RELIABILITY-BASED ANALYSIS OF PRECAST CONCRETE INSERTS

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ABSTRACT

The design and analysis of the inserts utilized in the precast concrete elements encompass a high level of uncertainty. This uncertainty arises from the complex mechanical behavior of these connections that leads to many different possible modes of failure. Prestressed/Precast Concrete Institute recommends its design handbook to design these inserts to withstand four times the actual weight of the concrete elements. The current study resembles the behavior of these inserts in terms of reliability analysis based on the statistical properties of the contributing parameters. The analysis recommends establishing a specific target of reliability index for each connection based on its complexity and the expected loading pattern. An example for the analysis of lifting sleeve connection is presented herein.

ملخـص

يشتمل تصميم و تحليل الأجزاء المعدنية المستعملة بالعناصر الخرسانية سابقة الصب على درجة عالية من اللامحدودية. و تنتج هذه اللامحدودية عن السلوك الميكانيكي المعقد لهذه الوصلات مما يؤدي إلى أنواع متعددة من نماذج الانهيار المتوقعة. و يوصي المعهد الأمريكي للخرسانة سابقة الإجهاد و سابقة الصب بأن تصمم هذه الوصلات على أن تتحمل أربع أضعاف وزن العنصر الخرساني. و لذا فتقدم هذه الدراسة تحليل لسلوك هذه الوصلات من منظور التحليل الإعتمادي لها بناءا على الخواص الإحصائية للعوامل المؤثرة على انهيار الوصلة. و توصي الدراسة بتحديد معامل إعتمادية مخصص لكل وصلة تبعا لتركيبتها و الأحمال المتوقع التعرض لها. و تحتوي هذه الورقة على مثال لتحليل وصلة رفع العناصر الإنشائية سابقة الصب كنموذج تطبيقي لمحتويات الدراسة.

INTRODUCTION

In the construction process of the electrical plant at Ayoun Moussa, Egypt (1997-2000), more than 8,000 precast concrete panels were utilized in the eight buildings of the plant. Most of the panels were typical within the same building but the panels at the corners and the openings demanded special configurations for dimensioning and handling. The total number of panel types designed and constructed came up to more than 800 types. One of the problems encountered in the design was the design of the steel inserts in the precast concrete panels. The lifting connection was the most complicated and the most effective one through the design and handling phases of the panels because of the high uncertainties of the connection. The uncertainties of these parameters rise dramatically with the increase of the number of parameters controlling the behavior as well as their variability.

The reliability analysis presented in this paper was conducted in order to investigate the most effective parameter on the design and behavior of such inserts. The reliability analysis considers the probabilistic distribution of the different parameters constituting the driving force and the resistance force for the connection. Calibrating the standard design processes in terms of reliability analysis is recommended (Melchers, 1987). Most of the recent codes consider the probabilistic distribution of the considered parameters. A number of pioneering attempts to calibrate a new generation structural design code to an existing code have been reported by Lind (1977), Ravindra and Galambos (1978), Ellingwood et al. (1980), and Mirza (1987).

BACKGROUND

The prime advantage of utilizing precast concrete elements is the high level of quality control that could be achieved in the workshop. In some projects, the field location is the purpose of utilizing precast elements either because of tight space for cast-in-situ concrete or because of the remote location of the project. On the contrary, the economy of utilizing precast elements is the prime disadvantage for small projects or unique structure. On the contrary, if the elements are being produced typically in the project or in similar projects the precast choice becomes feasible economically.

One of the major references for the design of the precast concrete elements is the PCI (Precast/Prestressed Concrete Institute) Design Handbook (1992). Design of precast elements as provided in that Handbook is based primarily on the basics and fundamentals recommended by ACI (American Concrete Institute) Building Code (1989). In the PCI design handbook, there are two phases to be considered in the design of these elements; the first is the manufacturing phase including stripping, handling, transporting, and erection activities. The second phase is concerned with the in-service conditions along with the corresponding dead, live, wind, and seismic loads. The design of precast concrete members needs an individual analysis because concrete strength at the manufacturing age is low. Besides, the supporting points and orientation are usually different than those of the panel at its final position. The wall type panels are extremely critical in this concern because of the large size and heavy weight versus high slenderness. Also, these panels are exposed, from manufacturing stage to erection stages, to loads different than those sustained at service stage.

Hardware inserts in the precast concrete elements is one of main differences between the conventional concrete and precast concrete design. These items are not related to the design of conventional concrete neither to the design of structural steel elements. Meanwhile, the loads exerted on these elements are variables with high level of uncertainty starting from the placement of these items up to the working phase of the precast concrete element. The loads applied to these items vary with the type of the precast element type, the category of the design with respect to utility and the accepted crack width, and the function of the insert itself.

RELIABILITY THEORY

Reliability analysis takes into account the major uncertainties in the design of a structure or element. The performance function, **Z**, in the reliability analysis of a structure can be defined as the resistance minus the loading as;

$$\mathbf{Z = R - S} \qquad \text{Eq. (1)}$$

where; R is the accumulated resistance forces and S is the driving forces for a specific mode of failure and load configuration.

When the performance function is greater than zero, a safe state exists and failure is defined otherwise. Numerous failure functions exist for any structure because it may fail in many different ways and because of different effects. These failures may be caused by a variety of loads such as dead, live, wind, seismic, snow, or combinations of each. It should also be kept in mind that failure does not necessarily imply structural collapse; it could be defines as exceeding other limits, such as deflection or crack width. This has lead to the performance function also being referred to as the limit state function or the failure function.

The resistance is a function of material properties and element or structure dimensions, while the load is function of the different types of the applied loads, material densities, dimensions of the structure, each of which is a random variable. The fundamental variables that define and characterize the behavior and safety of a structure are termed the “basic” variables. They are usually the variables employed in conventional structural analysis and design. Typical variables are dimensions, densities, materials, loads, and material strengths. The compressive strength of concrete is considered a basic variable in any concrete element even though it can be related to more fundamental variables such as cement content, water to cement ratio, aggregate size, grading and strength, etc. In general, the latter variables are not used in strength or safety analyses.

The probabilistic distribution attributes of each parameter in the analysis need to be established. Once the basic variables and their distributions are established, the simple limit state form, **(R – S)**, may be replaced with a generalized version expressed directly in terms of the basic variables. If the vector **X** represents the basic variables of the problem then the resistance **R** could be expressed as $\mathbf{R=G_R(X)}$ and the load effect as $\mathbf{S= G_S(X)}$. The functions $\mathbf{G_R}$ and $\mathbf{G_S}$ may be nonlinear, and in general the cumulative distribution function $\mathbf{F_R}$, must be obtained by multiple integration over the relevant basic variables:

$$F_R(r) = \int^r \,.....\int f_X\,(x)\,dx \qquad \text{Eq. (2)}$$

The resistance and load functions may not be independent such as when some loads act to oppose failure or when the same dimensions affect both functions. In this case, it is not valid to utilize the conventional integral for reliability analysis. Also, it is not applied when there are more than one mode of failure for the considered element or structure with different combinations for resistance and load functions. A more general formulation is needed in each of the mentioned cases.

Several techniques exist to perform structural reliability analyses. These techniques include Monte Carlo simulation, first-order second-moment methods (FOSM), and stochastic finite element. The Monte Carlo simulation technique was utilized in the reliability analysis of reinforced concrete elements by Grantt et al. (1978) and Mirza and McGregor (1989). These authors recommended this method because the complex nature of the design of RC elements. Monte Carlo simulation is a computer intensive method with thousands simulations being common. It depends on establishing a deterministic relationship derived between the performance of the system and each variable affecting the performance. The statistical properties of the distributions of all variables must be known to be utilized in the simulation. In this case of analysis, sampling each random variable X_i randomly to give a sample value $\hat{x}_i$. The limit state function $G(\hat{x})$ is then evaluated. The experiment is repeated many times, each time with a randomly chosen vector $\hat{x}$ of $\hat{x}_i$ values. Then the reliability index, β, can be determined from the relationship;

$$\beta = \frac{\mu_Z}{\sigma_Z} \qquad \text{Eq. (3)}$$

where; μ_z is the mean value of the performance function, , and σ_z is the standard deviation of the performance function. Obviously the number of trials required is related to the desired accuracy of the probability function. The process is performed to calculate the performance of a specified synthetic system in order to figure out the overall variability of the structural system.

ANALYSIS

The analysis of the inserts in precast concrete elements is governed primarily by the expected modes of failure that would be experienced under different cases of loading. Each mode of failure depends on a specific order of circumstances of the parameters contributing to this mode of failure. The general parameters for these modes are concrete compressive strength; yield strength of the insert, geometric configurations of the connection, and the applied load. Some of these parameters contribute to the resistance of the system and others contribute to the applied loads. In some cases, the same parameter contributes to the resistance and the load. For example, the concrete dimension in a precast concrete element contributes to the dead load applied to the element and to other attracted loads like wind and seismic loads. Meanwhile, it contributes to the strength of the element and the distribution of the stresses and strains.

The following steps are the general procedure to test the reliability of connections with hardware inserts in the precast concrete elements:

1. The expected modes of failure that would be affecting the design of this particularity are to be identified. The attributes and the boundaries of each mode are determined. Also, the different cases of loading and the corresponding sources of resistance need to be allocated.
2. The analytical model for each mode of failure in terms of the governing variables is derived utilizing the perspective of the boundaries set forth in step (1). The analytical model should quantify the mechanical capacities of the element as well as the coefficients those may be needed to simulate the actual behavior of the element to counteract the applied load.
3. The required statistical parameters for the employed variables are acquired from the previously reported results. These parameters represent the probabilistic distribution of the mechanical properties of the connection components, the configurations of the connection, and the applied loads. The sources of these data must be counted upon and representing the behavior of the considered connection as possible.
4. The numerical analysis for the different modes of failures is conducted thereafter utilizing the concluded statistical parameters. Each mode of failure would have a separate analysis procedure that brings about the reliability

indices for the connection design as per this specific mode of failure and the applied loads.

5. The reliability indices come out for each mode of failure are evaluated with respect to the other modes. The lowest reliability indices values are the governing values and their modes of failure are the most critical ones.
6. A sensitivity analysis for the design is developed for the effective variables in terms of reliability indices. This analysis rationalizes the susceptibility of the connection to the variation of the considered parameters and leads to the most appropriate level of safety required for the designed connection.

EXAMPLE

The example of lifting sleeve is presented here to simulate the reliability analysis for the inserts of precast concrete elements. The lifting sleeve system, as shown in Figure (1), consists of three main elements; the anchorage bar, the screwed sleeve embedded in the precast element, and the lifting bar. The weight of the element is transferred from the concrete element to the anchorage bar through bond between the steel bar and concrete. The next load movement is the transfer from the anchorage bar to the sleeve through direct bearing between the two elements. Finally, the load is transmitted from the sleeve to the lifting bar through the screw in both sides.

The main variables governing the design of this connection is the strength and thickness of concrete element, strength and diameter of anchorage bar, strength and diameter of sleeve, and strength and diameter of lifting bar. Also, the length of the groove in the sleeve, total length of the sleeve, length of the screw, and length of anchorage bar are contributing to the strength of this connection.

A limit state function is established for each expected mode of failure. This function is expressed utilizing the performance function. In the considered case, the load effect is the weight of the precast concrete element while the resistance is the counter forces that develop in the concrete and hardware components to through transmitting the weight to the lifting crane. This resistance takes different paths and values according to the mode of failure. The contribution of the constituent components of the resistance as well as the contribution of the weight in the reliability analysis depends completely on the statistical properties of each of these components.

The following modes are considered in this example for probable failure in accordance with the presented pattern:

Mode 1 The simplest mode of failure is the tension failure of the lifting bar. The control of this failure is easy to be controlled through the direct estimate of the required capacity as per the following relationship;

$$R = \frac{\pi\, d_b^2}{4}\, f_{yb} \qquad \text{Eq. (4)}$$

where; d_b and f_{yb} are the diameter and the yield strength of the lifting bar in cm and kg/cm^2, respectively.

Mode 2 The connection may fail due to tensile failure of the sleeve above the anchorage bar and under sleeve groove that holds the lifting bar. This failure is expected to be accompanied by shear failure in the concrete element. The shear failure is expected to be like a truncated pyramid at 45^o. The shear strength of concrete is assumed, as per PCI design handbook equation 6.5.3, equal to $\left(0.7\sqrt{f_c}\right)$ where, f_c, is the concrete cylinder compressive strength at the handling age in kg/cm^2. The anticipated resistance for this mode is simply represented by the following relationship;

$$R = \pi\left(\frac{d_s^2 - \left(d_b + 0.1\right)^2}{4}\right) f_{ys} + 2\left(\sqrt{2}\, l_{s1}\, t_c \times 0.7\sqrt{f_c}\right) \qquad \text{Eq. (5)}$$

where d_s is the exterior diameter of the sleeve in cm, f_{ys} is the yield strength of the sleeve in kg/cm^2, l_{s1} is the embedment length of the lifting bar in cm, and t_c is the thickness of the concrete element in the out-of-plane direction in cm.

Mode 3 The sleeve may rupture at the level of interacting with the anchorage bar. The mass of the sleeve is hollowed out at this location to allow for the anchorage bar to intermingle. This failure is adjunct to progressing shear failure in the concrete element as well. The resistance of the connection based on this mode of failure is worked out utilizing the relationship given below;.

$$R = \left[\frac{\pi\, d_s^2}{4} - d_s\left(d_a + 0.1\right)\right] f_{ys} + 2\left(\sqrt{2}\, l_{s2}\, t_c \times 0.7\sqrt{f_c}\right) \qquad \text{Eq. (6)}$$

where; d_a is the diameter of the anchorage bar in cm and l_{s2} is the distance from the centerline of the anchorage hole in the sleeve to the extreme fiber of the concrete element in cm.

Mode 4 The extension of the sleeve underneath the anchorage interaction is very important since it may experience longitudinal splitting at this location if the embedment is not sufficient. This mode is subject to the progress of shear failure in the concrete element, too. The shear strength of the sleeve is considered, 0.75 f_{ys}, as per the PCI design handbook equation 6.5.14, subsequently the strength of the connection for the mode of failure is predicted as;

$$R = 0.75\, d_s \left(l_{s3} - l_{s2} - \frac{d_a}{2} - 0.05 \right) f_{ys} + 2 \left(\sqrt{2}\, l_{s2}\, t_c \times 0.7 \sqrt{f_c} \right) \quad \text{Eq. (7)}$$

where; l_{s3} is the total embedment length of the sleeve in cm. This mode is restricted by additional resistance below the bearing area underneath the sleeve but this zone is susceptible for crushing at the expected shear failure of concrete. Hence, it is not represented in the previous mathematical expression.

Mode 5 The anchorage bar may fail under shear at the sides of the sleeve. This mode needs to be accompanied by the shear failure in concrete element as in other modes of failure. The shear strength of the anchorage bar is considered, 0.75 f_{ya} , as per the recommendation of the PCI, too. The estimated resistance as per this mode is developed utilizing the following relationship;.

$$R = 2 \times \pi \, \frac{d_a^2}{4} \times 0.75 \, f_{ya} + 2 \left(\sqrt{2}\, l_{s2}\, t_c \times 0.7 \sqrt{f_c} \right) \quad \text{Eq. (8)}$$

where; f_{ya} is the yield strength of the anchorage bar.

Mode 6 Bond failure between concrete and anchorage bar is the last but not least important mode to be considered for this connection. The bond resistance is presumed by substituting the tensile strength of the bar in the minimum required length relationship given in the PCI design manual equation 6.5.1. This substitution yields the following relationship;

$$R = 6.25 \sqrt{f_c}\, l_a \quad \text{Eq. (9)}$$

where; l_a is the length of the anchorage bar.

The PCI design handbook recommends, "…use embedded inserts and erection devices with pullout strength at least equal to 4 times the actual weight lifted". The reliability analysis conducted within the scope of this work considered this recommendation in the analysis. Hence, the configurations of the examples used in the analysis are arranged so as to satisfy the relationship $(R \geq 4\, S)$ for all the considered modes of failure. All strength reduction factors are neglected in the parameter configurations of the analysis, so that the reliability indices and their sensitivity are easy to be noticed and compared.

The original connection utilized in the analysis assumes total weight of 6.00 tons per connection with thickness of 15 cm for the concrete element. The concrete compressive strength at time of handling is 200 kg/cm^2 and the yield strengths of the anchorage bar, sleeve, and the lifting bars are 3600 kg/cm^2. The yield strengths for the previous three elements assumed to be the same through the whole analysis. The length of the anchorage bar is 300 cm and its diameter is 2.0 cm. On the other hand, the diameter of the sleeve is 4.20 cm while the lengths l_{s1}, l_{s2}, and l_{s3}, as illustrated in Figure (1), are taken as 10 cm, 13 cm, and 16 cm, respectively. Meanwhile, the diameter of the lifting bar is considered 2.80 cm.

The required statistical properties to perform the reliability analysis include the mean value, the coefficient of variation, and the distribution function of these variables. These data were obtained from the available research results reported by MacGregor (1983) for the mean and variability of compressive concrete strength, Mirza (1979-a) for its distribution, and Ellingwood (1980) for concrete density and the geometric parameters for anchorage bars and sleeves. The data reported by Mirza (1979-c) were utilized for the statistical properties of steel yield strength and Mirza (1979-b) for the thickness of concrete element. The parameters of the lengths of the sleeve and anchorage bar were assumed similar to that of the steel cross sectional area. Table (1) summarizes the utilized properties and the corresponding probabilistic distribution used for each variable. It is noted that the references utilized to obtain these properties were published originally two decades ago but also these references are still valid and being utilized by other researchers like Steinberg (1997) and Al-Harthy (1994). The iterations conducted through the Monte Carlo simulation in this analysis set as 10,000 iteration for each reliability index figure.

The analysis of the lifting insert connection utilized in the example showed the reliability indices presented in Figures (2)-(4). The reliability indices for the different modes of failure range from three to nine but the variability of the reliability indices for the same mode is considerably small. The most critical mode of failure is shown to be Mode 4. The high probabilistic value for failure in Mode 4 is rising from the dependency on two individual lengths with two spectrums of uncertainty that increases the standard deviation of the resistance value and subsequently decreases the reliability indices. The figures present the reliability indices for the six modes of failures with respect to the weight of concrete element, Figure (2), the compressive concrete strength, Figure (3), and the steel yield strength, Figure (4). The figures show that the reliability index for the same mode of failure experiences little variability with respect to any of the three variables mentioned here above.

Sensitivity analysis

It is impossible to establish a unique target for the reliability index for the whole process of design for a certain type of structure or for a certain project. Thus a cheap but critical component should have a higher β value, while a very low probability load combination could have a lower value of β. The results of Ellingwood et al. (1980) for steel and reinforced concrete beams provided reliability indices at the mean value of three. Those results are common for the case of dead and live loads for most of the codes for steel and reinforced concrete structures. For other cases of loading the reliability indices are different such as 2.50 for case when wind load is included and 1.75 for when the seismic load is included.

It is recommended to set a high reliability index for the inserts of the precast concrete panels due to the high uncertainties in the experienced loads. Hence, the design of the considered connection needs to account for different load factors and strength reduction factors so as to achieve the target index for all considered modes of failure. In response to this concept, the following parametric analysis presents the sensitivity of this particular connection with respect to the parameters most affecting the lowest sets of indices.

Based on the previous results, a reliability index of 6.0 is assumed to be the target in this analysis. The modes with lower indices were examined for enhancing their reliability. Mode 4 shows the lowest reliability indices; with the embedment length ($l_{s3} - l_{s2}$) is the governing term in the strength and reliability. Table (2) shows the configuration utilized to study the effect of the embedment length of the sleeve. While Figure (5) shows that to reach reliability index of 6.0, it is recommended to increase the total length of the sleeve to 20 cm despite that 16 cm satisfy the condition of $(R \geq 4\,S)$. Mode 5 is governed primarily by the bond strength developed between the anchorage bar and the concrete element. This is directly proportional to the length of the anchorage bar, l_a. Table (3) and Figure (6) demonstrates the effect of the variation of the length of the anchorage bar on the reliability of the connection considering this mode of failure. It is noticed that the increase of the length of the anchorage bar from 300 cm to 450 cm is required to reach the target of 6.0 for the value of β despite that 300 cm satisfies the four times capacity condition.

Figure (7) presents the relationship between the diameter of the lifting bar and the reliability index assuming that Mode 1 is the governing mode. Mode 1 reflects the capacity of the lifting bar that is directly proportional to the diameter of the bar. The figure shows the sensitivity of the reliability index with the decrease of the diameter of the lifting bar. It is noticed that 2.80 cm is the minimum diameter that satisfies the condition of the, 4S, capacity and the, $\beta =$ 6.0, condition. Table (4) provides the data utilized in this sensitivity analysis.

CONCLUSIONS

The following conclusion are obtained from the reliability analysis performed on the design of the steel inserts in the precast concrete element:

- The behavior of the inserts in the precast concrete elements is complicated that the actual mode of failure may not be determined in advance.
- The items utilized in the handling processes are exposed to higher uncertainties because of the different situations they experience.
- It is recommended to identify the anticipated modes of failure of the connection prior to assigning the required reliability level.
- The statistical properties of the considered parameters are needed to be utilized with respect to the specific function of the parameter in the particularity under consideration.
- It is required to set a different target of reliability for each item for of the designed structure according to the attributes of the item and the expected loads.
- The reliability level is to be assigned according to the probability of the loading case being considered as well as the complicated behavior of the connection under any critical mode of failure.
- The reliability index of 3.0 is the average for the steel and reinforced concrete structural elements but for the inserts of the complicated precast concrete element inserts the index need to be higher to account for the expected high uncertainties.
- The four times capacity set by the PCI design handbook need to be overviewed for each specific connection according to the function and the experienced stresses at the different stages.

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REFERENCES

Al-Harthy, A. S. and Frangopol, D. M., (1994), “Reliability Assessment of Prestressed Concrete Beams”, *Journal of Structural Engineering, ASCE,* Vol. 120, No. 1, pp. 180-199.

Building Code Requirements for Reinforced Concrete. (1989). ACI 318, American Concrete Institute, Detroit, USA.

Ellingwood, B., Galambos, T., MacGregor, J., and Cornell, C. (1980), “Development of a Probabilistic Based Load Criterion for American National Standard A58, Building Code Requirements for Minimum Design Loads in Buildings and Other Structures”, Nat. Bureau of Standards, Washington, D.C.

Grant, L. H., Mirza, S. A., and MacGregor, J. G., (1978), “Monte Carlo Study of Strength of Concrete Columns”, *ACI Journal*, August, pp. 348-358.

Lind, N. C., (1977), “Formulation of Probabilistic Design”, *Journal of Engineering Mechanics Division, ASCE*, 103, EM2, pp. 273-284.

MacGregor, J, G., Mirza, S. A., and Ellingwood, B., (1983), “Statistical Analysis of Resistance of Reinforced and Prestressed Concrete Members”, *ACI Journal*, May-June, pp. 167-176.

Mirza S. A., Hatzinikolas, M., and MacGregor, J. G., (1979-a), “Statistical Descriptions of Strength of Concrete”, *Journal of the Structural Division, ASCE*, Vol. 105, No. ST6, pp.1021-1037.

Mirza S. A. and MacGregor, J. G., (1989), “Slenderness and Strength Reliability of Reinforced Concrete Columns”, *ACI Structural Journal*, Vol. 86, No. 4, pp. 428-438.

Mirza S. A. and MacGregor, J. G., (1979-b), “Variation in Dimensions of Reinforced Concrete Members”, *Journal of the Structural Division, ASCE*, Vol. 105, No. ST4, pp.751-766.

Mirza S. A. and MacGregor, J. G., (1979-c), “Variability of Mechanical Properties of Reinforcing Bars”, *Journal of the Structural Division, ASCE*, Vol. 105, No. ST5, pp.921-937.

Mirza, S. A., (1987), “Probability-Based Strength Criterion for Reinforced Concrete Slender Columns”, *ACI Structural Journal*, Vol. 84, No. 6, pp. 459-466.

Melcher, R. E., (1987), “Structural Reliability; Analysis and Prediction”, Ellis Horwood Limited, Chichester, England.

“PCI Design Handbook”, (1992), Fourth Edition, PCI, Chicago, USA.

Ravindra, M. K., and Galambos, T. V., (1978), “Load and Resistance Factor Design for Steel”, *Journal of Structural Division, ASCE*, 104, ST9, pp. 1337-1353.

Steinberg, E. P., (1997), “Reliability of Structural Steel Haunch Connections for Prestressed Concrete”, *Journal of Structural Engineering, ASCE*, Vol. 123, No. 10, pp. 1382-1389.

Table (1) Statistical Properties

Property	**Mean**	**Coefficient of Variation**	**Probabilistic Distribution**	**Reference**
Concrete Compressive Strength (f_c)	$0.9*f_c$’	0.18	Normal	MacGregor (1983)
Concrete Thickness (t_c)	t_c	$\frac{1}{4t_c}$	Normal	Mirza (1979)
Concrete Density (γ_c)	γ_c	0.10	Normal	Ellingwood (1983)
Steel Yield Strength (f_y)	$1.1* f_y$	0.10	Lognormal	Mirza (1979)
Diameter of Sleeve (d_s)	d_s	0.01	Normal	Ellingwood (1980)
Diameter of Anchorage Bar (d_b)	d_b	0.01	Normal	Ellingwood (1980)
Lengths of sleeve’s components (l_s)	l_s	0.10	Normal	-
Length of anchorage bar (l_b)	l_b	0.10	Normal	-

Table (2) Parameters used to analyze the response to variable panel weight

Wt. per connection, (tons)	1.00	2.00	3.00	4.00	5.00	6.00
Concrete strength, (kg/cm^2)	200	200	200	200	200	200
Yield strength, (kg/cm^2)	3600	3600	3600	3600	3600	3600
Bolt diameter, (cm)	1.2	1.6	2	2.3	2.6	2.8
Sleeve diameter, (cm)	1.4	2.1	2.7	3.3	3.7	4.2
Sleeve length, s_{l1} (cm)	10	10	10	10	10	10
Sleeve length, s_{l2} (cm)	13	13	13	13	13	13
Sleeve length, s_{l3} (cm)	14	15	15	15	16	16
Diameter of anchor, (cm)	0.6	0.8	1.2	1.6	1.8	2.0
Length of anchor, (cm)	50	100	150	200	240	300
Thickness of slab, (cm)	15	15	15	15	15	15

Table (3) Parameter utilized to analyze the response to variable concrete compressive strength

Wt. per connection, (tons)	6.00	6.00	6.00	6.00
Concrete strength, (kg/cm^2)	200	300	400	500
Yield strength, (kg/cm^2)	3600	3600	3600	3600
Bolt diameter, (cm)	2.8	2.8	2.8	2.8
Sleeve diameter, (cm)	4.2	4.2	4.2	4.2
Sleeve length, s_{l1} (cm)	10	10	10	10
Sleeve length, s_{l2} (cm)	13	13	13	13
Sleeve length, s_{l3} (cm)	16	16	16	16
Diameter of anchor, (cm)	2.0	2.0	2.0	2.0
Length of anchor, (cm)	300	300	300	300
Thickness of slab, (cm)	15	15	15	15

Table (4) Parameters utilized to analyze the response to variable steel yield strength

Wt. per connection, (tons)	6.00	6.00	6.00	6.00
Concrete strength, (kg/cm^2)	200	200	200	200
Yield strength, (kg/cm^2)	4200	3600	2800	2400
Bolt diameter, (cm)	2.6	2.8	3.2	3.4
Sleeve diameter, (cm)	4.0	4.2	4.7	5.1
Sleeve length, s_{l1} (cm)	10	10	10	10
Sleeve length, s_{l2} (cm)	13	13	13	13
Sleeve length, s_{l3} (cm)	16	16	16	17
Diameter of anchor, (cm)	2.0	2.0	2.3	2.5
Length of anchor, (cm)	300	300	300	300
Thickness of slab, (cm)	15	15	15	15

Figure (1) Schematic arrangement of the lifting connection

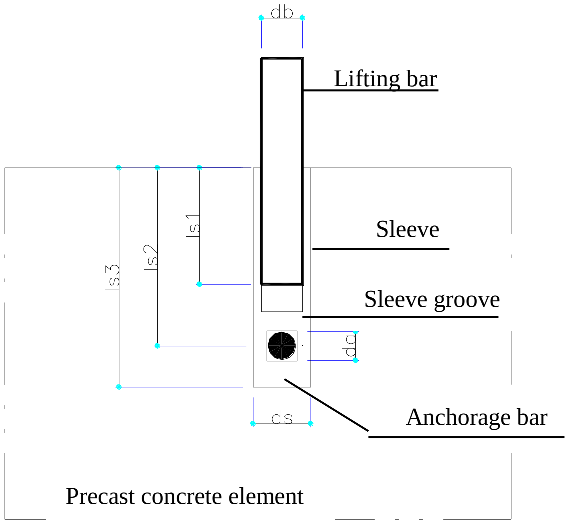
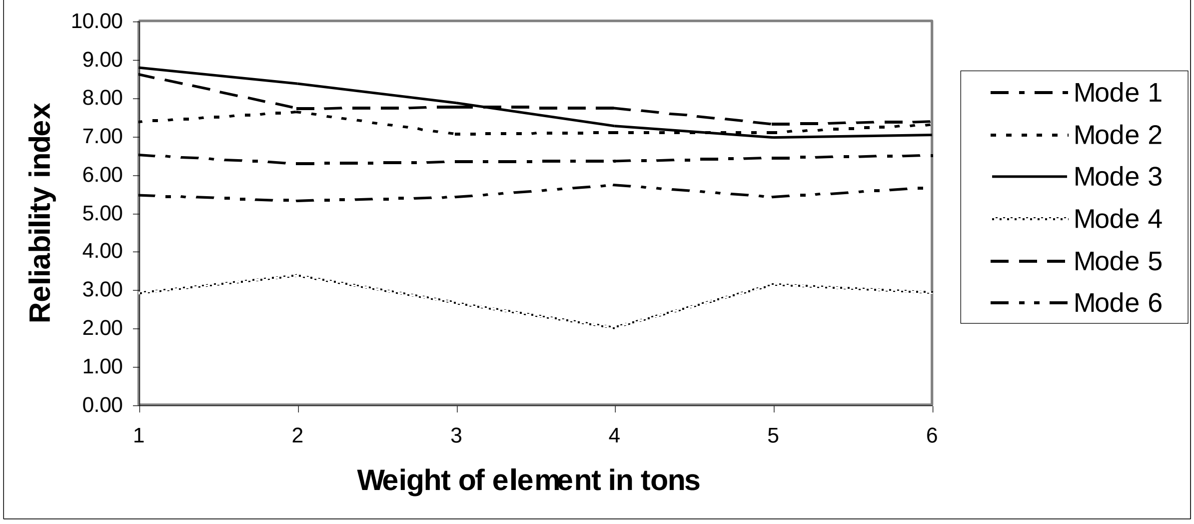


Figure (2) Reliability indices for lifting insert with respect to element weight



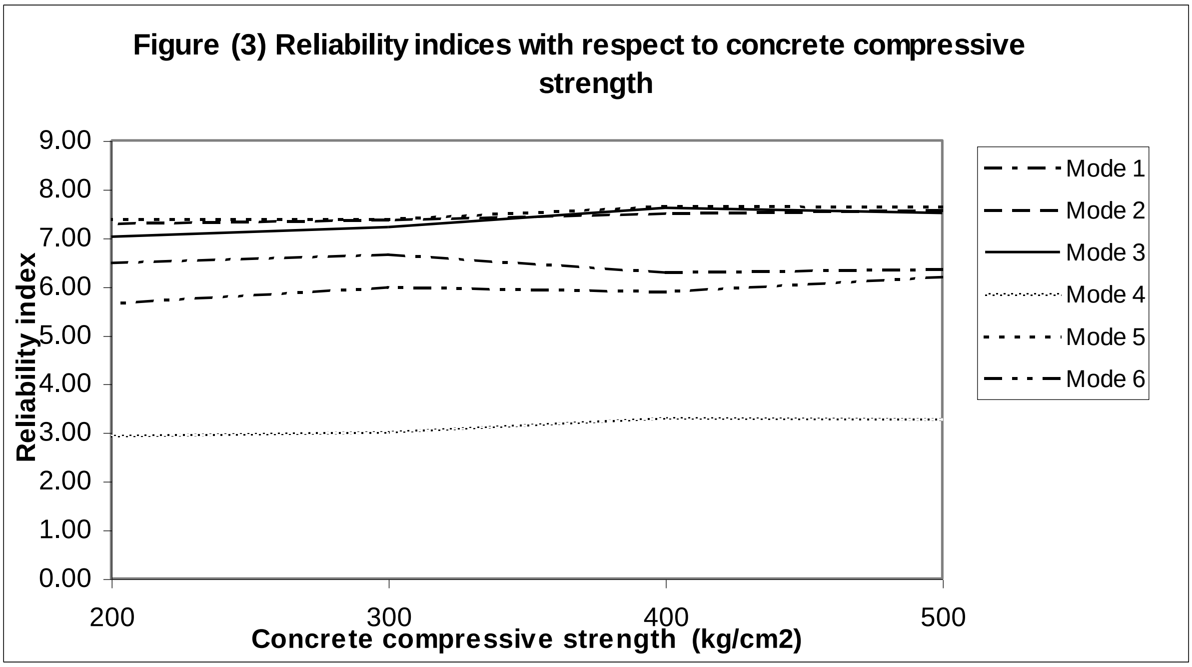


Figure (3) Reliability indices with respect to concrete compressive strength

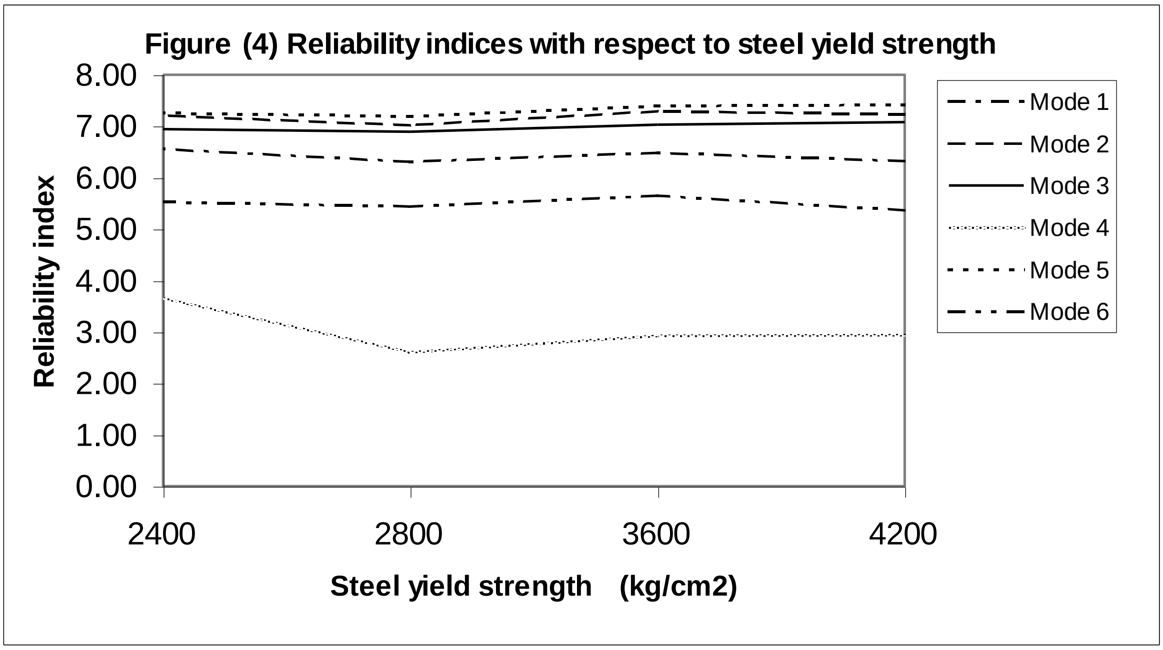


Figure (4) Reliability indices with respect to steel yield strength

Figure (5) Sensitivity of reliability index with respect to sleev embedment length

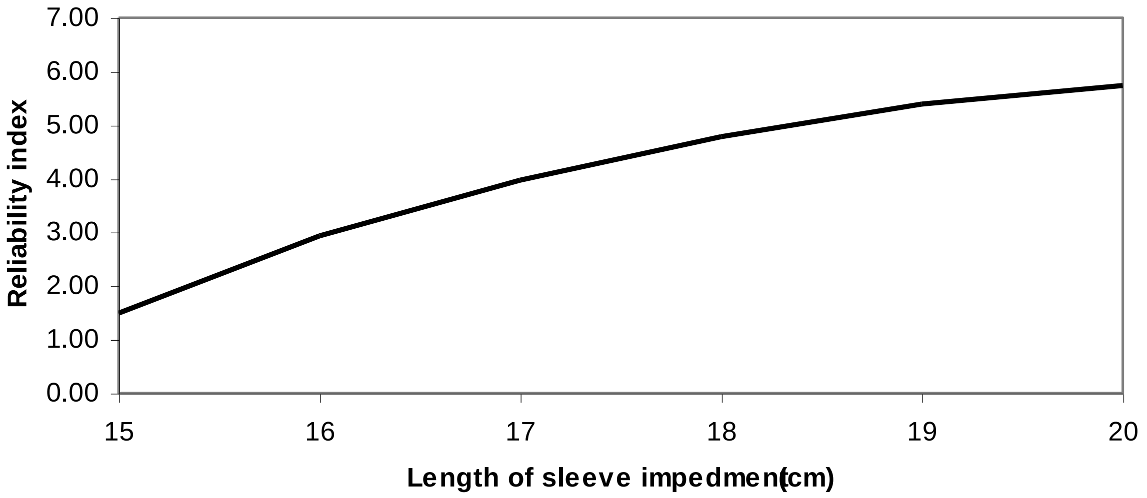


Figure (6) Sensitivity of reliability index with respect to length o anchorage bar

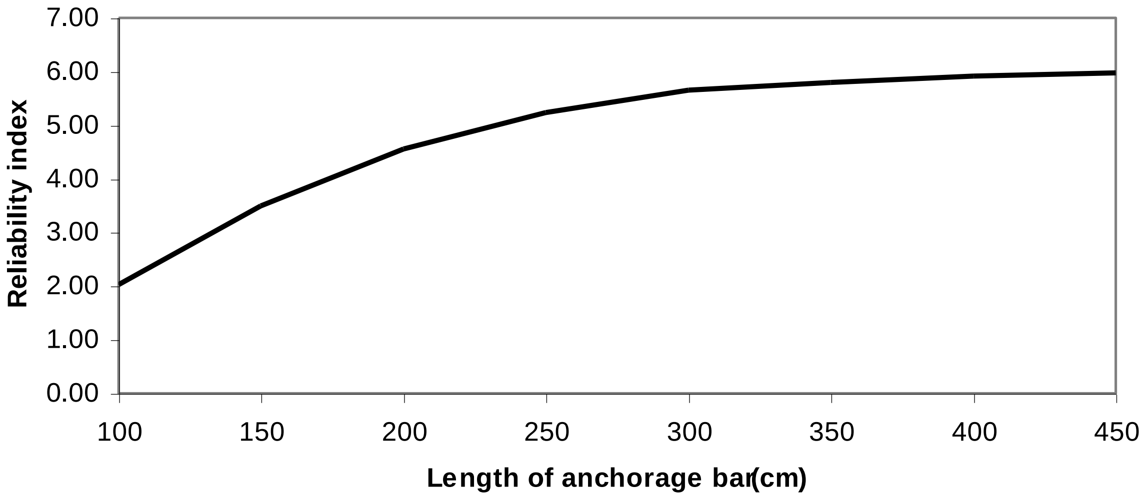


Figure (7) Sensitivity of reliability index with respect to the diameter
lifting bar