Volume Compressibility and Pore Pressure Response of Kutch Soils with Varying Plastic and Non-plastic Fines

Majid Hussain and Ajanta Sachan

Abstract Effect of fines content (FC) and its nature (*plastic fines and non-plastic fines*) on the volume compressibility and pore pressure response of 32 soils from 10 locations of Kutch (high seismicity region) is studied. Volume compressibility and pore pressure response of soils are studied and analyzed in the context of variations in plastic and non-plastic fines content. Volume compressibility increased with an increase in fines content: Influence of plastic fines is more compared to non-plastic fines. Fines content and nature of fines controlled the magnitude of excess pore water pressure generated within the soil mass. Plastic fines inhibited pore water pressure generation to a greater degree than non-plastic fines. Soils with same fines content but higher plastic fine content exhibited larger volume compressibility and lower pore water pressure evolution. Skempton's pore pressure parameter (A) of these soils indicated similar response as that of liquefiable soils. FC and nature of fines affected the degree of brittleness, which has been evaluated by obtaining undrained brittleness index (I_{B2}) with respect to pore pressure. The correlation between volume compressibility and pore water pressure response exhibited negative value ($R = -0.14$) indicating the opposite effect of FC.

Keywords Compressibility · Pore pressure · Fines content · Plastic fines · Non-plastic fines

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1 Introduction

Compressibility and shear strength behavior of soils are largely dependent on density, stress state, and boundary conditions of the soil. Effect of fines on the compressibility of soil is a function of amount and nature of fines present (Rutledge 1947; Lade et al. 2009). Loose silty sands, when subjected to drained and undrained conditions, exhibited contractive response under both the boundary conditions (Pitman et al. 1994). Pore pressure evolution might lead to strength reduction resulting in various types of failures like strain softening (SS), limited strain softening (LSS), and strain hardening (SH) (Castro 1969). The large positive pore water pressure causes spontaneous triggering with large movement and high velocity also known as static liquefaction. Many researchers have explained the mechanism of static liquefaction. Terzaghi (1956), Sladen et al. (1985) and Yamamuro and Lade (1997) articulated collapse of the metastable soil structure during undrained monotonic loading as a possible cause of static liquefaction. The phenomenon has drastic implications when encountered in silty sands. Such type of soils offers minimal resistance to earthquake liquefaction (Ishihara 1993).

Loose silty sands are highly compressible, which creates large excess pore pressure (Δu) during undrained shearing leading to static liquefaction (Yamamuro and Lade 1998; Sladen et al. 1985). The immediate consequences of large volume compressibility (m_v) and large excess pore pressure (Δu) are large deformations and low load-carrying capacity. Factors controlling m_v and Δu such as relative density, fines content, effective confining pressure, stress history, and anisotropy were explored recently by many researchers (Lee and Farhoomand 1967; Lade and Yamamuro 1997; Yamamuro and Lade 1997; Thevanayagam 1998; Yamamuro et al. 2008; Monkul and Yamamuro 2011). Most of these studies were carried out on clean standard sands and silts. Effect of non-plastic fines content was studied by many researchers by controlled and systematic addition of fines to the clean standard host sands (Shen 1977; Kuerbis et al. 1988; Pitman et al. 1994; Vaid 1994; Lade and Yammuro 1997; Yamamuro and Lade 1997; Yamamuro and Lade 1998; Salgado et al. 2000; Papadopoulou and Tika 2008; Sitharam and Dash 2008). Effect of plastic fines was studied by few researchers indicating a significant decrease in undrained strength of sand with an increase in plastic fines (Georgiannou et al. 1990; Georgiannou et al. 1991 and Abedi and Yasrobi 2010). The coupled effect of plastic and non-plastic fines is yet to be explored. While gradation of standard river sands is more or less uniform between particular grain sizes, the gradation of natural sandy soils, silty-sand, and clayey-sand changes with depth as well as horizontal distance. To understand the compressibility behavior and pore pressure response of such deposits, extensive research needs to be carried out under various boundary and loading conditions. In the current experimental study, m_v and Δu of naturally occurring soil samples from 10 locations at different depths from Kutch region are presented. Sitharam et al. (2004) and Ravishankar et al. (2005) studied the dynamic behavior of soil collected from the epicenter of 2001 Bhuj Earthquake. Variations in grain size distribution (GSD), fines content (FC), and nature of fine content (plastic and non-plastic) were observed. The dependency of volume compressibility and excess pore pressure response of soils was analyzed in the context of variations in plastic fines (clay) and non-plastic fines (non-plastic silt).

2 Material Properties and Specimen Preparation

Soil samples from 3000 km^2 area of Kutch region were collected. Basic geotechnical properties, viz. GSD, Specific Gravity (G_s), and Atterberg Limits, are given in Table 1 (Hussain and Sachan 2017). A series of isotopically consolidated undrained triaxial compression (CIUC) tests were performed (ASTM D4767-04 2004). Figure 1 shows the GSD of the soils used in the current study (Hussain and Sachan 2017). Triaxial tests were performed on cylindrical specimens of 50-mm-diameter and 100 mm height. Specimens were prepared at in situ dry density using moist tamping method at 8% water content. Saturation of the specimens was accomplished by back pressure saturation. It involved flushing the specimens with CO_2 for 45 min followed by water flushing. Water equivalent to 2–3 times the volume of the specimen was pushed through the specimen at low pressure. Following water flushing, backpressure was applied in two saturation ramps to acquire B value larger than 0.95.

3 Results and Discussion

CIUC triaxial tests were analysed in the context of volume compressibility (m_v) and pore pressure response (Δu, A, and I_{B2}) for 32 soils collected from 10 locations of seismically active Kutch region.

3.1 Volume Compressibility of Kutch Soils

Specimens were subjected to isotropic consolidation under the effective confining pressure of 100 kPa, after saturation. Volumetric strains experienced by soil specimens at the end of consolidation varied over a wide range, 1.02–7.84%. The corresponding volume compressibility, m_v, evaluated was found in between 10^{-4} and 7.8×10^{-4} m^2/kN. The volume compressibility of soils is controlled by factors, viz. density and fines content. Further silt (non-plastic) and clay (plastic) fractions have a different effect on the compressibility of soils. In the current study, both variables control the volumetric strains during consolidation. Figure 2 shows volume compressibility of the soils of Kutch Region as a function of FC, silt (non-plastic) content, and clay (plastic) content. Effect of FC, plastic fines, and non-plastic fines on volumetric compressibility of silty-sands and clayey-sands are shown in Fig. 2a. The response of all 32 soils is shown in Fig. 2b. In silty sands, compressibility

Table 1 Geotechnical properties of soils in Kutch region

Soil Name	Depth m	γ_{di} kN/m^3	G_S	GSD				FC %	Atterberg Limits			Soil Class
				G	S	M	C		LL	PL	PI	
				%	%	%	%		%	%	%	
Chang Dam		23°27.591’ N		70°24.408’ E								
S1 (L1)	0.5	15	2.67	6	78	11	5	16	-	-	-	SM
S2 (L2)	0.5	15.69	2.66	0	82	15	3	18	15.5	NP	NP	SM
S3 (L2)	1.5	15.70	2.68	5	76	17	2	19	20.0	NP	NP	SM
Kharoi		23°28.367’ N		70°23.330’ E								
S4	0.5	16.01	2.67	0	82	13	5	18	15.7	NP	NP	SM
S5	1.5	16.90	2.67	5	84	9	2	11	13.8	NP	NP	SP-SM
S6	2.5	16.00	2.67	1	86	11	2	13	12.7	NP	NP	SM
Suvai Dam		23°36.428’ N		70°29.821’ E								
S7	0.5	17.03	2.67	0	72	21	7	28	15.1	NP	NP	SM
S8	1	14.37	2.66	2	74	19	5	24	14.6	NP	NP	SM
S9	1.5	13.55	2.66	1	82	14	3	17	14.8	NP	NP	SM
Fatehgarh Dam		23°41.369’ N		70°48.057’ E								
S10	0.5	17.17	2.72	0	1	62	37	99	54.0	19.0	35	CH
S11	1.5	15.53	2.67	1	54	42	3	45	19.9	NP	NP	SM
S12	2.5	15.45	2.69	0	78	21	1	22	16.3	NP	NP	SM
Chobari		23°30.722’ N		70°20.881’ E								
S13	0.5	17.51	2.7	0	56	42	2	44	24.2	14.2	10.0	SC
S14	1.5	16.96	2.71	0	51	42	7	49	26.2	14.8	11.4	SC
S15	2.5	17.57	2.7	0	59	37	4	41	24.6	16.2	8.4	SC
Khadir		23°50.82’ N		70°14.39’ E								
S16	0.5	15.94	2.66	2	79	17	2	19	16.9	NP	NP	SM
S17	1.5	16.82	2.66	1	74	22	3	25	15.6	NP	NP	SM
S18	2.5	16.96	2.66	2	88	9	1	10	13.7	NP	NP	SP-SM
Tappar Dam		23°15.017’ N		70°07.586’ E								
S19	0.5	17.36	2.67	0	58	24	18	42	34.1	11.2	22.9	SC
S20	1.5	16.39	2.66	5	66	14	15	29	31.4	10.1	21.3	SC
S21	2.5	17.67	2.68	4	72	14	10	24	22.2	10.5	11.7	SC
Budharmora		23°20.634’ N		70°11.501’ E								
S22	0.5	17.71	2.68	2	69	21	8	29	23.2	14.6	8.6	SC
S23	1.5	14.27	2.71	1	34	46	19	65	44.3	15.7	28.6	CL
S24	2.5	12.26	2.7	2	18	57	23	80	65.8	26.9	38.9	CH
Banniari		23°24.299’ N		70°09.910’ E								
S25	0.5	13.37	2.74	0	17	81	2	83	26.4	NP	NP	ML
S26	1.5	14.59	2.75	0	5	68	27	95	47.2	18.6	28.6	CL
S27	2	16.26	2.68	0	68	26	6	32	24.6	11.6	13.0	SC
S28	2.5	17.60	2.69	1	78	13	8	21	28.0	11.7	16.3	SC
Shivlakha Dam		23°24.659’ N		70°35.128’ E								
S29	0.5	14.43	2.69	0	71	25	4	29	16.8	NP	NP	SM

(continued)

Table 1 (continued)

Soil Name	Depth m	γ_{di} kN/m^3	G_S	GSD				FC %	Atterberg Limits			Soil Class
				G	S	M	C		LL	PL	PI	
				%	%	%	%		%	%	%	
S30	1.5	14.88	2.7	1	88	9	2	11	17.4	NP	NP	SP-SM
S31	2	16.37	2.69	1	74	18	7	25	15.0	NP	NP	SM
S32	2.5	13.40	2.68	0	28	50	22	72	39.0	15.5	23.5	CL

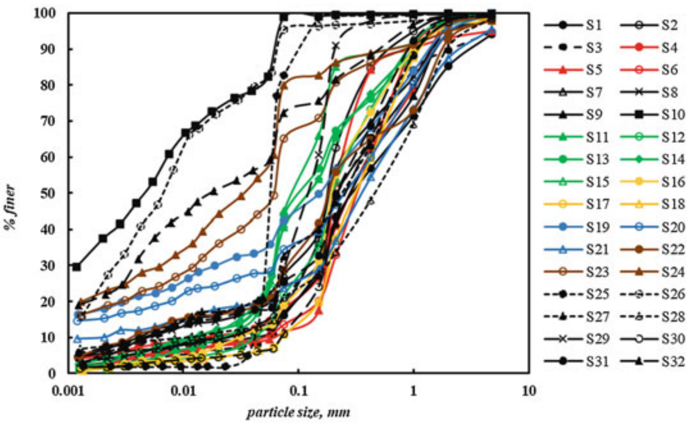

Fig. 1 Grain size distribution of Kutch soils

increased at a higher rate with increase in plastic fines as compared to the similar increase in non-plastic fines. Thevanayagam (2000) and Bandini and Sathiskumar (2009) reported that the effect of silt content on m_v was insignificant, which was aligned with the current study. However, such dependency was not found in other soils. The microstructure of a soil varies with FC and affects the mechanical behavior of the soil. FC of soils in the current study varied from 11 to 99%. FC in the silty sands varied from 11 to 49% with non-plastic fines varying from 9 to 42%. Smaller silt particles create a large number of unstable particle contacts, which exhibit higher compressibility (Yamamuro et al. 2008). As the soils in the current study are having GSD distributed over a wide range, particles with different microstructure tend to collapse and rearrange themselves in different configurations affecting the soil compressibility significantly (Zhao et al. 2013; Zhao and Zhang 2013). Evaluated values of m_v for the soils in the current study implied large settlements and highly contractive response under undrained loading conditions.

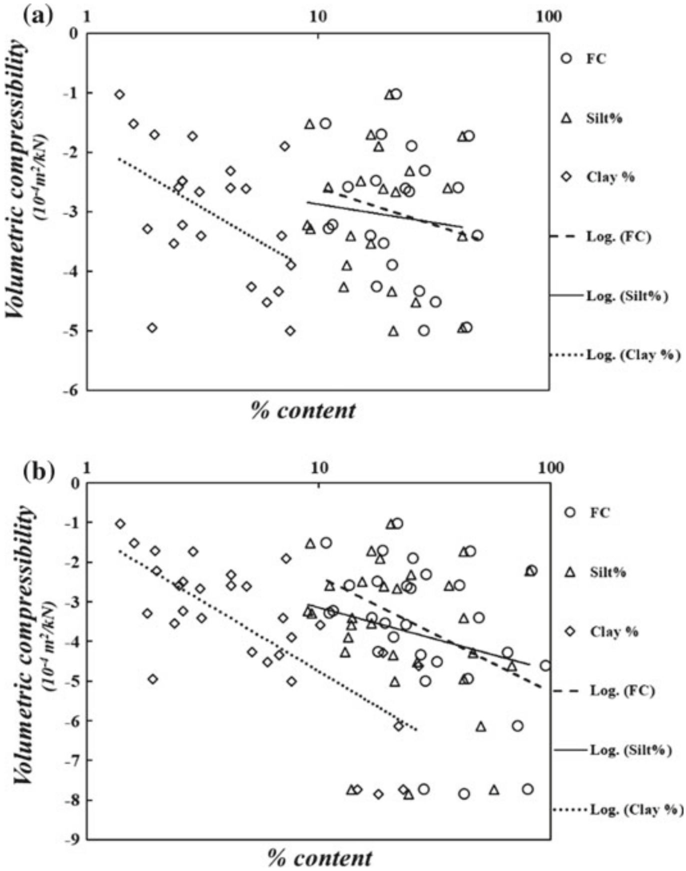

Fig. 2 Variation of volume compressibility, m_v, with FC and nature of fines for Kutch region **a** Silty-sands and clayey-sands, **b** All soils

3.2 *Pore Pressure Response of Kutch Soils with Varying Plastic and Non-plastic Fines*

After the isotropic consolidation was over, specimens underwent undrained shearing. Excess pore pressure evolution (Δu) varied over a wide range with strong dependency on FC and nature of fines present in the particular soil. Soils in the current study exhibited positive pore pressure evolution during undrained shearing varying from 63 to 98 kPa. Excess pore pressure (Δu) evolved at a faster rate during initial stages of loading, $\varepsilon_a < 0.5\%$. Excess pore pressure evolution (Δu) was observed to be greater than 95 kPa for silty sands indicating static liquefaction. It was observed that 50% of final Δu developed at peak stress (Table 2). Higher Δu meant reduced mean effective pressure leading to low load-bearing capacity of soils. Figure 3a, b, c shows the evolution of Δu during undrained shearing of soils collected from Chang Dam,

Table 2 Volume compressibility and pore pressure response of Kutch soils

Soil Name	NF* %	PF† %	FC %	ε_v%	Peak Parameters		Residual Parameters			
					ε_f %	u_f	ε_r	u_r	A_f	I_{B2}
Chang Dam										
S1 (L1)	11	5	16	-	-	-	-	-	-	-
S2 (L2)	15	3	18	2.5	0.45	46	25	96	1.1	0.52
S3 (L2)	17	2	19	3.5	0.42	48	25	96	1.3	0.50
Kharoi										
S4	13	5	18	4.3	0.6	57	25	92	1.6	0.38
S5	9	3	12	3.2	0.5	53	25	96	1.3	0.45
S6	11	2	14	2.6	0.48	52	25	98	1.2	0.47
Suvai Dam										
S7	21	7	28	4.3	0.54	51	23	95	1.4	0.46
S8	19	5	24	2.6	0.48	49	22	89	1.3	0.46
S9	14	3	17	3.4	0.42	45	25	97	1.2	0.53
Fatehgarh Dam										
S10	62	37	99	-	-	-	-	-	-	-
S11	42	3	45	1.7	0.55	51	25	95	1.1	0.46
S12	21	1	22	1.0	0.56	46	19	95	0.7	0.50
Chobari										
S13	42	2	44	5	0.76	53	25	85	1.1	0.37
S14	42	7	49	3.4	0.5	34	25	75	0.7	0.55
S15	37	4	41	2.6	0.5	28	25	76	0.7	0.63
Khadir										
S16	17	2	19	1.7	0.46	50	25	96	1.5	0.48
S17	22	3	25	2.7	0.54	53	25	93	1.4	0.43
S18	9	2	11	1.5	0.38	44	25	90	1.0	0.54
Tappar Dam										
S19	24	18	42	7.8	0.5	35	25	73	0.7	0.52
S20	14	15	28	7.7	0.5	41	25	72	0.7	0.43
S21	14	10	24	3.6	0.5	41	25	80	0.8	0.49
Budhar mora										
S22	21	8	29	5.0	0.5	44	25	90	0.7	0.37
S23	46	19	65	4.3	0.5	20	25	76	0.7	0.74
S24	57	23	80	7.7	0.5	15	25	66	0.5	0.77
Banniari										
S25	81	2	83	2.2	0.56	46	25	97	0.9	0.53
S26	68	27	95	4.6	0.5	21	25	70	0.6	0.7
S27	26	6	32	4.5	0.76	52	25	85	1.2	0.38
S28	13	8	21	3.9	0.7	46	25	85	1.4	0.46
Shivlakha Dam										
S29	25	4	29	2.3	0.47	44	25	96	1.1	0.54
S30	9	2	11	3.3	0.45	48	25	98	0.9	0.51
S31	18	7	25	1.9	0.5	37	25	96	0.7	0.61
S32	50	22	72	6.1	0.5	30	25	75	0.8	0.60

**Non-plastic fines (silt) †Plastic fines (clay)*

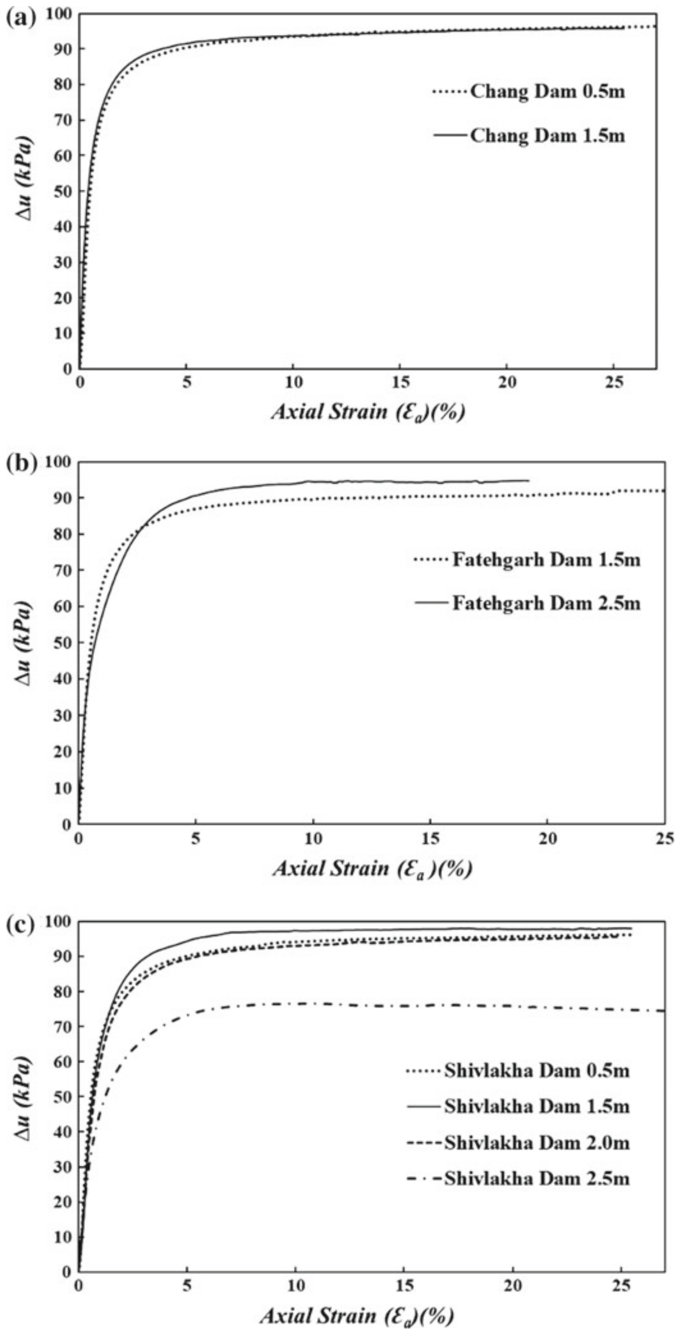

Fig. 3 Pore pressure response of soils from **a** Chang Dam, **b** Fatehgarh Damand, **c** Shivlakha Dam

Fatehgarh Dam, and Shivlakha Dam, respectively. Δu generated was observed to be very close to initial effective confining pressure (100 kPa) leading to the static liquefaction within the soil mass. A large fraction (greater than 0.5) of the ultimate excess pore pressure was observed at strain (ε_f) corresponding to peak stress. FC and nature of fines was observed to impact the excess pore pressure evolution in the natural soil deposits of high seismicity Kutch region.

Figure 4a, b shows the evolution of Δu during undrained shearing for soils collected from Tappar Dam and Chobari, respectively. With higher FC but lower plastic fines, the evolution of Δu of Chobari soils was observed to be similar to that of Tappar Dam soils, which have low FC but significant plastic fines. Lade and Yamamuro (1997), Hazirbaba (2005), Hazirbaba and Rathje (2009), and

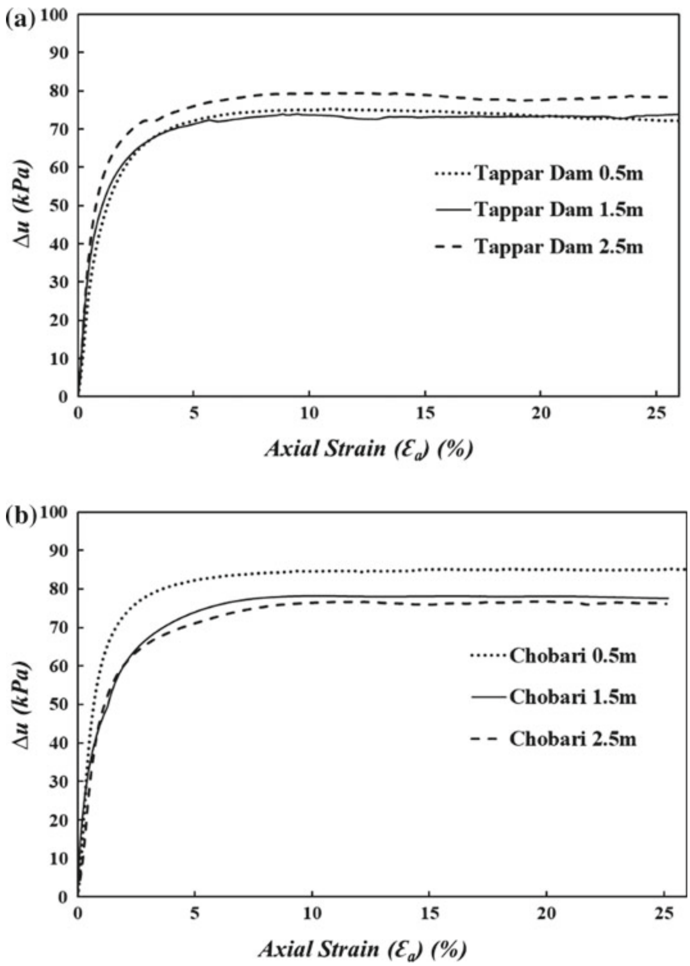

Fig. 4 Pore pressure response of soils from **a** Tappar Dam and **b** Chobari

Thevanayagam et al. (2002) reported similar findings for comparable non-plastic fines content. In the current study at similar non-plastic fines content, Δu is decreased due to the coupled effect of plastic and non-plastic fines. Findings in the current study agree with the study on plastic fines reported by Carraro et al. (2009).

Similar to Δu, Skempton's pore pressure parameter *(A)* at different strain levels was observed to be affected by FC and nature of fines. Figure 5a, b, c shows the evolution of parameter (*A*) for soils from Chang Dam, Fatehgarh Dam, and Shivlakha Dam respectively. Parameter *A* was found to evolve with axial strain. After gradual increase at lower axial strains, parameter A increased sharply due to continuously increasing Δu and reducing deviatoric stress, ε_d. For Fatehgarh Dam (at 2.5 m depth), *parameter A* attained values as high as 94 due to static liquefaction resulting from loss of effective confining stress due to high Δu.

Soils having significant fines and showing significant residual strength, parameter *A* stabilized at a constant value as shown in Fig. 6b. Parameter *A* for soils collected from Chobari was significantly higher as compared to soils from Tappar Dam. This was due to the higher fraction of plastic fines in soils from Tappar Dam. However, total fines content was higher in soils from Chobari.

Similar to undrained brittleness index (I_B) proposed by Bishop (1971), a parameter undrained brittleness index with respect to pore pressure (I_{B2}) was defined to quantify the Δu generation as shown in Eq. 1.

$$I_{B2} = \frac{u_{\mathrm{ultimate}} - u_{\mathrm{yield}}}{u_{\mathrm{ultimate}}} \tag{1}$$

where u_{ultimate} was the Δu at large strains and u_{yield} was Δu at initial peak stress. I_{B2} captured the evolution of Δu before and after yield. I_{B2} of the soils varied from 0.37 to 0.77 in the current study, as shown in Table 2. Figure 7 shows the variation of I_{B2} with FC, non-plastic fines and plastic fines for the silty sands in Kutch region. I_{B2} values for silty-sand soils (non-plastic fines) were observed to be on the lower side. However, higher values were obtained for soils containing significant plastic fines.

Lower I_{B2} corresponds to the higher fraction of ultimate Δu evolving at peak stress indicating softening behavior of soil. This was due to the higher development of Δu at early stages of shearing resulting in reduced mean effective pressure and softening behavior.

In the current study, majority of the soil samples (28 out of 32) had total fines content (FC) greater than 15%, which exhibited large volume compressibility and low shear strength. Typical design CBR values for such type of soil are in the range of 10–20% (Authority 2002). However, the total FC was found to be mainly composed of non-plastic fines. These soils could be used in the design and construction of subgrade and embankments systems by addressing the issues relating to large volume compressibility and pore water pressure. This might require compaction with lower lift thickness. Erodibility and siltation of silty sands can be prevented by using geo-textiles.

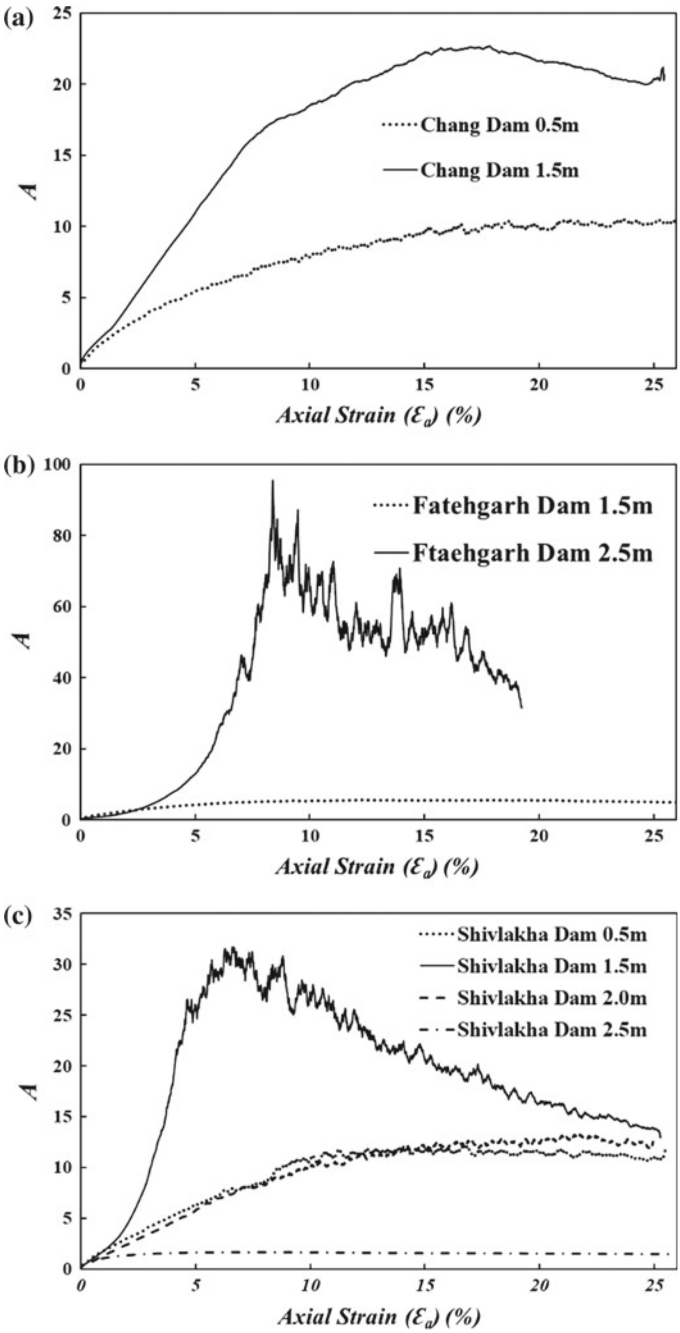

Fig. 5 Evolution of skempton's pore pressure parameter, *A*, for soils from **a** Chang Dam, **b** Fatehgarh Dam and **c** Shivlakha Dam

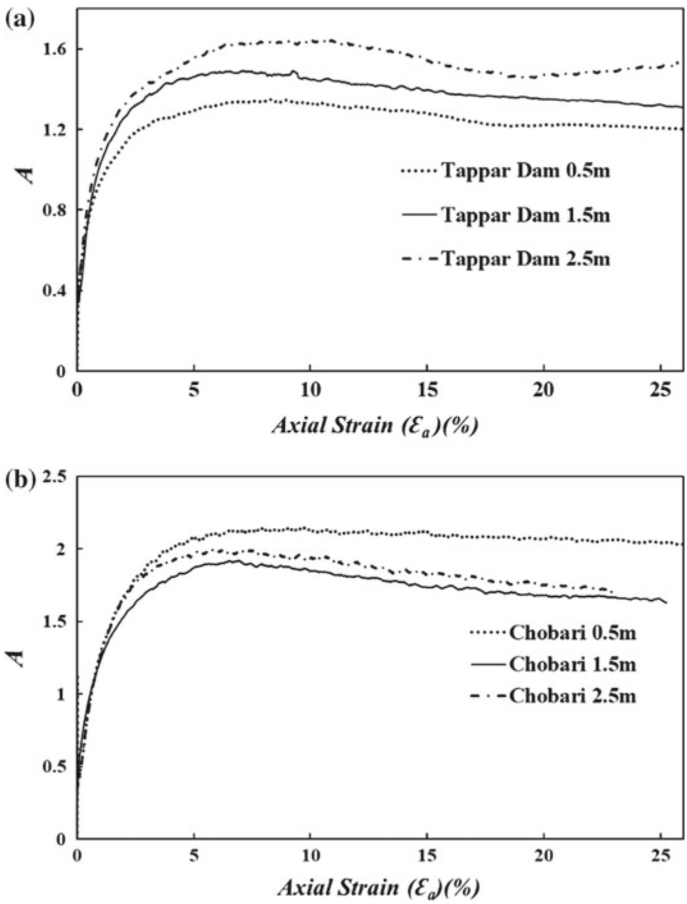

Fig. 6 Evolution of Skempton's pore pressure parameter, *A*, soils from **a** Tappar Dam and **b** Chobari

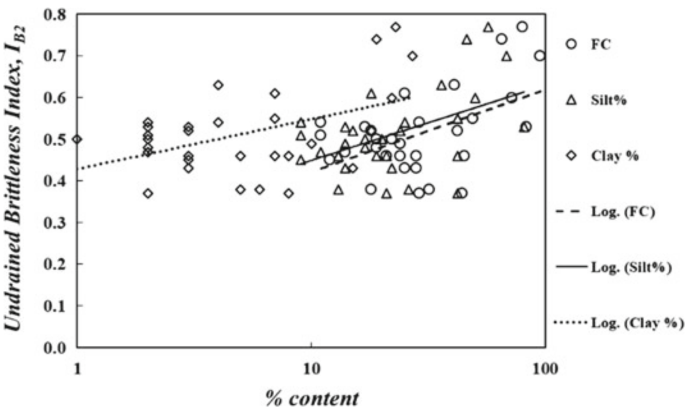

Fig. 7 Variation of I_{B2} with FC and nature of fines for soils in Kutch region

4 Conclusions

Volume and pore pressure response of Kutch soils were explored in the current study. Twenty out of the 32 soil samples classified as SM type soils. The response of silty sands was significantly affected by the FC, which was observed to be controlled by the relative proportion of plastic and non-plastic fines. Volume compressibility and pore pressure response of Kutch soil with varying plastic and non-plastic fines were studied. Key observations are mentioned below.

1. The volume compressibility, m_v, of soils increased with increase in FC. For similar FC, soils with a larger fraction of plastic fines exhibited higher m_v. Large m_v indicated larger consolidation settlement during loading.
2. Pore pressure response was affected by FC and nature of fines. Δu reached 98 kPa in soils with low FC. Soils with higher FC, Δu generation was observed to be controlled by the relative proportion of plastic and non-plastic fines. Plastic fines inhibited pore pressure generation to a higher degree compared to non-plastic fines. The similar response was observed for parameter A.
3. Variation in plastic and non-plastic fines exhibited a subdued effect on undrained brittleness, I_{B2}, with respect to pore pressure. Soils with higher plastic fines content exhibited increased I_{B2} indicating the resistance to pore pressure generation.
4. Large volume compressibility and pore pressure generation of Kutch soils need to be considered in the design of subgrades and embankments to protect them from settlement and shear failure.

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