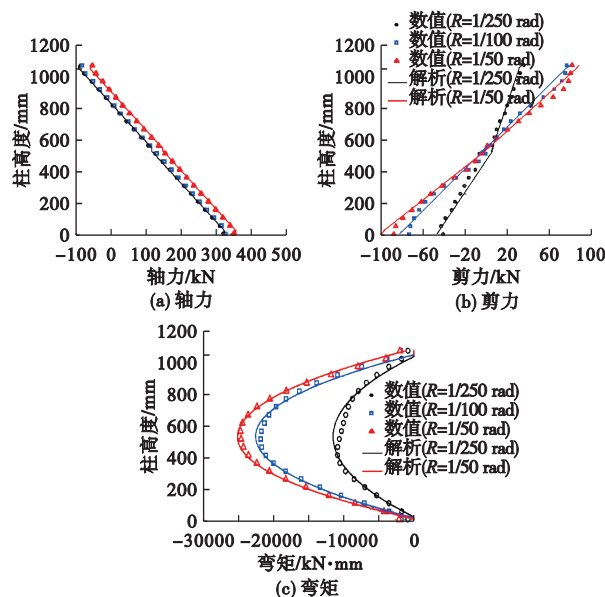


图 10 边缘框架柱内力计算结果对比
Fig. 10 Comparisons of internal forces of boundary frame columns

5 结 论

(1)带多块混凝土板的防屈曲钢板剪力墙的内藏钢板的应力状态可划分为带混凝土板的约束区域和不带混凝土板的非约束区域。约束区域为纯剪切应力状态,非约束区域为斜拉应力状态,分别给出了对应区域剪应力和正应力的计算表达式。

(2)柱内力划分为多区段进行计算,采用理论分析方法,确定了边缘框架柱的轴力、剪力及弯矩的解析计算表达式。

(3)与试验研究结果进行对比,验证了带多块混凝土板的防屈曲钢板剪力墙的数值模型的合理性和正确性。边缘框架柱的轴力、剪力及弯矩的解

析计算结果与数值分析结果较为接近,说明确立的边缘柱内力的解析计算表达式正确。研究成果可为该类钢板剪力墙的设计提供理论参考。

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In ternal forces demand calculation of boundary columns of buckling-restrained steel plate shear wall with multi-concrete panels

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Abstract: Small cross-sections of boundary elements easily induce the “internal tension” phenomenon of steel plate shear walls, making it difficult to fully utilize the seismic performance of buckling-restrained steel plate shear walls. The design of cross-sections of boundary elements is related to their internal force requirements, and analyzing the internal force requirements of boundary columns is meaningful. Based on the proposed buckling-restrained steel plate shear wall with multi-concrete panels (MBRSPSW), the analytical expressions for the internal force of the boundary column of the MBRSPSW were theoretically derived in this paper. Combined with the experimental research on the buckling-restrained steel plate shear wall horizontally assembled multi-concrete panels (H-MBRSPSW), its numerical model was established and verified. The internal force distributions of the boundary column obtained from numerical analysis and analytical calculation were further compared. The research results indicate that the inner steel plates in the MBRSPSW are divided into constrained regions and unconstrained regions. The axial force, shear force, and bending moment distributions of the boundary column from the analytical calculation results agree with those from the numerical analysis results, and the internal force calculation expressions of the boundary column are correct. The research results can provide a reference for the design of this category of steel plate shear walls.

Key words: buckling-restrained steel plate shear wall; multi-concrete panels; boundary columns; internal forces demand; analytical calculation

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Bézier triangular surface mesh generation and isogeometric plate and shell analysis

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Abstract: This paper presents a Bézier triangle meshing method that considers both clipped and non-clipped forms for a single NURBS surface. The proposed method is applied to analyze isogeometric Kirchhoff-Love shell structures. The process begins by interpolating NURBS surfaces into Bézier surfaces. Subsequently, the topological relationship between the clipping curve and each parameter node is calculated within the parameter domain. A Bézier contour curve set is then generated in the parameter domain by selecting points along the clipping curve. Utilizing this contour curve set, a triangular mesh is generated in the parameter domain. Finally, the Bézier triangle mesh in the physical domain is created through a mapping method. The adaptability and robustness of the algorithm are verified through three models, and the mesh quality is assessed. The results demonstrate favorable overall mesh quality. Building upon this foundation, the paper illustrates the application of a rotation constraint between Kirchhoff-Love shell elements, using the penalty function method with Scordelis-Lo's Roof shell model as an example. The accuracy of Kirchhoff-Love shell elements based on Bézier triangles is subsequently validated.

Key words: isogeometric analysis; Bézier triangular mesh; Bézier surface mapping; Curves trim; Kirchhoff-love shell