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液化场地桩基地震失效机制：现状与挑战

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摘 要: 震害调查表明, 多次破坏性地震中均发生液化场地桩基严重破坏的典型实例。针对地震土体液化诱发桩基破坏这一突出难题, 国内外学者围绕液化场地桩基抗震的紧迫需求, 采用多重研究方法, 揭示液化场地桩基弯曲、屈曲和沉降等多种失效模式, 取得系列丰硕研究成果。首先开展典型液化场地桩基震害实例的调研与分析, 总结液化场地桩基震害主要的表现形式和破坏模式, 剖析各种桩基失效模式触发机制和演化机制, 指出当前研究的重难点问题, 并给出液化场地桩基抗震研究进一步需要努力的方向。期望这项工作能够给研究者们提供一些有用的参考, 并助力于我国液化场地桩基抗震理论的不断发展和完善。

关 键 词: 地震; 液化场地; 桩基础; 失效机制; 震害实例

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Failure mechanism of pile foundations in liquefiable soils under seismic loading: status and challenge

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Abstract: The investigation of earthquake damage shows that the pile foundation in liquefiable soils has been severely damaged in many destructive earthquakes. In view of the prominent problem of pile foundation disaster induced by soil liquefaction, many scholars have adopted multiple research methods to explore the earthquake damage mechanism of pile foundation in liquefiable soils, aiming to meet the urgent need of aseismic design of pile foundation in liquefiable soils. And they have obtained fruitful and important research results, revealing the failure modes of pile foundation in liquefiable soils, such as bending failure, buckling failure and settlement failure. This study surveys and analyzes the typical cases of pile foundation earthquake damage in liquefiable soils, and summarizes the manifestations and failure types of pile foundation earthquake damage. On this basis, the paper dissects progress of different earthquake failure mechanisms of pile foundation, and points out the key and difficult points in the current research. Finally, the insufficiency and challenges of seismic research on pile foundation in liquefiable soils are revealed, and the further research directions are given. This work can provide some useful reference for the researchers, and it is beneficial to the development and perfection of seismic theory of pile foundation in liquefiable soils.

Keywords: earthquake; liquefiable soils; pile foundations; failure mechanism; earthquake damage cases

1 引 言

历次强震中均发生大量的砂土液化造成桩承

结构显著毁坏的典型事例, 如 1923 年日本关东大地震、1964 年日本新潟地震、1976 年中国唐山地震、1989 年美国旧金山大地震、1995 年日本阪神

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大地震、1999 年中国台湾集集地震、2008 年中国汶川地震、2010 年智利大地震、2015 年尼泊尔大地震以及 2017 年墨西哥大地震等^[1-9]。砂土液化对人民生命财产安全造成严重威胁。近几十年, 众多学者从历次震害中不断吸取教训和经验, 深入研究液化场地桩基地震失效机制, 促进液化场地桩基抗震规范的完善和不断发展^[10-20]。

本文将从典型的桩基强震破坏实例出发, 系统总结国内外液化场地桩基震害机制在理论分析和试验等方面的研究成果, 分析液化场地桩基破坏主要特征和重要类型, 剖析桩基震害主要诱因, 并指明这一领域亟需解决的关键科学问题, 以飨读者。

2 实例分析

自 1964 年日本新潟地震和美国阿拉斯加地震以来, 液化场地桩基抗震问题受到高度重视。国内外学者开展了大量液化场地桩基震害调查、模型和现场试验、理论分析等工作。震害调查作为研究液化场地桩基抗震问题最直接且有效的方法, 也是试验研究和理论分析的基础。然而, 震害调查机会难得、实例稀少, 可供广大研究者参考的公开资料较少。本文通过调研国内外相关文献, 摘选并剖析 17 个典型桩基震害实例。

实例 1: 1964 年日本新潟地震造成昭和大桥 (Showa Bridge) 坍塌。根据目击者回忆^[13], 推测出桥梁坍塌、土体发生液化侧扩流发生时间: 地震峰值加速度约在 20 s, 大桥在 70 s 后开始坍塌, 86 s 后岸坡发生液化侧扩流。这或许说明桥梁破坏并非由地震惯性作用或液化土体侧扩流引发。Bhattacharya 等^[3]研究发现, 土体液化导致桩基土体丧失约束, 在轴向荷载下桩基发生屈曲失效 (见图 1)。

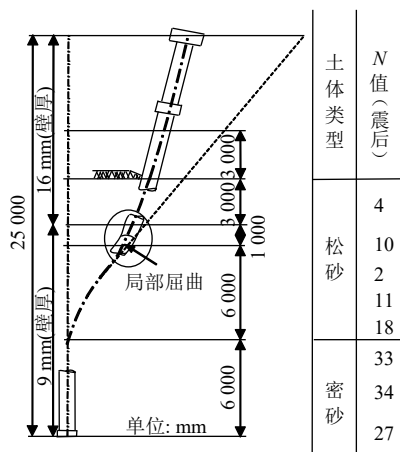


图 1 昭和大桥 P4 桩基破坏情况和土层条件^[3]

Fig.1 Damage of Pile 4 in Showa Bridge and soil profiles^[3]

实例 2: 日本八千代桥 (Yachiyo Bridge) 的基础形式是钢筋混凝土群桩, 其中桩径为 0.3 m, 桩长为 10 m。桥台和基础在 1964 年日本新潟地震中严重损坏, 桥台不均匀沉降和倾斜, 桩身出现弯曲裂缝 (裂缝集中于土层交界处), 岸边地表侧向变形达 6 m。Lin 等^[21]基于非线性温克尔地基梁模型法^[22-23]分析后认为, 日本八千代桥 (Hamada^[24]) 属于弯曲破坏, 而日本昭和大桥 (Hamada^[24]、Bhattacharya 等^[3, 25]) 是屈曲失效。

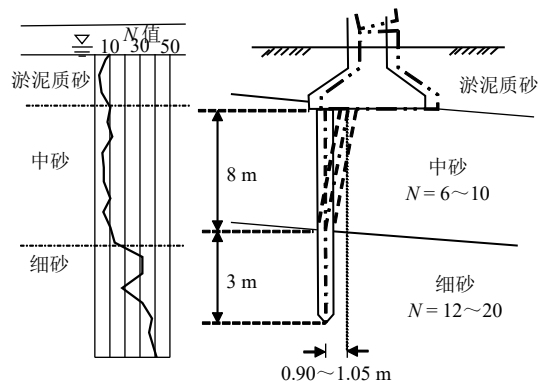


图 2 八千代桥的桩基损伤和变形情况^[24]

Fig.2 Damage and displacement of Yachiyo Bridge pile foundation^[24]

实例 3: 4 层的房屋基础为钢筋混凝土桩基^[24], 其中桩径为 0.35 m, 桩长为 12 m。建筑所处可液化土层厚约 10 m。1964 年日本新潟地震后, Hamada^[24]现场调查了 74 根基桩, 所有基桩均呈现相似的破坏特征, 即在距桩顶 2.5~3.5 m 和距桩端 2~3 m 位置出现破损。

实例 4: 唐山市陡河胜利桥是一座长 66 m 的五跨钢筋混凝土桥梁。1976 年唐山地震中桥梁倒塌^[26]。现场调查发现, 河岸两侧土体发生侧向流动, 桥台和桥墩倾斜严重, 近岸桩基折断后压在桥面板下, 其余桩基均向西倾斜, 并在接桩位置处折断, 整座桥梁塌落 (见图 3)。



图 3 1964 年唐山地震中胜利桥倒塌^[26]

Fig.3 The Shengli Bridge collapsed in 1964 Tangshan earthquake^[26]

实例 5: 天津某化工厂车间为 5 层框架结构^[26], 左侧采用筏基础, 右侧为桩基础。1976 年唐山地震后, 桩基几乎未沉降, 而左侧筏基础沉降约 0.3 m, 导致上部结构出现不均匀沉降, 并在沉降缝处发生局部破坏。

实例 6: 天津某糖库基础为 2×2 群桩基础^[26], 其中一侧两基桩长为 18 m, 另外一侧两基桩分别长 9 m 和 12 m。糖库位于可液化土层上, 土层厚 15 m。1976 年唐山地震后, 两根短桩由于土体液化失去承载力, 两根较长桩在偏心荷载作用下发生破坏。

实例 7: 1983 年日本海中部地震中某仓库发生严重震害^[24]。仓库基础是预应力混凝土桩基, 桩外径为 0.6 m, 壁厚为 0.1 m, 长为 18 m。地震后, 经现场开挖和检测, 可液化土层厚约 14 m, 桩在埋深 7.5 m 和 15 m 附近发生破损。

实例 8: 1993 年日本北海道地震中, 液化场地某 2 500 t 级水泥仓发生 3° 倾斜, 不均匀沉降达 0.9 m, 侧向位移达 0.45 m。Mori 等^[27-28]对桩基进行开挖、勘察, 通过钻孔相机勘测桩身破损状况, 见图 4。桩基损坏位置出现在桩顶或桩顶以下 1~2 m 处, 以及土层交界处。

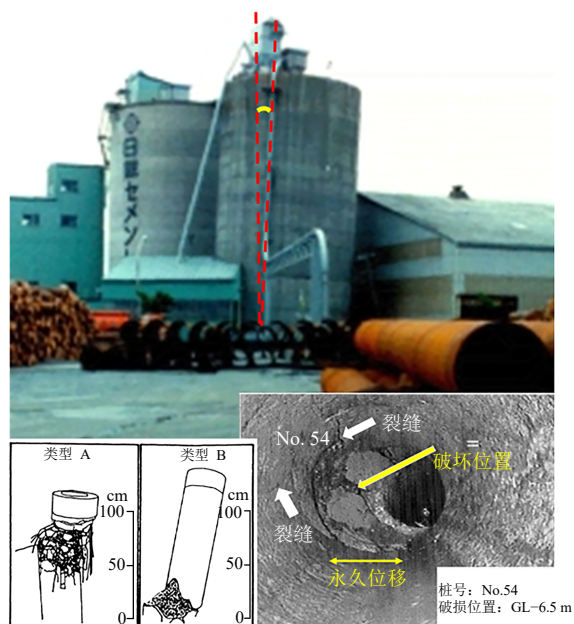


图 4 1993 年北海道地震中某水泥存储仓^[27]
Fig.4 A cement storage bin during the 1993 Hokkaido earthquake^[27]

实例 9: Lew 等^[29]现场考察了 1994 年美国北岭地震中一座建筑物, 建筑基础为嵌岩桩 (端承桩)。地震中建筑物并未发生显著破坏, 建筑物周围混凝土路面发生 3.8~5.1 cm 沉降。可见, 桩基能够控制

液化场地建筑沉降。

实例 10: Miwa 等^[30]考察了 1995 年日本阪神地震中一栋房屋状况, 房屋距离海岸挡土墙约 100 m, 基础是钢管桩, 其中桩径 0.4 m, 壁厚 9.5 mm, 长 27.5 m。场地可液化土层厚度约 8 m。地震后, 岸边土体侧向变形达 2~4 m, 距离挡土墙最近的桩基变形超过 1 m, 桩顶和土层交界处出现损坏。Miwa 等^[30]认为桩基响应主要是受土体位移控制, 场地完全液化前, 土体和桩基已经产生侧向变形。

实例 11: 某建筑物在 1995 年日本阪神地震中严重损坏^[8]。建筑物基础是预应力混凝土管桩, 其中桩径为 0.45 m, 长为 28 m, 场地人工填土层 (可液化土层) 厚度约为 8.5 m。经现场勘察, 地表沉降 0.3~0.6 m, 并存在喷砂冒水迹象。桩基在土层交界处破坏严重, 桩身向西南方向弯曲, 其中一根桩基在桩顶附近存在明显剪切裂缝, 见图 5。Fujii 等^[8]认为由于惯性荷载在桩顶附近产生剪切作用, 导致地震早期阶段桩顶附近出现剪切裂缝。之后, 液化土体侧向变形导致桩基在土层交界处弯矩过大, 发生弯曲破坏。

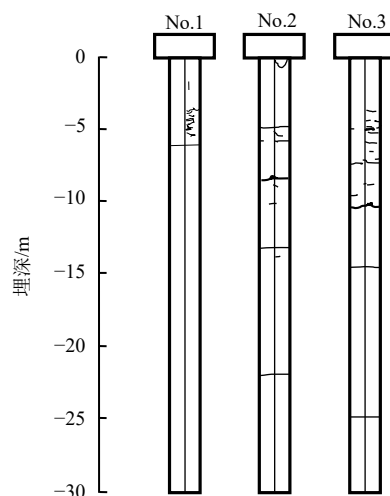


图 5 某 3 层建筑物的桩基损伤情况^[8]
Fig.5 Damage to the pile of a three-story building^[8]

实例 12: 某临海储油罐桩基础直径为 0.4 m, 长为 23 m, 可液化土层厚约 13 m^[31]。1995 年日本阪神地震后, 储油罐地基存在液化迹象, 储油罐周围土体最大侧向变形为 0.5 m, 最大桩顶位移达 0.6 m, 桩身裂缝集中于土层交界处, 桩身破损情况 (包括位移和裂缝) 见图 6。可见, 桩基在液化土层和非液化土层交界处破坏最为严重。

实例 13: 1995 年日本阪神地震后, Tazoh 等^[32]对某高速公路桥梁桩基开挖、勘察, 桩直径为 0.45 m, 长为 35 m, 桩顶和土层交界处出现大量裂缝, 桩顶

位移动达 1 m, 见图 7。Tazoh 等^[32]指出地震中桩-土间运动相互作用和地震后土体侧扩流导致了桩基损坏, 并建议设置板桩墙和加强配筋 (尤其是桩顶处) 防止桩基破坏。

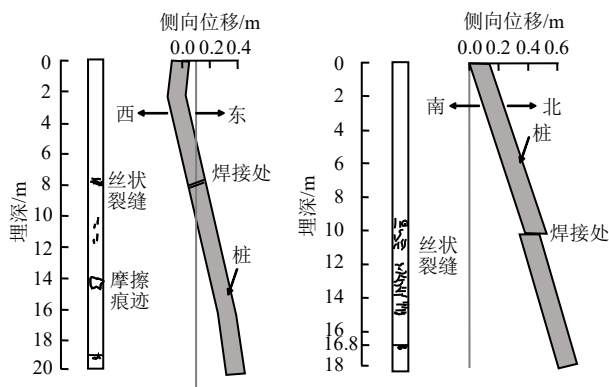


图 6 储油罐的桩基裂缝和变形^[31]

Fig.6 Cracks and displacement of storage tanks pile^[31]

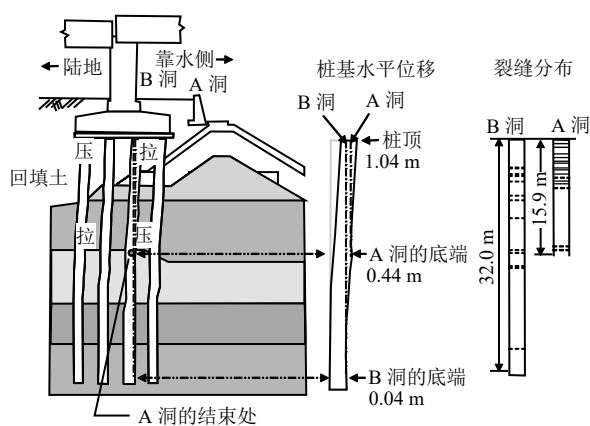


图 7 某高速公路桥梁桩基的裂缝和变形^[32]

Fig.7 Cracks and displacement of the pile of expressway bridge^[32]

实例 14: 某高架桥桥墩的基础是 2×2 现浇混凝土桩, 其中桩直径为 1.2 m, 长为 20.5 m。1995 年日本阪神地震后, 勘察显示桥墩没有损坏痕迹, 但桩身存在些许裂缝, 主要集中在桩顶处 (2 条裂缝) 和纵向配筋突变处 (3 条裂缝), 裂缝宽度在 2~4 mm 之间, 见图 8。Luo 等^[33]认为桩顶处裂缝是上部结构的惯性荷载导致, 而纵向配筋突变处的裂缝主要是土体变形导致。

实例 15: 2018 年汶川地震后, Zhou 等^[34]对岷江水沿岸的百花大桥勘察发现, 桥梁周围存在明显喷砂冒水迹象, 土体侧向变形达 0.4 m, 桥墩和土体间裂缝约 0.2 m。桥墩双柱破坏, 主梁脱落, 桥梁倒塌严重。显然, 土体液化侧扩流是造成桥梁破坏的重要原因。

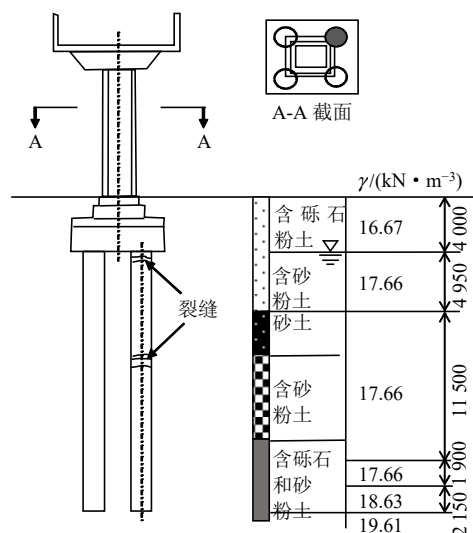


图 8 某高架桥土体剖面和桩基损伤 (单位: mm)^[33]

Fig.8 Soil profile and damage to the pile foundation of a viaduct (unit: mm)^[33]

实例 16: 2017 年墨西哥地震对当地结构和建筑物造成不同程度的损坏, Franke 等^[4]勘察发现端承桩抗震性能明显优于摩擦桩, 摩擦桩结构容易产生不均匀沉降和倾斜。结构和场地共振放大效应是结构破坏的重要原因。同时指出特殊场地条件下 (可液化土、人工填土和软土等) 建筑物的震害更为严重。

实例 17: 野马滩大桥在 2021 年青海玛多地震中出现严重的落梁现象, 场地喷砂冒水迹象明显, 黄勇等^[35]发现主梁、盖梁、支座、柱墩等均呈现出不同程度的损坏。Yuan 等^[36]认为野马滩大桥震害是由于场地液化破坏效应考虑不足造成的。

上述震害实例发生时间和破坏情况见表 1。综上所述, 液化场地桩基震害表现形式多为桩顶处裂缝、土层交界处裂缝和配筋突变处裂缝等; 从裂缝形状和发展趋势判别, 裂缝类型主要为弯曲裂缝、弯剪裂缝和剪切裂缝。桩基在土体侧向流动荷载和上部惯性荷载共同作用下易发生弯曲和剪切失效。随着土体液化, 桩周土约束减弱, 桩基在竖向荷载作用下存在发生屈曲失稳 (嵌岩桩) 和沉降 (非嵌岩桩) 的风险。同时, 土体液化增大桩-土-结构体系周期导致动力失效。显然, 桩基不同失效模式的触发条件存在差异性, 探究不同失效模式的机制是保证桩基地震安全的重要内容。

针对液化场地桩基不同失效模式, 国内外学者开展了大量桩基地震失效机制的研究工作, 本文分别归纳液化场地桩基不同失效模式的机制研究现状与挑战。

表 1 震后调查案例总结
Table 1 Summary of post-earthquake investigation cases

序号	年份	地震事件	来源	桩基损坏情况	结构损坏情况	桩基可能破坏类型
1	1964	新潟地震	昭和大桥 Yoshida 等 ^[13] 、 Bhattacharya 等 ^[25]	变形、折断	桥梁坍塌	屈曲、动力失效
2	1964	新潟地震	Yachiyo 桥 Lin 等 ^[21]	土层交界处弯曲裂缝	不均匀沉降、倾斜	弯曲破坏
3	1964	新潟地震	房屋基础 Hamada ^[24]	裂缝、变形	不详	弯曲破坏、屈曲
4	1976	唐山地震	唐山市陡河胜利桥 刘金砺等 ^[26]	变形、折断	桥梁坍塌	弯曲破坏、液化导致周期变化
5	1976	唐山地震	天津某化工厂车间 刘金砺等 ^[26]	不详	不均匀沉降、裂缝	未破坏
6	1976	唐山地震	天津某糖库 刘金砺等 ^[26]	较长桩基折断	不详	弯曲破坏、屈曲
7	1983	海中部地震	某仓库 Hamada ^[24]	裂缝和变形	不详	弯曲破坏、屈曲
8	1993	北海道地震	水泥仓 Mori 等 ^[27-28]	桩顶、桩顶下 1~2 m 处和 土层交界处裂缝、折断	3° 倾斜、不均匀沉降达到 0.9 m， 侧向位移 0.45 m	弯曲破坏、屈曲
9	1994	北岭地震	桩承建筑物 Lew 等 ^[29]	不详	周围土体发生了沉降，桩承建筑未 发生破坏	未破坏
10	1995	阪神地震	两层房屋建筑 Miwa 等 ^[30]	桩顶变形超过 1 m	沉降、倾斜，一层柱子破坏严重	弯曲破坏
11	1995	阪神地震	某桩承建筑物 Fujii 等 ^[8]	桩顶裂缝、土层交界处弯曲裂缝	不均匀沉降、倾斜	弯曲破坏
12	1995	阪神地震	日本某储油罐 Ishihara 等 ^[31]	桩顶位移 0.5 m、土层交界处裂缝	不详	弯曲破坏
13	1995	阪神地震	某高速公路桥梁 Tazoh 等 ^[32]	桩顶处裂缝、桩顶位移 1.04 m	桥面板移动 1.04 m、不均匀沉降	剪切破坏、弯曲破坏
14	1995	阪神地震	日本某高架桥 Luo 等 ^[33]	桩顶裂缝和土层交界处裂缝	未损坏	剪切破坏、弯曲破坏
15	2008	汶川地震	百花大桥 Zhou 等 ^[34]	不详	桥墩破坏、桥梁倒塌	不详
16	2017	墨西哥地震	Franke 等 ^[4]	不详	房屋等建筑轻微损坏	沉降破坏、弯曲破坏、剪 切破坏
17	2021	玛多地震	野马滩大桥 Yuan 等 ^[36] 、沮林 ^[37]	局部混凝土保护层脱落	落梁	剪切破坏、动力失效

3 桩基破坏机制

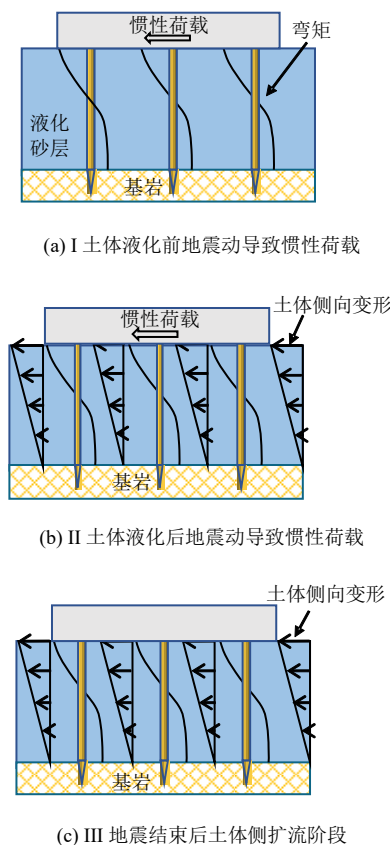
3.1 弯曲破坏

强震中液化场地桩基侧向荷载主要有运动荷载和惯性荷载。自 1964 年新潟地震以来，很多学者试图揭示液化场地桩基强震失效机制与成因条件，得到重要启示：液化场地桩基失效主要由侧向荷载作用于桩上产生过大弯矩所致^[23, 31]，即弯曲破坏机制。同时，国内外提出多种桩基弯曲失效理论，以 Ishihara^[38]的下行效应和上行效应概念、Tokimatsu 等^[20]的三阶段理论最为著名，见图 9。

Brandenberg^[39]认为传统桩基抗震设计中对土体侧扩流作用考虑不足，增加了桩基地震风险，并通过离心机试验揭示不同地震阶段桩基承受的惯性荷载和土体侧扩流荷载差异显著，比现行规范中的假设复杂。Tokimatsu 等^[40]通过振动台试验和数值模拟，研究上部结构固有周期和场地卓越周期对桩

基承受惯性荷载和运动荷载的影响，并认为二者共同作用导致桩基弯矩过大，从而引起桩基发生弯曲破坏。之后，Tokimatsu 等^[20]通过对 1995 年阪神地震现场调查和研究，将桩基弯曲破坏划分 3 个阶段：液化前、液化后和土体侧扩流阶段。孔隙水压力累积之前，上部结构的惯性力起主导作用，称为第 1 阶段；液化土体运动荷载随着孔隙水压力的增大，开始起到主导作用，称为第 2 阶段；震动结束后，特别是土体发生永久侧向位移时，运动荷载起主导作用，称为第 3 阶段。

针对液化侧扩流场地桩基抗震问题，Cubrinovski 等^[41]提出一种简化的液化侧扩流场地桩基内力计算方法。Ricardo 等^[42]开展多组离心机试验探究上部结构刚度对弯矩的影响，研究表明，最大弯矩始终出现在液化土与非液化土层交界处，峰值弯矩随上部结构刚度增大而减小。Liu 等^[43]通过离心机试验分析地震中孔隙水压力、弯矩、位移等

图9 三阶段失效分析理论^[20]Fig.9 Schematic diagram showing the pile “three stage” failure^[20]

响应规律, 给出液化场地桩基抗震设计建议。Tang 等^[44]通过振动台试验研究钢筋混凝土高承台桩基在土体液化过程中的动力特性, 通过宏观现象和内力分析认为其破坏模式为典型的弯曲破坏。Finn 等^[45]认为现有桩基设计未关注土体侧向变形导致的剪力和弯矩, 提出基于桩基弯曲破坏理论并考虑土体侧向变形的简化设计方法。Abdoun 等^[46]通过现场案例、理论分析和模型试验得到相同结论, 即土体液化侧扩流对桩基弯曲破坏具有不可忽视的作用。唐亮等^[47]通过振动台试验探究液化场地桩基动力特性和破坏机制, 认为地震动幅值显著影响桩基弯矩, 强震会导致桩基弯曲破坏。袁晓铭等^[48]认为非液化土层中, 惯性荷载是影响桩基侧向响应的主控因素, 此时属于“桩推土”; 而液化土层中土体位移起主要作用, 属于“土推桩”。Haskell 等^[49]指出群桩体系由于承台的约束作用, 导致桩基呈现不同的变形模式, 较小的桩间距呈现单曲率模式, 而较大的桩间距呈现双曲率模式。Liu 等^[50]开展不同桩型部分预埋桩基的拟静力试验, 桩基呈现出典型弯曲失效模式, 并指出桩帽旋转效应会加大群桩桩基的轴向荷载, 桩基数目增加会加剧群桩效应。梁发云等^[51]分别实

施低承台和高承台群桩离心机振动台模型试验, 高承台上部结构和承台的惯性效应更明显。低承台和高承台存在明显的桩基内力分布差异, 高承台桩基在承台与桩接触位置弯矩较小。

Ullah 等^[52]通过频域分析, 验证桩-土-结构体系动力响应是由运动相互作用和惯性相互作用共同影响的。进一步地, 刘春辉^[53]针对惯性荷载和运动荷载的相位关系开展模型及理论研究, 指出惯性荷载与运动荷载并非简单的数值叠加, 二者相位关系受到桩基刚度、上部结构质量、桩长和土层条件的显著影响, 最后给出基于位移的液化侧扩流场地桩基等效静力设计方法。Tran 等^[54]发现单桩和群桩位移响应在相位上存在明显差异, 单桩位移和土体位移呈现相反的趋势, 而 3×3 群桩位移则与土体位移基本吻合, 差异性由单桩和群桩体系刚度不同导致。Garala 等^[55]指出: ①地震动强度较低时, 由运动荷载引起的弯矩和由惯性荷载引起的弯矩同等重要; ②地震动强度较高时, 惯性荷载引起的弯矩占主导地位; ③现有方法中^[56-59]确定两层土界面处运动作用产生的弯矩, 低估了桩基弯矩响应。Jia 等^[60]通过大型振动台试验研究液化侧向扩展作用下群桩-桥梁体系破坏机制, 发现桩固定在持力层时, 桩顶和桩端易发生弯曲破坏。Saha 等^[61]分析侧扩流场地不同类型桥梁桩基弯矩需求, 建议设计中应综合考虑惯性荷载和流动土压力, 或者采取桩前板桩墙的加固方法, 防止桩基因土体侧向流动而发生弯曲破坏。以弯曲破坏机制为主要理论基础, 中国、日本、美国所推荐的液化场地桥梁桩基抗震设计规范中, 特别考虑土体液化对桩基工作状态的影响效应^[62-67]。应该说, 这些规范总体上代表了当前世界范围内对桥梁震害成因机制的最新认识与抗震设计的发展趋势。

水平场地桩基侧向荷载主要来源于上部结构惯性荷载, 液化侧扩流场地桩基侧向荷载还包括土体侧扩流产生的土压力 (存在上覆黏土层时, 土体侧向流动荷载更显著), 二者皆是造成桩基弯曲失效的原因, 见图 10。弯曲破坏机制尽管从最初单一惯性荷载的认识发展到运动荷载和惯性荷载耦合作用的认识, 逐渐揭示液化场地桩基弯曲破坏的机制。然而, 由于桩基侧向荷载的复杂性和耦合性, 难以准确解耦桩-土运动相互作用和惯性相互作用。

3.2 沉降破坏

软土和液化场地广泛使用端承桩控制建筑物沉降。实际上, 土体液化会改变桩侧摩阻力和桩端承载力, 引起基础和上部结构沉降。液化场地桩基沉降

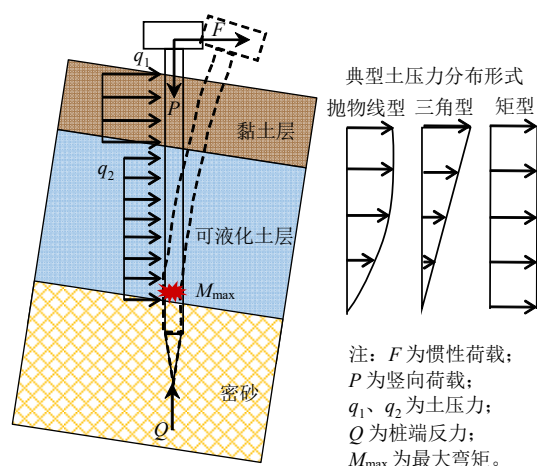


图 10 弯曲破坏示意图

Fig.10 Schematic diagram showing the pile bending failure

是当前岩土工程研究领域的一个重要课题。与桩基相比, 液化场地浅基础沉降破坏更为普遍且严重^[68], 本文仅讨论液化场地桩基沉降机制。

2010 年埃尔梅奥-库卡帕地震中某铁路桥桥墩沉降 0.5 m, Turner 等^[69]认为桥墩沉降是土层液化导致桩基竖向承载力不足引起的。De Alba 等^[70]指出饱和砂土中桩基沉降极易受到超孔隙水压力的影响, 随着超孔隙水压力不断累积, 桩基沉降增加, 并提出基于超孔隙的桩基沉降预测方法。Ter-Martirosyan 等^[71]对比分析土体液化对桩基沉降的破坏性影响效应, 研究表明, 土体液化效应显著影响基础沉降。Lew 等^[29]发现 Tokimatsu 法和 Ishihara 法未考虑细粒含量的影响, 导致桩基沉降预测可靠性较低, 并指出土层中细粒含量超过 35%, 应修正现有沉降计算方法。Knappett 等^[72]开展系列离心机振动台试验, 研究结果和 De Alba 等^[70]的研究结论基本一致。此外, 土体液化后再固结过程对桩基施加负摩擦力, 使得桩基更易沉降。Knappett 等^[72]采用球形孔洞膨胀问题预测液化土的地基承载力, 提出液化场地桩基沉降静力安全系数解析公式。王睿等^[73]建立地基固结和再固结过程中桩基轴力与沉降计算方法。Fellenius 等^[74]将桩侧摩擦力和桩-土相对位移相协调, 分析桩基沉降过程, 认为造成桩基沉降的主要原因是桩端土层液化而不是负摩擦力。该方法从桩-土相互作用力和位移协调性出发, 全面反映桩基受力机制, 是研究液化场地桩基沉降的有效方法。Hirai^[75]从弹性连续介质理论出发, 提出非均质土中矩形桩在竖向荷载作用下的沉降计算公式。

实际上, 群桩和单桩沉降有很大不同^[76-81], Knappett 等^[80]指出群桩在惯性荷载和运动荷载作用下前后晃动, 群桩两侧的桩受到较大的轴向循环荷载作用, 而且桩侧摩擦力(孔压增长阶段)大幅下降, 致使群桩基础不断产生沉降。而地震后, 由于土体固结对桩基施加负摩擦力导致桩基需要更大的桩端承载力。Stringer 等^[82-83]研究桩侧摩阻力传递规律, 发现砂土越松散桩基越易沉降, 孔压增长会降低摩擦力, 而孔压消散阶段会增加桩基竖向荷载, 二者都会造成桩基沉降。对于群桩而言, 桩间距是影响桩基承载力和沉降的重要因素。Patil 等^[84]指出总沉降和极限沉降随桩数、砂土内摩擦角和相对密实度的增大而减小, 地基沉降速率随土层相对密实度的增大而减小。Xu 等^[85]采用全耦合有限元方法, 研究地震动特性对饱和砂土中群桩沉降的影响。建议在工程设计中, 应注重考虑竖向地震作用下液化场地群桩沉降, 并建立基于荷载传递法的沉降估算方法。

事实上, 桩-土-结构体系刚度因土体液化而降低, 体系惯性荷载也相应变化。土体未液化时, 上部结构惯性力较大, 基桩承受较大的轴向动力荷载。反之, 土体液化后体系惯性荷载较小, 基桩承受较小的轴向动力荷载。而群桩效应和承台旋转效应使群桩两侧基桩的轴向循环荷载更为复杂, 使得群桩体系沉降计算更为棘手。

对于桩基沉降, 目前研究主要关注超孔隙压发展与桩侧摩擦力的关系、液化砂土再固结过程中的负摩擦力。显然, 液化场地桩基沉降有以下原因: ①液化土体丧失有效应力, 桩侧竖向摩阻力也逐渐丧失, 在附加动力轴向荷载循环作用下, 桩基产生沉降; ②地震结束后, 超孔隙水压力逐渐消散, 液化土体处于再固结阶段, 桩侧出现负摩阻力, 导致桩基产生沉降, 见图 11。附加动力轴向荷载与地震动特性密切相关, 如何将单桩沉降计算方法合理拓展到群桩基础将会是一件富有挑战性的工作。因此, 建议借助精细化有限元方法进一步探究液化场地群桩沉降。

3.3 屈曲失效

除侧向荷载外, 桩基还承受上部结构自重等轴向荷载(比如初始静荷载、附加动荷载)^[86-87]。为此, Fleming 等^[88]认为建立在软弱土层的桩基或桩基露出土层过长时, 需要考虑桩基稳定性问题, 并给出动力和静力设计建议。Dash 等^[89]认为弯曲-屈曲相互作用导致的桩基破坏是一种被忽视的破坏模式。Bhattacharya^[90]认为现存桩基稳定性需要被重新

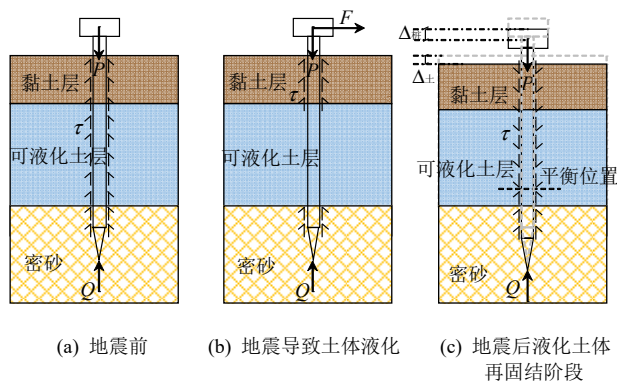


图 11 单桩沉降原理图

Fig.11 Schematic diagram showing the single pile settlement

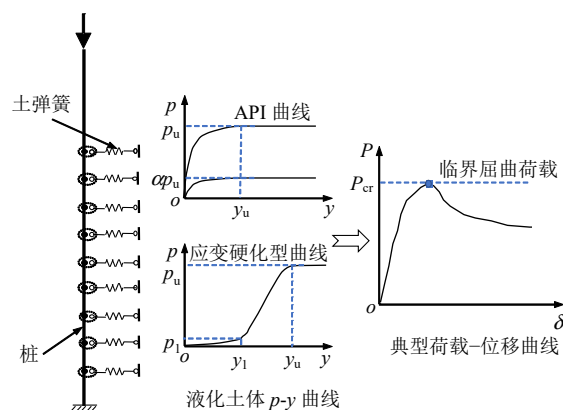
检验, 应综合考虑上部结构动力效应、 $P-\Delta$ 效应、桩基有效长度等因素, 并提出桩承结构屈曲失稳安全评价简化方法。之后, Bhattacharya 等^[25]归纳总结 15 个桩基震害案例, 提出基于长细比的液化场地桩基失效理论, 并通过离心机试验验证其失效理论的可靠性。同时, 指出日本最新规范仍存在设计缺陷——对桩基稳定性理论考虑不足^[91]。因此, 应在规范中重新考虑基于屈曲失效理论假定。Shanker 等^[92]通过理论解析求解考虑侧向土压力衰减的嵌沿桩临界屈曲荷载。研究表明, 液化土厚度、土体强度、长细比、桩基刚度和端部条件对桩基屈曲行为有显著影响。Bhattacharya 等^[93]针对轴向承载力 (屈曲失稳) 随液化发展逐渐降低的现象, 提出一种基于等效单自由度模型的桩-土体系频率计算简化公式。Knappett 等^[94]开展 6 组群桩屈曲失稳离心机试验, 认为场地液化后群桩存在屈曲失稳的可能性, 即使液化土层未达到完全液化状态, 也明显降低了桩基竖向承载力, 并指出, 静力分析方法并不能很好地刻画地震作用对群桩的影响效应。Gabr 等^[95]采用幂函数分布形式的土反力系数, 建立长细摩擦桩临界屈曲承载力计算模型, 分析侧向土反力分布形式对屈曲荷载的影响。Zhao 等^[96]基于桩侧摩擦力均匀分布和水平系数随深度线性增加的假设, 利用势能守恒理论推导失稳临界载荷和稳定计算长度。建议通过改善土体特性提高桩基稳定性, 桩柱刚度比较大、桩埋入比较小时, 土体特性对稳定性影响有限。同样类似的研究还有, Heelis 等^[97]提出面向任意端部条件、土体刚度和桩侧摩擦力的桩基屈曲荷载计算方法。Davisson^[98-99]、Mousavi^[100]、Lee^[101]、Liang^[102]等的研究多局限于线性理论中, 没有考虑桩基几何缺陷影响, 对非线性屈曲的研究较少。实际上, 桩基侧向位移显著降低桩基临界屈

曲荷载。因此, 传统特征值屈曲荷载计算方法不能准确反映桩基竖向承载力。Timoshenko 等^[103]给出考虑初始位移的弹性柱稳定性分析理论:

$$\delta = \delta_0 \frac{1}{1 - \frac{P}{P_{cr}}} \quad (1)$$

式中: δ_0 为桩顶的初始位移; δ 为考虑轴向荷载效应的桩顶侧向位移; P 为轴向荷载; P_{cr} 为欧拉临界屈曲荷载。当 $P/P_{cr} = 0.33$ 时, 考虑轴向荷载效应的桩顶侧向位移接近初始侧向位移的 1.5 倍, 此时产生的附加弯矩导致桩基发生塑性破坏。

随着计算机的发展, 考虑材料和几何非线性的数值计算方法应运而生。张效禹等^[104-107]基于非线性 Winkler 地基梁模型, 建立考虑土体液化效应、桩基特性和地震动特性的液化场地桩基屈曲临界荷载数值计算方法 (见图 12), 并建立考虑惯性荷载的桩基屈曲失效判别公式。随后, Tang 等^[108]在其研究基础上, 建立砂土液化过程中的桩基临界屈曲荷载判别公式。非线性 Winkler 地基梁模型中液化土体 $p-y$ 曲线是准确计算液化场地桩基临界屈曲荷载的关键。Tokimatsu^[109]、Dash^[110]、凌贤长^[111]、冯忠居^[112]等针对液化土体的 $p-y$ 曲线开展了大量研究, 此处就不再赘述。

图 12 临界屈曲荷载数值计算方法^[108]Fig.12 Numerical method of critical buckling load^[108]

Kheradi 等^[113]通过 1g 振动台试验和静力模型试验验证群桩基础屈曲的可能性和局部地基加固对群桩基础防屈曲失稳的有效性。Huded 等^[114]在上述研究基础上, 建立含非液化夹层的桩基非线性屈曲承载力计算方法, 并给出考虑非液化夹层的桩基屈曲设计图表。Ouyang 等^[115]将土体侧向流动的滑动力引入单元公式中, 发展倾斜场地桩基非线性屈曲

分析的欧拉-伯努利桩单元。针对屈曲机制, Haldar 等^[116]采用二维平面应变有限差分法, 研究砂土相对密实度、桩径、地震卓越频率和加速度峰值对液化深度、侧向位移和桩基弯矩的影响。Saeedi 等^[117]利用二维平面应变问题, 建立桩-土相互作用数值分析模型, 研究桩的截面属性、土体相对密实度和地震动峰值等参数对桩-土相互作用的影响, 研究表明, 液化深度随地震频率的增加而减少、弯矩随土体密度的增大而增大, 上部结构抗弯刚度大于桩基抗弯刚度时, 最大弯矩位置随抗弯刚度的增大而增大。Haldar 等^[118]针对前述学者们提出的弯曲破坏、屈曲失稳和二者相互作用引起的破坏, 研究土体参数、桩基参数和地震动参数对桩基破坏类型的影响, 认为液化土层厚度、桩基材料、桩径、土体相对密实度、地震动幅值和频率对桩基破坏模式起着重要作用。

目前, 屈曲失稳分析方法主要包括静力分析法 (见图 13)、能量分析法和有限元法等。静力分析法首先假设结构处于失稳变形状态, 分析部分结构受力状态, 列出力的平衡方程, 最后根据边界条件求解结构可能的位移和屈曲荷载^[119-120]。能量分析法基于虚位移原理, 对于特定的目标对象 (约束条件不变) 处于平衡状态的充分必要条件是: 所有的力在任意虚位移下所做的虚功之和为 0。桩-土体系的能量表达式为

$$\Pi = U_p + U_s + V_p + W_f \quad (2)$$

式中: Π 为桩-土体系总能量; U_p 为桩的弯曲应变能; U_s 为土体变形能; V_p 为桩顶荷载产生的能量;

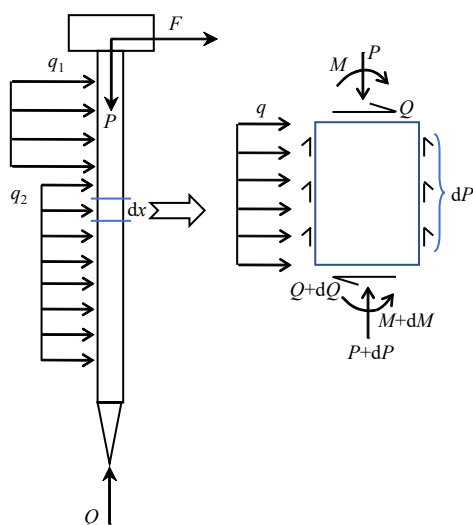


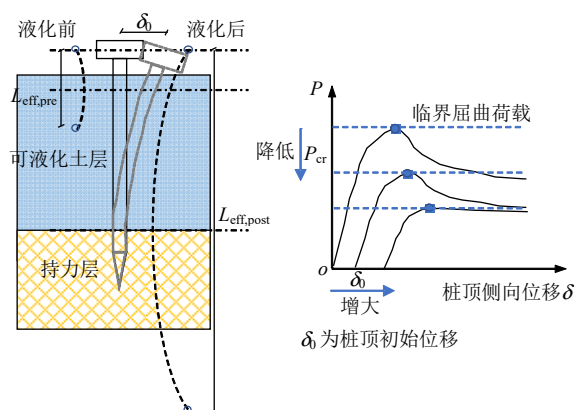
图 13 平衡微分方程示意图^[120]

Fig.13 Schematic diagram showing the micro-segment internal force analysis^[120]

W_f 为摩擦力产生的能量。此原理中不包含任何材料本构, 所以适用于任何体系^[121-122]。有限元法概念简单易懂, 并且随着计算机技术的发展, 有限元法的优势也越来越明显^[111-116]。

液化场地桩基屈曲失稳原因概括为以下两点:

①地震下土体液化导致桩基水平约束丧失, 桩基有效长度增加、临界屈曲荷载降低, 桩基发生屈曲失稳; ②桩基在侧向荷载作用下 (惯性荷载和运动荷载) 产生侧向位移, 侧向位移降低了临界屈曲荷载, 导致桩基屈曲失稳, 见图 14。



注: $L_{\text{eff,pre}}$ 为液化前有效长度;
 $L_{\text{eff,post}}$ 为液化后有效长度。

(a) 有效长度计算

(b) 荷载-位移曲线

图 14 屈曲失效分析

Fig.14 Schematic diagram showing the pile buckling

值得一提的是, 屈曲失稳最终的表现形式是弯曲破坏, 因此仅从现场震害调查难以准确推断出屈曲失稳。尚未有以屈曲破坏机制为理论基础的桩基抗震设计方法, 现有规范中多是以弯曲破坏理论进行设计, 并基于传统欧拉公式验算液化场地桩基屈曲荷载。事实上, 传统欧拉公式是基于理想长细杆推导得到, 并未充分考虑桩基几何和材料非线性。加之, 地震作用下液化场地桩-土相互作用强非线性行为, 致使传统欧拉公式难以准确反映桩基竖向承载力, 这无疑为桩基抗震设计埋下了隐患。

3.4 动力失效

除上述弯曲破坏、沉降破坏和屈曲失稳外, 液化场地桩基破坏形式还有剪切破坏、液化导致周期变化进而引发上部结构破坏 (动力失效) 和以上破坏模式的耦合破坏^[123-132]。与非液化场地不同的是, 对于均质单一液化土层, 砂土液化后桩-土-结构体系刚度变小, 周期延长, 刚度突变不明显。因此, 液化场地中桩顶处剪切破坏更可能出现在地震动初期^[8, 20], 此外, 桩基可能因土体侧扩流在土层交界

处出现剪切破坏。由土体液化效应导致桩基剪切破坏的实例和研究较少,不再详述。本节主要总结液化场地桩基动力失效研究。

自 20 世纪以来,人们认识到土-结构相互作用对于建筑物抗震分析和设计的重要性,特别是土-结构相互作用对结构周期影响显著^[133-135]。地震作用下,液化土体侧向约束能力变弱,桩-土-结构体系侧向刚度变化显著,整体呈现出强非线性行为。此时,桩基易因桩-土-结构体系基本周期延长而动力失效。对此,不少学者通过试验探究土体液化对桩-土-结构体系基本周期的影响,进而研究桩基在整个地震过程(包括孔压上升和消散)的动力响应。

Maheshwari 等^[136]以液化场地 2×2 群桩为研究对象,强调桩承结构分析和设计中考虑非线性和土体液化效应的重要性。液化场地桩承桥梁频繁出现桥梁中间跨坍塌,而靠近岸边的桥面保持相对稳定。Mohanty 等^[137-140]认为土体液化导致桩-土-结构体系基本周期增大,桩顶侧向位移需求增大。分析昭和大桥(1964 年新潟地震)和盘山大桥(1975 年海城地震)震害情况,发现桥梁中间桥墩的位移需求增加了 100%以上,而非中间跨的桥墩位移需求仅增加了 50%。之后,开展桩-土-桥梁体系振动台试验验证桥梁桩基基本频率随砂土液化而降低,桩基位移和弯矩需求也相应改变。室内振动台试验和野外振动试验研究^[141-143]同样表明,土体液化导致桩-土-结构体系频率和阻尼发生变化,从而改变桩-土-结构体系动力响应:随着超孔隙水压力发展,桩-土-结构体系阻尼比显著增加,在谱加速度减小和基本周期延长的共同作用下,惯性力显著衰减。小波变换能够有效地、直观地反映出局部信号特征,有利于分析结构瞬态振动特性。Hall 等^[144]采用小波变换方法,发现土体液化速度越快,体系频率衰减越明显;同时液化导致地震波基本周期延长和总频率带宽缩小。在结构体系频率降低和地震动高频分量滤波的共同作用下,发生共振,进而放大结构动力响应。Nasr 等^[145]针对非均质单一可液化倾斜场地大直径钢筋混凝土桩开展非线性动力分析,发现液化和非液化时桩基谱加速度明显不同,液化改变了结构原有的固有周期。

与静力失效模式相比,动力失效更加关注动荷载作用下结构自振频率、阻尼、振型等动力特性。由于液化土强非线性行为,桩-土-结构体系动力特性因土体液化而剧烈改变,比如,结构体系周期延长、阻尼系数增大(见图 15)。动力特性的改变致使桩基面临共振风险,一旦桩基发生共振,桩-土-结

构体系位移增大,桩基出现动力失效。

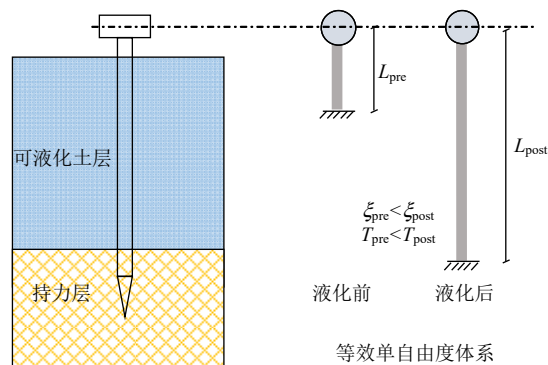


图 15 桩-土-结构体系周期变化图
Fig.15 Schematic diagram showing the period of pile-soil-structure system

针对液化场地桩-土-结构动力失效问题,求解考虑桩周土体液化状态(不同场地液化程度)的桩-土-结构体系动力特性更为困难。尽管大量学者基于现场试验和理论解析等方法分析了桩-土-结构体系周期变化规律及影响因素^[137-152]。但是,液化土强非线性行为致使现行分析方法难以准确计算土体液化过程中体系的动力特性。

3.5 耦合失效

可见,针对上述桩的几种常见破坏类型,国内外学者从物理模型试验、理论解析和数值模拟等方面开展了较多的细致工作,并在一定程度上揭示了液化场地桩基地震破坏机制。然而,Yoshida 等^[153]重新分析 Bhattacharya 等^[3, 25]调查的 15 个桩基震害事例,认为多数情况下,桩的轴向荷载达到屈曲荷载的 50%或更大时,桩基便发生严重破坏。随后,Dash 等^[89]研究表明,桩的抗震设计中,考虑单一失效机制可能不安全,同时考虑两种破坏机制耦合作用可能更保守。Tang 等^[154]研究发现,强震中侧向荷载作用下桩产生较大弯矩,随着轴向荷载增大,桩的弯矩和侧向变形势必增大,可能造成桩过早出现破坏。同样,桩周土液化变“软”时,较大轴向荷载下桩趋于产生较大的侧向挠度,在侧向扰动荷载作用下,可能很快达到失效状态^[30, 155]。桩上侧向荷载和轴向荷载相互作用称为弯曲-屈曲相互作用,对应桩的失效模式为弯曲-屈曲耦合失效机制^[89]。可见,液化场地桩的屈曲失效因侧向荷载的作用而导致临界荷载变小,桩的弯曲失效因轴向荷载效应而变得更易发生^[89, 94, 156]。液化场地桩基震害实质并不是单一的弯曲失效或屈曲失效模式,更多情况是,侧向荷载和轴向荷载共同作用下桩发生弯曲-

屈曲耦合失效^[89, 94]。虽然沉降破坏与屈曲失效不易同时发生, 然而, Franke 等^[4]指出桩基不均匀沉降容易造成承台转动, 进而使得桩顶弯矩过大, 形成塑性铰。同时塑性铰的产生将加剧承台扭转, 产生附加弯矩和轴向荷载, 致使桩基不均匀沉降进一步加剧, 直至桩基失效或者结构坍塌。Liu 等^[50]通过振动台试验发现群桩效应和桩帽旋转效应增大桩-土-结构体系的位移和弯矩, 进而影响桩基破坏。可见, 液化场地桩的弯曲失效因不均匀沉降而弯矩增大, 桩的沉降破坏因塑性铰的产生而变得更容易发生, 见图 16。显然, 液化场地中桩基失效不仅仅是由某单一机制造成的, 多种失效机制的耦合是造成桩基破坏的重要原因。然而, 世界范围内尚没有哪部规范系统考虑桩的耦合失效作用机制。近年来, 国内外学者已逐步认识到解决这一问题的严重性与紧迫性^[91, 157-158]。此外, 由于土体液化本质是超孔压发展的过程, 不同位置处地震作用产生的超孔压比不同, 导致场地水平和竖向液化范围受限。因此学者也逐渐意识到, 开展不同液化程度下桩-土-结构动力响应和失效模式研究的重要性和迫切性^[116, 159-160]。

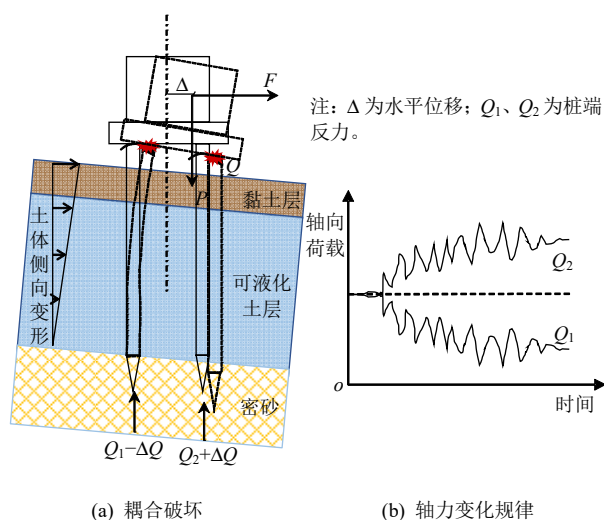


图 16 沉降和塑性铰耦合破坏示意图

Fig.16 Schematic diagram showing the coupled failure of settlement and plastic hinge

4 结论和建议

尽管各国抗震规范在历次地震中不断的发展完善, 但是液化场地桩基震害依然屡见不鲜, 本文从历次桩基震害案例出发, 归纳目前国内外对液化场地桩基地震失效机制的研究成果。关于震害事例和几种破坏模式机制的研究现状与挑战总结如下:

(1) 通过整理震害案例, 液化场地桩基破坏模

式可归为弯曲破坏、屈曲失效、沉降破坏和动力失效等, 还包括两种及以上破坏类型的耦合破坏模式。由于震害调查机会难得、案例稀少、施工费用高等缺点的限制, 野外足尺模型试验或许是一种合适的选择, 有利于弥补现有震害资料的不足。

(2) 弯曲破坏主要是地震作用下惯性荷载和运动荷载作用导致, 对于液化侧扩流场地, 地震后土体侧向流动是导致桩基弯曲破坏的重要原因。其中, 运动荷载和惯性荷载的物理耦合和叠加方式是当前研究的难点和重点。

(3) 液化场地土体超孔压增长导致桩侧摩擦力降低和超孔压消散阶段(再固结)的负摩擦力是研究桩基沉降的重点, 也是影响桩基沉降的两个主要原因。加之, 群桩效应和承台旋转效应的影响, 液化场地群桩基础沉降研究更为复杂, 应进一步研究群桩与单桩沉降差异性。同时, 工程中应尽可能防止基础产生不均匀沉降。

(4) 桩基几何和材料非线性、桩-液化土间强非线性行为是液化场地桩基临界屈曲荷载计算中的重大挑战。此外, 液化场地群桩屈曲失效机制研究更是匮乏。发展简洁且有效的液化场地桩基屈曲荷载计算方法和抗震设计方法是一项重要挑战。

(5) 由于土体液化导致桩-土-结构体系动力特性改变, 尤其是基本周期延长致使桩-土-结构体系动力失效的研究正处于初始阶段, 桩-土-结构体系的强非线性行为是该方向的研究难点。发展考虑桩周土体液化程度的动力特性确定方法是未来研究的一项重大挑战。

(6) 液化场地桩基破坏不仅由单一失效机制控制, 多种失效机制的耦合作用是造成桩基破坏的重要原因。因此, 开展桩基耦合失效作用机制迫在眉睫。

(7) 土体液化的本质是超孔隙水压力累积, 建立能够表征孔压时空特征和桩基响应规律的桩周土体液化状态评价指标, 准确量化桩周土体液化对桩基破坏性影响效应是未来亟需努力的一个方向。

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