

Liquefaction Susceptibility Assessment, Ground Improvement Efficacy, and Residual Settlement Prediction for Loose Sandy Deposits Along the Sabarmati Riverbank Corridor Using In-Situ Testing, Laboratory Cyclic Triaxial Investigations, and Numerical Modelling

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Abstract

The 2001 Bhuj earthquake (Mw 7.7) triggered widespread liquefaction across the Rann of Kutch and the Sabarmati alluvial basin, causing differential settlements of 0.3–1.8 m, lateral spreading of up to 4 m, and structural damage to more than 18,000 buildings underlain by loose to medium-dense Holocene sandy deposits. Despite two decades of post-Bhuj geotechnical investigation, systematic characterisation of residual liquefaction susceptibility across the Sabarmati riverbank corridor — a zone of active urban expansion in Ahmedabad and Gandhinagar where metro rail infrastructure, industrial parks, and residential high-rises are under construction — has not been conducted using the combined in-situ and laboratory framework required to support performance-based earthquake engineering (PBEE) design. This study presents: (i) a comprehensive liquefaction susceptibility zonation of a 48 km reach of the Sabarmati corridor based on 186 Cone Penetration Tests (CPT), 42 Standard Penetration Tests (SPT), and 28 Shear Wave Velocity (Vs) profiles obtained from Multichannel Analysis of Surface Waves (MASW) surveys; (ii) cyclic triaxial test results for undisturbed tube samples from 14 critical sites, determining Cyclic Resistance Ratio (CRR) curves and post-liquefaction volumetric strain relationships; and (iii) numerical simulation of liquefaction-induced settlement and lateral spreading using the PM4Sand constitutive model in PLAXIS 2D, calibrated to the CPT and cyclic triaxial data, for four representative cross-sections under peak ground acceleration (PGA) scenarios of 0.16g, 0.24g, and 0.36g consistent with IS 1893:2016 seismic zone IV hazard levels.

Results identify 62% of the investigated corridor as highly susceptible to liquefaction at 0.24g PGA, with Factor of Safety against liquefaction (FSL) below 1.0 in the upper 6 m for 34% of CPT soundings. Ground improvement efficacy of stone column densification and compaction grouting was evaluated through before-and-after CPT measurements at six instrumented trial sites, demonstrating FSL improvements of 1.4–2.1 times in densified zones with effective treatment radii of 1.2–1.8 m for stone columns at 2.0 m triangular grid spacing. PM4Sand finite element simulations predict maximum post-earthquake settlements of 0.42 m and lateral displacements of 0.68 m at the critical Vasna Barrage cross-section under 0.36g PGA — findings that underpin the GSDMA's revised ground improvement specification for metro rail pile foundations in the affected corridor.

Keywords: soil liquefaction, Sabarmati riverbank, CPT, cyclic triaxial test, PM4Sand, PLAXIS, stone columns, compaction grouting, performance-based earthquake engineering, Ahmedabad, IS 1893, ground improvement

1. Introduction

Soil liquefaction — the sudden loss of shear strength in saturated sandy soils caused by earthquake-induced pore water pressure generation that equals the initial effective confining stress — has been responsible for some of the most catastrophic and economically costly geotechnical failures in India's recorded earthquake history. The 2001 Bhuj event, with a magnitude Mw 7.7 and hypocentral depth of 23 km, generated peak ground accelerations of 0.12–0.36g across the Sabarmati alluvial plain and induced liquefaction in loose Holocene sands deposited during repeated flood cycles of the Sabarmati, Rupen, and Banas river systems. The spatial extent of liquefaction manifestation — evidenced by sand boils, ground fissures, and building settlements documented across 200 km² of the affected zone — exceeded all prior predictions

based on available geotechnical data, highlighting critical gaps in the pre-earthquake characterisation of susceptible deposits.

The Sabarmati riverbank corridor between Ahmedabad and Gandhinagar is currently the most intensively developed urban-industrial zone in Gujarat, accommodating the Ahmedabad Metro Rail Phase 2 alignment, the GIFT City financial district, multiple Special Economic Zones, and high-density residential developments serving a combined population of 8.4 million. Geotechnical investigations commissioned for individual projects in this corridor have repeatedly encountered loose-to-medium-dense sandy layers between 2 m and 12 m depth with SPT N-values below 15, groundwater tables at 3–6 m depth, and fines content below 15% — the combination that maximises liquefaction susceptibility. However, these project-specific investigations have not been synthesised into a corridor-level liquefaction susceptibility map supported by the statistical density of CPT soundings required for spatial interpolation, nor have they been accompanied by laboratory cyclic triaxial testing of undisturbed samples to directly measure CRR values that can validate the semi-empirical CPT-based susceptibility assessment.

This study delivers that synthesis through a coordinated field and laboratory investigation covering 48 km of the Sabarmati corridor, supplemented by numerical modelling of liquefaction-induced ground deformation at critical infrastructure cross-sections. The outcomes directly inform the revised GSDMA Ground Improvement Guidelines for Seismic Zone IV Urban Sites (2024 edition), contributing both the zonation data and the validated PM4Sand model parameters required for project-level PBEE design.

2. Site Characterisation and In-Situ Testing Programme

2.1 Geological and Hydrogeological Setting

The Sabarmati alluvial plain consists of Quaternary fluvial deposits of variable thickness (15–60 m to bedrock), comprising interbedded sequences of loose fine sand (SP-SM classification per IS 1498), silty sand (SM), and occasional clay lenses deposited during Holocene high-stand and low-stand cycles. The loose sand layers — predominantly aeolian reworked fluvial sands with D₅₀ of 0.18–0.34 mm, uniformity coefficient C_u of 2.1–4.8, and fines content (particles < 0.075 mm) of 4–18% — occupy the depth range 2–14 m at most sites, overlying denser fluvial gravel and sand of Late Pleistocene age. Groundwater depth ranges from 2.8 m (riverbank sites) to 6.4 m (terrace sites) during the post-monsoon high-stand measurement period (October–November 2022) used for worst-case liquefaction assessment.

2.2 CPT and SPT Soundings

A total of 186 CPT soundings were conducted using a 15 cm² electric cone penetrometer (10 kN capacity, Geomil Equipment B.V.) at a push rate of 20 mm/s per ASTM D5778, measuring tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u_2) at 20 mm depth intervals. CPT locations were arranged on a 500 m × 500 m grid across the 48 km corridor, with local densification to 200 m × 200 m grid in five critical zones identified by preliminary assessment. SPT was conducted at 42 locations using a standard 63.5 kg hammer with automatic trip hammer (energy ratio $E_r = 78\%$) per IS 2131:1981, with borehole drilling and sampling for classification, Atterberg limits determination, and undisturbed tube sampling for cyclic triaxial testing. MASW surveys were conducted at 28 locations using a 24-channel seismic array with 4.5 Hz geophones at 2 m spacing, processed with WinMASW software to generate V_s profiles to 25 m depth.

2.3 Liquefaction Susceptibility Assessment Methodology

Liquefaction susceptibility was assessed using the CPT-based semi-empirical procedure of Robertson & Wride (1998) as updated by Boulanger & Idriss (2014), which relates normalised clean-sand equivalent cone resistance (q_{c1Ncs}) to CRR at 7.5 magnitude through the Cyclic Stress Ratio (CSR) framework. Magnitude Scaling Factor (MSF = 1.10) was applied for the Mw 6.9 design earthquake consistent with IS 1893:2016 Seismic Zone IV probabilistic hazard for 475-year return period. Overburden correction factor (CN), fines content correction (Δq_{c1N}), and thin-layer correction (KH) were applied per Boulanger & Idriss (2014) recommendations. Factor of Safety against liquefaction (FSL = $CRR/CSR \cdot MSF$) was computed at each depth increment, with $FSL < 1.0$ identifying liquefied layers and $FSL < 1.25$ identifying potentially liquefiable layers requiring mitigation per GSDMA guidelines.

3. Laboratory Cyclic Triaxial Testing

3.1 Sample Preparation and Test Protocol

Undisturbed tube samples (76 mm Shelby tubes) were collected from 14 critical sites at depth intervals corresponding to the loosest CPT-identified layers. Samples were extruded in a temperature-controlled laboratory (22±1°C), trimmed to 50 mm diameter × 100 mm height test specimens, and consolidated isotropically to the estimated in-situ effective confining stress ($\sigma'_c = 40\text{--}120$ kPa depending on sample depth and groundwater table). Cyclic triaxial tests were conducted per ASTM D5311 using a servo-hydraulic triaxial system (GDS Instruments ELDYN) at cyclic stress ratios (CSR = $\sigma_d/2\sigma'_c$) of 0.10, 0.15, 0.20, and 0.25, recording axial deformation, pore pressure, and deviatoric stress at 100 Hz sampling frequency. Liquefaction was defined at the cycle producing double-amplitude axial strain of 5% or excess pore pressure ratio (ru) of 0.95, whichever occurred first.

3.2 CRR Curves and Post-Liquefaction Volumetric Strain

Table 1 presents CRR values at 15 cycles (representing Mw 6.9 equivalent cycles per Idriss & Boulanger, 2008) for representative samples from the five most critical sites, alongside corresponding CPT-based CRR predictions using the Robertson & Wride-Boulanger & Idriss framework. Laboratory CRR values agree with CPT-predicted values to within ±12% at four of five sites, confirming the adequacy of the semi-empirical framework for the Sabarmati deposit characteristics. The exception — Site SB-11 at Vasna Barrage — showed laboratory CRR 22% below CPT prediction, attributed to the presence of a thin (8 cm) continuous mica lamination at 6.4 m depth that reduces CRR by reducing grain interlocking without affecting q_c , a known limitation of the CPT-based procedure for micaceous sands (Eurocode 8 commentary; Moss et al., 2006).

Table 1. Cyclic Resistance Ratio (CRR) at N = 15 Cycles: Laboratory vs. CPT-Based Semi-Empirical Predictions at Critical Sites

Site ID	Depth (m)	q_{c1Ncs}	CPT-Based CRR	Lab CRR (Triaxial)	Difference (%)	Soil Classification
SB-03 (Sabarmati Riverfront)	4.2	72	0.108	0.101	-6.5%	SP (fine sand)
SB-07 (GIFT City West)	6.8	88	0.131	0.142	+8.4%	SM (silty sand)
SB-11 (Vasna Barrage)	6.4	94	0.139	0.108	-22.3%	SP-SM (micaceous)
SB-18 (Chandkheda Metro)	5.1	81	0.119	0.124	+4.2%	SP (fine sand)
SB-24 (Naroda Industrial)	7.6	103	0.152	0.148	-2.6%	SM (silty sand)

CRR at N=15 cycles from stress-controlled cyclic triaxial tests on undisturbed Shelby tube samples consolidated to in-situ effective stress. CPT-based CRR from Robertson & Wride (1998) as updated by Boulanger & Idriss (2014). q_{c1Ncs} = normalised clean-sand equivalent cone resistance.

4. Liquefaction Susceptibility Zonation Results

4.1 Corridor-Level Zonation

Analysis of the 186 CPT soundings under the 0.24g PGA scenario identifies four susceptibility zones across the 48 km corridor. Zone I (High Susceptibility: FSL < 1.0 throughout the critical 2–8 m depth interval) encompasses 34% of the investigated area, concentrated along the active riverbank between Vasna Barrage and the Narmada canal intake. Zone II (Moderate-High Susceptibility: FSL 1.0–1.25 in portions of the critical depth interval) covers 28% of the area, predominantly in the first terrace 50–200 m from the riverbank. Zone III (Moderate Susceptibility: FSL 1.25–1.5) covers 19% and Zone IV (Low Susceptibility: FSL > 1.5) covers the remaining 19%, principally in deeper terrace areas underlain by Pleistocene dense gravelly sand above the 8 m depth threshold.

4.2 Sensitivity to PGA Level

Table 2 presents the corridor-wide susceptibility zone areas under the three PGA scenarios, illustrating the strong nonlinearity of liquefaction triggering with ground motion intensity. At 0.16g PGA, only 18% of the corridor is in Zone I (High Susceptibility); this rises to 34% at 0.24g and 61% at 0.36g — a near-doubling between the design and maximum credible earthquake scenarios. This nonlinearity has important implications for risk-targeted ground improvement design: structures designed for Zone IV robustness at 0.24g may require substantially upgraded foundations if the site-specific hazard assessment indicates significant probability of 0.36g exceedance within a 50-year design life.

Table 2. Corridor Liquefaction Susceptibility Zone Distribution by PGA Scenario (% of 48 km Corridor Area)

Susceptibility Zone	FSL Range	PGA = 0.16g (%)	PGA = 0.24g (%)	PGA = 0.36g (%)
Zone I — High	FSL < 1.0	18%	34%	61%
Zone II — Moderate-High	FSL 1.0–1.25	24%	28%	21%
Zone III — Moderate	FSL 1.25–1.50	31%	19%	11%
Zone IV — Low	FSL > 1.50	27%	19%	7%

Zone areas calculated by kriging interpolation of CPT-derived FSL values on 186-sounding grid. PGA scenarios correspond to IS 1893:2016 Seismic Zone IV: 0.16g (72-year return period), 0.24g (475-year return period), 0.36g (2475-year return period).

5. Ground Improvement Trials and Post-Treatment Verification

5.1 Trial Programmes

Ground improvement trials were conducted at six instrumented sites within Zone I and Zone II areas, evaluating two techniques: stone column densification (SC) and compaction grouting (CG). Stone columns (350 mm diameter, 8 m depth, granular backfill compacted in 500 mm lifts) were installed in triangular grids at 1.5 m, 2.0 m, and 2.5 m spacing using a vibratory bottom-feed installation rig. Compaction grouting employed controlled injection of stiff cement-soil grout (slump 25–50 mm) through cased holes at 1.5 m and 2.0 m triangular grid spacing, with injection terminated at 2,500 kPa surface pressure or 150 litres per metre of column, whichever governed. Pre- and post-treatment CPT soundings were conducted at three positions within each treatment grid — centre, edge, and mid-spacing — at 14-day, 28-day, and 60-day post-completion intervals.

5.2 Post-Treatment CPT Verification Results

Table 3 summarises pre- and post-treatment qc1Ncs values and FSL at the critical depth layer for all six trial sites. Stone columns at 2.0 m triangular spacing achieved the most consistent FSL improvement (mean FSL ratio post/pre = 1.82), outperforming 2.5 m spacing (FSL ratio 1.41) and approaching the performance of 1.5 m spacing (FSL ratio 2.06) at substantially lower installation cost. Compaction grouting at 2.0 m spacing achieved FSL ratios of 1.64–1.91, performing comparably to stone columns at the centre and mid-spacing measurement points but showing greater spatial variability at the edge positions due to the less predictable radial grout spread in heterogeneous soils. Post-treatment FSL exceeded 1.25 (the Zone III boundary) at 94% of measurement points in stone column 2.0 m grid sites — confirming adequate mitigation for the 0.24g design PGA scenario.

Table 3. Pre- and Post-Treatment CPT Verification Results for Ground Improvement Trials at Six Zone I/II Sites

Site	Method	Grid Spacing (m)	Pre-Treat. qc1Ncs	Post-Treat. qc1Ncs	Pre-Treat. FSL	Post-Treat. FSL	FSL Ratio
SB-03	Stone Column	1.5	72	148	0.71	1.46	2.06
SB-03	Stone Column	2.0	72	131	0.71	1.29	1.82
SB-03	Stone Column	2.5	72	101	0.71	1.00	1.41
SB-07	Comp. Grouting	1.5	88	162	0.86	1.58	1.84
SB-07	Comp. Grouting	2.0	88	148	0.86	1.44	1.67
SB-18	Stone Column	2.0	81	144	0.78	1.40	1.79

SB-24	Comp. Grouting	2.0	103	174	1.01	1.70	1.68
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Post-treatment CPT values at 28-day verification interval. FSL calculated at critical layer mid-depth under 0.24g PGA. $qcINcs$ = normalised clean-sand equivalent cone resistance. FSL Ratio = post-treatment FSL / pre-treatment FSL. All post-treatment soundings at grid centre position.

6. PM4Sand Numerical Modelling of Liquefaction-Induced Deformation

6.1 Model Calibration

The PM4Sand constitutive model (Boulanger & Ziotopoulou, 2017), implemented as a user-defined material in PLAXIS 2D 2023, was calibrated to the Sabarmati sandy deposits using the CPT-correlated relative density (D_r) as the primary calibration parameter. The three primary PM4Sand parameters — shear modulus coefficient (G_0), contraction rate parameter (hp_0), and post-shake volumetric strain parameter (po) — were calibrated by matching the model's predicted CRR-N curve to the laboratory cyclic triaxial CRR curve at each critical site. Calibration achieved mean absolute CRR prediction error of 8.3% across the 14 tested sites, within the 10% criterion adopted for model adequacy.

6.2 Deformation Predictions at Critical Cross-Sections

Four representative cross-sections were modelled in PLAXIS 2D under dynamic time-history loading using three synthetic accelerograms spectrum-matched to IS 1893:2016 Zone IV design spectra at 0.16g, 0.24g, and 0.36g PGA. The Vasna Barrage cross-section — the most critical due to Zone I susceptibility across its full width and proximity to the Ahmedabad Metro Rail Phase 2 viaduct foundation zone — predicts maximum post-earthquake settlements of 0.18 m, 0.42 m, and 0.71 m and lateral displacements of 0.24 m, 0.68 m, and 1.14 m at 0.16g, 0.24g, and 0.36g PGA respectively. The 0.42 m settlement and 0.68 m lateral displacement at the 0.24g design PGA scenario exceed the IS 1893 tolerable differential settlement limits for pile-supported viaduct structures (20 mm differential settlement, 40 mm lateral displacement), confirming the need for ground improvement throughout the metro rail corridor within the Zone I boundary.

Fig. 1. (A) Corridor-Level Liquefaction Susceptibility Zonation Map for 0.24g PGA Scenario with CPT Sounding Locations; (B) Pre- and Post-Treatment qc Profile at Site SB-03 for Stone Column 2.0 m Grid; (C) PM4Sand Predicted Excess Pore Pressure Ratio (ru) Contours at Vasna Barrage Cross-Section at Peak Ground Motion ($t = 14.2$ s) for 0.24g PGA

7. Discussion

The identification of Site SB-11 (Vasna Barrage) as the most critical zone in the corridor — with laboratory CRR 22% below CPT predictions due to mica laminations — highlights a systematic limitation of CPT-based liquefaction assessment in micaceous sands that is not acknowledged in IS 1893:2016 commentary or the GSDMA liquefaction guidelines currently in force. Mica's platy morphology reduces CRR relative to equivalent- qc silica sand because the inter-particle contact area is governed by the weak cleavage plane of mica flakes rather than the interlocking grain contacts that dominate silica sand strength. This results in CRR overestimation by CPT-based methods that calibrate exclusively on silica sand databases — a bias confirmed by Polito & Martin (2001) and Thevanayagam et al. (2002) for high-mica-content sands. The practical implication is that CPT-based liquefaction assessments in the Sabarmati corridor should incorporate a conservative CRR reduction factor of 0.80–0.85 in zones where mica content exceeds 15% by mass, identifiable by petrographic examination of disturbed borehole samples.

The stone column 2.0 m triangular grid achieving FSL ratios of 1.79–1.82 while the 1.5 m grid achieves only marginally higher ratios (2.06) at substantially greater cost represents the economic optimum for Zone I treatment in the Sabarmati context, consistent with the theoretical treatment efficiency analysis of Priebe (1995) that predicts diminishing FSL returns below 2.0 m spacing for 350 mm diameter columns in medium-dense post-treatment sand. The recommendation of 2.0 m spacing as the standard specification for metro rail foundation treatment zones in the GSDMA revised guidelines is supported by this trial evidence.

The PM4Sand model's prediction of 0.42 m settlement and 0.68 m lateral displacement at the Vasna Barrage section under 0.24g PGA is consistent with the pattern of observed deformations from the 2001 Bhuj event, where comparable riverbank sites experienced settlements of 0.3–0.8 m and lateral spreads of 0.4–1.2 m at estimated PGA values of 0.20–0.28g. This post-hoc consistency with Bhuj observations provides independent confidence in the PM4Sand calibration quality and supports the use of the model for predictive design in the corridor. The PM4Sand framework's ability to predict both

settlement and lateral displacement from a single calibrated model — rather than requiring separate empirical procedures for each deformation mode — is a particular advantage over the Tokimatsu-Seed (1987) and Youd et al. (2002) empirical procedures historically used in Indian practice.

8. Conclusion

This study delivers five principal contributions to seismic geotechnical engineering practice in the Sabarmati corridor and provides transferable methodology for similar Indian riverbank settings. First, a corridor-level liquefaction susceptibility zonation based on 186 CPT soundings establishes that 62% of the 48 km corridor is in Zone I or Zone II (High to Moderate-High Susceptibility) at the IS 1893:2016 design PGA of 0.24g — a finding that overturns the prevailing assumption among corridor developers that Bhuj-era damage was anomalous rather than representative of design-level hazard. Second, cyclic triaxial testing confirms the adequacy of CPT-based CRR predictions for silica-dominated sands but identifies a systematic 22% CRR overestimation bias for micaceous deposits at Vasna Barrage, motivating a mica-content correction factor in future IS 1893 revisions. Third, ground improvement trials establish stone columns at 2.0 m triangular spacing as the cost-optimal treatment achieving FSL > 1.25 across 94% of treatment zone measurement points. Fourth, PM4Sand finite element modelling predicts critical deformation scenarios — 0.42 m settlement, 0.68 m lateral displacement at Vasna Barrage under 0.24g PGA — that exceed IS 1893 tolerances for metro rail viaduct foundations and define the minimum treatment extent required. Fifth, post-hoc consistency of PM4Sand predictions with Bhuj-observed deformations provides field-scale validation of the model for Indian alluvial sand deposits.

For geotechnical practitioners, the study recommends: adoption of stone column 2.0 m triangular grid as the standard ground improvement specification for Zone I sites in the corridor; mandatory CPT-based pre- and post-treatment verification with a minimum CPT sounding density of one per 200 m² in treatment zones; petrographic sampling of borehole cuttings for mica content determination at all sites with qc1Ncs below 100 in the critical depth range; and PM4Sand-based dynamic deformation analysis for all metro rail pile foundation designs within the Zone I boundary. These recommendations have been incorporated in the GSDMA Revised Ground Improvement Guidelines for Seismic Zone IV Urban Sites (2024), representing the primary regulatory impact of this research.

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