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自复位混凝土剪力墙抗震性能研究进展与展望

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摘 要: 强震后工程结构功能快速恢复, 是近年来可持续发展工程抗震的重要研究领域。由于通过释放墙片节点约束并沿墙高通长布置并后张无黏结预应力筋实现摇摆回复及倾覆控制, 地震动下摇摆和界面张合效应可使自复位混凝土剪力墙表现为无震损或低震损及小残余变形, 甚至无残余变形。作为可恢复功能结构体系重要组成部分的自复位混凝土剪力墙结构、技术、理论与设计方法正持续发展。在结构体系上, 包括受控摇摆单肢墙、混合摇摆单肢墙、竖向连接耗能联肢墙以及连梁耗能联肢墙结构4种自复位剪力墙, 均具备明确的自复位能力, 但耗能钢筋/部件组装程度导致了不同结构体系的耗能能力差异明显; 在设计方法上, 传统的基于力的抗震设计方法难以直接应用, 应确定相关性态极限状态和控制点并完善残余变形验算, 提出基于位移性态的自复位剪力墙抗震设计方法; 非线性分析方法上, 纤维模型、集中塑性模型、多弹簧模型以及实体有限元模型等均可较精确地进行自复位剪力墙结构的模拟分析, 但仍需有针对性考虑相关参数敏感程度。进而, 提出开展预应力筋弹性和非弹性受力行为及其摇摆转动中的变形能力与损失, 楼屋盖等水平连接对自复位能力和震损影响, 强震下预应力筋和耗能元件失效下自复位剪力墙倾覆, 多墙片拼装竖向多节点摇摆的自复位剪力墙高阶振型效应与残余变形预测等四方面研究, 为自复位混凝土剪力墙结构体系研发与应用提供理论和技术支撑。

关键词: 后张无黏结预应力; 可恢复功能; 自复位剪力墙; 抗震性能; 残余变形

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Seismic performance of self-centering concrete shear wall: State-of-the-art review and prospects

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Abstract: Post-earthquake resilience of engineering structures is an important research area of sustainable earthquake engineering in recent years. Restoration and collapse control are realized with the constraint at the base released and the wall panels post-tensioned by unbonded prestressing tendons, which can reduce damage level and even eliminate residual drift owing to the gap-opening mechanism. As an important component of earthquake resilient structural system, self-centering concrete shear wall system is developing continuously. Self-centering shear wall such as controlled rocking wall, hybrid rocking wall, coupled wall with vertical joint, and coupled wall with coupling beam can accomplish restoration, but various dissipating devices result in significantly different levels of energy dissipating. Traditional force-based design is not applicable, so displacement-based performance design should be presented with the improvement of residual drift control, definition of performance level, and section limit state. Fiber model, lumped plasticity model, multi-spring model, and finite element model can accurately simulate the behavior of self-centering shear wall, but there are some differences in parameter sensitivity. Thus, some current research challenges and prospects were presented to provide theoretical and technical support for the development and application of self-centering shear wall structure system; the elastic and inelastic behavior of prestressing tendon which can cause prestress loss and insufficient deformation capacity during rocking, the connection performance between self-centering shear wall and adjacent structural components such as the floor system and its influence on self-centering ability, collapse control with the failure of post-tensioning tendon and dissipating device, and utilization of multiple dissipating rocking interfaces between wall panels to mitigate higher mode effects and estimation of residual drift.

Keywords: unbonded post-tensioning; earthquake resilience; self-centering shear wall; seismic performance; residual drift

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由于地震作用的随机性和人类资源的有限性, 不可能无限制地使用资源去实现在强震下结构不破坏, 设防目标也不可能寻求绝对的安全^[1]。现有各国抗震设计规范尽管设计参数取值和方法存在

差异,但在罕遇地震下,基本都以控制倒塌,保障生命安全为设防目标.在这样的抗震设计理念的指导下,一般通过能力设计方法(capacity design)^[2]刻意弱化结构的某些部位,使其在地震作用下屈服形成塑性铰耗散能量,并从构造上保证塑性铰的变形能力和延性,使结构达到期望的破坏模式.虽然可以控制结构倒塌,但截面塑性的过度发展常常会导致结构在震后产生损伤和大的残余变形.

从国外近年来的几次大的破坏性地震中可以发现,结构的倒塌和人员伤亡都得到了一定的控制,但经济损失却异常巨大.除了非结构构件和设备的损坏之外,大部分的损失来源于震后结构的修复重建费用,以及漫长的修复和重建期导致建筑使用功能中断带来的经济损失.典型的如 2011 年新西兰基督城 Mw 6.3 级地震,死亡 185 人,但重建费用高达 40 亿新西兰元,约占新西兰 GDP 的 20%,某街区内 1 000 余栋建筑就由于残余变形过大(并未倒塌)而不得不拆除.

因此,如何降低震后修复难度,提高结构抗震韧性(resilience),近年来得到广泛关注.而从 20 世纪 90 年代开始出现并持续发展的自复位结构体系(self-centering structural system),由于独特的界面摇摆机制和后张预应力技术的应用,可有效控制结构震损和残余变形,降低震后修复费用和修复时间,提高城市的抗震防灾韧性.目前新西兰、美国、日本和中国均在着力开展自复位结构体系的相关研究,提出了摇摆桥墩、自复位框架、自复位剪力墙和自复位耗能支撑等多种自复位结构形式,相关性能分析、设计方法与技术也逐渐发展.

本文聚焦自复位混凝土剪力墙结构(以下简称自复位剪力墙或自复位剪力墙结构),介绍自复位混凝土剪力墙结构的自复位实现与工作原理,总结和分析近来自复位剪力墙的结构组成与结构体系、设计方法以及非线性模型与模拟方法等,进而,基于自复位剪力墙结构体系、技术与理论发展现状,提出在工作机理、分析模型和设计方法方面需解决的关键科学问题与展望.

1 自复位原理

在 1960 年的智利大地震中,一些较高的水塔结构由于发生了基础摇摆,结果在地震中免遭破坏. Housner^[3]针对这类结构提出了简化的刚性摇摆质量块模型,见图 1,分析了不同工况下摇摆质量块的响应.随后许多学者又针对底部释放约束的摇摆结构展开了研究^[4-12],包括摇摆柱、摇摆框架和摇摆核心筒等,发现了约束释放后产生的结构摇摆对结

构的保护作用.此类结构的思路为释放约束后结构发生摇摆,刚度降低,结构振动周期延长,减小了地震作用,在自重作用下震后结构将回复原位.但结构的摇摆变形缺乏控制,极有可能在强震下发生倾覆.

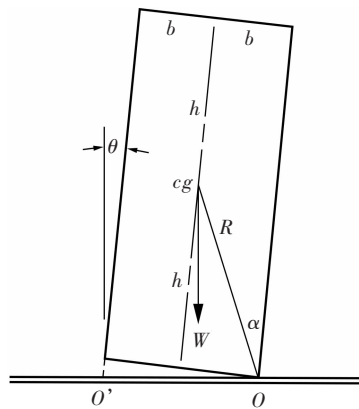


图 1 刚体摇摆质量块^[3]

Fig. 1 Rigid rocking block^[3]

在 20 世纪 90 年代美日联合进行的预制装配式混凝土结构抗震性能研究的 PRESSS 项目(precast seismic structural systems)^[13]中, Priestley 等^[14]首次提出将预制装配式结构与后张预应力筋结合,通过施加预应力控制结构的自由摇摆,提出了具备自复位能力的装配式节点连接,见图 2.

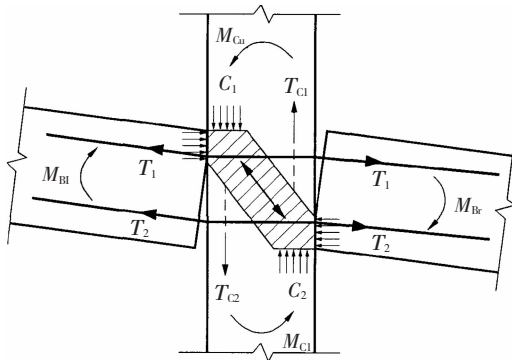


图 2 自复位混凝土框架节点^[14]

Fig. 2 Self-centering concrete frame joint^[14]

自复位涉及结构约束释放后地震输入下的摇摆回复及其倾覆控制,其基本工作原理为节点约束释放后形成的独特的界面摇摆机制.自复位常与装配式混凝土结构相结合,将预制构件通过后张预应力筋预压装配连接,而节点处钢筋和混凝土断开连接,即可形成自复位节点.在变形初始阶段节点近似固结;随变形增大,节点张开,界面发生摇摆,抗侧刚度主要由预应力筋提供,刚度会有大幅下降,变形行为可简化为双线性,卸载后预应力筋提供的回复力将使结构恢复到初始位置,见图 3(a).节点约束释放后结构周期延长,地震作用降低,但承载力和刚度不足,耗能能力偏低^[15],可在节点摇摆界面位置附加耗能元件,借助耗能元件的耗能能力,结构的变形将

变为旗帜型,见图 3(b).

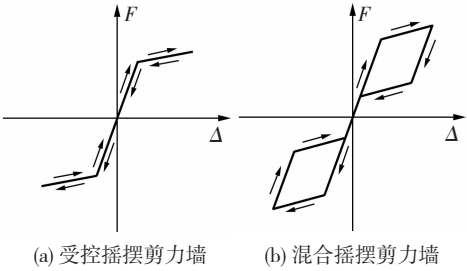


图 3 自复位节点的变形特征

Fig. 3 Hysteretic behavior of self-centering connection

若结构的摇摆不受控制,回复力的来源仅为结构自重,此类摇摆一般称为自由摇摆 (free rocking); 若在自由摇摆的基础上后张预应力筋对摇摆加以控制,则称为受控摇摆 (controlled rocking); 若在摇摆界面再安装耗能元件,结构还将具备一定的耗能能力,称为混合摇摆 (hybrid rocking). 考虑到自由摇摆缺乏控制,容易失去稳定发生倾覆,因此,着重分析具有后两类摇摆机制的自复位剪力墙.

相比于传统节点固结的结构,自复位结构的优势主要体现在:

- 1) 震损小. 节点约束释放,混凝土基本没有拉应力,通过在混凝土局部受压区配置间接钢筋约束,结构的损伤可以得到有效控制. 自复位节点类似于塑性铰,但非线性范围更加集中,除节点外结构的其余部分将基本保持弹性;
- 2) 残余变形小. 依靠高强预应力筋较强的弹性变形能力,结构变形能力大幅提高,在地震结束后,回复力 (包括结构自重以及后张预应力等) 将使结构恢复到初始位置,减小或消除残余变形,促使震后建筑快速恢复使用功能,减少因修复重建和建筑功能中断带来的经济损失和社会影响.

2 自复位剪力墙的分类与试验研究

按结构形式,自复位剪力墙可分为单肢墙和联肢墙两类. 单肢墙可分为受控摇摆墙和混合摇摆墙,联肢墙可分为通过竖向耗能元件连接的联肢墙和通过自复位连梁连接的联肢墙. 自复位剪力墙可由多片预制墙片竖向叠合,再沿墙高布置并后张无黏结预应力筋拼装而成;墙片端部的墙趾受压区域布置间接钢筋形成约束混凝土,以提高墙趾的碰撞能力和变形能力;需要保证拼装节点不发生滑移和剪切破坏,使其仅发生墙底摇摆界面张合.

2.1 单肢墙

2.1.1 受控摇摆墙

Kurama 等^[15]最早提出了受控摇摆墙模型,见图 4. Kurama 采用 DRAIN-2DX 软件进行了大量的

拟静力分析和动力响应分析,较为系统地研究了此类结构的抗震性能,发现该类剪力墙可以在低损伤情况下实现较大侧移,而且基本没有残余变形,但耗能能力差,变形偏大,见图 5.

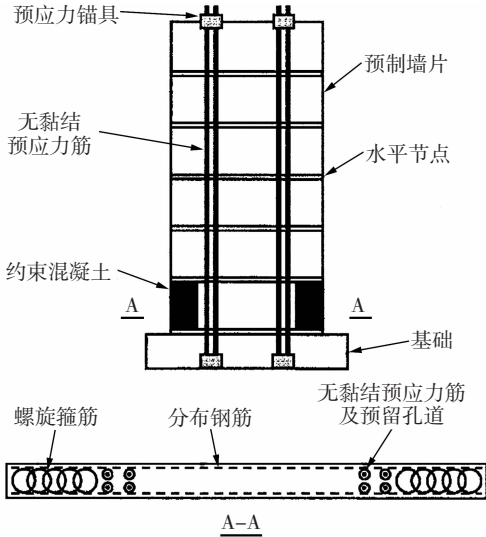


图 4 受控摇摆墙^[16]

Fig. 4 Controlled rocking wall^[16]

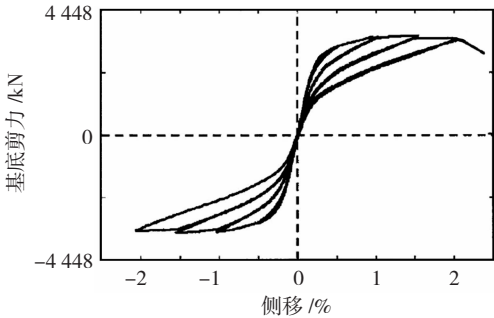


图 5 受控摇摆墙的滞回曲线^[16]

Fig. 5 Hysteretic curves of controlled rocking wall^[16]

Nazari 等^[17]对 4 片 5/18 受控摇摆墙进行了振动台试验 (图 6),研究了摇摆墙在不同地震水平下的动态响应,分析了高速加载条件下墙体与基础间碰撞的辐射阻尼作用. 试验发现碰撞过程中的弹性辐射耗能阻尼约为 1.5%,总耗能阻尼约为 4.2%,辐射阻尼占比可以达到总阻尼的 1/3 左右.

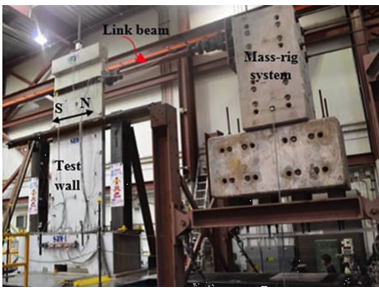


图 6 受控摇摆墙的振动台试验^[17]

Fig. 6 Shake-table test of controlled rocking wall^[17]

2.1.2 混合摇摆墙

为改善受控摇摆墙的耗能能力,Kurama^[18]提出

在剪力墙两侧安装黏滞阻尼器,见图 7,分析发现安装黏滞阻尼器后结构的最大层间侧移和楼面加速度大幅降低. Restrepo 等^[19]在基础摇摆界面设置了软钢滞回耗能元件来补充耗能. 采用的耗能元件为在摇摆界面附近一定长度范围内变截面的热轧钢筋,并在此范围内保证无黏结,以防止应变过度集中而导致钢筋过早断裂. 配置了耗能钢筋的剪力墙在 3% 侧移时等效黏滞阻尼系数达到了 14%, 试验结束后仅墙趾混凝土发生了轻微剥落,剪力墙基本没有残余变形,表现出了非常好的自复位能力,见图 8.

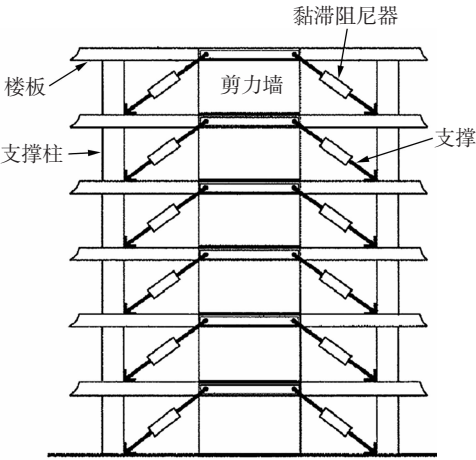


图 7 黏滞阻尼器耗能混合摇摆墙^[18]

Fig. 7 Hybrid rocking wall with viscous damper^[18]

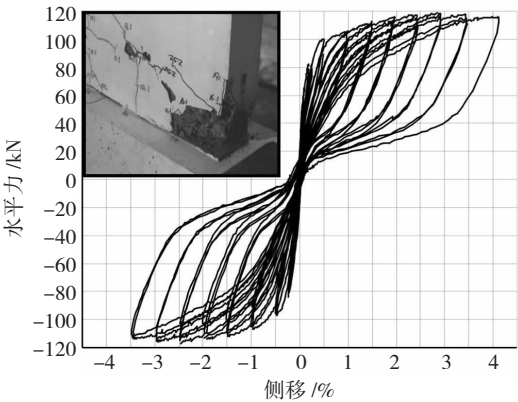


图 8 混合摇摆墙的滞回曲线^[19]

Fig. 8 Hysteretic curves of hybrid rocking wall^[19]

Smith 等^[20-21]根据 ACI 318^[22]、ACI - ITG 5.1^[23]和 ACI - ITG 5.2^[24]设计了 1 片等同现浇剪力墙和 5 片自复位剪力墙,进行了拟静力试验. 考虑到实际的施工情况,剪力墙由上下两部分墙片叠合拼装而成,见图 9. Smith 等^[25]在 ACI - ITG 5.1 和 ACI - ITG 5.2 的基础上完善了设计建议,编写了相应的设计指南,以使自复位剪力墙能够同时满足 ACI 318 和 ACI - ITG 5.1 中要求,同时提出了竖向拼接多墙片剪力墙控制水平接缝张开分析方法,使得摇摆只产生在墙 - 基界面.

Marriott 等^[26-27]提出将耗能元件外置(图 10),

代替在剪力墙内部锚固的普通耗能钢筋,方便在震后进行更换. 对 4 片采用不同外置耗能元件的剪力墙进行了高速拟静力试验和振动台试验,耗能元件包括软钢滞回耗能元件和速度黏滞型耗能元件两种. 试验发现破坏主要集中于外置的耗能元件上,通过更换耗能元件,结构的功能可以快速恢复,两种耗能元件结合使用可以比仅使用速度黏滞型耗能元件更好地控制结构的位移响应.



图 9 预制装配式混合摇摆墙^[28]

Fig. 9 Precast fabricated hybrid rocking wall^[28]



图 10 耗能元件外置混合摇摆墙^[26]

Fig. 10 Hybrid rocking wall with external damping elements^[26]

Henry 等^[29-32]提出了一种新型的带端柱的自复位剪力墙体系 (precast wall with end columns, PreWEC) 并进行了拟静力试验,如图 11 所示. 这种自复位剪力墙中部为摇摆墙,两侧为沿柱高通长布置预应力筋锚固的钢柱或混凝土柱,剪力墙与柱间安装 O 型连接件(图 12),通过 O 型连接件的弯剪变形耗能. Twigden 等^[33-34]对这种自复位剪力墙体

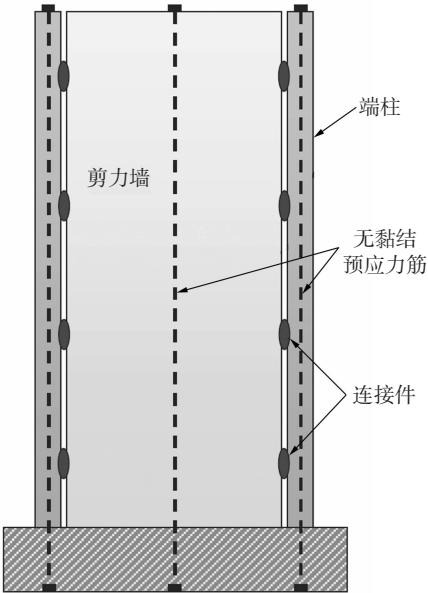


图 11 PreWEC 剪力墙^[31]

Fig. 11 PreWEC^[31]



图 12 墙柱间 O 型连接件^[34]

Fig. 12 O-connector between wall and column^[34]

系进行了振动台试验,研究了这种结构的动力响应与阻尼效应。

陈适才等^[35]建立了底部开缝后张预应力预制剪力墙的有限元模型,此类剪力墙在墙底部分设缝,中部与现浇剪力墙结构相同保持正常连接,保留了一定的抗弯和抗剪能力,同时通过后张无黏结预应力筋来保证剪力墙的自复位能力,见图 13。党像梁等^[36]设计进行了底部开水平缝预应力自复位剪力墙的拟静力试验,并进行了精细化的有限元模拟^[37-39]。试验结果表明此类剪力墙兼备现浇式剪力墙和自复位剪力墙的优点,不仅具有一定的耗能能力,还具有较好的自复位能力,变形能力较强,而且损伤和残余变形小。

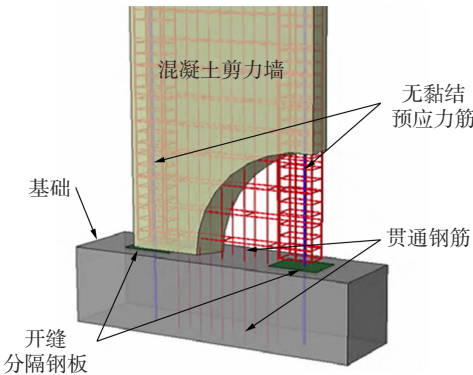


图 13 底部开水平缝预应力自复位剪力墙^[39]

Fig. 13 Self-centering shear wall with horizontal bottom slits^[39]

郭彤等^[40]设计了一种新型的自复位剪力墙结构(图 14),形式与 PreWEC 剪力墙相似,但剪力墙两侧为底部铰接的摇摆柱,并在底部墙片墙趾处外包钢板以防混凝土压溃。所采用的耗能元件为摩擦耗能元件(图 15),由钢板间内夹黄铜片,通过螺栓和弹簧垫圈预紧叠合而成。

2.2 联肢墙

2.2.1 通过竖向耗能元件连接的联肢墙

在 PRESSS 项目^[41]中的剪力墙即为采用了带竖向接缝的联肢剪力墙,相邻墙片间竖向接缝通过 U 型弯曲钢板耗能元件连接,见图 16。水平力作用下墙体产生摇摆,各摇摆墙片间的竖向接缝发生错动,U 型钢板在弯剪联合作用下屈服耗能。Sriharan 等^[42]对这种 U 型钢板连接件进行了往复加载试验,

并针对这种联肢剪力墙提出了简化的分析和设计方法^[43]。

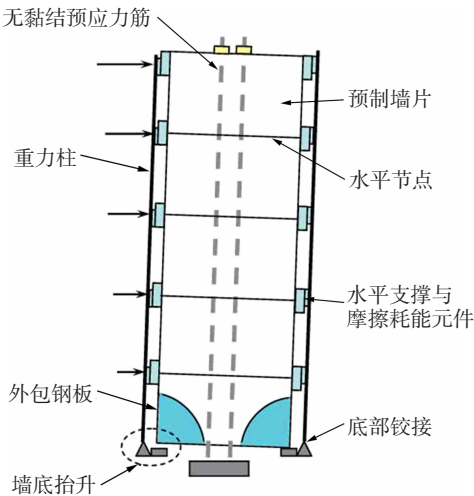


图 14 摩擦耗能元件耗能混合摇摆墙^[40]

Fig. 14 Hybrid rocking wall with friction damping elements^[40]

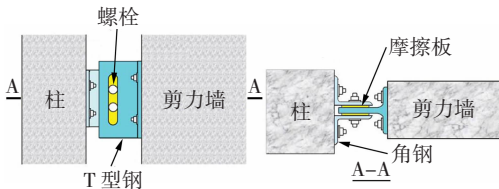


图 15 摩擦耗能元件构造^[40]

Fig. 15 Details of the friction damping element^[40]



图 16 U 型钢板连接件耗能联肢墙^[44]

Fig. 16 Coupled wall with U-shaped connectors^[44]

2.2.2 通过自复位连梁连接的联肢墙

Kurama 等^[45-49]提出将联肢墙间连梁由固结改为摇摆形式,见图 17,连梁与墙间通过后张预应力筋连接,通过连梁端部布置的上下角钢变形耗能。

Barbachyn 等^[50-51]将联肢墙间连梁弱化,提出了一种新的后张预应力混凝土连梁。这种连梁取消了斜向钢筋,但在截面形心配置了后张预应力筋,见图 18。试验发现这种连梁的自复位能力并不突出,但连梁的变形能力大幅提高,转动能力可达 11%,连梁的损伤集中于梁端,损伤表现为两端保护层混

混凝土剥落,而连梁的其余部分则基本保持弹性,连梁的整体破坏模式改变,而连梁变形延性提高的同时也提高了结构整体的变形能力。

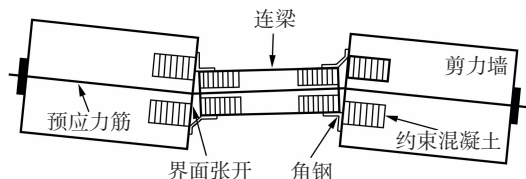


图 17 自复位耗能连梁^[49]

Fig. 17 Self-centering coupling beam^[49]

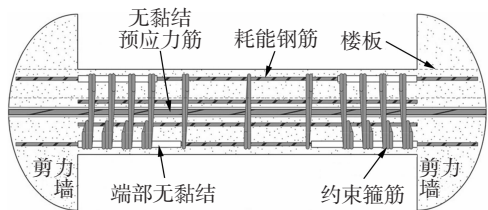


图 18 后张预应力连梁^[50]

Fig. 18 Post-tensioning coupling beam^[50]

3 分析计算理论与建模方法

3.1 直接基于位移的设计方法

3.1.1 基于力的设计方法的缺陷

随着抗震设计理论逐渐发展,人们意识到变形能够更直观地表达结构的损伤情况,从 20 世纪 90 年代起,基于性能的抗震设计(performance-based seismic design, PBSDB)^[52-54]不断发展,成为了结构抗震设计的主流方向之一。基于性能的抗震设计要求根据结构的重要性和用途确定其性能目标,使结构在地震中具备预期的功能,达到期望的变形水平。

目前各国规范中普遍采用的一般为基于力的设计方法(force based design, FBD),分析时需首先估计弹性状态下的基本周期,考虑到地震作用下使结构保持弹性并不现实,一般允许结构进入塑性,采用折减系数 R 对弹性反应谱进行折减。折减系数 R 的取值一般根据等位移原理确定,即在中等周期范围(0.6 ~ 2.0 s)内,理想弹塑性体系的变形与弹性体系变形相等,可近似认为折减系数 R 与延性需求 μ 相同。然而,自复位结构一般表现为旗帜型的变形特征,与理想的弹塑性滞回规则有较大差别,耗能能力差,延性更高(自复位梁柱节点位移延性可达 14^[14]),并不能满足等位移原理的要求^[55],显然,基于力的设计方法并不适合进行自复位结构设计。

Priestley 等^[56]提出了直接基于位移的设计方法(direct displacement-based design, DDBD)。直接基于位移的设计方法属于一种基于性能的抗震设计方法,但与其他方法的区别在于 DDBD 采用最大位移处的切线刚度而非初始弹性刚度来计算剪力,自复

位结构不会因为相对偏高的初始刚度和较高的延性而受到影响。此外, DDBD 方法采用位移谱而非加速度谱进行设计,在设计的初始阶段即可确定目标位移,一般不需要进行迭代过程,相比 FBD 方法, DDBD 方法计算获得的基底剪力更低^[57-58],从造价角度而言更为经济。

3.1.2 DDBD 的分析步骤

DDBD 的分析过程以 Shibata 等^[59]提出的等代结构为基础,基本步骤如下^[56]:

- 1) 确定目标位移 Δ_d ;
- 2) 根据变形模式,将多自由度体系转化为等效单自由度体系,对于楼层数少于 10 层的自复位剪力墙,变形模式可近似取为刚体转动变形;
- 3) 根据延性系数 μ 确定等效黏滞阻尼(equivalent viscous damping, EVD);
- 4) 根据位移谱确定等效单自由度体系的周期 T_e ;
- 5) 计算等效单自由度体系的等效刚度 K_e 。

$$K_e = \frac{4\pi^2 m_e}{T_e^2};$$

- 6) 将此刚度作为割线刚度计算基底剪力

$$F = V_{base} = K_e \Delta_d;$$

- 7) 分配剪力至结构各部分,进行截面设计。

详细的计算公式和参数取值可参考文献^[56]。

3.2 自复位剪力墙的极限状态与截面设计

在获得了截面的内力设计值后,需要对截面进行分析及配筋设计。根据自复位剪力墙的单调推覆曲线,其截面极限状态可分为消压、软化、屈服、耗能元件破坏和倒塌等几个状态,见图 19。在加载初始阶段,界面未发生摇摆,此时结构的变形类似于固结;随着水平力的增大,截面受拉边缘压应力为零,达到消压状态,此时摇摆界面的接触高度(受压区高度)逐渐减小,界面逐渐张开;随着摇摆幅值的提高,耗能元件屈服,结构开始有明显软化,此时刚度主要由预应力筋提供,刚度明显降低;在预应力筋屈服之后承载力基本不再提高,随后耗能元件和预应力筋发生破坏,结构失去稳定发生倾覆。

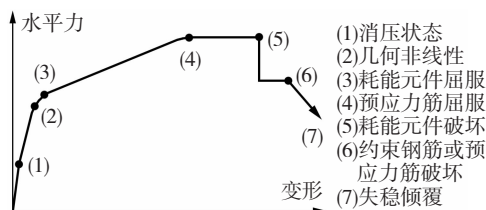


图 19 自复位剪力墙的截面极限状态^[60]

Fig. 19 Limit states of self-centering shear wall^[60]

虽然截面有多个极限状态,但一般取屈服和达到极限承载力时对应的两个极限状态进行截面配筋

设计. 由于节点约束释放,平截面假定并不满足,而且节点非线性高度集中,曲率近似为无穷大,因而常采用弯矩-转角方法进行分析,并开展具体的截面配筋设计计算^[24, 28, 61].

3.3 非线性建模与分析方法

非线性模拟分析中的分析模型主要包括纤维单元模型 (fiber model)、集中塑性模型 (lumped plasticity model)、多弹簧模型 (multi-spring model) 和连续有限元模型 (finite element model) 四类.

3.3.1 纤维单元模型

Kurama 等^[15]利用 DRAIN-2DX 软件建立了自复位剪力墙的纤维单元模型 (图 20),进行了单调推覆和动力时程分析,随后又将纤维单元模型应用在混合摇摆墙^[62]和自复位联肢墙^[45-46, 48]的模拟分析中. Erkmén 等^[63]将纤维单元模型的分析结果与已有自复位剪力墙试验数据进行了对比验证,发现对力-变形曲线模拟结果较为接近,但对预应力筋应力的模拟结果差距较大.

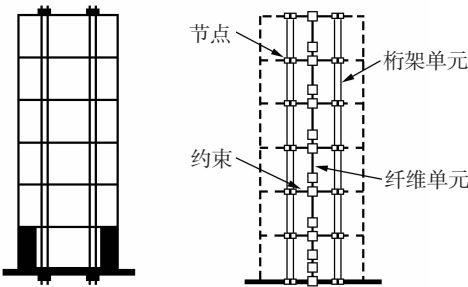


图 20 受控摇摆墙的纤维单元模型^[15]

Fig. 20 Fiber element model of controlled rocking wall^[15]

纤维单元模型分析的基础为平截面假定,混凝土截面被划分为若干纤维,钢筋和预应力筋采用桁架单元,预应力筋仅在两端约束自由度以实现无黏结,通过弯矩-曲率迭代分析,可以模拟从开始加载到最终破坏的全过程. 纤维单元模型的优势在于仅采用材料的单轴本构关系即可获得相当精确的模拟结果,但为了模拟摇摆,界面的张合是通过将节点附近单元抗拉强度和刚度设置为零来实现的^[64]. 虽然这样可在宏观上表达界面张开后结构的刚度变化,但却并不能真实反映结构的摇摆抬升和局部行为.

3.3.2 集中塑性模型

集中塑性模型是表达自复位结构节点弯矩-转角响应的一种宏观模型,见图 21. 通常情况下,摇摆界面由两个零长度的转动弹簧组成,分别代表界面的自复位的双线性变形特征 (自复位来源,如后张预应力) 和弹塑性耗能特征 (如滞回耗能元件),上部结构 (如柱、梁、墙或桥墩) 可近似简化为弹性梁单元. Pampanin 等^[65]最早采用这种模型来模拟自复

位混凝土框架中的梁柱节点,Parlemo 等^[66-67]建立了自复位桥墩的分析模型,Nazari 等^[17]利用这种模型对受控摇摆墙进行了模拟分析,Henry^[68]在双弹簧模型的基础上提出了一种七弹簧模型用于模拟 PreWEC 剪力墙,通过调整各弹簧参数,获得了较精确的模拟结果.

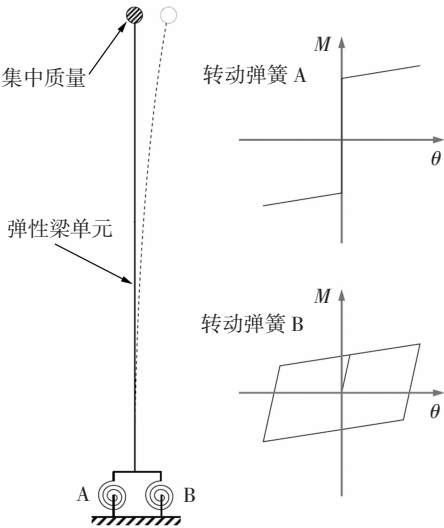


图 21 双弹簧模型^[68]

Fig. 21 Double-spring model^[68]

自复位结构的非线性行为高度集中于节点位置,结构的其余部分则基本保持弹性,符合集中塑性模型的假定,但这种模型同样无法表达摇摆界面局部的受力情况. 此外,为获得精确模拟结果,需要对摇摆界面进行深入的截面分析^[60]. 有学者提出了悬臂梁等效方法^[65, 69],即假定自复位悬臂梁与固结悬臂梁在达到相同侧移时截面应变分布相同,此过程与弯矩-曲率分析方法类似,但需要先假定受压区高度,通过不断迭代确定截面平衡. Sritharan 等^[42]通过分析发现自复位剪力墙一般在达到 0.5% 侧移后截面受压区高度基本不再变化,可通过预先定义受压区高度与转角关系,提出简化分析方法^[43],从而避免迭代过程,能较准确预测弯矩、受压区高度、预应力筋应力和混凝土墙趾受压应变,而且这种方法的准确性也在后续研究得到了进一步验证^[17].

3.3.3 多弹簧模型

多弹簧模型同样为一种宏观模型,见图 22. 在多弹簧模型中,摇摆界面处布置有多个轴向弹簧,将弹簧设置成仅能受压来实现界面的摇摆,预应力筋和耗能元件同样通过设置成弹簧来进行定义,上部结构则一般简化为弹性梁单元.

Conley 等^[70]采用多弹簧模型模拟了 PRESSS 项目中的联肢墙,在墙体底部每侧受压区形心各布置了一个弹簧,近似认为转动中心在两侧弹簧的位置. 但实际上截面转动中心应在中和轴,该假设将高

估预应力筋的伸长量和截面弯矩^[60]. Spieth 等^[71]提出了一种包含 10 个接触弹簧单元的模型,用于模拟自复位混凝土梁柱节点,同时考虑了不同弹簧数量和弹簧位置分布对于模拟精度的影响。

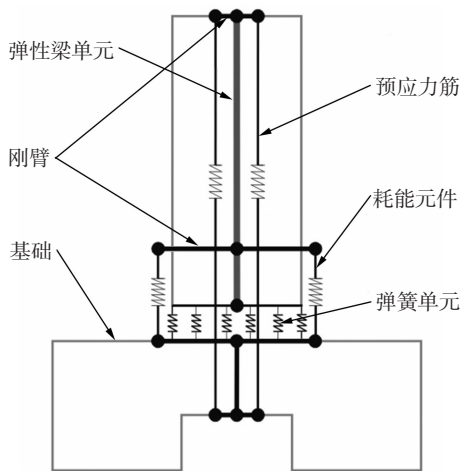


图 22 多弹簧模型^[60]
Fig. 22 Multi-spring model^[60]

Palermo 等^[72]对自复位混凝土梁柱节点和自复位剪力墙进行模拟分析发现,多弹簧模型相比集中塑性模型拟合程度更好,而且在考虑预应力筋的塑性之后能够取得更好的模拟结果. Marriott^[60]继续对已有的多弹簧模型进行了改进,提出了确定弹簧轴向刚度的方法,并根据试验的双向加载工况提出了三维多弹簧模型. 杨博雅等^[73]使用 OpenSees 在墙体与基础接缝处设置了一系列桁架单元,采用 CONCRETE01 材料(不考虑受拉)来实现墙体的摇摆抬升。

多弹簧模型的优点在于可对摇摆界面局部行为进行较精确模拟,包括应力、应变、中和轴高度以及构件的伸长效应^[74]. 但为获得较高精度的模拟结果,对自复位混凝土框架节点,至少需要 8 个弹簧单元^[75],而自复位剪力墙截面高度更大,至少需要 20 个弹簧单元才能更好地模拟刚度的退化^[76]. 弹簧数量提高能够提高模拟精度,但弹簧的参数较难确定,虽然考虑材料塑性而将弹簧设置为双线性更合适,但计算稳定性会降低^[60].

3.3.4 连续有限元模型

Kurama^[18]采用 ABAQUS 建立了自复位剪力墙的有限元模型,墙体采用平面应力单元,摇摆通过定义接触实现,获得了与纤维单元模型非常接近的结果. 马昕等^[77]通过实体有限元模拟,研究了软钢滞回耗能元件数量、位置和长度对剪力墙自复位性能及抗震性能的影响. 党像梁等^[78]分别建立了自复位剪力墙的实体单元、平面应力单元和薄壳单元的有限元模型,与试验结果进行了对比,比较了三种单元

的计算效率和精度,并验证了动力显式分析中采用壳单元进行模拟的可行性与合理性. Henry 等^[31]利用 ABAQUS 建立了 PreWEC 剪力墙的实体有限元模型,模拟结果与拟静力试验结果符合良好,并且通过建模分析了剪力墙摇摆抬升对相邻楼板的影响。

相比于其他三类单元,有限元模拟分析多采用通用有限元软件,计算效率最低,但有更多的单元类型和本构参数模型可供选择,通过精细建模,能够在分析过程中反映剪力墙的真实情况,如界面摇摆,受压区高度,耗能元件受力情况等。

4 研究展望

新西兰、美国和日本等国家近年来已针对自复位剪力墙进行了较为深入的研究,研究成果已经体现在相关规范和设计指南中^[23-24, 79-80],并将研究成果成功应用于一批采用了自复位技术的试点工程^[81-83],值得说明的是已有采用混合框架-自复位剪力墙体系的医院建筑在强震后基本无结构损伤,震后使用功能未中断^[83],说明了自复位结构体系在抗震设计理念上的先进性. 基于前述国内外研究现状的总结和分析,提出了需解决的关键科学问题。

4.1 预应力筋的变形能力

后张预应力钢绞线具有较强的弹性变形能力,作为经济可靠的回复力来源,在预应力混凝土结构和自复位结构中都有广泛的应用,但其应变状态没有引起足够的重视. 在自复位结构体系中,预应力筋在构件截面中心直线布置,反复的水平侧移导致预应力筋的应力在有效应力的基础上发生反复的变化(图 23),这种变化在进入大变形状态后更加明显,应变也会在有效预应力对应的应变的基础上反复增加。

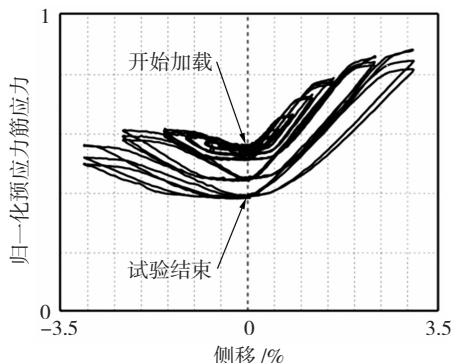


图 23 拟静力滞回加载中预应力筋的应力变化^[28]

Fig. 23 Stress development in post-tensioning tendon subjected to cyclic loading^[28]

虽然钢绞线在普通的单调拉伸试验中延伸率基本可以达到 7% 左右,但实际上由于锚具中夹片对于钢绞线截面的削弱作用,钢绞线在达到较小应变时即可能发生单丝破断,见图 24. Bruce 等^[84]通过

对 12.7 mm 直径 1 860 MPa 级 1×7 钢绞线的反复加载试验发现,钢绞线在发生单丝断裂后,卸载至张拉应变时预应力就已全部丧失,钢绞线刚度也有较大下降.已有的有关钢绞线的非线性滞回加载试验发现钢绞线发生单丝断裂时的延伸率约在 1~4%^[84-88].对于自复位结构,在摇摆界面张开后,结构的抗侧刚度则主要由预应力筋提供,预应力筋的破断和屈服是控制结构倒塌和保证其自复位能力的关键,如何提高预应力筋的变形能力有待于进一步研究.



图 24 预应力钢绞线的单丝破断^[89]

Fig. 24 One-wire fracture of the post-tensioning strands^[89]

4.2 自复位单元与相邻结构的相互作用

自复位剪力墙自身刚度和承载力偏低,很少作为主要抗侧体系独立出现,常与框架结构结合形成框架-摇摆/自复位剪力墙体系^[90-95].剪力墙作为整体性关键构件,可以控制框架实现整体屈服破坏机制,避免出现薄弱层,自复位剪力墙提供的回复力也可以降低结构的残余变形.

自复位剪力墙需要与相邻结构形成可靠连接以保证协同工作,但剪力墙摇摆过程中的墙体抬升会对连接构件^[96]和相邻的结构构件^[68, 97]造成严重损坏.虽然框架对整体结构屈服后刚度的提升有助于控制残余变形^[98],但模拟分析中常常忽略的构件,如楼板、填充墙和楼梯等,却会在一定程度上阻碍回复力的作用,削弱结构的自复位性能^[99].

4.3 冗余度与鲁棒性的提升

为了实现摇摆机制,自复位剪力墙在节点处一般仅通过预应力筋和耗能元件连接,构件内部的纵向受力钢筋和混凝土在节点处断开,约束的释放使得此类结构缺乏足够的冗余度.实际上,在已有的试验研究中预应力筋和耗能元件破坏的情况并不少见.Yamashita 等^[100]进行的自复位混凝土柱振动台试验中发现锚具夹片破坏,导致预应力失效,此时预应力筋的应变仅为 1.05%;Restrepo 等^[19]进行的混合摇摆墙拟静力试验中,在墙体侧移达到 3% 时耗能钢筋发生断裂;在 UCSD 户外振动台完成的三层预制装配式混凝土结构的振动台试验中发现,在设计水平地震下,墙体内部的耗能钢筋发生断裂,且预

应力钢绞线存在不均匀张拉,直接导致随后在最大设计地震水平下钢绞线断裂^[96].

这充分说明尽管自复位剪力墙在控制震损和残余变形方面有着明显的优势,但却面临着冗余度和鲁棒性不足的问题,结构缺乏足够的安全储备.在极端情况下,剪力墙的摇摆将失去控制,结构的承载力大幅降低,自复位能力丧失,极有可能发生倾覆或倒塌.而通过引入随摇摆幅值提高而分步激活的多组耗能元件,或者结合多种耗能摇摆方式,可以建立抵抗倒塌的多道防线^[101],但如何进行分析设计还有待于进一步的试验验证.

4.4 高阶振型的影响与多节点摇摆的利用

能力设计的目标是使得结构在预期位置产生塑性铰,对剪力墙而言则希望塑性铰产生在墙底.塑性铰形成后,基底弯矩便不再增加,但结构的其余部分的弯矩和剪力仍可能继续增长^[102],高阶振型的分析可以通过分析对应的底部铰接体系来实现^[103].随着地震动幅值的提高,塑性铰弯矩的限制导致一阶振型受限,而铰接体系高阶振型占比提高导致剪力合力作用点降低,在塑性铰弯矩不变的情况下,基底剪力相比静力加载情况(线性分布,近似一阶振型)更大,而且这种现象在拥有长自振周期和高延性需求的结构中更加明显^[104].

自复位节点的耗能摇摆与塑性铰类似,因此,上述问题在自复位剪力墙中同样存在,研究中也发现了这种剪力的放大效应^[16, 76, 105-107].考虑到自复位结构一般为装配式结构,Wiebe 等^[103]提出通过允许多节点摇摆的形式降低高阶振型的作用,通过设置额外的耗能摇摆界面,新增摇摆界面处的弯矩将被限制,见图 25,而弯矩的微分为剪力,弯矩降低后剪力同样会降低^[108].

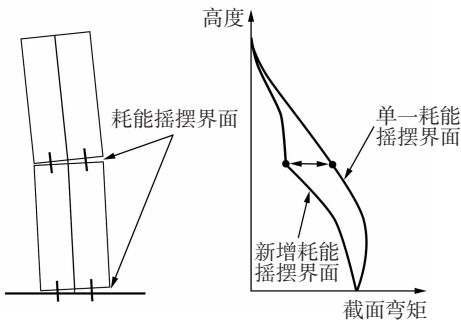


图 25 新增摇摆界面后对弯矩和剪力的控制作用^[108]

Fig. 25 Reduction of moment and shear force with additional rocking joint^[108]

虽然有学者指出设置多个耗能摇摆界面/塑性铰后变形的提高并不大^[103, 109],但释放约束会降低结构的刚度,对结构刚度和变形的影响需要进一步验证.在设置多个耗能摇摆界面后,可以尝试在控制

内力的同时实现多节点多层次耗能,从整体层面上提升剪力墙的冗余度和鲁棒性。

4.5 残余变形的控制与预测

各类耗能元件的使用提高了自复位结构的耗能能力,但也导致了残余变形增大的可能。为保证自复位能力,Stanton 等^[110]最早提出了一种基于静力平衡的残余变形控制方法,即要求预应力提供的回复力应能够使耗能钢筋屈服来实现自复位,这一思想也体现在许多规范和标准中^[24, 79, 80, 111]。但是在真实的地震过程中,结构在震后还将进行自由衰减振动,最终的残余变形有减小的趋势^[112-113],最终的残余变形可能只产生在很小的范围内,见图 26,说明根据拟静力试验等低速加载条件下获得的结构残余变形可能并不准确。

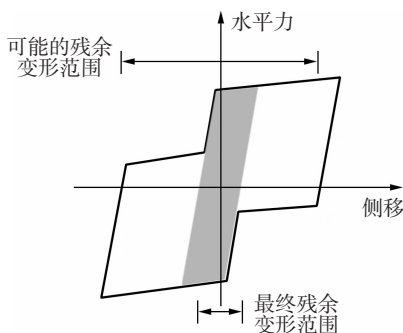


图 26 结构可能残余变形范围^[99]

Fig. 26 Likely range of residual drift^[99]

结构的损伤与变形密切相关,因而基于性态的抗震设计能够比较好地控制结构损伤,但却不一定能够很好地控制残余变形。以往的抗震设计中往往只关注结构在地震过程中的最大响应和结构的耗能能力,但是最终的残余变形也是评估结构震后安全性与修复费用的重要指标。虽然已有学者分析了控制残余变形大小的影响因素^[55, 112-117],但如何能够在设计阶段精确预测残余变形仍需要大量的分析研究来实现。

5 结 论

通过对国内外研究现状的总结和分析,可获得的结论为:

1) 通过对自复位剪力墙结构工作原理、技术优势和理论发展现状的总结和分析,可知作为可恢复功能结构体系的重要组成部分,自复位剪力墙在抗震设计理念上具有明确的先进性。

2) 应在自复位剪力墙结构体系研发,性能、分析与设计方法等方面进行深入系统的研究,为这种具有明确优势的结构体系的应用提供技术和理论支撑。

3) 尚应对预应力筋的受力行为、楼屋盖等水平连接影响、自复位剪力墙的冗余度和鲁棒性、多墙片

拼装竖向多节点摇摆的自复位剪力墙高阶振型效应,以及残余变形分析与预测等进一步开展研究。

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