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Designing with Composite Materials

Projet avec des matériaux composites à base de fibres

Bemessungsgrundlagen für Faserverbund-Werkstoffe

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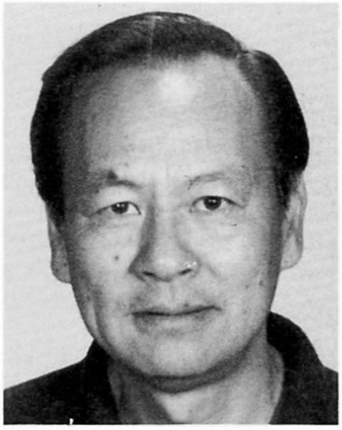

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SUMMARY
Ultra-high stiffness and strength fibers can be made into useful structures through laminates from unidirectional plies oriented at selected angles. Other outstanding properties of composite materials include forty percent lighter than aluminum, high endurance limit, high corrosion resistance, and novel fabrication and assembly techniques. Cost, joining, and inspection are some of the negative factors. Design process has been arbitrary and inconsistent. It is the intent here to outline our approach to design which should make composite structures, large or small, reliable and cost effective.

RÉSUMÉ
Des fibres de rigidité et de résistance extrêmement élevées peuvent être transformées en des structures utiles par l'intermédiaire de couches laminées unidirectionelles orientées selon certains angles. D'autres propriétés extraordinaires de ces matériaux composites comprennent un poids de quarante pourcent inférieur à celui de l'aluminium, une limite d'endurance élevée, une résistance à la corrosion élevée, ainsi que des techniques nouvelles de fabrication et de montage. Les coûts, l'assemblage et l'inspection sont quelques-uns des facteurs négatifs. L'étude du projet a été jusqu'ici arbitraire et irrationelle. L'intention de cet article est de montrer une approche du projet qui devrait rendre les structures composites – grandes ou petites – fiables et économiques.

ZUSAMMENFASSUNG
Fasern hoher Festigkeit und sehr hoher Steifigkeit können in Laminaten mit geeignet ausgerichteten Fasern nutzbringend Verwendung finden. Andere hervorragende Eigenschaften dieser Verbundmaterialien sind die Gewichtseinsparung von 40 % gegenüber Aluminium, die hohe Dauerhaftigkeit, die Korrosionsbeständigkeit und neuartige Herstellungs- und Montagetechniken. Hohe Kosten, die Notwendigkeit des Zusammenfügens und die Ueberwachung sind einige der negativen Faktoren. Die Bemessung geschah bisher mit fallweisen, nicht widerspruchsfreien Annahmen. Es wird hier der Versuch unternommen, ein Bemessungsvorgehen aufzuzeigen, welches Verbund-Bauelemente, seien sie gross oder klein, zuverlässig und kostengünstig werden lässt.

1. INTRODUCTION

Composite materials have been recognized as viable engineering materials for numerous applications in the aerospace industry. The stiffness of unidirectional and laminated composites is predictable and can be applied to structures with confidence. The strength, on the other hand, is empirical and continues to improve as basic data, particularly those under combined stresses, become available. It is the purpose of this presentation to cover some of the recent developments in the prediction of the strength of the laminated composite. With more reliable strength prediction the design process can be simplified considerably, and the degree of empiricism and subjective judgement can be reduced proportionally.

Glass fibers were found to possess unusually high strength many years ago. Many structures have been made from fiberglass, the first modern composite material, since the 1940's. Greater applications of fiberglass continue to emerge, including those for large buildings and nearly every sporting good from canoes to tennis rackets. Glass fibers have one glaring deficiency: low Young's modulus. Until boron fibers were discovered in the late 1950's, the stiffness required for aircraft structures limited materials to various aluminum alloys. Boron fibers made from a vapor deposition process became the first "advanced composite" which could produce structures with stiffness comparable to that of aluminum. Later graphite fibers, made from polyacrylic nitrile fibers through heating and stretching, made composite materials more competitive than aluminum. Later Du Pont's aramid fibers, with a trade name of Kevlar, emerged with the lowest density of all fibers fit for structural applications. More recently new organic fibers continue to emerge. One example is the polyethylene fiber, which is lighter that water.

Unidirectional fibers are combined into unidirectional plies or woven into fabric forms for further processing. The plies and fabric may be preimpregnated with matrix materials. The plies and fabric can be placed over one another to form multidirectional laminates. This process is often done by hand, but some automated layup machines are available. Other popular processing methods include:

Filament-winding of pressure vessels and pipes where either "prepreg" or wet winding can be used. The latter process puts resin on the fiber bundle before it is placed on a mandrel.

Pultrusion of simple and complicated cross-sections where fibers are pulled through dies.

Numerous compression, injection and transfer molding processes developed for reinforced and unreinforced plastics.

2. BASIC STIFFNESS AND STRENGTH DATA

Elastic constants and strengths of orthotropic plies in x-y plane are usually based on four constants (longitudinal, transverse and shear moduli and Piosson's ratio), and five strength data (longitudinal tensile and compressive strengths X and X', transverse tensile and compressive strengths Y and Y', and longitudinal shear strength S).

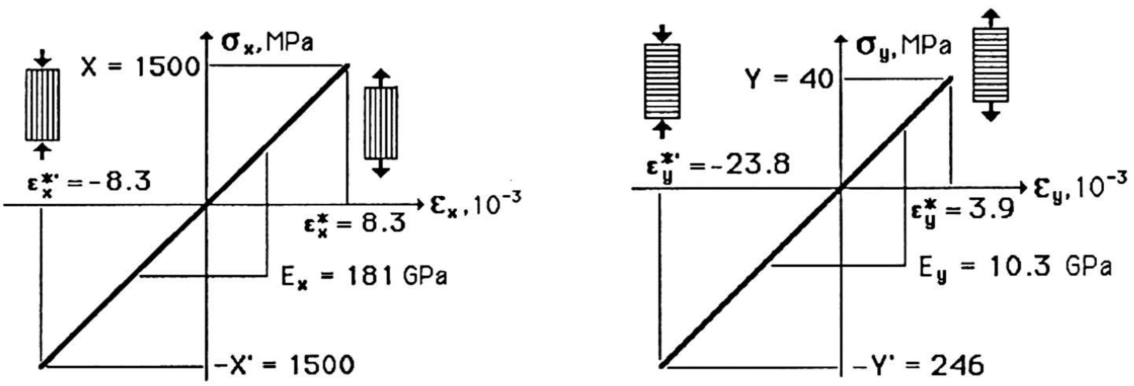


Figure 1 Uniaxial tensile and compressive stiffnesses and strengths of [0] and [90] T300/5208 graphite/epoxy composite material.

In the following table, such data for typical composites are listed. All of the orthotropic stiffnesses and strengths of each composite are also listed. Of the composite materials listed, the first eight are unidirectional plies; the last two are woven fabric.

Table 1 Stiffnesses, fiber volume, specific gravity, unit ply thickness, and strengths of various composite materials

Type	CFRP	BFRP	CFRP	GFRP	KFRP	CFRTP	CFRP	CFRP	CCRP	CCRP
Fiber/cloth	T300	B(4)	AS	E-glass	Kev 49	AS 4	IM6	T300	T300	T300
Matrix	N5208	N5505	H3501	epoxy	epoxy	PEEK	epoxy	Fbrt 934	Fbrt 934	Fbrt 934
Ply eng'g constants and data						APC2		4-mil tp	13-mil c	7-mil c
Ex,GPa	181.0	204.0	138.0	38.6	76.0	134.0	203.0	148.0	74.0	66.0
Ey,GPa	10.30	18.50	8.96	8.27	5.50	8.90	11.20	9.65	74.00	66.00
nu/x	0.28	0.23	0.30	0.26	0.34	0.28	0.32	0.30	0.05	0.04
Es,GPa	7.17	5.59	7.10	4.14	2.30	5.10	8.40	4.55	4.55	4.10
v/f	0.70	0.50	0.66	0.45	0.60	0.66	0.66	0.60	0.60	0.60
rho	1.60	2.00	1.60	1.80	1.46	1.60	1.60	1.50	1.50	1.50
ho,mm	0.125	0.125	0.125	0.125	0.125	0.125	0.125	0.100	0.325	0.175
Quasi-isotropic constants										
E,GPa	69.68	78.53	54.84	18.96	29.02	51.81	78.35	56.24	52.67	47.07
nu	0.30	0.32	0.28	0.27	0.32	0.30	0.30	0.32	0.32	0.32
G,GPa	26.88	29.67	21.35	7.47	10.95	19.88	30.23	21.37	19.89	17.8
Max stress, MPa										
X	1500	1260	1447	1062	1400	2130	3500	1314	499	375
X'	1500	2500	1447	610	235	1100	1540	1220	352	279
Y	40	61	51.7	31	12	80	56	43	458	368
Y'	246	202	206	118	53	200	150	168	352	278
S	68	67	93	72	34	160	98	48	46	46
Max strain, eps E-03										
x	8.29	6.18	10.49	27.51	18.42	15.90	17.24	8.88	6.74	5.68
x'	8.29	12.25	10.49	15.80	3.09	8.21	7.59	8.24	4.76	4.23
y	3.88	3.30	5.77	3.75	2.18	8.99	5.00	4.46	6.19	5.58
y'	23.88	10.92	22.99	14.27	9.64	22.47	13.39	17.41	4.76	4.21
s	9.48	11.99	13.10	17.39	14.78	31.37	11.67	10.55	10.11	11.22

Comparing directionally dependent properties, it is difficult to judge the relative merits of composite materials. It is a common practice to compare the longitudinal properties of composite materials against one another, as well as against isotropic materials like aluminum and steel. Such comparisons are not useful because composite materials are not usually used in unidirectional forms subjected to uniaxial tensile loads. In most cases, bidirecional loads are encountered. Laminated composites having several ply orientations are required. Fortunately for each stiffness component that varies drastically with ply orientation there is an invariant, a constant that is associated with the area under the curve, shown in

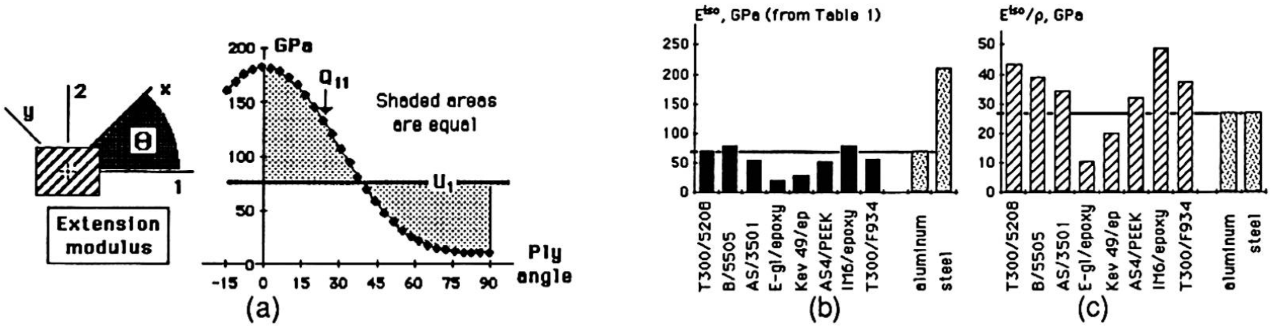


Figure 2 (a) The angular dependency of a typical stiffness component, and its associated invariant U_1.
(b) Effective quasi-isotropic constants of various composite materials, and aluminum and steel.
(c) The same quasi-isotropic stiffness normalized with respect to specific gravity.

Figure 2(a). For each orthotropic material listed in Table 1 we list the equivalent quasi-isotropic constants. It is more meaningful to compare these isotropic constants, on an absolute basis in Figure 2(b); and on a relative basis in Figure 2(c). These comparisons are conservative because we have not claimed the advantage of directional properties which depend on the applied load, and the specific laminate stacking sequence. Suffice to say, the comparisons here make composite materials look less attractive than when the longitudinal properties are used, but they represent the minimum advantages of composite materials.

3. EXAMPLES OF APPLICATIONS

After the discovery of boron filaments in the 1960's, US Air Force began many programs to promote aircraft structures made of composites. The F-111 horizontal stabilizer was the first flight-worthy composite component.

Production of composite stabilizer for the F-14 in the early 1970's was another major milestone. That was followed by the composite stabilator for the F-15, and composite rudder and stabilizer for the F-16. In the early 1980's, Boeing 767 used nearly two tons of composite materials in its floor beams and all of its control surfaces. The USSR giant transport, Antonov 124, have a total of 5500 kg of composite materials, of which 2500 kg are graphite composites. The all-composite fin box of the Airbus Industrie A310-300 is an impressive structure in its simplicity. Nearly all emerging aircraft have extensive use of composites; examples include the Dassault-Breguet's Rafale, Saab-Scania JAS-39 Gripen, and the European Fighter Aircraft (EFA) of Britain, West Germany, Italy, and Spain.

An all-composite toe plane was built here in Helsinki about ten years ago. The Beech Aircraft's Starship 1 is another all-composite airplane, and is currently undergoing flight test. In 1986, another all-composite airplane that set a world record in nonstop flight around the world is the Voyager designed and built by Burt Rutan and his coworkers. The plane was ultra light as expected. But it also showed amazing toughness and resilience against many stormy encounters. High strength graphite composites are used in the dual rudders of the revolutionary 12-meter yacht, the USA, of the St. Francis Yacht Club's entry to the America's Cup challenge. Both the Voyager and the USA have converted composite materials from a high technology domain into household words. High visibility is an important ingredient for the growth and acceptance of composite materials as viable engineering materials.

There are other unique features offered by composite materials which have no counterpart in metals. Shear coupling, for example, is a built-in characteristic of an off-axis or unbalanced laminate. Aeroelastic tailoring is an application of the shear coupling characteristic such that an airfoil can be tuned to control the bending /twisting ratio. Composites can also possess zero or negative expansion coefficients. The characteristic is utilized to build dimensional stable structures such as microwave antennas.

Materials and processing advances have been instrumental to the growth of our technology. Graphite and aramid fibers became commercially available in the early 1970's. Epoxies are available for various use conditions. More recently, higher temperature matrix materials and thermoplastics have emerged for more demanding applications of the future.

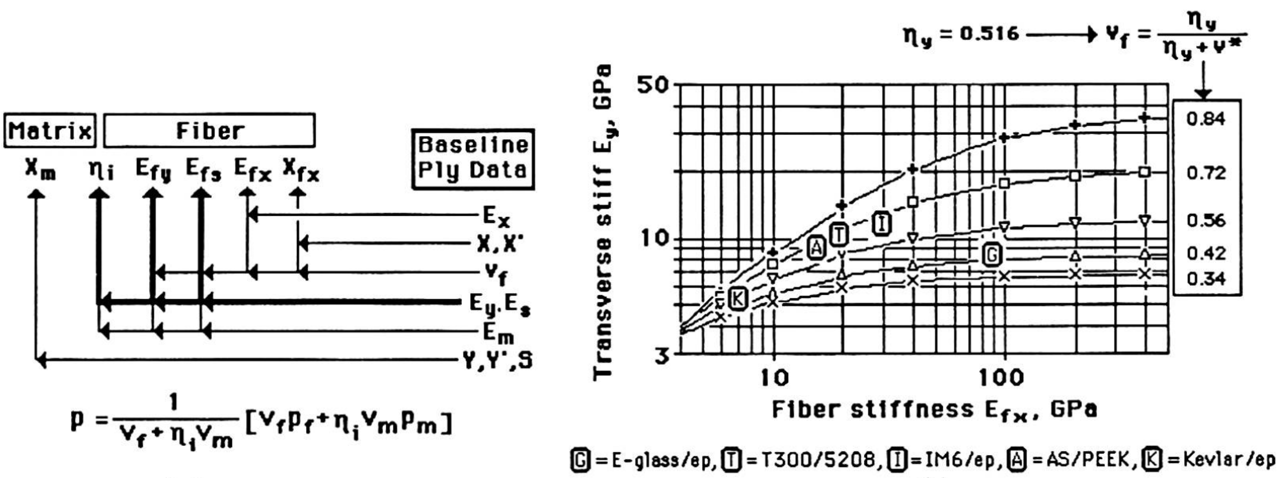
In the mean time, the high technology of composites has spurred applications outside the aerospace industry. The sporting goods is a major outlet of our material. Hundreds of tons of graphite composites were used for tennis and squash rackets, and golf shafts each year since 1983. These rackets and composites are synonymous. Applications in other than boats and rackets include bicycles, oars for rowing, and just about any equipment where weight, stiffness, and strength are important.

Examples of large structures using composite materials are limited at this time. Space structures can be large. So can off-shore drilling platforms where a number of applications are being investigated. The tethers for tension leg platform (TLP), the risers, and the legs of fixed platforms are possible future applications. US Army has built mobile bridges using composite materials.

At a conference held by the Engineering Society of Detroit's conference in December 1985, an automotive industry executive saw the impact of composite materials on the automobile industry to be as great as if not greater than that of electronics. Such high expectation of composites is good for our technology. The acceptance of composites can be greatly enhanced if the cost is lowered; the design, simplified. We would like to address primarily the design issue which is intimately related to the cost.

4. INTEGRATED MICRO-MACROMECHANICS ANALYSIS

Micromechanics is used to relate the properties of fiber and matrix to those of a unidirectional ply. There are many analytic models for this purpose, but we recommend a modified rule-of-mixtures relation shown in the following figures:



(a) (b)

Figure 3 (a) The back-calculated fiber and matrix stiffnesses from the baseline ply data, and the modified rule-of-mixtures formula.
(b) The result of this micromechanics model showing the transverse stiffness as a function of fiber transverse stiffness, volume fraction and the stress-partitioning parameter etc. The position occupied by typical composite materials having epoxy matrices are also shown.

The powerful integrated micro- and macromechanics analysis is achieved by combining the micromechanics, such as the one recommended above, with the classical laminate plate theory, examples for the latter with be shown in the next section. With an integrated micro-macromechanics model, we can show the contribution of the constituent properties to those of the resulting composite material. Thus composite laminates can be optimized from selecting the best combination of the constituents and ply orientations. The additional dimension in materials design is unique with composite materials. The integrated micro-macromechanics model simplifies the design process.

5. STIFFNESS OF SYMMETRIC LAMINATES

The simplest form of multidirectional composites is a symmetric laminates subjected to in-plane and flexural loads. For the in-plane loads case, we usually assume that the strain across the laminate thickness is constant. This is shown in Figure 4(a). The resulting stress in each ply will be piece-wise constant, as shown in Figure 4(b). We then only need to calculate the effective laminate stiffness which is simply the rule-of-mixtures relation or the average stiffness across the laminate thickness. This is shown in Figure 4(c). These effective constants can be used directly in the stress analysis of symmetric laminates subjected to any boundary conditions, just like the case for isotropic plates.

The effective Young's modulus along any axis of a laminate must be systematically calculated using laminate plate theory. As indicated in Figure 4(c) we must take the average of the plane stress modulli [Q] to find the in-plane stiffness [A]*. By matrix inversion we find the compliance [a]*. Then engineering constant such as the Young's modulus along the 1-axis is obtained from the reciprocal of the appropriate compliance component. It is therefore impossible to guess the value of the resulting engineering constant. The parallel spring model does not work.

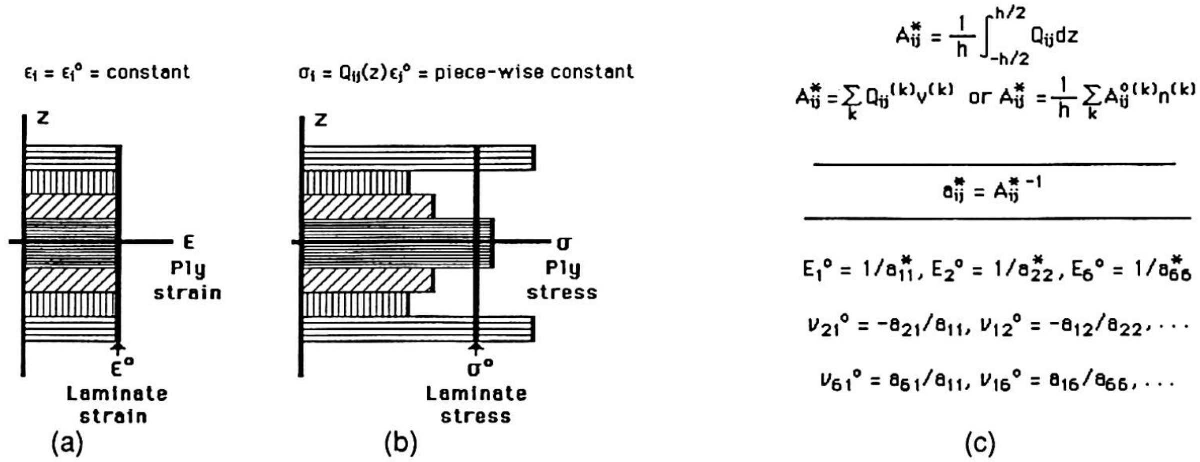


Figure 4 (a) The assumed constant laminate strain across the laminate thickness.
(b) The variation of ply stresses resulting from different ply orientations.
(c) The effective in-plane laminate constants for symmetric laminates.

In Figure 5(a), it is shown that the difference between an off-axis Young's modulus of a unidirectional composite is significantly different from that of an angle ply. Had the parallel spring model worked, the two moduli would have been the same. In Figure 5(b) we compare the effect of angle plies on the effective Young's modulus of various common laminates. In Figure 5(c) we show the effective shear moduli as functions of various laminates.

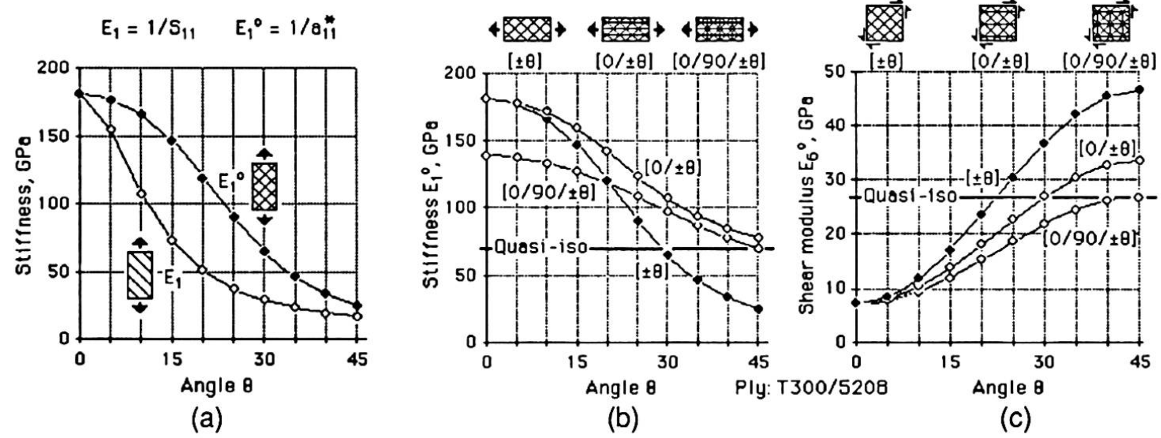


Figure 5 (a) Comparison between the Young's modulus of an off-axis unidirectional and angle-ply laminate.
(b) Young's moduli of various laminates as functions of angle-plies with different angles.
(c) Shear modulli of various laminates as functions of angle-plies with different angles.

In the case of flexural loading of a symmetric laminate, we need to assume that the strain across the thickness is a linear function of the thickness. The flexural stiffness is now a weighted average of the plies. The positions of the plies are now important. Once the flexural stiffness is known, the compliance and associated engineering constants can be derived from the same relations; i.e., the flexural compliance is obtained from the matrix inversion of the stiffness matrix, and engineering constants and coupling ratios are obtained from reciprocals and ratios of the compliance components. These relations are shown in Figure 6(a).

In Figures 6(b) and 6(c), we show the convergence of the flexural stiffness to that of the in-plane when the laminate approaches a homogenized plate with increasing repeating sublaminates. For a

T300/5208 [0/90] cross-ply laminate in Figure 6(b) the two orthogonal stiffnesses are equal. The two corresponding flexural stiffnesses converge toward that value. For laminates where the ratio of the thickness of the two angles are different from unity, say, 2 to 1 as in Figure 6(c), The flexural stiffness converges toward different values as the laminate approaches a fully homogenized state.

$$D^*_{ij} = \frac{12}{h^3}\int_{-h/2}^{h/2} Q_{ij} z^2 dz$$

$$D^*_{ij} = \frac{12}{h^3}\,\frac{1}{3}\sum_k Q_{ij}^{(k)}\left[(z^{(k)})^3-(z^{(k+1)})^3\right]$$

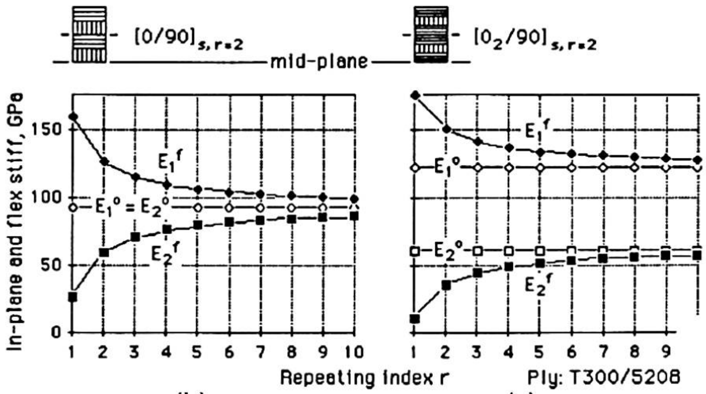
$$d^*_{ij} = D^{*\,-1}_{ij}$$

$$E_1^f = 1/d^*_{11},\ E_2^f = 1/d^*_{22},\ E_6^f = 1/d^*_{66}$$

$$\nu_{21}^f = -d_{21}/d_{11},\ \nu_{12}^f = -d_{12}/d_{22}, \dots$$

$$\nu_{61}^f = d_{61}/d_{11},\ \nu_{16}^f = d_{16}/d_{66}, \dots$$

(a)



(b) (c)

Figure 6 (a) Flexural stiffness, compliance and engineering constants of a symmetric laminate.
(b) Convergence of the Young's modulus in flexure toward that in in-plane of a [0/90] laminate.
(c) Same as (b) for a different cross-ply laminate.

6. FAILURE ENVELOPES

From Table 1 we showed that the strengths of composite materials, like the stiffnesses, are highly directionally dependent. It would be optimistic to compare the longitudinal strength of unidirectional composite with that of an isotropic material unless the composite materials is used to carry uniaxial tensile load. Since composite materials are weak in carrying biaxial loads, laminates of multidirectional plies are made to achieve biaxial strength. Of all possible multidirectional laminates, the quasi-isotropic laminate represents the minimum performance of the composite material. It is therefore a conservative basis for comparing various composite materials as well as with isotropic materials.

In Figure 7(a) we show the relative longitudinal strength of various composite materials with aluminum. This comparison is optimistic. A better comparison is shown in Figure 7(b) where the quasi-isotropic strength is compared with that of aluminum. This comparison is further normalized with respect to the relative specific gravities of aluminum over that of the composite material. Note that many CFRP's have strength advantages over aluminum between 10 to 50 percent.

The ply strengths in the principal stress plane, and the corresponding ultimate strains in the principal strain space can be mapped. All failure criteria envelopes must pass through these four strength points. These points are shown in Figure 7(c), for T300/5208, whose ply properties are listed in the first column of Table 1.

Having those four points, various failure criteria can be drawn through these points to represent the strength of a composite material under the influence of combined stresses.

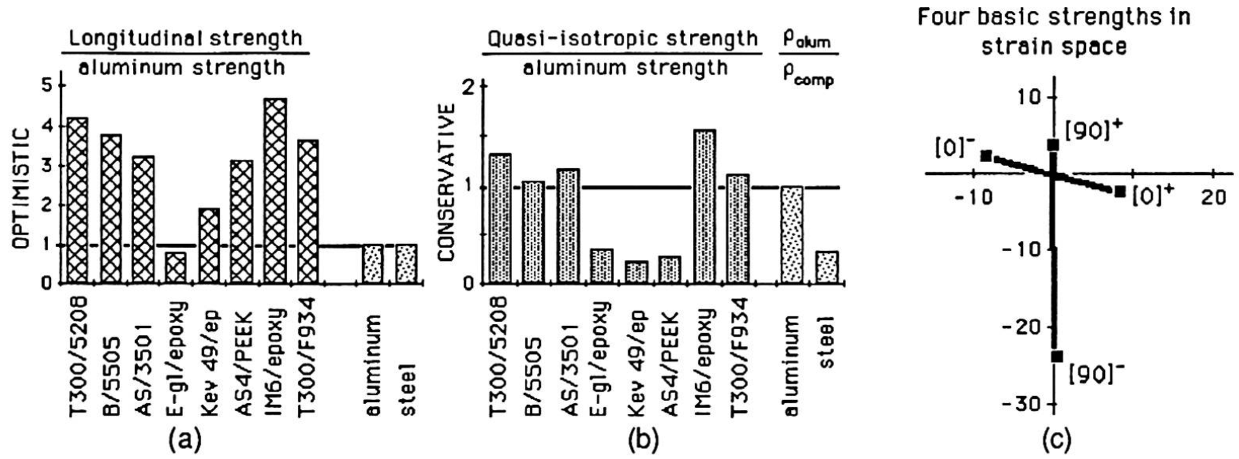


Figure 7 Basic strength comparisons, in (a) and (b), and (c) four basic strengths plotted in strain space.

The failure criterion of isotropic material is most frequently described by the von Mises criterion which is based on a scalar product (a quadratic invariant), and is fully interactive. The same criterion can be shown in strain space. The scalar product or quadratic criteria can be applied to orthotropic material in two as well as three dimensional stresses or strains. For a transversely isotropic material, which is an accurate representation of a unidirectional composite, the resulting criterion is straight forward:

Quadratic criterion in stress space:

$$F_{ij}\sigma_i\sigma_j+F_i\sigma_i = 1 \qquad F_{xx}\sigma_x^2+2F_{xy}\sigma_x\sigma_y+F_{yy}\sigma_y^2+F_{ss}\sigma_s^2+F_x\sigma_x+F_y\sigma_y = 1$$

$$F_{xx}=\frac{1}{XX'},\ F_{yy}=\frac{1}{YY'},\ F_{ss}=\frac{1}{S^2},\ F_x=\frac{1}{X}-\frac{1}{X'},\ F_y=\frac{1}{Y}-\frac{1}{Y'}$$

$$F_{xy} = \text{interaction term} = F_{xy}^*\sqrt{F_{xx}F_{yy}}$$

For closed conic surfaces: $-1 \leq F_{xy}^* \leq 1$

Quadratic criterion in strain space:

$$G_{ij}\epsilon_i\epsilon_j+G_i\epsilon_i = 1 \qquad G_{xx} = F_{xx}Q_{xx}^2+2F_{xy}Q_{xx}Q_{xy}+F_{yy}Q_{xy}^2$$

$$\vdots$$

$$G_y = F_xQ_{xy}+F_yQ_{yy}$$

Figure 8 Formulation of quadratic failure criteria in stress and strain spaces.

In Figure 9 we show quadratic failure envelopes as functions of the normalized interaction term. All envelopes must pass through the four basic uniaxial strengths shown in Figure 7(c). The degree of interaction will determine the best fit envelope among the possible shapes shown in Figure 9. Biaxial test data are required for the determination of this interaction term. A number of experimental methods are available for this purpose.

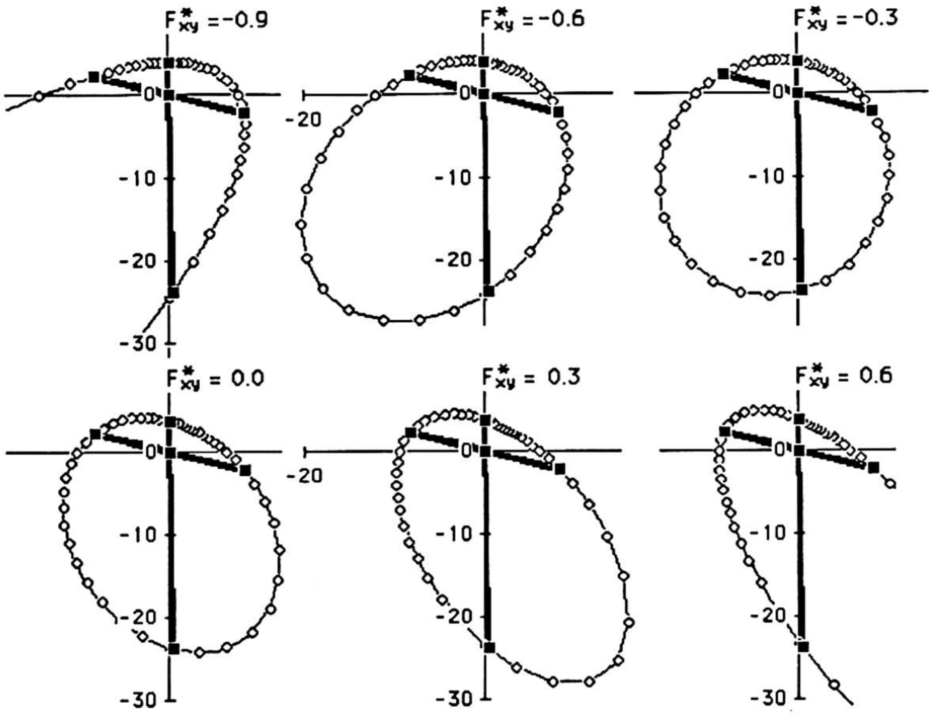


Figure 9 Various possible failure envelopes for T300/5208 as functions of the interaction term.

7. FIRST-PLY FAILURE ENVELOPES

When multidirectional plies are bonded together to form a laminate the effective stiffness of the laminate and the stresses and strains in each ply can be calculated using classical laminated plate theory. From the resulting ply stress or ply strain the failure criterion can be used to determine the strength of the

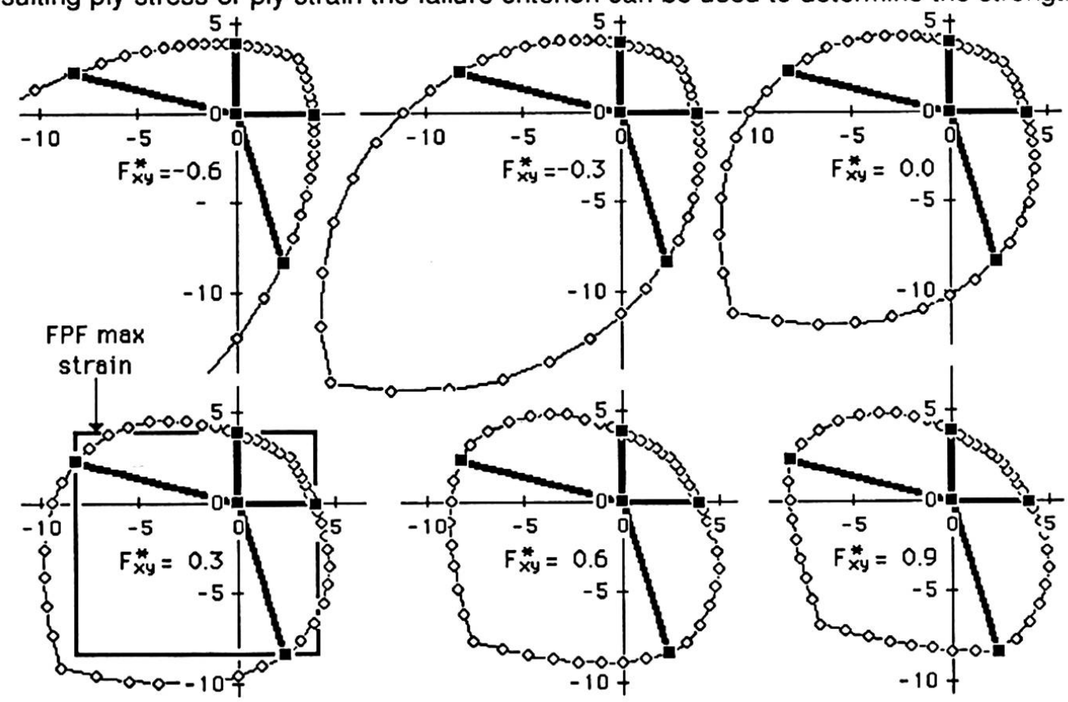


Figure 10 Various first-ply-failure envelopes for T300/5208 as functions of the interaction term.

laminate. Following the pattern of Figure 9 where failure envelopes of a unidirectional composite as functions of the interaction term are plotted, we show in Figure 10 the inner envelopes of a [π/4] laminate:

8. SENSITIVITY OF THE INTERACTION TERM

The effect of the interaction term of the quadratic failure criterion on the failure envelopes of [π/4] laminate is shown in Figure 10 for T300/5308. The range of variation goes from -0.6 to 0.9, at intervals of 0.3.

The four anchor points in the FPF envelopes above are the transverse tensile failure and longitudinal compressive failure for [0] and [90]. The [45] and [-45] do not control in the principal strain plane for this composite material. As a comparison, the FPF based on the maximum strain criterion is simply a box drawn through those four anchor points. A value of 0.3 for the interaction term in the figure above will approximately match the quadratic and maximum strain failure envelopes.

9. LAST-PLY-FAILURE

The use of laminated plate theory beyond the first-ply-failure is not strictly valid because matrix/interface failures in the form of periodically dispersed cracks begin to reach a saturation level. Laminated plate theory is valid for continuous media; i.e., where cracks do not exist. However, the laminated plate theory can be used for predicting the Last-Ply-Failure(LPF) by a simple empirical method as illustrated in the following stress-strain diagram:

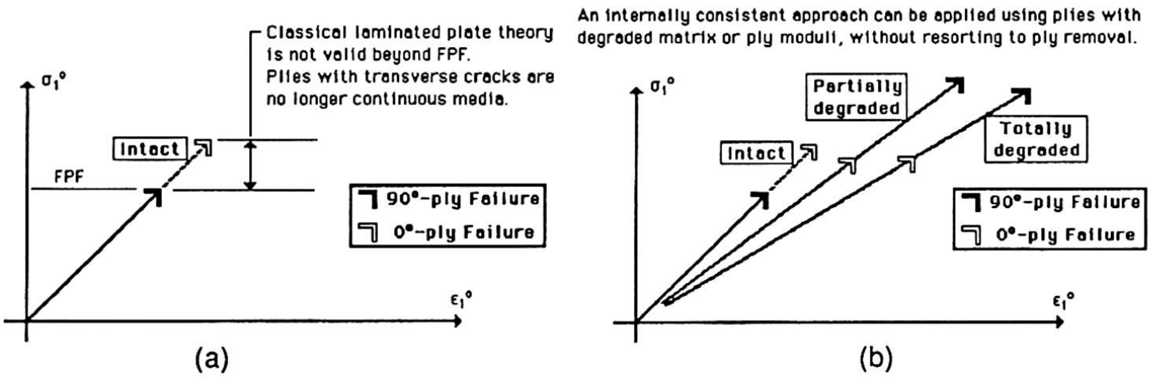


Figure 11 Schematic stress-strain curves of [0/90]s laminate for various degrees of damage. The ply failures for each degree of damage are also indicated.

If an applied uniaxial load is first applied to the [0/90]s laminate beyond the FPF and then is decreased to zero and reloaded, we would expect the stress-strain curve to follow the partially degraded curve with a reduced laminate stiffness because of the cracking of the 90 degree ply as shown in Figure 11(b). The laminated plate theory can be used to predict the macroscopic failure with an degradation of the matrix stiffness for the 90 degree ply. We assume that the ultimate strength or Last-Ply-Failure(LPF) of a laminate is reached when all plies are saturated with transverse cracks, and the prediction of the LPF is

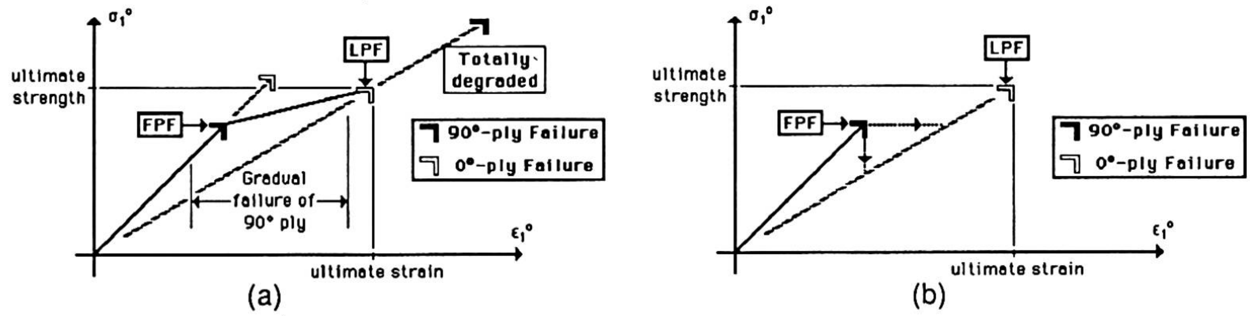


Figure 12 (a) Simplified prediction of the LPF based on the laminate with totally degraded plies.
(b) Loading path beyond the FPF for load and displacement controlled tests.

corresponding to the total degradation of all plies and the ply with the lowest failure load corresponds to the ultimate strength of the laminate. For design purposes, the FPF and LPF are important. Thus, and for convenience of computation, the partially degraded model is ignored. This simplified approach is shown in Figure 12.

The transition between the FPF for the intact plies to the eventual LPF along the degraded plies depends on many factors. The path can be idealized as a horizontal line if the loading machine is load controlled. If the machine is displacement controlled, the path can be idealized as a sudden drop in the load, followed by increase along the stiffness of degraded plies, as shown in Figure 12(b). In actual testing, the transition is smooth because the stiffness between the intact and degraded plies is small. For example, for practical laminates, the loss of laminate stiffness due to total matrix degradation is less than 10 percent. We therefore recommend a smooth transition shown in Figure 12(a). If the applied load increases monotonically, the stress-strain curve is expected to follow the intact line, then deviate from the FPF point to the LPF point on the totally degraded line. Thus, the key for the determination of the last-ply-failure is the degradation factor.

10. DEGRADATION FACTOR

It is simple to introduce a method of describing plies with cracks by using continuous plies with lower matrix stiffness. This is shown in the following:

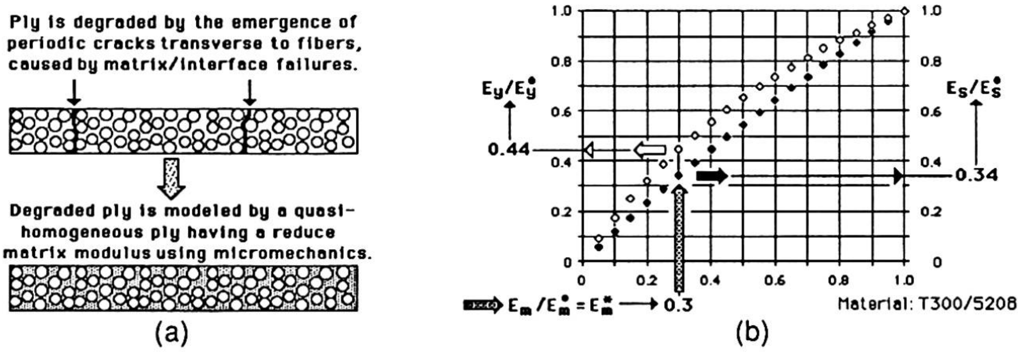
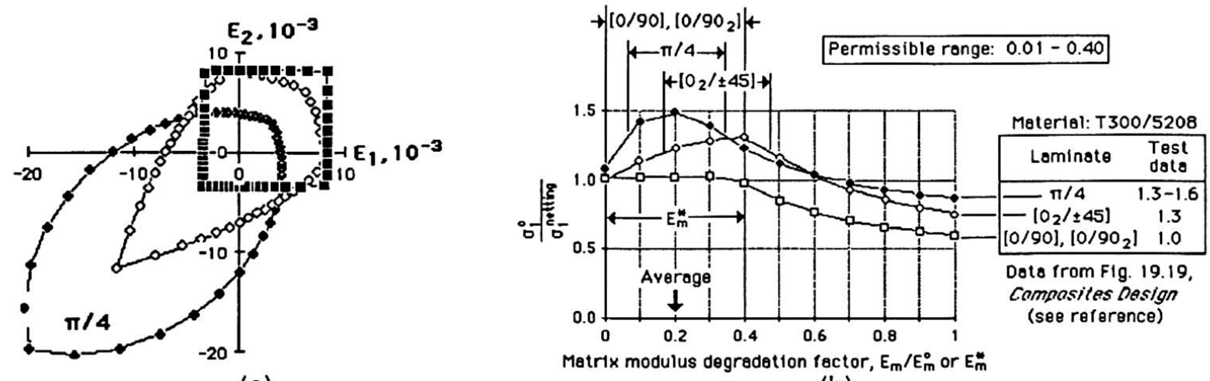


Figure 13 (a) Replacement of degraded plies by quasi-homogeneous plies.
(b) Relative reduction in transverse and shear moduli due to reduction in matrix stiffness.

The cracked plies are replaced with a continuum of quasi-homogeneous plies of lower stiffness so that the conventional stress analysis can be applied. Using a simplified micromechanics formula, based on the stress partitioning parameters, we can show the reduced transverse and shear stiffnesses resulting from a reduced matrix modulus. In the above Figure 13(b), as an example for T300/5208, for a fractional degradation of matrix stiffness to 0.3 the corresponding reduction in transverse stiffness and shear stiffness are to 44 and 34 percent respectively.

In summary, the FPF and LPF are related by the matrix degradation factor; the FPF is for the intact ply or when the degradation factor is unity. When the degradation factor decreases the shape of the failure envelope changes. Failure envelopes and failure strengths of [π/4] laminates with various degree of degradation are shown in Figure 14(a):

In Figure 14(a) the failure envelopes are obtained using the quadratic failure criterion as discussed in section 6. It is shown in this figure that when the value of the matrix degradation factor approaches to zero, the failure envelope converges to that of the maximum strain or netting analysis. Thus in the limiting case the results of maximum strain or netting analysis can be recovered from the quadratic criterion.



(a) (b)

Figure 14 (a) Failure envelopes of [π/4] laminate for several degradation factor, solid diamond for E_m^*=1.0, intermediate for E_m^*=0.3, and rectangle is the netting analysis with E_m^*=0.001.
(b) Normalized tensile failure strength of several laminates versus matrix degradation factor. Comparison with test data are also shown.

11. DETERMINATION OF DEGRADATION FACTOR

The degradation factor can be determined from test data obtained from laminates. In Figure 14(b) the tensile strength calculated based on quadratic criterion for various values of the degradation factor is normalized by that of netting analysis. The tensile strength prediction by netting analysis is the product of ply tensile longitudinal strength and the fraction of [0] plies in the laminate. In the limit when the degradation factor approaches zero, the netting analysis prediction, as shown above, should prevail. For most composite materials, such extent of degradation is not likely to occur. In this figure, test data from various laminates are used to select the most probable value for the degradation factor. Assuming that the ultimate tensile strength of a laminate is equal or higher than than that of the netting analysis and comparing the test data, Figure 14(b) suggests that, for T300/5208, a value between 0.01 and 0.40 for the matrix degradation factor is permissible. An exact value is not critical for the prediction of the LPF envelope. A summary of the values of the degradation factors for typical composite materials that can be used in predicting the LPF is shown as follows:

Table 2 Degradation of stiffness and strength based on matrix degradation.

Degrade	T300/52	IM6/epoxy	AS4/PEEK	E-gl/epoxy	Kev/epoxy
E_m	3.40	3.40	3.40	3.40	3.40
E_m^*	0.20	0.10	0.10	0.07	0.02
E_y	10.30	11.20	8.90	8.27	5.50
E_y^*	0.31	0.14	0.17	0.07	0.05
E_s	7.17	8.40	5.10	4.14	2.30
E_s^*	0.27	0.11	0.16	0.08	0.06
X'	1500	1540	1100	610	235
X'*	0.77	0.64	0.70	0.60	0.57
ν_x^*	0.20	0.10	0.10	0.07	0.02
$(F_{xy}^*)^*$	0.20	0.10	0.10	0.07	0.02

* means fractional degradation from intact to degraded matrix.
Longitudinal compressive strength are degraded, not linearly, but to the 0.2th power; i.e., $X'^* = [E_s^*]^{0.2}$.
Poisson's ratio and interaction term in the quadratic failure criterion are linearly degraged with the matrix stiffness.

12. RULE-BASED LIMIT AND ULTIMATE STRENGTHS

Using the degradation factor, the FPF and LPF envelopes can be obtained in a consistent manner. For T300/5208, with a factor of 0.3 for the LPF, we can show these two envelopes:

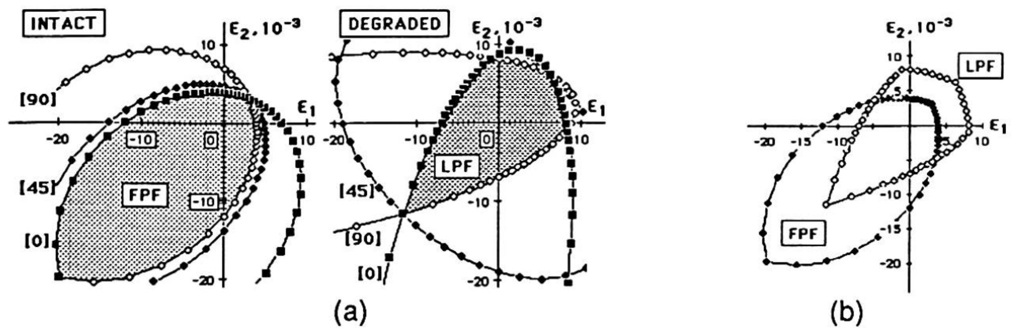


Figure 15 (a) Failure envelopes of a [π/4] laminate before and after degradation.
(b) FPF and LPF envelopes together after superposition.

Now we can establish a rule-based design criterion. These rules depend on the relative values of FPF, LPF, and the desired factor of safety. We would like to recommend the following rules:

Rule 1: ultimate strength = MAX(FPF, LPF)

Rule 2: limit* strength = ultimate/safety factor

Rule 3: limit strength = MIN(FPF, limit*)

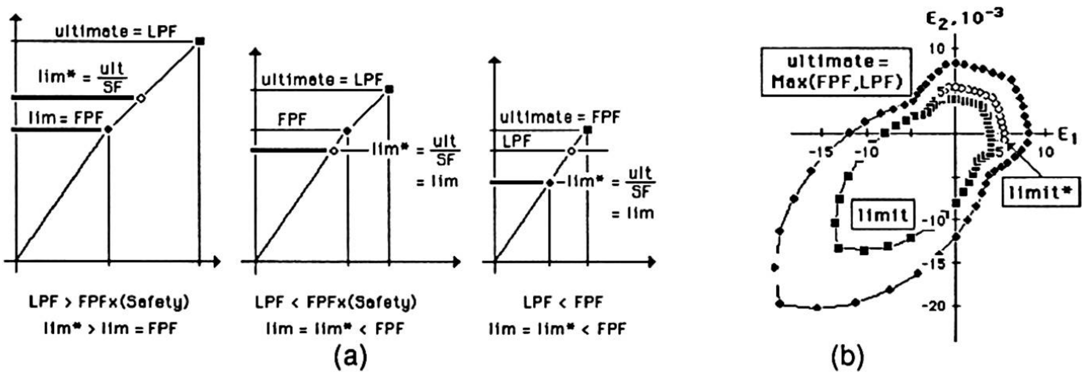


Figure 16 (a) Illustration of ultimate, limit*, and limit strengths.
(b) Ultimate, limit*, and limit failure envelopes for T300/5208 with a safety factor of 1.5.

Limit* is an ultimate based strength where matrix/interfacial cracks are tolerated; e.g., in filament-wound pressure vessels. Limit strength is more conservative where stresses beyond FPF are not permitted. However, at the same time, the limit strength takes full account of post-FPF capability to provide the maximum desired safety. These rules are shown in the Figure 16(a) above. With these rules, the ultimate, limit* and limit envelopes, with a safety factor of 1.5, are shown for T300/5208 in Figure 16(b).

13. SIMPLE DESIGN METHODS

The above mentioned design rules can easily be implemented in any design method. We would like to discuss, for example, two such simple design methods that are being used by engineers in sizing composite structural components.

13.1 Mic-Mac Spreadsheet

This is an integrated micro-macromechanics analysis, or Mic-Mac for short, which can be best performed using one of several spreadsheets now available in all personal computers. The example presented below is based on Microsoft Excel for the Apple Macintosh computer, which can be transferred to Lotus 123 for the IBM PC computer.

stacking	READ ME	Theta 1	Theta 2	Theta 3	Theta 4		Ply mat	T3/N52[SI]	
	[ply angle]	**0.0**	**90.0**	**45.0**	**-45.0**	[repeat]	h, *	h, E-3	[Rotate]
	[ply *]	**1.0**	**1.0**	**1.0**	**1.0**	**1.0**	8.0	1.0	**0.00**
strength ratios	R/intact	0.598	0.325	0.383	0.383	R/FPF	0.325	safety	**1.50**
	R/degrade	0.559	0.618	0.974	0.974	R/LPF	0.559	R/lim*	0.373
						R/ult	0.559	R/lim	0.325
loads & laminate moduli	{N}, MN/m or k/in		{N}lim	{N}lim*	{N}ult	{E°}lim	E^ul/E^lr	<alph>E-6	<beta>
	compon't 1	**1.00**	0.325	0.373	0.559	69.68	0.91	1.52	0.0401
	compon't 2	**0.00**	0.000	0.000	0.000	69.68	0.91	1.52	0.0401
	compon't 6	**0.00**	0.000	0.000	0.000	26.88	0.89	0.00	0.0000
stresses & strains at limit, et al.		<sg°>	<sg°>lim	<sg°>lim	<sg°>ult	<ep°>E-3	<ep°>lim	<ep°>lim*	<ep°>ult
	compon't 1	1000	325	373	559	14.35	4.67	5.91	8.87
	compon't 2	0	0	0	0	-4.25	-1.38	-1.87	-2.81
	compon't 6	0	0	0	0	0.00	0.00	0.00	0.00
micromech variables		T opr	c, moist	vol/f	Em	Efx	Xm	Xfx	Em/Em°
	Baseline	22.0	0.005	0.70	3.40	259	40.0	2143	0.20
	[Modified]	**22.0**	**0.005**	**0.70**	**3.40**	**259**	**40.0**	**2143**	**0.20**
	Mod/Base	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

Figure 17 Example of Mic-Mac spreadsheet for designing composite structural components.

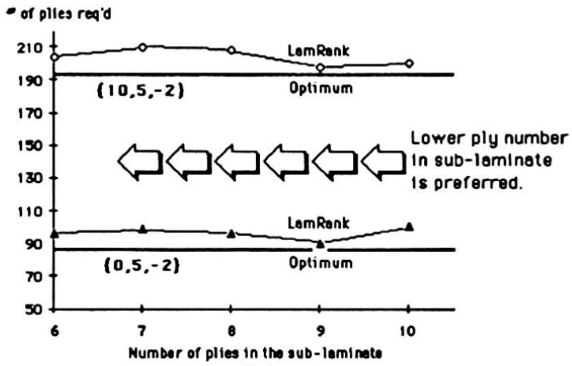
The eminent feature of the Mic-Mac spreadsheets is that all the micromechanics and macromechanics variables are integrated together by the interlinked cells of the spreadsheets. Thus designers can immediately see the effect of any micromechanics variable on the final design and this tool is extremely useful for the sensitivity study of any design variable.

13.2 Design by Ranking

Another method of design can be based on ranking of a family or families of sublaminates. The use of sublaminates in design has two major advantages. One, splicing of the total laminate with a number of sublaminates reduces manufacturing cost and is less prone to make mistake in lamination. Two, instead of having all the plies of same angles stacked up together, if the laminate is made up with sublaminates, the interlaminar stresses can be reduced and consequently the laminate will be stronger against delamination.

Number of plies	2 orientations	3 orientations	4 orientations
2	3	6	10
3	4	10	20
4	5	15	35
5	6	21	56
6	7	28	84
7	8	36	120
8	9	45	165
9	10	55	220
10	11	66	286
Total	65	285	1000

(a)



(b)

Figure 18 (a) Number of sublaminates in a family for given number of ply orientation and plies.
(b) Influence of number of plies in a sublaminate on the design thickness of the laminate for two load cases.

A large number of sublaminates can be ranked in optimizing a laminate. The number of sublaminates in a laminate family is a function of the number of plies and number of orientations in a sublaminate. Figure 18(a) shows the number of possible sublaminates in a family of given number of orientations (between 2 and 4) and total number of plies (between 2 and 10). It is shown in Figure 18(b) that, for six or more number of plies in a sublaminate, the laminate ranking method can yield virtually an optimum laminate.

14. CONCLUSIONS

We have shown an approach to extend the common failure criteria from the FPF to LPF using a matrix degradation factor. Then rules for design can be based on limit* or limit, using any value of safety factor. This approach can be illustrated in the following flow diagram:

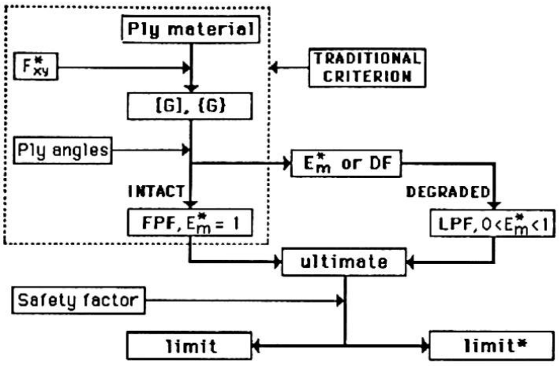


Figure 19 Flow chart of a laminate design.

While we recommend the quadratic criterion, but the approach outlined here can be applied to any other failure criteria. The only change necessary will be replacing the [G] and {G}, the strength parameters in strain space by whatever appropriate parameters corresponding to the chosen failure criterion.

The degradation factor proposed here is preferred over the ply removal method. It is also preferred over complete degradation; i.e., assigning a value approaching zero. The stiffness of a degraded structure can also be used to estimate the degree of degradation.

We believe the approach that we propose is simple, easy to understand, and easy to compute. It is internally consistent, and extend the utility of laminated plate theory for the prediction of the LPF of a laminate. This approach is better than using two unrelated models; e.g., laminated plate theory for FPF, and netting analysis for the burst pressure. Much work remains to be done for the proposed approach.

The experimental data are always difficult to assemble. But with a framework such as ours, we believe that the critical data can be obtained systematically.

REFERENCE

Stephen W. Tsai, *Composites Design*, 3rd ed, Think Composites, Dayton (1987).

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Advances Made in Materials Related to Concrete and Applications

Progrès dans les matériaux proches du béton et applications

Fortschritte in der Anwendung von beton-orientierten Zusatzwerkstoffen

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SUMMARY
This paper is focused on materials related to concrete and briefly describes the development and application of ultra-high strength concrete, reinforcing fibers, new admixtures and high performance reinforcing bars. And, further, introductory description on polymer-concrete composites is presented. Problems requiring further study is also discussed.

RÉSUMÉ
L'article se concentre sur les matériaux proches du béton et décrit brièvement le développement et l'application de bétons à résistance extrêmement élevée, de fibres de renforcement, de nouveaux additifs, d'armatures de renforcement à haute performance. Une introduction est présentée sur les matériaux composites béton-polymère. Des domaines sont indiqués pour des recherches futures.

ZUSAMMENFASSUNG
Dieser Beitrag behandelt beton-orientierte Zusatzwerkstoffe, die Entwicklung und Verwendung von Beton höchster Festigkeit, Bewehrungsfasern, neuartige Zusatzmittel, Stähle hoher Qualität und Polymerbeton. Abschliessend werden die Probleme für weitere Forschungsarbeiten besprochen.

1. Introduction

A review of the development of structural engineering in modern times will show that the advances made in design and construction of structures have been in step with improvement of existing materials and creation of new materials. The expectations gathered on new materials in today's age of technological revolution are extremely great. Especially, at present, in large-scale technological development such as nuclear fusion and space exploration, and spheres closely related to our livelihood such as microelectronics and medicine, new materials comprise keys to making progress in revolutionizing technologies, and it is said that one who controls materials controls technology. Although the revolutionizing of construction and structural technology is not as spectacular as in the beforementioned fields, the technological background regarding the necessity for developing new materials is the same.

Conventional materials used in structural engineering have mainly been steel, concrete, and on a smaller scale, wood. A new material may be defined as "a material manufactured under close control to draw out a new function not possessed by a conventional material or to greatly improve performance." Of course, new materials will include composite materials that satisfy the demands for more complex, higher-level items that are "lighter, yet stronger," or "more ductile and durable," through combinations of the merits of conventional materials. It should be noted further that for these new matrials to be used practically as supports for advanced technology, it will be necessary to aggressively develop processes for lowering costs and to make strong efforts for promoting practical use.

Development and application of new materials concentrating on cement concrete-related items as sturctural materials will be briefly discussed in theis paper.

2. Improving Performance of Concrete

Concrete is one of the oldest artificial materials, is used in a wide range of forms, and today, is still one of the most important construction materials. It is thought that this composite material based on cement will contunue to retain its position as the greatest construction materiral for at least the next half-century while appearing in various new combinations and forms. The features of this mateiral are that its raw materials are abundant and readily available almost everywhere, a little energy is required for manufacturing so that the cost is low compared with other materials, and in additon to such advantages, it has the properties of stability and durability when used as a construction material in various kinds of evironments. However, it is clear as to the properties of concrete that there remains still room left for improvement and higher performance as a construction material, and therefore, besides its use as a general purpose material, concrete is being developed as a material adjusted to be usable under special conditions also.

2.1 High-strength Concrete

With the advances made in concrete technology it has become possible for high-strength concrete of compressive strength about 70 to 80 MPa to be produced with comparatively ease. Development of high strength concrete with years is, for instance, as shown in Table 1. This is owed largely to the development of good-quality, high-range water-reducing admixtures, while it is being called for to further develop technology making possible attainment of high strength over around 130MPa without special materials or costs [1] [2]. These high-strength concretes have been put to practical use in individual members and precast products such as piles, but for such concretes to be applied actural cast-in-situ work there must be improvements made in placement techniques and build-up of quality control systems along with progress in concrete manufac-

Table 1 Increase in strength of concrete with years (MPa)

	Normal	High	Ultra-high
1950'S	<30	>30	-
1960'S	<30	30∿50	-
1970'S	30∿50	50∿100	-
1980'S	30∿50	50∿100	>100

Table 2 Concrete for CONDEEP SP Gullfaks C

Cement (SP30-4A mod.)	430kg
Silica fume	20kg
Sand (0∿5mm)	920kg
Coarse aggregate (5∿20mm)	860kg
Water	1651
Superplasticizer	61
Slump	240mm
W/C	038
Mean 28-d cube strength	83MPa
Standard deviation	5.4MPa

turing techniques.

With 1970 as a turing point, there has been much demand for concrete of high strength that can be obtainded by ordinary methods without using special materials such as resins (polymers) or special technologies, and in the Chicago area of the United States, close to 20 buildings have been built since 1972 using concrete columns of compressive strength of 62 MPa as shown before in Table 1. Following this, there have been similar cases in various regions, and according to a recent report, a building using concrete of 131 MPa (age, 56 days) has appeared. [3] In Japan, too, where considerable earthquake resistance is demanded, there are buildings (apartment houses) up to 40 stories including those presently under construction which have employed concrete of compressive strength of 50 to 60 MPa and slump of around 180 mm. There are also many reports of use in members of bridges and other structures, a prominent example being the use in large quantity of high-strength concrete for construction of oil exploration rigs for North Sea oilfield development in Norway. With improvements made in the qualities of materials such as cement, aggregates, and admixtures, concrete of 28-day cube characteristic strength 75 MPa (w/c =0.38, slump = 240 mm, standard deviation 5.4 MPa) is presently being made and placed by pump at the Gullfaks C rig as giving in Table 2. [4] High-strength concrete is usually made with high cement content, low water-cement ratio, ordinary aggregates, chemical admixtures, and pozzolanic admixtures. The mix proportions differ greatly depending on the required strength, age, material characteristics, and place of application. Although there are essentially no fundamental differences from concrete of normal strength with repard to batching, mixing, conveying, placing, and controlling techniques, special consideration is necessary for securing uniform-quality materials to obtain high strength.

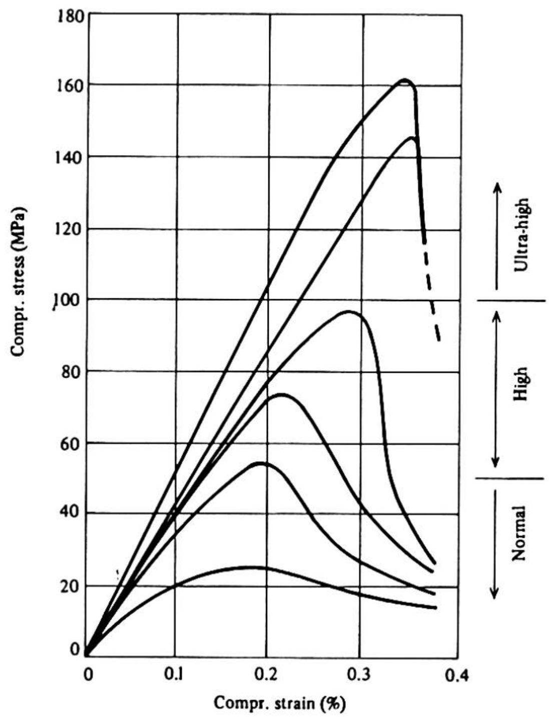

Fig. 1 Stress-strain curves for concretes of various strength

Considering the fact the behavior of high-strength concrete under loading is of slightly brittle nature, as shown in Fig. 1 reexamination is required concerning cross-sectional design, but there are substantial improvements, for example, with regard to reduction in creep and shrinkage, increase in density, etc. [5] It is clearly known, therefore, that the utilization of high-strength concrete to columns which are subjected to vertical forces is quite advantageous economically with respect to reduction in member cross section, early removal of forms, and securing of durability. However, with regard to utilization in beams, it may be said there is room left for study on such as the minimum quantity of tensile reinforcement, the necessity for lateral confinement, etc. from the standpoint of assuring toughness. Furthermore, besides problematic points in construction such as the influence of material characteristics, consolidation and mixing procedures, is is emphasized that there is a necessity for still more study on subjects such as properties of lightweight high-strength concrete and high-strength fiber-reinforced concrete, and application of high-strength concrete in earthquake zones.

2.2 Silica Fume Concrete

A substance that is being watched with extremely close attention as a material in making high-strength concrete is silica fume (micro silica). This is a by-product obtained in manufacturing silicon and ferrosilicon alloys and average particle diameters are from 0.1 to 0.2 microns (Specific surface of the order of 20,000 m^2/kg). It is an amorphous glassy meterial with Sio_2 content generally over 90 percent. Use in concrete was tried for the first time in the 1950s in Norway, and spread rapidly in various parts of the world from 1970. The objective at the beginnig was to reduce cement content in obtaining identical strength, but it may be said that at present the main purpose of using silica fume is the manufacture of concrete possessing high strength and high durability. [6] The main producing countries of silica fume are the U.S.A., the U.S.S.R., Norway, and Canada. The annual production in Japan is around 26,000 Mg (1984).

Since silica fume is non-crystalline and, moreover, ultra-finely-divided, the hydration activity is great and pozzolanic action occurs at an early stage when mixed with cement. Consequently, concrete using this material has good strength gain at the initial stage, and that compressive strength is proportional to the binder-water ratio is the same as with ordinary concrete. Strength gain can be explained by both pozzolanic reaction and by microfiller effect. An attempt has been made using efficiency factor K to show how many times more effect silica fume has over the same quantity of cement. In essence, with W as unit water content, C as unit cement content, Si as unit silica fume content, and K as efficiency factor, water-binder ratio can be expressed as $W/(C + KSi)$. Examples of the values of K obtained for various kinds of strength are given below in Table 3. [7]

Table 3 Values of K

Curing	Compressive strength	Tensile strength	Flexural strength
Moist curing	2 - 5	3 - 8	4 - 16
Dry curing	2 - 5	0 - 3	0 - 3

The amount of silica fume used is normally between 5 and 10 percent (volumetric ratio) of the cement content, but since it is a finely-divided powder, there is a tendency for water demand to be increased as the addition to concrete is increased, and because of this, it is necessary to use a high-range water-reducing agent to prevent decrease in strength and increase in drying shrinkage. Bleeding of fresh concrete is also greatly reduced. The structural system of silica fume concrete is close-textured and pore diameters are extremely small compared with concrete in general so that there is a trend for resistance to freezing and thawing to be increased, but very little confirmation of this has been done experimentally, and it is thought the effect of improvement by air entrainment cannot be ignored. Since silica fume is an extremely fine powder, tanks, special trailers, special container bags and the like are used in transportation, while in consideration of surer handling and mixing, the material is made into the form of granules or slurry.

It is thought silica fume will attract even more attention in the future because of its special performances as an admixture. However the drawback of this material is that the amount of production is too limited to be used widely.

2.3 Fiber Reinforced Concrete

Attempts to strengthen cement mortar and concrete using various types of fibers have been made from a long time ago to improve low tensile strengths and brittle natures which are major drawbacks of these materials, and asbestos-cement products and reinforced concrete are typical examples. Recently, fiber reinforced concrete (hereafter "FRC) using steel bifers, alkali-resistant glass fibers, vinylon fibers, carbon fibers, aramid fibers, etc. have been finding practical use. The properties of various fibers for FRC are given in Table 4.

Table 4 Various fibers used for reinforcing cement concrete :

Kind of Fiber		Tensile str. MPa	Mod. of Elasticity GPa	Specific wt
steel	carbon	500∼1000	195∼200	7.8
	zinc-galvanized	500∼1000	195∼200	7.8
	stainless	500∼1000	195∼200	7.8
alkali-resistant glass		2500	75	2.8
Carbon	Low elast. (pitch-based)	800∼1100	43	1.63
	High elast. (Pan-type)	2000∼3000	200∼400	1.7∼1.9
asbestos		560∼ 980	84∼140	2.9
aramid		3100	75	1.39
vinylon		900∼1500	31∼ 37	1.29
polypropyrene		550∼ 760	3.5	1.5
concrete		0.5∼2.5	10∼ 30	1.0∼2.3

The forms of reinforcement by fibers are mainly with short fibers for three-dimensional random reinforcement and two-dimensional random reinforcement by direct spraying. Recently, however, there have been research and development works going on where continuous fibers are solidified into deformed bars using various kinds of resin, with these bars employed for uniaxial reinforcement, biaxial reinforcement in mesh or net form, and triaxial reinforcement in truss or lattice form.

FRC with short fibers dispersed randomly in all three dimensions within the cement matrix will have a significantly lower reinforcing efficiency of the fibers mixed in compared with the cases of reinforcement by uniaxial continuous fibers and by short fibers randomly dispersed in two dimensions due to the effects of alignment of the short fibers and the length of fibers (effect of adhesion strength of fibers and matrix).

The present situation is that mass-production techniques for reinforced concrete members using continuous fibers have not yet been established, while two-dimensional random reinforcement by short fibers is limited by the method of spraying. In contrast, when short fibers can be dispersed three-dimensionally in random fashion inside a mixer and the mixture cast and molded in forms in the same way as conventional concrete, the lowness of reinforcing efficiency of the fibers can often be more than offset when the improvement in productivity and other improvements in physical characteristics due to reinforcement are considered. It should be noted here that with the current mixing techniques using mixers, approximately 5 percent (by volume) is the limit to addition of short fibers in cement matrices, workability being impaired extremely and scatter in strength sharply increased at higher rates of addition.

Steel Fiber Reinforced Concrete

Steel fiber reinforced concrete (SFRC) was first tested in the United States in 1971, with the first large-scale application made in 1973. States of the art and problematic points have been widely reported [8] and, therefore, details will be omitted here. Maximum-size aggregates of 10 to 20 mm are generally used for SFRC, and from the standpoint of workability, mix designs are fairly rich with high sand-aggregate ratio. It has been considered that when steel fibers are added to concretes containing an aggregate with exceeding a maximum-size of 20 to 25 mm, the performances of the resulting concretes would not be favorable. However, examination of the results with smaller-sized aggregates shows that because of high cement content there are drawbacks such as large volume change due to heat evolution and the tendency for cracks to occur readily when cross sections are thick due to shrinkage of cement paste. As a consequence, there are studies being made, in which rather than aiming to reduce unit cement content, while giving consideration to workability, maximum size of aggregate is increased to about 40 mm, aggregate quantity is increased, and in accordance, long steel fibers (made longer than maximum aggregate size to, for example, 50 to 60 mm) are added. Using such fibers, which are hooked or corrugated, concretes having better impact strengths and fatigue strengths have been obtained compared with conventional streight fibers (length, 19 mm). [9]

Glass Fiber Reinforced Concrete

Details concerning glass fiber reinforced concrete (GFRC) will also be omitted. Through development of alkali-resistant glass fibers started around 1968 in the United Kingdom, use of these fibers has gradually increased, and they are being utilized in various construction materials such as curtain walls and panels, but the alkali resistance of the glass fibers is still not perfect, and this has been an important research topic. To cope with this, it has been attempted to lower the degree of alkalinity in concrete using admixtures such as silica fume and blended cement, while cement for GFRC (calcium silicate-Hauyne-slag-based cement), which does not produce $Ca(OH)_2$ during the process of cement hydration and has very low OH^- ion concentration in the hardened system, has been developed, and good test results have been reported to draw attention. The components in comparison with ordinary cement are shown in Table 5. [10]

Table 5 Chemical composition of GFRC cement

Cement	SiO_2	Al_2O_3	Fe_2O_3	CaO	SO_3
GFRC cement	23.2	13.8	1.1	47.5	9.3
Ordinary portland cement	22.2	5.2	3.1	64.8	2.0

Carbon Fiber Reinforced Concrete

Carbon fibers for reinforcing cement concrete may be classified into two types-one is pitch-based fibers made from pitch and the other is pan-type fibers made from acrylic fibers. Research on carbon fiber reinforced concrete (CFRC) was first started by M. A. Ali, J. A. Waller and others in the 1970s, and in these studies, pan-type fibers were used, mainly aligned unidirectionally, by which it was shown that high tensile and flexural strengths were obtained in proportion to the fiber content (for example, tensile strength of 50 MPa and flexural strength of 100 MPa with content of 5 percent). [11] However, pan-type fibers are expensive and there have been few cases of practical use. In contrast, research has been done in Japan for development and application of low-cost pitch-based fibers. Successful use in large quantities for highe-strength, high-quality, lightweight cladding tile panels and curtain walls made by mixing short fibers 3 to 10 mm in length and 15 to 20 μm in diameter mixed in mortar at rates of 2 to 4 percent (volumetric) and cured by autoclaving has been reported. For example, in the ARK high rise office building (37 stories) in Tokyo, 5540 lightweight CFRC curtain wall of 1.47 m width, 3.76 m height, and 1 Mg mass were manufactured, in which about 170 Mg pitch-based carbon fiber was used. The tensile stress-strain curves for normal CFRS is, for example, shown as in Fig. 2 [12]

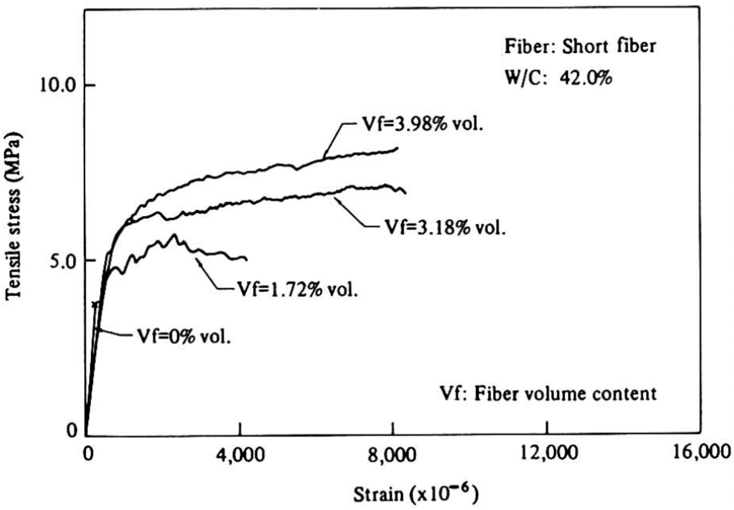

Fig. 2 Tensile stress-strain curves for CFRC.

Aramid Fiber

Aramid fibers possess physical properties intermediate between pitch-based and pan-type carbon fibers. The flexural strength of aramid fiber reinforced mortar is approximately the same as with carbon fibers. This fiber is lower in heat resistance compared with carbon fibers.

Vinylon Fiber

Vinylon fibers indicate physical properties similar to those of pitch-based carbon fibers, but have good bond with cement matrix, and are highly resistant to alkali. The flexural strength of vinylon fiber reinforced mortar is almost the same as that with pitch-based carbon fibers.

3. Development of New Reinforcing Bars

In recent years, opportunities for using reinforcing bars of higher strengths and larger diameters have increased as reinforced concrete structures of larger scale and higher stories have come to be designed and constructed. This topic will be considered below together with the development of salt-resistant reinforcing bars required for improving the durability of reinforced concrete structures.

3.1 High-strength Reinforcing Steel

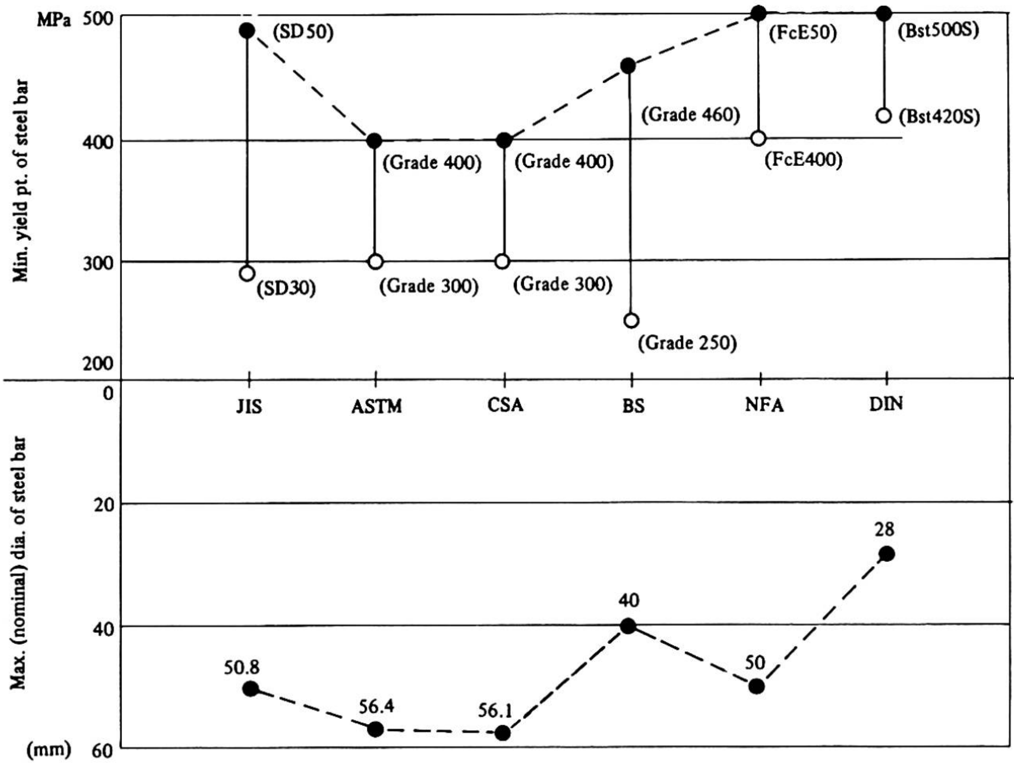

Fig. 3 Min.yield pt. and max. nominal dia. of steel bars specified in various countries.

The limits of minimum yield points of various types of reinforceing bars and upper limits of bar diameters standardized at present in various contries are shown in Fig. 3. [13] The limits to high-strength of reinforcing bars in Japan, Germany, France, and the United Kingdom are around σ_y = 50 MPa, while in the United States and Canada they are lower at around 400 MPa.

In Japan, although there is a standard for SD 50 (σ_y = 490 MPa), there have been almost no cases of actual use up to now, and practical studies have been with small-diameter bars (diameter: 10 to 16 mm), while concrete strengths have mostly been in a range of 20 to 30 MPa.

There is a limit to high strengh of reinforcing steel that can be effectively utilized in reinforced concrete members. Factors which may be considered to affect the upper limits are the following:

1) Ultimate flexural strengths of beam and column cross sections
2) Flexural ductilities of beam and column cross sections

3) Shear Strengths of beam and column members
4) Bond strength, etc.

In case of using high-strengh reinforcing bars in beams, there may be a limit to the bar stress for the most effective use from the standpoints of limitations in crack width at the serviceability limit state and/or the fatique limit state. In contrast, in application to a column in which the action of axial compressive force is predominant, the problems of cracking limit due to bending moment and fatigue of reinforcing bar due to repetitions of live load are decreased. As a consequence, a large reduction in column cross section becomes possible through the combination of high-strength concrete and high-strength steel of large diameter, and an economic advantage can also be adequately secured. According to recent studies on the use of SD 50 it is shown that the more varieties of designs can be made the higher the concrete strength, and that if adequate shear reinforcement is provided sufficient flexural ductility can be secured for earthquake resistance even when SD 50 is used. Studies also indicate that, sufficient safety can be secured on bond strength so long as excessively large-diameter bars are not used, and moreover, in case cover is made thick correspondingly to bar diameter. In this way, reduction in reinforcing steel quantity and consequential ease of reinforcing bar placement are made possible through the use of SD 50, while for the same flexural strength, reduction in member cross section becomes possible when an equal quantity to low-strength reinforcing bars is used. On the other hand, a kind of deformed prestressing bar produced by induction heat heat treatment of dia. 6.4 to 13 mm having yiels point f_y = 1275 MPa and tensile strength f_u = 1422 MPa can be used for shear reinforcement (including spiral) in beams and columns of building in Japan, although the allowable stress intensity is limited to 196 MPa for service load and 588 MPa for ultimate load (usually earthquake load). These advantages of using high strength bar is to be pursued more, and future developments are looked forward to.

3.2 Large-diameter Reinforcing Bars

Utilization of higher Strength as well as larger diameter of reinforcing bars is conceivable for the purposes of manpower reduction, mechanization of reinforcement work and improvement in construction precision in large-scaled reinforced concrete structures. Accordingly, D 57 (#18) has been standardized under ASTM in the United States, and an even larger D 64 (#20) has been developed also in Japan. For this bar, a thread-like deformation along the bar length has been adopted for ease of mechanical jointing. [14] Anchorage of these reinforcing bars is of mechanical type using anchor nuts and anchor plates, the anchoring mechanism of which consist of the bond force of the specified embedment length and the bearing force by the nuts and plates.
Problems in using reinforcing bars of such extremely large diameter may be said to be the following;
1) increase in weight per individual bar, 2) increase in bending radius of reinforcing bar, 3) reduction in joint-forming efficiency, 4) impairment of bond and splitting strength of members 5) impairment of cracking properties of members (increase in crack width).

Because of these problems, development work is going on concerning pre-assembling in placement of large-diameter bars, efficient placing equipment, and sure mechanical joints, along with which experimental studies are being made on mechanical properties of members. Fig. 4 shows examples of the influences of various-diameter reinforcing bars on cracking properties of beams, the dimension of which is 30 x 65 x 120 cm (b x h x I) for small beams and 60 x 130 x 1040 cm for large ones. It has been reported that when using such large-diameter bars, such measures as inserting reinforcing bar grills-steel mesh composed of 10- to 13-mm diameter bars with spacing of 10 to 20 cm, for example, at the cover (10 to 30 cm) - are quite effective. [14]

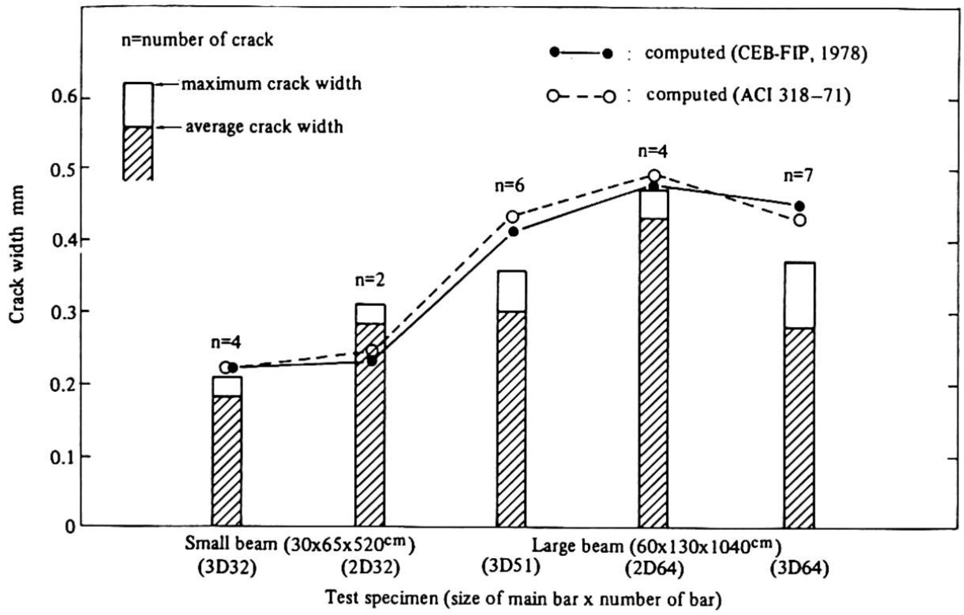

Fig. 4 Comparison of crack width in beams with smaller dia. of bars and those with large ones. (σ_s=1800 Kgf/cm^2, static load)

3.3 Salt-resistant Reinforcing Bars

Corrosion of reinforcing bars in concrete has been aggravated by the use of marine sand as aggregate, spreading of de-icing chemicals in winter on the bridge slab, and penetration of chlorides into offshore structures. Since the use of epoxy resin-coated reinforcing bars was started in the United States around 1970 as a measure to prevent corrosion, such bars have come to be used increasingly in the United States, Canada, Middle East countries, Japan, and the United Kingdom, and it is a standard now in 48 states of America for this type of bar to be used in bridges. Such reinforcing bars are made by electrostatic powder coating with coat thicknesses normally 150 to 300 μ, and averaging about 200 μ. It is necessary for care to be exercised concerning lower bond strength with concrete because of the coating, some amount of limitations in bending fabrication of bars, and preventing peeling off of coats during construction. It has also been reported that these bars are approximately double in cost compared with ordinary reinforcing bars, but service life is 10 to 14 times longer. [15]

Meanwhile, several attempts have been made to improve salt resistance through adjustments of the composition of reinforcing steel. That is, salt-resistant reinforcing steels are being developed by selecting appropriate composition systems aiming to impart corrosion resistance several times greater than normal, with rapid corrosion and deterioration not resulting even when carbonation of concrete has occurred, while properties as reinforcing steel such as fatigue resistance, fire resistance, and bond strength are not impaired. As examples, a nickel (Ni) alloy (about 3.5 percent) containing a small amount (not more than 0.3 percent) of tungsten (W), and a 2 percent chromium (Cr) steel have indicated good performances. Exposure tests are being carried out along with laboratory research works and the success of such development research is eagerly awaited. [16], [17]

4. New Chemical Admixtures

4.1 Superplasticizers

It has been mentioned previously in this paper that the use of a high-range water-reducing admixture is indispensable for manufacturing high-strength concrete containing materials such as silica fume. The use of so-called fluidified concrete made by adding fluidifier to base concrete of comparatively stiff consistency and agitating to increase fluidity with the aim of improving workability of stiff-consistency concrete has been increasing. This method was developed in West Germany and was introduced in Japan around 1975. In Japan the fluidifiers, or superplasticizers as they have come to be called, of naphthalene sulfonate base and others were developed, which were different from the fluidifier (melamine base) of West Germany, and which at first had been used exclusively in building construction on experimental bases, the demand increased in the civil engineering field also. As guidelines on superplasticized concrete were established in 1983 by both the Japan Society of Civil Engineers and the Architectural Institute of Japan, and also recommended practices (draft) were prepared, demand has since increased sharply and the total demand in 1987 is estimated to have been more than 7 million m^3 in terms of the volume of concrete.

The rapid increase in the use of superplasticized concrete in Japan had the background described below. In essence, concrete of higher slump came to be used in the architectural field, and especially, with unit water content increased due to use of richer mixes in placing by pump and by the increased use of crushed stone, adverse effects in quality and durability of concrete and structures had been of concern. Consequently, base concrete of medium slump (8 to 12 cm) was fluidified in the field to higher slump (18 to 21 cm) by using a superplasticizer with the objectives being prevention of cracking, reduction of heat of hydration of cement, and improvement of durability through reductions in unit water content and unit cement content. In particular, a principle was set up aiming for unit water content and slump not to be more than 185 kg/m^3 and 18 cm, respectively, to secure durability of concrete, and as a result the necessity to use superplasticizers was increased even more. On the other hand, in the civil engineering field, improvements in workability and efficiency have been sought imparting fluidity to conventional stiff-consistency concrete through the use of superplasticizers.

Initially, addition of a superplasticizer to ready-mixed concrete was done at the jobsite in the form of delayed addition and high-speed mixing by agitator immediately before placement. However, because of problems of noise and time-dependent changes in slump after fluidification, the method of adding superplasticizer intermittently to recover slump was devised and a retarding-type superplasticizer with which slump loss is smaller was developed so that the method in which all that needs to be done is to add the superplasticizer at the ready-mix plant is beginning to come into practical use. For example, a certain superplasticizer contains a new, reactive polymer which changes its molecular sturcture in concrete. This polymer is normally a finely-divided powder of several microns insoluble in water which becomes water-soluble affected by the surplus alkali produced as hydration of cement progresses, and functions as a dispersing agent. The result is a gradual supply of dispersing agent making it possible to maintain slump constant as time elapses as shown in Fig. 5. [18] Fluidifying operations are thus simplified for advantages in control so that the use of superplasticized concrete is thought to increase even more. The strength characteristics and durability of concrete fludified in this manner have been studied, and it has been ascertained that they are more or less the same those of base concrete.

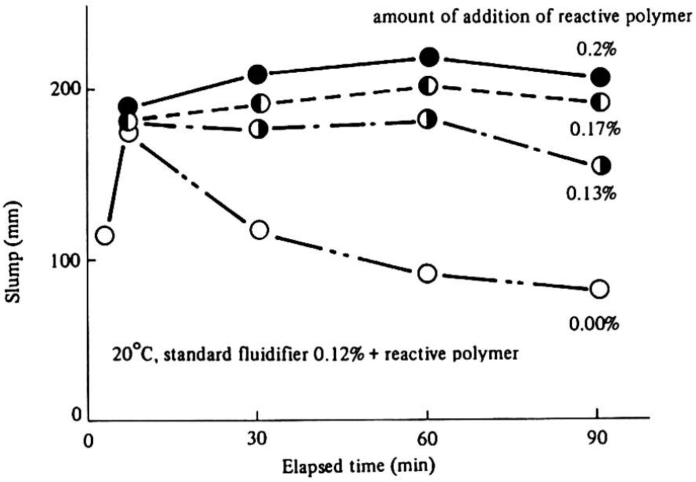

Fig. 5 Effect of addition of reactive polymer on slump

4.2 Admixtures for Underwater Concrete

Underwater placemetn of concrete was attempted as early as the mid-nineteenth century, and subsequently, work was done to prevent segregation of materials in water and to secure quality of concrete to satisfy structural requirments, which has evolved in the forms of placement by tremie, prepacked concrete, and development and improvement of equipment. The objects of underwater concrete placement have shifted recently from small-scale of members to offshore platforms, docks and harbor facilities, large-scale of bridge piers, etc. The nuature of underwater concrete has changed from a mere filler material to an important repair material for major underwater structures, properties of which high quality and good durability are demanded. [19]

For this purpose, development work has been going on for special admixtures that are not limited by construction methods and type of equipment used, that keep segregation in water to a minimum and make possible placement of high-strength, durable underwater concrete of increased reliability. The first attempt was made in West Germany in 1977, where an admixture consisting of a water-soluble polymer was added to impart viscosity to concrete making it more difficult for cement and aggregate to become segregated while falling through water. Subsequently, similar development work was done in various countries and these admixtures are being widely used. In Japan, such admixtures have been in actural use since 1981 with approximately 10 products having been made available up to the present for placement of a cumulative volume of approximately 140,000 m^3 of special underwater concrete. The results of screening analyses of fresh underwater special concrete and normal concrete are compared in Fig. 6. A design and construction manual for this type of concrete has also been published. [20]

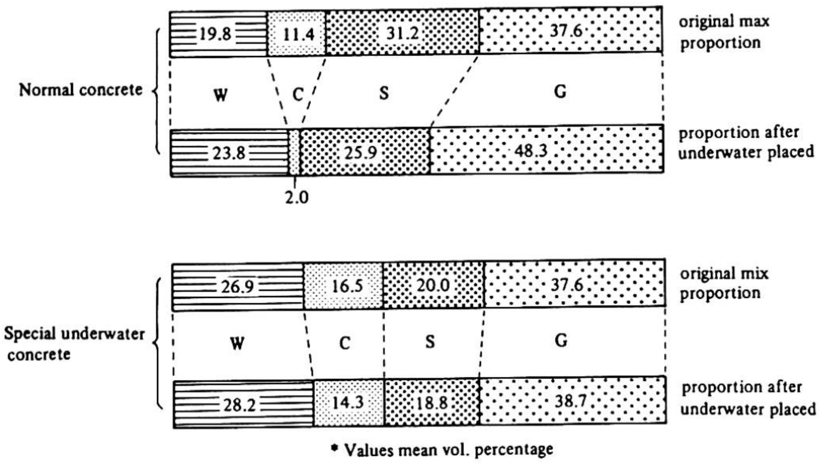

Fig. 6 Wet screening analysis of underwater special concrete and permal concrete

The main components of the special admixtures being used are of two kinds-cellulose base and acrylic base water-soluble polymer. These materials are all either of powder or granular form. The materials are also required to be high in safety when used in under water concrete. The features in quality of special underwater concrete when fresh are as follows:

(i) Excellent resistance to segregation when subjected to washout action of water
(ii) High viscosity and plasticity, and excellent self-levelling and selt-compacting properties
(iii) Little occurrence of bleeding and laitance
(iv) Regarding time of setting, retardation with cellulose base admixture and no change with acrylic base admixture
(v) Increased resistance in case of conveying by concrete pump

When too much amount of special admixture is used there will be objectionable results such as extreme retardation or excessive viscosity for poor workability, and in many cases the admixture is used in combination with an air-entraining agent, water-reducing agent, superplasticizer, etc., and with silica fume, granulated blast-furnace slag, fly ash, and the like. When selecting an

Table 6 Type of Special Admixture and Combination with Superplasticizer

Special Admixture	Superplasticizer
Cellulose base	Melamine sulfonate type (triazine base)
Acrylic base	Naphthalene sulfonate
	Melamine sulfonate base (triazine)
	Acrylic base
	Polycarbonic acid base

air-entraining agent or water-reducing agent, it will be necessary to give consideration to compatability with the special admixture for underwater concrete as shown in Table 6. [20]

4.3 admixture for Highly Durable Concrete

An attempt to greatly improve concrete quality by means of an admixture that has drawn attention is described below.

A certain type of water-insoluble glycol ether derivative greatly reduces drying shrinkage of concrete and shows good durability in relation to permeability, progress of carbonation, and penetration of chloride ions, while an amino alcohol derivative in the constitution of concrete has the functions of absorbing oxygen gas and adsorbing chlorine ions. Taking advantage of these properties and using the two in a suitable combination there is a possibility for a concrete of high durability to be obtained. As an example Fig. 7 shows strengths, drying shrinkages, permeability coefficients, carbonation depths, and chlorine ion penetration depths of mortar and concrete containing 3 percent glycol ether derivative and 1 percent amino alcohol derivative, both by weight of cement, compared with plain or air-entrained concrete. [21] The result show the excellent effect of these admixtures, and it is reported that judging by the rate of increase in carbonation depth it is not a wild dream that concrete of service life longer than 500 years can be made so long as the water cement ratio of concrete is below 50% and the cover of reinforcement is over 40 mm.

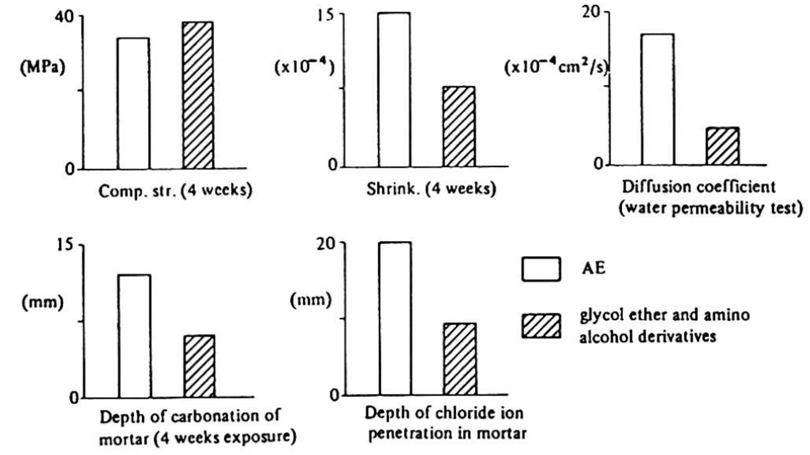

Fig. 7 Durability of concrete containing both glycol ether and amino alcohol derivatives

5. Concrete-Polymer Compositers

It has been approximately 30 years since research and development began on concrete-polymer compositer with the aim of improving the performance of cement concrete. The forms of introducing polymer into concrete may be classified according to the three of a) poymer modified (cement) concrete (or mortar) (PCC or PPCC), b) polymer concrete (PC), and c) polymer impregnated concrete (PIC). The degrees of improvement it performance achieved all differ, but the principal effects of polymer introduction are improvements in mechanical properties of concrete (for example, ratio between compressive strength and tensile strength), watertightness, freeze-thaw resistance, and resistance to chemicals. However, there are some among physical properties which are adversely affected, resulting, for example, in increased thermal expansion and reduced heat resistances.

5.1 Poymer Impregnated Concrete

Work on polymer impregnated concrete began from the latter 1960s in the United States, Europe, and Japan, typical monomers for impregnation being methyl metacrylate and styrene. Many results were reported, but research and development on this has not been actively pursued of recent. The reasons are 1) that the menufacturing process is complex and energy consumption in the drying and polymerization processes is large, 2) the performances obtained are not greatly different from those of polymer concrete (resin concrete), and a balance between performance and cost is unobtainable, and 3) methods of measuring poymer impregnation ratio and depth are not available to make quality control difficult. The last and largest case of application in the United States was a road at Grand Coulee Dam, and the construction cost was \$51/m^2 (1982). [22] However, concrete of ultra-high strength, 196 to 275 MPa in terms of compressive strength was developed in the 1970s through the application of impregnation techniques, while compressive strengths of 225 to 255 MPa have been obtained experimentally through the addition of silica fume and superplasticizer, autoclave curing, and polymetacrylate methyl impregnation. However, these high-strength products have not found practical use as yet. With regard to application of on-site polymer impregnation methods, there have been experiments made with the purposes of improving surface hardness and strength, watertightness, chemical resistance, chlorine ion penetration resistance, and carbonation resistance of existing structures, with work done in repairs of pavements and dams, and corrosion protection of factory floors, but practical applications are still insufficient.

5.2 Polymer Modified Cement Concrete (or Mortar)

In manufacturing polymer modified cement concrete (or mortar), polymer is added in the form of a dispersion (latex or emulstion) to ordinary cement concrete (or mortar) at the time of mixing. Polymer dispersions widely used in Japan are SBR (styrene-butadiene rubber) latex, and EVA (ethylene vinyl acetate) and PAE (polyacrylic ester) emulsions, annual consumption exceeding 100,000 Mg. In the United States and Europe, water-soluble polymers such as unsaturated polyester resin are used fairly widely, but these are rarely used in Japan because of high prices. In general, polymer-cement ratios used are in a range of 5 to 30 percent. When polymer-cement ratio is increased, tensile and flexural strengths, extensibility, adhesion, watertightness, chemical resistance, etc. are improved, but surface hardness is reduced. As an example, the increases in resistance of polymer modified cement concrete (mortar) to penetration of chlorine ions (Cl^-) are shown in Table 7. [23] This indicates that the use of polymer cement modified concrete is effective for prevention of salt damage to reinforced concrete structures, and this type of concrete is being applied in repairs and overlays of bridge decks deteri-

Table 7 Apparent chloride ion diffusion coefficient of latex-modified mortars and concretes

Type of mortar	Polymer-cement ratio, percent	Apparent chloride ion diffusion coefficient, cm²/s	Type of concrete	Polymer-cement ratio, percent	Apparent chloride ion diffusion coefficient, cm²/s
Un-modified	0	6.4×10^{-8}	Un-modified	0	2.2×10^{-8}
SBR-modified	10	6.4×10^{-8}	SBR-modified	10	1.9×10^{-8}
	20	3.9×10^{-8}		20	9.3×10^{-9}
EVA-modified	10	4.4×10^{-8}	EVA-modified	10	7.9×10^{-9}
	20	2.4×10^{-8}		20	1.0×10^{-8}
PAE-modified	10	3.8×10^{-8}	PAE-modified	10	6.2×10^{-9}
	20	4.4×10^{-8}		20	5.8×10^{-9}

orated by de-icing salts and of parking structure floor slabs.

Since the physical properties of polymer itself are greatly dependent on temperature, the temperature dependence of polymer modified cement concrete is prominent, and it may be considered that the temperature limit for use is approximetely 150°C. According to outdoor exposure tests, it appears that the weather resistance of Polymer modified cement concrete (or mortar) is not of concern for practical purposes. Polymer modified cement concrete has the merit that ordinary concreting techniques can be applied without alteration, but there is little use for it as concrete per se. Applications are as pavement material, floor material, repair material, corrosion protection material, and adhesive material, and it is looked forward to that uses will be expanded hereafter.

5.3 Polymer Concrete (Resin Concrete)

Various liquid resins are used as binders for poylmer concrete, but what are most common are epoxy resin, unsaturated polyester resin (polyester-styrene base), and methyl metacrylate (MMA) (monomer). Besides the above, furan, urethance, polyester amid, and vinyl ester are also used. Although differing according to circumstances in the individual countries, MMA and plyester are the lowest in cost. Polyester used is mostly for precast factory products, but MMA is also often sold generally in prepackaged form. This is because of the flammability and disagreeable odor of MMA.

In case of using polymer concrete for structural purposes, steel bars for reinforced concrete, prestressing steel rods, and FRP rods are used for reinforcement of members, while for reinforcement of polymer concrete itself, reinforcing materials such as steel fibers and glass fibers are used. In general, the amount of polymer contained as binder is 9 to 25 wt%. The properties of polymer concrete depend to a great extent on the type and properties of the binder and the properties of aggregate. Manufacturing of polymer concrete (mortar) is done using batch-type forced-mixing mixers and continuous-mixing type mixers. In the United States, a continuous-mixing mixer with capacity of 250 kg/min for repair and overlay of bridge decks has been developed. The advantageous points of polymer concrete for overlays are the good adhesion to old concrete, the feasibility of finishing to thin cross sections (to about 13 mm) for low dead weight, the unnecessity to place approach slabs, the good mechanical properties, and the shortness of curing time.

A noteworthy application of PC is the development in Japan of a method used for automatic construction of small cross-section shield tunnels (inside diameter 120 cm, thickness 10 cm) having lengths of 107 m and 200 m. The compressive strength of specimens cored from lining concrete was 88.9 MPa with standard deviation of 11.4 MPa. [24] The strength of reinforced plastic composite pipe using polyester mortar was comparable to steel pipe with maximum diameter attained as much as 5.2 m. The production is 30,000 to 40,000 ton annually. Other than the above, a recent trend is that which takes advantage of the excellent vibration damping properties of polymer concrete for use in machine tool beds and fabrication of machinery parts, West Germany and Switzerland having taken the lead in development.

6. Closing Remarks

The advances in new design and construction of structures have been made together with the development of new materials and improvement of existing ones. In the field of construction, what are required of construction materials are the capabilities to be made into large size and fill large-volume demand, be of high strength, satisfy demands for durability over a long term in a man-made environment, and be of favorable benefit-cost ratio. For such objectives, fundamentally steel and concrete will continue to comprise the mainstream of structural materails, with new varieties respectively developed. This paper has been centered on concrete and briefly describes the development and application of new admixtures, high-performance reinforcing bars, and further, polymer concretes as means of making concretes of high performance. Probelms requiring further study have been discussed, and the necessity for mutual cooperation between construction engineers and materials engineers has been pointed out.

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Computer Infrastructure and Artificial Intelligence for the Design Office

Infrastructure informatique et intelligence artificielle dans un bureau d'ingénieurs

Computer-Infrastruktur und künstliche Intelligenz im Bauingenieurbüro

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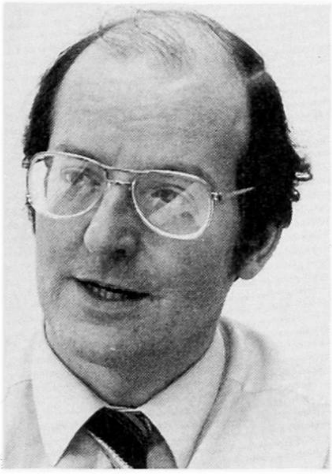


Born 1940, started his civil engineering career as a student with British Railways where he spent five years on a variety of projects. He then moved to the contractor George Wimpey for further site experience before joining a consultant. He has worked with Ove Arup Partnership since 1964, first as a designer then on computer technical support. His responsibilites include policy for Information Technology and hardware acquisition.

SUMMARY
This paper is a transcription of the presentation given by David Taffs who illustrated his talk with numerous slides. It shows the type of work undertaken by a Consulting Engineer and how small a contribution the computer makes to many projects. Other projects are dominated by computer use. The computer infrastructure to support many offices of all sizes is described including communications equipment. A mix of old and new is recommended and adoption of international standards. A direction to future development is given and reference is made to experiences to date with Expert Systems.

RÉSUMÉ
Cette contribution est le compte-rendu de la présentation – illustrée de nombreuses diapositives – de David Taffs lors du Congrès. Elle montre le genre d'activités dans unbureau d'études et la très modeste contribution de l'ordinateur dans de nombreux projets. Dans certains cas la contribution de l'ordinateur est essentielle. L'auteur décrit l'infrastructure informatique et les moyens de communications nécessaires à de nombreux bureaux d'études de grandeur variable. L'auteur est en faveur d'une combinaison d'équipements anciens et nouveaux et souhaite l'adoption de normes internationales. Il indique une direction pour de futures développements et mentionne les expériences réalisées à ce jour avec les systèmes experts.

ZUSAMMENFASSUNG
Dieser Beitrag von David Taffs wurde durch zahlreiche Diapositive illustriert. Er behandelt die Arbeit eines Ingenieurbüros und zeigt, wie bei vielen Projekten der Beitrag des Computers gering ist. In anderen Projekten macht die Rechnerbenützung den wesentlichen Teil aus. Es wird die Rechnerinfrastruktur zur Unterstützung zahlreicher Büros verschiedener Grösse mit den Uebertragungseinrichtungen beschrieben. Eine Mischung von alten und neuen Geräten und die Anwendung internationaler Normen wird empfohlen. Die Richtung der zukünftigen Entwicklung und die Erfahrungen mit Expertensystemen werden ebenfalls beschrieben.

Lecture presented by David Taffs:

Good morning everyone.

The other day I looked in my database and my database is a little box, it has got drawers and hanging files, but it contains information. The older ones of you might call it a filing cabinet, but the computer manufacturers assured me, it's a database. I looked in my database and I found that it was about 10 years ago since you last invited me to speak to IABSE; on that occasion it was in Bergamo in Northern Italy. In about half an hour you will be forgiven for thinking that things have changed very little in 10 years - there is just a little more colour now with our use of computers.

What I am going to do is run through, very quickly, a number of applications just to give you a feel for the way that we use our computers and then go on into the type of hardware that we are using and I hope, if there is time, we finish off with expert systems, artificial intelligence and where we think that is going.

So I start with my first slide which shows highway design and in this particular area of application I don't think anything more needs to be said. It is an obvious area in which to apply computers, we have been doing it for 20 odd years, and I think we are all fairly successful at it. So we do not need to say anything more about it.

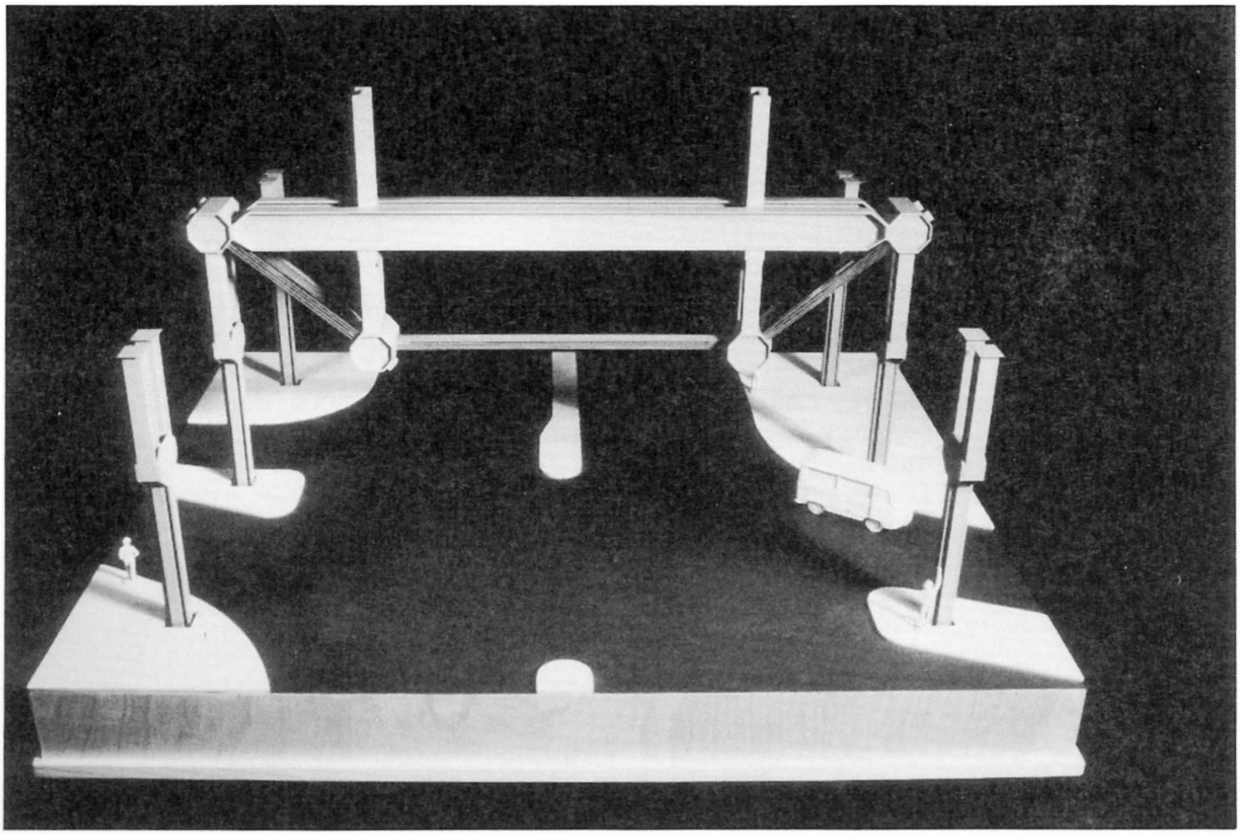
Moving on from highway-design where there is a very clear benefit into an area where we see somewhat less benefit - and that is in site-modelling. But nevertheless, we can model our sites although they may be quite small in some cases. We feed in the data from survey information and then go on and add in, if we wish, the construction sequence for people like contractors to look at the way in which the project is going to be built. This will in turn help the contractor and others to realize how to handle the materials on the site and how best to organize it.

In areas where the structure is very well defined, there is no problem. We can produce computer systems to completely analyse what is there and building a structure such as lattice steel microwave towers, where the dishes at the top may vary in number and orientation, only requires a degree of interaction between the designer and the computer to produce the necessary data which then in turn can be fed through to automatic drawing production.

20 odd years ago we were playing with all sorts of computer applications. Where there is a very well defined system building it is possible to go as far as deciding where the power points are going to appear, where all the services in the project will appear and all of this could be built into the computer system quite happily. Unfortunately, in my particular office we get very few of that type of project to deal with. This (Figure 1) is more representative of the sort of problem we get and I hope, in talking about it in this way, I get across to you how relatively unimportant CAD and computer-tools are to us in our office.

We are looking at the Menil Museum in the United States. The interior lighting was a very important facet of the project and we had to devise ways in which the roof structure would give the correct

1. Ferro cement and ductile iron roof structure of the Menil museum.



2. Transfer structure over six lane highway in London.

lighting and not cast any unwanted shadows within the interior of the structure and so, the design evolved. We came out with a solution where the roof beam bottom cords were in ferro cement and the top cords in ductile iron. Ductile iron has the characteristic of being able to be cast into very fine detail, it is more fluid than other cast irons, so the castings are more accurate. It does not require any heat-treatment, so you do not get distortion. The ferro cement was given a texture that matched with the ductile iron.

Now, the design was by Renzo Piano, the Italian architect, and ferro cement, you would imagine, was very much an Italian material. In fact, we could find no one in Italy who seemed to be able to remember how to construct ferro cement and we had to scour the world looking for people who could do this. It is a fairly convoluted story about how we eventually found an individual or an organisation that could do the work and also ways in which the quality assurance of that structure evolved. No amount of earlier thoughts on CAD systems would have been of very much use in a situation like that. As a consequence of that particular design and project, just the week before last there was another meeting in Italy resulting from an invitation to prepare another museum in the USA. The design team sat around a table, working upon the concept of that design and there was not a computer in sight! I cannot see a situation where that is likely to be for a very long time.

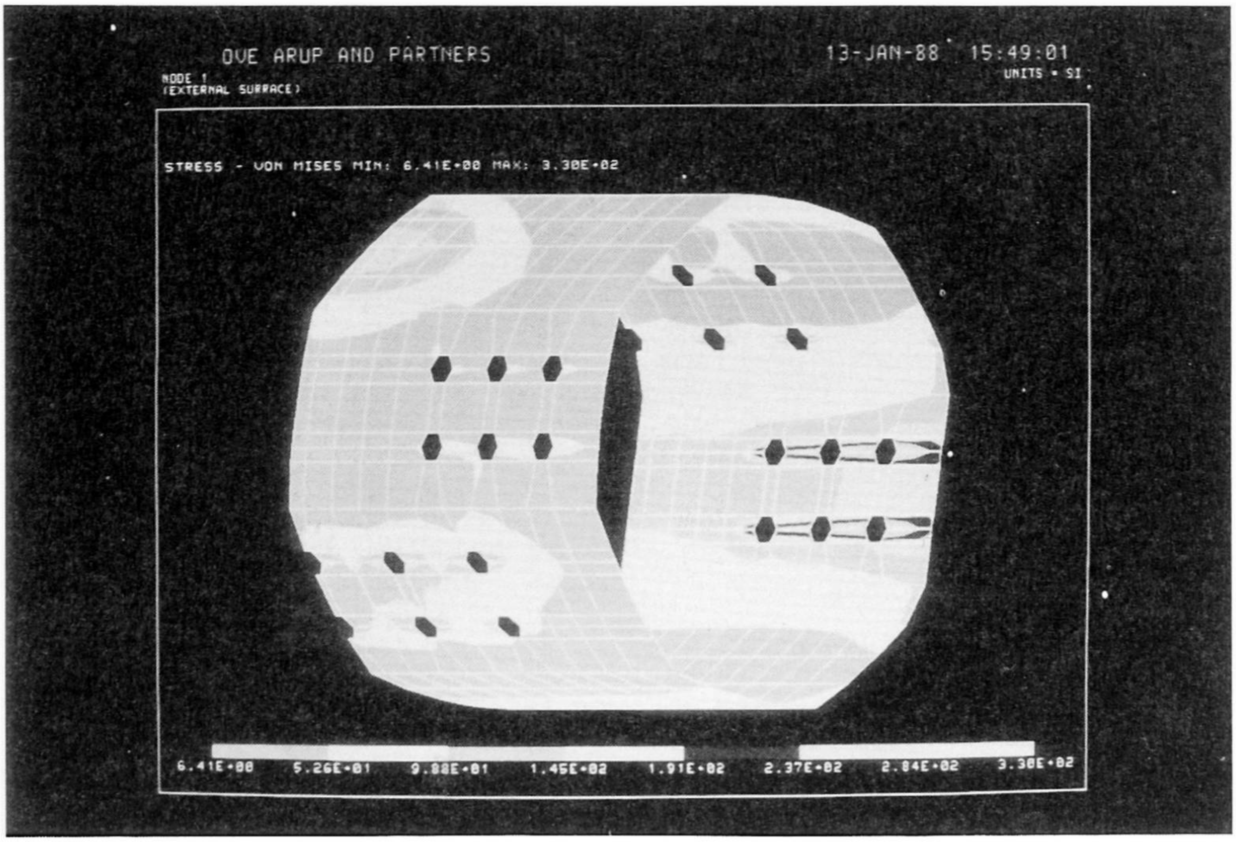
Where we do use the computer, is more in analysis, in particular areas of analysis - and here is a project that is going up in London at the moment (Figure 2). It is an 18-storey-building that fits across a six-lane highway, a dual carriage highway. Here you see the road intersections, and the problem was designing the transfer structure underneath the building. The obvious solution would have been a normal truss but the crossbracing of that truss at the ends, where you have your maximum shear, would have clashed with an area that had to be left clear for pedestrians to walk through. So the design chosen was an inverted truss. The tension members were bars that allowed for take up of tolerances during the construction. We use our computers simply to tell us a little bit about the particular casting that you see in Figure 3 - an analysis of the structure.

Very much the sort of thing that we were seeing in Bergamo ten years ago, but without the colour.

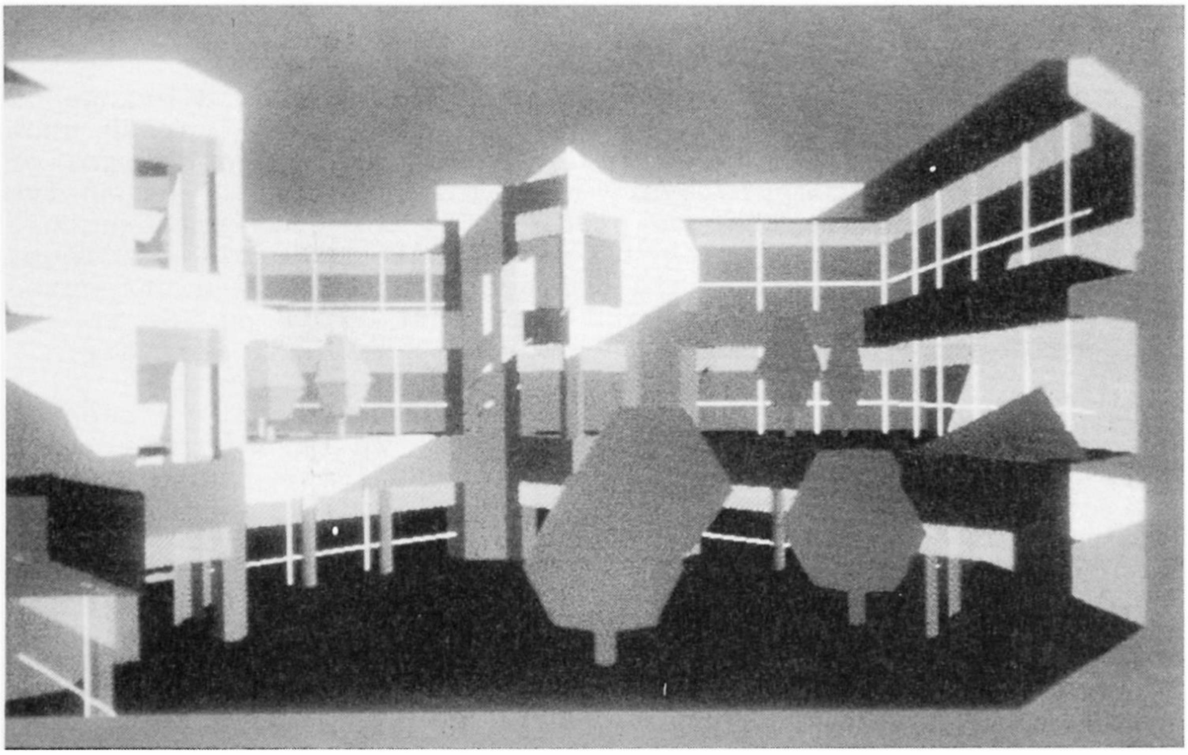
One pleasing aspect of this particular project was that we could use the computer to produce drawings and those drawings in turn were passed to an artist called Ben Johnson. He is well-known in the UK and specializes in painting mechanical objects and building structures. Having been suitably inspired by some of the work we were doing on this scheme he produced a painting some three metres high, of the transfer structure steelwork. So that to me is one of the more pleasing uses of CAD.

Visualization is where we see computers used quite often at the moment. Actually the cost benefits are very debatable but the results look quite attractive. In Figure 4 you see a courtyard in the middle of a hospital complex. In other cases you are looking at larger landscapes and bringing in existing buildings.

Interior shots as well, these are popular, generating images quite quickly. We do find that we need particular types of computer



3. Stress diagram of a node in the transfer structure.



4. Visualisation of courtyard in a hospital complex.

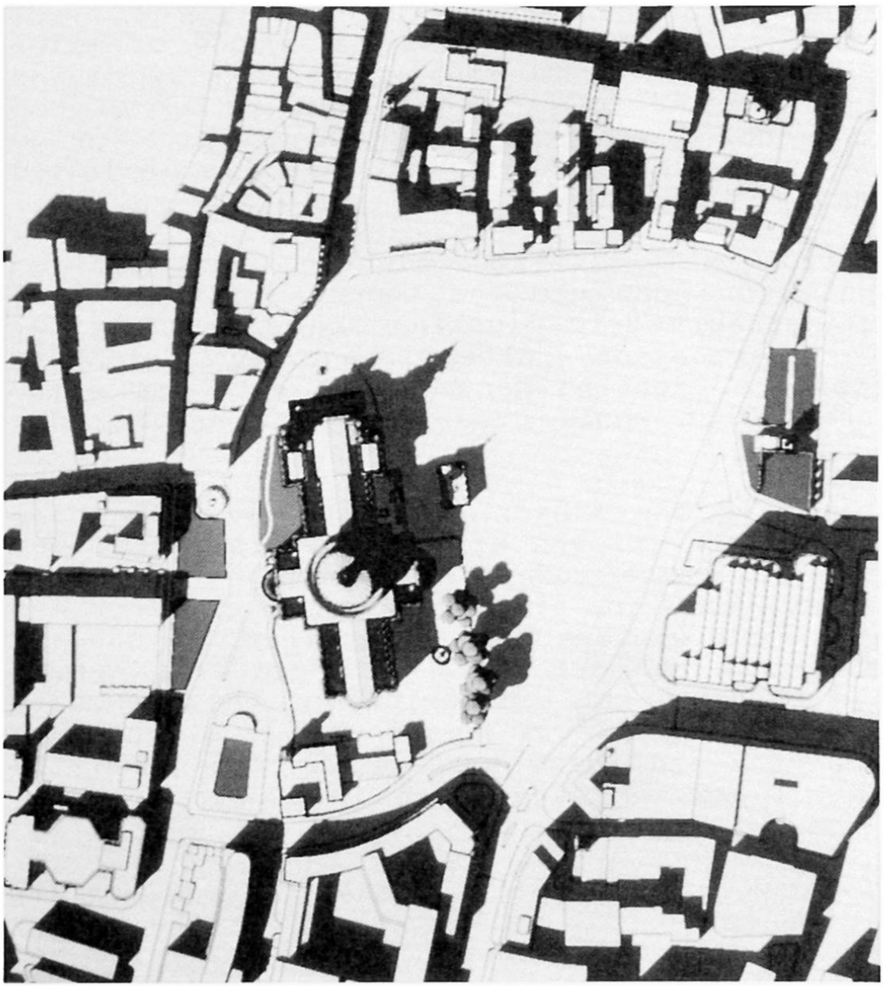
graphics software in order to generate the information we need within the timescale we are given for the project. There often is not time to sit down and drive an elaborate graphics system. It is also very surprising how often we get caught out when we look at the proprietory systems. One of the first slides I showed you on highway design was generated on a system that is very strong in the UK in the motor industry. Therefore, we thought if we bought it we ought to have no problem considering the complexity of some of those motor engineering structures. But in fact we hit a limit - as soon as we started to work on the system, we hit a limit in terms of the file sizes it could hold and it is remarkable that we keep coming up against these problems. I tend to think of ourselves as being fairly modest in our demands on CAD and we hear of all these other wonderful things that are happening and yet, every time we dabble, we hit limits of one sort or another and these systems cannot cope. I would like to mention our 3D-work. We find we need particular pieces of software, particular techniques to get the information we need fast enough. Having built a 3D model, analysis that can be either lighting or thermal analysis and things like this flow from it.

There is a project that is currently being tackled in London (Figure 5). You see an area around St. Paul's Cathedral in the centre and our interpretation of St. Paul's. Those of you who are avid readers of the magazine CAD/CAM will recognize that we are on the front cover of this month's issue. The cover shows a view of structural steelwork of a project, again in London, which is called Broadgate, a very massive fast-track project. The model of the steel was created through the CAD-system to enable the architects to play with different configurations of steel and to get a fair idea of what the final product would look like.

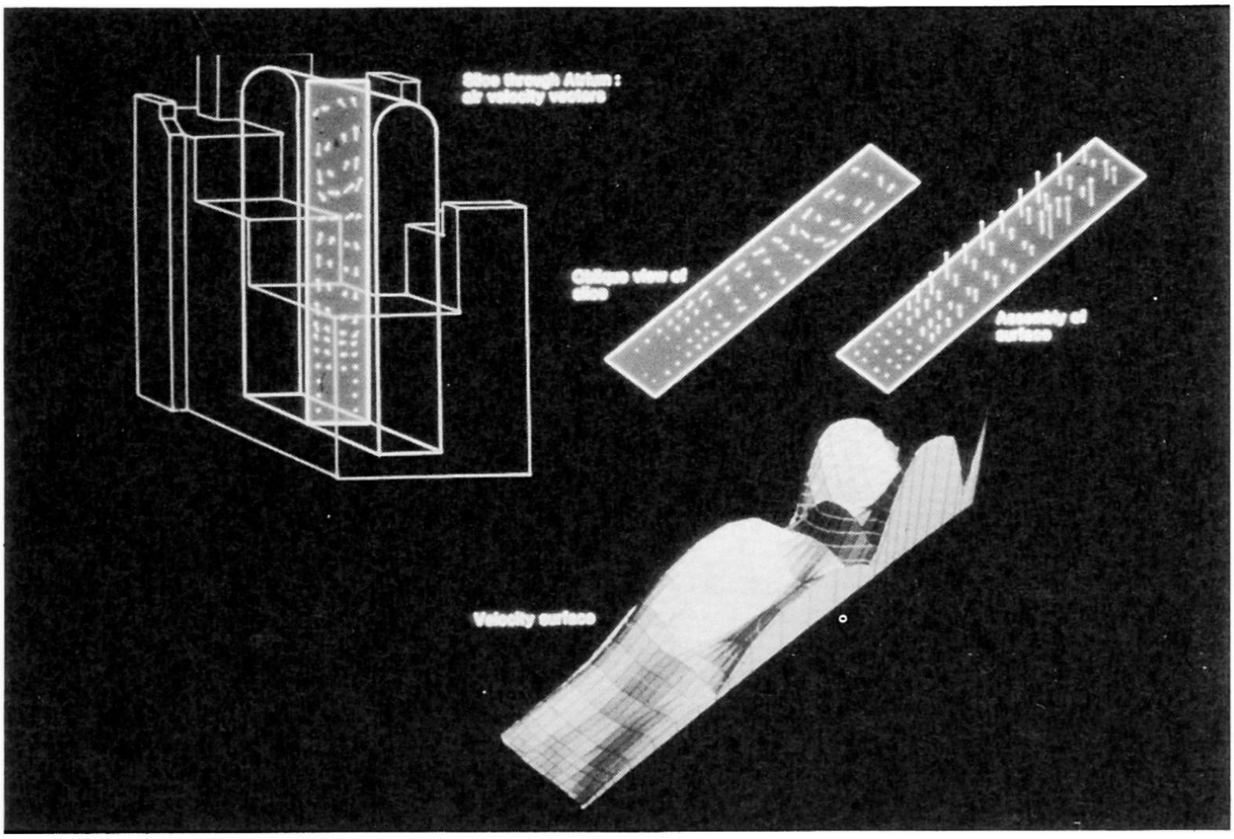
In your company today I thought I should at least show a bridge or two, perhaps a cable stay bridge. Alternatively a more garish shot for bridge engineers who are colourblind. But in bridge design, of course, we do use the computer very much more for numerical analysis, for getting the data that we need. Being the complex structure it is, we have construction stages to consider, variable loadings, the automatic positioning of vehicle loads or uniformly distributed loads, use of influence surfaces and this sort of thing. So we are using the computer in the traditional way of numerical analysis.

Light weight structures, of course, lend themselves to this because of the amount of geometrical calculation that we have to deal with and it is not only just for the stressing up of the fabric within those light weight structures but also for producing the cutting patterns for making the structure afterwards. So these are all by-products of the visualization.

Simulation is another area and in this particular case we are looking at the design of a flask for carrying radioactive waste. The client was very concerned about quality assurance and as a consequence we ran a number of simulations. Although we dropped this flask from great heights and it had not broken, someone came up with the idea that if it fell across a railway track and at a particular angle across that track, it would be at its most vulnerable state if it were then hit by a high speed express train. So we ran through a computer simulation of that particular event. Then the client did something unusual. He sat all the design team down on



5. Computer model of development area around St. Pauls in London.



6. Simulation of air movements in atrium of the Lloyds building in London.

seats, on a ramp, rather like yourselves, and placed the flask across the track. An express train was started a couple of miles further down the track so that it would get up to full speed and the whole of the design team had to sit and watch whether or not their analyses were correct. Fortunately, few of us have to go through that experience. Thankfully, it all worked out as predicted so there was not too much to worry about.

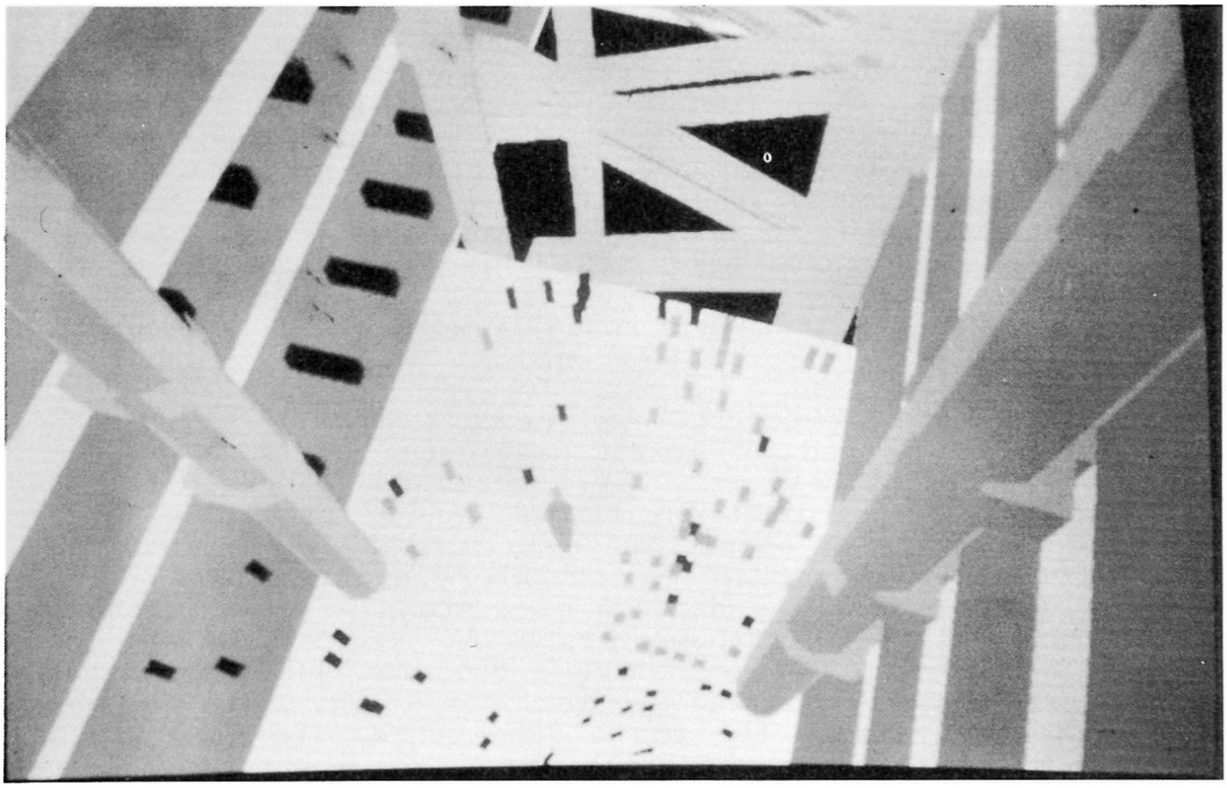
Another area of application is in lighting. Lighting design, interior lighting and, again, one can use the computer for handling different types of luminaires, strip lighting and spot lighting. There is quite a lot of calculation, rather like a very intensive finite element analysis. The lighting can also be not just simulated for individual rooms but whole areas and with natural day lighting as well.

Another project in London is the Lloyds Building. In the centre there is a very high glass atrium, and off to the sides are open plan offices, where the insurance underwriters sit and do their trading throughout the day. They are sitting on the open floor and there was concern that in our winters very cold air would cascade down the side of this atrium and perculate to where the traders were sitting. So we had to carry out simulations of the air movement. So we set up a model of the atrium itself (Figure 6) and then carried out various analyses and these surface diagrams show the peaking of velocity of air movement and where it occurred at the different sections of the building.

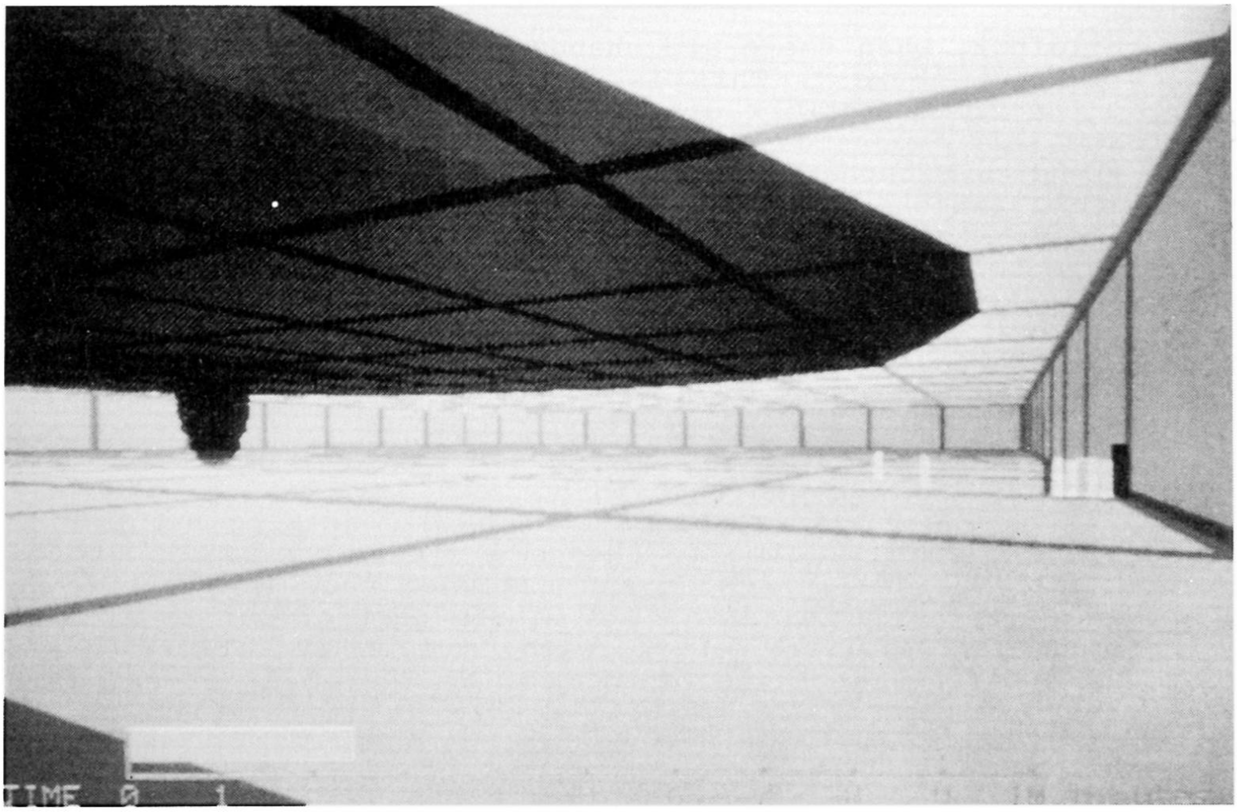
The view in Figure 7 is looking down into the base of that atrium, but the very fact that we see people walking about at the bottom tells me that we are looking at a different type of analysis, and that is one of fire. With fire engineering we are talking about the movement of people within buildings. Typically, with a very large building you are looking at the speed of evacuation of people from those spaces and at all the constraints that are imposed in terms of rate of exit, distance to travel and the different speed of travel of the people. So one can produce a simulation of people moving at different speeds according to their abilities or disabilities and seeing how long it takes to evacuate the building. So one starts a fire in the computer model, you see a few flames at the bottom there and you see the smoke starting to billow up and then, watching that smoke spread and again watching the time it takes for that random sample of people to evacuate the building (Figure 8).

I want to tell you that I have yet to find a need in our particular work to carry out 3-dimensional modelling of building services. I often hear it said how important it is to carry out modelling and I have yet to find the need for it. It normally takes longer to produce the model and there are so many factors which the model cannot sensibly take into account that it is not worth the effort.

Let us take a quick look at a trend that is growing, and that is into intelligent buildings. By intelligent buildings I do not mean cabling of buildings. Invariably, when people talk about intelligent buildings today, they talk about more cabling, special cabling in buildings. That is not what we mean. We are talking about computer control of buildings and its environment. This is a new development and there are very few examples in existence. The other new



7. View of floor of Lloyds atrium taken as part of fire simulation exercise.



8. Smoke cloud advancing over members of public exiting to the right.

area of development is facilities management. What I mean by facilities management is where clients are asking for the computer data to be passed on to them to enable them to carry out maintenance and subsequent planning of the structure after completion.

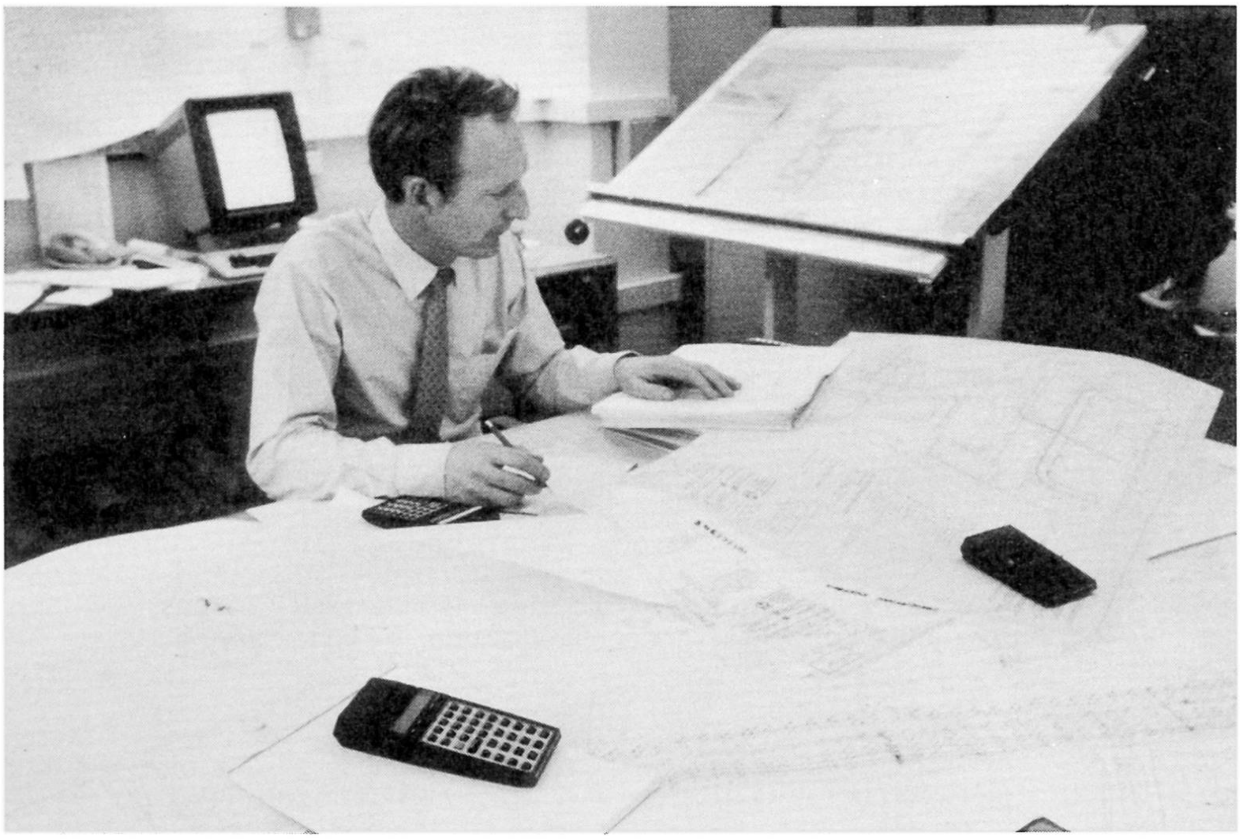
Right. So let's look now at the computer infrastructure behind these applications to give you some idea of what we are currently using. We are interested in what we can make work today. Typically, one would hear about these marvellous things 10 years ago in conferences but I am talking about struggling today to make the things that we heard about 10 years ago actually work. That is the difference.

We have networks of computer systems. We have about 16 buildings in London. There is a need to intercommunicate. Design groups are split as new projects come in. They have to be relocated together in different buildings, different floors, and all the time we have this dynamic movement of people which in turn means, moving equipment to allow them to get to the computers and use them as they need to be used. Within the network of systems we have a range of computers. There is a DEC-10 computer, a VAX II/785 computer. There are a couple of Primes, one of which a 9755 has just been changed for a 6350. To those of you who are into computing, I say its amazing that we keep hitting the top of the range. The 6350 when we bought it a few months ago was the biggest, the most powerful system that Prime had. For the little bit of work that we do, to find ourselves having to use the biggest and the best that they have, I find staggering. What, I wonder, is the rest of the world doing.

Within buildings, when there are changes, we have to reanalyse the segments of the network. This is done by somebody working on a PC-system. He has a network diagram in front of him on a screen. Over to the left hand side here, you see some fairly thick cables. That is standard Ethernet cabling, where a trial is being conducted between a PC (which is the Apricot-PC), through the little grey boxes which are the transceivers which take the input from the PCs, and another computer. We are down to things like speed and concurrency of movement of the data and the data-transfers, of course, always turn out to be far less than the rated data-transfer speed. Ethernet we are told is 10 megabits a second. We are still looking for the manufacturer who can deliver that.

Behind the scenes of course you have got archiving, masses of data to store and look after and that is no mean feat having to do all of that. We have database machines that handle nothing but a Pick database. Again when we came to choose a system we looked at the financial institutions in London and saw what they were using. To us, handling our financial data is a minute part of our total processing load. The suppliers came forward, saw our specification and offered us a particular style of system. We looked at it and said that sounds quite good, but we will double it in terms of power. They said you won't need that, it is far too much. We insisted they doubled the size of their offering and within months we soaked it to the point where it was hardly able to run. What I am showing you now is the latest machine that has just arrived. It is again the biggest available, the latest technology. It is the first one that McDonald Douglas have supplied in the world and which gives us, they claim, performances of 24 to 30 mips of power, just to handle one small part of our activity. And again I put the question: What

9. Testing data transfer speeds through Ethernet cables (visible to the left).



10. Design office hardware from CAD to programmable calculators.

is the rest of the world doing? If we in a small part of our data processing have to get the biggest models, where does that leave the financial institutions? Are they maybe just playing with their computer systems? I don't know. - That's a question for later maybe.

With our level of computing activity we need back-up and all sorts of other things. You are looking at a UPS-system (uninterruptable power supply). Now the point I want to make here is, as you become more dependent on the computer systems, you do not have the time to fall back onto manual systems of any sort and, therefore, you have to have all the support services in place and this is a big headache for us, to achieve all of this.

Constantly playing with wiring, with cabling, with autodial-modems, the ability to go out and communicate with offices across the world - all of this requires extra effort and support. You get very congested cabling if you go for mini computer technology with multiple colour work stations. Line analysers are needed to install and check out RS232-type of communication or whether you are into much faster Ethernet. You need different analysers for the different networks, go- faster boards and graphics boards and all sorts of lovely things.

I should tell you about telephone systems as well because the telephones have to be reprogrammed every time people move about and every time we want to introduce a new feature. We keep statistics on their use, so we know who is doing what and where extra line capacity is needed. We are also expected to exchange data with all the other systems our clients are using. For this we use various devices and always some bespoke programming. We also have a continuous training programme on all topics, which is a constant headache to us.

We have quality assurance to worry about in software, making sure that the software is correct. There are even mistakes in chips. Those of you that may well be using an early Intel 386-chip should be aware, if you have not already heard, that for a while there was a basic error in the 386-chip logic. So if you have done a lot of calculations with a 386-chip, I hope you are feeling comfortable at the moment. If you want to check with Intel they will tell you the serial numbers affected. They are printed on the chip and are easily visible.

Another subject is documentation. We help our quality assurance by paying attention to documentation. Distribution of software is a problem. We have set up a system for distributing our software with an automatic machine for copying and verifying disks. The endless supply of equipment requires staff to deal with the demand and the packing and unpacking and all the insurance requirements that we have.

And then, finally, into our offices and the sort of set-up you see: PCs, laser printers, little dot matrix printers. For the heavy specification production we get dedicated word processing systems. For the design office we have CAD-stations and more CAD-stations. I think this is an important slide to note (Figure 10). You see little programmable calculators, where software is being stored. They do not change, they are just laying about there, people will grab the particular calculator they want. So every scale of computing has its place.

Onto colour output and a particular SUN workstation. We are using the 386i workstation as our standard now and that has again only just been announced. The operating system software still has bugs, but we are working on that, and hoping to get better.

Quickly, then, I have about one minute I think, on expert systems. An expert system has been set up by a UK government body which is spending a lot of money and wanting reassurance about the way that money is being spent. So they commissioned an expert system.

We are the consultants who are advising the government body on the project which is about stabilising old limestone mineworkings. The idea is to get corroberation of our recommendations. Our knowledge was fed into the knowledge base. Not surprisingly now, the expert system tells the government body that what we are advising them is correct. So those of you who understand these systems, may not be impressed by the exercise.

In other areas of expert systems practical applications are a long way off. Simple operations with expert systems we find are taking hours and hours, very much longer than it would take to simply phone up an expert and seek advice.

I have run out of time. Thank you.

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Integration of Computer Applications in Structural Engineering

Intégration de l'ordinateur dans le projet de systèmes structuraux

Integration der Computeranwendung bei der Tragwerksplanung

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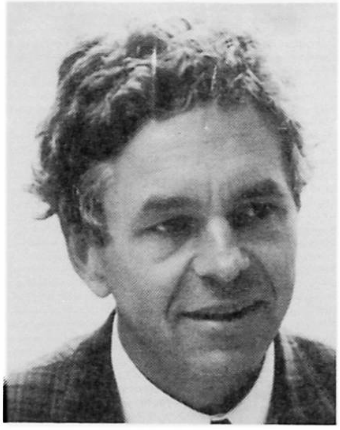

Wolfgang Haas was born in 1942, and obtained his civil engineering degrees at the Universities of West Berlin and Stuttgart. He was engaged on various projects concerning the development of EDP programs for civil engineering problems. Wolfgang Haas is technical manager of RIB/RZB.

SUMMARY
The integration of computer applications has a long tradition in structural engineering. The first attempt to achieve integrated systems was made as early as the 1960's. In contrast to this, most structural design is done today with separate programs for structural analysis and design. The paper describes the current state of the art in practical applications and trends for further development.

RÉSUMÉ
L'intégration des applications informatiques a une longue tradition dans le génie des structures. Le premier essai de réaliser des méthodes intégrées a été fait dès 1960. En opposition, la plupart des projets est couramment faite par des programmes non intégrés de projet et de calcul. Ce manuscrit explique de façon détaillée les applications pratiques de la technique actuelle ainsi que les tendances de développement.

ZUSAMMENFASSUNG
Die Integration der Computeranwendung in der Tragwerksplanung hat eine lange Tradition. Die Entwicklung der ersten integrierten Systeme begann bereits in den sechziger Jahren. Im Gegensatz dazu wird die praktische Anwendung immer noch vom Einsatz von Einzelprogrammen für den Entwurf und die Berechnung von Tragwerken geprägt. Der Bericht beschreibt den gegenwärtigen Stand der Technik der praktischen Anwendung und Tendenzen für künftige Entwicklungen.

1. INTRODUCTION:

The integration of computer applications has a long tradition in structural engineering. As early as in the sixties and seventies, the first integrated systems such as ICES (Integrated Civil Engineering System), [1], GENESYS [2] and IST (Informationssystem Technik [3] were developed.

Although these overall integrated systems were not accepted by the majority of the structural engineers, subsystems for special applications such as ICES STRUDL for structural analysis or ICES COGO for geometric computations were used to a far greater extent.

Almost parallel to these efforts towards integrated systems, a great variety of isolated programs for the analysis of structural members such as foundations, continuous beams, frames and slabs were developed. They are usually applied in a "desintegrated" manner and are the 'state of the art' software widely used for structural analysis.

The late seventies and early eighties saw the upcome of the application of graphic interactive techniques, i.e. CAD programs in structural design. They are well-established for the production of engineering drawings such as structural general arrangements, reinforcement drawings and structural steel drawings.

Today we can observe many efforts to integrate the existing landscape of disintegrated programs for structural analysis and design. First successes can be observed in the integration of CAD programs and finite element programs, for example for slab analysis or the analysis of shearing walls.

Another field of activities is to enable the exchange of CAD data between the architect, the structural engineer and building services engineers, even if they use different CAD systems.

The key to any integration is the definition of a common set of data which can be shared or exchanged between the different disintegrated programs and program systems.

2. INTEGRATED STRUCTURAL ANALYSIS:

Structural analysis by computers can be divided into the following subproblems:

- data entry of the structural system and the loads
- computation of displacements and internal forces
- superposition of load cases
- dimensioning
- presentation of the structural system and the results of the analysis.

For smaller structural problems such as continuous beam analysis, the integration of all subproblems does not represent any problem since all steps of the analysis are done within one program.

If the problems become more complicated, especially if finite element programs are used, the integration of the above-mentioned 5 subproblems is no longer self-evident. Usually, they are grouped in three programs as follows:

- the preprocessor, for graphically-oriented data entry, in many cases restricted to the generation of the finite element merits
- the structural analysis program for the computation of the displacements and internal forces
- the post-processor or the superposition of the results, the dimensioning and the graphical presentation of the results.

Typical examples are shown in figs. 1 and 2. Fig. 1 shows a computer display during the generation of an element net of reinforced concrete offshore platform.

Fig. 1 - Finite element net of 1 reinforced concrete offshore platform.

Fig. 2 - shows on the left side a graphical representation of the principal moments and on the right side a plot of the reinforcement as a result of the dimensioning postprocessing.

The graphically-orientated highly interactive preparation of input data for structural analysis programs is not restricted to the generation of finite element nets but can be applied to describe the complete structural problem including loads, supports and material properties.

In order to understand the impact of this new type of program to structural analysis, it is necessary to keep the traditional application of structural analysis programs in mind.

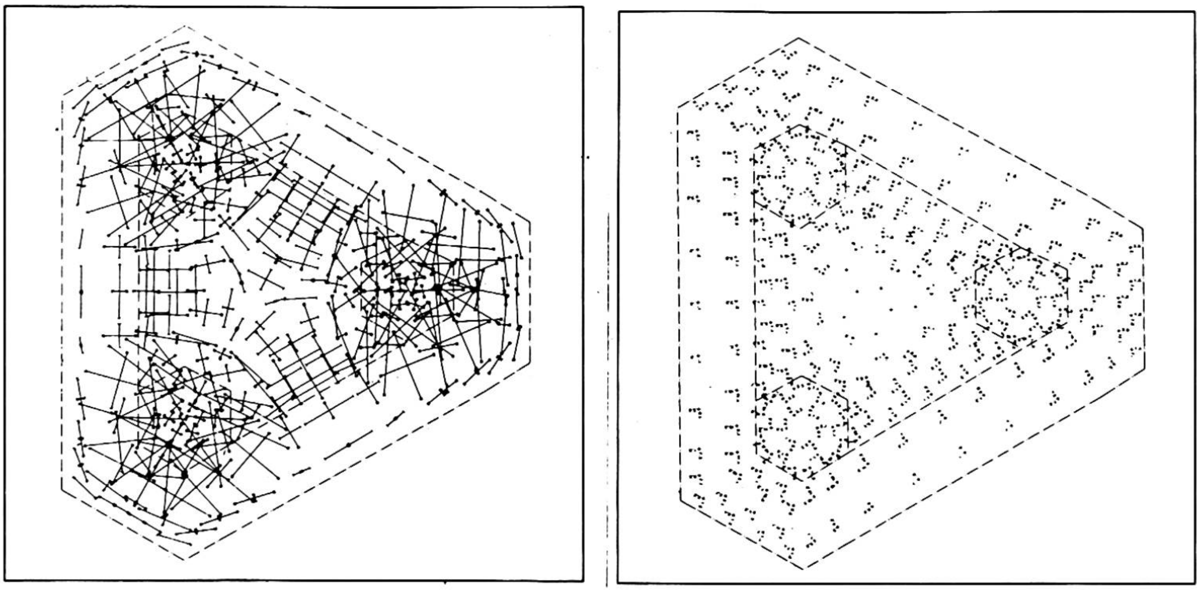

Fig. 2 - Principal moments and reinforcement dimensions for a reinforced concrete offshore platform.

The structural engineer supplies sketches of the structural system and loads. Fig. 3 shows a typical example of a plane frame.

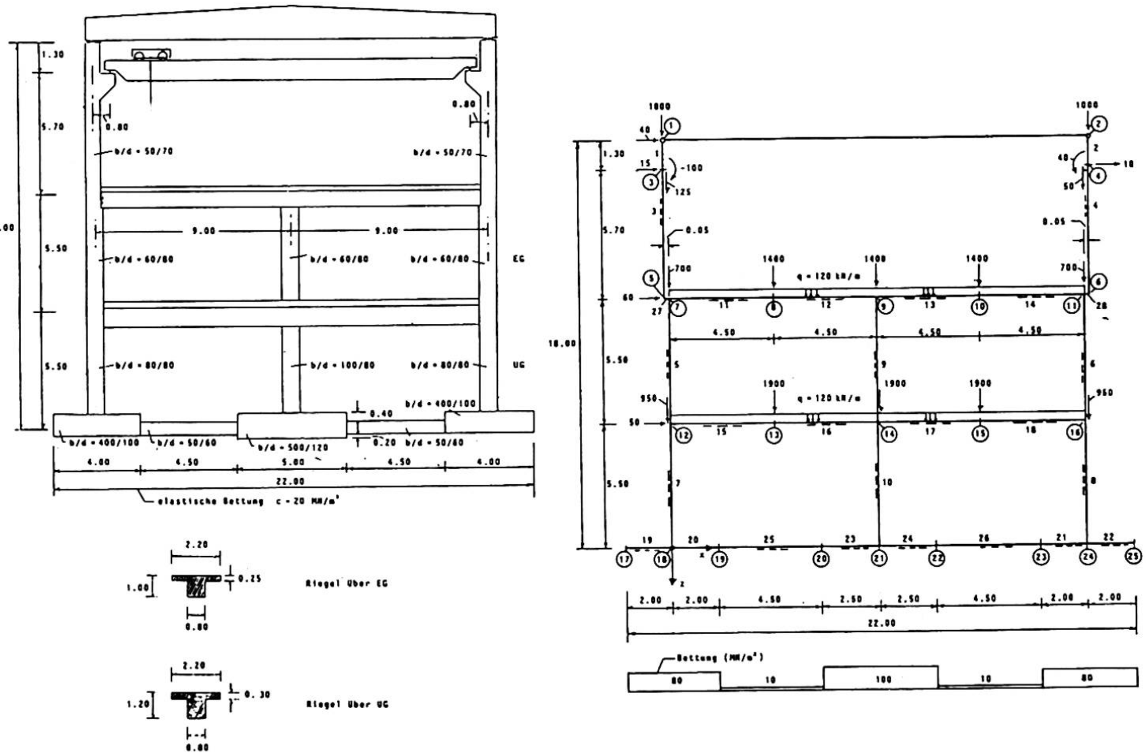

Fig. 3 - Typical sketch of a structural system with loads.

These graphically-orientated data are now translated into alphanumeric data, i.e. text and numbers. 'Graphic' is thus converted to 'alpha numeric'. With this procedure, not only has the number of data risen remarkably, but it is also susceptible to faults and very difficult to verify. In order to examine the abundance of alpha-numerical data values more effectively, so-called control plots are drawn up.

The fact that this 'historically grown' application form could be improved considerably by entering the structural system and the loads directly and graphically was first recognised at universities in the USA and transformed into prototypes for graphically-orientated structural analysis programs [4], [5]. In the meantime, such programs are commercially available.

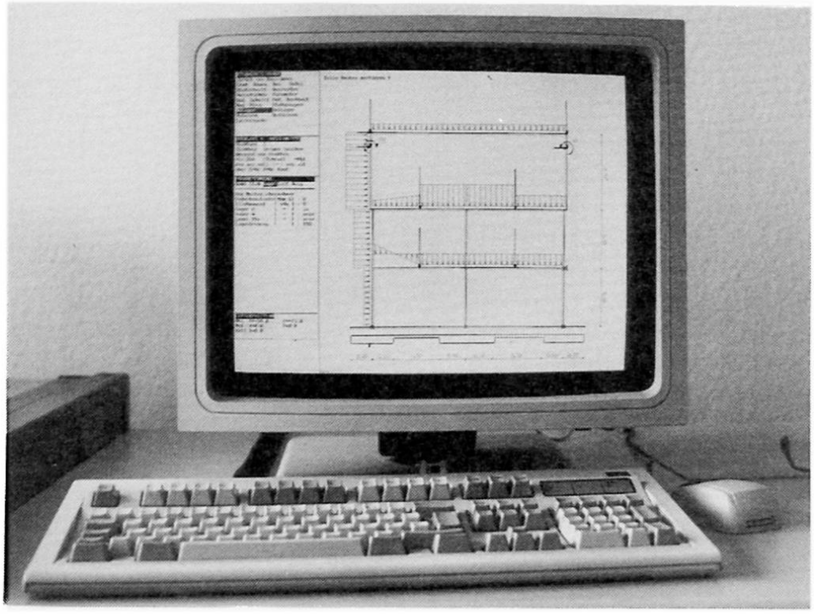

Fig. 4 - Graphic interactive preparation of input data for a structural analysis program of plane frames [6].

Fig. 4 shows a picture taken from a monitor during the preparation of input data of the presented structural system. It is evident that this manner of application is much more rational and considerably less susceptible to errors than the traditional input via alphanumeric input data.

3. INTEGRATED STRUCTURAL DESIGN:

3.1 Application of CAD programs for the production of structural general arrangements:

The CAD technology is well-accepted for the production of structural general arrangements. A reason for this good acceptance is, that this application has hardly any demands which exceed the performance of normal CAD systems for building design. With standard functions such as:

- the union of building components of the same material (i.e. walls in a floor plan),
- layer technique for the selective representation of building members and drawing items,
- the generation of regular parts of the construction,
- automated hatching of sections,
- semi-automatic dimensioning,

even complicated structural general arrangements can be created economically.

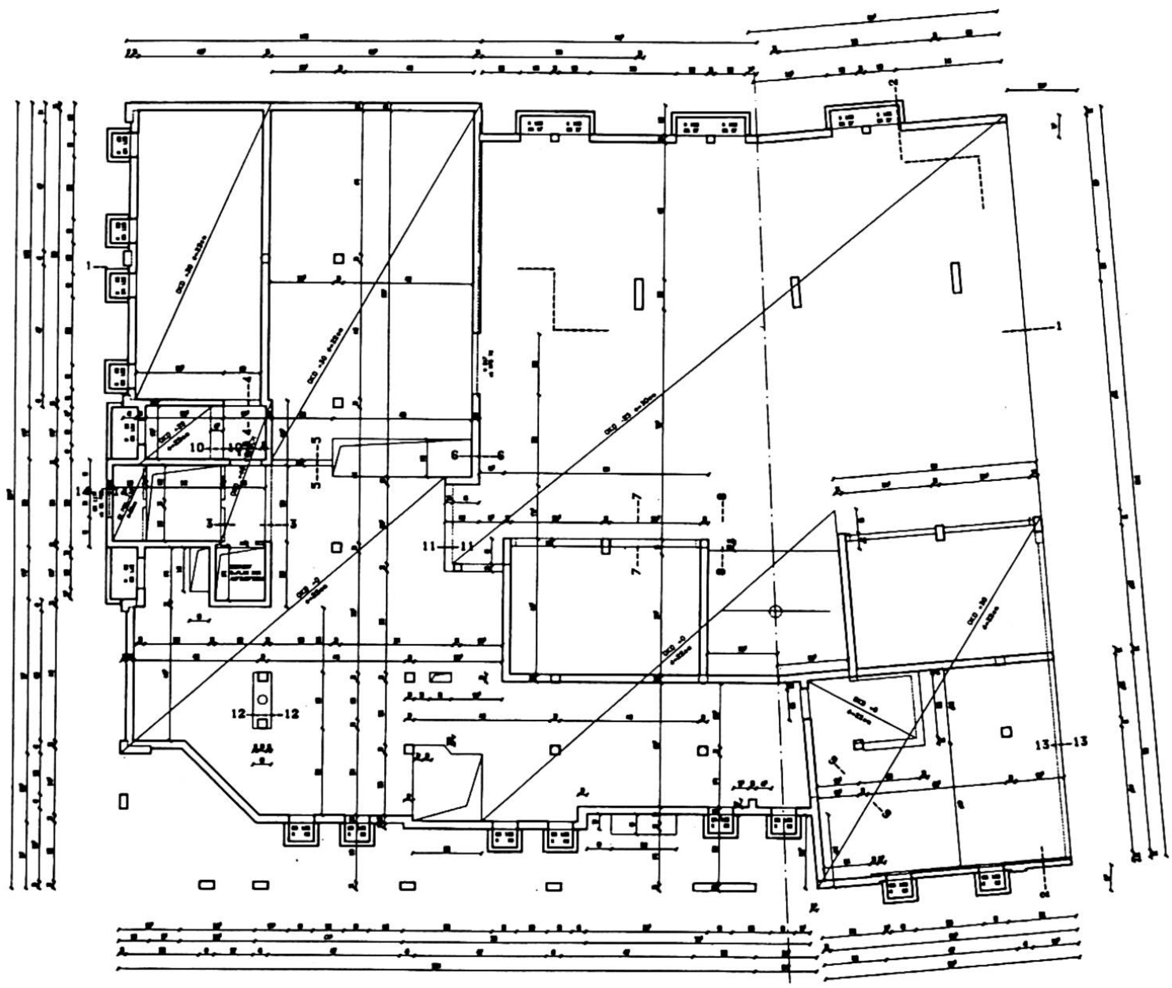

Fig. 5 - Floor plan, generated with CAD

If the structural situation is mainly prismatic, i.e. in the case of a floor plan (fig. 5), 2D systems are surely sufficient. If the structural situations become more complicated, i.e. in the case of the foundation of power plants or other engineering constructions or if several sections must be drawn in order to describe a structural situation completely, such as in stairways (fig. 6), then the application of 3D systems is more advisable. With such a 3D system, a 3D building model can be generated in the computer from which sections can be derived automatically. If the structural situation becomes very complicated, it can even be inspected in a perspective representation.

3.2 Application of CAD programs for reinforcement drawings:

Programs for reinforcement drawings were developed as early as the seventies. The starting point was the fact that for many programs for structural analysis, the geometric dimensions of the structural members are available as input data.

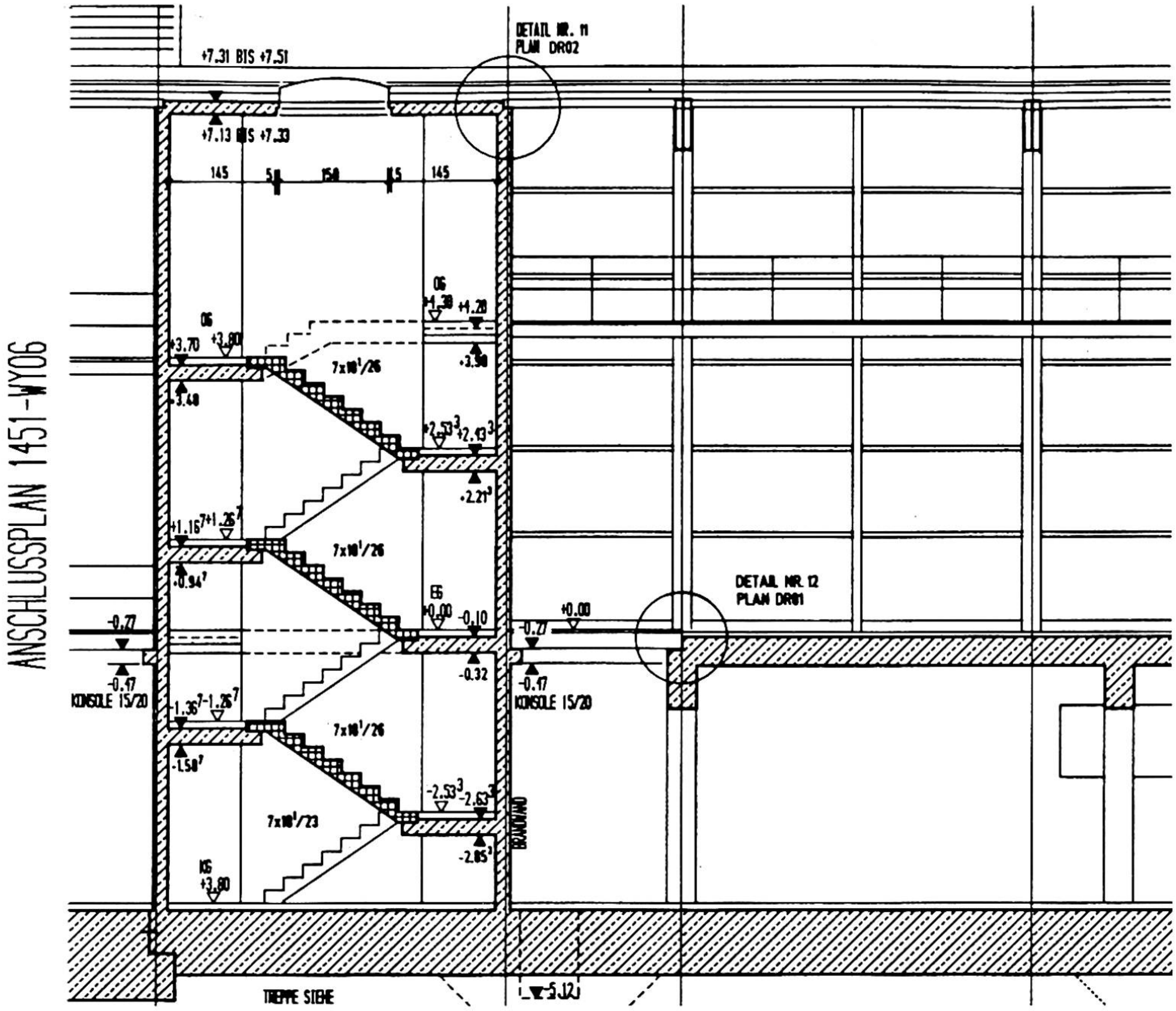

Fig. 6 - Stairway generated with CAD out of a 3D building model.

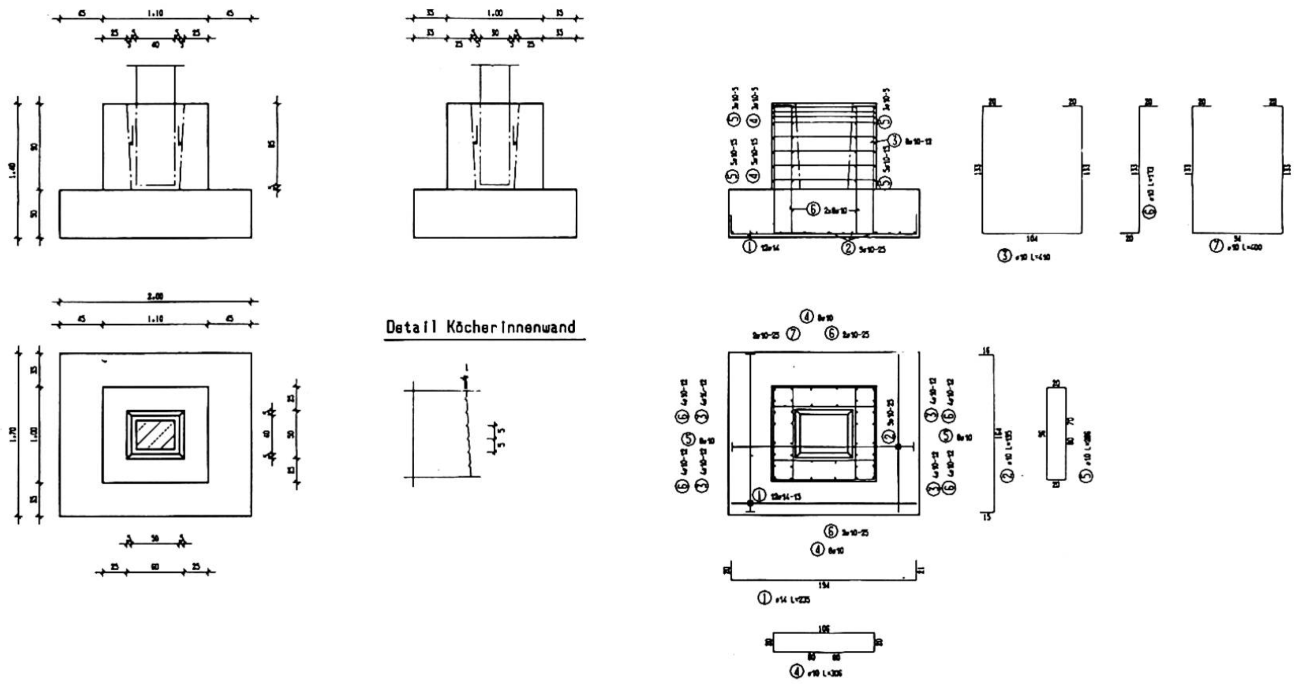

Fig. 7 - Reinforcement drawing of a foundation

The dimensioning parts of the programs calculate the cross-sections of the reinforcement. If the user provides additional data, such as preferred reinforcement diameters, reinforcement forms, etc, then reinforcement drawings can be produced automatically. The main applications of these parametric design programs are standardised building members, such as pre-fabricated, concrete members, foundations (fig. 7), columns and beams. It should be emphasized at this point, that these programs are not CAD programs as they are currently understood, since CAD techniques such as graphic interactive input are not used.

When the structural situations become more complicated, then the preparation of input data for the parametric design programs becomes a time-consuming, cumbersome task, where many errors may occur. It is here that CAD techniques, i.e. graphic interactive techniques, have definite advantages compared with the parameter controlled input techniques.

For the graphic interactive design of reinforcement with CAD programs, special input techniques have been developed. The necessity for such special input techniques will be demonstrated by means of two examples.

The first example concerns the arrangement of web reinforcement within a flat slab (fig. 8). At first glance it seems to be a generating problem which can be solved with standard CAD techniques. If one considers it more closely, then one recognises that special techniques have to be applied here in order to be cost-effective. The web reinformenent is arranged in longitudinal and transversal direction with overlap. If several strips are laid in longitudinal direction, the overlaps can be grouped alternatingly or non-alternatingly. If the web reinforcement overlap the floor strips, one should have the choice to cut the web reinforcement on the borders or to evenly distribute the overlaps.

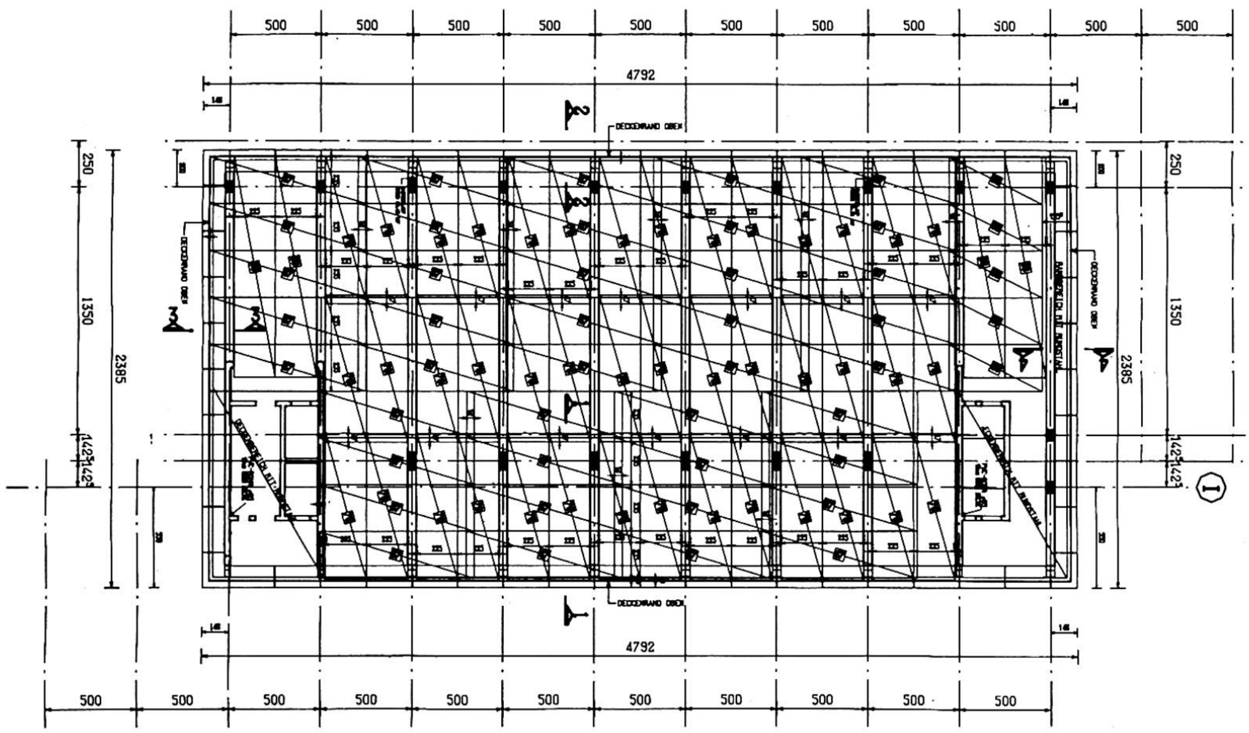

Fig. 8 - Reinforcement drawing of a flat slab

Concerning bar reinforcement, the CAD program must allow the user to describe the shape of the reinforcement bar freely or relative to the shape of the building member. For both types of reinforcements, web reinforcement and bar reinforcement, special lists must be produced automatically. These two examples only show a small number of the special problems to be solved.

Such CAD programs for the design of reinforcement are not restricted to standardised building components, but can be applied for the design of the reinforcement of arbitrary building components. Beside flat slabs (fig. 8), even special constructions such as tunnel pipes, communication towers or bridges (fig. 9) can be dealt with.

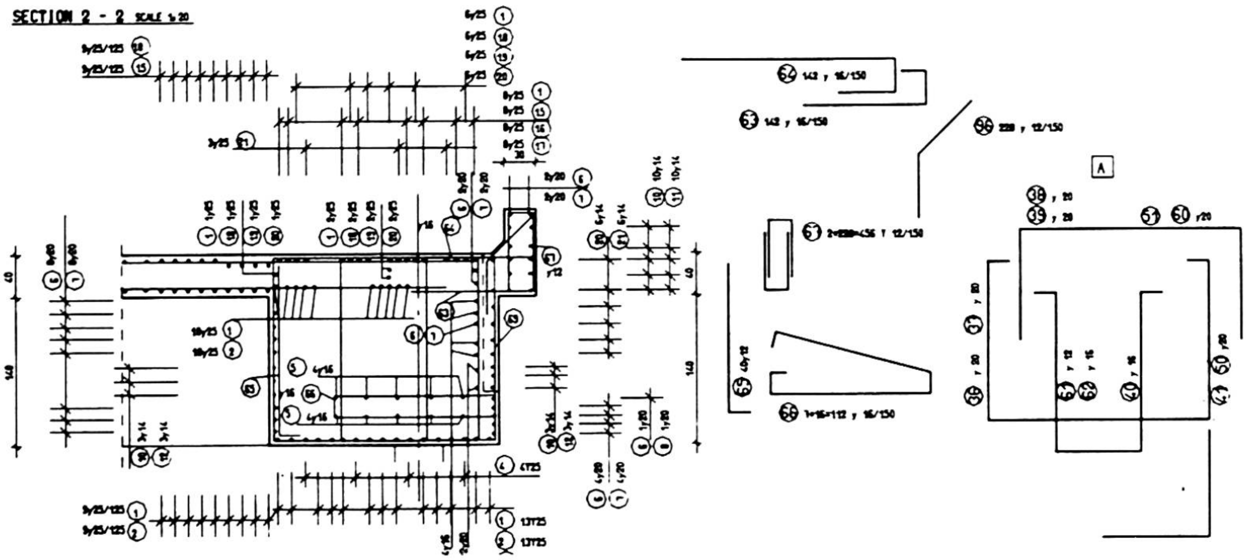

Fig. 9 - Reinforcement drawing of a detail of a bridge

3.3 Integration CAD - structural analysis:

In all applications of data processing to building design, there is a strong trend towards integration. In structural design the first integrated systems were based on the parametric design technique for reinforcement drawings. A typical example is the integrated program system for continuous beam design, described in [7].

Today, many CAD programs provide the possibility to describe finite element nets, loads and supports for structural analysis programs relative to architectural drawings or general structural arrangements.

The first application of this type of integration was the integrated analysis and design of flat slabs [8]. One prerequisite for this integrated design was that the finite element method could be established in West Germany as a standard method for the analysis of flat slabs.

Recently, a similar effective finite element program has been developed for the analysis of walls with openings. The problems of the structural analysis of walls in building design will be described on the basis of fig. 10. The wall has regularly- or irregularly-distributed openings. This structural behaviour of parts of the wall resembles beams. Slabs are connected to the wall. Obviously, this kind of structural systems cannot be analysed with programs, based on classical plane stress analysis in a cost-effective manner.

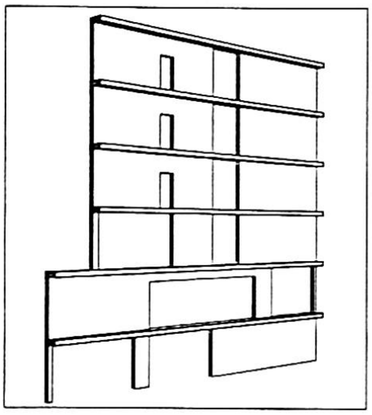

Fig. 10 Typical concrete wall

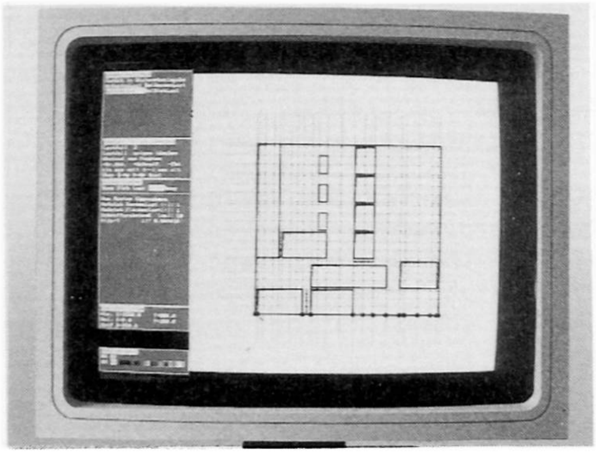

Fig. 11 - Graphic interactive input of a concrete wall

The recently-developed program for the analysis of concrete walls [9] is based on a modified hybrid plane stress finite element [10]. The parameters of this element can be determined in such a way that it has the exact stiffness of a uniaxial beam element with arbitrary cross-sectional values. This program allows one to model complex wall structures such as the one shown in fig. 10 with few elements and nonetheless obtain accurate results which can be used to directly calculate the reinforcement.

The structural design starts with the definition of the finite element net relative to the architectural drawings or a general structural lay-out. Fig. 11 shows the graphic screen during the preparation of these input data. Extensive generating possibilities are provided which allows one to define the element net economically. The support conditions, the loads and additional information for the calculation of the reinforcement can be described in a similar way.

The program for structural analysis gives the following results:

- graphical representation of the structural system;
- displacements;
- internal forces;
- dimension of the reinforcement.

These results can be shown in listings or graphically (fig. 12). These results allows one to check the analysis easily and to design a reinforcement which corresponds to the actual structural behaviour.

4. INTEGRATION OF ARCHITECTURAL AND STRUCTURAL DESIGN:

4.1 Organisational aspects:

The building design is interdisciplinary. The architect draws the lay-outs in which he first presents the rooms, their dimensions, functions and equipment. The structural engineer calculates the structure and prepares the structural general lay-out and reinforcement drawings. The Individual engineers responsible for heating, ventilation, air-conditoning, sanitary and electrical technology draw their respective installation plans on the basis of the architectural drawings or the structural general lay-out. This type of planning is presented schematically in fig. 13.

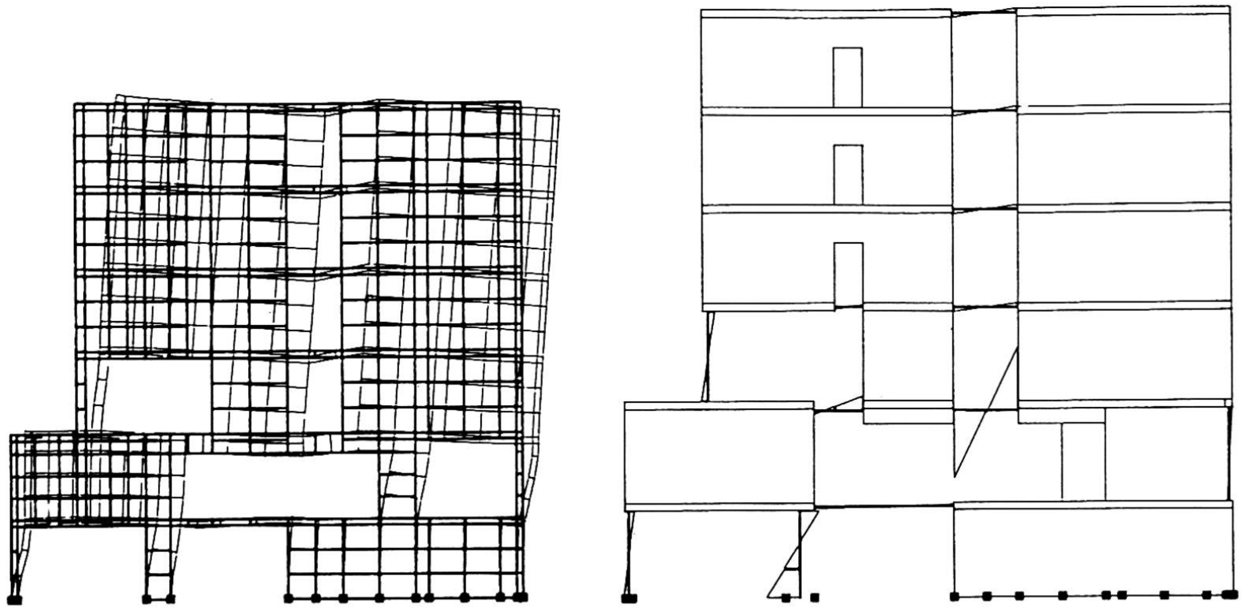

Fig. 12 - Examples of results of the concrete wall analysis
left : displaced structure
right: bending moments in "beam-like" members.

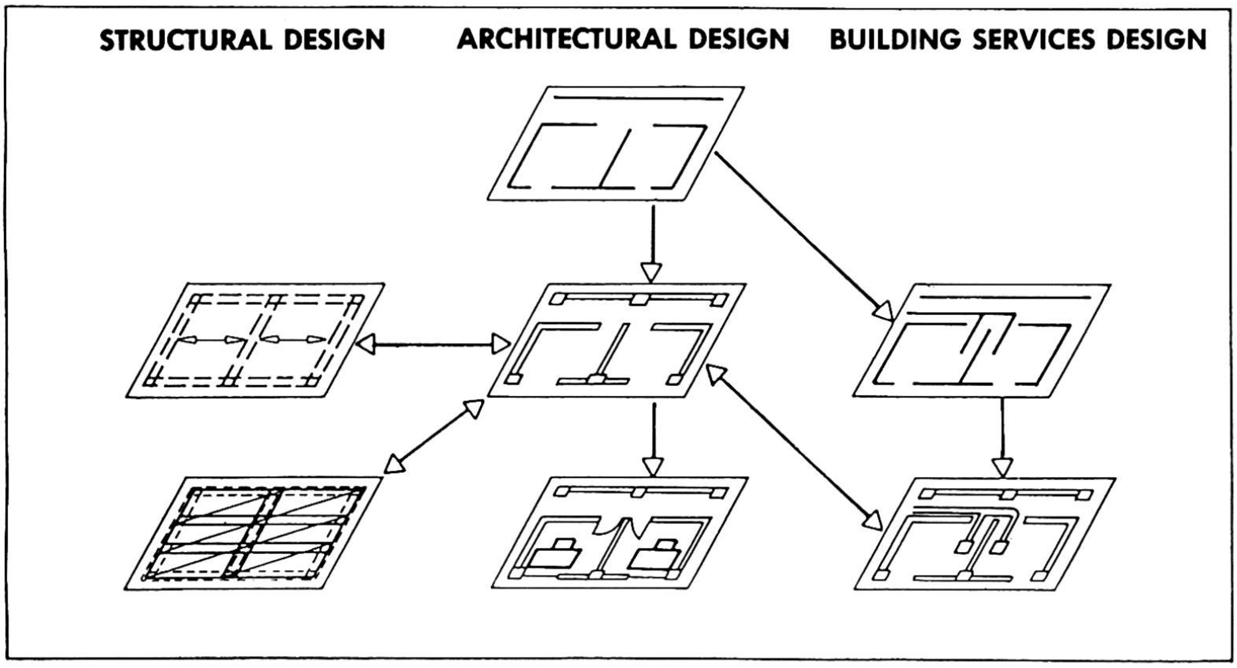

Fig. 13 - Interdisciplinary building design

In building design, particlarly for industrial buildings, the layer technique can be applied effectively to this interdisciplinary design process. With this layer technique a drawing, as presented in fig.14, can be divided into different transparent layers. These layers can be overlaid arbitrarily. Each of the architects and engineers involved in a common building project may visualise the layers of the other architects or engineers and thus coordinate the planning of his part in a better way, than with conventional methods.

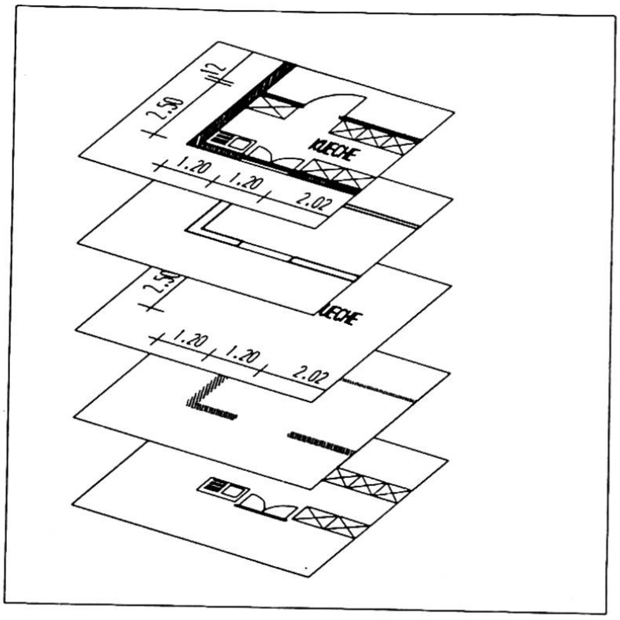

Fig. 14 - Layer technique

A good layering system and classification is decisive for a successful application of this technique and a prerequisite for an integrated building design. If this classification is carried out effectively, then the ideal that no line must be drawn twice can almost be achieved. This is explained in fig. 15.

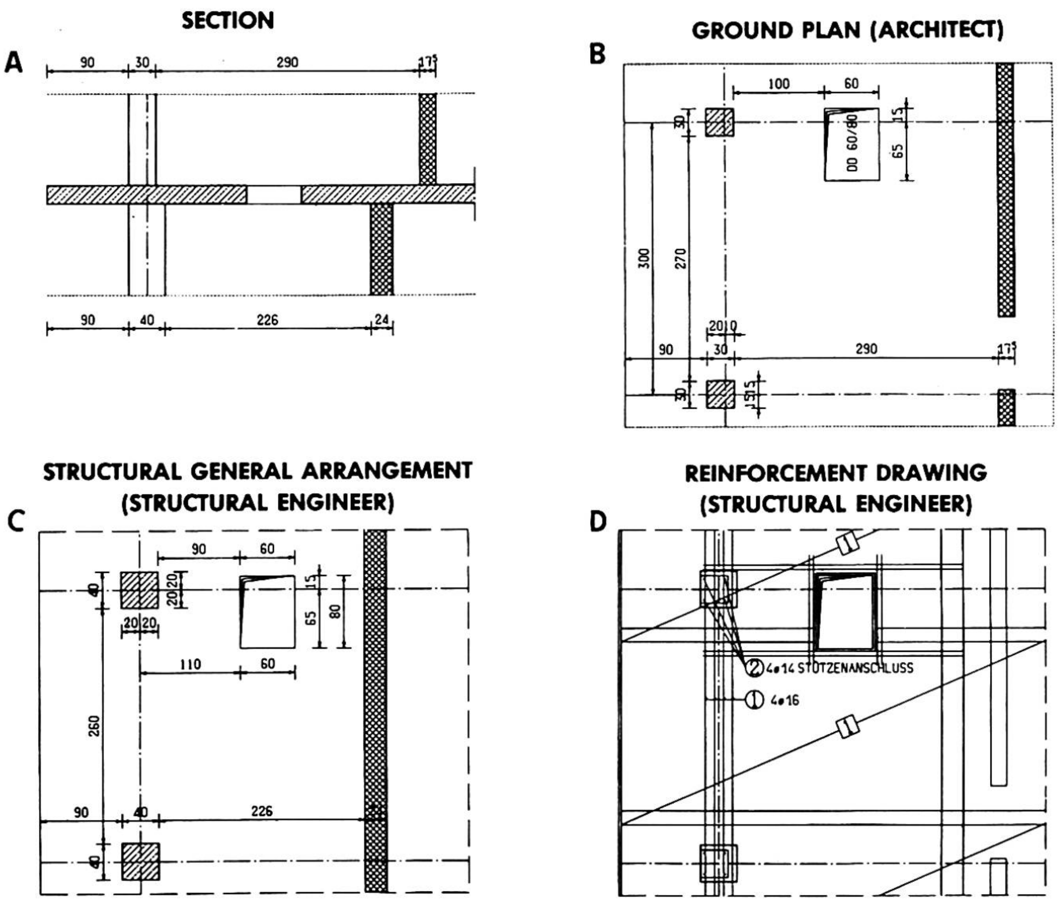

Fig. 15 - Application of the layer technique

Fig. 15A shows the vertical section of a typical building situation. The cross-section of the columns changes from storey to storey and the inner walls are shifted.

Fig. 15B shows the corresponding architectural lay-out. It also presents the slab together with its openings and walls and columns which are located above it.

Fig. 15C shows corresponding part of the general structural lay-out representing columns and walls beneath the slab together with the slab.

In the reinforcement plan, presented in fig. 15D, all building components are shown in many cases. Beside the building components below the slab, one also needs the building components above the slab for the connecting reinforcement of the columns and walls above the slab.

What can be done in order to reach the ideal case of each line having to be drawn once only in those 3 plans? One must present the building components below the slab, the slab itself and the building components beneath the slab, each in separate layer as well of course as the hatching and dimensioning. Only then does one obtain the architectural plan overlaying the layers of the slab and the layer of the building components above the slab, the general structural lay-out by overlaying the layers of the slab and the layer of the building components below the slab and the reinforcement plan by overlaying all three layers.

The effective application of the technique within one building project requires, however, an agreement between all involved architects and engineers upon a uniform naming and administration of the layers. Only then is it guaranteed that the structural engineer will actually find the requested lay-out of the architect or that the installations engineer will find the desired layer of the structural engineer or architect. Reports [11] and [12] will give an indication of the organisational problems which must be solved when the overall planning within a planning company is carried out on one CAD system within an office. Should architectural, structural and installation planning be carried out in different offices with different systems, then an additonal problem to overcome would be the transfer of CAD data between different CAD systems.

4.2 CAD Data Exchange:

Before discussing the situation in the AEC (Architecture, Engineering, Construction) industry, it is useful to look at the situation in other industries.

In Germany, several neutral exchange formats are used outside the AEC industry.

- VDA-FS [13] (Format for the exchange of surfaces for the automotive industry)

- VDA-IS [14] (IGES subset for the automotive industry).

It is interesting to note that inside the automotive industry a subset of IGES is accepted and used as an intermediate format, to exchange CAD data between different types of CAD systems.

Inside the German AEC industry IGES [15] or other neutral exchange formats [13] and [14] are not accepted. If exchange of CAD data is practised at all between different systems, then only with the help of so-called direct translators which transfer the 'sending' CAD system format directly into the 'receiving' CAD system data format. Reports on practical experience are shown in [16] - [18].

The situation abroad is similar. With regard to the exchange of CAD data in Great Britain [20] and in the USA [21], two such reports show experience in these countries. Here also the existing neutral formats are almost not accepted. Direct translators are mostly used.

In 1986 a special AEC committee was founded in Germany as a subcommittee of the German DIN committee, which contributes to the ISO standardisation concerning this topic. The goal of this AEC committee is to establish a neutral data format for the exchange between different CAD systems suitable for AEC purposes on the basis of national and international standards.

This committee consists of representatives of all types of institutions engaged in building design. The committee decided on a two-step approach in order to establish the neutral exchange format:

- rapid development of a 2D exchange format on the basis of existing standards until the end of this year;

- in 1988 beginning of a contribution to the ISO work and discussion of existing reference models.

A first version of the 2D exchange format, based on the international standard STEP with the name of STEP_2DBS (STEP 2D building subset) is currently available.

The future standard STEP was chosen as a basis, since this format will be established in the nineties. Then STEP_2DBS can be extended to exchange 3D data an alphanumeric information in order to become a comprehensive product definition exchange format for AEC purposes.

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Inspection, Assessment and Maintenance

Surveillance, évaluation et maintenance

Überwachung, Zustandbewertung und Unterhaltung

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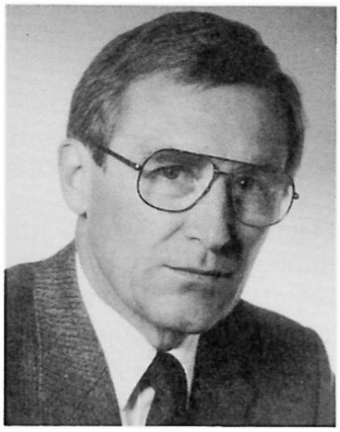

Manfred Wicke, born 1933, graduated in Civil Engineering and took his Dr.techn. degree at Vienna Technical University. For a 12 year period he firstly worked in and later on headed the design office of a firm. He was mainly involved in design work of buildings, bridges and power plants. Since 1971 he is full professor for concrete structures at Innsbruck University. Since 1977 he inspected more than hundred bridges.

SUMMARY
This introductory paper deals with the inspection of existing structures as well as the evaluation of established damages and defects and the overall assessment of its maintenance condition. These factors are the basis for the determination of appropriate methods for maintenance and repair. The diverse influential factors are described here as well as inferences which can be made with regard to applicable procedures. Using examples, particularly from the field of bridge construction, a picture of the present status of inspection procedures will be provided.

RÉSUMÉ
Cette introduction traite du contrôle des bâtiments déjà existants, de l'évaluation des dommages et des insuffisances constatés, ainsi que de l'analyse de l'état actuel. Ces données constituent la base pour le choix des mesures de réparation à prendre. L'auteur décrit les différents facteurs d'influence et en déduit les procédures à appliquer. Quelques exemples, notamment ceux du domaine de la construction des ponts donnent un apercu de l'état actuel des possibilités de contrôle.

ZUSAMMENFASSUNG
Diese Einführung behandelt die Überwachung von bestehenden Bauwerken sowie die Bewertung der festgestellten Schäden und Mängel und die Gesamtbeurteilung des Erhaltungszustandes. Diese stellen die Voraussetzung für die Auswahl geeigneter Instandsetzungsmaßnahmen dar. Es werden die verschiedenen Einflußgrößen beschrieben und daraus Folgerungen für die anzuwendenden Verfahren abgeleitet. Anhand von Beispielen, insbesondere aus dem Brückenbau, wird Einblick in den gegenwärtigen Stand der Überwachung vermittelt.

1. INTRODUCTION

Over the past few years, the question of the structural maintenance of buildings has gained substantially in importance. Even a decade ago, primarily historical buildings or railway bridges were mentioned in connection with the term "maintenance". Bridge construction provided the point of departure for other structural engineering work being confronted with the subject of maintenance, and the scope of that theme spread to cover buildings, even residential buildings. At this time, one would even be justified in stating that the question of structural maintenance will eventually play a lesser or greater role in all types of constructional works.

One of the reasons for the growing interesting in maintenance can certainly be traced to the tremendous building boom which has taken place throughout the world since World War II. Initially, it was the industrialized nations which began with the re-constructional activities; it did not take long for the pre-war stand to be attained and the amount of construction work being done far surpassed anything previously. Furthermore, active building began in all developing nations with the close of the colonial period. With the exception of short-term economic slumps, the last four decades have witnessed florishing building activities on a worldwide basis, which have brought about a tremendous increase of existing structures.

It is relatively easy to estimate the economic importance of structural maintenance. If one assumes, in a more or less random fashion, that maintenance costs amount to approximately 1 % those of rebuilding, then after a half a century of consistant construction activity, one-half of the annual building budget would be utilized for carrying out maintenance measures. When details with regard to the actual situation are known, it is possible to calculate differentiated predictions of the annual maintenance costs; such a calculation was, for example, carried out for the bridges of the West German Federal Highway system (Fig. 1).

Since that estimate was made, many owners, expecially highway administrators, have recognized the enormous amount of funds which will be necessary for maintenance in the future. This situation has naturally led to the circumstance that innumerable international experts and associations are increasingly directing their efforts toward answering the questions posed by structural maintenance. The catalog of questions on this subject is comprehensive as well as covering many levels.

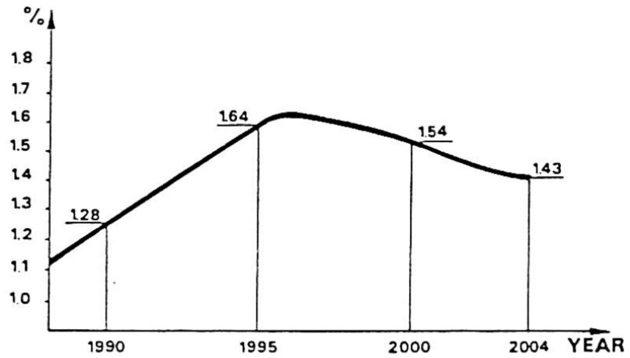

Fig. 1 Predicted annual maintenance cost in percent of rebuilding [acc. 1]

The following discussion is an attempt to present the methods and procedures which are available and being utilized at this time for structural maintenance.

The purpose of all structural maintenance procedures is ensuring that the intended lifetime of a structure is maintained throughout the entirety of its functional life. It is therefore necessary to prevent the reliability of a structure from sinking below a predefined limit. Eventual reductions in reliability must be recognized at as early a stage as possible, to allow the timely commencement of maintenance and repair work. This can be accomplished with regularly scheduled inspections of the structure, or inspections subsequent to accidental or extraordinary circumstances. The maintenance status of the structure which was determined during the inspection can be utilized in conjunction with subsequent, more detailed inspections to present a basis for maintenance and repair decisions.

The questions of maintenance do, however, not only concern themselves with existing structures, but als have repercussions on new construction works. Structures which are easy to maintain continue to gain in importance, since apparently, one is beginning to realize that the economy of a structure is calculated by means to the overall costs throughout its functional life; the overall costs are comprised of the initial expenses as well as those which are incurred for maintenance.. The opinion that only the minimization of the initial building costs is important should finally be laid aside.

2. CATEGORIZATION

It has proven effective to subdivide the extensive field of structural maintenance according to diverse criteria. In so doing, one should utilize at least four categories, such as:

- Type of construction
- Type of structure
- Environmental conditions
- Maintenance procedures

2.1 Types of Construction

The type and extent of necessary maintenance measures are extremely dependent on the type of construction involved. During the categorization phase, it is recommended that the division be made according to convential methods, for instance:

- Stone construction
- Masonry / brick construction
- Reinforced concrete construction
- Prestressed concrete construction
- Composite construction
- Steel construction
- Timber construction
- Other

Within certain fields of construction, for example steel construction, structural maintenance already enjoys a long history. Other fields, such as concrete construction, have only begun to consider the problem of structural maintenance recently.

2.2 Type of Structure

Here it is also recommended that the conventional categorization methods are followed. One should determine between:

- Bridges
- Civil engineering works
- Buildings and
- Residential buildings

It was primarily bridge construction/engineering which provided the foundation for concern with the questions of structural maintenance. Naturally, a vast amount of experience and information is already available in the field of railway bridges, whereas road bridge construction has only recently begun to consider these questions.

2.3 Classification of Environmental Conditions

Environmental aggression continues to gain in importance. The intensity with which the environment attacks structures covers an extremely broad range, making it logical to classify the aggressivity of the environment. The following categories have developed in the field of concrete construction:

Class 1: Dry environment
Class 2: Humid environment
 a) without freezing
 b) with freezing
Class 3: Humid environment with freezing and use of road salt
Class 4: Marine environment
 a) without freezing
 b) with freezing
Class 5: Aggressive chemical actions
 a) mild attack
 b) intermediate attack
 c) severe attack

These classes can also be utilized for any other types or methods of construction.

2.4 Maintenance Procedures

For this section, one can apply the time at which necessary maintenance procedures must be carried out to enable making a subdivision. The procedures begin with the inspection and documentation of the structure's present condition; these are then followed by the evaluation and assessment of the situation encountered. These factors represent the prerequisites for the selection of the appropriate maintenance measures. Thus, a distinct separation is made between the initial inspections and the selection of the necessary maintenance activities. Maintenance is then divided into servicing and repairs; the former shall be defined as measures to maintain the expected condition, whereas the latter means the returning to that condition.

3. INSPECTION PROCEDURES

3.1 Inspection Schedule

The nature and extent of the necessary number of inspections shall be established in an inspection schedule. This schedule should be determined by those executing the project and turned over to the owner at the time of structure's acceptance. The persons responsible for carrying out the inspections and the scope of associated responsibilities should be established within the schedule as well; in most countries, legal regulations determine that the owner himself is responsible for the structure's maintenance. In addition, the time intervals between the regular inspection tours should also be established. It is advisable that diverse kinds of inspections which differ in terms of extent, qualifications of personnel, as well as in frequency be carried out.

Within the field of bridge construction, detailed working inspection schedules, which, in some countries, have attained the status of obligatory guidelines or standards, are already being utilized. Such inspection schedules generally recognize several different types of inspections, as well as continuous monitoring by road service crews (Fig. 2).

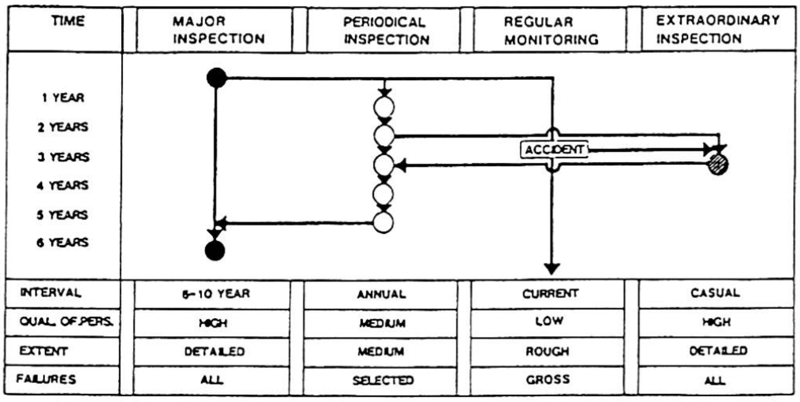

Fig. 2 Inspection schedule for bridges

Major inspections take place every six or ten years, whereas the intermediate inspections are carried out on an annual basis.

The personnel responsible for the major inspection should be more highly qualified - at least the head of the inspection team must have had an appropriate education to permit his determining what effects the detected defects and damages could have on the carrying capacity of the structure. The person or persons responsible for the annual inspection need not have such a high degree of schooling, but must be capable of detecting and describing damages as such; usually, special training is sufficient.

The major inspection involves close inspection of all accessible surfaces of the superstructure as well as the substructure. Generally, simple tests, such as measuring the concrete cover, estimating the strength of the concrete with a rebound hammer, determining the depth of carbonatization or measuring crack widths, are carried out as well. Should the inspector suspect that something is amiss, special tests would include drill core tests, determination of chloride penetration depth, measuring length and deflection, ultrasound tests, endoscopic and radiographic tests, measurement of the electric potential, as well as taking dynamic tests. Special tests, like the dye penetration procedure and the magnetic powder method, are used for steel and/or composite structures to detect cracks; the thickness of the coatings being utilized is also measured. Annual inspections are limited to determining new damage and/or the progress of existing damage. In addition, selected damage sites, for instance cracks, can be measured on a predetermined basis.

It would be advantageous if similarly detailed inspection schedules could be established for civil engineering works as well. For standard buildings, on the other hand, it would suffice to provide simpler procedures. Here it would be vital that the project contractor indicate all points of the structure which deserve particular attention, as well as establishing the type of inspection to be carried out, its extent and time intervals.

Regardless of construction type, any structure which has undergone unusual circumstances such as fire, vehicle collision, avalanche activity, etc., must undergo a detailed inspection analagous to the major inspection described above.

3.2 Equipment and Tools

The kinds of equipment and tools used during the inspection procedure depend on the type of structure. The equipment is used to reach otherwise inaccessible surfaces on sections of the structure to enable visual inspection. In the field of bridge construction, specially designed bridge inspection devices which allow direct inspection of the underside of bridges as well as the upper zones of the supporting piers and abutments from the roadway have been developed (see Fig. 3 and 4).

Fig. 3 Placing of inspection device

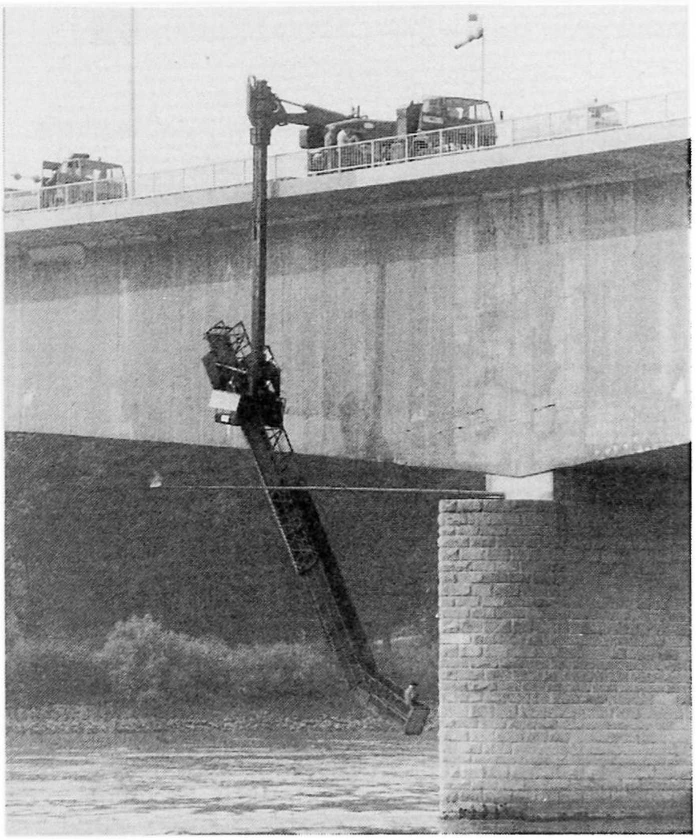

Fig. 4 Inspection device in working position

By using an auxiliary device, a suspended basket, it is possible to inspect entire piers from top to bottom (see Fig. 5). Lifting devices are also used to reach the undersides of bridges from the ground surface below, though the use of such equipment requires adequate access roads. A more modern version of such lifts enable inspection of the bridge understructure from the roadway (Fig. 6).

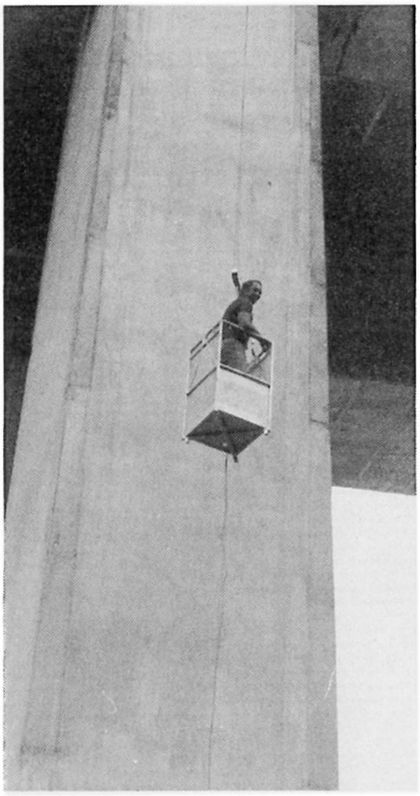

Fig. 5 Suspended basket

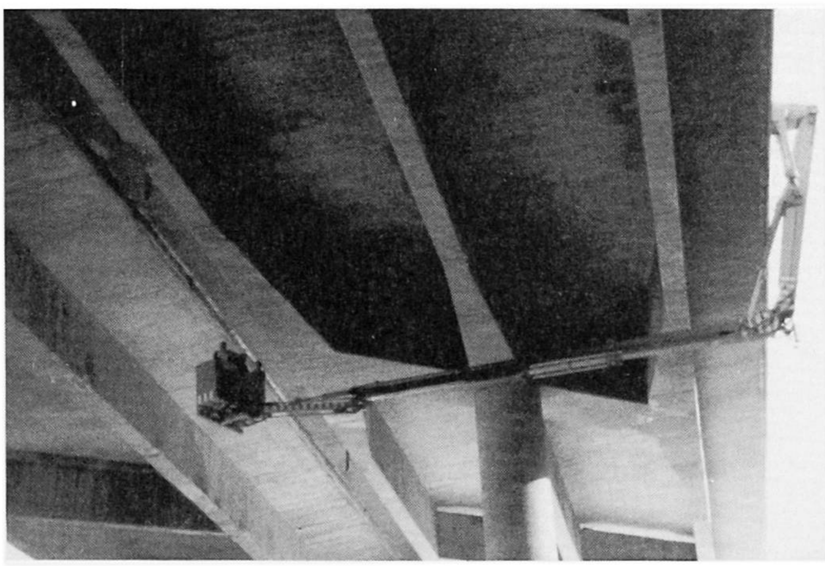

Fig. 6 Lifting device

Inside buildings, access to the surface of the construction is usually hindered by facings or other interior works. It is therefore necessary to plan that facing be removable at critical sites. Generally, a ladder is sufficient for the inspection of the structure at such locations. Outer surfaces could be reached by means of lifting devices or inspection platforms installed directly on the building.

Simpler tools for inspection purposes such as measuring devices, those for on-site surface inspections or for taking samples shall be carried by the inspection crew. The inspection of steel structures requires additional tools to ascertain the condition of joining materials. Furthermore, smaller testing apparatus such as crack microscopes, endoscopes, magnetic detection devices, extensometers and coating thickness gauges are required. Rapid information with regard to the depth of carbonatization and/or chloride content is also gained by the use of indicator solutions. To carry out the special inspection procedures discussed above in Section 3.1, specific sampling devices must also be taken along by the inspection crew.

4. DOCUMENTATION

4.1 Purpose of Documentation

Documentation should provide sufficient information with regard to the maintenance condition determined during inspection activities. These records serve as the foundation for the evaluation of the structure. They should also enable consequent and precise determination of eventual changes in the damage situation by recording information taken at periodic inspections. To accomplish these goals, the maintenance situation shall be recorded in a number of ways.

4.2 Graphic Representation

Graphic representation is particularly suited in providing a rapid summary of the situation and should therefore be included in any sort of documentation. Overstated drafting precision is not required, since schematic drawings are sufficient; for example, bridges which are curved according to sideview and/or overview can be drawn as straight lines. It is far more important to select a method of presentation from which the greatest amount of information can be obtained. The damage diagram of a T-beam bridge has been chosen to illustrate this point (Fig. 7).

On this diagram, the cracks on both vertical surfaces were drawn into only one view of the web, though different markations were used for either side, whereby cracks resulting from shear were clearly clearly distinguishable from those caused by torsion. Had one, on the other hand, mapped the cracks on a developed view of the surfaces, the patterns would probably not have been so distinct. Moreover, it is also advantageous to present the structure it as it is viewed from the inspection device.

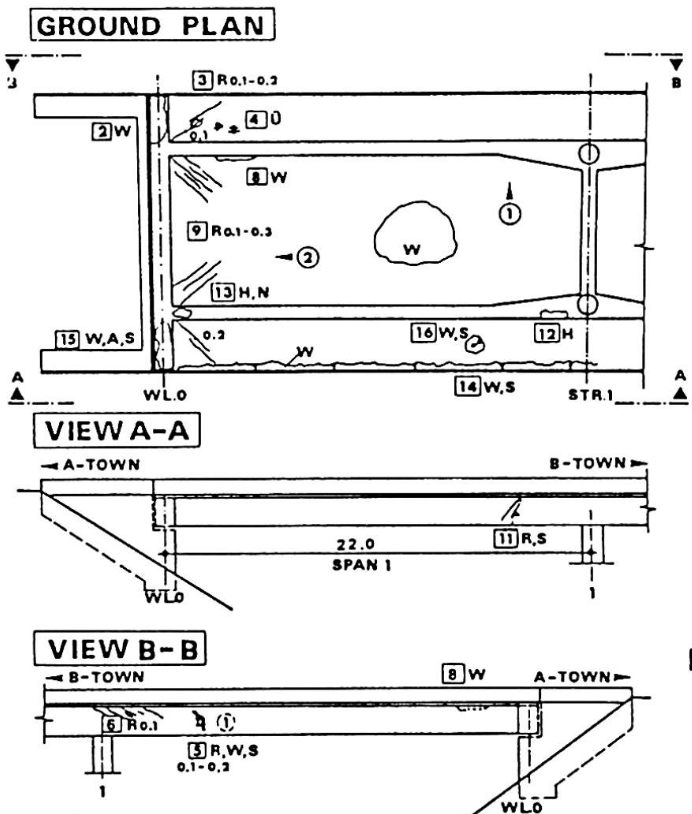

Fig. 7 Schematic drawings

It is not usually necessary to take exact measurements of the damage site. The drawing should, however, facilitate finding the location of the damage site on the structure as well as serve as the basis for evaluations. In the field of bridge construction, for example, it is often sufficient to provide meter markations on the girders and piers, and then approximating the position of the damage site between two such marks. More precision is not necessary for drawings used for this purpose. In contrast to this rule, the location of damaged tendons must be drawn in exactly as measured to enable their identification.

To increase the amount of information which can be contained on such a drawing, texts should be avoided and symbols indicating the individual types of damage should be used. It is immaterial what symbols are chosen to represent damages, though they should not require extensive explanations to make them understandable. A standardization of such symbols should, however, be desirable, to facilitate informational exchange (Fig. 8).

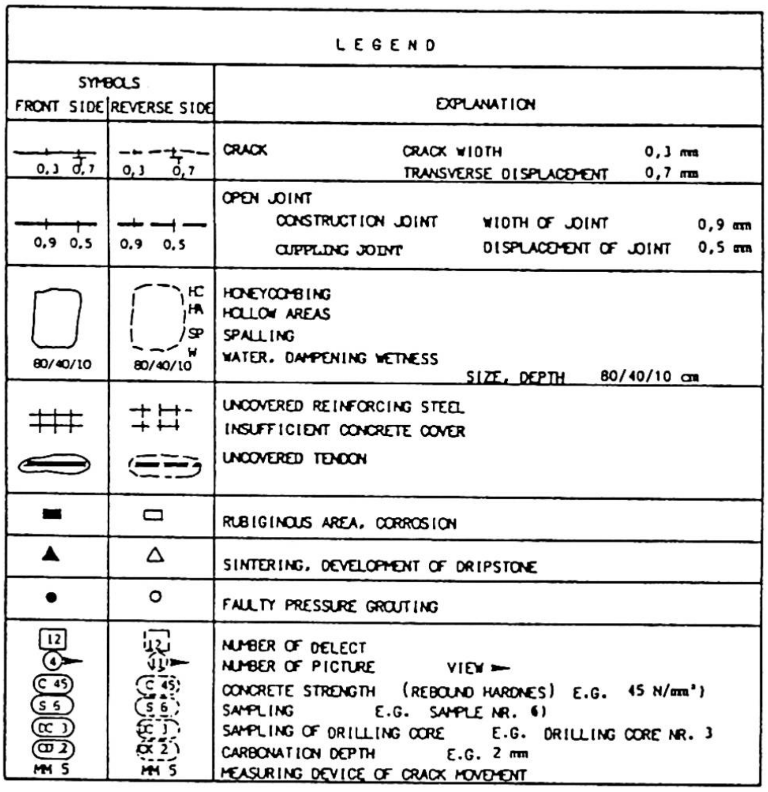

Fig. 8 Symbols of defects

4.3 Listing and description of damage sites

A verbal explanation should supplement the graphic representation, whereby all details which could not be reflected in the symbols should be provided. Insofar as conclusions can be drawn from the visual inspections, they should als be mentioned. Furthermore, comments as to whether and/ or which typ of additional inspections procedures would be useful in clarifying the cause of the damage should also be included.

When numerous damage sites have been detected, their consecutive numeration is expedient. A list can then be prepared according to those numbers. While the compilation of similar defects is necessary for the repair work tender, it is not required for documentation purposes. Should the list be prepared with a computer, an additional list according to the type of damage can be presented without significant additional expenses.

4.4 Photographs

Coloured photographs increase the informational value of any type of documentation. Black and white photos should no longer be used. One must remember that not every type of damage is equally suited for photographic presentation. For example, colour photographs of cracks serve no practical purpose, whereas those of rust and wet spots could prove extremely informative. Colour photography should therefore applied with care, though should it promise informative results, film should not be spared. It is also necessary that the photographs be clearly assigned to the damage sites; the simplest way of accomplishing this is by writing the number of the photograph on the drawings as well as by noting the photo number on the damage list.

In addition to photographs taken of surficial damages, those taken during endoscopic activities can prove extremely valuable for documentation purposes. The latter process is imperative during the inspection of the condition of grouting around tendons as well as of inaccessible voids (Fig. 9).

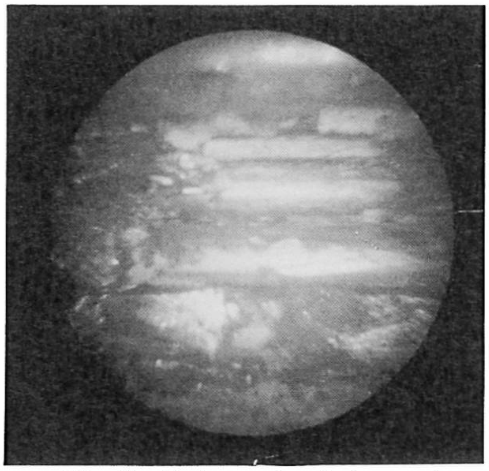

Fig. 9 Endoscopic photograph

4.5 Test Results

The results of the various tests which were carried out during the inspection procedures are to be included in the documentation. Such results would include, for example, measurements of the depth of carbonatisation on concrete structures or the thickness of protective coatings on steel structures. Insofar as samples were taken and the associated laboratory tests carried out, test results should also be included in the documentation. The applied sampling and/or testing methods must be mentioned in the test records or in the documents.

4.6 Computer-aided Documentation

Since the personal computer has become almost ubiquitous, it seems logical that this tool be applied for damage documentation as well. However, this does not mean merely using a word processing unit instead of a typewriter for writing the documentation. The full advantages of using data processing for documentation procedures are first realized when one includes additiona data processing functions. These auxiliary systems should at least provide the capability of using the damage documentation for the preparation of repair work tenders. Integrated evaluation procedures would be desirable to assist in preparing the assessment of maintenance conditions. A more detailed discussion will be presented below in Chapter 6.

5. **ASSESSMENT OF DAMAGES**

In assessing damages, it is important to include not only the type of structural damage, but the method of construction, the type of construction, and the aggressivity of the environment as well. An additional assessment tool can be gained from the degree of damage, whereby it must be realized that the damage situation can change as damage progresses. One must also make the distinction as to whether the damage negatively influences the structure's bearing capacity, its serviceability, its durability or a combination thereof. The evaluation of individual types of damage as well as their extent must be included in the assessment of the overall condition of the structure.

5.1 Types of damages

Depending on the type of structure involved, damages can be ordered according to the structural group, whereby subdivision according to appearance is more useful than one based on causes or causative agents. In such a manner, it is possible to maintain the same method of classification form the beginning of the inspection process through the documentation procedures; this also avoids having to determine causes for damage during the inspection phase. The following major damage categories can be established according to the main methods of construction:

Concrete construction:

Concrete damage, cracks and open construction joints, damages to normal reinforcement (Fig. 10), defects in prestressing tendons (Fig. 11) and wet spots (Fig. 12)

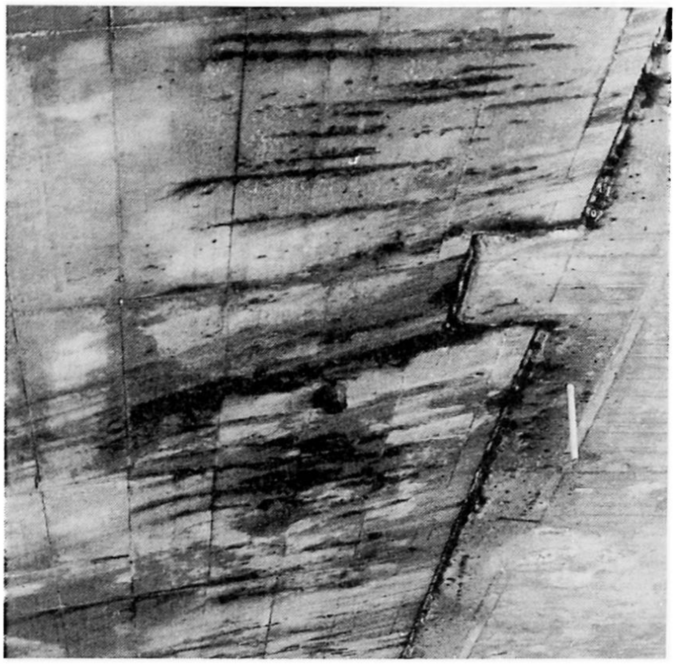

Fig. 10 Corrosion of reinforcing steel

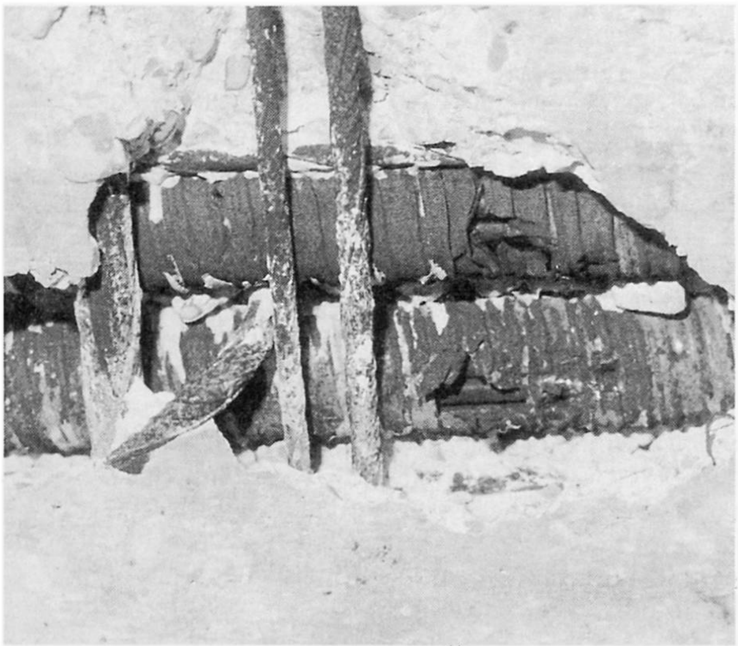

Fig. 11 Ungrouted ducts

Fig. 12 Wet spot

Particular effects with regard to the reliability of the structure can be assigned to each of the above-mentioned subdivisions. For example, a crack parallel to a bar is an indication that the bar is corroding and thus presents a problem with regard to durability. Should making the appropriate repairs be neglected, this could result in the splitting of the concrete cover, which in turn would mean more rapid rusting, again influencing the strength of the structure in a negative manner. On the other hand, a crack parallel to a tendon leads to the assumption of missing or insufficient grouting in the duct. Because of the voids within the duct, rusting does not cause the development of bursting pressure and spalling as a warning signal does not occur. It is thus not possible to exclude the reduction of the tendon cross-section and thus the reduction in load capacity and/or fatigue strength, which mean that rapid action must be taken.

The subdivision also provides a good foundation for drawing conclusions about the type of repair work which is needed. A constraint crack, which for instance might have been caused by the cooling of the concrete after placement (setting shrinkage subsequent to hydration heat), would not undergo significant width changes in the future and therefore could be injected with rigid synthetic resin. This repair method could, however, not be applied to a crack whose width continued to change because of loading, since new cracks would develop.

5.2 Environmental Aggression Classes

The same type of damage could be categorized differently according to diverse environmental aggression classes. For instance, wetness is generally harmless on reinforced steel structures. However, in combination with freezing and road salts (Aggression Class 3), it could mean a significant source of danger. Environmental aggression primarily affects the durability of a structure. If repairs are not carried out at an early enough time, the structure's serviceability as well as bearing capacity could also be affected.

5.3 Type of Structure

The type of structure plays a role with regard to the environmental influences which are associated with its function, and thus the statements made in the previous section can be applied here. A dry environment is expected inside residential and/or office buildings, whereas road bridges in climatic zones with winter weather are exposed to frost and road salt action. Naturally, a structure's function is not necessarily coupled with a particular class of environmental aggression, as indicated by the example of a jetty wall at a river or marine harbour.

Furthermore, the type of structure provides information as to whether dynamic action is to be expected or whether primarily static loads are shown. When dynamic loading is apparent, damages are also to be evaluated with regard to their influence on the fatigue safety of the structure.

5.4 Supplementary Inspections

In some cases, the results of the inspections and documentation are insufficient in evaluating damages. Here, additional inspections should be initiated to help clarify the cause of the damage and thus make assessment possible. Such additional tests would include static or dynamic calculations which utilize realistic values for structural materials and material laws, as well as measurements of shape changes under known circumstances. An example is provided in Fig. 14, which shows the width measurements of a crack on a prestressed concrete bridge, whereby it would have to be determined whether the crack was caused by constraint or by overloading. For this reason, measurements were taken during loading of the structure with a vehicle as well as under the conditions produced by daily temperature changes. The results clearly show that constraint forces dominate. Measurements taken after repair work had been completed showed that it had accomplished its purpose.

Structural steel:

Local deformation, insufficient joining material, damages to protective coating (Fig 13) and hairline cracks

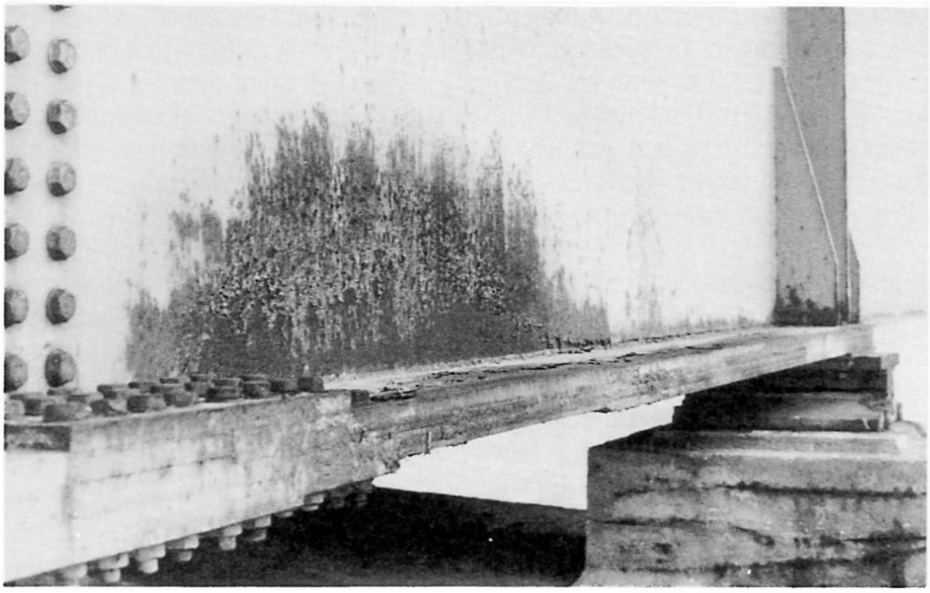

Fig. 13 Corrosion of steel structure

Composite construction:

All damages describes above under mass concrete construction and steel, as well as insufficient composite joining material

Bridge construction:

Damages to bearings, road expansion joints, insulation, drainage, etc.

Timber construction:

Changes in material, insufficient connections, cracks and wetness

Masonry/Brick work construction:

Cracks, wet spots, changes in material and plaster damages

Of course, other main groups can be selected for the categorization process. In any case, more detailed segregation into subdivisions is necessary. The manner in which damages belonging to several groups must also be established, for instance in the case water-carrying cracks. Subdivision does not necessarily have to be carried out on the basis of phenomenological criteria; here, classification according to cause could be useful in determining which measures should be applied for maintenance purposes. The subdivision of cracks in prestressed concrete have been provided below is representative for all types of damages.

Cracks caused by external loading

Cracks caused by the introduction of concentrated forces

Cracks caused by constraint and self-equilibrating stresses

Surface cracks

Cracks along reinforcement bars

Cracks along tendons

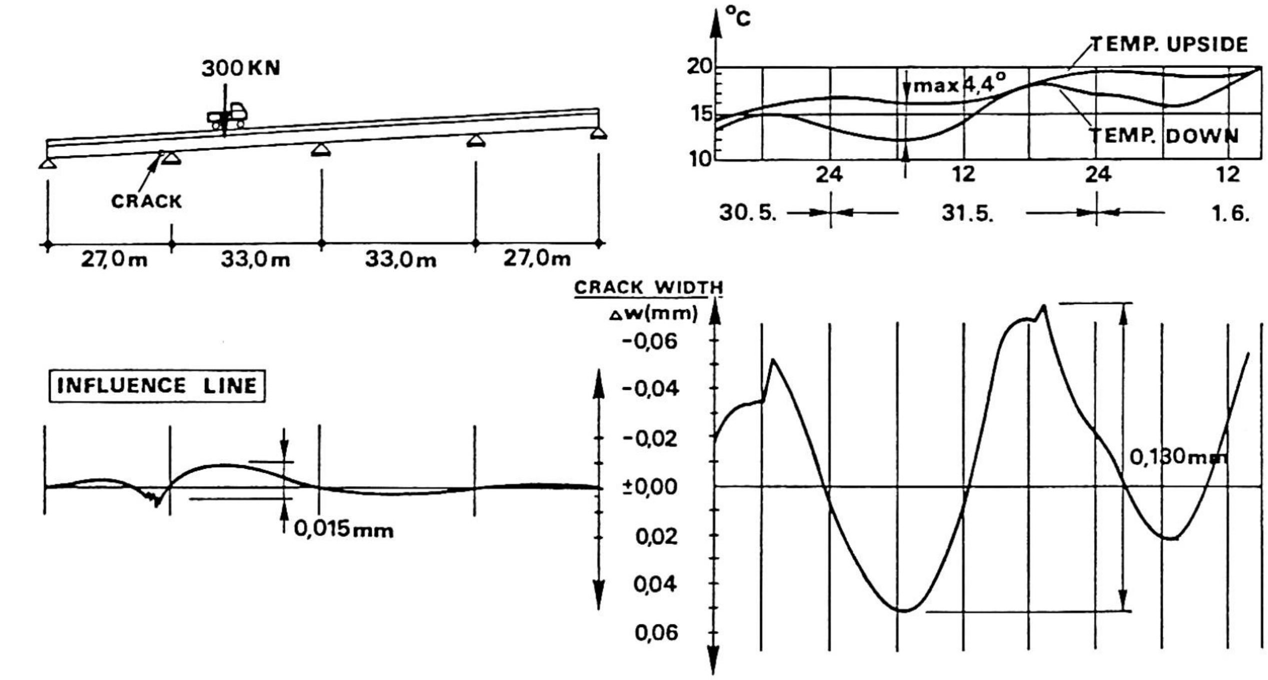

Fig. 14 Results of insite measurement of crack opening

6. ASSESSMENT OF MAINTENANCE CONDITION

A compreheensive assessment of all damages and defects established on a structure should reflect its maintenance condition and provide indications of the urgency of repair. The evaluations with regard to individual damages and their extent, as described above in Chapter 5, shall provide the foundation of this overall assessment.
Owners who have a large number of structures to maintain are also confronted with having to establish priorities for the maintenance work required, while taking available funding into consideration. To facilitate this activity, special assessment procedures have been developed, which can, however, due to the great quantity of data, only be carried out with electronic data processing.

An example of such an assessment procedure is one which was developed during a research project which dealt with road bridges constructed of concrete [2]. This procedure simplifies assessment process in a schematic manner to enable evaluation being carried out with the help of a blank form. A basic damage value "G" is assigned to each type of damage; "G" is then multiplied by four factors (k_1 to k_4), so that an evaluation of the damage can be calculated as follows:

$$G \cdot k_1 \cdot k_2 \cdot k_3 \cdot k_4$$

where

$0{,}5 < k_1 < 1$ is the factor of extent

$0{,}5 < k_2 < 1$ is the factor of intensity

$0{,}3 < k_3 < 1$ is the structural factor

and $1 < k_4 < 10$ is the factor of urgency

The factor of extent reflects the areal extent of a type of damage and/or the frequency with which it appears. The factor of intensity expresses the degree of damage. The structural factor takes the affect of a defect on the carrying capacity of a section or entire structure into consideration. The factor of urgency is applied to express the rapidity with which repairs of the particular type of damage should be carried out. To obtain a value expressing the overall condition, one adds the values attained for each type of damage.

Of course, though this schematic procedure cannot make decisions in place of the person responsible in individual situations, it does present a valuable tool for making such decisions. The procedure described has been tested on about 100 bridges and presented a clear picture of the bridges which should be given priority with regard to repair work (Fig. 15).

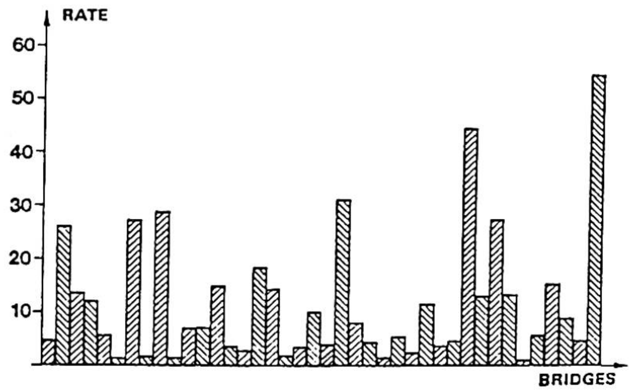

Fig. 15 Rating of maintenance situation

7. DEVELOPMENTAL TENDENCIES

Further developments would be desirable in two fields - in the field of inspection methods and in the field of assessment systems. Both fields promise good opportunities for further research as well as in the practical application of such results.

With respect to inspection methods, procedures which up until now have been primarly manual in nature, should be expanded and have engineering character. Indications of such a direction are provided by dynamic procedures as well as by the sound emission method. Here, the problem of adopting the equipment to the requirements of structural inspection as well as the improvement of result interpretation should be taken into consideration. A further point worth mentioning would be the application of video cameras for damage recording; these could also be used in conjunction with thermography.

Documentation procedures will continue to develop toward data processing and storage, whereby stored data can be used for assessment and tendering procedures. Another logical development would be replacing photographs with video clips. In conjunction with scanners (image recognition systems), electronic evaluation of the optical information would also be conceivable.

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Challenge of Highway Bridge Evaluation, Operation and Maintenance

Défis dans l'évaluation, l'exploitation et la maintenance des ponts routiers

Ueberwachung und Unterhaltung von Autobahnbrücken: eine Herausforderung

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SUMMARY
Comprehensive bridge management techniques must be used to meet the challenge of bridge needs in the 1990's and beyond. Bridge maintenance and rehabilitation are more cost effective than replacement. Much larger portions of available resources must be directed towards rehabilitation and maintenance.

RÉSUMÉ
Les techniques globales de gestion des ponts doivent être appliquées pour répondre aux défis des besoins en ponts dans les années 1990 et suivantes. La maintenance et la restauration des ponts sont plus économiques que leur remplacement. Une plus grande partie des ressources disponibles devrait être utilisée dans des programmes de restauration et de maintenance.

ZUSAMMENFASSUNG
Umfassende Organisationstechniken sind erforderlich, um den Herausforderungen der Brückenunterhaltung über die 90er Jahre hinaus gerecht zu werden. Brückenunterhaltung und -Verstärkungen sind kosteneffektiver als Ersatz durch Neubauten. Deshalb ist ein weit grösserer Teil der verfügbaren Mittel für Verstärkung und Unterhaltung zu verwenden.

I. BRIDGE STATUS

The 576,000 existing highway bridges in the United States pose a formidable challenge to those of us responsible for their continued safe and efficient operation.

Continuing traffic growth, a few spectacular collapses and the general public perception that bridges should last forever contribute to this challenge. The U.S. Secretary of Transportation reported last year that $50.4 billion would be required to bring all deficient highway bridges up to today's standards.[1] This estimate is based upon inspection data gathered by all States and submitted to the Federal Highway Administration for inclusion in the National Bridge Inventory.

2. ANNUAL PROGRAM

Each year the United States spends between $5 and $6 billion for new, replacement or rehabilitated bridges. Between 8,000 and 10,000 bridge improvement projects are begun each with these funds. About 6,500 to 7,000 of these bridge projects are funded through the Federal-aid highway program. The remainder are funded by individual States or local governments.

3. MAJOR QUESTIONS

While these are impressive figures for any country in the world, several key questions should be asked:

- Is the Federal program large enough?
- Are the right bridges being improved?
- Are the right replacement, rehabilitation and maintenance decisions being made?

4. FUTURE NEEDS

The National Bridge Inventory data, if properly structured, can provide a basis for answering these critical questions. The data included is described in the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, January 1979."[2] Figure 1 shows a histogram of the Nation's highway bridges. It can readily be seen that the majority of bridges built in the United States are between 15 and 35 years old. A more dramatic histogram is shown in Figure 2. This illustrates the deck or roadway surface area in square feet or square meters. No matter which unit of measurement is used, the important thing to note is that a full 40 percent of the deck area of existing highway bridges in the United States is represented by bridges between 15 and 35 years old. The tremendous bridge building boom of the 1950's and 1960's as the Nation carried out its Interstate highway construction program is the principle reason for this anomaly in the histogram.

It should be noted that the widespread use of bridge deck protective systems to prevent chloride induced corrosion of concrete reinforcing steel did not begin in earnest until the mid 1970's in the United States. Because of higher priority demands on available funds, the majority of these pre-1975 bridge decks have not been retrofitted with protection systems. As a result, bridge deck and superstructure rehabilitation needs are expected to continue to grow in the near future.

Existing Deck Area - All Bridges

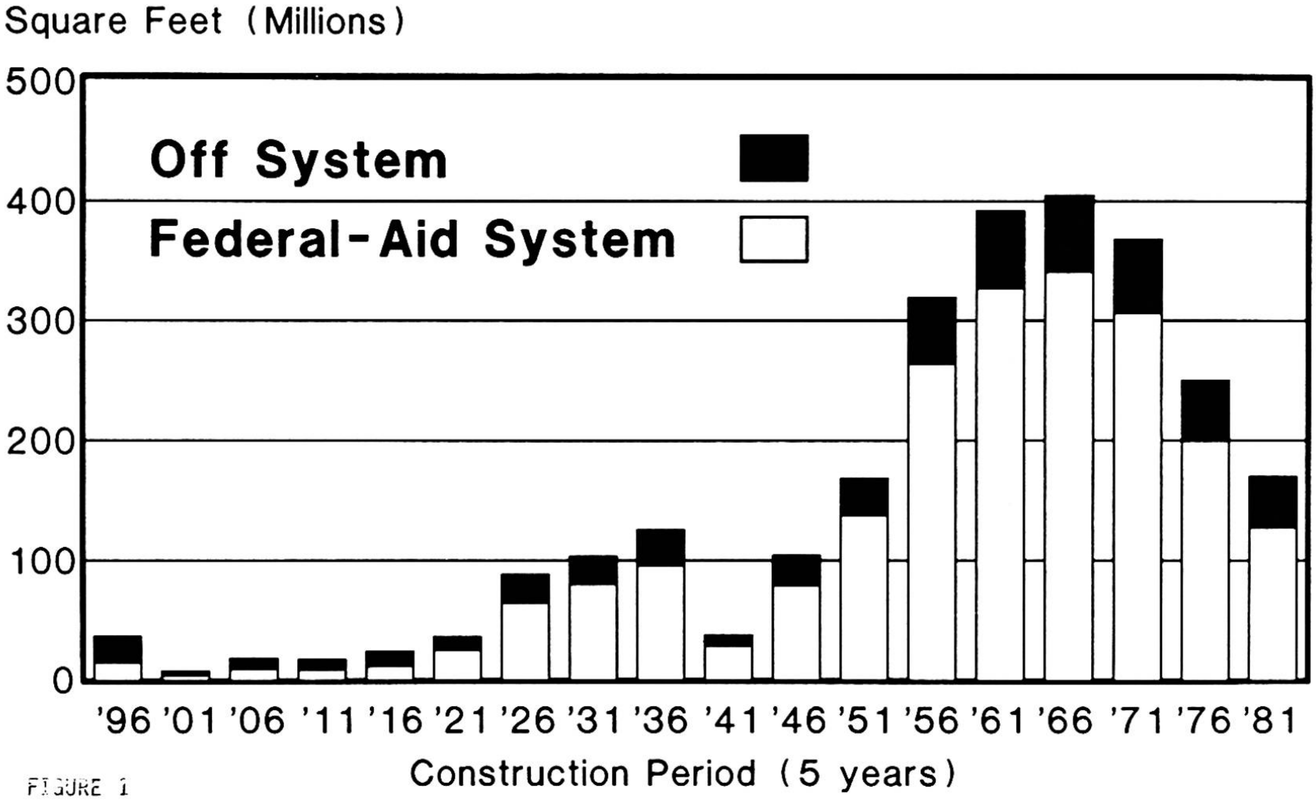

FIGURE 1

Number of Existing Bridges

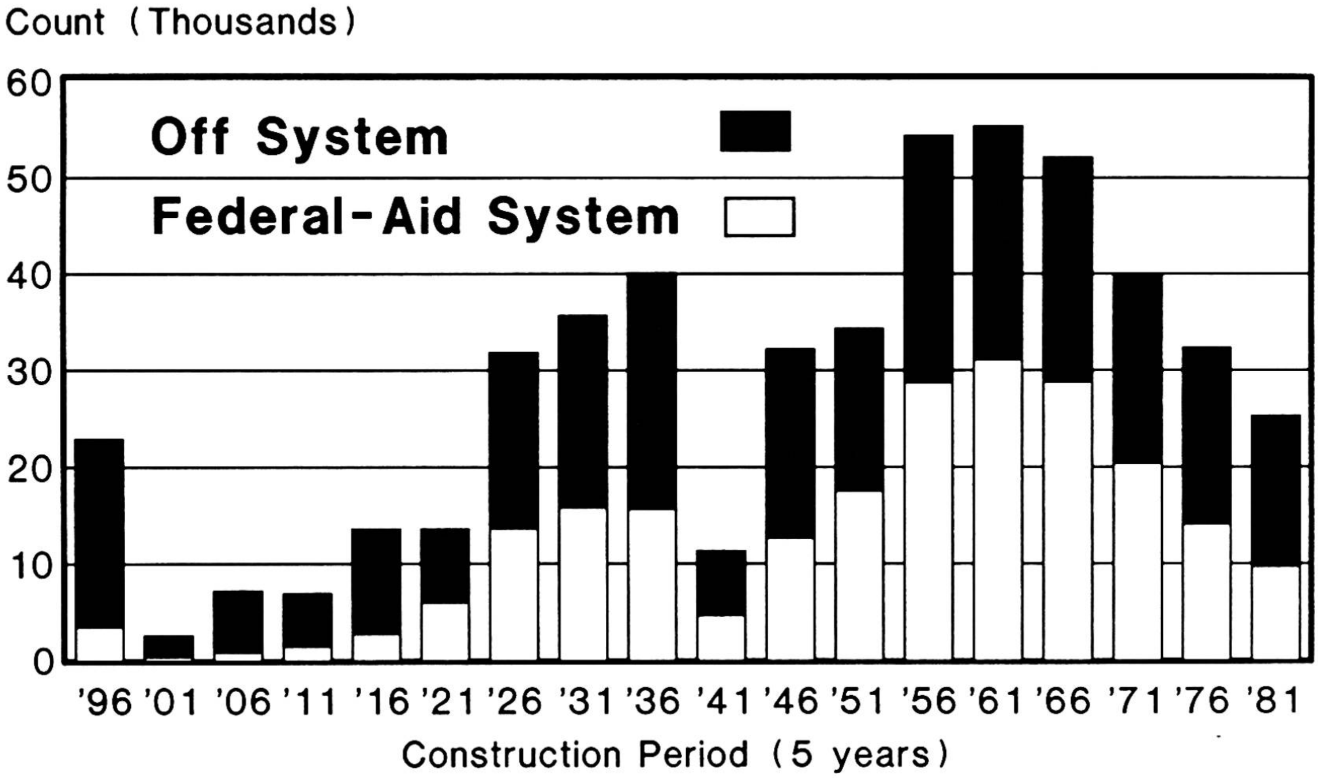

FIGURE 2

A recent study of the useful service life of existing bridges by the staff of the Federal Highway Administration indicates that the average bridge in the United States is replaced when it is about 70 years old and must be rehabilitated sometime during its midlife. Comparison of this life expectancy with the fact that 40 percent of the deck area of existing bridges is represented by bridges between 15 and 35 years old suggests strongly that bridge replacement, rehabilitation and maintenance needs will rise sharply during the next 10 years and beyond.

5. FUTURE PROGRAM SIZE

Current fiscal policies indicate that while Federal programs to improve bridges will not decrease in the immediate future, they are not expected to increase significantly either.

6. THE CHALLENGE

Therein is the challenge. Bridge needs are predicted to increase but resources are predicted to stay about constant.

7. COMPREHENSIVE BRIDGE MANAGEMENT PROGRAMS PROVIDE THE ANSWER

Comprehensive bridge management systems which incorporate the most current engineering, managerial and systems technology will provide the basis for meeting the challenge. A comprehensive bridge management system must include the following elements:

- Data collection and management
- Data Base
- Analysis
- Needs, predictions, options and costs
- Program formulation and Planning

Fortunately most States can use the National Bridge Inventory data as a beginning for their bridge management systems. Methods for manipulating and evaluating the data base are available. Several States have defined minimum tolerable levels of service of bridges and have developed formulas, based upon engineering judgement and empirical studies, to measure relative deficiencies of their highway bridges. Figure 3 illustrates the minimum and desirable levels of service used by the State of North Carolina.

It cannot be overstressed that the validity of any system depends largely upon the uniformity, accuracy and currency of the data base. If the inventory data is flawed, the entire system will suffer. The integrity of any management system for a large array of structures depends principally upon the validity of the inventory and appraisal data.

8. NEEDS DEFINITION

Needs cannot be determined without a universally accepted definition. One governmental unit's definition of needs will be another's luxury. One Country's substandard bridge may be entirely adequate in another Country. This is also true for States within the United States. The best definition of needs that we have been able to agree on is one which defines needs on the basis of benefits to the

North Carolina Level of Service Goals
Bridge Capacity Goals

Road Over Functional Classification	Single Vehicle Capacity (Metric Tons) Acceptable	Desirable
Interstate & Arterial	NP	NP
Major Collector	45.4 Tons	NP
Minor Collector	14.5 Tons	NP
Local	14.5 Tons	NP

NP = Not Posted (capacity = 30.5 Tons for single vehicles)

Clear Bridge Deck Width Goals
for Two Lane Routes

Road Over Functional Classification	Current ADT	Clear With (Meters) Acceptable	Desirable
Interstate & Arterial	ADT $\leq$ 800	6.7	9.8
	801 - 2000	7.3	11.0
	2001 - 4000	7.9	12.2
	Over 4000	8.5	12.2
Major & Minor Collectors	ADT $\leq$ 800	6.1	7.3
	801 - 2000	6.7	8.5
	2001 - 4000	7.3	9.1
	Over - 4000	7.9	9.1
Local	ADT $\leq$ 800	6.1	7.3
	801 - 2000	6.7	8.5
	2001 - 4000	7.3	9.1
	Over 4000	7.9	9.1

ADT = Average Daily Traffic

Bridge Vertical Underclearance Goals

Road Under Functional Classification	Underclearance (Meters) Acceptable	Desirable
Interstate & Arterial	4.27	5.03
Major & Minor Collectors	4.27	4.57
Local	4.27	4.57

FIGURE 3

user of the facilities. This definition of needs is universal to any array of structures. Two forms of this definition of needs are:[3]

- Needs are the least cost actions to make up the gap between existing conditions and standards which are socially optimal by virtue of maximizing net benefits to society.
- Needs are the actions that maximize the net benefits for each bridge, and thus represent the socially optimal choices from a broad range of alternatives.

Applying either of these definitions to a number of improvement options for an individual bridge should result in a unique choice for improvement which is optimal for the bridge users.

A comprehensive bridge management system will provide a systematic procedure for making bridge programming decisions which is markedly different from applying engineering expertise on a bridge-by-bridge basis.

9. INCREMENTAL BENEFIT/COST RATIO

One of the best methods of applying bridge management techniques to bridge improvement decisions is to determine the alternative improvement options for each substandard bridge, estimate the improvement cost for each alternative and estimate the user costs incurred by the public for each alternative. Often the higher cost improvements result in net benefits which are smaller than less costly improvement alternatives.

The incremental benefit/cost ratio is determined by taking each increment of benefit and dividing it by each increment of cost. At some point there will be an increment of benefit which equals the increment of cost. This the optimal point for improvement. Figure 4 illustrates this procedure.

If this process is repeated for all substandard bridges and the projects are listed in the order which lists the highest incremental benefit/cost ratio projects first, the resulting list will be the optimal list of projects in priority order.

The process can be readily computerized.[4]

Most recently, researchers from North Carolina State University applied the process to a group of 25 bridges and made some interesting observations.[5] Some of the conclusions are:

- The process is sound and superior to use of empirical priority ranking methods.
- The process provides near optimal sets of alternatives under budget constraints and optimal project sets under conditions of no budget restraints.

10. POLICY IMPLICATIONS

The use of the incremental benefit/cost techniques to select the best bridge improvement options on a system-wide basis has some interesting policy implications. Some of these are:

- For the normal range of discount rates, it is almost always better to rehabilitate or perform heavy maintenance on a bridge than replace it.
- It is in the public interest to spend about 6 percent of the replacement cost of a bridge each year to keep it in service. Put another way, if the discount rate is 6 percent, it is in the public interest to spend up to 6 percent of the replacement cost of bridges each year to keep them in service for an additional year.
- The long term trend in the United States should be to increase bridge maintenance budgets dramatically. Bridge rehabilitation budgets should be significantly increased and bridge replacement budget needs should drop correspondingly.
- Better bridge rehabilitation and maintenance techniques coupled with comprehensive bridge management systems are required to meet the challenge of bridge operations in the 1990's and beyond.
- Similar conclusions will probably apply to the evaluation, operation and maintenance of other large groups of structures such as buildings, dams, airfields and the like.

COMPARING ALTERNATIVES

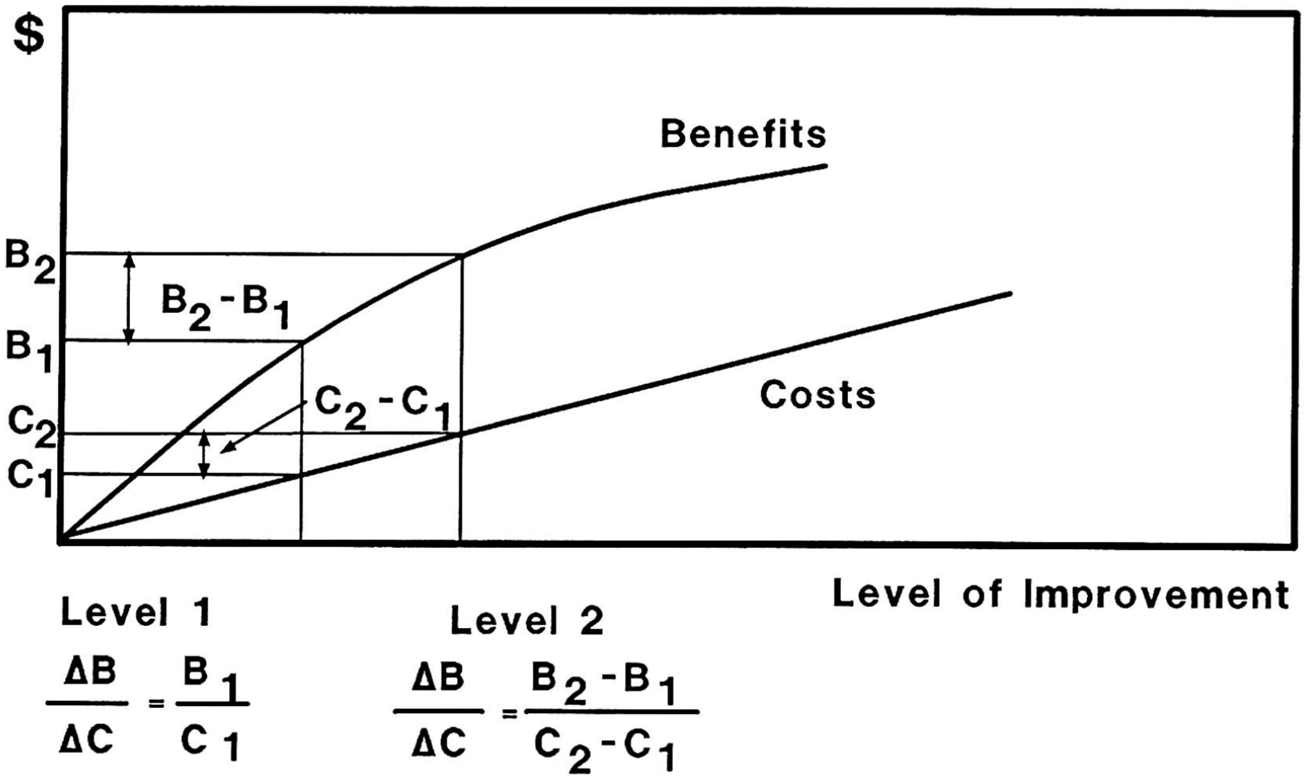

FIGURE 4

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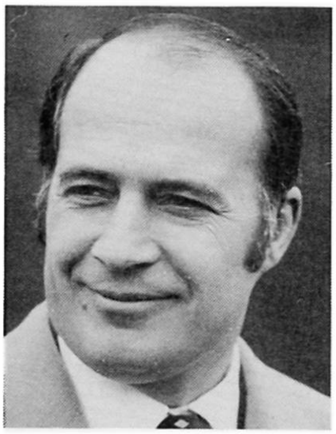
Betonsandwichwände
Wechselwirkungen zwischen Bauphysik und Konstruktion

Sandwich Walls Made of Concrete
Relations between building physics and building construction

Murs sandwich en béton
Relations entre la construction et la physique du bâtiment

Erich CZIESIELSKI
Prof. Dr.
Technische Universität Berlin



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ZUSAMMENFASSUNG
Es wird zu aktuellen Problemen Stellung genommen und es werden Weiterentwicklungsmöglichkeiten von Betonsandwichwänden aufgezeigt. Behandelt werden: Wärmebrücken durch Verbindungsmittel, Fugen, Ausbildung von Frostschürzen, transparente Wärmedämmungen, inkorporierte Heizungen, Sanierung von Korrosionsschäden durch Wärmedämmung.

SUMMARY
The paper deals with sandwich walls made of concrete, especially with actual problems and the possibility of the further development of such sandwich walls. The points treated are: thermal bridges, joints, thermal insulation in the earth (frost protection) transparent thermal insulation, incorporation of heating systems, repairing of corrosion damage by thermal insulation.

RÉSUMÉ
L'auteur prend position sur des problèmes actuels et montre des possibilités de perfectionnement dans le domaine des murs sandwich en béton. Les sujets abordés sont: les ponts thermiques dûs aux moyens de raccordement, les joints, l'isolation thermique dans la terre (gel), les isolations thermiques transparentes, les chauffages incorporés, la réparation des dégâts de corrosion par une isolation thermique.

1. ZWECK UND ZIEL DES BEITRAGES

Die Bauphysik ist die mathematisch naturwissenschaftlich Grundlage für das Konstruieren. Die Wechselwirkungen zwischen der Bauphysik und dem Konstruieren sollen am Beispiel von Betonsandwichwänden dargestellt werden. Hierbei soll zunächst auf folgende aktuelle Fragestellungen eingegangen werden:

a. Stellen die stählernen Verbindungsmittel zwischen Vorsatzschale und tragender Wand Wärmebrücken dar?
b. Fugenausbildung,
c. Ausbildung von Frostschürzen,
d. Sanierung von Korrosionsschäden.

Weiterhin sollen mögliche Weiterentwicklungen bei Außenwänden aufgezeigt werden:

1. Wände mit inkorporierten Heizungen/Kühlsystemen,
2. Transparente Wärmedämmungen.

2. AKTUELLE FRAGESTELLUNGEN BEI BETONSANDWICHWÄNDEN

2.1 Wärmebrücken

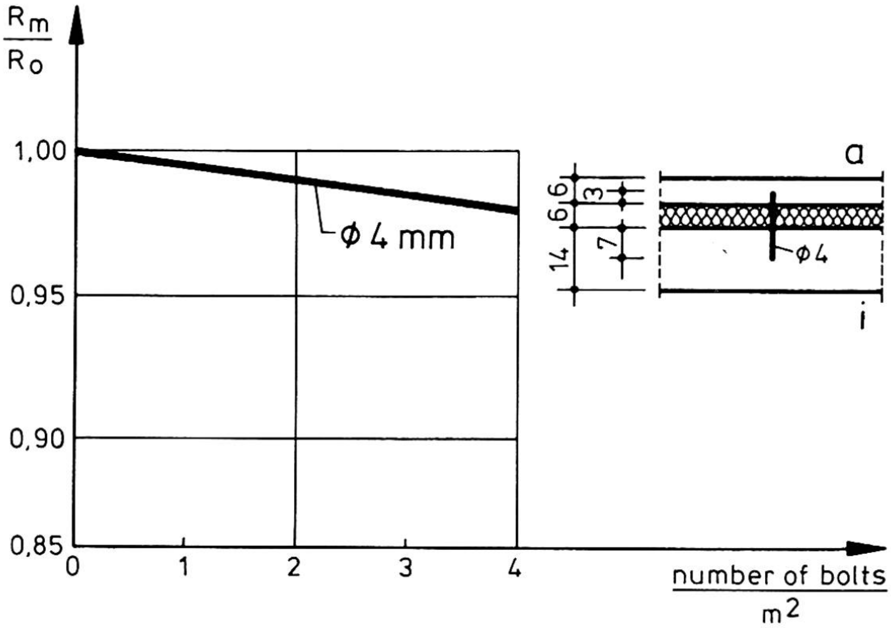
Bei der Konstruktion mit Betonfertigteilen ist in der Regel der erforderliche Wärmeschutz ohne besonderen Aufwand zu erreichen. Problematisch ist hingegen die quantitative Beurteilung von Wärmebrücken. - Entsprechend dem heutigen Wissensstand gibt es zwei Möglichkeiten, um den Einfluß von Wärmebrücken zu erfassen:

a. Versuchstechnische Verfahren nach der Heizkastenmethode,
b. analytische Verfahren.

Zur systematischen Erfassung von Wärmebrücken und zur Aufzeigung von Phänomenen wurden bisher beide Verfahren angewendet und die hinreichende Übereinstimmung beider Verfahren hinsichtlich der Ergebnisse nachgewiesen. Wegen des erheblichen Versuchsaufwandes und der bei den Versuchen bestehenden Fehlermöglichkeiten haben sich in letzter Zeit die analytischen Verfahren durchgesetzt, wenn auch die Erfassung räumlicher Wärmeströme an geometrisch komplizierten Konstruktionen Schwierigkeiten bereitet.

Um den Einfluß der Wärmebrückenwirkung von stählernen Verankerungen zwischen Vorsatzschale und tragender Wand zu erfassen, wurden sowohl Versuche als auch Berechnungen durchgeführt. Für stabförmige Verbindungsmittel sind in den Bildern 1 bis 3 die Ergebnisse dargestellt. Es ist ersichtlich, daß die Verankerungen auf den Wärmedurchlaßwiderstand und die innere sowie äußere Oberflächentemperaturen einen nur geringen Einfluß besitzen, wenn man von einer bauüblichen Anzahl von Verankerungsstählen ausgeht ($n \leq 4$ Stähle/m^2).

Zur Abtragung des Eigengewichtes der Betonvorsatzschalen von Sandwichwänden werden Ankerkonstruktionen unterschiedlichster Bauart verwendet (Bild 4). Meß- und Rechenergebnisse zeigen, daß die Abminderung des Wärmedurchlaßwiderstandes durch die Anker relativ gering ist. Der prozentuale Einfluß eines solchen Ankers auf den Wärmedurchlaßwiderstand einer Außenwand wird auch von der Fläche der Außenwand bestimmt: Je nach Art des Ankers und der Größe der



R_m $[\frac{m^2 \cdot K}{W}]$ thermal resistance with bolts

R_o $[\frac{m^2 \cdot K}{W}]$ thermal resistance without bolts

Bild 1 Wärmedurchlaßwiderstand einer Wand mit Wärmebrücken (R_m) bezogen auf den Wärmedurchlaßwiderstand einer Wand ohne Wärmebrücken (R_o)

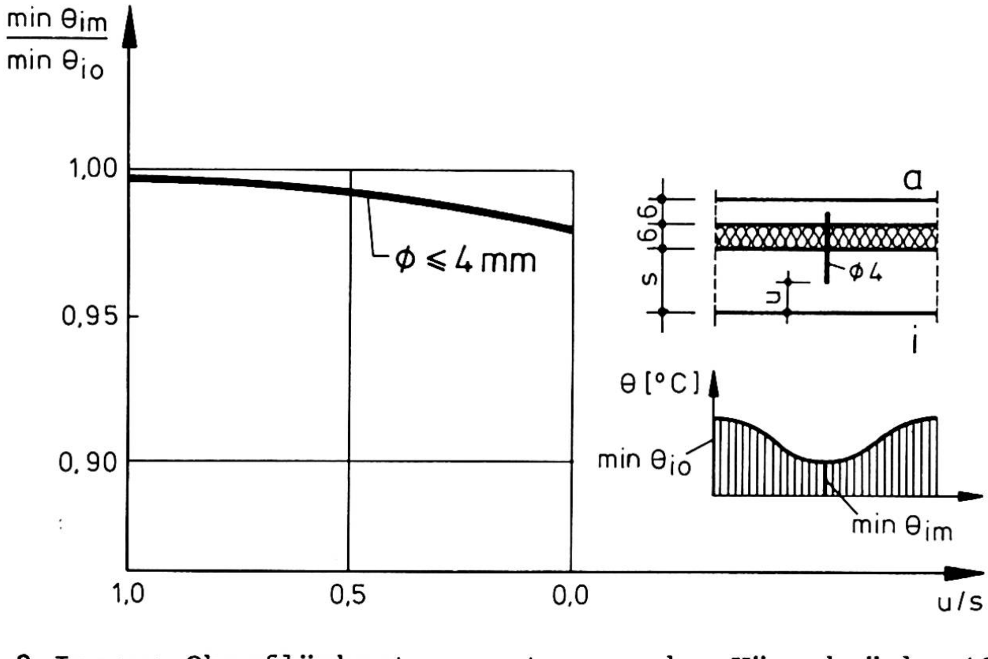


Bild 2 Innere Oberflächentemperatur an der Wärmebrücke (θ_{im}) bezogen auf die Temperatur im "ungestörten" Wandbereich (θ_{io})

Wände wird durch den Anker der Wärmedurchlaßwiderstand um maximal 3 % gemindert.

Näherungsweise wird empfohlen - insbesondere bei den heute vorherrschenden größeren Dicken der Wärmedämmstoffe -, den errechneten Wärmedurchlaßwiderstand einer Betonsandwichwand insgesamt um ca. 5 % zu mindern (2 % für die vier Stähle Ø 4/m² und ca. 3 % für den tragenden Anker).

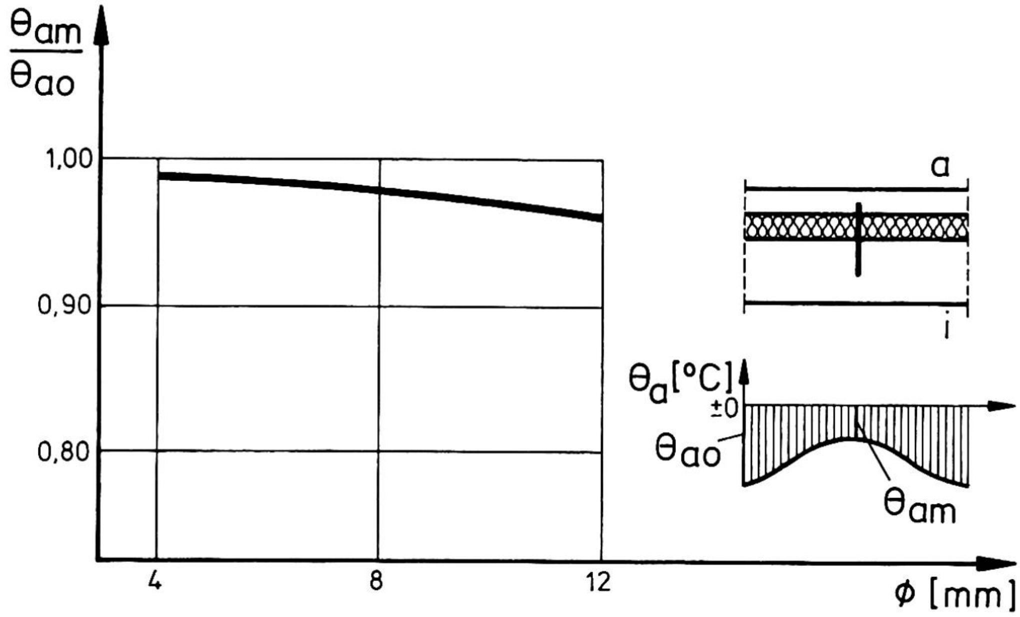


Bild 3 Äußere Oberflächentemperatur an der Wärmebrücke (θ_{am}) bezogen auf die Außenlufttemperatur (θ_{ao})

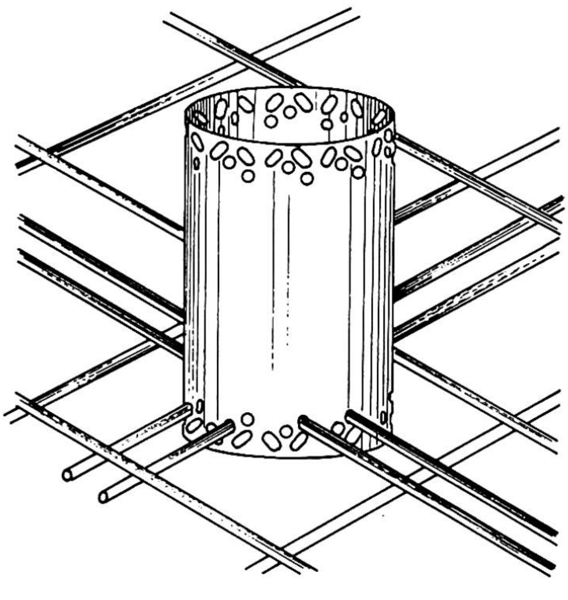


Bild 4 Anker zum Abtrag des Eigengewichts der Vorsatzschale ("Manschettenanker" - System deha)

Diese Empfehlung steht nur scheinbar im Widerspruch zu in der Literatur [1] angegebenen Meßergebnissen; dort wurden Abminderungen des Wärmedurchlaßwiderstandes zwischen 13 % und 41 % für eine 1 m^2 große Wand angegeben. Geht man von bauüblichen Größen der Außenwände von $A \geq 5$ m^2 aus und werden die durch falsch verlegte Wärmedämmplatten verursachten Wärmeverluste ausgeschlossen, so kann auch mit diesen Ergebnissen die pauschale Empfehlung bestätigt werden, den Wärmedurchlaßwiderstand einer Betonsandwichkonstruktion um 5 % abzumindern, um den Einfluß sämtlicher stählerner Verankerungsstähle zu erfassen. Bei dieser Empfehlung wird vorausgesetzt, daß die beim Einlegen der Wärmedämmplatten im Bereich der Anker beschädigte Wärmedämmung nachgebessert wird und daß die Wärmedämmplatten zweilagig verlegt werden.

2.2 Fugen

Es gehört zum Stand der Technik bei Betonsandwichwänden, die Fugen entweder mit Dichtungsmasse zu schließen, was als problematisch bekannt ist, oder belüftete Fugen auszuführen. Bei den belüfteten Fugen treten in letzter Zeit dann Probleme auf, wenn im Bereich der Horizontalfuge die Betonüberdeckung zu gering ausgeführt wurde (Korrosion der Bewehrung, s.Abschnitt 2.3, Bild 7). Es besteht die Möglichkeit, die belüfteten Fugen z.B. mit einbetonierten PVC-Profilen auszuführen (Bild 5), wobei die Probleme der Betonüberdeckung weitgehend vermieden werden. - Eine Alternative besteht darin, die Fugen mit vorgefertigten Bändern z.B. aus Polysulfid zu überkleben (Bild 6). Die bisherigen Erfahrungen seit ca. 15 Jahren in Deutschland bestätigen diese Art der Fugenabdichtung.

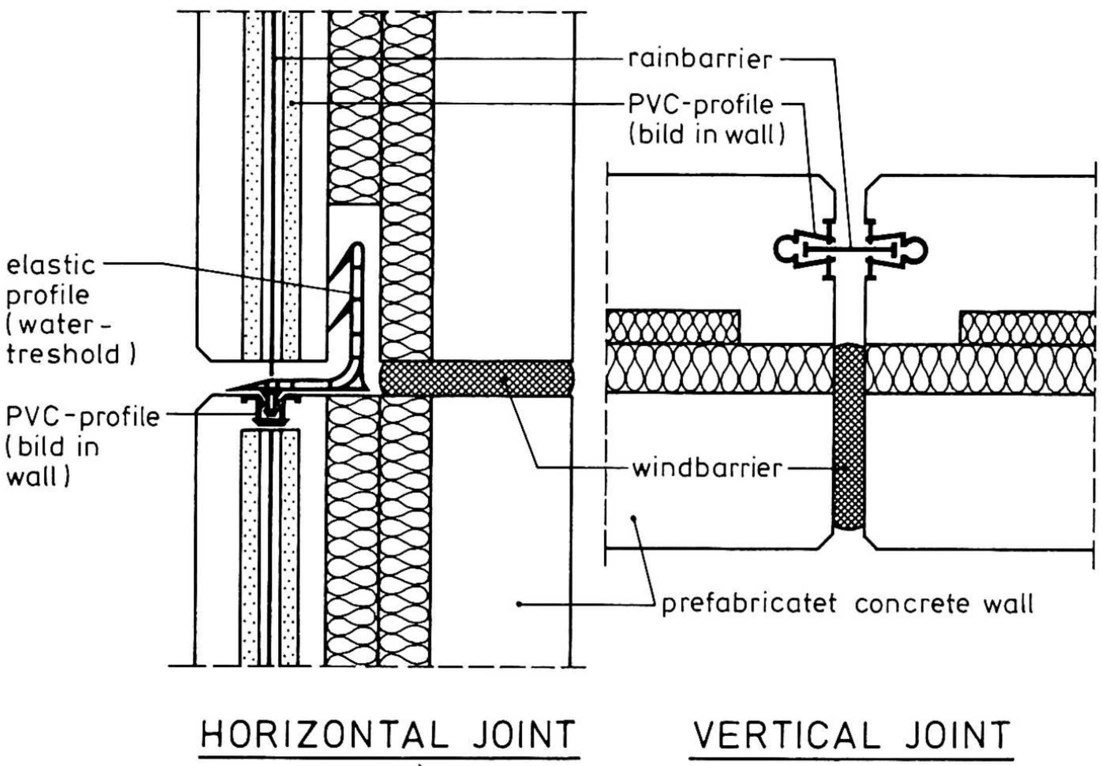


Bild 5 Belüftete Fugenkonstruktion (System Eurofit)

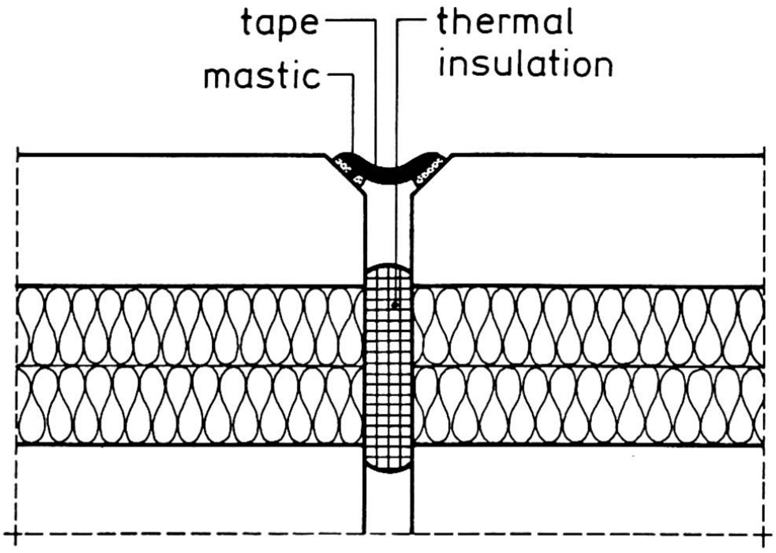


Bild 6 Abdichtung mit einem Fugenband aus Polysulfid, Silicon o.ä.

2.3 Sanierung von Korrosionsschäden

Die Bewehrung in den Vorsatzschalen - insbesondere im Bereich der Wandränder - ist in der Vergangenheit oft so verlegt worden, daß die erforderliche Betondeckung nicht eingehalten wurde. Betonabplatzungen infolge der korrodierenden Bewehrung waren die Folge (Bild 7).

Bild 7 Betonabplatzung im Bereich offener Horizontalfugen infolge Korrosion der Bewehrung

Eine Instandsetzung der Wände nach der heute üblichen Technik des Sanierens ist die Regel, wobei die Vielzahl der Arbeitsgänge eine äußerst sorgfältige handwerkliche Arbeit verlangt, damit der Schaden dauerhaft beseitigt wird (Bild 8).

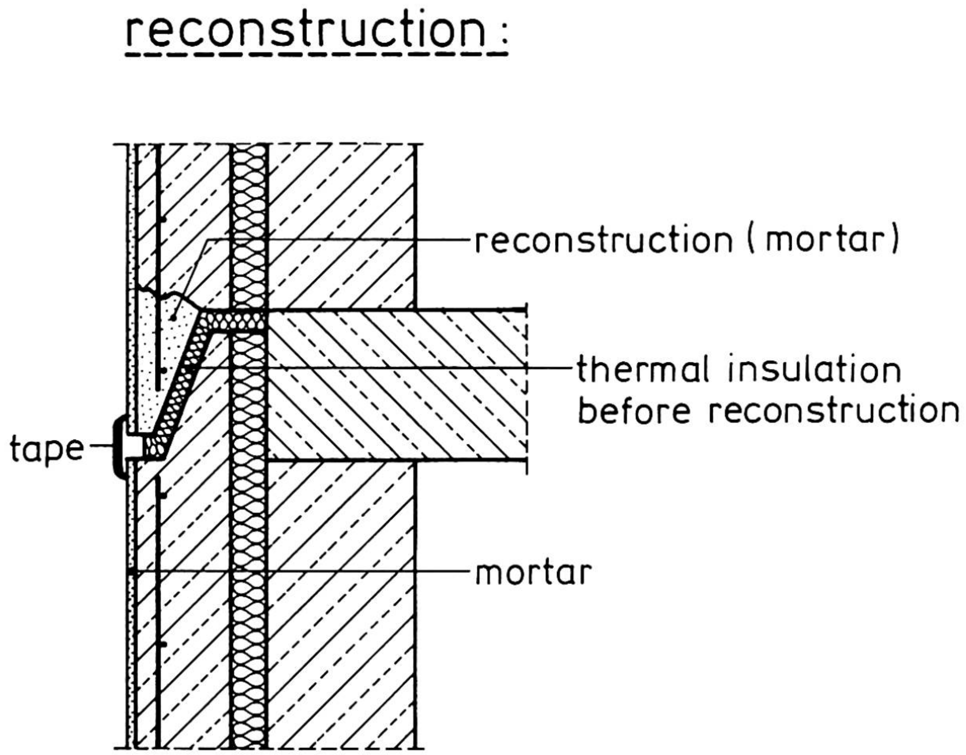


Bild 8 Reparatur der geschädigten Horizontalfuge durch ein Betonsanierungssystem

Ein anderer Weg, den Korrosionsprozeß zu stoppen und den Schaden zu beheben besteht darin, die gesamte geschädigte Fassade mit einem Wärmedämmverbundsystem zu versehen.

Dem Gedanken, durch ein Wärmedämmverbundsystem den Korrosionsprozeß zu stoppen, liegt die Überlegung zugrunde, daß zur Korrosion eines Bewehrungsstahles drei Voraussetzungen gleichzeitig erfüllt sein müssen:

- Die Passivierung der Stahloberfläche im Beton muß aufgehoben sein - durch Karbonatisierung oder schädliche Salze -,
- Sauerstoff muß zutreten können und
- ein Elektrolyt muß vorhanden sein, d.h., der Beton muß ausreichend feucht sein.

Die sinnvollste Maßnahme im nicht chloridbelasteten Hochbau ist es, die Karbonatisierung des Betons so rechtzeitig einzudämmen, daß während der geplanten Lebensdauer des Bauwerks der Karbonatisierungshorizont die Stahlbewehrung nicht erreicht. Dies ist bei Gebäuden mit sichtbaren Korrosionsschäden nicht mehr möglich.

Die zweite Möglichkeit besteht darin, den Zutritt von Sauerstoff zu verhindern. In diffusionsoffenen Baustoffen wie Beton ist dies nur direkt am Stahl möglich, z.B. durch dichte Kunststoffbeschichtungen; dies ist einer der Wege der "klassischen" Betoninstandsetzung.

Als dritte Möglichkeit der Korrosionshemmung bleibt noch, den Beton dauerhaft so trocken zu halten, daß mangels ausreichendem Elektrolyten keine Betonstahlkorrosion stattfinden kann.

Bisherige Untersuchugnen haben gezeigt, daß durch ein nachträglich aufgebrachtes Wärmedämmverbundsystem die Feuchte in der tragenden Betonwand und in der Vorsatzschale sinkt:

- Im ursprünglichen Zustand ist die tragende Schale der Wand deutlich trockener als die längere Zeit durchfeuchtete Vorsatzschale,

- nach Aufbringen der zusätzlichen Wärmedämmung wird die Tragschale geringfügig trockener als vorher,

- während die Vorsatzschale nicht nur deutlich trockener als vorher, sondern auch noch trockener als die tragende Wandschale im ursprünglichen Zustand wird.

Da die Vorsatzschale nach Aufbringen eines Wärmedämmverbundsystems (Bild 9) keine Schlagregenbeanspruchung mehr erhält, dürfte die Bewehrung somit bei normal genutzten Innenräumen nicht weiter korrodieren. Diese auf theoretischem Wege gefundene Erkenntnis wird

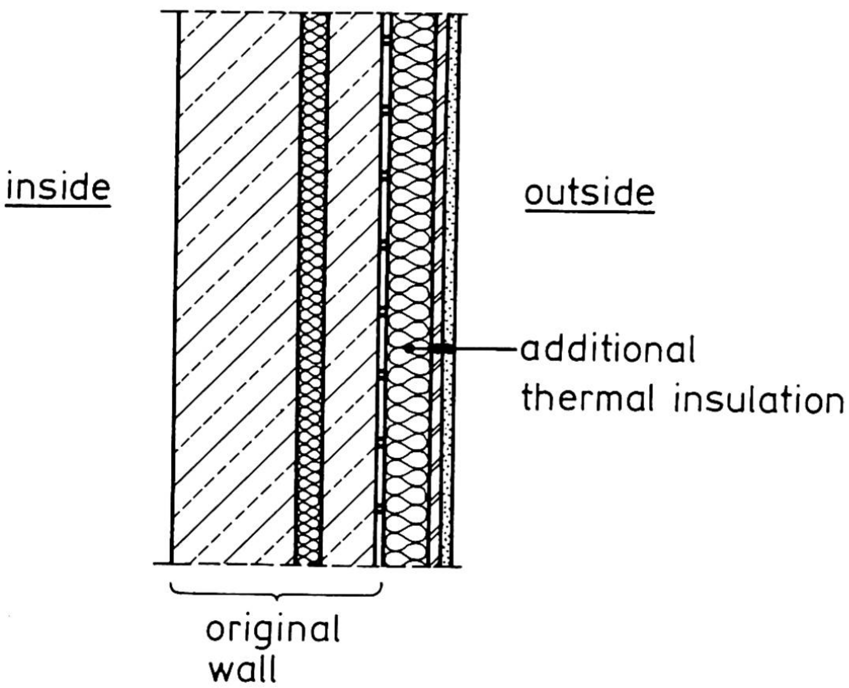


Bild 9 Sanierung der korrodierten Bewehrung durch nachträgliches Aufbringen einer Wärmedämmung

durch Beobachtungen an ausgeführten Bauten vollinhaltlich bestätigt: Betonwände, die im Wohnungsbau und ähnlich genutzten Gebäuden ausgeführt wurden, sind an den zum Raum hin orientierten Seiten in der Regel erheblich karbonatisiert, so daß für die innenliegende Bewehrung die passiverende Schutzschicht verlorengegangen ist; dennoch sind noch nie Korrosionsschäden im Beton beobachtet worden, bei denen die Betonoberflächen dem Rauminnern hin zugewandt waren.

Zusammenfassung: Durch Aufbringen einer zusätzlichen äußeren Wärmedämmung (Bild 9) im Bereich der Wohnungsaußenwände kann die Bewehrungskorrosion in den Vorsatzschalen nachhaltig gehemmt werden. Darüber hinaus wird ein zusätzlicher Wärmeschutz erzielt. - Bestätigungsversuche werden zur Zeit durchgeführt.

2.4 Frostschürzen

Im Industriebau werden in der Regel Frostschürzen entsprechend Bild 10 ausgeführt. Anstelle der nur bedingt wirksamen massiven Frostschürze können auch wärmedämmende Platten aus extrudiertem Polystyrol verwendet werden, die bereits bei einer Einbindetiefe von 45 cm in das Erdreich ein Auffrieren des Bodens wirksam verhindern. - Für Hallenkonstruktionen, bei denen für die Außenwände aus statischen Gründen kein tragendes Streifenfundament vorhanden sein muß, stellt die in Bild 11 dargestellte Konstruktion - auch unter Berücksichtigung des Bauablaufs - eine mögliche Konstruktionsvariante dar.

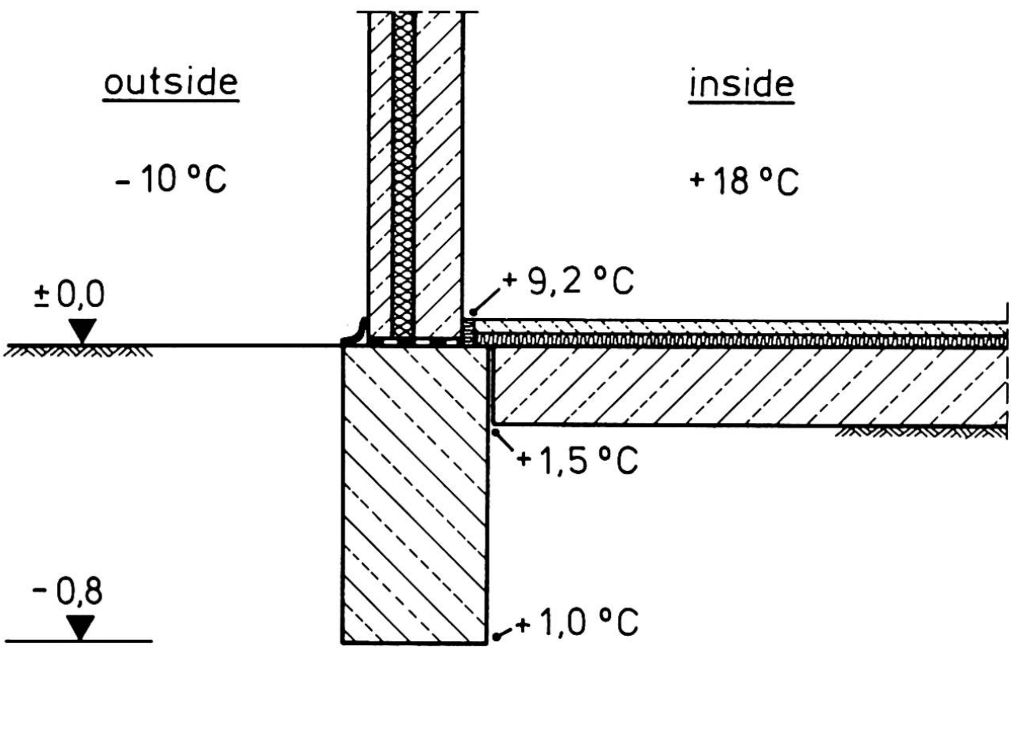


Bild 10 Übliche Frostschürze, die durch ein Streifenfundament gebildet wird

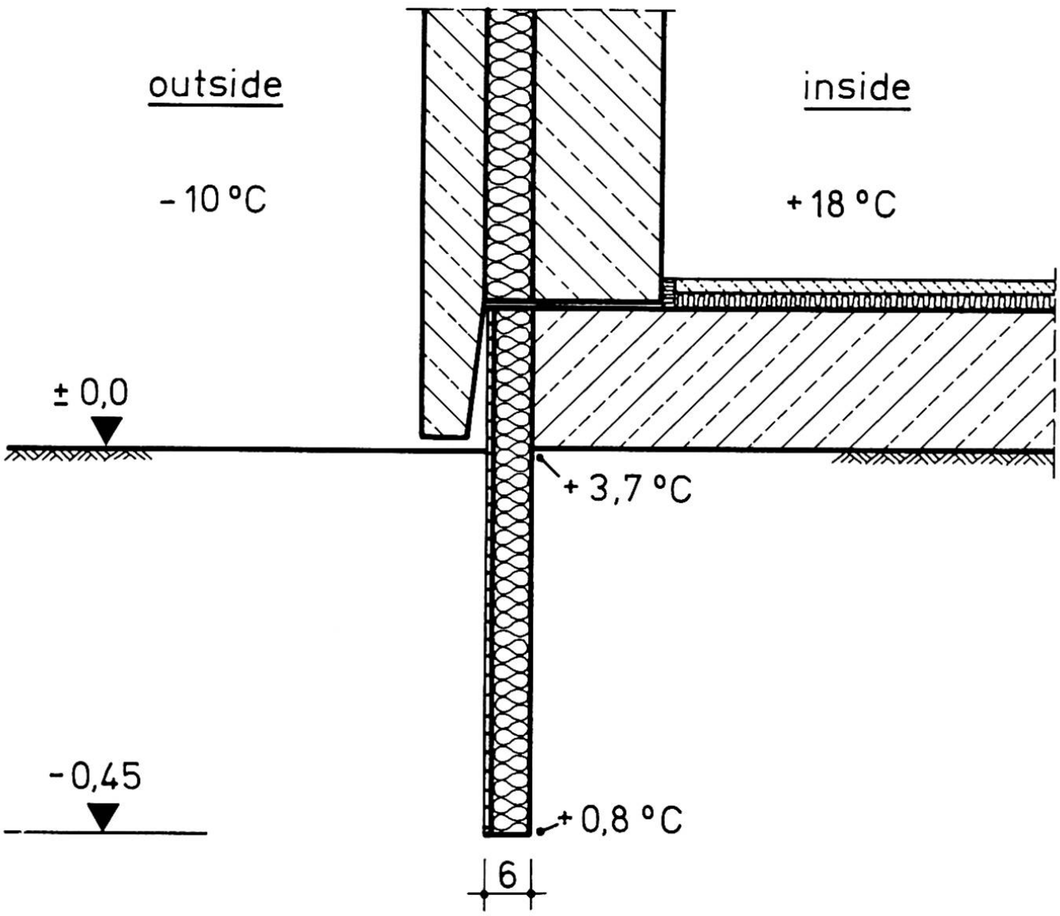


Bild 11 Frostschutz unterhalb der Bodenplatte durch eine Wärmedämmung (extrudiertes Polyst yrol), die durch eine Kaschierung stabilisiert wird

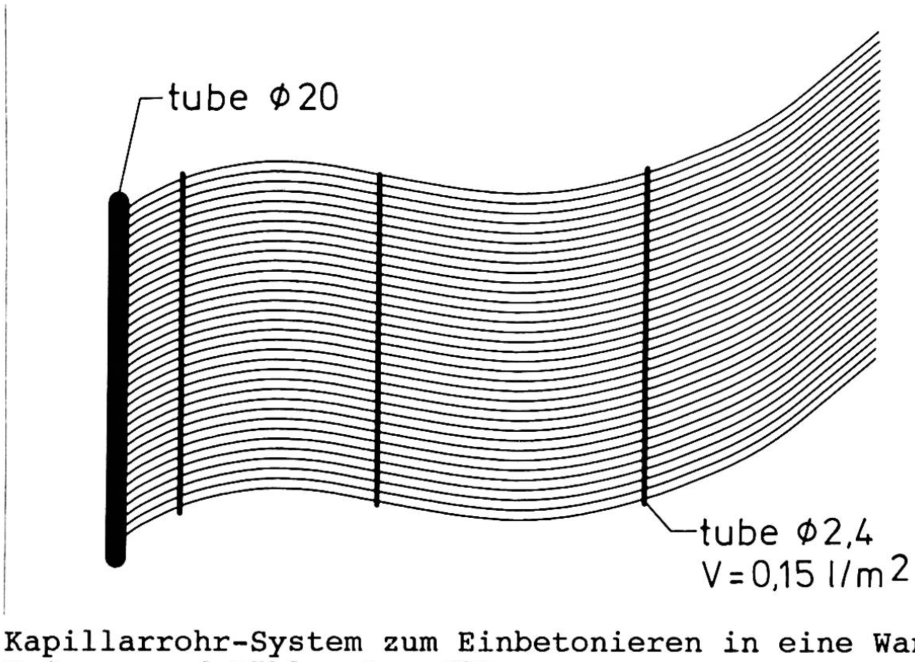


Bild 12 Kapillarrohr-System zum Einbetonieren in eine Wand als Heizung und Kühlsystem [2]

3. MÖGLICHKEITEN DER WEITERENTWICKLUNG VON BETONSANDWICHWÄNDEN

3.1 In die Wände eingebaute Heiz- bzw. Kühlsysteme

Die Wirtschaftlichkeit von vorgefertigten Bauteilen ist in immer höherem Maß dann gegeben, wenn in die Bauteile Teile des Ausbaues einbezogen werden. Es bietet sich an, in die Wände eine Warmwasserheizung, die gleichzeitig auch zur Kühlung herangezogen werden kann, zu inkorporieren.

In Deutschland ist ein Rohrsystem - ein sogenanntes Kapillarrohrsystem entwickelt worden (vgl. Bild 12) [2]. Das Rohrsystem besteht aus einer Vielzahl von flexiblen Röhren mit einem Durchmesser von nur 2,4 mm, die in zwei Sammelrohre Ø 20 mm münden. Aufgrund ihres geringen Durchmessers und des engen Abstandes der Rohre wird eine gleichmäßige Erwärmung der Wand erreicht, während die Tragfähigkeit der Wand gemindert wird. Tragfähigkeitsversuche haben ergeben, daß unbewehrte Wände aus Normalbeton, in die das Rohrsystem einbetoniert war, eine um ca. 20 % geringere Druckbeanspruchung aufwiesen im Vergleich zu Wänden, in die das Rohrsystem nicht einbetoniert war. - Die Bilder 13 und 14 zeigen den Einbau der Rohre in die Wände. - Die Bilder 15 und 16 zeigen die Wände unter Normalkraftbeanspruchung im Bruchzustand. Der Bruch wurde durch das am Wandkopf liegende Sammelrohr eingeleitet (erhöhte Querzugspannungen). Die verminderte Querzugfestigkeit am oberen bzw. unteren Rand der Wand kann durch eine Bügelbewehrung ausgeglichen werden. - Weiterhin wurde festgestellt, daß durch das Rohrsystem ein verringerter Verbund zwischen den an das Rohrsystem grenzenden Betonschichten vorhanden war.

Für die Heizung wird Wasser mit einer Vorlauftemperatur von 30 °C verwendet. Soweit die Außenwandflächen nicht ausreichen, um die erforderliche Wärmemenge in den Raum abzugeben, können Innenwände oder Decken ebenfalls mit dem Rohrsystem versehen werden.

Eine Untersuchung einer Wand, bei der in die Rohre abwechselnd + 60 °C warmes Wasser und Wasser von + 12 °C geleitet wurde (Temperaturschock), hat die Wand ohne Risse überstanden.

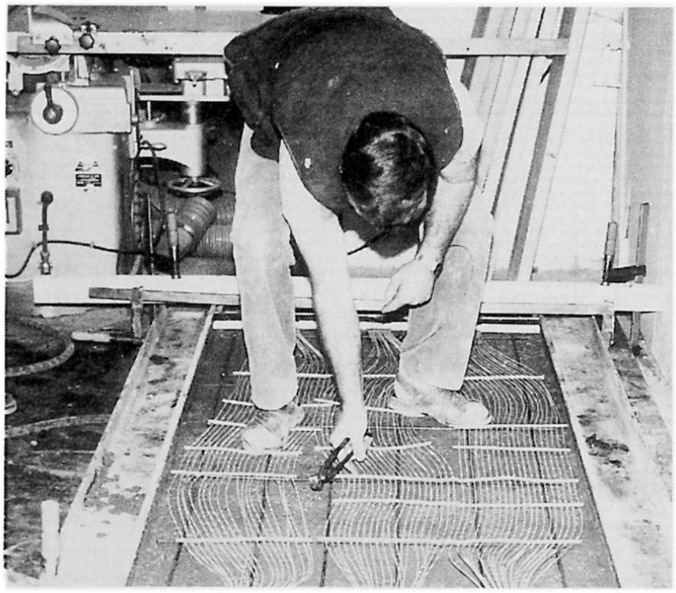


Bild 13 Verlegen der Rohrmatte

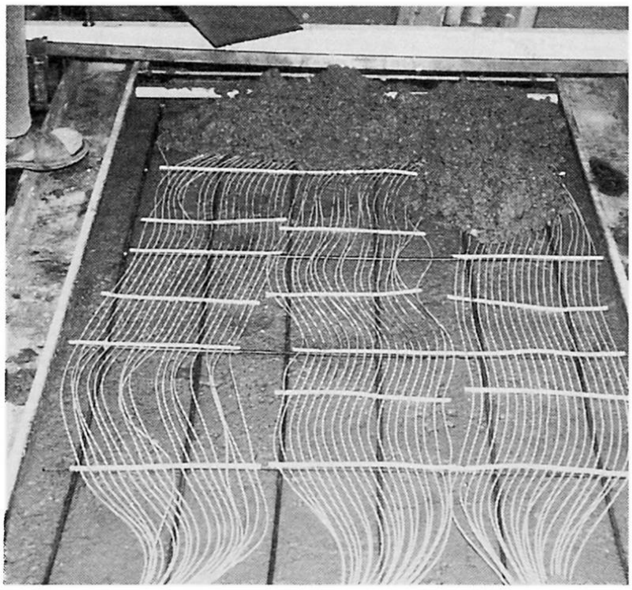


Bild 14 Einbau der Rohrmatte

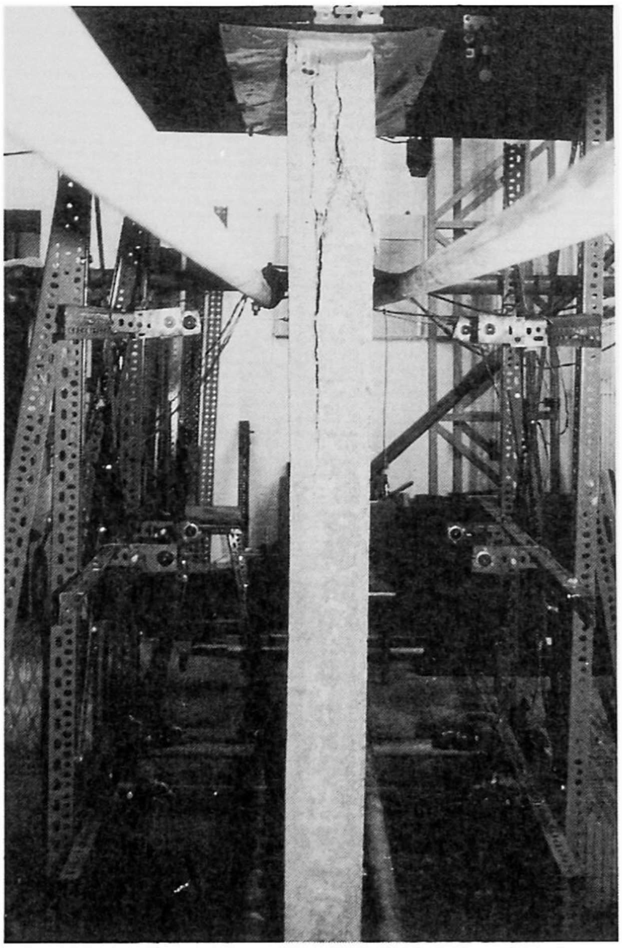


Bild 15 Wand mit Rohrmatte bei der Bruchbelastung

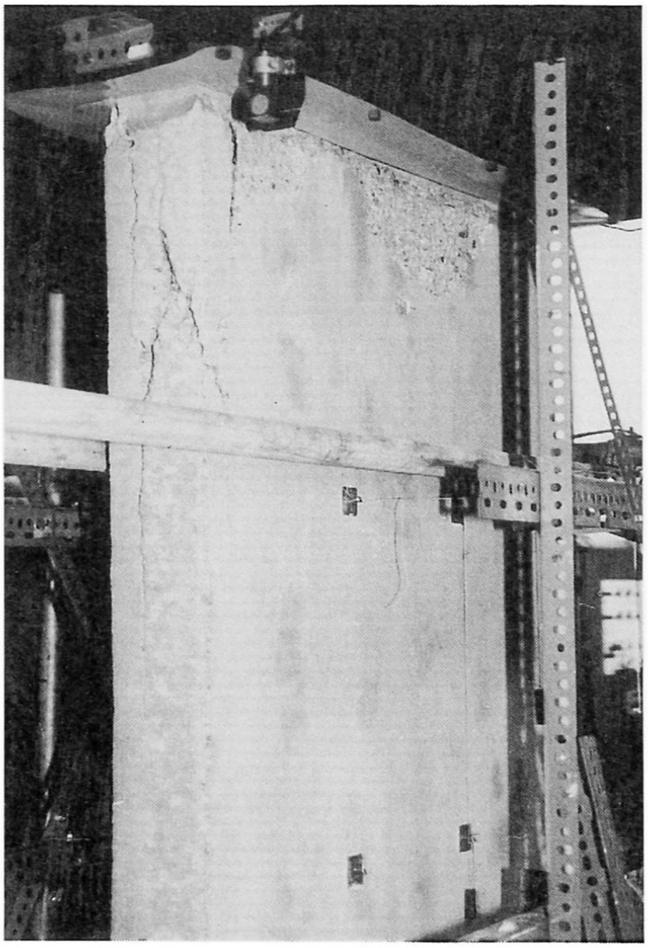


Bild 16 Bruchbild

Wird in das Rohrsystem kühles Wasser (Leitungswasser) eingespeist, so kann die Wandfläche auch als Kühlsystem wirken. Durch die Kühlung der Räume über die Wandflächen entfällt ein Nachteil der konventionellen Klimaanlagen, bei denen kühle Luft in die Räume geblasen wird, wodurch Zugerscheinungen auftreten. - Mit dem Rohrsystem können Kühllasten bis zu 80 W/m^2 aus den Räumen ohne Luftbewegung abgeführt werden. Wasser eignet sich im Vergleich zur Luft besser für den Wärmeabtransport, da ihre spezifische Wärmekapazität wesentlich größer ist. Hinzu kommt, daß für die Kühlung keine Kältemaschinen benötigt werden; es reicht in der Regel ein Verdunstungskühler, um die den Räumen entzogene Wärme an die Außenluft abzugeben bzw. es kann die Wärme auch an das Erdreich im Keller des Gebäudes abgegeben werden. - Bisherige Vergleichsberechnungen haben ergeben, daß mit der "Wandkühlung" ein besseres Raumklima mit wesentlich geringeren Energiekosten erreicht wird.

Um beim Einbringen von Befestigungsmitteln in die Wände die Rohre nicht zu beschädigen, empfiehlt es sich, durch Temperaturmessungen (Indikatorpapier) die Lage der Rohre vor dem Bohren zu orten. Sollte dennoch ein Rohr beschädigt werden, so kann durch ein Einstecken einer heißen Nadel in das Bohrloch das Rohr wirksam wasserdicht verschweißt werden.

3.2 Transparente Wärmedämmung

Die Entwicklung transparenter Wärmedämmungen kann dazu beitragen, daß der Energiebedarf eines Gebäudes in hohem Maße eingeschränkt wird [3]. Die Wirkung der transparenten Wärmedämmung nutzt die Durchlässigkeit des Glases und anderer transparenter Materialien beim Auftreffen kurzwelliger Wärmestrahlen aus. Die von der Sonne abgestrahlte kurzwellige Wärmestrahlung geht durch das Glas und die transparente Wärmedämmung hindurch und trifft auf die dahinter liegende tragende Wand (Bild 17). Die massive Wand ist nicht strahlungsdurchlässig; die Wärme wird absorbiert und die Wand wird dabei erwärmt. Die absorbierte Wärme kann nicht wieder nach außen abgegeben werden, da einerseits die transparente Wärmedämmung eine Wärmetransmission nach außen weitgehend verhindert und andererseits die von der Wand ausgehende langwellige Wärmestrahlung von den transparenten Materialien ebenfalls am Durchgang behindert werden.

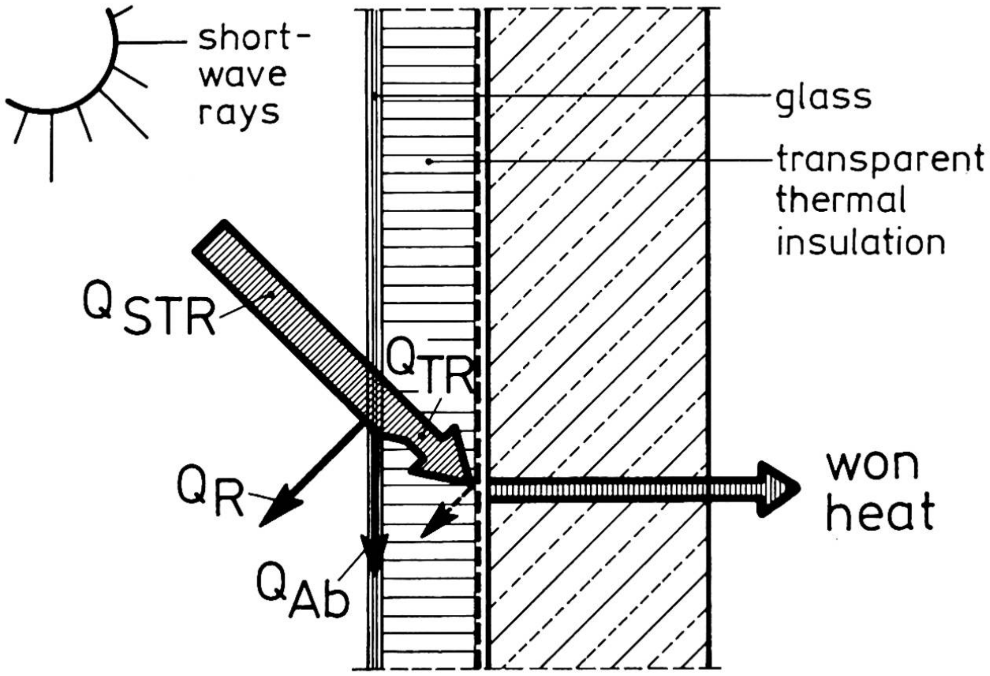


Bild 17 Wärmegewinn durch eine transparente Wärmedämmung

Bei den heute ausschließlich verwendeten opaken (nichttransparenten) Wärmedämmstoffen wird die von der Sonne ausgehende Wärmestrahlung nicht durchgelassen; sie wird an der Außenoberfläche der Wand absorbiert und zum überwiegenen Teil nach außen abgegeben, so daß nur ein geringer Wärmegewinn entsteht.

In Bild 18 ist ein möglicher Wandaufbau dargestellt. An transparenten Wärmedämmstoffen sind bisher untersucht worden [3]: Acrylschaum, Glasfaservlies, Glaskugeln, Wabenkonstruktionen. Es ist darauf zu achten, daß die Dämmstoffe nicht durch die Erwärmung geschädigt werden; dies gilt insbesondere für Dämmstoffe aus organischen Materialien.

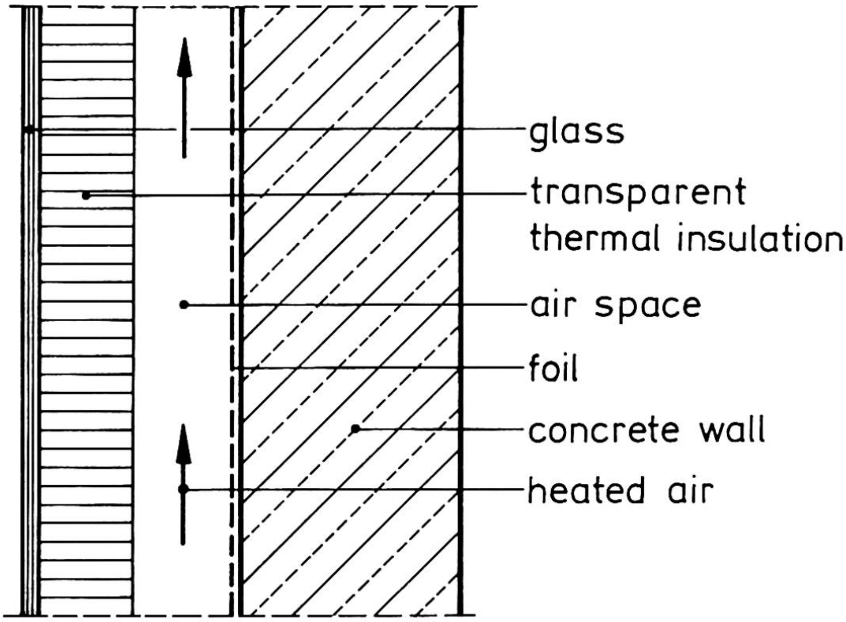


Bild 18 Wandaufbau mit einer transparenten Wärmedämmung

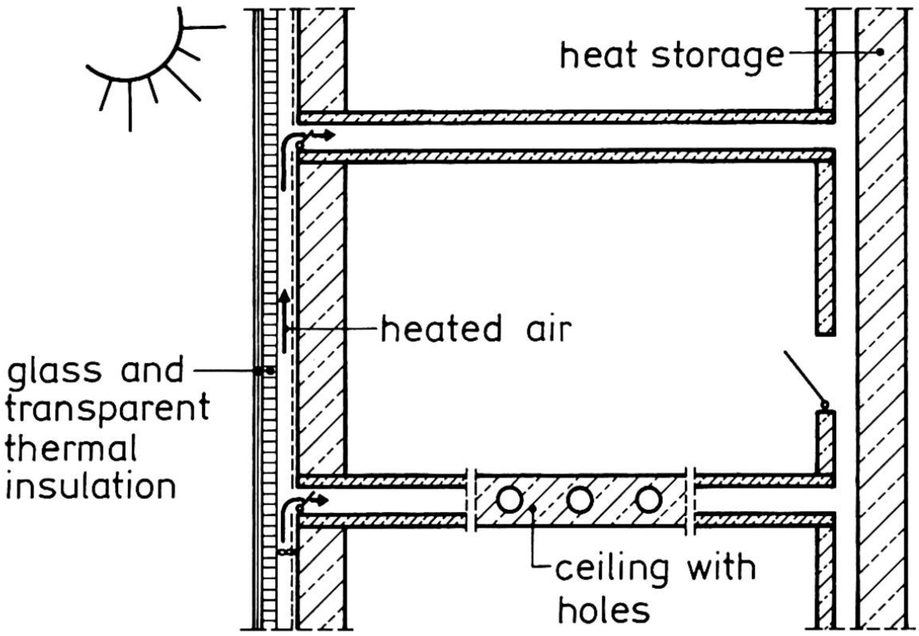


Bild 19 Prinzip der Konstruktion eines Gebäudes zur Nutzung der Wärmestrahlung

Da die Wärmestrahlung nicht konstant anfällt (z.B. nicht während der Nacht) und auch nicht mit gleicher Intensität während eines Tages, muß für die Nutzung der intermittierend anfallenden Wärmestrahlung ein Speicher (z.B. massive Wände oder Decken) vorgesehen werden. Der prinzipielle Aufbau eines Gebäudes, bei dem die Wärmestrahlung genutzt wird, ist in einer Prinzipskizze dargestellt (Bild 19). In der Bundesrepublik Deutschland stehen bereits einige Testhäuser, in denen die Wirksamkeit der transparenten Wärmedämmung untersucht wird.

Ein vorläufiges Zwischenergebnis dieser Untersuchungen ist, daß selbst dann, wenn nicht sämtliche Außenwandflächen eines Gebäudes mit transparenten Wärmedämmstoffen versehen wurden, der Engergiebedarf zur Heizung des Gebäudes erheblich reduziert wird: der spezifische Jahreswärmeverbrauch für Häuser mit einer opaken Wärmedämmung beträgt im untersuchten Fall 47,0 kWh/m^2, während das gleiche Testhaus mit einer transparenten Wärmedämmung einen Energiebedarf von nur 9,9 kWh/m^2 aufwies (Engergieeinsparung ca. 79 %).

Theoretisch denkbar wäre es, auch in der Bundesrepublik Deutschland, wo nur mäßige Wärmestrahlungsintensitäten auftreten, völlig auf eine zusätzliche Heizung zu verzichten. Das Problem, das sich dann stellt, ist, wie der Überschuß an Wärme vom Gebäude - insbesondere im Sommer - ferngehalten werden kann. Denkbar sind außen vor der Verglasung angebrachte Sonnenschutzvorrichtungen oder Glaseinfärbungen, die bei einer bestimmten Temperatur den Durchgang der Wärmestrahlen verhindern. Entsprechende Untersuchungen und weiterführende Forschungen werden zur Zeit in Deutschland durchgeführt, damit dieses Konzept praxisreif gemacht wird und auch von den Architekten akzeptiert wird.

4. ZUSAMMENFASSUNG

Es wurde gezeigt, daß die Bauphysik die mathematisch naturwissenschaftliche Grundlage für das Konstruieren ist. - Für Betonsandwichwände sind einige in der Feuchtwelt diskutierte Fragen auf der Grundlage bauphysikalischer Erkenntnisse beantwortet worden. Weiterhin wurden Weiterentwicklungsmöglichkeiten im Bereich von Wänden aufgezeigt.

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Building Physics and Design

Physique des constructions et projet

Bauphysik und Entwurf

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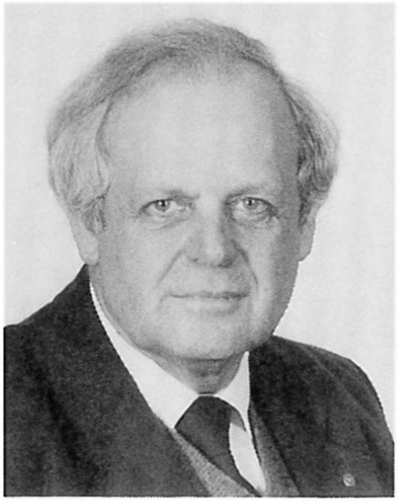

Bo Adamson, born in 1925, received his doctor's degree at the Royal Institute of Technology, Stockholm, Sweden 1955. He is professor in Building Science since 1964 at Lund University, Sweden. He is involved in research concerning energy conservation of new and existing buildings, heat and mass transfer in building materials and components as well as heat and mass balance in buildings.

SUMMARY
Building physics is involved in all stages of building design and concerns mostly heat and mass transfer in materials and components as well as radiation. The paper discusses problems as heat flow in ground, temperatures in non-transparent building components, energy transfer through windows, moisture migration and storage in materials and structures, fire exposed structures and air movement in rooms and spaces. The paper shows that simulation of the physical processes is the basic for an increased knowledge but it is often stated that the agreement between simulations and measurements in occupied buildings could be better. This is discussed in a section "Theory and practice" in the end of the paper.

RÉSUMÉ
On retrouve la physique des construction à tous les stades des projets de construction, ceci concerne surtout le transfert calorifique et de masse dans les matériaux et les composants, ainsi que la radiation. L'article traite de problèmes tels que la circulation de chaleur au niveau du sol, les températures dans les composants non-transparents des constructions, le transfert d'énergie à travers les fenêtres, la migration et le dépôt d'humidité dans les matériaux et les structures, l'exposition au feu des structures et le mouvement de l'air dans les chambres et les espaces. Ce travail montre que la simulation des processus physiques est une bonne base pour une meilleure connaissance mais on constate souvent que la corrélation avec les mesures effectuées dans des constructions habitées pourraient être meilleures. Ceci est discuté dans le chapître "Theory and practice" en fin d'article.

ZUSAMMENFASSUNG
Die Bauphysik spielt in allen Stadien des Gebäudeentwurfs eine wichtige Rolle, vor allem was den Wärme- und Feuchtetransport in Materialen und Bauteilen betrifft. Dieser Beitrag behandelt den Wärmefluss in den Untergrund, Temperaturen in nicht-transparenten Bauteilen, Energie-übertragung durch Fenster, Feuchtetransport und -aufnahme in Baumaterialen, feuerexponierte Bauten und die Luftbewegung in Räumen und Zwischenräumen. Es wird gezeigt, dass die Simulation der physikalischen Vorgänge das Verständnis verbessert. Im Abschnitt "Theorie und Praxis" wird die Tatsache diskutiert, dass die Uebereinstimmung zwischen Simulation und Messungen am Gebäude oft besser sein könnte.

1 INTRODUCTION

Natural science is the basis of building design. Other necessary desciplines are aesthetics, function, psychology and physiology. Within natural science mechanics, physics and chemistry are used, and we have subjects such as:

$\left.\begin{array}{l}\text{- building statics}\\ \text{- building dynamics}\end{array}\right\}$ structural mechanics
- building physics
- building chemistry
- building biology

The most interesting topics in building design are those in which more than one subject is involved, as for example structural mechanics and building physics. Other interesting topics of current interest are rot, mould and smell in buildings, in which structural mechanics, physics, chemistry and biology are involved.

Building physics covers subjects such as:

1) heat transfer by conduction, convection and radiation as well as heat storage by sensible and latent heat
2) mass transfer and storage (moisture, air and gases)
3) heat and mass balances in rooms and buildings
4) fire and fire resistance
5) lighting
6) acoustics

All these subjects, except item 5, are more or less related to structural mechanics. Subjects such as energy conservation and air pollution are not specifically mentioned despite the fact that they are very important. They need different areas of physics for their solution.

The aim of this paper is to give an introduction to some problems chosen from research fields of which the author has some knowledge.

2 HEAT FLOW IN THE GROUND

Heat flow in the ground influences the foundation structure in two ways. Firstly, if the ground consists of frost heaving soil the foundation can be affected by frost heaving forces. Secondly, the heat flow determines floor temperatures and heat consumption. Since the end of the 1960s computer simulations have made possible the solution of two- and three-dimensional transient heat transfer problems which also consider latent heat and temperature-dependent thermal conductivity and heat capacity. Figure 1 shows isotherms in a ξ-z section (a vertical section through the diagonal of a 10x10 m building in north Sweden), Adamson, et al 1972.

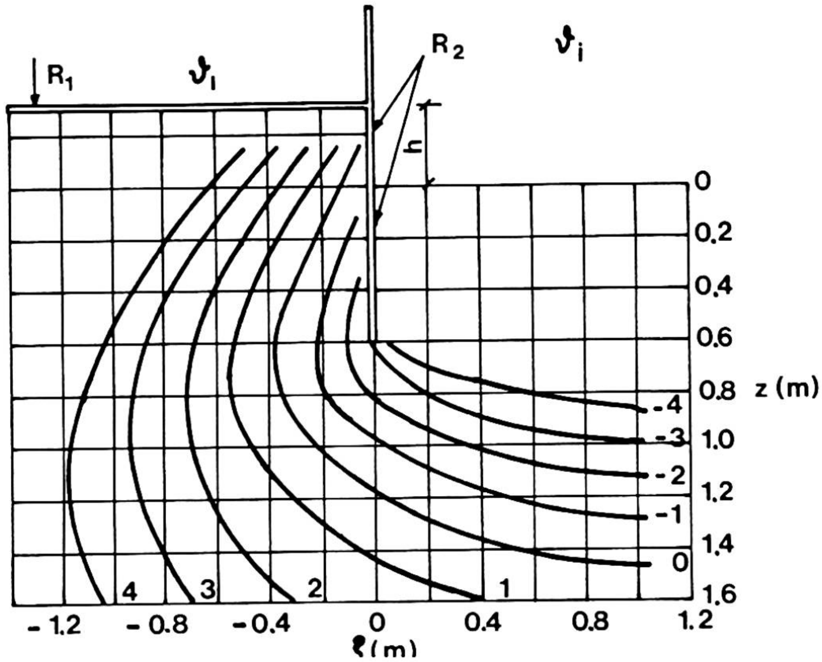

Fig.1 Isotherms, 4 weeks after the lowest outdoor temperature, in a vertical ξ-z section of a house of 10x10 m foundation area. Indoor temperature = +20 ^{o}C and outdoor air temperature = +4.4+17.4 cosωt. Snow-free ground. h=0.3. Floor insulation R_1=1.08 m^2K/W and foundation wall insulation R_2=1.08 m^2K/W for $-0.3 \leq z \leq 0.6$ m.

3 TEMPERATURES IN NON-TRANSPARENT BUILDING COMPONENTS

Building components such as walls and roofs are influenced by air temperatures and radiation as well as evaporation and condensation of moisture. Very often surface temperatures are calculated with standardized surface heat transfer coefficients. Air movement, shortwave (solar) and longwave radiation must be taken into account if a reasonably accurate temperature is to be calculated. A number of computer programs which can calculate surface temperatures and temperatures within the component by considering convection, radiation and conduction, are available. Normally such programs do not take evaporation and condensation into account. Unfortunately, outdoor climatic conditions such as solar and longwave radiation to the sky are not well known. Moreover, future shading by other buildings and vegetation is not known at the design state.

It has been shown that the colour of the outer surfaces of a building is of critical significance for both surface temperatures and the heat flow through the component. Adamson, 1987 shows that the heat flow through walls and roof can be increased by 5-15% in Chinese climate by changing the colour from dark to white.

If the temperature stresses or deformations are to be calculated it is necessary to carry out an accurate calculation in which all the above parameters are taken into consideration. We need better simulation methods and experimental validation of the programs. Otherwise a calculation will be merely a conjuring trick which deludes both the performer and the audience.

4 ENERGY TRANSFER THROUGH WINDOWS

The glazed part of a window has earlier been treated in a very simple way. The U-value has normally been calculated without considering daylight radiation and windows have been regarded as very heat consuming building components.

Attention is now given to the transmission of solar energy into rooms and it is part of the heat supply to heated buildings. In climates with high temperatures solar radiation will increase the cooling load. An accurate simulation of the energy transfer through the glazed part of a window is necessary.

Selective coatings and gases other than air between panes have given better insulation properties to sealed glass units but have also changed the temperatures of the sealed unit. Besides the heat balance, calculation of temperatures in the glass material and in the seal must take the energy absorption and energy transfer into consideration in an accurate way. The JULOTTA program, Källblad 1987 gives an example of such an accurate method. Table 1 sets out heat requirements and maximum room air and glass temperatures of glazed parts with various properties. The glass temperature can rise from 36 to 56 ^{o}C if a specific selective coating and argon are applied to a double glazed sealed unit. The annual heat requirement will for this specific building decrease by 15%.

Glass unit	Selective coating 1) Transm	Absorp	Emiss	Annual heat requirement	Annual max temp Room air surface	Inner glass
				kWh/yr	^{o}C	^{o}C
Double glazed 2)				11629	32.8	35.9
Triple glazed 2)				10533	32.7	40.6
Quadruple glazed 2)				10072	32.7	43.9
Double glazed with selective coating and argon:						
Old type: (gold)	0.58	0.15	0.09	10127	32.7	43.8
Sb-doped SnO_2	0.55	0.30	0.13	9984	34.0	56.2
New type (silver)	0.64	0.15	0.12	10079	33.3	44.0

1) on normal 4 mm glass
2) normal 4 mm glass

Table 1 Annual heat requirement and maximum room air and glass temperatures for an apartment in Swedish climate. South facing glass area =7% and north facing glass area =3% of the floor area.

5 MOISTURE MIGRATION AND MOISTURE STORAGE IN MATERIALS AND STRUCTURES

Moisture conditions in materials and structures are of great concern in new buildings. Rot, mould and smell are problems in all types of climates. In cold climates energy conservation and extreme insulation have been blamed for this, but the reason is of course that architects and engineers have not designed and/or constructed the building in a proper way. Knowledge of migration, condensation and storage of moisture is limited and, moreover, that knowledge is poorly disseminated among architects and engineers.

For very well defined boundary conditions and for materials which are very well known from the point of view of moisture, such as well defined concrete, moisture migration can be calculated also for a transient flow. For most materials such as brick, aerated concrete and wood, future moisture conditions cannot at present be predicted very accurately. The materials are not homogeneous and they vary from one consignment to another. Moreover, the moisture conditions are dependent on the moisture history of the material, and that history can cover many months. Knowledge of the behaviour of components and structures must be based on a qualitative knowledge of migration, condensation (evaporation) and storage of moisture combined with analysis of the behaviour of components and structures in practice. Calculations must take into consideration the variation of boundary conditions and moisture properties.

Research concerning moisture migration etc has to be divided into two groups:

- basic research (increased knowledge) of migration, condensation (evaporation) and storage of moisture

- analysis of components and structures in laboratories and in situ from a heat and moisture point of view

The "Moisture Research Group" (Fuktgruppen) at the Lund Institute of Technology covers both aspects. Among topics the following can be mentioned:

- study of moisture migration in porous materials by the "Moment"-method
- moisture diffusion of concrete, cement plaster and water-cement paste of high moisture content
- moisture migration in wood of high moisture content
- heat and moisture conditions in ventilated crawl spaces
- moisture conditions in walls of aerated concrete

Preliminary results are published in annual reports.

6 FIRE EXPOSED STRUCTURES

In the last few decades, great advances have been made in the mathematical modelling of compartment fire and the design of fire-exposed loadbearing structures. The fully developed or postflashover compartment fire is most widely studied. During the past 20 years a number of simulation models have been developed. Hamerthy and Mehaffey, 1983 have classified 14 mathematical models of compartment fire.

The fundamental characteristics for a full description of the postflashover fire are the variation in time of the following factors (Pettersson, 1983):

1) rate of heat release
2) gas temperature
3) geometrical and thermal data for external flames
4) smoke and its optical properties
5) composition of the combustion products, particularly toxic and corrosive gases

Factors 1-3 concern structural safety.

The models available are applicable to compartments of room size such as dwellings, ordinary offices, schools, hotels etc. For compartments of very large volume such as industrial buildings and sports halls a postflashover model gives an unsatisfactory description of the worst thermal condition of the structure. In such cases a preflashover model of the conditions near the structural members will give a better prediction of the worst case.

For timber structures exposed to fire, an analytical model for calculation of the rate of charring and moisture conditions in uncharred portions is necessary. Fredlund, 1988 has studied this problem. Heat transfer in the material is assumed to take place as conduction and convection when the volatile pyrolyses products and the vapour move in the pore system of the wood. The original moist wood material is assumed to have four phases: active wood which on pyrolyses produces gases, charcoal which oxidises at the material surface, moisture in the liquid phase, and vapour.

The theoretical analysis of the pyrolysis of moist wood involves nonlinear heat and mass balance equations which are solved numerically.

Figure 2 shows an experiment where a piece of pine is exposed to an energy source, Fredlund, 1988.

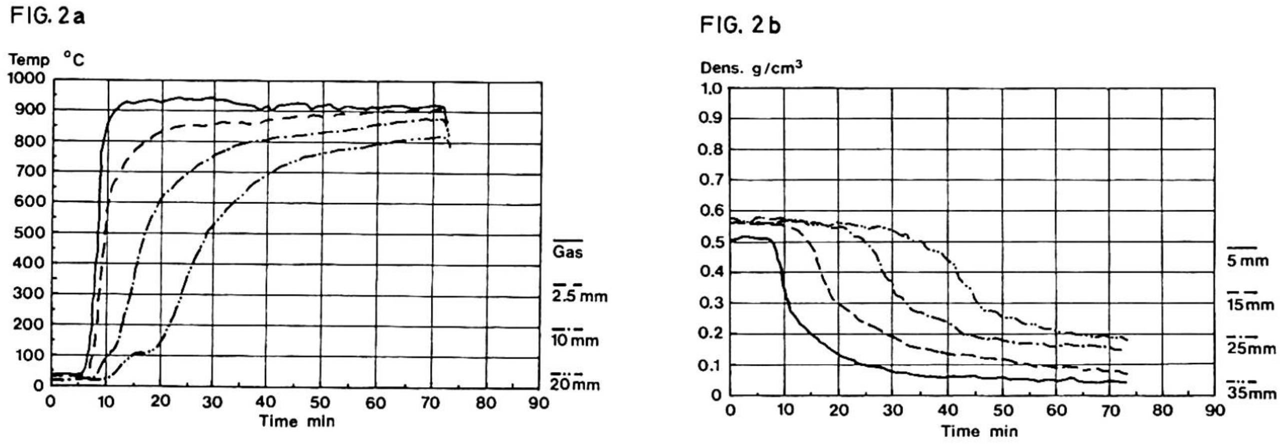

Fig.2 Temperature (a) and density of material (b) in a test where Pine (moisture content =13.5%) is exposed to 95 kW. (Fredlund, 1988).

7 AIR MOVEMENT IN ROOMS AND SPACES

Air movement in rooms and spaces is of interest for many reasons such as

- convective heat transfer in closed spaces
- temperature distribution in large rooms
- smoke distribution from a fire
- ventilation efficiency in ventilated rooms

Air movement in closed spaces has been simulated by a number of mostly two-dimensional mathematical models. Figure 3 shows calculations made by de Vahl Davis and Mallinson, 1975 for a space containing oil and Figure 4 shows the horizontal temperature distribution in the middle of a sealed glass unit with different Rayleigh numbers, Jonsson, 1985. It is shown that the air stands still if Ra=8000.

Air movement in very large rooms is of interest from many points of view. It influences:

- heat consumption
- ventilation efficiency in occupied zones
- temperature distribution in the room
- smoke distribution in the event of fire

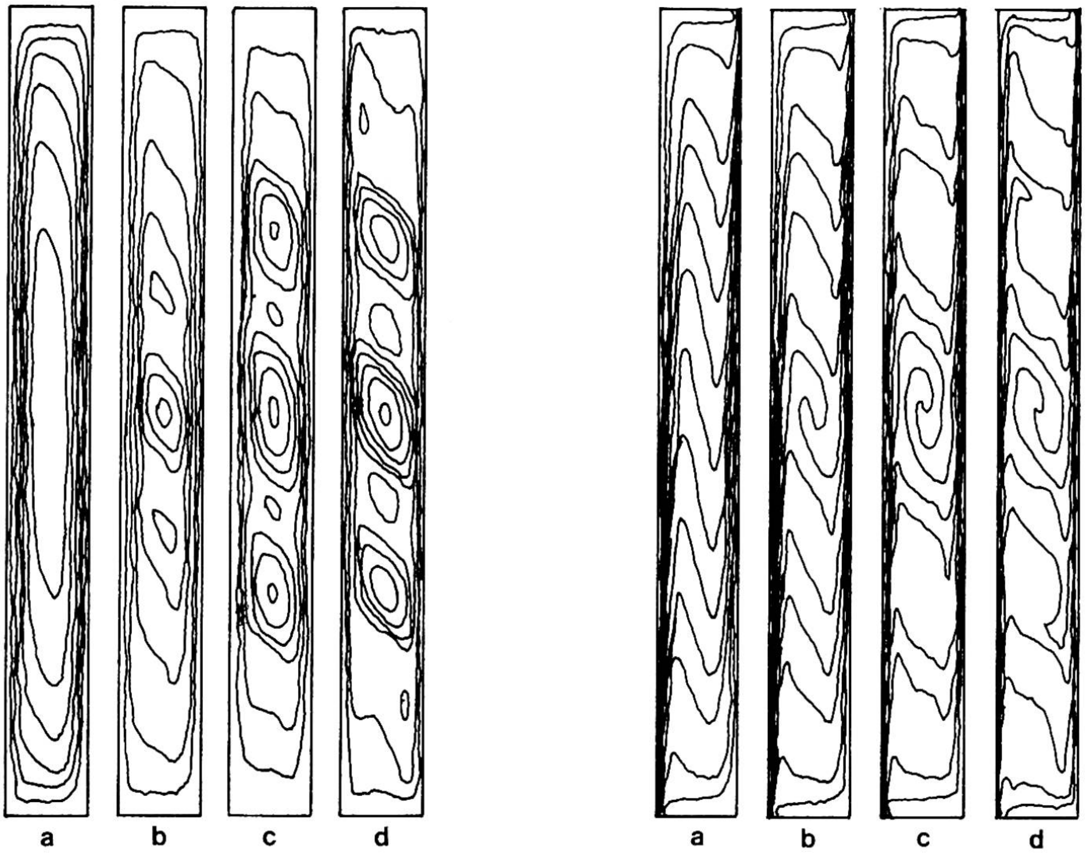

Fig.3 The stream function and isotherms in a space (height L and width d) for L/d = 10 and Pr = 1000 (oil) and Ra = a) $2.4*10^5$, b) $5.0*10^5$, c) $9.4*10^5$ and d) $33*10^5$ (de Vahl Davis and Mallinson, 1975).

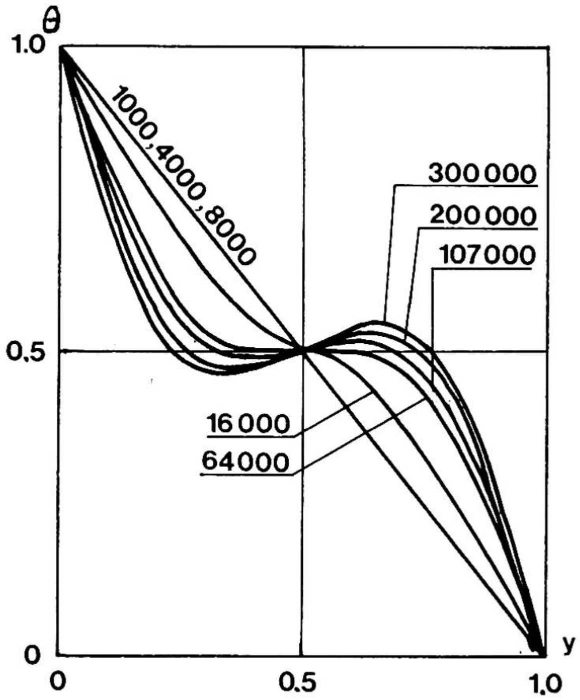

Fig.4 Horizontal temperature distribution in an air-filled vertical space at x/L=0.5 (middle). L/d=20, Ra=1000, 4000, 8000, 64000, 107000, 200000 and 300000. (Jonsson, 1985).

Air movement and the temperature distribution in glazed spaces between buildings or attached to buildings are of interest nowadays. Insolation will increase the temperature of floor and walls, and will give rise to air movements and temperature gradients in the space.

A number of simulation programs have been developed and used for the above mentioned problems, as for example PHOENICS, TEACH and FLUENT, see Whittle, 1986.

Computer programs can of course also be used for prediction of air movements in normal rooms. However full scale tests are normally used. They can also give results for occupied rooms under non-isothermal conditions.

8 HEAT BALANCE IN ROOMS AND BUILDINGS

Design of heating and cooling equipments and determination of energy requirements for heating and cooling necessitate accurate calculation of the heat balances of the spaces in a building. Calculation of indoor conditions in uncooled buildings during the hot season also needs accurate prediction of room temperatures and moisture conditions.

A number of programs have been developed and used for prediction of loads, energy requirements and temperatures. The programs have been compared by calculating the same case. Such comparisons have been carried out within the IEA, 1981 (International Energy Agency), Energy Conservation in Buildings and Community Systems Programme. The result is that large differences occur in the calculation of heating and cooling loads and energy requirements. This is due to:

- differences in interpretation of drawings and specifications
- differences in the algorithms used
- differences in the numerical treatment

Comparisons between simulations and measurements are rare, especially for occupied buildings.

A conclusion is that simulations have difficulty in accurately predicting the thermal performance of a building. If the weakness of a program is known the programs can be successfully used for parametric studies, i.e. study of the influence of the variation of a parameter (insulation of walls, number of panes in windows, orientation of windows etc) on temperatures, loads and energies. Such parametric studies have been carried out in our department. They concern different types of climate, Swedish, European, Chinese and Algerian climates. Table 2 shows how improved insulation and reduced ventilation can give an unheated multistorey residential building in the Beijing climate reasonable indoor temperatures also during the coldest days of the year when the outdoor temperature is below -10 ^{o}C.

Case					Room air temperature (^oC)					
Apart-ment	Ven-tila-tion ach/h	Panes	U (W/m^2K) Outer walls	Roof	Middle storey T_{min}	T_{-100}	T_{-500}	Upper storey T_{min}	T_{-100}	T_{-500}
Middle	1.1	2	0.87	0.31	3.0	5.3	7.4	2.5	4.7	7.0
"	0.5	"	"	"	6.5	9.5	11.8	5.9	9.0	11.2
"	"	3	"	"	7.8	10.6	12.9	7.1	10.0	12.4
"	"	"	0.56	"	8.7	11.5	14.0	8.0	11.0	13.5
"	"	"	0.33	0.17	10.0	13.0	15.7	9.6	12.9	15.4
"	"	4	"	"	10.9	13.7	16.2	10.5	13.5	16.1
End	"	"	"	"	9.3	12.1	14.4	8.9	11.9	14.2

Table 2 Annual minimum temperature T_{min} and temperatures T_{-100} and T_{-500} fallen short of 100 and 500 hours per annum, respectively in an unheated building with different ventilation rates, numbers of panes and U-values of outer walls and roof. Glass area facing south is 14% of the apartment floor area. BEIJING 1984. (Adamson, unpublished).

9 COMBINED HEAT AND MOISTURE BALANCE

In a Nordic climate, indoor moisture conditions are not critical for human comfort. In a warm and humid climate, however, indoor moisture conditions play an important part for comfort. During certain periods of the year, moisture conditions may become very uncomfortable if the apartments are not suitably planned and designed.

Calculation of the combined heat and moisture balance is of great interest, and studies have started. This is a field of future research.

10 THEORY AND PRACTICE

This paper has dealt with simulation of physical processes in buildings. It is often stated that the agreement between simulations and measurements could be better, especially for occupied buildings. This has caused more "practical" people to ask: Why use an immense program to calculate the heat requirement of an occupied building when the difference between calculated and measured consumption can be more than 10%? This is a very good question and shows the weakness of simulations. The result of a simulation can only be as good as the input data. If the input data are not accurately known, which they normally aren't in occupied or used buildings, the agreement between theory and practice will not be very convincing.

However, simulations and parametric studies are very good tools to increase our knowledge. Simulation of frost penetration near a foundation was used to shape building regulations. It was possible to study how variations in soil properties, snow cover, insulation etc influenced frost penetration. Simulations of the development of fire and smoke movement also result in regulations and recommendations. Parametric studies concerning indoor temperatures, heating and cooling requirements have resulted in regulations and handbooks.

Conditions in occupied buildings for which a number of parameters are unknown or badly known can probably be predicted by using simplified models in which uncertain parameters are given as an average value and a variation.

Theory is a simulation of nature. At best it simulates nature in a very good way. Practice is nature and therefore a very good theory and practice will coincide. A "practical man" has, at best, extensive experience of real cases (design, construction or operation of a building). But he cannot use his knowledge unless the situation from which the experience originates is repeated. Some circumstances are probably changed, but for the "practical man" it seems that the case is the same. Without a theoretical background it is hardly possible to use experience. A theory gives an understanding of the physical behaviour and gives a framework which helps to organize various experiences.

Building codes and recommendations are simplified. It is most essential that they are physically correct. They should also reflect the theoretical background and serve as an educational instrument. They do not always fulfil that requirement.

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