IJESRT
INTERNATIONAL JOURNAL OF ENGINEERING SCIENCES & RESEARCH TECHNOLOGY

NONLINEAR BEHAVIOUR OF CYLINDRICAL WALL OF GROUND ELEVATED EMPTY RC SILO UNDER WIND LOAD

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DOI: 10.5281/zenodo.199563

ABSTRACT
Nonlinear analysis of civil structures gives as an analyst's tool and entered the practicalworld of design engineering. Most FEA applications undertaken by design engineers were limited to linear analysis.It has been demonstrated that the ultimate strengths of some typical two-story planar frames (under gravity loads) determined by inelastic analysis are 10–30% higher than those estimated from the first-hinge elastic approach. Design of silos using the nonlinear analysis has described in this thesis. In this thesis it has shown that nonlinear analysis of two models i.e., Model 2.3 and Model 2.4 the dimension of that silos are height 6.4&12.8m respectively. Analysis has done on that silo by incorporating the nonlinear properties to those models as M25 grade concrete properties and changing step module values.It has explained that how the stresses vary in case of nonlinear analysis.

KEYWORDS: Fea, Linear Analysis,Non Linear Analysis.

INTRODUCTION
Over the last decade, finite element analysis (FEA) stopped being regarded only as an analyst's tool and entered the practical world of design engineering. However, until recently, most FEA applications undertaken by design engineers were limited to linear analysis. Such linear analysis provides an acceptable approximation of real life characteristics for most problems design engineers encounter. Nevertheless, occasionally more challenging problems arise, problems that call for a nonlinear approach. A decade ago, engineers recognized FEA as a valuable design tool. Now they are starting to realize the benefits and greater understanding that nonlinear FEA brings to the design process. In some applications, it is sufficient to assume that the material remains elastic, i.e. that the deformation process is fully reversible and the stress is a unique function of strain. However, such a simplified assumption is appropriate only within a limited range, and in general must be replaced by a more realistic approach that takes into account the inelastic processes such as plastic yielding or cracking. Inelastic analysis covers structural aspects such as: incremental analysis, limit analysis, shakedown analysis, and optimal design, beam structures subjected to bending and torsion, yield line theory of plates, slip line theory, size effect in structures, creep and shrinkage effects in concrete structures. System design by inelastic analysis provides several important advantages over conventional design method. For one, the system strength can be directly assessed from the analysis without the need for calculating effective length factor or checking the member-based beam-column interaction equations in the specification. For another, inelastic analysis may lead to the design of lighter and more economic structures. The ultimate load carrying capacity of a structural steel system with even a modest capacity to redistribute loads can be much larger than what is determined by the design of individual members. It has been demonstrated that the ultimate strengths of some typical two-story planar frames (under gravity loads) determined by inelastic analysis are 10–30% higher than those estimated from the first-hinge elastic approach. Perhaps the most significant advantage of design by inelastic analysis is that it is better able to capture the system behavioural characteristics as they currently are understood. Since inelastic analysis can explicitly indicate the failure mode(s). It becomes possible to identify different performance limits in design (e.g., first yielding or hinge, incipient system instability, etc.). Design by inelastic analysis will force the engineer to think carefully about the behaviour of the structure as an integrated system.

ISSN: 2277-9655
[Kumar* *et al.*, 5(12): December, 2016] Impact Factor: 4.116
IC™ Value: 3.00 CODEN: IJESS7

DESCRIPTION OF MODELS

Model-1.1: Ordinary Silo (refer Fig. 3.2-1)

The Model-1.1, which has been considered by *Samanta and Datta (2005)*,includes a 350 MT capacity coke silo supported on six Nos. of circular columns of height 6.0m from the top of foundation. The column heads are connected with a ring beam of depth 1000mm at level (+) 5.0m, which is supporting the entire silo volume together with the conical hopper supporting the material.The total height of the silo is 16.5m above the ground level, whereas total height of wall including roof slab is 11.5m. The thickness of the wall has been assumed as 200mm. At the topmost level (+16.5) m, the silo wall is connected to the slab beam system of the roof and at the bottom (+5.0m) it's connected to the ring beam. The other relevant general arrangements have been shown in Fig. 3.4

FIGURES:

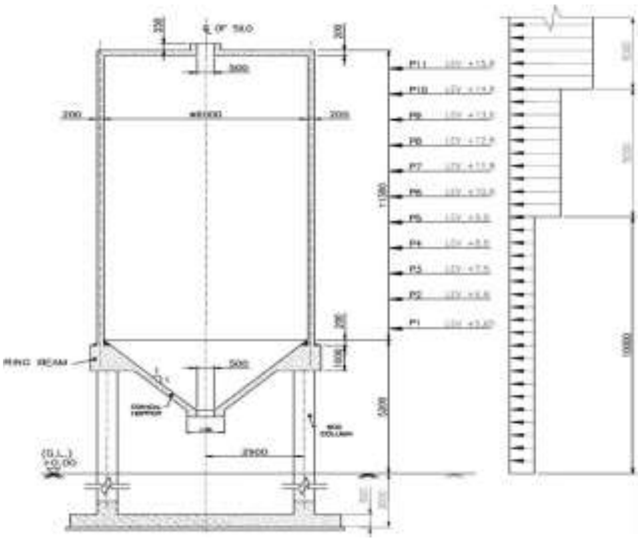
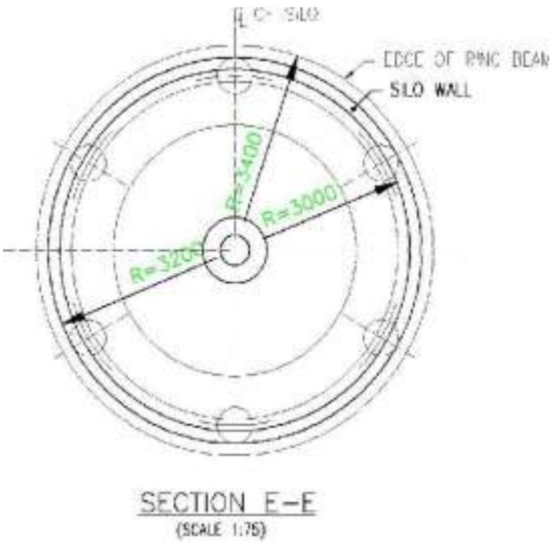


Fig. 3.3:Vertical Section of Elevated RC silo (Ordinary)

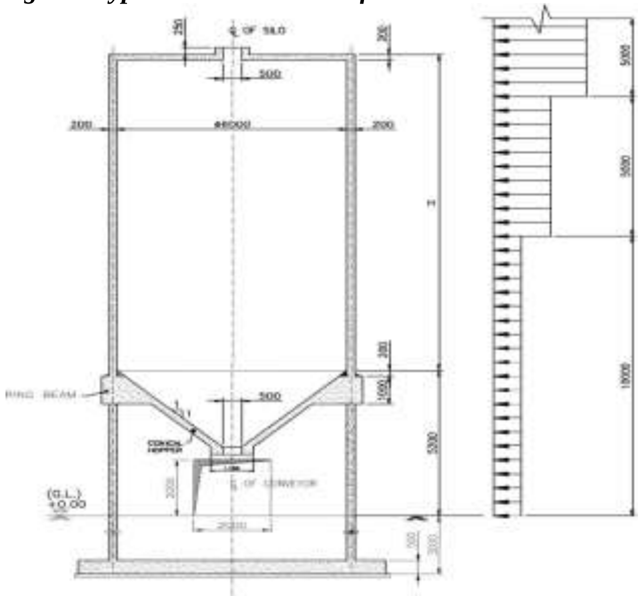


Model-2 (ordinary silo with H-D variations)

Model-2 includes total seven Nos. of silo structures of varying height to diameter ratios. It is here mentioned that the basic configuration and geometries such as, supporting columns, outer diameter of silo shell, bottom ring beam, roof slab system, thickness of wall of all the silo structures have been kept identical with that of the Model-1.1(ordinary Silo).

ISSN: 2277-9655
Impact Factor: 4.116
CODEN: IJESS7

Fig. 3.5:Typicalvertical section of model with H/D variation



Numerical Method in the Abaqus (FEM Software):
After getting the silo models ready in the Abaqus, loaded the models in the Abaqus finite element software. And performed the nonlinear analysis in the Abaqus with few modifications as described below steps wise.

PART MODULE:
In Abaqus analysis, the silo structure has modelled into 4 parts, those are column, ring beam, roof opening and roof slab with the prescribed dimension.Part moduleconsidering modelling space 3D, deformable type, Base Feature is solid shape and Extrusion type.

MATERIAL PROPERTY MODULE

For Plastic analysis the Concrete was modelled using smeared cracking model, as implemented by ABAQUS/CAE User's manual to define the properties of plain concrete outside the elastic range. Only concrete grade M25 is used in this model. Here in smeared cracking model, all the compressive stress and plastic strain as provided in Table 3.7.2-1, were given as input in the data table and in the sub option Tension stiffening was introduced with displacement of 0.08mm for M25. In general, isotropic behavior in elastic and plastic model were given to concrete and steel separately, since the stress strain ratio of mainly concrete is assumed to be constant in every direction. The given input of concrete and steel was taken as 25000MPa and 200000MPa for young's modulus and with Poisson's Ratio 0.17 and 0.3 respectively. Mass Density of Concrete and steel was taken as 24x10-6 N/mm3 and 78 x10-6 N/mm3 respectively. Finally, Plastic model was defined for steel to Columns Ring Beam Silo Model Roof Slab yield at relatively low temperatures under a static loading and creep effects wereunimportant. The yield stress of plastic metal along with plastic strain as defined in Table 3.7.2-2, were given as input in the data table.

STEP MODULE:
In step 1 a static General step has been chosen for the loading condition and interaction was given on the initial step. The default time period for the step given was 1 unit time (as given in default) and initial size has been given 0.1 in the incrimination tab.Automatictime incrimination has been chosen which starts incrimination using the value entered for the initial increment size. Full Newton's method was takenas a numerical technique for solving nonlinear equilibrium equations and geometric nonlinearity was considered. In the Field output request based on the analysis procedure of the step for the whole section, stresses (S.MISSES), Displacements (U), and reaction forces (RF),were given. Two history output request are created, in which Abaqus generates Displacement (U3, UT) and Forces/Reactions (RF3, RT) at two Sets position in the model, for theX–Yplots on load and displacements.

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METHOD-2 (FINITE ELEMENT METHOD):

Unlike method-1 (Analytical method), entire silo including the 6nos. of columns, ring beam on the top of the columns,conical hopper,roof slab, top ring beam, pilaster etc. have been modeled withAbaqusin three dimension with fixed support condition at the top of the foundation as shown in the relevant Figs. All the parts of the silo structures are generated as three dimensional, deformable solids elements in Abaqus(version 6.9). “P” and “H” type convergence tests were performed in order toselect the type of element and mesh density to be used in the static analysis of silo models under static wind load. Linear and quadratic hexahedral and tetrahedral elements provided byAbaquswere tested. The silo wall except the ring beam, conical hopper, top ring beam as the case may be of all the silo geometries considered were modeled using 8-noded standard linear hexahedral elements with incompatible modes (C3D8I) whereas, linear tetrahedral elements with hybrid formulation (C3D4H) were used to model ring beam with conical hopper, roof slab and top ring beam. In all cases, element sides have been maintained close to 200 to avoid any mess distortion effect on the results. It is here mentioned that the medial axis algorithm technique is used for mess generation. The total elements, type of elements, nos. of each type of elements and total nodes generated for all models are shown in table: 3.5-1. The behavior of the material (concrete) is assumed as isotropic and elastic with Young’s modulus = 25000MPa,Poisson’s ratio = 0.17 andmass density = 2.4E-009 t/mm^3. Thecalculatedwindload (equivalent concentrated forces) has been distributed among the nodes along the periphery as well as height of a particular model (model 1.1) of height 11.5m for a case study. In all models, the windload has been applied as surface load for studying the overall response of the wall of all the silo structures. The detail calculation of nodal forces (point loads), surface loads and development of Fourier equation is shown next in tabular form. The support condition of all the structures has been simulated as fixed with ring beam at (+) 5.0m level supporting the conical hopper, as the ring beam is sufficiently rigid compared to the same of the silo wall. There may be a lateral sway of the ring beam as a whole depending on the slenderness of thecolumns supporting 38wall as the case may be, but same will take place together with the silo wall. The 3D silos modeledsuch a way, have been analyzed usingAbaqusto get the values of deformation and stresses developed at different level. Finally, the design stresses are noted for all the cases mentioned. The results obtained for model-1.1 in both the method of analysis are compared. The comparative studies for all the models analyzedby method-2 have alsobeen done based on stiffening and variation of height to diameter ratios.

ASSESSMENT OF WIND LOAD AS POINT LOAD(FOR MODEL-1.1 ONLY):

Since the wind load varies both with height as well as in plan, such variation of wind load as derived for Method-1 has beencalculated at levels P1to P11as shown in Fig.-1. For representation of the wind pressure variation, equivalent concentrated forces were calculated using MS excel spreadsheet to apply at the 24 nos. of nodes created along the silo periphery at all levels.The magnitude of concentrated force applied at a particular node was calculated by multiplying the pressure intensity to the contributing area.The concentrated forces were applied at an interval of one meteralong the height except the two levels at thebottom of the shell as shown in Fig. 3.4-2(Case-I). First level is at a height of 0.65m from the top of ring beam and the 2nd level is 1.15m from the first. The wind load acting on the discrete columns and the ring beam along with hopper was not considered here assuming its contribution to the silo wall deformation being negligible and the intension of this investigation is to find deformation of the wallonly. The detail calculation of nodal forces has been shown next in Table 3.5-1 to Table 3.5-5.

RESULTS AND DISCUSSION

General:

Results obtained in method-1and method-2 are presented, discussed and compared for model -1.1 where wind load has been applied as equivalent concentrated forces (point loads). Second, results obtained from nonlinear analysis for the model 2.4 has compared with the same model which is a done by the linear analysis. Drawn the graphs and obtained the deflected configuration for the model 2.4 by the both linear and nonlinear analysis. Compared the critical stress values for the same model in windward, leeward and 78° windward directions with the nonlinear analysis results.

Results for Linear Analysis (For Model-2): (Ref. [3])

Model-2.5 (wall height = 12800, H/D=2.0)

As soon as the deformed configuration and stress contour are examined after application of wind load as point forces, some irregularities have been observed onthe deformation pattern of the silo wall, which is obvious. Hence an attempt has been made to apply the same wind load as surface load after partitioning the surface of the silo wall. May 16, 2016 Fig. 4.4-1shows deflected configuration and stress contours of Model-2.5 with deformation scale factor of 5000 analyzed by finite element method. Fig.4.4-1 shows Von Mises and vertical stress contours of entire silo wall, whereas Figs. 4.4-3 shows cross sectional distortion of wall at maximum stress level and maximum displacement level. From the deformed configuration (refer Fig. 4.4.*1*) it is understood that the maximum increase in diameter takes place at about mid height whereas the maximum decrease in the same occurs above the mid height. Whereas, from the distorted cross section (refer Fig. 4.4-3) it is said that the same does not take exactly the shape of an ellipse, center is slightly shifted to the wind direction and also the ovalisation of cross section is clearly observed.The summary of Von Mises stress (σvon), longitudinal stress (σvert) and hoop stress(σhoop) have been listed in Table 4.2.-1. The maximum value of hoop stress is -0.218MPa occurs at (+) 11.2m level i.e. at the mid height of the wall in the winddirection as in case of models with lower H/D ratios. Maximum value of hoop stressat 780 to wind direction is 0.197 MPa taking place at the same height.Vertical stresses are critical at the wall (leeward side as well as 78degree to windside)just above the junction of ring beam and wall and which may be occurred due tolocal bending of same. Values are even greater than the maximum hoop stress values.

Results for Nonlinear Analysis (For Model-2):
Model-2.5 (wall height = 12800, H/D=2.0)
Nonlinear analysis has performed on model height 12.8m and H/D ration is 2 and got the deformed shape of the silo, Hoop stresses, Von Mises stresses, and vertical stresses. Drawn the graphs for the variation of stresses along with the silo height.

The summary of Von Mises stress (σvon), longitudinal stress (σvert) and hoop stress(σhoop) have been listed in Table. The maximum value of hoop stress is - 0.221MPa occurs at (+) 11.2m level i.e. at the mid height of the wall in the wind direction in outside nodes. Maximum value of hoop stress at leeward wind direction is 0.107 MPataking place at the (+) 5.2m height. Critical value of vertical stress is -.0.2340 MPaoccurs at leeward side at (+) 5.20m level and maximum Von Mises stress also occurs at the same location, but the value is about 0.2800 MPaThe maximum values of the Von Mises Stresses at windward direction in inside and outside the nodes are 0.2488 MPa& 0.2089 occurs at height (+) 12.2m respectively.The minimum values for the same stress in 780 windward direction has occurred 0.2761 at (+) 5.20m level.

FORMULAES:
[F] = [K] * [d]
$\sigma = (1-)Do^{el}:(\varepsilon - \varepsilon^{pl}) = D^{el}:(\varepsilon - \varepsilon^{pl})$
$\varepsilon = \varepsilon e + \varepsilon p$
$\varepsilon_e = \sigma / E_c$

TABLES

Model	Diameter(D) of silo wall	H/D ratio	Height(H) of silo wall(mm)
Model-2.1		1.00	6400
Model-2.2		1.25	8000
Model-2.3		1.50	9600
Model-2.4	6400mm	1.80	11500
Model-2.5		2.00	12800
Model-2.6		2.25	14400
Model-2.7		2.50	16000

Table:3.3-values for compression stress and plastic strain for M25

Compression stress (mm)	nom	True	true	Plastic strain
		nom(1+nom)	ln(1+nom)	$true^{-(true/E)}$
0	0	0	0	
7.5	0.00030	7.50225	2.9996E-04	0
15	0.00170	15.02550	1.6986E-03	1.0975E-03

20	0.00300	20.06000	2.9955E-03	2.1931E-03
25	0.00350	25.08750	3.4939E-03	2.4904E-03
20	0.00390	20.07800	3.8924E-03	3.08924E-03

CONCLUSION

The walls of silo structures are prone to ovalisation, when subjected to wind Loads and this should be taken care of for proper design and detailing of such type of structures against wind load. It has been tried first to study such phenomenon for an elevated single RC silo wall by analytical method (method-1) and for this investigation an annular ring of silo wall of height unity at a particular level, subjected to highest intensity of wind pressure has been considered. It is found that in this method of analysis only hoop stresses are obtained for a particular level considered, that too much on conservative side. And secondly tried with numerical method that is static linear analysis, compared with the analytical method. So the author deals with the behavior of the wall of said structures under wind load in empty condition numerically (i.e.method-2) using finite element package ABAQUS (version-6.14), also author deals with the static Nonlinear analysis and results has compared with static linear analysis. For this analysis wind load has been applied to the entire silo wall in terms of equivalent concentrated forces (point loads) and the detail calculation of the same is shown in results and discussion chapter. The method laid down in the relevant IS code of practice. From the observation of the results obtained in finite element method of analysis it is clear that the Von-Mises stress values higher in nonlinear analysis compared to linear analysis, especially in Leeward direction. In the view of hoop and vertical stresses values are significantly lower nonlinear analysis compared to linear analysis. So, by this it has been understood that effect of nonlinearity observed in Von Mises stress compared to two other stresses. In addition to that analytical method(i.e.method-1) provides empirical values only in the hoop direction, whereas FE modeling provides not only the hoop stress values but also longitudinal /vertical stresses that are equally significant and the same does not follow the typical pattern expected from the beam bending theory.

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